FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

OAK STREET BASIN AND STORM DRAIN DESIGN

FINAL DESIGN DATA REPORT VOLUME II

CONTRACT FCD – 2009C021

October, 2012

Prepared For:

Flood Control District of Maricopa County
2801 West Durango Street
Phoenix AZ, 85009

In Cooperation With:

City of Mesa
20 East Main Street, 5th Floor
Mesa, Arizona 85211

Prepared By:

Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
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Job No. 0912

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MEETING MINUTES

Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021
Date: 1/19/2010
Location: FCD - Operations Building
Purpose: Kick-Off Meeting - Mtg-01

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:	Action:
	Introductions - see list below for attendees	
	Old Business:	
	New Business:	
1	**Project Overview -** Afshin gave an overview of the project, explained that it represents the final, upstream element of the drainage system which includes the McDowell Road storm drain and the Hermosa Vista storm drain. Scott added that the project will provide flood protection to the Thunder Mountain subdivision as well as a number of other properties that lie downstream of Hawes Road and downstream of McDowell Road. He also explained that the project should limit flows and sediment discharges into the Thunder Mountain subdivision.	
2	**MCDOT Involvement -** Afshin said that we need to involve MCDOT because much of the work will be done in County R/W.	***Afshin to invite MCDOT to our monthly meetings***
3	**87th Street Storm Drain Lateral -** Afshin brought up the option of running a lateral up 87th Street. The group agreed that it would be best to discuss this option, as well as construction access issues, during the value engineering effort after the 30% submittal.	
4	**GIS Data to be Supplied by District -** Topographic Mapping, parcel maps, aerial photos, etc	***G&B to request data from Eric Feldman and copy Afshin with the e-mail request (completed 1/20/10)***
5	**Hydrology Data to be Supplied by District -** HEC-1 models and version of DDMS that was used by Kimley Horn	***Steven to e-mail the Kimley Horn HEC-1 model and the appropriate version of DDMS to Mark***
6	**District Landscape references to be supplied by District**	***Jon to provide CD (Completed 01/22/2010)***
7	**City Utility Data and As-builts**	***Shahir will upload the relavent City records to the District's FTP site and give e-mail notification to Mark and Afshin***
8	**Title Report and Legal Description for Basin Property**	***Mark to send an e-mail request to Afshin for the title report and ALTA property survey, if one exists.***

9	**Right of Entry -** The group agreed that no right of entry is necessary for the field survey effort or the plant inventory work, but for the geotechnical work, it was agreed that rights of entry were necessary, if on private property.	***Mark to e-mail a list of properties (by tax parcel number) that will be impacted by the project so that Afshin can order title reports.***
10	**Geotechnical Investigation -** Group discussed location and timing for borings, boring operations need to be sensitive to the desert.	***G&B to provide layout of boring plan for District review and to get the process started***
11	**Hydrologic Analysis -** Verification of Watershed Boundary and Flow Splits	***Steven to conduct a field visit with G&B on Monday (1/25/10) in order to verify the watershed boundary (completed 1/25/10)***
12	**Public Meetings and PAAC Meetings -** The group had some discussion on these meetings and agreed that the first would be held around the 30% plan stage.	
13	**Utility Coordination -** Gary M. told us to make sure that we order fiber optic maps from COX and QWEST since they come from different departments. He also said that he would help us with utility contacts for the area for the conflict review notices.	
14	**Monthly Meeting Schedule -** The group agreed that the third Tuesday of each month at 1:00 is a good time for the monthly meeting.	***Afshin to send out invitation for the monthly meeting (completed 1/22/10)***
15	**Project Schedule -** Jon Loxley said that it would be helpful to provide a flowchart along with the project schedule to be able to see the required sequence of tasks.	***Mark to get the project schedule to Afshin and look into preparing a flowchart***
16	**Projected Billings**	***Mark to get projected billings to Afshin***
	Schedule:	
	Key Dates: 30% Submittal: April 15, 2010 Public Meeting No. 1: May 2010 60% Submittal: July 1, 2010 Public Meeting No. 2: August 2010 90% Submittal: Sep. 15, 2010 100% Submittal: Dec.1, 2010	
	NEXT MEETING: Feb 16, 2010 - 1:00 p.m. at the District	

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
x	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Jon Loxley, FCD	jonloxley@mail.maricopa.gov	602-506-2956
x	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
x	Nicole Scheider, FCD	nicolescheider@mail.maricopa.gov	602-506-6762
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771

	Report of Meeting	Gavan & Barker Inc.
	Oak Street Basin and Storm Drain Design	3030 North Central Avenue, Suite 1530 Phoenix, AZ 85012 Phone 602-200-0031
	Contract FCD 2009C021	
	Date: 2/16/2010	
	Location: FCD - New River Conf. Room	
	Purpose: Monthly Meeting - Mtg02	

Item No.	Item/Notes:	Action:
1	**Introductions -** see list below for attendees	***Afshin to invite MCDOT to our next monthly meeting***
	Old Business:	
2	**Data Collection -** Data collection is nearly complete. Shahir has requested an additional drainage report from the engineer (Granite mountain Drainage report). Steven Tucker to get appropriate version of DDMS to Gavan & Barker.	***Mark to talk to Steven about obtaining the proper version of DDMS.***
3	**Geotechnical Investigation -** The geotechnical investigation will be done in two phases. Phase one will be the 4 soil borings plus any necessary rock coring for the basin property and Phase two will be the soil borings for the storm drain alignment. Phase one will be done now and phase two will be done after completion of 30% plans. Afshin authorized the use of the allowance for rock coring, as necessary within the basin.	***Gavan & Barker to flag the access route for the drill rig equipment. Gavan & Barker shall also flag the cactus that need to be protected. Kip to provide schedule to Gavan & Barker so that Afshin can notify the neighboring property owners.***
4	**Utility Designating -** Utility designating shall be done for the dry utilities located in Oak Street from Hawes Road to 87th Street and in Hawes Road from Oak Street to the south about 200 feet. Afshin authorized the use of the allowance for the utility designating but ask that we send him an estimate of the dollar amount to be used.	***Mark to send an e-mail to TBE (with a copy to both Afshin and Shahir) authorizing TBE to proceed. The e-mail shall indicate the utilities to designate and the approximate length of each. Any additional designating shall be approved in advance before TBE proceeds.***
5	**Right of Entry -** Afshin has started the process of obtaining the right of entry for the storm drain soil borings.	
6	**Watershed Boundary -** The upper watershed area will be included in the hydrologic model, but only a portion of the flow from the apex of the alluvial fan will be assumed to flow to Oak Street. Preliminary estimates made by Steven Tucker and Gavan & Barker was that the 10% to 20% of the upper watershed contributes to the Oak Street flood flows. This split flow will be further assessed as the hydrology is developed.	
7	**Project Schedule -** The project schedule was distributed to the team members.	

	New Business:	
8	**Thunder Mountain Access - The d**esign team needs access to Thunder Mountain (through automated access gates) in order to measure existing washes and culverts.	***Nicole to talk to Thunder Mountain representative about obtaining a key code to enter the subdivision (completed 2/16)***
9	**PAAC Meetings and Public Meetings -** Public meeting number 1 is scheduled for May 11. PAAC meeting should be about 2 weeks before public meeting.	***Nicole and Jon L. to put together invitation list for PACC meeting. Public meeting will be discussed at next monthly meeting.***
10	**Plant Inventory -** The plant inventory will cover the entire basin property. It will include trees of 4 inch in caliper and larger plus saguaros (total height including arms), ocotillos, barrels, and an approximate number of the smaller, miscellaneous cactus.	
	Schedule:	
	Key Dates: 30% Submittal: April 16, 2010 Public Meeting No. 1: May 2010 60% Submittal: July 12, 2010 Public Meeting No. 2: August 2010 90% Submittal: Sep. 21, 2010 100% Submittal: Dec.8, 2010	
	NEXT MEETING: Mar 23, 2010 - 1:00 p.m. at the District	

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
x	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
x	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
x	Jon Loxley, FCD	jonloxley@mail.maricopa.gov	602-506-2956
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
x	Nicole Scheider, FCD	nicolescheider@mail.maricopa.gov	602-506-6762
x	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872

Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021

Date: 3/25/2010

Location: FCD - New River Conf. Room

Purpose: Monthly Meeting - Mtg03

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:	Action:
1	**Introductions -** see list below for attendees	
	Old Business:	
2	**PAAC Meetings -** A PAAC meeting will be held in early May.	***Nicole, Jon and Afshin will get together to establish a list of invitees for the PAAC meeting.***
3	**Public Meeting -** A public meeting will be held in late May.	***Nicole to determine availability of Mesa's fire house. Gavan and Barker will prepare graphic of project limits for post card advertisement.***
4	**Geotechnical Investigation -** RAMM drilled 4 soil borings within the basin and discovered weathered granite in all 4 borings. RAMM believes that most of the rock, even at deeper depths, can be excavated with conventional excavation equipment, but that some of the area, particularly the deeper cuts at the northeast corner may require rock excavation techniques. Mark suggested having RAMM conduct the seismic surveys and possibly some test pits to supplement the soil boring data.	***Mark to send a copy of RAMM's preliminary geotech report to Afshin and Shahir (completed 4-7-10)***
5	**Right of Entry -** Afshin is coordinating with the property owner for right of entry to drill the storm drain soil borings.	
	New Business:	
6	**Hydrology/Granite Mountain Diversion -** The Granite Mountain subdivision was designed with a diversion/distribution channel along its southern edge. Mark explained that it appears that the diversion channel will divert the majority of the flow to Hawes Road, but it was difficult to know for sure without the hydrologic model for Granite Mountain.	***Shahir will contact Wood-Patel to get their HEC-1 model (completed 3-29-10)***
7	**Storm Drain Design -** Mark presented a schematic alignment for the storm drain which included a storm drain in Hawes to capture the diverted flow described in item 6 and suggested that some improvements may have to be done to the Granite Mountain channel to make sure that the 100-year flow is all captured. Gary S. expressed concern with changing the channel in Granite Mountain. In addition Jon L. expressed a concern that we are diverting flow in the Oak Street storm drain that would otherwise flow through Thunder Mountain. Afshin suggested that an onsite meeting should be held to review the storm drain alignment in the field.	***Afshin will set up a field review meeting.***

8	**Plant Inventory/Basin Design -** John summarized the plant inventory and pointed out some proposed design changes to avoid removal of existing trees and cactus. He said there are about 25 trees impacted, but only about 7 of these are reasonably salvageable. We will need to make a determination if any trees will be salvaged. The fact that the basin concept includes a dike/dam at the southwest corner was discussed. It was suggested that the District's Dam Safety group be consulted to make sure the concept is acceptable.	***Afshin to ask the District's Dam Safety group to take a look at the concept and advise.***
9	**Schedule -** Mark asked to move the 30% submittal back to April 30th due to the complexities with the hydrology.	**Mark to send a revised schedule to Afshin.**
	Schedule:	
	Key Dates: 30% Submittal: April 30, 2010 Public Meeting No. 1: May 2010 60% Submittal: July 12, 2010 Public Meeting No. 2: August 2010 90% Submittal: Sep. 21, 2010 100% Submittal: Dec.8, 2010	
	NEXT MEETING: Apr 20, 2010 - 1:00 p.m. at the District	

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
x	Jon Loxley, FCD	jonloxley@mail.maricopa.gov	602-506-2956
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
x	Nicole Scheider, FCD	nicolescheider@mail.maricopa.gov	602-506-6762
	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872

	Report of Meeting	Gavan & Barker Inc.
	Oak Street Basin and Storm Drain Design	3030 North Central Avenue, Suite 1530 Phoenix, AZ 85012 Phone 602-200-0031
	Contract FCD 2009C021	
	Date: 4/13/2010	
	Location: Thunder Mountain Ramada	
	Purpose: Monthly Meeting / Field Review - Mtg04	

Item No.	Item/Notes:	Action:
1	**Introductions -** see list below for attendees	
	New Business:	
2	**Oak Street Inlet, East of 87th Street (390cfs) -** The team conducted a field review of the wash that crosses Oak Street just east of 87th Street. This wash represents the largest concentration of flow that will be intercepted by the storm drain. Mark explained that the plan is to construct a special drop inlet structure in the front yard of the homeowner's property which lies on the north side of Oak Street. Scott asked if the inlet could be built on the south side of Oak Street so that disruption of the homeowner's front yard could be avoided. He also suggested another option wherein the inlet in the front yard would be reduced in size to pick up a portion of flow with the rest being captured on the south side of Oak Street. Mark said that either of those options could work, but they would not provide flood protection for Oak Street. John suggested that the frontage road could be eliminated which would move the inlet closer to Oak Street and out of the Homeowner's front yard.	***Gavan & Barker to show the proposed special drop inlet structure on the 30% submittal with an opening in the flood containment wall that will allow about 25cfs to pass downstream into the Thunder Mountain subdivision.***
3	**Oak Street Inlet, 86th Street extended (230cfs) -** The team also looked at this wash which represents the second largest concentration of flow that will be intercepted by the storm drain. Mark explained that the plan is to construct a special drop inlet structure in the front yard of the homeowner's property similar to the one east of 87th Street. Scott's comments at that location also apply to this location. However, at that location there is not an opening in the Thunder Mountain Wall so no by-pass will be	***Gavan & Barker to show the proposed special drop inlet structure on the 30% submittal.***
4	**Sediment -** Steven said that the inlets which are to allow low flows to bypass should be designed in a manner which encourages the sediment load to be captured in the new storm drain. That way the bypass flows should only be carrying smaller sediment (suspended load) which should minimize downstream maintenance issues.	
5	**Basin Design -** The team discussed the basin design and height of the berm. Scott said that we need to check to make sure that the berm will not fall within ADWR's definition of a jurisdictional dam. He also said that we should endeavor to limit the height of the berm, regardless whether it is jurisdictional.	***Mark checked ADWR's definition of jurisdictional dam and found that the berm is not jurisdictional. It is about 6 feet high relative to the spillway elevation. According to District criteria, the 6 foot height requires a spillway designed for the one-half PMF compared to a berm that is less than 6 feet which only requires a spillway designed for the 100-year flood.***

6	**Hawes Road Storm Drain -** Mark presented a schematic alignment for the storm drain which included a 48" storm drain in Hawes to capture the diverted flow from the Granite Mountain channel. The inlet location was visited in the field. There was some concern with building the inlet on private property, but it was agreed that we would include it on the 30% submittal.	
7	**Title Reports -** Scott said that title reports should be obtained for those properties where we know we're going to need R/W.	***Afshin to have the real estate division order the title reports.***
8	**Flows through Thunder Mtn. -** Bob Nowak and Vince Kelly asked that we try to minimize the flow of water and sediment that enters their subdivision. They said that removing sediment is a constant maintenance problem.	
9	**Thunder Mtn. Channel Maintenance -** Bob Nowak and Vince Kelly asked if the flood control district could provide them with a maintenance plan for their floodwater channels. Scott said that he would look into it for them.	**Scott to get back to the HOA members on District procedures for channel maintenance.**
	Schedule:	
	Key Dates: 30% Submittal: April 30, 2010 Public Meeting No. 1: May 2010 60% Submittal: July 12, 2010 Public Meeting No. 2: August 2010 90% Submittal: Sep. 21, 2010 100% Submittal: Dec.8, 2010	
	NEXT MEETING: May 18, 2010 - 1:00 p.m. at the District	

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
	Jon Loxley, FCD	jonloxley@mail.maricopa.gov	602-506-2956
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
	Nicole Scheider, FCD	nicolescheider@mail.maricopa.gov	602-506-6762
x	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
x	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872
x	Vince Kelly (Thunder Mtn. HOA)		480-201-8896
x	Bob Nowak (Thunder Mtn. HOA)		480-354-1697

Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021

Date: 5/18/2010

Location: FCD - New River Conf. Room

Purpose: Monthly Meeting - Mtg05

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:	Action:
1	**Introductions -** See list of Attendees below	
	Old Business:	
2	**Geotechnical Investigation -** The group agreed to exercise the allowances for seismic refraction surveys and test pit excavations to better define the subsurface conditions at the basin. For the storm drain, it was decided that soil borings alone will be adequate for now.	*G&B to send a cost estimate for the allowance items to Afshin and Shahir along with a plan for the test pits and SD soil boring locations.*
3	**PAAC Meeting/Public Meeting -** PAAC meeting to be held 6:30 on 5/25 and public meeting to be held at 6:00 on 5/27.	*Preparations to be discussed in separate meeting on 5/19/2010.*
	New Business:	
4	**Utility Coordination -** 30% plans to be sent to the utility companies. There is a potential conflict with Mesa's gas line and water line along Hawes Road where the storm drain is aligned in between the city utilities (gas and water) and the SRP power poles. It was agreed that the sewer line that runs diagonlly under the Oak/Hawes intersection will be potholed since there is little clearance to the proposed SD. Wait for this pothole until alignment is set, have it done for 60%.	*G&B to send 30% plans to SRP, COX and QWEST and copy Gary Maiers with the letters. Shahir to discuss Hawes Road utility issues with City staff.*
5	**404 Permitting -** The Granite Mountain subdivision will cut off one of the washes that was identified as jurisdictional in the ADMP.	*Bob to research the JD that was done for the Granite Mountain subdivision and advise the team on how to treat it for this project.*
6	**Right-of-Way -** The title reports that were acquired for three separate properties on the east end of Oak Street indicate that there is not R/W for Oak Street. Instead, there is an ingress/egress easement which may not provide the necessary rights to construct a storm drain. Consequently, it appears that R/W and/or drainage easements will be required to construct the new SD.	
7	**Granite Mountain Channel Inlet -** The group agreed on the concept of constructing a new inlet at the west of the Granite Mountain channel to be sized to accept the entire 100-year design flow. Hydraulic capacity calculations will be conducted for the upstream channel to estimate the amount and location of flows that can be expected to spill out of the channel. Inlets will be added along Hawes Rd to collect any flows that are determined to spill out of the channel.	*G&B to determine the capacity of the Granite Mountain Channel*

8	**Hawes Road Channel Freeboard -** The current design does not provide the required freeboard (according to the District's Hydraulics Manual) in the Hawes Road channel	*G&B to demonstrate that the project will not increase the flood potential along the Hawes Road channel. Look at Thunder Mountain original report to see what flows were to get to the channel under original concept and compare to design flows. Include this analysis in the report as justification for not using the design freeboard criteria.*
9	**Storm Drain Design/Drop Structures -** The 30% design was based on a maximum pipe velocity of 20fps in accordance with the scope of work. The design incorporates a number of 4' drop structures to reduce the slope enough to achieve a maximum of 20fps.	
10	**Sediment Transport -** Preliminary calculations indicate that the bypass pipes can not convey the expected sediment load from the 100-year flood.	
11	**Basin Spillway/Freeboard -** the 30% basin plan incorporates a spillway designed to convey the 100-year flood.	*Steven to review the design of the Culver-Hawes basin to determine why a spillway was not installed.*
12	**Food for Thought -** The group discussed the idea of changing the basin design from an offline basin to an inline basin which will preclude the problem of inadequate sediment conveyance capacity in the bypass pipes.	*The maintenance of the basin would be an issue if the basin is inline. Some benefits are reduced cost, pipe system will not be plugged with sediment.*
14	**Field Survey -** The group agreed that the Granite Mountain channel should be surveyed in order to determine its conveyance capacity. This effort is outside the scope of services so the survey allowance will be used, if necessary.	*G&B to submit an allowance request to Afshin and Shahir before beginning survey work on the Granite Mountain channel.*

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
x	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
x	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
	Dennis Holcomb, FCD	dbh@mail.maricopa.gov	602-506-4074
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
	Nicole Scheider, FCD	nicolescheider@mail.maricop	602-506-6762
x	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
x	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872
	Vince Kelly (Thunder Mtn. HOA)		480-201-8896
	Bob Nowak (Thunder Mtn. HOA)		480-354-1697

Report of Meeting

Oak Street Basin and Storm Drain Design

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Contract FCD 2009C021

Date: 6/15/2010 & 6/28/2010

Location: FCD - New River Conf. Room

Purpose: Monthly Meeting (Mtg06) & 30% Comment Resolution Meeting

Item No.	Item/Notes:	Action:
1	**Introductions -** See list of Attendees below	
	Old Business:	
2	**404 Permitting -** Bob has requested the Granite Mountain 404 permit from Wood-Patel to help determine the status of the wash that crosses Hawes Road at the NE corner of the basin site. This wash was not identified as being jurisdictional in the Wood Patel grading plan for the subdivision, but it was determined as jurisdictional with the Spook Hill ADMP. Bob also stated that he did not see any obstacles to obtain a 404 permit for the plan, but that mitigation will probably be required which can either be done onsite in a dedicated mitigation area or to pay mitigation fees to have it done offsite.	*Bob is going to follow up with Ash Patel to obtain their JD for the washes in the Granite Mountain subdivision.*
	New Business:	
3	**30% comment (Oversize Inlets to account for uncertainty in upstream diversions - use curb opening inlets on Oak Street) -** Scott suggested that we incorporate extra inlet capacity on the downstream side of Oak Street to capture any flows that might side step the inlets and/or flows that might spill out of the upstream inlets if they become clogged. At the Thunder Mountain wall, he suggested limiting the amount of flow that can get through the wall openings by reducing the size of the openings and installing inlets to capture larger flows that exceed the capacity of the reduced wall openings.	
4	**30% Comments - Basin Design (Freeboard, Perimeter Maint. Road, move basin north and east to preserve natural edge) -** The group agreed that the basin plan should be refined to include a maintenance road that will run around the entire perimeter like at the Culver - Hawes Basin. Dan said that the road needs to be at least 14 feet wide and should be covered with at least 3 inches of ABC. The group also agreed that the basin should have 1 foot of freeboard below the spillway elevation and that Gavan & Barker perform sediment transport calculations to verify that the expected volume of sediment inflow from the 100-year event will not exceed the volume associated with the 1 foot of freeboard. Scott asked that we endeavor to preserve as much of the perimeter vegetation as possible.	

5	**30% Comments - Hawes Road Channel (Design Capacity, Freeboard, High Velocities, Culvert under Street Entrance) -** Mark explained that the drainage report for Thunder Mountain does not include design flows for the Hawes Road Channel and that the report indicated that the flows were to be culverted under Hawes Road instead of being collected in a channel. Mark also explained that the McDowell Road storm drain was designed based on the assumption that 200cfs will discharge to the Hawes Road Channel from the Oak Street SD system. The 30% design plans are based on only 180 cfs discharging to the channel, but even though the 180 cfs is contained in the channel, there is inadequate freeboard to be in conformance with District freeboard standards. The group agreed that a low wall could be used to provide the freeboard in locations where adequate freeboard does not exists.	
6	**30% Comment - Sediment Transport (Traps at Inlets vs. Design to Pass Sediment, Storm Drain Sediment Capacity, Eliminate Basin Bypass Pipes) -** Mark explained that G&B preliminary sediment calculations would suggest that the basin bypass pipes will not be capable of conveying the incoming sediment from the larger upstream pipes. The group agreed that the best solution is to construct the basin as an online basin, thereby eliminating the basin bypass pipes and allowing the sediment to drop out in the basin.	
7	**30% Comments - Drop Structures (4' Drop Inhibit O&M, Length of Drop, Velocity Exceeds 15fps, Velocity Rings in lieu of Drops) -** Dan said that the 4 foot drop is acceptable if the maintenance steps are positioned in a manner that will allow access to both the higher and the lower pipes. The group agreed that the storm drain can be designed for 20 fps just so the EGL is kept below the ground surface. Scott also suggested that we determine how many drop structures would be required to limit the velocity to 15 fps to help justify the higher allowable pipe velocity.	
8	**30% Comments - Inlet Bypass Flows (Bypass at all Thunder Mountain Wall Openings, Bypass Flows along Hawes Road, 404 Washes) -** Scott suggested that we eliminate the inlet bypasses and instead consider local inflow downstream of the inlets as the low flow bypass. The group agreed that the large inlet at the upstream end of the Oak Street SD should be moved southerly, into the frontage road in conformance with the homeowners' suggestion that we received at the public meeting. This will require some reconstruction of the driveway access from Oak Street. Afshin said that he will have the District's real estate division review the title reports to make sure that this option can be done.	*Gavan & Barker to calculate the local inflow that is not captured by the inlets and coordinate with Bob to determine if that flow will be adequate to preserve the downstream 404 jurisdictional washes. Gavan & Barker shall also return the title reports to Afshin.*

9	**30% Comment (Curved vs. Straight SD Alignment) -** The group agreed that curved storm drain alignments would be best to help prevent sediment deposition in the pipes. Gavan & Barker will provide more information on the 60% plans regarding the curves and whether they can be done with beveled pipes or if pre-manufactured bends will be required. It was also discussed, after Dan left the meeting, that District maintenance staff should be consulted to make sure that they don't have any maintenance concerns with the curved sections.	*Group to discuss the curved pipe sections with Dan at the next monthly meeting*
10	**30% Comment - Hawes Rd SD Alignment (Separation from 20" Waterline, Separation from 2" gas line, Align under Basin Berm, Place Upstream Inlet in Granite Mountain Basin) -** Shahir said that the City would like to get as much separation as possible from their 20" W and 2" Gas. Mark will send a revised alignment to Shahir for the City's approval. The group agreed that the Granite Mountain inlet should be left in its current location which is inline with the upstream channel that runs along the Subdivision's south line. In that way the initial flows will enter the storm drain before they spill into the Subdivision's basin which will better meet the design intent of the subdivision's drainage infrastructure.	*Gavan & Barker to send revised Hawes Road storm alignment to the City for their review.*
11	**30% Comment - Oak Street SD Alignment (Align north of EP, East of 23+00 Align north or South of EP) -** The group agreed that the storm drain should stay in its current alignment which is in the right-of-way and which is not in conflict with existing utilities. Sahir asked that G&B respond to the City's comments regarding the alignment. The lack of vehicular access during construction was discussed and the notion of opening Quill Street to the east, temporarily during construction, was brought up.	*Afshin to talk to the District's Real Estate division about gaining temporary neighborhood access along Quill Street to alleviate vehicle traffic during construction on Oak Street.*
12	**30% Comment - Alignment of Basin Inlet/Outlet Pipes (Connect Basin Outlet to Hawes Rd. SD, Connect East Inlet to Oak Street SD) -** The group agreed that we will connect the east inlet into the Oak Street SD. The comment regarding the connection of the basin outlet pipe to the Hawes Road SD becomes a mute point with the inline basin concept.	

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6/15	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522	
6/15 & 6/28	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076	
	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031	
	Dennis Holcomb, FCD	dbh@mail.maricopa.gov	602-506-4074	
6/15 & 6/28	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.cor	602-200-0031	
	Nicole Scheider, FCD	nicolescheider@mail.mari	602-506-6762	
6/15 & 6/28	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.co	602-200-0031	
6/28	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771	
6/28	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292	
6/15 & 6/28	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872	
6/15	Jeff Riddle, FCD	jrr@mail.maricopa.gov	602-506-4602	
6/28	Daniel Michael, FCD	dam@mail.maricopa.gov	602-506-4873	
	Vince Kelly (Thunder Mtn. HOA)		480-201-8896	
	Bob Nowak (Thunder Mtn. HOA)		480-354-1697	

Report of Meeting

Oak Street Basin and Storm Drain Design

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Contract FCD 2009C021
Date: 7/20/2010
Location: FCD - New River Conf. Room
Purpose: Monthly Meeting (Mtg07)

Item No.	Item/Notes:	
1	**Introductions -** See list of Attendees below	
	Old Business:	
2	**404 Permitting -** Bob Stevens said that the wash that runs through the Granite Mountain subdivision that was considered jurisdictional at the time of the ADMP was not identified as a 404 wash with the development of Granite Mountain and, therefore we should not have to consider it jurisdictional. Scott's suggestion that we eliminate the inlet bypasses and instead consider local inflow downstream of the inlets as the low flow bypass at 404 washes was discussed. Bob thought that the inflow from the local inflow might be too small relative to the normal flow in the wash, but he said that he would present it that way to the Corps and get their opinion.	*Gavan & Barker to provide the local inflow calculations to Bob Stevens.*
3	**Hawes Road Storm Drain Alignment-** The alignment of the Hawes Road storm drain, relative to the city's utilities and the SRP power poles was discussed and it was decided to keep the 33 foot offset that is presented in the 30% plans. That offset provides a separation of about 2.5 feet between the new trench wall and the theorectical, old trench wall for the 20" waterline. It also provides a separation of about 2 feet from the new trench wall to the SRP power poles.	
	New Business:	
3	**Utility Potholing -** Mark presented the proposed location for 17 potholes for the project.	*Gavan & Barker to prepare a pothole list and exhibit and get it to Afshin and Shahir for approval. Gavan & Barker will also request a proposal from TBE for the pothole work.*

4	**Basin Re-Design -** John Barker presented a very rough outline of the redesigned basin. It had to get significantly larger to account for the requirement of a perimeter maintenance road and to provide approximately 11% more volume due to changing the design to an online basin as opposed to the offline concept presented in the 30% plans. Dennis provided design parameter priorities if we have to modify the design to preserve a reasonable buffer width on the north and east sides of the basin. He said the first, most important priority is to keep the shape (that is preserve the ridges that run down into the basin), second is to keep the undulating slopes (he would be willing to forgo the variable slopes in favor of preserving the shape), and third is to keep the island in the bottom of the basin (Dennis thinks the island is the least important feature). Dennis also said that we could consider vertical (or near vertical) walls with railings if that will help reduce the footprint of the basin.	
5	**Oak Street Storm Drain Alignment -** Gordon expressed a concern with local traffic access during construction, particularly in the narrow street section that lies just west of 87th Street. He asked that the design team consider moving the pipe alignment to the south to provide as much space as possible between the new trench wall and the north edge of pavement.	
6	**Schedule -** A revised schedule was presented that proposed a tentative 60% submittal date of August 23rd and a 100% submittal date of January 12th, although it was discussed that this schedule may be too aggressive considering the coordination that still has to take place on the basin re-design and the property owner coordination.	

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	Bob Nowak (Thunder Mtn. HOA)		480-354-1697

Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021

Date: 8/17/2010

Location: FCD - New River Conf. Room

Purpose: Monthly Meeting (Mtg08)

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:	
1	**Introductions -** See list of Attendees below	
	Old Business:	
2	**404 Permitting -** Mark described the local inflow maps that Gavan & Barker previously delivered to Afshin and Bob.	*Bob is going to meet with the Corps to determine what the 404 permitting requirements will be for the project.*
	New Business:	
3	**Geotechnical Report -** Mark briefly described the results of the geotechnical investigation and explained that based on RAMM's findings, Gavan & Barker has introduced steep slopes within the basin where the geotechnical report indicates that the rock will be stable with vertical or near vertical cut slopes.	*Gavan & Barker submitted a copy of the geotechnical report to Afshin.*
4	**Utility Potholing -** Utility potholes being conducted by TBE should be completed by 8/20/10.	
5	**Right-of-Way -** Right-of-way requirements for the Oak Street storm drain were discussed and Scott suggested that the required right-of-way should be identified as drainage easement.	*Gavan & Barker to send a strip map to Afshin showing the new Drainage Easements that will be required along Oak Street*
6	**Basin Re-Design -** John Barker presented the re-designed basin and explained that the undisturbed buffer area along the north side of the basin property has largely been preserved. He also said that the elimination of the bypass pipes has allowed preservation of more of the perimeter vegetation.	
7	**Schedule -** Mark said that the 60% plans will be submitted on August 31, 2010.	*Gavan & Barker to send Afshin a revised schedule along with an explanation for the extra time needed to complete the project.*

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X	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076	
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Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021

Date: 9/21/2010

Location: FCD - New River Conf. Room

Purpose: Monthly Meeting - Mtg09

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:	Action:
1	**Introductions -** See attached list of attendees.	
	Old Business:	
2	**404 Permitting -** Bob described the submittal package that will be included in the 404 permit. Bob also will talk to Ron Fowler about the 404 permit and to see if the local flows that will bypass the inlets are adequate to feed the downstream wash segments. If they are not, either low flow by-passes are to be provided to feed the downstream washes from the main channel or mitigation will have to be provided.	***Bob*** *is going to talk to Ron Fowler about the 404 permit to determine the impacts of the proposed project.*
	New Business:	
3	**PAAC Meeting and Public Meeting No. 2 -** Due to Afshin's absence from the meeting, this topic will be discussed at a later date.	***Mark and Afshin*** *to convene and schedule the PAAC and Public Meeting for some time in October.*
4	**Utilities and Potholing -** Mark described the need to pothole the existing joint utility trench along the west side of the Hawes Rd channel (due to the proposed gravity wall along the west side of the channel). It was agreed that it is not necessary to pothole the small 25 pair telephone cable along Hawes Rd, north of Oak Street. Furthermore it was agreed not to pothole the existing waterline within Oak St. at the location where the SD crosses it. Instead, use existing pothole data and water valve nut elevations to estimate the elevation for the waterline and replace the ACP pipe with a DIP during construction.	***Mark*** *to set up a utility coordination meeting for sometime in October with: FCD, City of Mesa, SRP, COX and QWEST. The meeting is to be held at the city of Mesa.*
5	**Hawes Road SD Alignment -** Mark said that the Hawes Rd. SD alignment could move further to the west based on pothole information that indicates the 20" water line and 2" gas line were 2 feet further to the west than initially thought. However, Fritz suggested leaving the SD where it is shown in order to provide more separation from the water line because one never knows how the water main might act if exposed. Therefore it was agreed to leave the SD alignment as is because it is easier to brace the existing electrical poles than it is to deal with potential water line problems.	
6	**Oak Street SD Alignment west side of 87th Street -** Mark presented two options to maintain traffic flow during construction through the constricted section just west of 87th St. The first was to provide alternative access on Quill St., the second one was to obtain a TCE on the "constricting" property and remove & replace the wall in order to add extra width to keep traffic flowing during construction. However, Mark Lewis stated that the available 14 feet, without removiong the exisitng wall, is enough (8 ft minimum) to pass one-way traffic with a flagger. He added that at the end of each work day, the trench will have to be backfilled in order to allow for two-way traffic at night. Therefore, it was decided to leave the algnemtn where it is and plan to require one-way traffic with a flagger.	

7	**Backfill Requirements -** Mark Lewis and Fritz Huber provided the following suggestions for trench backfill: 1) Use MCDOT special provisions for pipe bedding slurry (half a sack of concrete per cubic yard of aggregate). 2) Under pavement (Oak St. SD), recommend using slurry backfill all the way up to the subgrade of the pavement, this would speed up the backfill process and avoid having to use compaction equipment and compaction testing to achieve the required backfill compaction requirements. Fritz also said it will be important to remove the requirement of waiting 24 hours before backfilling the slurry, because the trench will need to be backfilled at the end of each work day. 3) Under unpaved areas (Hawes Rd. SD), recommend using slurry backfill up the storm drain spring line with the remainder of the bedding and backfill per MAG standards. However, allow for alterntive backfill if the contractor deems it cheaper to backfill the whole trench with slurry. 4) Use slurry backfill around all manholes	
8	**Alternative to Drop Structures -** Mark explained that the drop structures will be expensive and difficult to construct in the available right-of-way. An alternative to the drop structures is to replace them with short segments of steep pipe; using either pre-fabricated bends or manholes to achieve the vertical deflections. The short pipe segments would be subject to velocities greater than 20 fps, but to counteract the potential pipe abrasion, the short segments would be specified to be Class V pipe which has a thicker wall and made with higher strength concrete.	***Gavan & Barker*** *to provide an exhibit and explanation to the District for their approval of the proposed pipe chutes.*
9	**Curved Pipe Sections -** It was agreed that it is impractical and overly expensive to build the storm drains on a curve. The curved sections in the storm drains will be replaced with pre-manufactured bends or manholes; whichever is the most economical.	

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Report of Meeting (revised 9/24/10)

Oak Street Basin and Storm Drain Design

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Contract FCD 2009C021
Date: 9/16/2010
Location: FCD - New River Conf. Room
Purpose: Review of 60% Basin Design

Item No.	Item/Notes:	Action:
1	**Introductions -** See list of Attendees below	
2	**Basin Slopes in Granite Bedrock -** Dennis explained that the District would prefer steep rocky slopes, without landscaping, on the north and east sides of the basin in order to reduce the basin footprint and eliminate the potential problem of top soil eroding off of the basin slopes. He said the railing could be placed at the top of the steep slopes. He also said that they had envisioned vertical slopes at the bottom of the basin instead of the 1.5H:1V slopes indicated in the 60% plans. Mark expressed a concern with vetical walls at the bottom of the basin without additional railing, if higher than about 3 feet, because of the potential for someone to climb over the uphill railing, hike down the uphill slopes and then be surprised by a vertical wall; creating a potential fall hazard for the public as well as maintenance staff. Scott suggested that we estimate the depth to the moderately weathered bedrock and establish that as the top of the steep rocky slopes, above which the slopes will be 4H:1V, or flatter. He also said that we could provide direction to the contractor in the special provisions that will allow the top of slope to vary based on field conditions. He suggested that RAMM could monitor the excavation and make decisions in the field regarding the slope stability of the weathered granite. Dennis suggested that a field trip should be arranged to observe existing cut slopes in the vicinity so that we can all see what the basin slopes might look like and how they might hold up to the erosive effects of runoff.	***Afshin*** *to schedule a field meeting to observe existing granite cut slopes.* ***Gavan & Barker*** *to revise the basin design to incorporate steeper, rocky slopes along the north and east sides of the basin and* ***(added 9/24/10 - reset the limits of disturbance line to preserve a larger area of native vegetation.*** *The revised design will be submitted for review by District staff prior to submittal of the 90% plans).*
3	**Potential Spillway for Wash at NE Corner of Basin -** Several members of the District staff expressed a desire to provide a rocky spillway for the wash that enters at the NE corner of the basin in lieu of the current pipe design. Kip explained that the upper portion of the granite bedrock is very weathered and would be susceptible to erosion from flows spilling over the basin edge. There was considerable discussion surrounding the spillway idea including the use of a ford crossing for the maintenance road and grouted rip rap (or some other type of erosion protection) for the spillway.	***Gavan & Barker*** *to evaluate alternative spillway ideas for the wash that enters at the NE corner of the basin in lieu of the current pipe design.*

4	**Basin Landscaping in Rocky Soils -** Harry and Dennis suggested that we provide at least 12 inches of over excavation and top soil replacement where we intend to plant native seed mix and to have at least 3 feet of over excavation and soil replacement where we place Tall Pot trees. Mark suggested, as a cost saving measure, to allow granite rock mounds in the bottom of the basin, rather than over excavate the entire basin floor. **(added 9/24/10 - Harry asked that we review the landscape plan to ensure that salvaged plant materials would be located in areas with sufficient soil depth to accommodate the existing root ball, enable replanting and encourage survival, particularly on basin side slopes that may not have enough topsoil plating and/or sufficient highly-weathered rock).**	***Gavan & Barker*** *to revise the basin landscaping areas in rocky soils which will include a combination of leaving the granite bedrock without vegetation, over excavating 12 inches in areas for desert grasses and over excavating 36 inches in areas where Tall Pot trees are to be planted.*
5	**Basin Excavation -** Excavation in the slightly weathered granite in the bottom of the basin could be difficult and it was suggested that blasting be allowed in order to attract competitive bids for the basin excavation. Scott said that we could allow blasting at the contractor's option just so we adequately cover it in the special provisions. **(update 9/24/10 - Scott reviewed the notion of blasting with FCD Construction staff and, due in great part to public perception and potential negative reaction, basin excavation must be accomplished by mechanical means only)**	***Gavan & Barker*** *to provide language in the special provisions that makes it clear that no blasting will be allowed on the project*

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X	Harry Cooper, FCD		

Report of Field Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021

Date: 9/27/2010

Location: Project Site

Purpose: Field Review of Oak Street SD and Rock Cuts

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:
1	**Oak Street SD at Constricted Section west of 87th Street -** The group visited the constricted section in front of the home that lies west of 87th Street. The width between Thunder Mountain's wall and the resident's wall in this segment of Oak Street is only about 33 feet wide. Fritz Huber and Mark Lewis both thought that the 14 foot offset shown in the 60% plans is the most appropriate location for the storm drain and that it provided enough space for the Contractor to flag traffic around the trenching operation. Scott added that we should revise the plans to continue the 14 foot offset farther to the west in order to minimize reconstruction of the circular driveway and landscape in the front yard of the house to the west that lies between station 25+00 and 26+50.
2	**Rock Cut Observation on Range Rider Trail -** The group visited a roadway rock cut slope on the south side of Range Rider Trail about 1/4 mile east of Oak Street. Subsequent review of the grading plans for the Granite Mountain subdivision indicates that this cut is 28 feet deep, so it is very similar to the proposed cut at the northeast corner of the Oak Street Basin. The group discussed creating a similar rock cut face on the north and east sides of the basin. Harry Cooper said that he would provide sample plans from an ADOT project that provides guidance for treatment of rock cut slopes and clearing limits that could be used for the basin construction (Afshin since provided said information to Gavan & Barker on 10-13-10).
3	**Spillway at Hawes Road Basin -** The group visited the Hawes Road Basin to observe the spillway at the northeast corner which accepts inflow from an upstream wash. In lieu of the piped inlet shown at the northeast corner of the Oak Street Basin 60% plans, it was agreed that we would employ a spillway design similar to the Hawes Road Basin; including the construction of a concrete cutoff wall that will protect the perimeter maintenance road and prevent upstream head cutting.
4	**Other Aesthetic Elements -** Mark, Dennis and Harry visited the locations of other, above ground significant project elements; including the two large inlet structures that will be located along the north side of Oak Street and the flood wall along the Hawes Road Channel. Dennis and Harry were satisfied that the design approach to the inlet structures; incorporating short, painted, stuccoed walls to contain the floodwaters was an appropriate treatment. In the case of the flood wall on Hawes Road, Dennis suggested that we paint the wall to match the color of the Thunder Mountain subdivision wall; instead of using integral color concrete as shown in the 60% plans.

Attended	Name & Organization:	Email:	Phone:
X	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
X	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
X	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
X	Dennis Holcomb, FCD	dbh@mail.maricopa.gov	602-506-4074
X	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
X	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
X	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872
X	Mark Lewis, FCD	mkl@mail.maricopa.gov	602-525-6786
X	Fritz Huber, FCD	frh@mail.maricopa.gov	602-390-7802
X	Harry Cooper, FCD	harrycooper@mail.maricopa.gov	602-506-2956
X	Kip Reese, RAMM	kreese@rammeng.com	480-921-8100

Report of Meeting

Oak Street Basin and Storm Drain Design

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Contract FCD 2009C021
Date: 10/19/2010
Location: FCD - New River Conf. Room
Purpose: Monthly Meeting - Mtg10

Item No.	Item/Notes:	Action:
1	**Introductions -** See attached list of attendees.	
	Old Business:	
2	**404 Permitting -** Bob spoke to the Corps and they told him that the previous JD done for the ADMP should be used for the Oak Street SD project, regardless of the subsequent upstream Granite Mountain subdivision JD that did not identify the wash crossing Hawes Road as jurisdictional. The group agreed to eliminate the inlet bypass structures in favor of paying in lieu fees for offsite mitigation. Bob to work with the Corps to determine the amount of mitigation required.	
	New Business:	
3	**PAAC Meeting and Public Meeting No. 2 -** Mark and Afshin agreed that the PAAC Meeting is to be held either on November 3rd or 4th and the Public Meeting is to be held one week later on either November 9th or 10th (dates are dependant on room availability). The mailers for the public meeting are prepared, the only thing missing is the date of the meeting.	***Grant Wegner*** *and* ***Lori Pearce*** *to set up dates for PAAC and Public Meetings (done 10-21-10, PAAC meeting has been set up for the Nov. 3 and public meeting will be Nov. 9)*
4	**Utilities and Potholing -** Mark and Gary agreed on a time for the utility coordination meeting. The meeting is to be held on November 4th at the City of Mesa around 1pm.	***Mark*** to send out invites to: FCD, City of Mesa, SRP, COX and QWEST (done 10-25-10, utility meeting has been set up for the Nov. 4, 1:00 at the city of Mesa).
5	**Drop and Junction Structures -** Gavan & Barker previously provided the District and the City of Mesa with a chute design as an alternative to the drop structures. Both the District and the City rejected the alternative, citing concerns of high velocities within the chute section, abrasion to the velocity rings, and sediment deposition. As an alternative to the cast-in place structures, the District recommends pre-cast junction and drop structures. The design of the pre-cast structures is to be included in the construction plans. Due to the large size of these structures, they might have to be pre-cast in several pieces and erected on site. Gavan & Barker shall coordinate design of pre-cast structures with District staff including Mark Lewis, Jeff Riddle and Kumar. After the meeting, Gary Shapiro sent an e-mail indicating that the District would prefer the junction structure to be a pre-fabricated wye.	***Mark*** *to contact pipe manufacturers to determine the feasibility of fabricating an 84" x 72" x 45 degree wye.*
6	**Handrail at Drop Inlets -** The group agreed that handrails at the drop inlets are not necessary. Due to the size of the grates, access hatches at each inlet should be provided, in order for O&M to be able to access the drop structure.	***Scott*** *to send Gavan & Barker an example access door used on the BHOC project (done 10-21-10)*

7	**Hawes Road Channel -** Mark explained that the existing channel is 4" thick shotcrete with wire fabric. The 60% plans called for shotcrete to match the existing channel both in terms of structure and texture. The District, however, prefers concrete and requires rebar instead of wire fabric. The District will review their requirements and advise if shotcrete with rebar is an acceptable alternative to concrete. In regards to the floodwall/containment curb, the group agreed that the best location for it is at its current location immediately adjacent to the channel.	***Gary** to check to see if shotcrete is acceptable in lieu of concrete. (done 10-19-10, Gary sent e-mail indicating that the channel is to be 6" concrete with rebar, shotcrete not acceptable).* ***Mark** to meet with MCDOT in order to address their comments about moving the floodwall outside of the MCDOT right-of-way.*
8	**Inlet at Douglas Property -** It was agreed that the inlet along Hawes Road should be moved to the Douglas property and a drainage easement be obtained.	
9	**Concrete vs. Block Walls -** The group agreed that smooth painted concrete walls should be used at the inlets in the front yards and smooth stained concrete walls at the basin.	***Harry** to provide Gavan & Barker with the 3 preferred O&M paint colors.*
10	**Storm Drain Plans -** Mark explained that the maximum deflection that is achievable by pulling joints is 3/4", which for a 84" pipe equates to a deflection of about 0.5 degrees. In order to obtain larger deflections, a pipe collar will be required.	***Gary** to check to see if pipe collars will be acceptable in lie of pre-fabricated fittings (done 10-19-10, Gary sent e-mail indicating that deflections should be done using pre-fabricated bends).*
11	**Basin Design -** Due to time constraints, the basin design issues will be discussed in a separate meeting. The group agreed to a meeting sometime next week. Mark did explain, however, that MCDOT did not want to show any disturbance limits within their R/W surrounding the basin.	***Mark** to discuss disturbance limits with MCDOT.*

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Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021
Date: 10/26/2010
Location: FCD - New River Conf. Room
Purpose: Basin Design Meeting

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602 200 0031

Item No.	Item/Notes:	Action:
1	**Introductions -** See attached list of attendees.	
	Old Business:	
2	**PAAC Meeting and Public Meeting No. 2 -** Mark confirmed that the PAAC Meeting is scheduled for 11/3/10 and the Public Meeting is to be held on November 9th. Lori Pearce prepared the mailers for the public meeting and the District was to mail out. The understanding is that Gant Wegner was coordinating the contacts for the PAAC and the mailing.	*Scott will follow up with Gant to verify PAAC and Public meeting coordination.*
	New Business:	
3	**Basin adjustments:** John discussed the adjustments made to the basin design to reduce the overall basin disturbance footprint. The buffer areas on the south and west sides have increased slightly from the 60% plans footprint, with 50 feet minimum on the west side and 40 feet minimum on the south side. The slopes on the north and east sides of the basin were increased to generally 1:1 or steeper for the bottom 10 - 20 feet. Although the use of steeper rock faces will help in reducing the footprint, the impact is not too significant because the steeper slopes are in the lower elevations of the basin. Drainage swales on the uphill side of the maintenance road were added which also increased the footprint. Storage calculations are yet to be made, but it appears that the footprint will be less than the 60% plans, but relatively close to what was presented in the first public meeting. The team generally concurred with the concept of the steeper slopes, but would like to see more buffer area along Hawes Rd. and Oak St. if possible.	
4	**Buffer on Hawes Rd.:** Harry pointed out the significant vegetation along Hawes Rd. and recommends shifting the berm toe of slope to the existing trees drip line if possible to preserve the large trees. Dennis concurred and recommends steepening the flatter slopes (flatter than 4:1) in order to preserve more vegetation and along Hawes Rd.	*G&B will strive to save the larger trees and provide a larger buffer along Hawes Rd.*
5	**Construction Access and Stockpiling:** The team discussed the approach to how/where the contractor will stockpile the various topsoil, salvaged rock, and top dressing materials, in addition to the location of the plant nursery. Scott recommended allowing the contractor to coordinate these activities either on or off site. **(Added 10-29-10) Harry added: The team agreed that no construction areas or temporary nurseries are to be developed outside of the basin footprint for the stockpiling of materials or holding of salvage plant materials.**	
6	**Rock face aesthetics:** Dennis and Harry provided examples of the desired rock face appearance with the goal of avoiding the mini-benching look. **(Added 10-29-10) Harry added: Add language in the SPs to clearly identify the intent of the work related to this item so the Contractor understands that additional work/refinements will occur per field direction given by the Engineer and the District's Landscape Architect. Be sure to include time in your Post Design Services Contract for LA and Engineer field direction services to accomplish this task.**	

7	**Over Excavation at basin bottom:** John discussed the concept for the basin bottom treatment with rock faces/ledges left un-excavated, over-excavated planting pockets, and seed areas. Dennis/Harry concurred with the concept except the 1-ft over excavation for seed mix is not necessary. Preference would be to deepen the pockets and reduce the surface size to approximately the drip line of proposed trees. Mark discussed the potential rough surface for the maintenance road in the bottom. Team agrees that a specified gradation of on-site rock material could be used to smooth out/make the road useable for trucks. **(Added 10-29-10) Dennis recommended that the under-excavated islands and "formations" in the basin bottom should be sized large enough to be visually significant and scaled relative to the size of the basin.** **(Added 10-29-10) Harry added: Add language in the SPs to clearly identify the intent of the work related to this item so the Contractor understands that additional work/refinements will occur per field direction given by the Engineer and the District's Landscape Architect. Be sure to include time in yo Design Services Contract for LA and Engineer field direction services to accomplish this task.**	
8	**Allowance for rock staining:** With the uncertainly of the amount exposed rock surfaces and the necessity for staining, the team agreed to provide an allowance for rock staining. **(Added 10-28-10) The allowance will be biddable based on a square yardage measured as an approximate projection parallel to the slope.** **(Added 10-29-10) Harry added: Add language in the SPs to clearly identify the intent of the work related to this item so the Contractor understands that additional work/refinements will occur per field direction given by the Engineer and the District's Landscape Architect. Be sure to include time in your Post Design Services Contract for LA and Engineer field direction services to accomplish this task.**	
9	**Spillway:** Mark explained the concept for the spillway at the northeast corner of the site. A concrete dip section in the maintenance road with 6-ft cutoff walls, similar to the outlet headwall of an ADOT box culvert. The top 2-3 feet of loose soil material will be removed and the natural rock surface will be the spillway. The team concurs with the approach and all agree that there may be the need to repair the spillway in the future depending on the erosiveness of the rock surface. The team agreed that the roadway dip section should be raked concrete with a stain applied.	
10	**Railing:** John discussed the placement of a barrier railing above the steeper slopes. The railing will be placed 3 - 5 feet away from the top of slope and be stained. **(Added 10-28-10) Scott added: the railing layout shall be smoothed out, removing sharp corners.**	
11	**Cacti Salvaging:** The team discussed the approach to salvaging the non-tagged cacti. John explained the approach to salavaging a percentage of the transect estimated smaller cacti and cholla in addition to field tagged plants. The team concurred to use the percentage approach, and specify that the remaining cholla will be scattered on the ground surface; and then provide an allowance for the salvaging and transplanting of saguaros, barrels, and hedgehogs that are in addition the required numbers shown on the plans. **(Added 10-29-10) Harry added: Add language in the SPs to clearly identify the intent of the work related to this item so the Contractor understands that additional work/refinements will occur per field direction given by the Engineer and the District's Landscape Architect. Be sure to include time in your Post Design Services Contract for LA and Engineer field direction services to accomplish this task.**	*G&B will include specifications to address.*

12	**Saguaro Method of Measurement:** John explained the approach identified in the 60% plans to pay for saguaros based on four categories: Once moved, 0-6', >6-12',>12-20'. Based on the ADOT method, this approach reduces the effort required by the District to measure and compare contractor's pay requests. The bidders will be able to provide unit prices for the four categories based on the linear feet provided in the plant inventory. The pros and cons were discussed, and the team concurred with using this method rather than the price per linear foot method.	
13	**Desert Cobble:** Harry explained the District's desire to provide a surface cobble to the finished slopes to emulate the existing ground surface. The team discussed the approach and determined that providing this treatment over all the slopes may be too costly and that the existing rocky soils should provide this aesthetic with water settling. Surface cobble will be collected, stockpiled, and applied to more highly visible slopes in the areas that receive the top dressing material. **(Added 10-29-10) Harry added: Reinforce the definition of Top Dressing to include: all dead/decaying organic materials of visual interest (ie saguaro ribs and skeletons, dead ocotillo and cholla trunks), all surface cobble larger than 3". Topsoil shall be excavated to a 12" depth. Quantities should be calculated to ensure minimal excess or shortage of topsoil during plating work.**	
14	**Seed mix:** Harry mentioned that he has some additions he would like to include in the seed mixes and will provide. The District recommends applying the seed mix to all portions of the site including exposed rock surfaces.	
15	**PAAC Exhibits:** The District will bring the photos of the rock slopes and G&B will bring the current grading plan. The photos will be mounted for the public meeting.	

Attended	Name & Organization:	Email:	Phone:
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x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
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Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021

Date: 11/3/2010

Location: Mesa Fire Station #216, Community Room

Purpose: Project Aesthetics Advisory Committee Meeting #2

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

	Item/Notes:
1	**Introductions -** see list below for attendees
2	**Project Update -** Mark provided a project update which included a description of the following plan refinements: *1) Elimination of Basin Bypass Pipes* - The basin bypass pipes which were meant to allow low flows to bypass the basin and drain directly to the Hawes Road channel have been eliminated from the project. This includes both the Hawes Road and the Oak Street bypass pipes. The primary reason for eliminating the pipes is due to their susceptability of clogging from anticipated upstream sediment loads. *2) New Curb along Hawes Road Channel* - The hydraulic analysis indicates that, although the existing channel has sufficient capacity to convey the basin outflow, it does not meet the minimum freeboard requirements. Therefore, a curb has been added (painted to match the color of the existing Thunder Mountain wall) to some segments of channel where additional freeboard is necessary. The height of the curb is generally 6 to 18 inches high with one short segment to be 24 inches high.
3	**Questions from last PAAC Meeting -** The following questions were asked at the first PAAC meeting. *1) Future upstream development* - Mark explained that much of the undeveloped land in the upstream watershed is US Forest Land or Usery Mountain Park land, neither of which will be developed. The privately owned undeveloped land is zoned R1-90 indicating 2 acre lots and, if it is developed, the City of Mesa will require storm water retention, so there should not be an increase in runoff due to future development. *2) Inlet bypass flows* - Mark explained that while some local runoff will continue to collect in the downstream washes, the plan is to not build bypass structures at the inlets. *3) Basin drain time* - Mark said that the basin will take 9 hours to drain during a 100-year flood; most floods will drain in much less time.
	4) Funding for Construction - Scott and Shahir explained that the funding for the project is intended to be a cost share between the County and the City, but the storm drain bond issue that the City needed to fund their share of the project was not added to the bond initiative. Alternative funding is being explored, but in the meantime, the project is not fully funded
4	**Refinement of Basin Design -** John presented an updated basin plan which includes the addition of a maintenance road along the east and north sides of the basin and significantly more exposed rock slopes than what was preseneted at the first PAAC meeting. He explained that the geotechnical exploration found rock, below depths of 2 to 5 feet. So the plan has been refined somwhat to take advantage of the rock by incorporating rock cut faces into the north and east sides of the basin which will be steep and will have little or no vegetation. These rock slopes will also require the addition of a railing at the top of the rock slopes for safety reasons. Besides the aesthetic advantages of creating natural looking rocky slopes, the steeper slopes also allowed the footprint of the basin to remain essentially the same as the initial design, even with the addition of the maintenance road and the extra volume required due to eliminating the basin bypass pipes.
	He also explained that elimination of the Hawes Road basin bypass pipe enables more of the existing vegetation along Hawes Road to be preserved. In addition, the existing wash that enters the basin property will be allowed to spill over the rock slope of the basin, rather than constructing a special inlet structure as shown in the initial design plans.
5	**Inlet Design:** Mark presented the inlet design that will be employed to intercept runoff at the natural washes. The design includes the use of a drop inlet structure similar to the ones constructed within the Granite Mountain subdivision along Range Rider Trail. The drop structure design allows the inlet to be constructed without regrading the channel bottom; perserving the character of the natural wash. The inlets will include low walls that blend with the landscape and contain the floodwaters; helping to direct flows into the drop inlets.

6	**Discussion:**
	New Curb along Existing Channel - Bob expressed a concern with liability of the curb being an attractive nuisance for kids to walk on and potentially fall down the steep, concrete channel slope. Scott said that we would endeavor to limit the height of the curb to reduce the potential risks. Bob also expressed a concern with damage to the landscape during construction. Scott explained that the project plans and specifications will require the Contractor to repair any irrigation facilities that are damaged and to replace any plants that are destroyed during construction.
	Inlet Bypass Flows - Both Tom and Bob were in favor of not having inlet bypass flows discharging to the downstream washes. Scott cautioned them that the Corps of Engineers still may require the bypass at the eastern most wash into the Thunder Mountain subdivision because it has been identified as a jurisdictional wash under Section 404 of the Clean Water Act.

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x	Bob Nowak (Thunder Mtn. HOA)	jrnowak79@earthlink.net	480-354-1697

Report of Meeting

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021
Date: 11/16/2010
Location: FCD - New River Conf. Room
Purpose: Monthly Meeting - Mtg11

Item No.	Item/Notes:	Action:
1	**Introductions -** See attached list of attendees.	
	Old Business:	
2	**404 Permitting -** Bob asked for more information regarding residual flows in the 404 washes.	***Gavan & Barker*** *to provide Bob with the 2-year residual flow (and backup calculations) for the 404 wash through Thunder Mountain*
	New Business:	
3	**Hawes Road Channel Curb -** Mark explained that the Thunder Mountain community is concerned with the bigger curb that is higher than 12 inches from both a safety and aesthetic standpoint. If we relax the freeboard requirement to the results of Eq. 6.25 (Hydraulics Manual) or one foot, whichever is greater; we would reduce the length of the curb from 1300 feet down to about 600 feet. More importantly, the height of the curb would be 6 inches or less in all but one short segment. The minimum one foot freeboard requirement is what is usually applied to subcritical channels, whereas two feet minimum is applied to supercritical channels. The Hawes Road Channel does flow at supercritical depth, but the peak discharge is relatively small at 182 cfs and the depth of flow is shallow; potentially providing justification for relaxing the freeboard requirement.	***Steven*** *to review this request with Amir and get back to us (Done 11-19-10: Steven told us to use 2 foot minimum as outlined in the Hydraulics Manual)*
4	**Oak Street Paving -** It was agreed that we would re-pave the entire length of the Oak Street storm drain, even though MCDOT only asked for the pavement in their R/W. We will re-pave the entire street from the basin to the last inlet, east of 87th Street.	
5	**Junction Structure vs Pre-Cast Wye -** It was agreed that we would call oput a pre-cast wye in the plans, even though there may only be one manufacturer (Ameron) who can constrcut the wye.	
6	**Revised Basin Design -** The revised basin design submitted on 11-10-10 is acceptable to the District with some added planting areas described by Harry. Afshin is going to check with the Operations and Maintenance division to make sure the design is acceptable to them.	***Afshin*** *to gain concurrence from Operations and Maintenance staff on revised basin design (Done 11/19/10: Afshin said that O&M reviewed and accepted the new basin design)*
7	**60% Review Comments**	
	Strom Drain Stationing - Agreed with the Group and later with Scott Vogel that we can keep stationing on the storm drain. Scott also asked that we keep all stationing unique. That is, keep that stationing on Oak Street different from Hawes Road.	
	Inlet on Hawes Road - The group agreed that inlets are not necessary along the edge of pavement on Hawes Road. The flows associated with the swales are very small and some flow needs to flow over Hawes to feed the downstream 404 wash.	
	Retaining Wall at Granite Mountain - The group agreed to have the wall protected in place rather than to have it removed and replaced.	

	Inlets behind Sidewalk on Oak Street - The group agreed and later confirmed with Scott Vogel to not install inlets in the landscape area behind the sidewalk, accept at the western most inlet on the Oak Street storm drain before it discharges to the basin. At this inlet, Scott asked to include an inlet behind the sidewalk that will connect to the new curb opening inlet.		
	Bollards/Railing in Front of Drop Inlets - The group agreed to not install bollards and or railing in front ot the drop inlets. The feeling is that the angled grate should do an acceptable job of preventing debris clogging.		
	Inlets at Thunder Mountain Wall Openings - The group agreed to install inlets at the two western most wall openingas, but not at the large 404 wash opening at the east end of the strom drain.		
11	**Other -** Afshin provided revised drainage easement boundaries to Gavan & Barker to have incorporated into the plans.		
Attended	**Name & Organization:**	**Email:**	**Phone:**
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Report of Meeting

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021
Date: 12/21/2010
Location: FCD - New River Conf. Room
Purpose: Monthly Meeting - Mtg12

Item No.	Item/Notes:	Action:
1	**Introductions -** See attached list of attendees.	
2	**Utility Coordination**	
	Additional Pothole - Mark explained that an additional pothole is necessary to determine if the new inlet at Oak St. SD Station 175+82 is in conflict with the existing SRP electric conduits.	*Gavan & Barker will send request to Afshin for additional pothole.*
	SRP Crossing at Oak Street Station 166+10 - SRP is going to make a field visit to determine if their facilities were all located with the pothole. If they find that they have more facilities than what the pothole indicated, Gavan & Barker will send TBE back out to re-dig the pothole.	
	Utility Crossing/Conflict Summary - A summary of the utility crossings and utility conflicts will be distributed once SRP completes their field investigation.	*Gavan & Barker will distribute a summary of utility crossings/conflicts to the utility companies and the District.*
3	**Right-of-Way -** Mark explained that a R/W map was prepared using the District's guidelines, but that it needed to be reviewed.	*Scott and Afshin will review the R/W map and let Gavan & Barker know if any revisions are necessary.*
4	**Oak Street Residents -** Afshin said that he would schedule a meeting with the residents along Oak Street that will be impacted by the new drop inlets.	*Afshin will schedule a meeting with the Oak Street residents.*
5	**Douglas -** Afshin asked that Gavan & Barker schedule a meeting with Cliff Douglas to explain the need for the inlet at the SW corner of his property.	*Gavan & Barker will schedule a meeting with Mr. Douglas for the first week in January.*
6	**MCDOT -** Afshin asked that we follow up with Sakda regarding the CLSM trench backfill and explain that the permanent backfill will not be placed over the CLSM until it has had at least 24 hours to cure.	*Gavan & Barker will send a follow up e-mail to Sakda.*

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
x	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
	Dennis Holcomb, FCD	dbh@mail.maricopa.gov	602-506-4074
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872
	Jeff Riddle, FCD	jrr@mail.maricopa.gov	602-506-4602
	Daniel Michael, FCD	dam@mail.maricopa.gov	602-506-4873
x	Mark Lewis, FCD	mkl@mail.maricopa.gov	602-525-6786
	Fritz Huber, FCD	frh@mail.maricopa.gov	602-390-7802
x	Harry Cooper, FCD	harrycooper@mail.maricopa.gov	602-506-2956
x	Gant Wegner, FCD	gantwegner@mail.maricopa.gov	602-506-7841

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021
Dates: 1/31/2011 and 2/2/2011
Location: FCD - New River Conf. Room
Purpose: Monthly Meeting - Mtg13 and 90% Comment Resolution

Item No.	Item/Notes:	Action:
1	**Introductions** - See attached list of attendees.	
2	**Structural Engineering** - Discussed Kumar Hanumaiah 90% Comments. Kumar was concerned that the drop structure dimensions are too small; not providing enough space between the outside wall of the connecting pipes and the inside wall and/or slab of the drop structure.	*Premier will coordinate with Kumar on the dimensions of the drop structures (completed 2/10/11).*
3	**Utilities** - Discussed Gary Maiers 90% Comments	*Gavan & Barker will prepare a written response to Gary's comments (completed 3/9/11).*
4	**Mesa 90% Comments** - Discussed Mesa 90% Comments	*Gavan & Barker will prepare a written response to Mesa's comments (completed 3/9/11).*
5	**Landscape** - Discussed Harry Cooper 90% Comments	*Gavan & Barker will prepare a written response to Harry's comments (completed 3/9/11).*
6	**Engineering** - Discussed Gary Shapiro 90% Comments	*Gavan & Barker will prepare written response to Gary's comments (completed 3/9/11).*
7	**Hydrology/Hydraulics** - Discussed Steven Tucker 90% comments	*Gavan & Barker will prepare written response to Steven's comments (completed 3/9/11).*
8	**Hydraulics** - Discussed Rafeal Pacheco 90% comments. Agreed to have RAMM conduct a geotechnical analysis of the basin embankment.	*Gavan & Barker will request a proposal from RAMM to conduct the geotechnical analysis (completed 2/7/11). Gavan & Barker will prepare a written response to Rapheal's comments (completed 3/9/11).*
9	**Other** - Discussed 404 permit. Bob Stevens expressed concern over diminished flows in downstream 404 washes; particullarly the wash through Thunder Mountain. Mark Gavan suggested providing the Corps with the post project downstream flows to demonstrate that there will still be substanital local inflow that will get to the washes.	*Gavan & Barker to send Bob watershed maps and post project peak discharges for the downsteam 404 washes (Completed 2/1/11).*

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
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x	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
x	Dennis Holcomb, FCD	dbh@mail.maricopa.gov	602-506-4074
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
x	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
x	Steven Tucker, FCD	slt@mail.maricopa.gov	602-506-4872
x	Harry Cooper, FCD	harrycooper@mail.maricopa.gov	602-506-2956
x	Kip Reese, RAMM	kreese@rammeng.com	480-921-8100
x	Kumar Hanumaiah, FCD		
x	Sergio Oliden, Premier	soliden@pre-engr.com	480-829-6000
x	Rajesh Dholokia, Premier	rdholokia@pre-engr.com	480-829-6000
x	J Rapheal Pacheco, FCD	josepacheco@mail.maricopa.gov	

Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021
Date: 5/9/2012
Location: FCD - New River Conf. Room
Purpose: Monthly Meeting

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:	Action:
1	**Introductions -** See attached list of attendees.	
2	**Geotechnical Field Exploration -** Kip Reese presented the results of RAMM's field exploration efforts and explained that they are in the process of conducting the laboratory analysis. They have found cemented soils at shallow depths along most of the dam alignment, rather than the granite that they were expecting to find. He said the cemented material should be suitable, competent material for the dam foundation and will likely be somewhat easier to excavate than granite. He said that the cemented soils are typically only a few feet below the surface soils, but are deeper within the existing washes. He also said that results of the penetration testing would indicate that the washes may be fairly well incised in the cemented material, with fairly steep side slopes.	
3	**RCC Dam Design -** Brett presented an initial draft of the dam cross section and explained that the dam foundation soils will not be an exact elevation, but instead will be undulating and irregular. He showed photos of the Spook Hill foundation as an example. The group agreed that we will have a design profile with calculated base bid quantities and include allowances in the contract for additional excavation and additional RCC to cover the Contractor's expense if he has to go deeper to reach competent material. Afshin mentioned that Mike Greenslade was concerned with the notion of placing the dam on cemented soils. Brett said that he would talk to Mike. Dennis said that it may be desirable to expose the downstream face of the dam, since it will only be a few feet high and will look like a wall. For that reason, as well as limiting disturbance, the downstream face of the dam should be as vertical as possible.	***Brett to talk to Mike Greenslade about founding the dam on the cemented soils.***
4	**Basin Redesign and Outlet Storm Drain Alignment -** Mark explained that the initial dam design cross sections presented by AMEC may result in more disturbance to the existing vegetation along the downstream face than was originally envisioned. Dennis said that, while we need to preserve as much vegetation as possible, we can revegetate those areas that are disturbed and possibly add more salvaged plant material to the perimeter of the dam to replace vegetation that has to be removed. Mark also presented an alternative alignment for the basin outlet pipe that positions the outlet structure farther east in a location along the maintenance road where the elevation of the road is above the spillway elevation; providing dry access during times when the basin is full to the top of spillway. Scott said that it would add costs to move the outlet farther east and that it may not be necessary. Mark added that the new position would make it more susceptible to sediment deposition from the Oak Street storm drain. **(post meeting discussion: for reasons of the added costs and the increased potential for sediment clogging, the decision was made to leave the basin outlet structure in the original location that was presented during the FMEA).** There was also a discussion on the possibility of adding salvaged trees to the perimeter of the basin, but since the FCD will be performing O&M and irrigation would have to be added to the project to establish the relocated trees, it was decided planting salvaged trees should not be the first option. Instead, we agreed to strive to make the current vegetation scheme work with the new RCC dam layout.	
5	**Project Schedule and Need for 95% Submittal -** Mark said that the project is on schedule except for the PAAC meeting which was scheduled to occur in the first week of May. There was much discussion regarding the need, purpose and the schedule for the PAAC meeting. It was agreed that we would schedule a PAAC meeting in the first week of June in order to show the PAAC members the change in the basin design and obtain feed back and possibly explore with them alternative aesthetic treatments for the of the dam. In regard to the 95% submittal, the group agreed that we would wait and see the amount of comments on the 90% submittal before we decide whether a 95% submittal is necessary.	***Afshin to schedule a PACC meeting for the first week of June (June 4 - June 6)***

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
x	Dennis Holcomb, FCD	dbh@mail.maricopa.gov	602-506-4074
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
x	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
x	Jake Serago, FCD	jakeserago@mail.maricopa.gov	602-506-4872
	Jeff Riddle, FCD	jrr@mail.maricopa.gov	602-506-4602
	Daniel Michael, FCD	dam@mail.maricopa.gov	602-506-4873
	Mark Lewis, FCD	mkl@mail.maricopa.gov	602-525-6786
	Fritz Huber, FCD	frh@mail.maricopa.gov	602-390-7802
	Harry Cooper, FCD	harrycooper@mail.maricopa.gov	602-506-2956
	Gant Wegner, FCD	gantwegner@mail.maricopa.gov	602-506-7841
	Michael Greenslade, FCD	mdg@mail.maricopa.gov	602-506-1501
x	Brett Howey, AMEC	brett.howey@amec.com	602-733-6021
x	Rafael Pacheco, FCD	josepacheco@mail.maricopa.gov	602-506-7841
x	Pedro Melo-Rodriguez, FCD	pedromelorodriguez@mail.maricopa.gov	602-506-0612
x	Kip Reese, RAMM	kreese@rammeng.com	480-921-8100

Report of Meeting

Oak Street Basin and Storm Drain Design
Contract FCD 2009C021
Date: 6/20/2012
Location: Mesa Fire Station #216, Community Room
Purpose: Project Aesthetics Advisory Committee Meeting #3

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

	Item/Notes:
1	**Introductions -** see list below for attendees
2	**Project Update -** Afshin reviewed the project background and explained the purpose of the PAAC meeting. Mark G. provided a brief review of project features: storm drains, basin, and outlet channel. Discussed the basin change from an earthen berm to a RCC dam due to potential seepage concerns. The characteristics of the dam were discussed including: the lowered elevation compared to the earthen berm design; the RCC concrete material; the RCC slopes; and benching to try to avoid the need for a railing at the top of the dam.
3	**Discussion:**
	Downstream Face of RCC Dam - Bill P. stated he prefers to fill against the exterior side of the dam so that the dam face is not exposed. The group discussed possible benefits of having a step up (exposed face) to demarcate a grade change to possibly eliminate a need for trespasser fall protection.
	RCC Color - Jerry S. stated he thought the concrete should be integrally colored, not stained. Harry C. provided information on the stain products that the FCD uses.
	Plant Salvage and Landscaping - John B. provided brief review of the proposed landscape salvage and planting for the basin.
	Perimeter Fencing - Discussion about the location of the wire access control fence. It is currently setback from the property line and may need to be relocated to the property line if County Land Management requires. PAAC discussed that it would be desirable to leave the fence set back so it is not as visible, like the Culver/Hawes Basin. Scott V. stated that the FCD will check the location of the fence at the Culver/Hawes basin, and if it is not on the property line then the Oak St. Basin can follow suit.
	Summary - General consensus was approval of the basin/RCC dam change with the recommendation to fill on the outside of the dam if it does not require the loss of additional existing vegetation. Furthermore, the PAAC recommended that the railing, if required, be installed on the inside slope to screen it from view.

Attended	Name & Organization:	Email:	Phone:
x	Harry Cooper, FCD	harrycooper@mail.maricopa.gov	602-506-2956
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
x	Gant Wegner, FCD	gantwegner@mail.maricopa.gov	
x	Pedro Melo-Rodriguez, FCD	pedromelorodriguez@mail.maricopa.gov	
x	Tom Smith (Thunder Mtn. HOA)	tksmithpe@gmail.com	480-380-3182
x	Cliff Douglas	machopolly@msn.com	480-986-9204
x	Jerry Seeman	Azgulch@yahoo.com>	480-380-9535
x	Bill Puffer	wjp5207@msn.com	480- 986-2655
x	Vince Kelly (Thunder Mtn. HOA)	vdennislee@msn.com	480-201-8896
x	Bob Nowak (Thunder Mtn. HOA)	jrnowak79@earthlink.net	480-354-1697

Report of Meeting

Oak Street Basin and Storm Drain Design

Contract FCD 2009C021

Date: 6/21/2012

Location: FCD - New River Conf. Room

Purpose: Monthly Meeting

Gavan & Barker Inc.
3030 North Central Avenue, Suite 1530
Phoenix, AZ 85012 Phone 602-200-0031

Item No.	Item/Notes:	Action:
1	**Introductions -** See attached list of attendees.	
2	**Geotechnical Field Exploration / Analysis -** RAMM has delivered a digital version of the DRAFT geotechnical report to Gavan & Barker for review. Afshin said that the District would like to go ahead and review the report now, prior to submittal of the 90% plans. He also said that a digital copy for review would be acceptable and ask that we include a copy in WORD so that comments can be made digitally on the WORD document.	***Gavan & Barker to submit the DRAFT report (PDF and WORD versions) along with digital copies of the previous reports to Afshin.***
3	**RCC Dam Design -** Mark and Brett presented the preliminary RCC Dam design and explained that, for design purposes, the bottom of the dam is going to be set two feet below the refusal depth on the penetration tests conducted by RAMM. This should provide a fairly conservative estimate of the limits of foundation excavation. AMEC is developing the RCC mix design. Brett said that the Contractor will have the option of using onsite aggregates, but that the mix design is being developed based on commercially available aggregate. Mike said that we should make sure that Fritz Huber reviews the specification for the RCC.	***Afshin to include Fritz Huber on the review team of the 90% plans and specs; most particularly for the review of the RCC Specifications***
4	**Basin Outlet Pipe Design -** Mark explained that the revised outlet pipe alignment that was proposed in the FMEA will not work. The required increase in pipe length coupled with the lower available (lower dam) requires a much larger pipe than will fit over the top of the downstream sewer. Therefore, the plan is to go back to the original design alignment for the outlet pipe which results in a much shorter pipe length, but due to the lower head, the outlet pipe will have to be upsized from 42 inches to 48 inches. This will require a minor redesign of the outlet structure. Afshin asked that we obtain a price proposal from Premier for the outlet structure redesign. Mike asked that we document this change from the FMEA, as well as any other changes (like the cemented soils versus granite bedrock) within the Dam Design Report. The outlet pipe will be excavated in the cemented soil and the pipe trench will be backfilled with CLSM. Scott mentioned to make sure that Premier check with Kumar on the design revision before finalizing the structure plans.	***Mark to send a price proposal to Afshin for the redesign of the basin outlet structure. Premier to talk to Kumar about the redesign of the outlet structure.***
5	**Basin Design -** Mark presented the revised basin design and explained that in order to have a sound foundation for the RCC, there will be a bench about 12 feet wide adjacent to the RCC Dam that is 4 to 6 feet above the basin floor. From an aesthetic standpoint the group agreed to make the bench more freeform to match the rest of the basin, add fill against the inside face of the dam to hopefully eliminate the need for railing (see results of PAAC meeting), add tall pot tree pods adjacent to the dam (Brett to provide guidance on minimum separation between the trees and the RCC dam). It was also discussed that the islands in the middle of the basin may not be desirable depending on the appearance and consistency of the cemented soils. Scott suggested that we include language in the special provisions that would allow the District to require the contractor to grade the bottom of the basin flat at the same contract unit price bid for basin excavation.	***Gavan & Barker to add language to the special provisions which will give the District the option of having the bottom of the basin graded flat.***
6	**Utilities -** Afshin asked if the new design would result in any changes to the utility conflicts previously identified. Mark said that since the outlet pipe is going to built in the same alignment as the previous design, there won't be any changes required for the utility relocation work. The telephone and electric conduits that had to be relocated for the 42 inch pipe will still need to be relocated in the same manner to accommodate the proposed 48 inch outlet pipe.	
7	**Results of PAAC Meeting - Mark explained that the general consensus was approval of the basin/RCC dam change with the recommendation to fill on the outside of the dam if it does not require the loss of additional existing vegetation. The PAAC also recommended installing the railing, if required, on the inside slope of the dam to screen from view.**	***Afshin to set up meeting with O&M to check maximum fall heights allowable within the basin***

Attended	Name & Organization:	Email:	Phone:
x	Afshin Ahouraiyan, FCD	afa@mail.maricopa.gov	602-506-4515
	Bob Stevens, FCD	rbs@mail.maricopa.gov	602-506-4073
x	Gary Maiers, FCD	gsm@mail.maricopa.gov	602-506-0522
x	Gary Shapiro, FCD	ghs@mail.maricopa.gov	602-506-3076
x	John Barker, Gavan & Barker	jbarker@gavanbarker.com	602-200-0031
x	Dennis Holcomb, FCD	dbh@mail.maricopa.gov	602-506-4074
x	Mark Gavan, Gavan & Barker	mgavan@gavanbarker.com	602-200-0031
x	Omer Karovic, Gavan & Barker	okarovic@gavanbarker.com	602-200-0031
x	Scott Vogel, FCD	csv@mail.maricopa.gov	602-506-4771
	Shahir Safi, City of Mesa	shahir.safi@mesaaz.gov	480-644-4292
x	Jake Serago, FCD	jakeserago@mail.maricopa.gov	602-506-4872
	Jeff Riddle, FCD	jrr@mail.maricopa.gov	602-506-4602
	Daniel Michael, FCD	dam@mail.maricopa.gov	602-506-4873
	Mark Lewis, FCD	mkl@mail.maricopa.gov	602-525-6786
	Fritz Huber, FCD	frh@mail.maricopa.gov	602-390-7802
x	Harry Cooper, FCD	harrycooper@mail.maricopa.gov	602-506-2956
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	Pedro Melo-Rodriguez, FCD	pedromelorodriguez@mail.maricopa.gov	602-506-0612
x	Kip Reese, RAMM	kreese@rammeng.com	480-921-8100

SURVEY

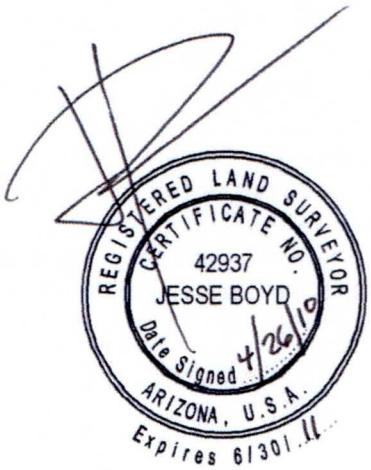
PREMIER ENGINEERING CORPORATION

April 26, 2010

Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

Attn: Mark T. Gavan, PE

Re: Land Survey Report
Contract FCD 2009C021
Oak Street Basin and Storm Drain



Mr. Gavan, et al.:

As per Section 3.2.8 "Survey, Photogrammetry, and Mapping" of Flood Control District of Maricopa County's Consultant Guidelines Third Edition – December 1, 2003 – Revision 1, this memo summarizes our methodology and details key aspects of the information used as a basis of our land survey work, as well as the results of field measurements and data analysis. All work was performed at my direction and under my supervision.

Project units are international feet.

The project coordinate system is based on Arizona State Plane Central Zone 0202 (NAD '83- 2007) and was established by scaling AZSPC 0202 by a "grid-to-ground" factor of 1.00017 about the Cartesian origin (i.e., X = 0.00, Y = 0.00). Said scale factor was calculated using Trimble Geomatics Office Version 1.63 Build 10 for a point near the center of the project; for simplicity's sake, the scale factor was rounded to the nearest one-hundred thousandth.

The project vertical datum is NAVD '88. No control, benchmarks, or ERMs in the project area with published NGVD '29 elevations were uncovered. A conversion to NGVD '29 was calculated using the National Geodetic Survey VERTCON software. This computed value was found to be -1.95 feet (NGVD '29 elevations are lower than NAVD '88 elevations by approximately 1.95 feet).

The primary geodetic control point used as a basis of the horizontal and vertical datums for the project was NGS Control Station A365 (PID DU0654). See attached data sheet for positional information, monument description, horizontal and vertical order, etc. Real-time kinematic GPS observations were made to the northwest and southwest corners and the east and west quarter-corners of Section 33, Township 3 North, Range 7 East. Measurements consisted of a minimum of two 90-second RTK observations with separate initializations. Field horizontal measurements were in substantial agreement with GDACS-published values; no delta exceeded 0.08', and GDACS-published values were adopted for the purposes of this survey. Vertical measurements yielded varying results: the northwest corner was found to be approximately four tenths higher than as published in GDACS, and the southwest corner was found to be approximately two tenths lower. The east and west quarter-corners matched

substantially (within two hundredths of a foot). Measured elevations were adopted for the purposes of this survey, as appropriate. A control map is attached.

All topographic survey measurements were made using RTK GPS observations of at least 5 seconds. Elevations were modeled using Geoid '03. The level of precision inherent in this methodology was deemed appropriate for the project, as the project area has significant relief; typical roadway profiles and slopes of other drainage features such as washes varied from as much as 1.5% to 4% or more.

No conventional terrestrial survey methods were used on the project. Therefore, no field survey notes were recorded. Electronic notes, data, and other relevant information are included as printed attachments herein and provided on CD-ROM as specified in Section 3.2.8.

Sincerely,

Premier Engineering Corporation

Jesse Boyd, RLS, EIT
Assistant Department Manager – Survey

6437 W. Chandler Blvd., Suite 1 • Chandler, AZ 85226 • (480) 829-6000 • Fax (480) 829-6016

Coordinate System Details

Project : 09034_TGO_012910

User name	rmunoz	**Date & Time**	9:27:13 AM 4/23/2010
Coordinate System	US State Plane 1983 (at ground)	**Zone**	Arizona Central 0202
Project Datum	NAD 1983 (Conus)		
Vertical Datum	NAVD '88	**Geoid Model**	AZ 03
Coordinate Units	International feet		
Distance Units	International feet		
Height Units	International feet		

Coordinate System
Coordinate System : US State Plane 1983(at ground)
Zone : Arizona Central 0202
Datum : NAD 1983 (Conus)
Ellipsoid Name : Geodetic Ref System 1980
Geoid Model : AZ 03
Site : Not selected

Ellipsoid
Ellipsoid Name : Geodetic Ref System 1980
Flattening 1/f : 298.257
Semi Major Axis : 20925646.32546ift

Datum Transformation : Three Parameter
WGS84 to Geodetic Ref System 1980
Translation X : 0.00000ift Rotation X : N/A
Translation Y : 0.00000ift Rotation Y : N/A
Translation Z : 0.00000ift Rotation Z : N/A
Scale Factor : N/A ppm

Transverse Mercator Projection
Projection Origin False Origin
Latitude : 31.000000000°N False Northing : 0.00000ift
Longitude : 111.916666667°W False Easting : 700000.00000ift
Height : N/A False Elevation : N/A
Scale Factor : 0.99990000

Shift grid name : None
Azimuth at projection centre : N/A
Azimuth at equator : N/A
Projection Parallel 1 : N/A
Projection Parallel 2 : N/A
Projection Ferro Constant : N/A
Projection Point 1 Latitude : N/A
Projection Point 1 Longitude : N/A
Projection Point 2 Latitude : N/A

Projection Point 2 Longitude	:	N/A
Projection grid name	:	N/A

Local site settings

Project latitude	:	30.980683900°N
Project longitude	:	114.150208423°W
Project height	:	2000.00000ift
Ground scale factor	:	1.0001700000
False northing offset	:	0.00000ift
False easting offset	:	0.00000ift

GPS Site Calibration Details

Horizontal Adjustment

North Origin	:	N/A	Translation North	:	N/A
East Origin	:	N/A	Translation East	:	N/A
Scale	:	N/A	Rotation	:	N/A

Vertical Adjustment

North Origin	:	N/A
East Origin	:	N/A
Vertical constant correction	:	N/A
Slope North	:	N/A
Slope East	:	N/A

Network Adjustment Parameters

Longitude Deflection	:	N/A
Latitude Deflection	:	N/A
Azimuth Rotation	:	N/A
Network Scale	:	N/A
Distance Scale	:	N/A
Distance Constant	:	N/A
Height Constant	:	N/A

Back to top

Questions concerning the VERTCON process may be mailed to NGS
Latitude: 33.46615

Longitude: 111.64987

NAVD 88 height: 1754.395f

Datum shift(NAVD 88 minus NGVD 29): 1.952 feet

Converted to NGVD 29 height: 1752.443 feet

The NGS Data Sheet

See file dsdata.txt for more information about the datasheet.

```
DATABASE =  ,PROGRAM = datasheet, VERSION = 7.82
1        National Geodetic Survey,   Retrieval Date = APRIL 23, 2010
 DU0654 ***********************************************************************
 DU0654  FBN          -  This is a Federal Base Network Control Station.
 DU0654  DESIGNATION -  A 365
 DU0654  PID          -  DU0654
 DU0654  STATE/COUNTY-  AZ/MARICOPA
 DU0654  USGS QUAD    -  BUCKHORN (1982)
 DU0654
 DU0654                          *CURRENT SURVEY CONTROL
 DU0654
 DU0654* NAD 83(2007)-  33 26 55.04243(N)     111 40 27.33950(W)      ADJUSTED
 DU0654* NAVD 88      -       473.781  (meters)     1554.40   (feet)  ADJUSTED
 DU0654
 DU0654  EPOCH DATE   -         2007.00
 DU0654  X            -  -1,967,632.751 (meters)                      COMP
 DU0654  Y            -  -4,950,915.610 (meters)                      COMP
 DU0654  Z            -   3,495,825.975 (meters)                      COMP
 DU0654  LAPLACE CORR-            2.98  (seconds)                     DEFLEC09
 DU0654  ELLIP HEIGHT-          444.311 (meters)          (02/10/07)  ADJUSTED
 DU0654  GEOID HEIGHT-          -29.47  (meters)                      GEOID09
 DU0654  DYNAMIC HT   -         473.215 (meters)     1552.54  (feet)  COMP
 DU0654
 DU0654  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
 DU0654  Type    PID    Designation                        North   East  Ellip
 DU0654  ----------------------------------------------------------------------
 DU0654  NETWORK DU0654 A 365                               0.20   0.20   0.51
 DU0654  ----------------------------------------------------------------------
 DU0654  MODELED GRAV-      979,428.4    (mgal)                       NAVD 88
 DU0654  OBS GRAVITY -      979,430.7    (mgal)                       GRAV_OBS
 DU0654
 DU0654  VERT ORDER   -  FIRST     CLASS I
 DU0654
 DU0654.The horizontal coordinates were established by GPS observations
 DU0654.and adjusted by the National Geodetic Survey in February 2007.
 DU0654
 DU0654.The datum tag of NAD 83(2007) is equivalent to NAD 83(NSRS2007).
 DU0654.See National Readjustment for more information.
 DU0654.The horizontal coordinates are valid at the epoch date displayed above.
 DU0654.The epoch date for horizontal control is a decimal equivalence
 DU0654.of Year/Month/Day.
 DU0654
 DU0654.The orthometric height was determined by differential leveling and
 DU0654.adjusted in June 1991.
 DU0654
 DU0654.Photographs are available for this station.
 DU0654
 DU0654.The X, Y, and Z were computed from the position and the ellipsoidal ht.
 DU0654
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DU0654.The Laplace correction was computed from DEFLEC09 derived deflections.
DU0654
DU0654.The ellipsoidal height was determined by GPS observations
DU0654.and is referenced to NAD 83.
DU0654
DU0654.The geoid height was determined by GEOID09.
DU0654
DU0654.The dynamic height is computed by dividing the NAVD 88
DU0654.geopotential number by the normal gravity value computed on the
DU0654.Geodetic Reference System of 1980 (GRS 80) ellipsoid at 45
DU0654.degrees latitude (g = 980.6199 gals.).
DU0654
DU0654.The modeled gravity was interpolated from observed gravity values.
DU0654.The observed gravity was obtained from relative gravimeter ties
DU0654.to the IGSN71 gravity network.
DU0654
DU0654;                      North         East      Units Scale Factor Converg.
DU0654;SPC AZ C      -   271,528.917   235,896.024   MT  0.99990626   +0 08 01.0
DU0654;SPC AZ C      -   890,842.90    773,937.09    iFT  0.99990626   +0 08 01.0
DU0654;UTM  12       - 3,701,226.362   437,333.472   MT  0.99964841   -0 22 18.0
DU0654
DU0654!              -  Elev Factor  x  Scale Factor =   Combined Factor
DU0654!SPC AZ C      -   0.99993025  x   0.99990626  =   0.99983652
DU0654!UTM  12       -   0.99993025  x   0.99964841  =   0.99957869
DU0654
DU0654                            SUPERSEDED SURVEY CONTROL
DU0654
DU0654  ELLIP H (09/30/99)  444.345  (m)                          GP(      ) 3 1
DU0654  NAD 83(1986)-  33 26 55.03352(N)    111 40 27.34301(W) AD(      ) B
DU0654  NAD 83(1992)-  33 26 55.04154(N)    111 40 27.33851(W) AD(      ) A
DU0654  ELLIP H (09/04/92)  444.402  (m)                          GP(      ) 3 1
DU0654  NAD 83(1986)-  33 26 55.03352(N)    111 40 27.34301(W) AD(      ) 1
DU0654  NAVD 88 (08/15/94)  473.78   (m)        1554.4     (f) LEVELING    3
DU0654  NAVD 88 (09/04/92)  473.78   (m)        1554.4     (f) LEVELING    3
DU0654  NGVD 29 (??/??/92)  473.195  (m)        1552.47    (f) ADJ UNCH    1 1
DU0654
DU0654.Superseded values are not recommended for survey control.
DU0654.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
DU0654.See file dsdata.txt to determine how the superseded data were derived.
DU0654
DU0654_U.S. NATIONAL GRID SPATIAL ADDRESS: 12SVC3733301226(NAD 83)
DU0654_MARKER: DB = BENCH MARK DISK
DU0654_SETTING: 66 = SET IN ROCK OUTCROP
DU0654_SP_SET: IN DRILL HOLE IN ROCK OUTCROP
DU0654_STAMPING: A 365 1967
DU0654_MARK LOGO: CGS
DU0654_MAGNETIC: O = OTHER; SEE DESCRIPTION
DU0654_STABILITY: A = MOST RELIABLE AND EXPECTED TO HOLD
DU0654+STABILITY: POSITION/ELEVATION WELL
DU0654_SATELLITE: THE SITE LOCATION WAS REPORTED AS SUITABLE FOR
DU0654+SATELLITE: SATELLITE OBSERVATIONS - February 09, 2010
DU0654
DU0654  HISTORY     - Date     Condition        Report By
DU0654  HISTORY     - 1967     MONUMENTED       CGS
DU0654  HISTORY     - 1975     GOOD             NGS
DU0654  HISTORY     - 1981     GOOD             NGS
DU0654  HISTORY     - 19840323 GOOD
DU0654  HISTORY     - 1986     GOOD             AZDT
DU0654  HISTORY     - 1987     GOOD             NGS
DU0654  HISTORY     - 19920120 GOOD             NGS
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DU0654  HISTORY      - 19920204 GOOD             NGS
DU0654  HISTORY      - 19940203 GOOD             NGS
DU0654  HISTORY      - 19940203 GOOD             NOS
DU0654  HISTORY      - 19940216 GOOD             NOS
DU0654  HISTORY      - 19960304 GOOD             CHANCE
DU0654  HISTORY      - 19980916 POOR             AZ-013
DU0654  HISTORY      - 19981102 GOOD             NGS
DU0654  HISTORY      - 20000329 GOOD             MCDOT
DU0654  HISTORY      - 20060927 GOOD             USPSQD
DU0654  HISTORY      - 20090602 GOOD             ENTEL
DU0654  HISTORY      - 20100209 GOOD             CAP
DU0654
DU0654                            STATION DESCRIPTION
DU0654
DU0654'DESCRIBED BY COAST AND GEODETIC SURVEY 1967
DU0654'4.3 MI NE FROM BUCKHORN.
DU0654'ABOUT 1.0 MILE EAST ALONG U.S. HIGHWAYS 60, 70, 80 AND 89 FROM THE
DU0654'POST OFFICE AT BUCKHORN, THENCE 2.5 MILES NORTH ALONG BUSH HIGHWAY,
DU0654'THENCE 0.8 MILE SOUTHEAST ALONG A TRAIL ROAD, IN S 7, T 1 N, R 7 E, AT
DU0654'THE FOOT OF THE WEST SLOPE OF THE HIGHEST HILL IN THE VICINITY, 53
DU0654'FEET SOUTHEAST OF THE CENTER LINE OF THE TRAIL ROAD AROUND THE HILL,
DU0654'SET IN THE TOP OF A ROCK OUTCROP, 0.7 FOOT NORTH OF A METAL WITNESS
DU0654'POST, AND ABOUT 5 FEET ABOVE THE LEVEL OF THE TRAIL ROAD.
DU0654
DU0654                            STATION RECOVERY (1975)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1975
DU0654'ABOUT 8.7 MILES EAST ALONG U.S. HIGHWAY 60, 70, 80 AND 89 FROM
DU0654'DOWNTOWN MESA TO BUSH HIGHWAY, THENCE 2.5 MILES NORTH ALONG BUSH
DU0654'HIGHWAY, THENCE 0.8 MILE SOUTHEAST ALONG A TRAIL ROAD TO THE FOOT OF
DU0654'THE WEST SLOPE OF THE HIGHEST HILL IN THE VICINITY, 53 FEET SOUTHEAST
DU0654'OF THE CENTERLINE OF THE TRAIL ROAD AROUND THE HILL, SET IN THE TOP OF
DU0654'A ROCK OUTCROP, 0.7 FOOT NORTH OF A METAL WITNESS POST, AND ABOUT 5
DU0654'FEET ABOVE THE LEVEL OF THE TRAIL ROAD.  1/4, SEC 7, T 1N, R 7E.
DU0654
DU0654                            STATION RECOVERY (1981)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1981
DU0654'RECOVERED IN GOOD CONDITION.
DU0654
DU0654                            STATION RECOVERY (1984)
DU0654
DU0654'RECOVERED 1984
DU0654'RECOVERED IN GOOD CONDITION.
DU0654
DU0654                            STATION RECOVERY (1986)
DU0654
DU0654'RECOVERY NOTE BY ARIZONA DEPARTMENT OF TRANSPORTATION 1986 (JRT)
DU0654'THE STATION WAS RECOVERED AT THIS DATE.
DU0654'THE STATION WAS FOUND IN GOOD CONDITION.  PREVIOUS TO REACH
DU0654'DESCRIPTION WAS ADEQUATE.
DU0654
DU0654                            STATION RECOVERY (1987)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1987 (MRM)
DU0654'THE STATION WAS RECOVERED AT THIS DATE.
DU0654'
DU0654'THE STATION IS LOCATED ABOUT 7.2 KM (4.5 MI)
DU0654'EAST-SOUTHEAST OF SIGNAL BUTTE,
DU0654'7.2 KM (4.5 MI) NORTH OF SR 360 (SUPERSTITION FREEWAY) AND AT THE
```

DU0654'BOTTOM OF THE WEST END OF A SMALL HILL.
DU0654'OWNERSHIP--UNKNOWN.
DU0654'
DU0654'TO REACH THE STATION FROM THE JUNCTION OF STATE ROUTE 360
DU0654'(SUPERSTITION FREEWAY) AND POWERS ROAD IN MESA, GO
DU0654'NORTH FOR 3.2 KM (2.0 MI) ON POWERS ROAD TO APACHE TRAIL.
DU0654'CONTINUE STRAIGHT AHEAD AND GO NORTH FOR 4.0 KM (2.5 MI) ON BUSH
DU0654'HIGHWAY TO MCKELLIPS ROAD.
DU0654'TURN RIGHT AND GO EAST FOR 0.8 KM (0.5 MI) ON MCKELLIPS ROAD TO A
DU0654'DIRT ROAD ON THE RIGHT, AT THE END OF A BLOCK WALL.
DU0654'TURN RIGHT AND GO SOUTHEAST FOR 0.3 KM (0.2 MI) ON THE DIRT ROAD TO
DU0654'A DIRT ROAD LEFT.
DU0654'CONTINUE STRAIGHT AHEAD AND GO SOUTHEAST FOR
DU0654'0.1 KM (0.05 MI) ON THE DIRT ROAD TO THE WEST BASE OF A SMALL HILL
DU0654'AND THE STATION, BETWEEN TWO LARGE PALOVERDE TREES.
DU0654'
DU0654'THE STATION IS A STANDARD CGS BENCH MARK DISK
DU0654'STAMPED---A 365 1965---,
DU0654'SET INTO A DRILL HOLE IN THE TOP OF ROCK OUTCROP OF 0.5 METERS
DU0654'LARGEST DIMENSION. LOCATED
DU0654'15.2 METERS (50.0 FT) SOUTHEAST FROM THE APPROXIMATE CENTER OF THE
DU0654'TRAIL ROAD AROUND THE HILL,
DU0654'9.7 METERS (31.8 FT) NORTHEAST FROM A PALOVERDE TREE AND
DU0654'8.3 METERS (27.2 FT) SOUTH FROM A PALOVERDE TREE.
DU0654'
DU0654'CALIFORNIA FAA AIRPORTS, 1987.
DU0654'
DU0654'THIS STATION SUITABLE FOR GPS SURVEYS.
DU0654'
DU0654'DESCRIBED BY S.E. RANDALL.
DU0654
DU0654 STATION RECOVERY (1992)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1992
DU0654'14.5 KM (9.00 MI) EASTERLY ALONG U.S. HIGHWAY 60 (MAIN STREET) FROM
DU0654'THE JUNCTION OF STATE HIGHWAY 87 (COUNTRY CLUB DRIVE) IN MESA, THENCE
DU0654'4.0 KM (2.50 MI) NORTHERLY ALONG NORTH POWERS ROAD, THENCE 0.9 KM
DU0654'(0.55 MI) EASTERLY ALONG EAST MCKELLIPS ROAD, THENCE 0.3 KM (0.20 MI)
DU0654'SOUTHERLY ACROSS COUNTRY, NEAR THE CENTER OF A 1 BY 2-FOOT AREA OF
DU0654'OUTCROPPING BEDROCK ALONG THE WEST SLOPE OF A HILL, 5.0 M (16.4 FT)
DU0654'EAST OF THE BASE OF THE HILL, AND 1.0 M (3.3 FT) SOUTH OF A WITNESS
DU0654'POST.
DU0654
DU0654 STATION RECOVERY (1992)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1992
DU0654'THE STATION IS LOCATED ABOUT 12 KM (7.5 MI) NORTHEAST OF MESA, 4 KM
DU0654'(2.5 MI) EAST-SOUTHEAST OF FALCON FIELD, 0.3 KM (0.2 MI) SOUTH OF
DU0654'MCKELLIPS ROAD, JUST NORTHWEST OF THE FALCON HILL ELEMENTARY SCHOOL,
DU0654'ON THE WEST END OF A LOW RIDGE AND AT THE TOE OF THE HILL.
DU0654'OWNERSHIP--UNKNOWN.
DU0654'TO REACH THE STATION FROM THE JUNCTION OF STATE HIGHWAY 360 AND POWER
DU0654'ROAD (EXIT 16) ABOUT 14.4 KM (8.9 MI) EAST OF THE JUNCTION OF STATE
DU0654'HIGHWAYS 87 AND 360 IN MESA, GO NORTH ON POWER ROAD FOR 3.29 KM
DU0654'(2.04 MI) TO ITS JUNCTION WITH U.S. HIGHWAY 60 (MAIN STREET).
DU0654'CONTINUE NORTH ON POWER ROAD FOR 3.98 KM (2.47 MI) TO A CROSSROAD.
DU0654'TURN RIGHT, EAST ON MCKELLIPS ROAD FOR 0.98 KM (0.61 MI) TO A DIM
DU0654'ROAD RIGHT, JUST BEFORE REACHING A CANAL CROSSING. TURN RIGHT,
DU0654'SOUTH, CROSSING A SLIGHT HUMP, THEN TURN RIGHT, SOUTHWEST ON A DIRT
DU0654'ROAD ACROSS AN OPEN AREA FOR 0.28 KM (0.17 MI) TO TREES ON THE LEFT.

DU0654'TURN LEFT ACROSS THE FIELD FOR 0.08 KM (0.05 MI) TO A ROW OF BOULDERS
DU0654'BETWEEN TWO TREES AND THE STATION 15 M (49.2 FT) BEYOND.
DU0654'THE STATION IS SET IN A DRILL HOLE IN A 0.3 M (1.0 FT) X 0.5 M
DU0654'(1.6 FT) X 0.1 M (0.3 FT) HIGH OUTCROP BETWEEN TWO PALO VERDE TREES
DU0654'AND 1.0 M (3.3 FT) ABOVE THE SURROUNDING TERRAIN. LOCATED 55.0 M
DU0654'(180.4 FT) WEST AROUND BASE OF RIDGE FROM THE NORTHWEST FENCE CORNER
DU0654'OF SCHOOL PLAYGROUND, 10.0 M (32.8 FT) NORTHEAST OF A PARTIALLY DEAD
DU0654'PALO VERDE TREE AND 8.5 M (27.9 FT) SOUTH OF A PALO VERDE TREE.
DU0654
DU0654 STATION RECOVERY (1994)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1994 (JDR)
DU0654'RECOVERED AS DESCRIBED. NOTE-CONTINUE NORTH ON POWER ROAD PAST MAIN
DU0654'STREET FOR 2.3 MI (3.7 KM) TO JENSEN ROAD. TURN RIGHT HEADING EAST FOR
DU0654'0.55 MI (0.89 KM) TO A FIELD ON THE LEFT. THERE IS A DIRT ROAD LEADING
DU0654'TO A HILL OF ROCK OUTCROPS. STATION IS ABOUT 600 FT (182.9 M) FROM
DU0654'ROAD AND IS BETWEEN TWO TREES.
DU0654
DU0654 STATION RECOVERY (1994)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL OCEAN SERVICE 1994 (JDR)
DU0654'RECOVERED AS DESCRIBED. NOTE-CONTINUE NORTH ON POWER ROAD PAST MAIN
DU0654'STREET FOR 2.3 MI (3.7 KM) TO JENSEN ROAD. TURN RIGHT HEADING EAST FOR
DU0654'0.55 MI (0.89 KM) TO A FIELD ON THE LEFT. THERE IS A DIRT ROAD LEADING
DU0654'TO A HILL OF ROCK OUTCROPS. STATION IS ABOUT 600 FT (182.9 M) FROM
DU0654'ROAD AND IS BETWEEN TWO TREES.
DU0654
DU0654 STATION RECOVERY (1994)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL OCEAN SERVICE 1994 (RAH)
DU0654'THE STATION IS LOCATED ABOUT 7.5 MI (12.1 KM) NORTHEAST OF MESA, 2.5
DU0654'MI (4.0 KM) EAST SOUTHEAST OF FALCON FIELD AND 0.2 MI (0.3 KM) SOUTH
DU0654'OF MCKELLIPS ROAD, JUST NORTHWEST OF FALCON HILL ELEMENTARY SCHOOL ON
DU0654'THE WEST END OF A LOW RIDGE AND THE TOE OF A HILL. TO REACH THE
DU0654'STATION FROM THE JUNCTION OF U.S. HIGHWAY 60, STATE HIGHWAY 360 AND
DU0654'POWER ROAD, GO NORTH ON POWER ROAD FOR 4.3 MI (6.9 KM) TO JENSEN ROAD.
DU0654'TURN RIGHT ON JENSEN ROAD FOR 0.5 MI (0.8 KM) EAST TO A TRACK ROAD
DU0654'JUST PAST A BLOCK WALL AROUND A HOUSING DEVELOPMENT, AND RUNNING
DU0654'NORTHEAST FROM A LOW GRASSY AREA ON THE LEFT. FOLLOW TRACK ROAD
DU0654'NORTHEAST TO WEST TOE OF A HILL AND STATION ON THE RIGHT BETWEEN TWO
DU0654'PALO VERDE TREES. DESCRIBED AND RECOVERED BY RALPH A. HARRELL, CHIEF
DU0654'OF NOS PARTY.
DU0654
DU0654 STATION RECOVERY (1996)
DU0654
DU0654'RECOVERY NOTE BY JE CHANCE AND ASSOCIATES 1996 (KHB)
DU0654'RECOVERED AS DESCRIBED. UPDATES TO THE DESCRIPTION FOLLOW TO REACH
DU0654'THE STATION FROM THE JUNCTION OF US HIGHWAY 60 (MAIN STREET) AND STATE
DU0654'HIGHWAY 87 (COUNTRY CLUB DRIVE) IN MESA, GO EAST ON US HIGHWAY 60 FOR
DU0654'14.4 KM (8.95 MI) TO POWER ROAD (EXIT 189 ON US HIGHWAY 60) . PROCEED
DU0654'NORTH ON POWER ROAD FOR 4.0 KM (2.50 MI) PAST MAIN STREET TO JENSEN
DU0654'ROAD. TURN RIGHT ON AND GO EAST ON JENSEN ROAD FOR 0.88 KM (0.55 MI)
DU0654'TO A SMALL PARK ON THE NORTH SIDE OF THE ROAD AND THE STATION
DU0654'APPROXIMATELY 182.9 M (600.1 FT) FROM THE ROAD, ON THE SOUTH SIDE OF A
DU0654'ROCK HILL THE STATION IS LOCATED 26.1 M (85.6 FT) SOUTH-SOUTHWEST FROM
DU0654'A PARK SIGN
DU0654
DU0654 STATION RECOVERY (1998)
DU0654
DU0654'RECOVERY NOTE BY MARICOPA COUNTY ARIZONA 1998 (LOC)

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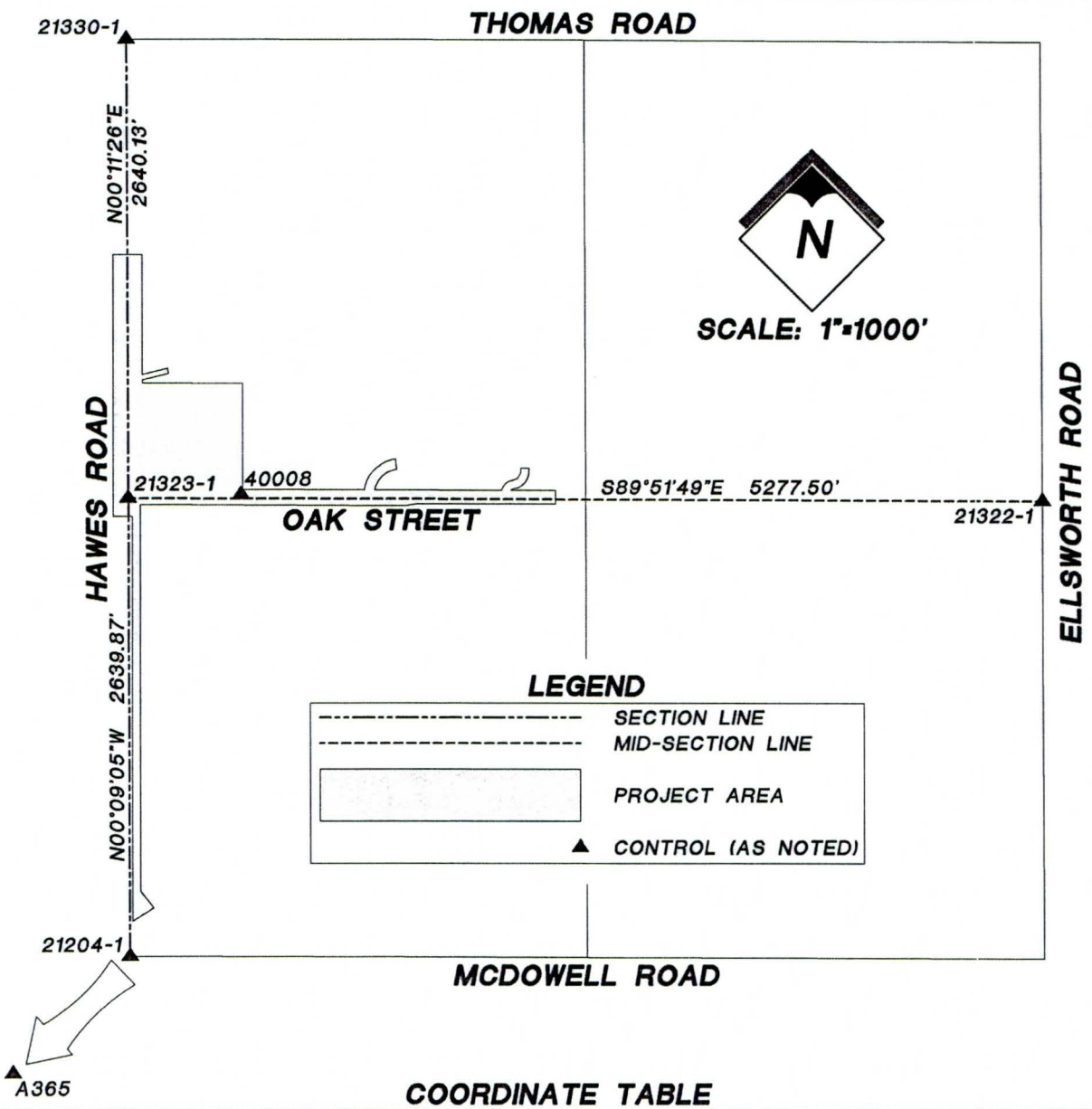
DU0654'RECOVERED AS DESCRIBED.
DU0654
DU0654                          STATION RECOVERY (1998)
DU0654
DU0654'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1998 (CSM)
DU0654'THE STATION IS LOCATED IN THE EAST SECTION OF MESA, AT THE CITY OF
DU0654'MESA, FALCON HILL PARK LOCATED ON JENSEN STREET, 0.96 KM (0.60 MI)
DU0654'EAST OF ITS JUNCTION WITH POWER ROAD, IN ROCK OUTCROP AT THE WEST BASE
DU0654'OF A HILL ON THE EAST SIDE OF THE PARK.  OWNERSHIP--CITY OF MESA.  TO
DU0654'REACH THE STATION FROM THE EXIT RAMP AT THE JUNCTION OF U.S. STATE
DU0654'HIGHWAY 60 (EXIT 188) AND POWER ROAD IN MESA, GO NORTH ON POWER ROAD
DU0654'FOR 6.96 KM (4.30 MI) TO THE JUNCTION OF JENSEN STREET ON THE RIGHT,
DU0654'TURN RIGHT, EAST ON JENSEN STREET AND GO 0.96 KM (0.60 MI) TO THE EAST
DU0654'SIDE OF FALCON HILL PARK ON THE LEFT AND A HOUSE NUMBERED 7231 ON THE
DU0654'RIGHT.  TURN LEFT, NORTH ON A NARROW TRAIL ROAD BETWEEN THE LANDSCAPED
DU0654'AREA AROUND THE PARK AND GO 0.08 KM (0.05 MI) TO THE STATION ON THE
DU0654'RIGHT.  THE STATION IS A US COAST AND GEODETIC SURVEY BENCH MARK DISK
DU0654'SET IN A DRILL HOLE IN A 0.55 M (1.80 FT) BY 0.40 M (1.31 FT) ROCK
DU0654'OUTCROP AT THE BASE OF THE HILL, DIRECTLY EAST AND ACROSS THE GRASSY
DU0654'AREA OF THE PARK FROM A VOLLEYBALL NET IN THE CENTER OF THE PARK.  IT
DU0654'IS 42.05 M (137.96 FT) NORTH-NORTHEAST OF LIGHT POLE NUMBER 24, 29.6 M
DU0654'(97.1 FT) SOUTH-SOUTHEAST OF LIGHT POLE NUMBER 29, 26.7 M (87.6 FT)
DU0654'SOUTH OF THE SOUTH LEG OF A PARK RULES AND REGULATION SIGN, 17.83 M
DU0654'(58.50 FT) EAST-SOUTHEAST OF LIGHT POLE NUMBER 28, 10.21 M (33.50 FT)
DU0654'EAST-SOUTHEAST OF THE EAST CONCRETE CURBING AROUND THE PARK AND 10.05
DU0654'M (32.97 FT) NORTHEAST OF A SMALL 6-FT TALL SAGUARO CACTUS.
DU0654
DU0654                          STATION RECOVERY (2000)
DU0654
DU0654'RECOVERY NOTE BY MARICOPA CO DOT 2000 (KRH)
DU0654'RECOVERED AS DESCRIBED.
DU0654
DU0654                          STATION RECOVERY (2006)
DU0654
DU0654'RECOVERY NOTE BY US POWER SQUADRON 2006 (RHC)
DU0654'RECOVERED IN GOOD CONDITION.
DU0654
DU0654                          STATION RECOVERY (2009)
DU0654
DU0654'RECOVERY NOTE BY ENTELLUS INC 2009 (WEW)
DU0654'RECOVERED IN GOOD CONDITION.
DU0654
DU0654                          STATION RECOVERY (2010)
DU0654
DU0654'RECOVERY NOTE BY CENTRAL ARIZONA PROJECT 2010 (JEM)
DU0654'FOUND IN GOOD CONDITION.

*** retrieval complete.
Elapsed Time = 00:00:00
```



COORDINATE TABLE

POINT ID	NORTHING	EASTING	ELEV	RECOVERED DESCRIPTION
21330-1	902671.9702	781476.7955	1858.75	3" CITY OF MESA BCHH
21323-1	900031.8574	781485.5810	1801.88	3" CITY OF MESA BC (0.2' BELOW GRADE)
21204-1	897391.9947	781492.5512	1754.21	2-1/2" GENERAL LAND OFFICE BCHH
21322-1	900019.2863	786763.0620	1967.55	2-1/2" GENERAL LAND OFFICE BC (0.9' ABOVE GRADE)
A365	890994.2513	774068.7403	1554.40	4" US COASTAL AND GEODETIC SURVEY BC
40008	900051.8051	782132.9573	1821.66	1/2" REBAR

•HORIZONTAL COORDINATES REPRESENTED HEREON ARE NAD 83 GROUND COORDINATES THAT HAVE BEEN SCALED ABOUT THE CARTESIAN ORIGIN BY 1.00017. ELEVATIONS SHOWN HEREON ARE GPS (RTK) DERIVED AND BASED ON THE NAVD 88 DATUM.

OAK STREET STORM DRAIN SURVEY REPORT

PREMIER
ENGINEERING CORPORATION
6437 W CHANDLER BLVD, STE 1
CHANDLER, AZ 85226

REGISTERED LAND SURVEYOR
CERTIFICATE NO.
42937
JESSE BOYD
Date Signed 8/30/12
ARIZONA, U.S.A.
Expires 06 / 30 / 2014

PROJECT No.	DRAWN BY	DATE
2009034	RSM	APR. 26, 2010
SHEET No.	CHECKED BY	REVISED
1 OF 1	JIB	AUG. 30, 2012

```
[Headers]
Title=09034_TGO_012910
Description=AZSPC0202_REVISED-061709_NGS-GDACS
Reference=
Field surveyor=
Computer operator=
Date=1/29/2010
Vertical datum=NAVD '88
Equipment=
[Units]
Height=1
Distance=1
Coordinate=1
Velocity=0
Area=2
Angle Format=0
Lat Long Format=2
East before North=0
Distance Type=2
Angle Units=0
Inch to Feet Scales=1
Null String=?
Lat Long Display=1
Distance Decimal Places=5
Coordinate Decimal Places=5
Area Decimal Places=5
Azimuth Types=1
Degrees Format=0
Grads and Mils Format=2
Azimuth Format=0
Coordinate Display=0
StatusBarDisplayKey=0
Vector Display=0
Terrestrial obs Display=0
WGS Display Type=0
Separate Thousands=0
Grade Format=0
Side Slope Grade Format=1
Grade Ratio Style=0
Stationing Format=1
Volume=1
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Terr Obs at Ground=0
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GEOTECHNICAL

Geotechnical Engineering Report
Oak Street Basin and Storm Drain Design
Oak Street and Hawes Road
Maricopa County, Arizona
FCD Contract No. 2009C021
RAMM Project No. G17544

For:
Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

◆◆◆

By:
Ricker-Atkinson-McBee-Morman & Associates, Inc.
2105 South Hardy Drive, Suite 13
Tempe, Arizona 85282

R·A·M·M

RICKER • ATKINSON • McBEE • MORMAN & ASSOCIATES, INC.

Geotechnical Engineering • Construction Materials Testing

Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

July 26, 2010

Attention: Mark T. Gavan, P.E.

Subject: Geotechnical Engineering Report
Oak Street Basin and Storm Drain Design
Oak Street and Hawes Road
Maricopa County, Arizona
FCD Contract No. 2009C021

RAMM Project No. G17544

Attached to this letter is the Geotechnical Engineering Report for the Oak Street Basin and Storm Drain Design, to be located in Maricopa County, Arizona.

The proposed construction consists of a detention basin approximately 10 to 24 feet in depth and approximately 2,600 lineal feet of storm drain varying in depth from approximately 8 to 15 feet. Portions of the storm drain will be located within the existing asphalt concrete paved Oak Street roadway. The results of our field explorations; laboratory testing; and engineering analysis, evaluation and recommendations are presented in the report. We provided preliminary geotechnical engineering recommendation in a letter dated March 10, 2010.

The attached report was prepared based on project and site data available at this time and was prepared in a manner and to the standards of local geotechnical engineering practice. Our services did not include evaluations for the presence of hazardous materials, area subsidence resulting from groundwater withdrawal or other geologic hazards.

If you have any questions, please do not hesitate to call.

Respectfully submitted,
RICKER, ATKINSON, McBEE, MORMAN & ASSOCIATES, INC.

Registered Professional Engineer (Civil) CERTIFICATE NO. 49948 KIP E. REESE Date Signed 7-26-2010 ARIZONA U.S.A.

Expires – 9/30/2012
By: Kip E. Reese, P.E.

Registered Professional Engineer (Civil) CERTIFICATE NO. 10971 KENNETH L. RICKER Date Signed 7-26-2010 ARIZONA U.S.A.

Expires – 3/31/2013
Reviewed By: Kenneth L. Ricker, P.E.

/dcw
Copies to: Addressee (5 mgavan@gavanbarker.com)

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REPORT

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INTRODUCTION

This report presents the results of our geotechnical engineering services for the Oak Street Basin and Storm Drain Design, to be located in Maricopa County, Arizona. The scope of our services included performing a field exploration program, laboratory analysis and geotechnical engineering evaluation, analysis and recommendations. The geotechnical recommendations presented herein include those for anticipated excavation conditions, site development, and material use and requirements. We provided preliminary geotechnical engineering recommendations in a letter dated March 10, 2010. We would be pleased to discuss with you any additional recommendations you may require. In addition, we are available to review project specifications and plans for conformance with our recommendations at no charge to you.

This firm should be notified for additional evaluation and recommendations if the project design parameters (location, type, size, pipeline alignment, etc.), site use or conditions encountered during construction differ from those presented herein.

PROPOSED CONSTRUCTION

The proposed construction consists of a detention basin approximately 10 to 24 feet in depth and approximately 2,600 lineal feet of storm drain varying in depth from approximately 8 to 15 feet. Portions of the storm drain will be located within the existing asphalt concrete paved Oak Street roadway.

SITE CONDITIONS

A visual reconnaissance of the proposed retention basin site was conducted by a field geologist for RAMM. The existing site surface soil and bedrock conditions were documented at the time of our reconnaissance. At the time of our reconnaissance and field exploration, the detention basin site was a gently rolling, undeveloped parcel which sloped gently downward to the southwest. Overall topographic relief was 26 feet. The site was crossed by two, relatively shallow, wide washes. The site surface conditions in the retention basin area consisted predominantly of granite-derived silty sand type soils. Scattered outcrops of decomposed to highly weathered granite bedrock were noted primarily in the eastern one-half of the site. The granite bedrock, where exposed, appeared to be massive with few, widely spaced joints apparent.

Vegetation on-site consisted of a slight to moderate coverage of native grasses, shrubs, cacti and trees.

Oak Street and Hawes Road along the storm drain alignment consisted of two lanes of asphalt concrete roadways and unpaved shoulders. Oak Street sloped gently downward to the west with an overall topographic relief of 70 feet. Hawes Road sloped gently downward to the south with an overall topographic relief of 30 feet.

FIELD EXPLORATIONS

Subsurface conditions in the proposed basin area were explored by drilling four test borings to depths of 14 to 22 feet, at locations accessible to our drill rig, and excavating four test pits to depths of 9 to 18 feet, as shown on the Site Plan in Appendix A1-1. Subsurface conditions along the proposed storm drain alignment were explored by drilling five test borings to depths of 14 to 19 feet, as shown on the Site Plans in Appendices' A1-2 and A1-3. Sediment transport analysis samples were obtained by hand excavation to a depth of 2 feet at four locations designated by the Maricopa County Flood Control District, as shown on the Site Plans in Appendices A1-2 and A1-3. A seismic refraction survey consisting of four seismic survey lines was performed in the proposed basin area by Geological Consultants, Inc. (GCI), the results of which are presented in Appendix C.

The test borings were drilled with a CME 45 drill rig using 7-inch diameter, hollow-stem and 4-inch diameter, single-flight augers. The drilling equipment and crew were provided by D&S Drilling, Inc. The test pits were excavated with a John Deere 310 rubber-tired extendahoe with a 1- or 2-foot wide bucket equipped with ripper teeth. The backhoe was provided by Sunstate Equipment and the operator was provided by D&S Drilling, Inc. The test boring and test pit locations were determined in the field by a field technician from our firm, who also directed the drilling. During the field explorations, representative disturbed and undisturbed samples were obtained, the test borings and test pits logged and soils and bedrock field classified by our field technician. The relatively undisturbed samples (ring samples) and driven disturbed samples (standard penetration tests (SPT)) were obtained by driving a 3-inch diameter, ring-lined, open-end sampler and a 2-inch diameter, split spoon sampler, respectively, into the soil with a 140-

pound hammer dropping 30 inches. The results of the field explorations are presented on the attached Test Boring and Test Pit Logs.

Existing pavement section measurements were performed at two locations along Oak Street, at locations determined by RAMM and marked in the field by a field technician with RAMM, as shown on the Site Plans in Appendix A1-2 and A1-3. The pavement was cored with a 4-inch diameter core barrel. The core was removed and measured. The asphalt concrete cores were retained and the core locations backfilled and patched with cold mix asphalt concrete.

LABORATORY ANALYSIS

Representative samples obtained during the field explorations were subjected to the following tests in our laboratory.

Type of Test	Type of Sample	Number of Samples Tested
Sieve Analysis, Percent Passing No. 200 Sieve & Atterberg Limits	Representative	8
Percent Passing No. 200 Sieve & Atterberg Limits	Representative	2
Swell Potential	Remolded	2
Maximum Density-Optimum Moisture	Representative	2
Moisture Content/Dry Density *	Undisturbed	7
Moisture Content *	Disturbed Ring/SPT	11
pH/Minimum Resistivity	Representative	6
Soluble Salts, Sulfate, Chloride **	Representative	6

* Reported in the test boring logs
** Testing performed By Motzz Laboratory

The results of the laboratory analysis are presented in Appendix B.

SURFACE AND SUBSURFACE CONDITIONS

General:

The results of the test borings and test pits are presented in Appendix A in the Test Boring and Test Pit Logs. Soil moisture contents were described as slightly damp. No groundwater was

observed in the test borings or test pits during the field exploration.

Proposed Basin:

The description of surface and subsurface conditions is based on our visual reconnaissance and geologic mapping of the site, the results of our field explorations and the results of the seismic refraction survey performed by GCI. The surface and near-surface soils consisted of silty sand containing some gravel colluvial, alluvial and residual type soils. These soils occur as a result of surface erosion, stream deposition and weathered-in-place bedrock (decomposed granite, Dg). These soils were encountered to depths of 1 to 10 feet in our test borings and test pits. These soils are loose to dense and have non-plastic to low plasticity fines.

The surface and near-surface bedrock consisted of granite bedrock. The granite bedrock outcrops are generally localized in areal extent and are primarily located within the south, east and north portions of the site. The granite bedrock was encountered near the surface in some of our test borings and test pits and extended to the full depths of our exploration (9 to 22 feet). The granite bedrock was moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth and capped with moderately to heavily cemented soils at isolated locations. Auger refusal occurred at a depth of 17.5 feet in Test Boring 2 and backhoe refusal occurred at depths of 9 to 18 feet in the test pits in this material. Limited surface expression indicated moderately to widely spaced, near vertical jointing in the granite bedrock. The joints had a general northwest to southeast and east-northeast to west-southwest trend.

Proposed Storm Drain:

Asphalt concrete pavement consisting of 2 inches of asphalt concrete on 6 inches of base material was encountered in Test Boring 8. In general, the near surface soils encountered in the test borings along the storm drain alignment and extending to depths of 5 to 12 feet consisted of silty sand containing some gravel. These soils were loose to medium dense and had non-plastic to low plasticity fines. Possible silty sand utility trench backfill was encountered to a depth of 5 feet in Test Boring 6. Underlying this deposit, and extending to the maximum depths of our exploration (14 to 19 feet), granite bedrock was encountered. The granite bedrock was

moderately soft to hard, hardness increasing with depth, and decomposed to slightly weathered, weathering decreasing with depth. Auger refusal occurred at depths of 14 to 19 feet in the storm drain alignment test borings in this material.

Pavement Section Measurements

The results of our existing pavement section measurements are presented below:

Core ID – Location*	Asphalt Concrete (inches)	Base Material (inches)
AC-1: WB	2.25	8.00
AC-2: EB	3.00	8.00

* Note – WB = westbound. EB = eastbound.

DISCUSSION OF TEST RESULTS

Remolded samples of the surface soils and excavated bedrock from the site exhibited no to low swell potential, following wetting, when tested in the laboratory.

PAVEMENT REPLACEMENT

The storm drain excavations will extend through the existing pavement along portions of the storm drain alignment. We anticipate that pavement disturbance will be limited to relatively narrow trench cuts and as such, pavement replacement, where required, should be in accordance with Section 601.6 of the Maricopa Association of Governments (MAG) specifications for Pavement Replacement and Surface Restoration, as supplemented by the Maricopa County Department of Transportation (MCDOT).

SITE DEVELOPMENT RECOMMENDATIONS

Permanent Cut Slopes - Basin:

The proposed basin will extend approximately 10 to 24 feet below existing site grades. Permanent cut slopes in the silty sand containing some gravel colluvial, alluvial and residual type soils and in the decomposed to highly weathered granite bedrock should stand at slopes no steeper than 1.5H:1V. Cut slopes lacking erosion control should be sloped no steeper than 4H:1V and will require increased maintenance over the life of the project. The effects of erosion

may be reduced somewhat if the cut slope is vegetated or faced with riprap and diversion ditches are constructed above the cut slope. Permanent cut slopes in the moderately weathered to unweathered granite bedrock should stand at near vertical slopes.

Construction Excavation and Temporary Construction Cut Slopes:

The proposed storm drain will extend approximately 8 to 15 feet below existing site grades. The following criteria are presented to aid in development of excavation plans.

1. Dewatering should not be required.

2. Unbraced temporary slopes in the silty sand native soils, silty sand backfill soils and decomposed to highly weathered granite bedrock should be constructed no steeper than 1.5H:1V. Unbraced temporary slopes in the moderately weathered to unweathered granite bedrock should be constructed no steeper than near vertical. As an alternative to shoring, trench boxes or other forms of slope stabilization may be used for this project.

3. Surface areas behind the crest of excavations should be graded so that surface waters do not pond within 10 feet of the crest or drain into the excavation.

4. Heavy material stockpiles should not be placed within 10 feet of the crest. Similarly, heavy construction equipment should not pass or be parked within 10 feet of the crest.

5. The sides and crest of excavations and slopes should be monitored daily for evidence of movement or potential problems.

The design of any bracing or shoring systems should be reviewed by a qualified geotechnical engineer. Also, observations should be made by the geotechnical engineer during excavating to evaluate site conditions and determine if modifications are necessary in excavation procedures. If unbraced slopes are utilized, some surface raveling, erosion, and spalling should be expected unless measures are taken to stabilize exposed cut surfaces.

Surface Drainage:

Most soils will undergo some degree of volume change as the result of wetting. The degree of volume change will depend on the type of soil, swell potential, natural soils structure or degree of compaction (if a fill). These volume changes could result in movements in overlying non-structure elements including sidewalks, planters, etc. Therefore, good site and surface drainage away from these elements is required. In addition, water should not be allowed to pond within 10 feet of elements which are sensitive to movements.

Excavatability:

The excavatability of site materials is difficult to evaluate based only on the exploration equipment used during this design report. Therefore, we recommend that the contractor evaluate the excavatability of site materials by performing test excavations with the size and type of equipment the contractor plans on using at the site. For design purposes the following paragraph presents our best analysis as to the excavatability of site soils.

The near surface soils and underlying granite bedrock to depths of 9 to 18 feet can probably be removed with conventional excavating equipment. Excavations will likely be slower and more difficult to accomplish with depth due to the increasing hardness of the granite bedrock and may require heavy excavating equipment, rock excavation techniques and/or blasting. Caving should be expected in the silty sand soils and in utility trench backfill soils. OSHA requires all excavations over five feet in depth, in which personnel are to enter, be either braced or sloped in accordance with OSHA regulations. Excavations in the decomposed to highly weathered granite bedrock will likely generate gravel-sized material and smaller. Excavations in the moderately weathered granite bedrock may generate gravel- and cobble- sized material depending on the excavation methods employed. Excavations in the slightly weathered to unweathered granite bedrock may generate cobble- and boulder-sized material depending on the excavation methods employed.

Workability:

Wetting site soils such that moisture contents are at or above optimum could result in some soil pumping under dynamic loadings such as heavy construction equipment driving over the area. In

areas where severe pumping has damaged subgrade conditions, the area should be allowed to dry until soils are workable without pumping or the wetted areas removed and replaced with drier site soils.

Corrosion Potential:

As part of this investigation, laboratory pH and Minimum Resistivity, Soluble Salts, Sulfates and Chloride testing of site soils and excavated bedrock was conducted. Based on these results, there appears to be a low to high potential for corrosion to buried metal structures and pipelines and a low to moderate potential for corrosive to concrete in contact with site soils and excavated bedrock. The results of the laboratory testing are included in Appendix B, and should be made available to material suppliers and corrosion experts for review.

MATERIALS SUITABILITY AND REQUIREMENTS

Site Soils and Excavated Granite Bedrock:

Along the proposed alignment and within the proposed basin, the site soils and excavated granite bedrock exhibit no to low plasticity. These soils and excavated bedrock may be used as fill in all areas outside of paved areas and unpaved roadways. All materials should be free of organics, debris, rubble and material greater than 4 inches in size.

Imported Soils:

Fill required beyond that available from site sources and for use as trench backfills outside of paved areas and unpaved roadways, should be imported soils meeting the following requirements:

Maximum Particle Size ---------- 4 inches
Maximum Swell Potential ---------- 1.5%*

* Based on a sample which is remolded to 95% of the ASTM D698 maximum dry density at a moisture content of 2 percent below optimum, placed under a surcharge load of 100 psf and wetted.

Controlled Low Strength Material:

Controlled low strength material (CLSM) used as trench backfill within paved areas and unpaved

roadways should conform to the requirements of Maricopa Association of Governments (MAG) Specifications for ½ sack CLSM.

Riprap:

Riprap used as erosion control material should conform to the requirements of MAG specifications for riprap (Section 703).

Base Material:

Base material used below pavement should conform to the requirements of MAG Specifications for Aggregate Base (Section 702). Existing base materials and milled asphalt pavement may be used as base material provided it meets the base material specification.

Asphalt Concrete Pavement:

Asphalt concrete pavement materials should conform to the requirement of MAG Specifications for Asphalt Concrete as supplemented by MCDOT.

Pipe Bedding:

Materials used as pipe bedding should be granular soils which meet the requirements of Section 601 of the MAG Specifications. Based on the results of the laboratory testing, the site soils and excavated granite bedrock will be suitable for use as bedding provided material greater than 1 ½ inches in size is removed. Excavated granite bedrock should be blended such that nesting does not occur.

SITE PREPARATION AND GRADING PROCEDURES

Basin and Trench Excavation and Pavement Replacement Areas:

Recommendations presented in the previous sections of this report are based upon the following site preparation and grading procedures. Therefore, all earthwork should be accomplished with observation and testing by a qualified technician under the direction of a registered geotechnical/materials engineer. The following apply to basin and trench excavation areas and pavement replacement areas.

1. Clear and grub the site by removing and disposing of all vegetation, any trash and debris, and any rubble and remnants of former developments. In trench excavation in paved areas remove all existing pavement materials.

2. Moisture condition and place all fill and backfill materials required to achieve specified grades. Fill materials should be moisture conditioned, placed and compacted in horizontal lifts of thicknesses compatible with the compaction equipment being used.

3. Compact subgrade, fill, backfill, subbase fill or base material to the requirements presented in MAG Section 601 as supplemented by MCDOT.

4. Place asphalt concrete in accordance with MAG Specifications Section 310 and 702, as applicable, using materials which comply with Section 321 of the MAG Specifications as supplemented by MCDOT.

APPENDIX A
FIELD EXPLORATIONS

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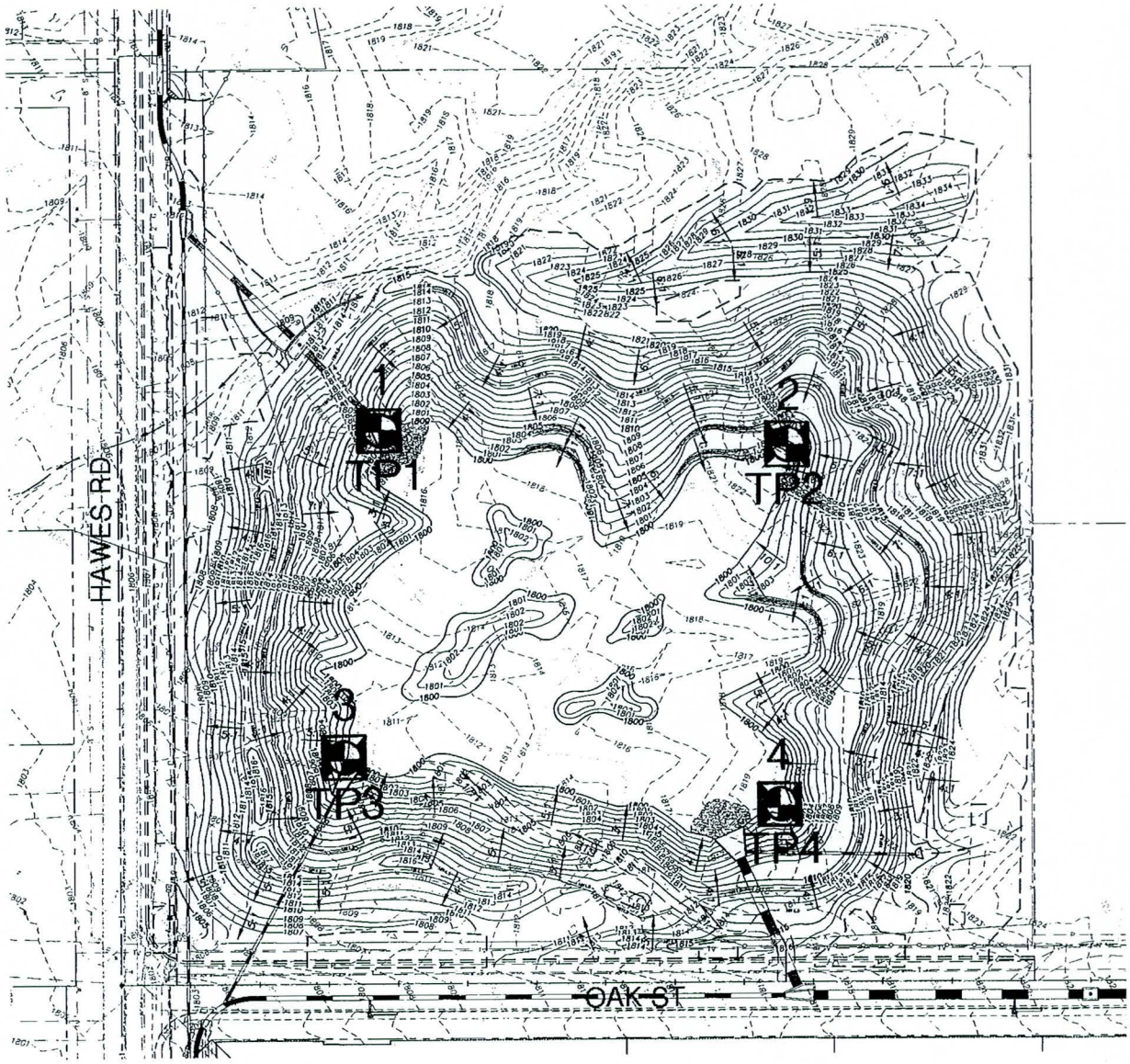

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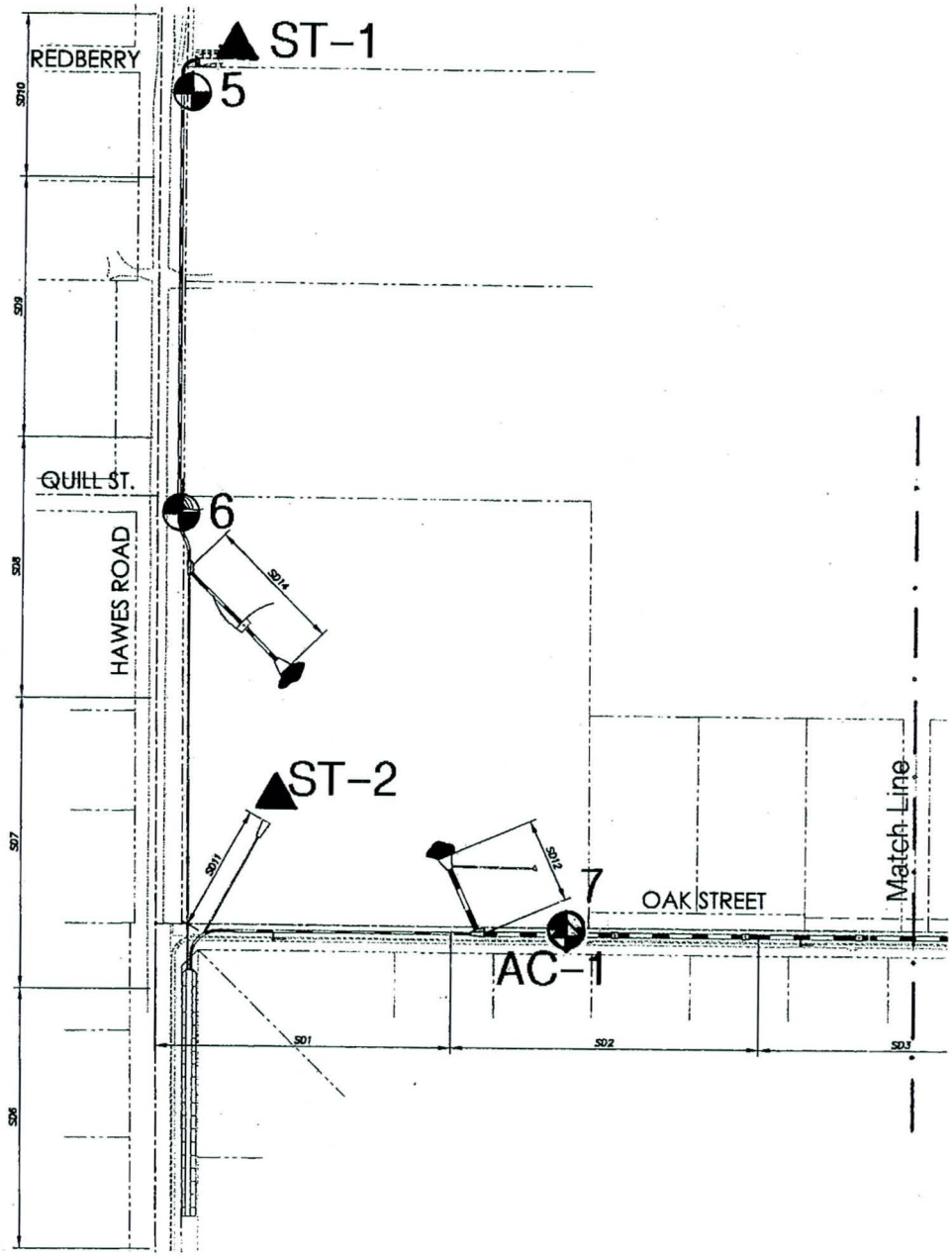
Note: See Site Plan A1–2 for Storm Drain Borings

Test Pit Location
Test Boring Location

Not To Scale

SITE PLAN



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Note: See Site Plan A1-1 for Basin Borings / Test Pits

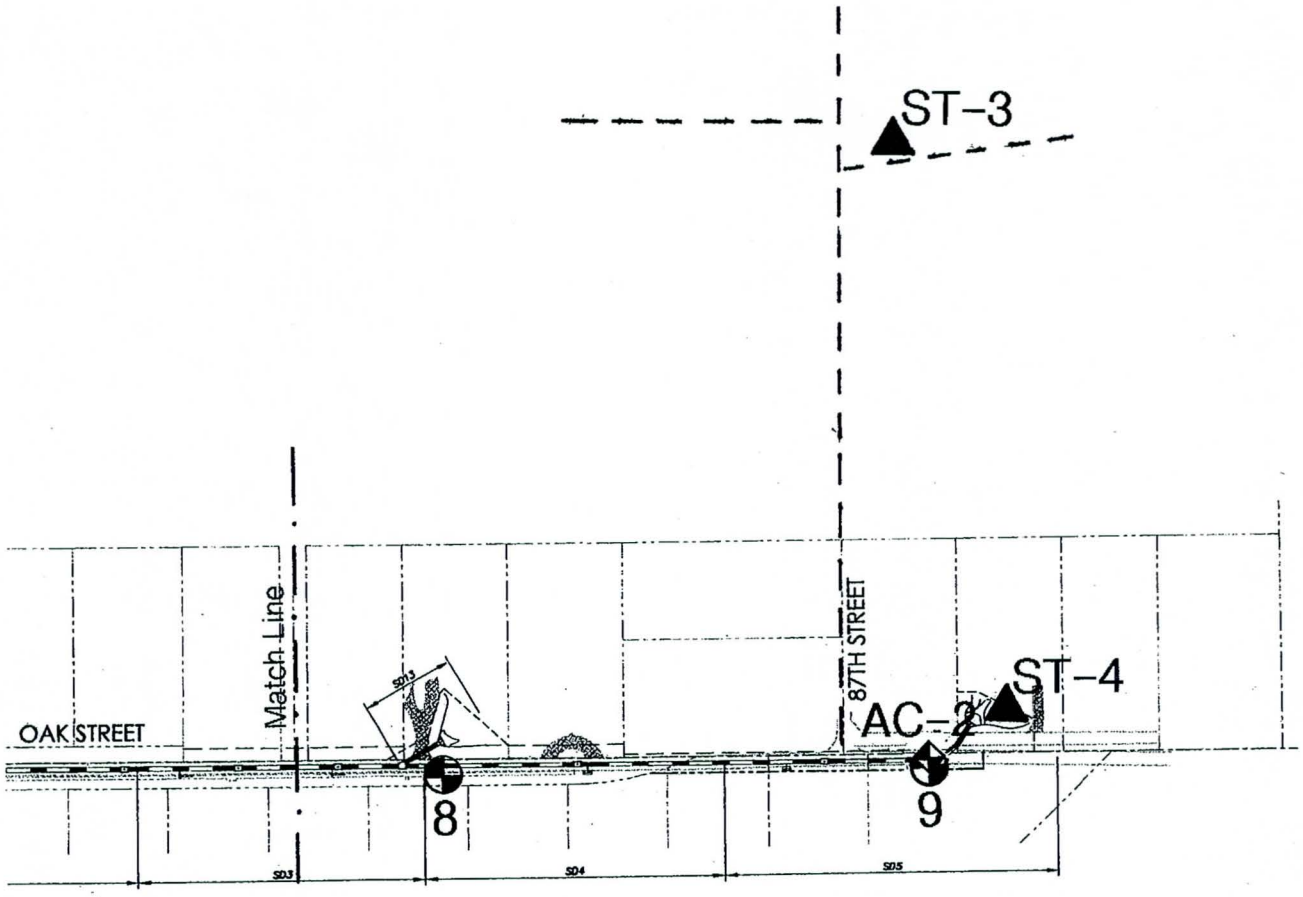
Sediment Transport Location

Pavement Section Location

Test Boring Location

Not To Scale

SITE PLAN



N

▲ Sediment Transport Location

◆ Pavement Section Location

◓ Test Boring Location

Not To Scale

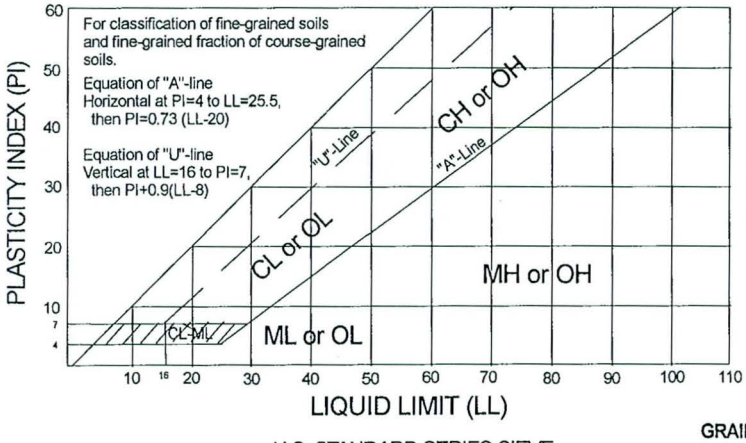
SITE PLAN

LEGEND

CLASSIFICATION OF SOILS

ASTM Designation: D2487-00
(Based on Unified Soil Classification System)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests					Soil Classification: Group Symbol	Soil Classification: Name
COARSE-GRAINED SOILS More than 50% retained on No. 200 Sieve	Gravels More than 50% coarse fraction retained on No. 4 Sieve	Clean Gravels Less than 5% fines	Cu > 4 and 1 < Cc < 3		GW	Well graded gravel
			Cu<4 and/or 1>Cc>3		GP	Poorly graded gravel
		Gravels with Fines More than 12% fines	Fines classify as ML or MH		GM	Silty gravel
			Fines classify as CL or CH		GC	Clayey gravel
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines	Cu > 6 and 1 < Cc < 3		SW	Well-graded sand
			Cu<6 and/or 1>Cc>3		SP	Poorly graded sand
		Sands with Fines More than 12% fines	Fines classify as ML or MH		SM	Silty sand
			Fines classify as CL or CH		SC	Clayey sand
FINE-GRAINED SOILS 50% or more passes the No. 200 Sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI>7 and plots on or above "A" line		CL	Lean clay
			PI<4 or plots below "A" line		ML	Silt
		Organic	Liquid Limit - oven dried / Liquid limit - not dried	<0.75	OL	Organic clay / Organic silt
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line		CH	Fat clay
			PI plots below "A" line		MH	Elastic silt
		Organic	Liquid limit - oven dried / Liquid limit - not dried	<0.75	OH	Organic clay / Organic silt
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor				PT	Peat



TEST BORING LOG DEFINITIONS

Blows per foot using 140 pound hammer with 30 inch free-fall.

Depth, feet	Blows/Foot: C	Blows/Foot: N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description

C = Continuous Penetration Resistance (2 inch diameter rod)
N = Standard Penetration Resistance (ASTM D1586)
R = Penetration Resistance (3 inch diameter ring line sampler)

GRAIN SIZES

U.S. STANDARD SERIES SIEVE: 200 | 40 | 10 | 4 | 3/4" | 3" | 12"
CLEAR SQUARE SIEVE OPENINGS

SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

MOISTURE CONDITION (INCREASING MOISTURE →)

DRY | SLIGHTLY DAMP | DAMP | MOIST (Plastic Limit) | VERY MOIST | WET (SATURATED) (Liquid Limit)

CONSISTENCY CORRELATION		RELATIVE DENSITY CORRELATION	
CLAYS & SILTS	BLOWS/FOOT*	SANDS & GRAVELS	BLOWS/FOOT*
VERY SOFT	0-2	VERY LOOSE	0-4
SOFT	2-4	LOOSE	4-10
FIRM	4-8	MEDIUM DENSE	10-30
STIFF	8-16	DENSE	30-50
VERY STIFF	16-32	VERY DENSE	OVER 50
HARD	OVER 32		

*Number of blows of 140 lb hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split-spoon sampler (ASTM D1586).

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 1

Elevation: Not Determined Datum: --- Date: 2-24-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
		12	R	NR			Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
5		48	R	NR			
10							
15							
20							Stopped drilling at 20 feet. No groundwater observed. NR=No Recovery
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 2

Elevation: Not Determined Datum: --- Date: 2-24-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
		50/8"	R	*	5		Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
5		50/4"	N		2		
10							
15							
							Auger refusal at 17.5 feet. No groundwater observed.
20							* Sample too disturbed to determine density.
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 3

Elevation: Not Determined Datum: --- Date: 2-24-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		5	R	104	6	SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
		10	R	110	2		
5		20	R	118	1		
10		50/8"	R	NR			Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
15							Stopped drilling at 14 feet. No groundwater observed. NR=No Recovery
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 4

Elevation: Not Determined Datum: --- Date: 2-24-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
		50/8"	R	NR			
5							
		50/5"	R	NR			Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
10							
		50/2"	N	NR			
15							
		50/2"	N	NR			
20							
							Stopped drilling at 22 feet. No groundwater observed. NR=No Recovery
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 5

Elevation: Not Determined Sta: 33+00, 45'RT Date: 6-4-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5		50/5"	R	105	3		
10		50/5"	R	109	2		Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
15							
20							Auger refusal at 19 feet. No groundwater observed.
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 6

Elevation: Not Determined Sta: 26+50, 20'RT Date: 6-4-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
		13	R	109	2		Note: Possible utility trench backfill.
5							
							Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
		35	R	*	3		
10							
15							
							Auger refusal at 17 feet. No groundwater observed.
20							
							* Sample too disturbed to determine density.
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 7

Elevation: Not Determined Sta: 16+40, 40'LT Date: 6-4-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5		50/9"	R	*	4		
10		50/5"	N		4		
							Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
15		50/1"	N		1		
							Auger refusal at 17 feet. No groundwater observed.
20							* Sample too disturbed to determine density.
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 8

Elevation: Not Determined Sta: 24+00, 28'RT Date: 6-4-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
							2" Asphalt Concrete on 6" Base Material
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5							5
		50/9"	R	*	NR		Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
		50/3"	N		1		
10							10
		50/1"	N		1		
15							15
		50/1"	N		1		
20							Auger refusal at 19 feet. 20 No groundwater observed. NR=No Recovery * Sample too disturbed to determine density.
25							25

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 9

Elevation: Not Determined Sta: 31+50, 15'RT Date: 6-4-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5		50/5"	R	106	3		
							Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
10		50/3"	N		2		
		50/4"	N		1		
15							Auger refusal at 14 feet. No groundwater observed.
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: TP 1

Elevation: Not Determined Datum: _____ Date: 6-3-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5							Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
10							
15							
20							Stopped excavation at 18 feet. No groundwater observed.
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: TP 2

Elevation: Not Determined Datum: Date: 6-3-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5							Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
10							Backhoe refusal at 10 feet. No groundwater observed.
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: TP 3

Elevation: Not Determined Datum: Date: 6-3-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5							
							Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
10							
							Stopped excavation at 10 feet. No groundwater observed.
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: TP 4

Elevation: Not Determined Datum: Date: 6-3-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
							Granite Bedrock, mottled brown/grey, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
5							
10							Backhoe refusal at 9 feet. No groundwater observed.
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: ST 1

Elevation: Not Determined Datum: --- Date: 7-22-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SW	Sand, Some Gravel; brown, slightly damp, loose, non-plastic fines.
							Stopped excavation at 2 feet. No groundwater observed.
5							
10							
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: ST2

Elevation: Not Determined Datum: --- Date: 7-22-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SW-SM	Sand, Some Gravel, Trace Silt; brown, slightly damp, loose, non-plastic fines.
							Stopped excavation at 2 feet. No groundwater observed.
5							
10							
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: ST3

Elevation: Not Determined Datum: --- Date: 7-22-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SW	Sand, Some Gravel; brown, slightly damp, loose, non-plastic fines.
							Stopped excavation at 2 feet. No groundwater observed.
5							
10							
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: ST4

Elevation: Not Determined Datum: --- Date: 7-22-10

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SW-SM	Sand, Some Gravel, Trace Silt; brown, slightly damp, loose, non-plastic fines.
							Stopped excavation at 2 feet. No groundwater observed.
5							
10							
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

APPENDIX B
LABORATORY ANALYSIS

R·A·M·M

LABORATORY TEST RESULTS

Date: 2-Jul-10

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Percent Passing No. 200 Sieve, Atterberg Limits, Percent Expansion (ASTM D1140, D4318, D4546)

SAMPLED BY: RAMM/Miller

RESULTS:

Sample Source	Percent Retained No. 4 Sieve	Percent Passing No. 200 Sieve	Liquid Limit	Plasticity Index	Percent Expansion*	Remolded Dry Density (pcf)	Remolded Moisture Content (%)
2 @ 0'-5'	21	17	24	5	0.7	126	5
3 @ 5'-10'	27	7.5	N/A	NP	0.0	130	4

* Based upon sample remolded to 95% of the estimated maximum dry density at 2% below the estimated optimum moisture content, with a surcharge pressure of 100 psf.

LABORATORY TEST RESULTS

Date: 26-Jul-10

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Sieve Analysis, Percent Passing No. 200 Sieve, Atterberg Limits (ASTM C136, D1140, D4318)

SAMPLED BY: RAMM/Schirmer

RESULTS:

Sample	Atterberg Limits		Sieve Size - Accumulative Percent Passing											Soil
Source	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.*
5 @ 9'-13'	21	2	11	14	18	25	38	61	86	100				SM
6 @ 0'-5'	27	5	11	15	18	24	33	50	74	100				SC-SM
7 @ 10'-15'	27	3	9.5	12	14	17	22	40	74	100				SP-SM
Composite:														
8 @ 0'-5' &														
8 @ 5'-10'	22	1	12	17	21	29	41	62	84	100				SM
9 @ 5'-10'	N/A	NP	8.7	12	16	23	36	57	83	100				SW-SM
TP1 @ 7'-16'	N/A	NP	4.6	8	12	20	30	44	64	100				SW
TP2 @ 3.5'-10'	29	5	1.0	1	2	3	6	13	29	76	82	93	100	GW-GC
TP4 @ 5'-9'	27	3	6.2	9	13	19	27	41	62	99	100			SW-SM
ST-1 @ 0'-2'		NP**	2.5	4	6	12	26	50	78	100				SW
ST-2 @ 0'-2'		NP**	5.0	8	14	22	33	48	68	100				SW-SM
ST-3 @ 0'-2'		NP**	2.5	4	7	15	30	55	79	100				SW
ST-4 @ 0'-2'		NP**	5.2	8	12	20	37	68	91	100				SW-SM

NP = Non-Plastic

* Unified Soil Classification System

** Non-plastic Visual

LABORATORY TEST RESULTS

Date: 2-Jul-10

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Percent Expansion (ASTM D4546)

SAMPLED BY: RAMM/Schirmer

RESULTS:

Sample Source	Percent Expansion*	Remolded Dry Density (pcf)	Remolded Moisture Content (%)
Composite: 6 @ 0'-2' & 6 @ 2'-5'	0.0	126	6
Composite: 8 @ 0'-5' & 8 @ 5'-10'	0.0	121	6

* Based upon sample remolded to 95% of the estimated maximum dry density at 2% below the estimated optimum moisture content, with a surcharge pressure of 100 psf.

LABORATORY TEST RESULTS

Date: 2-Jul-10

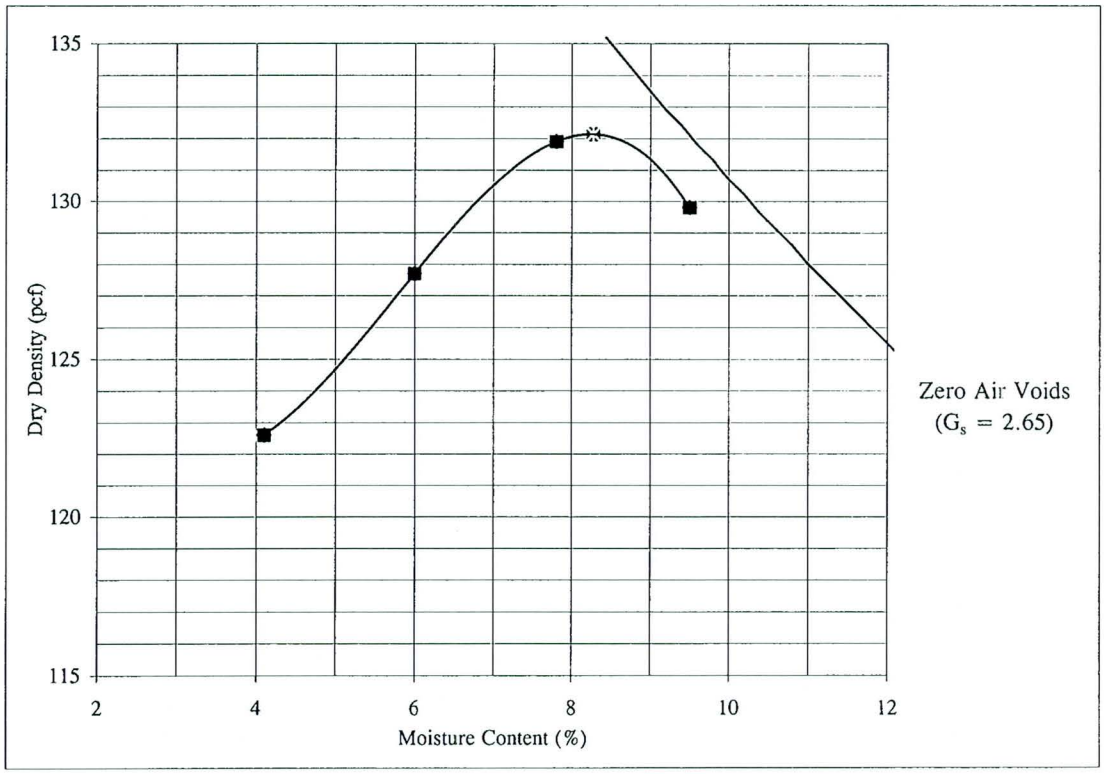
SAMPLE SOURCE: 6 @ 0'-5'

TESTING PERFORMED Maximum Density-Optimum Moisture Determination (ASTM D698 Method A)

SAMPLED BY: RAMM/Schirmer

RESULTS:

Maximum Density (pcf) = 132.1 Optimum Moisture (%) = 8.3



LABORATORY TEST RESULTS

Date: 2-Jul-10

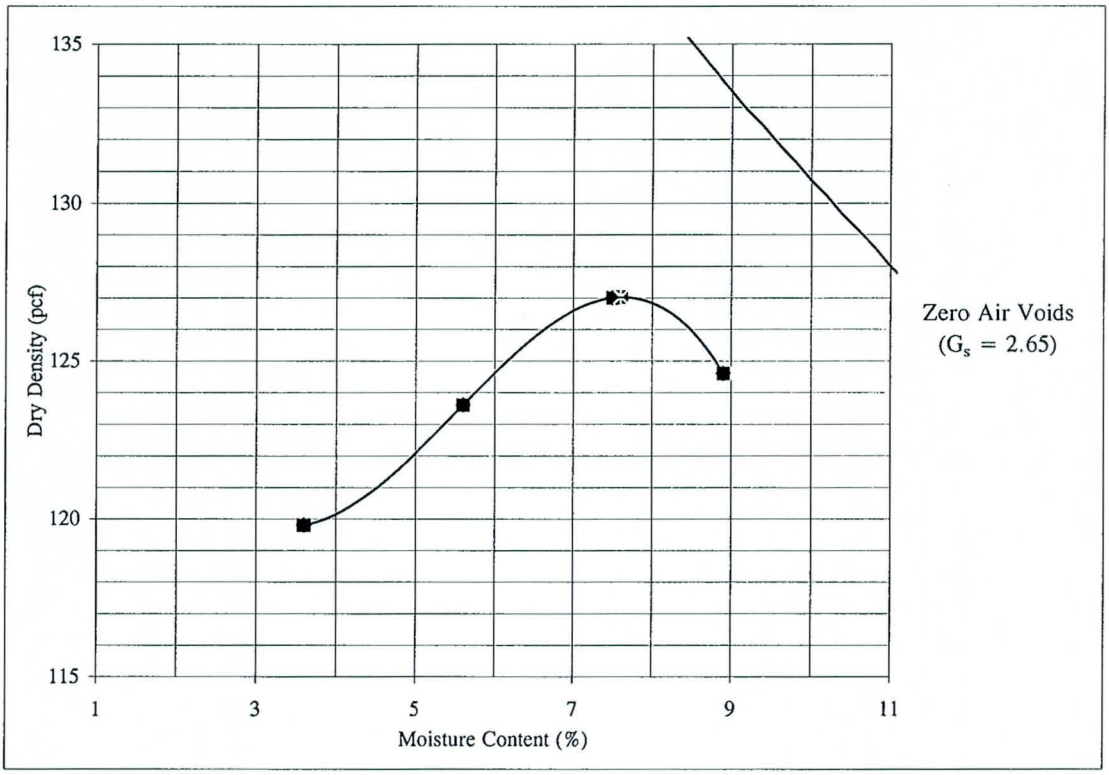
SAMPLE SOURCE: Composite: 8 @ 0'-5' & 8 @ 5'-10'

TESTING PERFORMED Maximum Density-Optimum Moisture Determination (ASTM D698 Method A)

SAMPLED BY: RAMM/Schirmer

RESULTS:

Maximum Density (pcf) = 127.0 Optimum Moisture (%) = 7.6



LABORATORY TEST RESULTS

Date: 2-Jul-10

SAMPLE SOURCE: As noted below

TESTING PERFORMED: pH, Minimum Resistivity (ADOT 236a)

SAMPLED BY: RAMM/Schirmer

RESULTS:

Sample Source	pH	Minimum Resistivity (ohm-cm)
5 @ 9'-13'	8.5	2096
6 @ 0'-5'	8.5	2028
7 @ 10'-15'	8.7	1758
8 @ 0'-5'	8.4	1960
9 @ 5'-10	8.2	5476
TP1 @ 7'-16'	7.8	541

MOTZZ LABORATORY, INC.
2113 S 48th St Suite 109
Tempe, AZ 85282

Soil Analysis Report

Ricker-Atkinson-McBee-Morman
Ken Ricker
2105 South Hardy Dr.#13
Tempe , AZ 85282-1921

Project: G17544
Sampler:
Date Received: 6/7/2010
Date Reported: 6/8/2010
PO Number: G17544

Lab Number: 901919-01 TP-1 7-16'

Soluble Salts, Sulfate & Chloride	Method	Result	Units	Levels
Soluble Salts	ARIZ 237b SS	0.18	%	
Sulfate, SO4	ARIZ 733	0.12	%	
Chloride	ARIZ 736	0.034	%	

Lab Number: 901919-02 5 9-13'

Soluble Salts, Sulfate & Chloride	Method	Result	Units	Levels
Soluble Salts	ARIZ 237b SS	0.028	%	
Sulfate, SO4	ARIZ 733	0.0060	%	
Chloride	ARIZ 736	0.0088	%	

Lab Number: 901919-03 6 0-5'

Soluble Salts, Sulfate & Chloride	Method	Result	Units	Levels
Soluble Salts	ARIZ 237b SS	0.026	%	
Sulfate, SO4	ARIZ 733	0.0048	%	
Chloride	ARIZ 736	0.0050	%	

Lab Number: 901919-04 7 10-15'

Soluble Salts, Sulfate & Chloride	Method	Result	Units	Levels
Soluble Salts	ARIZ 237b SS	0.058	%	
Sulfate, SO4	ARIZ 733	0.027	%	
Chloride	ARIZ 736	0.0033	%	

Lab Number: 901919-05 8 0-10'

Soluble Salts, Sulfate & Chloride	Method	Result	Units	Levels
Soluble Salts	ARIZ 237b SS	0.020	%	
Sulfate, SO4	ARIZ 733	0.0041	%	
Chloride	ARIZ 736	0.0026	%	

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Tempe, AZ 85282

Soil Analysis Report

Ricker-Atkinson-McBee-Morman
Ken Ricker
2105 South Hardy Dr.#13
Tempe , AZ 85282-1921

Project: G17544
Sampler:
Date Received: 6/7/2010
Date Reported: 6/8/2010

PO Number: G17544

Lab Number: 901919-06 **9 5-10'**

Soluble Salts, Sulfate & Chloride	Method	Result	Units	Levels
Soluble Salts	ARIZ 237b SS	0.022	%	
Sulfate, SO4	ARIZ 733	0.0015	%	
Chloride	ARIZ 736	0.0025	%	

APPENDIX C
SEISMIC REFRACTION SURVEY

R·A·M·M

GEOLOGICAL CONSULTANTS INC

KENNETH M. EUGE, R.G.

Report Prepared for:

Mr. Ken Ricker, P.E. & Mr. Kip E. Reese, P.E.
Ricker, Atkinson, McBee & Morman & Associates, Inc.
2105 S. Hardy Drive, Suite 13
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Report Prepared by:

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June 9, 2010
GCI Project No. 2010-125

Prepared by:
Kenneth M. Euge, R.G.

FINAL REPORT
SEISMIC REFRACTION SURVEY
OAK STREET BASIN

OAK STREET BASIN & STORM DRAIN DESIGN
OAK STREET & HAWES ROAD
MARICOPA COUNTY, ARIZONA

FCD CONTRACT No. 2009C021

RAMM Project No. G17544

NOTICE

The geologic and soils observations, findings, interpretations, conclusions and recommendations presented in this report area based on the results of the geophysical survey conducted at selected locations within the project site as defined in the Scope of Work agreed to by Ricker, Atkinson, McBee & Morman & Associate, Inc. (RAMM) and Geological Consultants Inc. (GCI) which included: (1) observation of the soil and rock materials exposed at the site, (2) seismic refraction surveys, and (3) geological interpretation, modeling, and analysis of the seismic survey data gathered. The services provided by GCI to RAMM were performed according to generally accepted geologic principles and practices used by members of the geologic profession in this locale at the time if this study.

It must be recognized that subsurface geologic conditions may vary from place to place and from those found at locations where measurements or surveys are made by the investigator. Generalized geological and excavateability recommendations presented in this report are based on the results of this investigation and it may not be possible for others to accurately correlate the geological and excavateability results to test explorations or investigations conducted by others. No warranty or representation, either expressed or implied, is or should be construed regarding geological conditions at locations other than those evaluated as part of this study.

The professional opinions, conclusions and recommendations presented in this report relate only to the project and locations specified in this report. If any changes are made in the project, the conclusions and/or recommendations in this report shall not be considered valid unless the changes are reviewed and the conclusions and recommendations of this study are modified and approved in writing by Geological Consultants Inc.

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A Seismic Refraction Survey Method

FINAL REPORT

SEISMIC REFRACTION SURVEY
OAK STREET BASIN

OAK STREET BASIN & STORM DRAIN DESIGN
OAK STREET & HAWES ROAD
MARICOPA COUNTY, ARIZONA
FCD CONTRACT No. 2009C021
RAMM PROJECT No. G17544

1.0 INTRODUCTION

1.1 General

This report presents the results of seismic refraction surveys conducted as part of a geotechnical investigation for the Flood Control District of Maricopa County's proposed Oak Street Basin and Storm Drain Design. The geophysical surveys were performed within the proposed Oak Street Basin that is located northeast of the intersection of Oak Street and Hawes Road in a Maricopa County parcel near Mesa, Arizona (Figure 1). The proposed basin, within the 10-acre parcel that is presently undeveloped native desert terrain, will be formed by excavating approximately 10 to 24 feet below the existing ground surface. Developed residential parcels surround the proposed basin area that is bounded on the south by Oak Street and on the west by Hawes Road (Figure 2)

The objective of the seismic refraction survey is to provide supplemental subsurface information for the geotechnical investigation conducted by RAMM and to determine the relative excavateability characteristics of the site soil and bedrock within the limits of the proposed basin.

The general geology of the Oak Street Basin site is relatively simple within the upper thirty to 50 feet of the stratigraphic section. The predominant geologic unit consists of massive, variably weathered granite bedrock that is overlain by dense to very dense gravelly residual soil derived by the in-place weathering of the underlying granite and by dense to loose accumulation of gravelly overburden slope wash debris and localized sandy ephemeral stream channel deposits soil.

Based on information deduce from the analysis of the seismic refraction survey data, combined with the interpreted drill hole data provided by RAMM (2010), the native, in-place overburden soil thickness is expected to be variable within the proposed basin area ranging from about nil to about ten feet thick.

1.2 Scope of Work

The scope of work used to accomplish the objectives of the qualitative excavateability evaluation and the seismic refraction surveys included the following activities:

- Project coordination with RAMM as required to complete the work in a timely and efficient manner.
- Prior to conducting the seismic refraction surveys, GCI reviewed the field drill hole logs from the site that were provided by RAMM (2010).
- Field explorations, included four seismic refraction survey lines as the locations depicted in Figure 2.
- Cursory field geological visual observations were made of the geologic and soil material exposed at the site, along with the review of the drill hole logs, to assist with the interpretation and modeling of the seismic refraction survey data.
- This seismic refraction survey report was prepared to document the field investigation, data modeling and interpretations, and our conclusions and recommendations relative to the site stratigraphy profile and rock excavateability. This report also includes photographs, supporting graphics, and tables.

The seismic refraction surveys were conducted on June 4, 2010 by GCI with Mr. Kenneth M. Euge, R.G. in responsible charge of the field work and with the assistance of Mr. Mike Blakely, Field Engineering Technician with RAMM. Existing ground conditions were evaluated based on visual examinations made during the field reconnaissance, results of the seismic refraction surveys, and our professional experience in this type of geological environment. No direct

subsurface explorations, such as drilling, coring, or trenching was conducted by Geological Consultants Inc. as part of this study. However, subsurface explorations including bore hole drilling and sampling and backhoe test pits were conducted by RAMM (2010) for the geotechnical investigation of the proposed Oak Street Basin area .

2.0 CONCLUSIONS AND RECOMMENDATIONS

This section summarizes our conclusions and recommendations derived from the data research and evaluation, the geological observations made at the site, and the seismic refraction surveys conducted by GCI at the Oak Creek Basin site. The analysis and interpretation of the geological data gathered during this investigation and our interpretations are also used to develop related recommendations as they pertain to possible constraints to the excavation and construction of the proposed Oak Street Basin depicted on the design plans provided by RAMM to GCI.

2.1 General Geology The Oak Street Basin site is presently undeveloped desert terrain consisting of gently undulated low hills and knobs that have been dissected by ephemeral stream channels that cross the site from the northeast to the southwest. Our interpretation of the site geology is based on the observations of the soil and bedrock materials exposed within and adjacent to the project site, review of the drill hole logs prepared by RAMM (2010), the results of our analysis of the seismic refraction survey data, and our prior experience on other projects in the vicinity of the Oak Street Basin.

The near surface granite bedrock is expected to be strongly weathered to locally decomposed while at greater depths the bedrock is expected to be moderately to slightly weathered and very hard. Within the weathered bedrock zone, core stones of very hard, well-indurated granite will likely be encountered during excavation of the basin. The granite is light orange brown to reddish gray and it is very coarse crystalline to porphyritic containing large, interlocking crystals of potassium feldspar and plagioclase. As stated previously, the granite is massive and it is very dense where decomposed to very well indurated where slightly weathered. Although no large bedrock outcrops were observed in the vicinity of the seismic survey lines, the bedrock is expected to be broken by joints and fractures. Localized bedrock balds were encountered exposing very weathered decomposed granite capped by thin accumulations of caliche. The secondary caliche layer appears to have been deposited along the contact boundary between the granite bedrock and the overlying overburden soils.

2.2 Seismic Refraction Surveys

Interpreted stratigraphic relationships with the proposed Oak Street Basin developed from the seismic refraction survey data combined with the observations made of the site geology and the examination of the drill hole logs prepared by RAMM (2010), are presented in the seismic refraction survey sheets Figures 3 through 6 and provide the basis to define the qualitative excavateability (Table 1) of the soil and bedrock expected to be encountered within the proposed basin area.

Interpretation of the seismic refraction surveys suggest that the proposed Oak Street Basin excavation will encounter overburden soils with relative low seismic velocity ranging from about 1,000 feet per second (fps) to 1,300 fps to depths ranging from nil to about 10 feet below existing native grades. Below the overburden soils, very weathered to slightly weathered granite bedrock will be encountered with seismic velocities that range from less 3,200 fps to about 3,800 fps where decomposed to strongly weathered and greater than 6,400 fps where the moderately to slightly weathered granite bedrock is encountered. See Figures 3 through 6.

The relatively low seismic velocities of the overburden soils and the embankment fill indicate these soils should be relatively easy to excavate. Although the composed and strongly weathered granite bedrock may be excavateable, granite core stones where encountered could require secondary fragmentation. The high seismic velocities (greater than 6,000 fps) indicate the granite bedrock will be difficult to excavate and could require special fragmentation methods such as blasting to facilitate excavation (Table 1).

Based on our interpretations of the seismic data, conclusions presented regarding the depth to various velocity zones are believed to be reasonable at the location of the seismic survey line(s). The conditions characterized by indirect seismic methods along the seismic survey lines probably represent subsurface conditions that could be found in adjacent areas of the site. Refer to Section 3 for additional information regarding the seismic refraction surveys and qualitative excavateability. Also, refer to Appendix A for discussion of seismic refraction survey limitations.

2.3 Excavateability

The excavation constraints described in this report (Tables 1 and 2) are, in our opinion, reasonable for the locations where the seismic refraction surveys were conducted. The ultimate soil/bedrock excavateability is dependent on many factors (variably cemented soils, presence of core stones, bedrock and soil physical properties, excavation methods, size and age of excavation equipment, level of effort applied by the contractor, etc.) and it may not be possible to correlate these factors with the results of the seismic refraction surveys conducted for this investigation. The excavation contractor must exercise caution, and assume associated risks, when attempting to extrapolate these data to other areas where seismic surveys have not been conducted. There is no guarantee that the seismic refraction survey results (Table 1) or the qualitative excavateability (Table 2) can be duplicated by others. We recommend this information be used with caution and only as guidelines

Prospective contractors or others involved with excavation at this site should review this report for information that might be used as part of their evaluation criteria for selecting equipment that may be used to excavate or fragment site soils or granite bedrock material expected to be encountered at this site. However, the contractors using or making interpretation of this information, for any reason, do so at their sole risk. No site-specific testing has been conducted at this site to verify the information provided in this report or the results of the seismic refraction surveys (Table 1); however, four backhoe test pits excavated at the site by RAMM encountered decomposed or weathered granite bedrock at depths ranging from about three feet to about seven below native ground surface and refusal at 9 to 10 feet in two of the test pits (personal communication; RAMM, 2010). Refer to the RAMM reports for additional information regarding the test pit excavations. No site-specific equipment performance evaluations been conducted relative to ripping or excavating site materials or to determine equipment suitability for this site.

2.4 Construction Vibrations

If heavy vibration-producing equipment, such as tractor-mounted, hydraulic impact hammers (ram-hoe), or blasting is used to assist with the excavations made at this site, the contractor should be required to keep ground vibrations from any construction source within applicable safe limits for surrounding structures including buildings and utilities.

Blasting could be used for the granite bedrock fragmentation and for the secondary fragmentation of granite core stones. If blasting or heavy, tractor-mounted vibratory hydraulic impact hammers are used for rock fragmentation at this site, it should be conducted by a contractor experienced with controlled blasting or the use of heavy impact hammers in residential areas in similar granitic/geologic terrain. When blasting is proposed, the blasting should be conducted according to appropriate sections of the current International Fire Code with any amendments effective as of the date the proposed blasting, or other rules and codes mandated by the Flood Control District of Maricopa County, municipal, State and Federal agencies that have jurisdiction relative to blasting operations. We recommend preconstruction and pre-blast surveys be made of all structures within 500 feet of the proposed blasting or high intensity construction-induced vibrations.

We recommend adequate "safety zones" be established and maintained around the proposed excavation sites during construction. Likewise, we also recommend that construction equipment operations and blasting should be monitored during construction to assure the ground vibrations are within safe limits. We recommend the construction/blasting vibrations be limited to less than one inch per second for residential areas and for sensitive structures or components such as buried gas and water lines unless more restrictive allowable vibration limits are specified by other regulatory authorities. The purpose of the preconstruction surveys and construction vibration monitoring is to limit liability for property owners, the contractor, and other involved parties.

3.0 SEISMIC REFRACTION SURVEYS

Seismic refraction surveys were conducted within the proposed Oak Street Basin to indirectly investigate subsurface conditions and to assess the potential excavateability (rippability) of the bedrock and soil.

3.1 Site Specific Seismic Surveys

Following the completion of a site reconnaissance to identify the survey location, seismic refraction surveys were made at the sites depicted in Figure 2. In addition to defining the qualitative excavateability of the site soil and bedrock, the seismic surveys were also conducted to evaluate the residual soil overburden thickness, to determine the depth to the granite bedrock and the thickness of weathered bedrock zones that may be present within the proposed basin excavation area.

GCI and RAMM selected the preliminary seismic survey line locations. However, because of the excavation and subsequent backfill of backhoe test pits in the seismic survey line area and to minimize the damage to existing vegetation along the seismic survey lines, minor adjustments to the selected location were made by GCI in the field to accommodate these site conditions. A rough position survey to approximately locate the seismic survey line was conducted by Geological Consultants Inc. using taped measurements from existing drill hole locations and a hand-held GPS unit. Using an arbitrary datum, a surveyor level was used to measure the relative elevations of the survey line shot points and the geophone sensor locations (Figures 3 through 6). The estimated accuracy of the level survey is estimated to be about 0.1 feet. Photographs were also taken at each of the seismic refraction survey lines (Figures 7 and 8).

Three shot points are used along each seismic survey line to evaluate possible non-horizontal subsurface boundary conditions (buried sloping surfaces, cementation zones, bedrock boundaries, etc.) that could be expected in this type of geological terrain, and to improve the accuracy of the seismic wave velocity determinations. The shot points were located at 5-foot offsets from each end of the seismic lines and at the center of each line except at the 'A' shot point for survey line OAK-S4 where an 8-foot offset was used to avoid excessive vegetation and loose surface soils. The seismic survey lines were run over a length of 110 feet plus the shot point offset at each end of the line to achieve adequate depth penetration and to identify the subsurface zones that could influence the basin excavation. In our opinion, because of the low background noise and the

quality of the low and high seismic velocity contrasts in the recorded seismic data, we expect that 30 feet to 40 feet of depth penetration, or more, was achieved at each of the seismic refraction survey sites.

As with any type of geophysical investigation method, there are limitations to its usefulness and application. Refer to Appendix A for additional information regarding seismic refraction surveys and their limitations.

3.2 Equipment

Travel-time data for the seismic traverses were obtained using Geometrics Inc. Model S12 SmartSeis™ 12-Channel Exploration Seismograph. Seismic wave arrivals are detected with digital grade vertical geophones with a dual hum-bucking coil and a frequency response above 14 Hz natural frequency. The first geophone vibration sensor was placed at seismic line Station 0 with subsequent geophones placed on 10-foot centers to the end of the survey line. The seismic shock wave is produced by repeated impacts of a 16-pound sledge hammer onto a soft-steel striking plate. Hammer impacts were made at 5-foot offsets (8-foot at OAK-S4 'A'-end) from each end of each seismic line traverse and at a shot point located at the center of survey lines. The distance from the impact station to the geophones and the travel time recorded for each station is stored in the seismographs onboard computer. If the field seismic data plots indicated the possible presence of anomalous subsurface conditions or spurious noise coincident with the hammer impacts, repeated impacts are used to verify the initial data reading or to correct the data. Topography, outcrops, and other natural or man-made features found along the seismic survey lines that might influence the data interpretations are annotated with the field data plots.

3.3 Results

Interpretations of the seismic survey data obtained at the project site were computer-modeled using Seismic Refraction Interpretation Programs (SIP) (Rimrock Geophysics, 2000). The results of the analysis suggest the presence of a distinctive subsurface stratigraphic profile along each of the seismic survey lines. Seismic velocities, calculated zone thicknesses, and depth to velocity zone boundaries correlated to each of the interpreted bedrock or soil types are summarized in Table 1. The qualitative excavateability of the rock and soil units related to their respective P-wave velocities encountered along each of the seismic survey lines is summarized in Table 2. The interpreted geologic velocity profiles and the distance-travel time data plots are

depicted in Figures 3 through 6, inclusive. The elevation data depicted on the Y-axis of the Depth Cross-Sections is based on information derived from level survey using an arbitrary datum. The data depicted on the figures can be used to scale depths to different velocity layers below the ground surface. The estimated accuracy of the velocity layer boundaries is approximately 20 percent. The calculated depth to the granite bedrock/overburden soil contact, determined from the seismic refraction survey, correlates very well with the depth to bedrock encountered in the drill holes and backhoe test pits logged by RAMM (2010).

Information presented relative to the thickness and depth below grade of the various stratigraphic/velocity zones is believed to be reasonable at the location of the seismic survey lines.

3.3.1 Excavateability

The estimated qualitative excavateability summarized in Table 2 is based on our interpretations of the seismic survey data, understanding of the site geological conditions, and professional experience. Because the seismic velocities used to determine qualitative excavateability may vary from 10 to 20 percent, and due to the variability of the subsurface material, qualitative excavateability listed in Table 1 and Table 2 may overlap at the transition from one constraint category to the next.

The qualitative excavateability summarized in Table 2, along with our interpretation of the subsurface materials (Table 1) are provided so that a prospective contractor can relate seismic velocities to the subsurface materials they can expect to encounter within the required excavation depth. Although a backhoe may be able to excavate and fragment low velocity material and a D9 with a single shank ripper might be able to rip moderate to high velocity material, there are no guarantees due to the wide range of variables summarized in Section 2.3 that effect equipment suitability and material fragmentation (rippability) and excavateability. A heavy rock trencher may be able to excavate the alluvial soils cemented to a rock-like consistency but progress should be expected to be very slow. Large boulder-size rock fragments or coherent tabular blocks could require secondary fragmentation.

Table 1
Seismic Survey Line Calculated Depth/Velocity Ranges[1]
Oak Street Basin

Survey Line No.	Calculated Depth Range (ft) @ Shot Point[2]			Average Seismic Velocity (ft/sec)	Interpreted Geologic Description	Qualitative Rippability
	A	B	C			
OAK-S1 (Figure 3)	0 - 2.5	0 - 1.5	0 - 3.5	1,066	Residual Soil; Slope Wash Debris; Local Stream Channel Deposits	Slight
	2.5 -19.0	1.5 - 15.5	3.5 - 16.5	3,828	Decomposed Granite, Strongly to Moderately Weathered Granite Bedrock	Marginal
	> 19.0	> 15.5	> 16.5	6,393	Moderately to Slightly Weathered Granite Bedrock	Severe
OAK-S2 (Figure 4)	0 - 3.0	0 - 3.0	0 - 2.0	1,217	Residual Soil; Slope Wash Debris; Local Stream Channel Deposits	Slight
	3.0 - 25.5	3.0 - 29.5	2.0 - 33	6,538	Moderately to Slightly Weathered Granite Bedrock	Severe
	> 25.5	> 29.5	> 33.0	14,126	Slightly Weathered to Unweathered Granite Bedrock	Severe
OAK-S3 (Figure 5)	0 - 5.5	0 - 5.0	0 - 3.0	877	Residual Soil; Slope Wash Debris; Local Stream Channel Deposits	Slight
	5.5 - 21.0	5.0 - 26.5	3.0 - 25.5	3,348	Decomposed Granite, Strongly to Moderately Weathered Granite Bedrock	Marginal
	> 21.0	> 26.5	> 25.5	8,757	Moderately to Slightly Weathered Granite Bedrock	Severe
OAK-S4 (Figure 6)	0 - 1.0	0 - 3.5	0 - 8.0	1,335	Residual Soil; Slope Wash Debris; Local Stream Channel Deposits	Slight
	1.0 - 12.5	3.5 - 16.0	8.0 - 18.5	3,276	Decomposed Granite, Strongly to Moderately Weathered Granite Bedrock	Marginal
	> 12.5	> 16.0	> 18.5	8,978	Moderately to Slightly Weathered Granite Bedrock	Severe

1) Refer to Figure 2 for seismic survey line locations.

2) Shot Point A offset -5 feet from line start (@ OAK-S4 'A'end Shot Point offsest 8-feet); Shot Point B @ line center; Shot Point C offset 5 feet from line end.

Table 2
Qualitative Excavateability Relative to Soil/Bedrock Type & P-Wave Velocity
Oak Street Basin

Unit	Average P-Wave Velocity (feet per second)	Excavateability Constraints
Overburden Soils or Very Weathered Decomposed Granite	< 3,000	**Slight**- Should be able to excavate using conventional earthmoving equipment. Could contain localized caliche cemented overburden soil and hard granite core stones that may require special excavation methods for effective fragmentation.
Decomposed and Slightly to Moderately Weathered Granite Bedrock	3,000 to 6,000	**Marginal**- Potentially difficult to excavate or marginally rippable with conventional equipment; a large tractor dozer with a single-shank ripper could be effective. Boulder-size, hard, well indurated granite core stones and localized caliche-cemented zones could be encountered in this velocity zone. These boulder-size core stones and the caliche-cemented soil could require special fragmentation methods to excavate effectively.
Moderately to Slightly Weathered Granite Bedrock	> 6,000	**Severe**- Blasting could be required for effective fragmentation. May be locally rippable along weathered joint/ fracture planes. Very heavy dozers with single-shank rippers and/or high torque heavy track-mounted excavators with hydraulic hammers may be effective but slow for fragmentation. Boulder-size core stones and boulder-size rock fragments could be generated.

Tables and charts have been developed by others (Caterpillar , 1997) for similar geological bedrock material. This information may be used by prospective contractors as part of their evaluation criteria for selecting equipment that may be used to excavate or fragment site soils or rock material. However, no site-specific testing has been conducted at this site to verify equipment performance relative to excavating bedrock within the Oak Street Basin excavation area. Therefore, the contractor must exercise caution, assume associated risks, and realize there are no guarantees if these charts and tables are used to determine the equipment suitability for this site.

All naturally deposited soil, soil cemented to a rock-like consistency, and bedrock material are "excavateable" and "rippable" to some degree. If a dozer, heavy excavator, or backhoe is determined by the equipment operator or contractor to have experienced "refusal" or encounters "unrippable" material that is not "excavateable," it is likely a determination that has been made by the equipment operator or the contractor because of the poor economics or profitability associated with the limited ripping ability or low production volume of the particular soil or rock material being excavated by the particular piece of equipment.

4.0 BIBLIOGRAPHY

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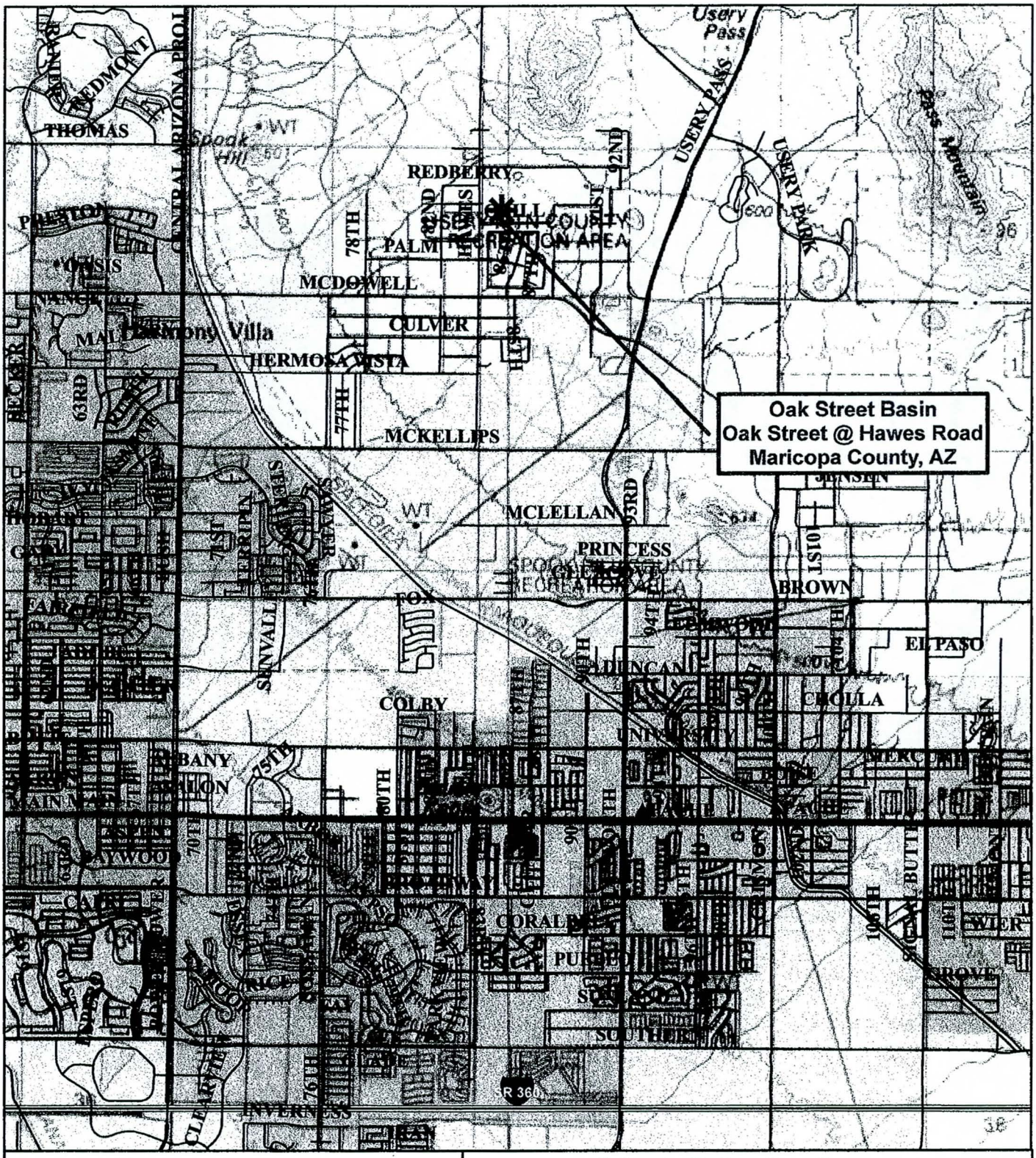
Ricker, Atkinson, McBee, Morman & Associates Inc. (RAMM); 2010; consultants report entitled "Preliminary Geotechnical Recommendations, Oak Street Basin and Storm Drain Design, Oak Street and Hawes Road, Maricopa County, Arizona"; FCD Contract No. 2009C021; prepared for Gavan & Barker Inc., Phoenix, AZ; March 10, 2010; RAMM Project No. G17544; 3 p.

Rimrock Geophysics; 2000;Seismic Refraction Interpretation Programs (SIP) computer programs.

FIGURES



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1:60,000

Base Map: USGS 30 x 60
Mesa Quadrangle

Seismic Refraction Survey
Oak Street Basin & Storm Drain Design
Figure 1
Location Map

GEOLOGICAL CONSULTANTS INC.
2333 West Northern Ave, Suite 1A
Phoenix, AZ 85021
phone 602-864-1888
fax 602-864-1899

Not to Scale

GEOLOGICAL CONSULTANTS INC.

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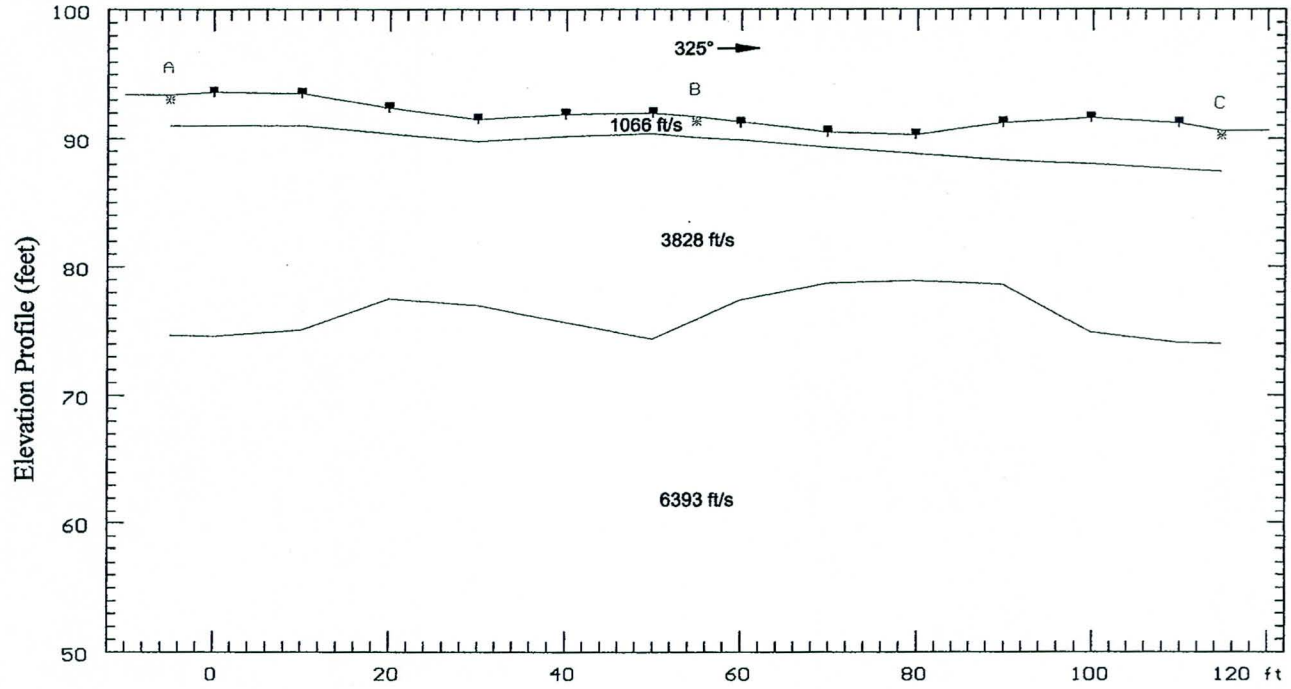
Seismic Survey Line by GCI (approx.)

◎ Drill Hole by RAMM (approx.)

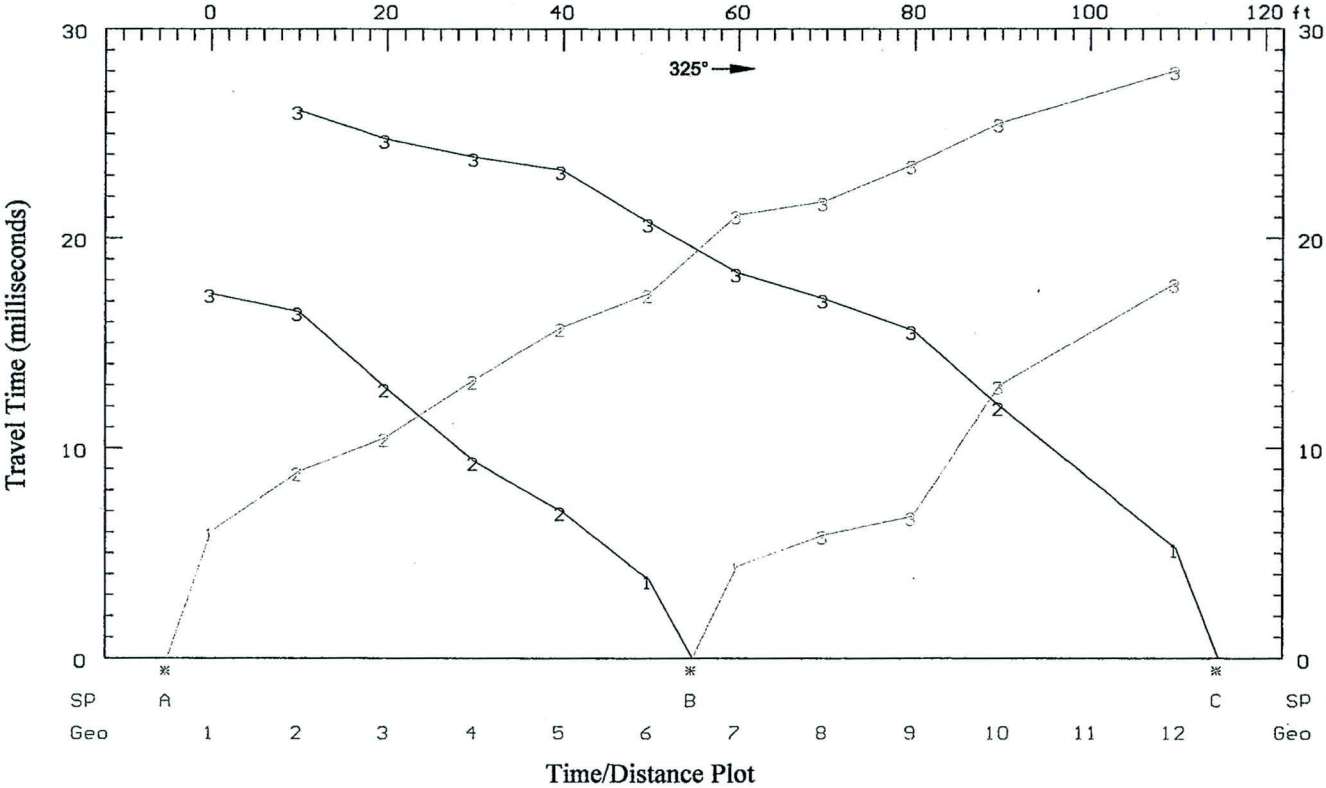
Refer to Figures 3 thru 6 for Seismic Velocity Profiles and Time-Distance Plots.

Base photo by Google/USGS USDA, June 2010.

Seismic Refraction Survey
Oak Street Basin & Storm Drain Design
Seismic Survey Line Location Plan
Figure 2



Interpreted Velocity Profile



Time/Distance Plot

Refer to Figure 2 for survey line location.

Seismic Refraction Survey
Oak Street Basin & Storm Drain Design
Seismic Survey Line OAK-S1
Figure 3

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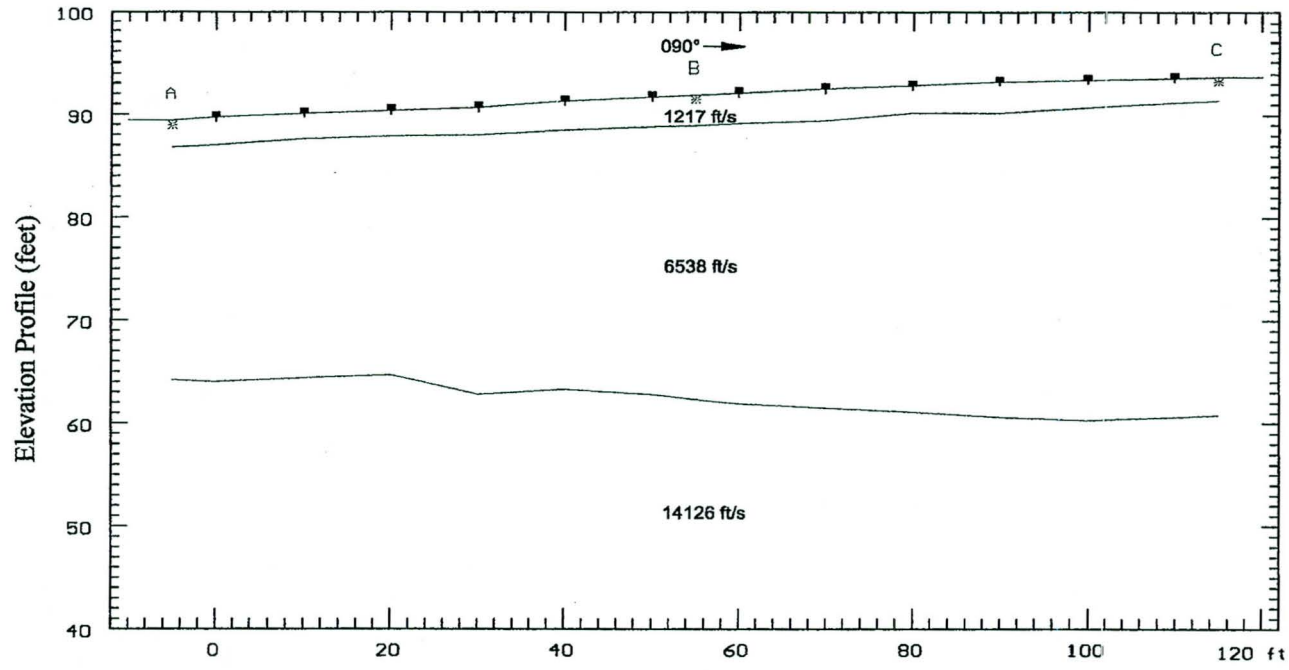
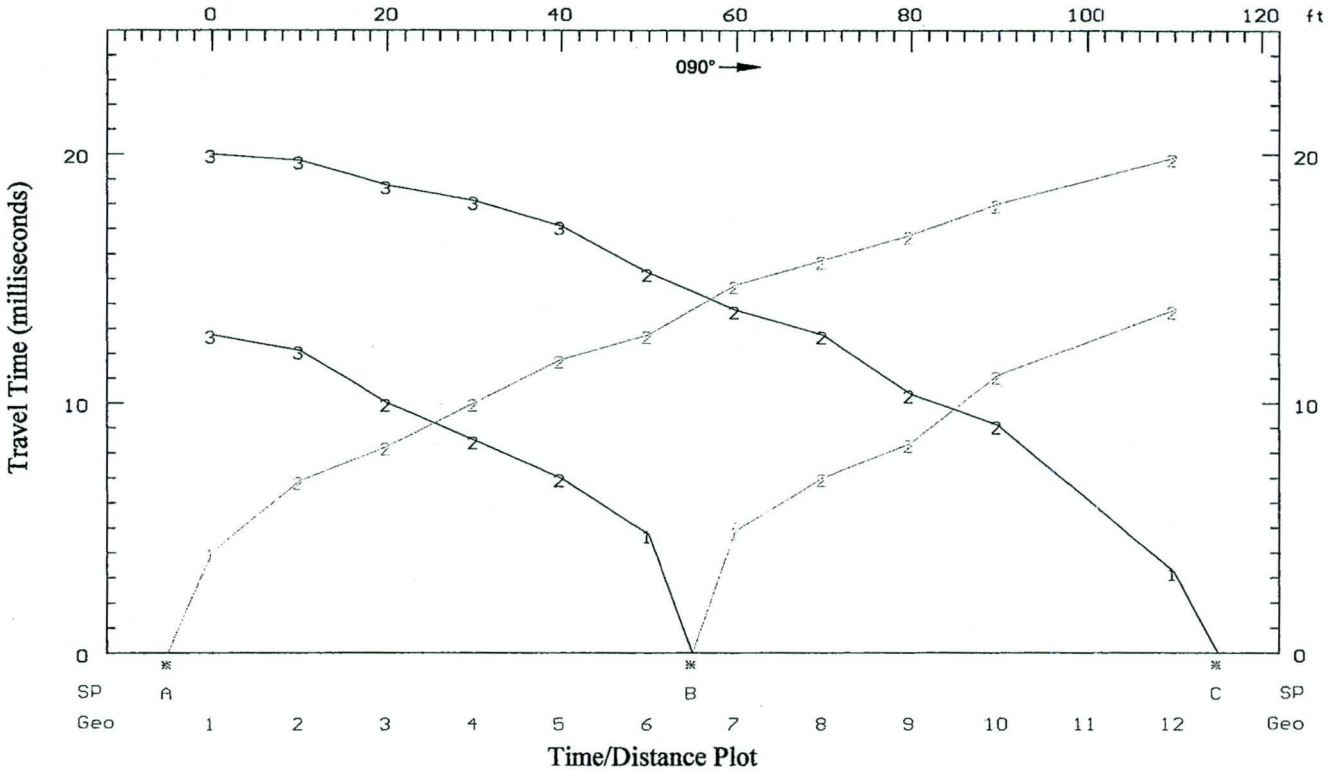


Interpreted Velocity Profile



Time/Distance Plot

Refer to Figure 2 for survey line location.

Seismic Refraction Survey
Oak Street Basin & Storm Drain Design
Seismic Survey Line OAK-S2
Figure 4

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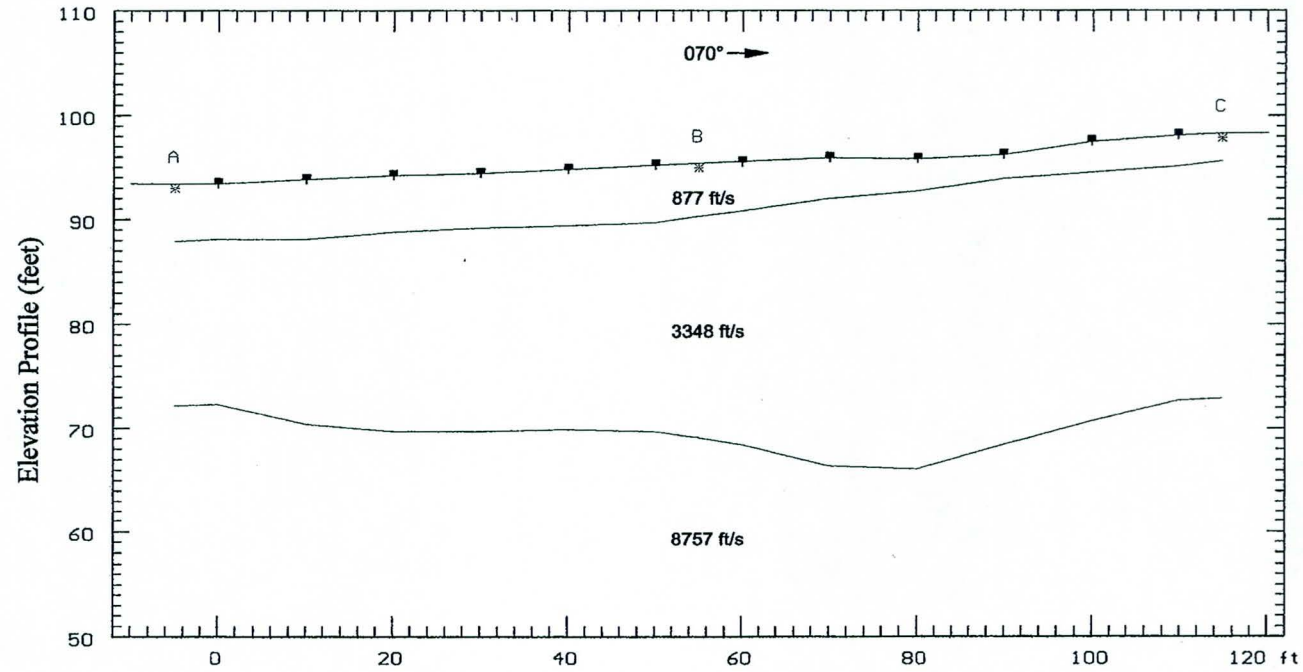
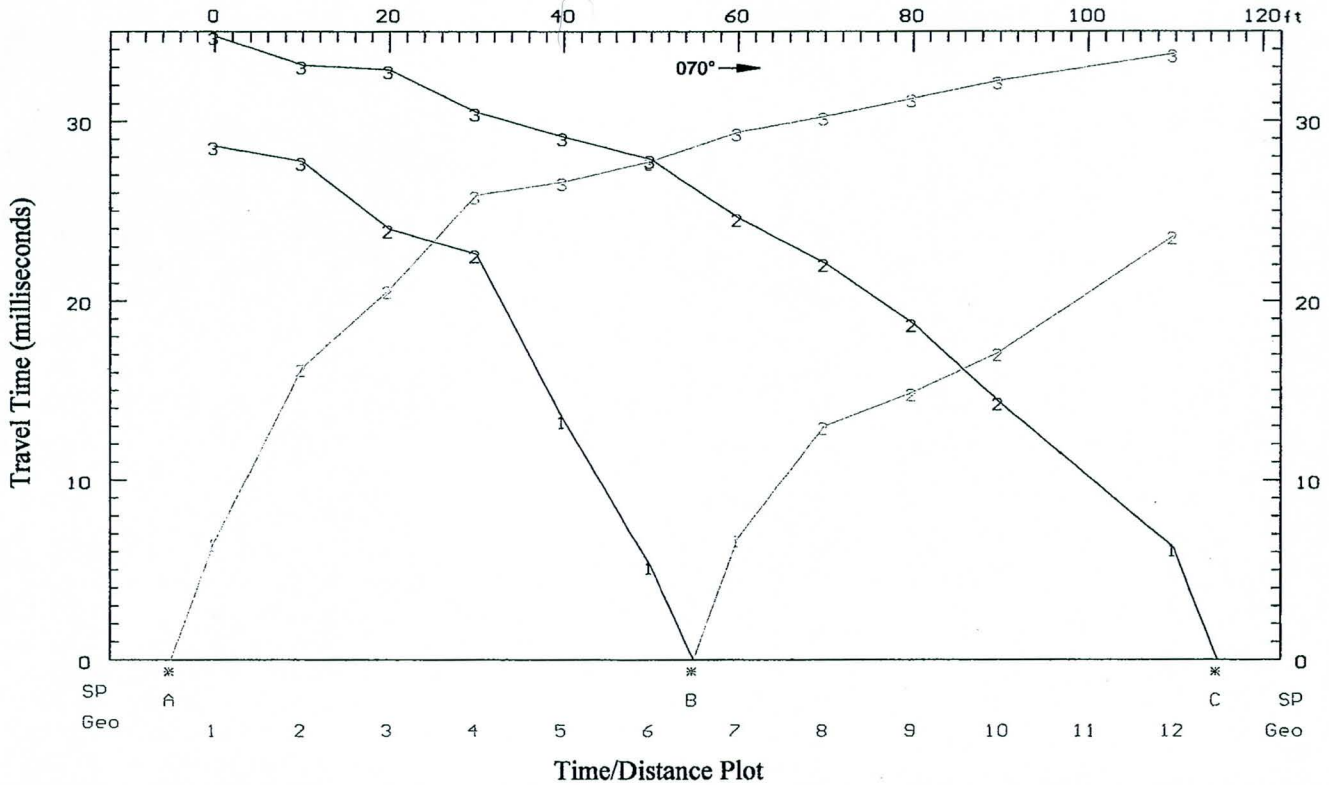


Interpreted Velocity Profile



Time/Distance Plot

Refer to Figure 2 for survey line location.

Seismic Refraction Survey
Oak Street Basin & Storm Drain Design
Seismic Survey Line OAK-S2
Figure 5

Geological Consultants Inc.
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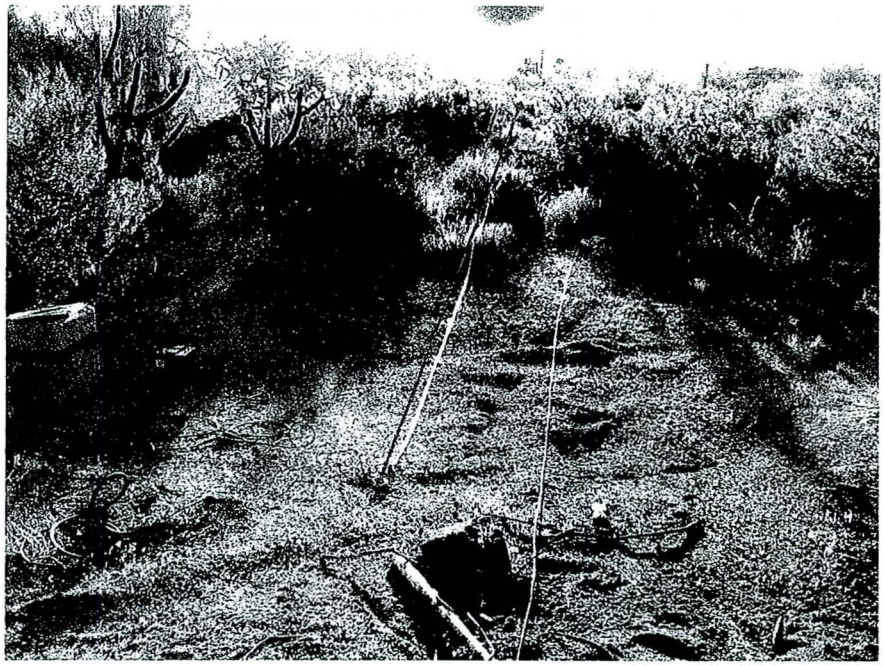

Photo 8-1: Alignment of Seismic Survey Line OAK-S3 looking East-Northwest. RAMM drill hole B1 approximately 9 feet offset from line end at geophone A.

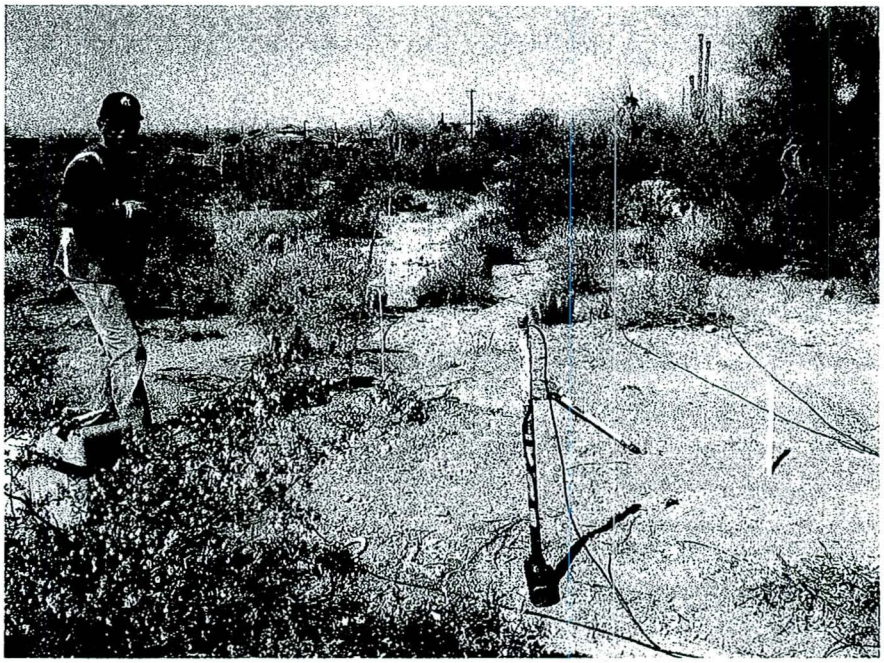

Photo 8-2: Alignment of Seismic Survey Line OAK-S4 looking Southwest. RAMM drill hole B4 approximately 9 feet North of seismic survey line end at geophone A. B4 drill hole location indicated by white stake at right of view. Disturbed area around drill hole caused by backfilling of backhoe test pit.

Refer to Figure 2 for survey line location.

Photographs by K. Euge, R.G. June 4, 2010.

Seismic Refraction Survey
Oak Street Basin & Storm Drain Design
Seismic Survey Line Photos OAK-S3 & S4
Figure 8

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APPENDIX A

SEISMIC REFRACTION SURVEY

APPENDIX A

SEISMIC REFRACTION SURVEY

A.1 GENERAL

In general, seismic wave velocities are related to the hardness, consolidation, and density of the materials through which seismic (shock) waves travel. Seismic velocities of subsurface soils and bedrock can be correlated to some of the physical properties of the material with reasonable levels of confidence. As with rock rippability (ease of excavation) for example, the Caterpillar tractor Company has correlated ranges of seismic velocities in different rock and soil materials to qualitative estimates of rippability for their D-9 tractor with a mounted hydraulic No.9 ripper.

The use of seismic velocities measured in various soils and rock types are considered reasonably conservative for evaluating soil and rock characteristics by "indirect" shallow geophysical seismic methods. Some general correlations are as follows:

- Soil, loose surface material, alluvium and strongly weathered and broken bedrock has velocities ranging from 500 feet per second (fps) to 1,200 fps;
- Moderately hard, slightly to moderately cemented, dense alluvial and colluvial sediments and moderately weathered and broken bedrock range from 1,200 fps to 3,000 fps;
- Very dense, hard, well-cemented soils and moderately competent bedrock range from 3,000 fps to 6,000 fps;
- Sound, relatively homogeneous or tightly jointed bedrock and uniformly, strongly cemented soils (silica hardpan, caliche, calcrete, etc.) have seismic velocities greater than 6,000 fps.

Soils and rock with velocities of less than 3,000 fps can usually be excavated with conventional earth moving equipment. Where materials with velocities in excess of 6,000 fps are found, blasting would normally be required for efficient fragmentation. However, if the rock is thinly bedded, jointed, or fractured, it may be possible to break the rock with heavy ripping using a single shank ripper or large ram-hoe. The resulting fragments will be of a size consistent with the fracture spacing and the progress of the excavation would be very slow.

The intermediate material (velocities between 3,000 fps and 6,000 fps) would likely require heavy equipment and possibly the localized use of jack- hammers, hydraulic breakers (ram-hoes), or selective blasting to provide cost-effective excavation.

A.2 DATA COLLECTION

Shallow seismic refraction data are normally collected along seismic survey lines consisting of 12 geophones spaced 10 feet apart. This geometry provided coverage of about 110 feet along each survey line. However, due to site constraints or other limitations, the seismic survey line length may be shortened or lengthened. Refer to Figure 2 for the seismic survey line locations. Seismic waves were generated at shot points located at 5-foot offsets from each end of the line and at the center to measure shallow materials (near-surface) seismic velocities. Depending on the project requirements and the type of soil or bedrock conditions anticipated at depth, data may be recorded from larger offsets (10 feet or more) past the line ends to measure the deeper velocities. Data are also recorded from both line ends so the effect of layer inclination, or dip on velocity boundaries, could be calculated. This geometry provided at least 20 feet, or more, of penetration at most line locations.

A.3 REFRACTION SEISMIC SURVEY LIMITATIONS

The seismic survey data presented in this report are derived from and interpreted from an indirect geophysical investigative technique (seismic refraction surveys) employed at the specific locations indicated and from observations made of the surface geologic conditions exposed at the site. The interpretations made at the specific seismic survey sites are believed to be reasonable based on the information available at the time of this study. The interpretations may not represent, nor are they intended to represent, the subsurface conditions at other locations.

Geologic contacts between rock and soil units are approximate, may be either gradual or abrupt, and the calculated depths could vary from 10 to 20 percent or more. Geological and geotechnical information provided others and our experience on similar projects in similar geological terrain were considered in the interpretations of subsurface conditions.

A.4 REFRACTION DATA PROCESSING

Seismic Refraction Interpretation Programs (SIP) computer programs by RIMROCK GEOPHYSICS, were used to analyze seismic data obtained in the field. The programs calculate average velocities of any number of layers assuming the multilayered intervals do not include velocity inversions or "hidden" zones (i.e., a high velocity zone over a low velocity zone). Thicknesses of each layer, except for the lowermost layer, are calculated along with the dip (inclination) angle of the layer boundary. The depth below the ground surface to each layer boundary is also provided.

Input data, velocity of each layer and seismic wave arrival times, obtained during the field work are checked by the computer program to assure that they satisfy reciprocity at least within 20 percent. These data are used to develop a meaningful geological model used to interpret subsurface stratigraphic conditions.

Supplement No. 1 – Foundation & Pavement Design Recommendations

R·A·M·M

RICKER • ATKINSON • McBEE • MORMAN & ASSOCIATES, INC.
Geotechnical Engineering • Construction Materials Testing

Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

December 16, 2010

Attention: Mark T. Gavan, P.E.

Subject: Geotechnical Engineering Report
Oak Street Basin and Storm Drain Design
Oak Street and Hawes Road
Maricopa County, Arizona
FCD Contract No. 2009C021

RAMM Project No. G17544
Supplement No. 1

This letter provides foundation and pavement design recommendations for the Oak Street Basin and Storm Drain Design, to be located in Maricopa County, Arizona. The project will include drop structures to depths below existing grades of 20 feet, retaining walls, and pavement reconstruction along Oak Street.

Foundation Design Recommendations

Spread and Mat-Type Footings:

The proposed drop structures and retaining walls can be supported on shallow spread and mat-type footings. The proposed footings should be founded on undisturbed site soils, undisturbed decomposed to slightly weathered granite bedrock and/or compacted fill. Footings thus founded may be designed using allowable bearing pressures and bottom of the footing depths of embedment below finished grade presented in the following table.

	Foundation Type				
	Spread		Mat		
Bearing Material	Depth of Embedment (feet below finish grade)	Bearing Pressure (psf)	Depth of Embedment (feet below finish grade)	Bearing Pressure (psf)	Modulus of Subgrade Reaction (pci)
Soil/Compacted Fill*	1.5	1500	1.0	1000	250
	2.0	2000	1.5	1500	300

	Foundation Type				
	Spread		Mat		
Bedrock	1.5	3000	0.0	2000	750
	2.0	4000	0.5	2500	750

*Note: Bearing pressure may be increased to 3000 psf for footings founded in areas where depth of excavation is at least 10 feet below existing site grades.

Finished grade is defined as the lowest adjacent finished grade within 5 feet of the perimeter of the structure or walls. Structural loads should not exceed 8 kips per linear foot for walls. All footing excavations should be reviewed by a representative of the geotechnical engineer prior to placing reinforcing steel or concrete. Any loose, disturbed or unstable soils should be removed from the bearing surface and replaced with MAG cement/AB slurry or as otherwise directed by the geotechnical engineer.

The allowable bearing capacity should be applied to maximum, design dead plus live loads and may be increased by one-third when considering temporary loads such as transient wind or seismic loads. A one-third increase may also be used for toe pressures due to eccentric or lateral loadings, assuming the entire footing bearing surface remains in compression. The weight of the footing concrete below grade may be neglected in dead load computations. The recommended minimum footing width is 1.33 feet for continuous wall footings. A Site Class designation of B should be used for the site per the 2000, 2003 and 2006 International Building Code (IBC). The soil profile and site class designations are based on the general geology of the area in which predominantly granite bedrock exists to depths over 100 feet in the immediate vicinity of the site.

The estimated total and differential footing settlements, for anticipated loading conditions and anticipated presence of water at or near footing level, are less than 3/4 inch for foundations bearing on undisturbed site soils and and/or compacted fill and less than ¼ inch for foundations bearing on undisturbed decomposed to slightly weathered granite bedrock.

Lateral Earth Pressures:

The following tabulation presents the recommended lateral earth pressures and base friction values which should be used in the lateral design of footings, keyways, below grade structures and earth retaining systems. The lateral pressures are equivalent fluid pressures for average anticipated conditions.

Backfill Pressures (Site Soils and Import):
- Unrestrained walls ---------- 32 psf/ft
- Unrestrained walls (saturated) ---------- 80 psf/ft
- Restrained walls ---------- 55 psf/ft
- Restrained walls (saturated) ---------- 90 psf/ft

Passive Pressures
- Continuous (Site Soils) ---------- 350 psf/ft
- Continuous (Site Soils, saturated) ---------- 235 psf/ft
- Continuous (Granite Bedrock) ---------- 500 psf/ft
- Continuous (Granite Bedrock, saturated) ---------- 400 psf/ft

Coefficient of Base Friction:
- Concrete to soil (dry and saturated) ---------- 0.45
- Concrete to granite bedrock (dry and saturated) ---------- 0.60

The following material properties and parameters may be used for structure and retaining wall design:

	Material Properties/Parameters		
	Undisturbed Soil	Undisturbed Bedrock	Backfill
Unit Weight (pcf, dry)	110	150	125
Angle of Internal Friction (degrees)	32	45	35
Cohesion (psf)	0	0	0

Site Development Recommendations

Workability:

Wetting site soils such that moisture contents are at or above optimum could result in some soil pumping under dynamic loadings such as heavy construction equipment driving over the area. In foundation areas, some pumping is not detrimental to foundations provided the specified percent compaction is achieved. However, in flexible pavement areas where pumping has occurred, and in foundation areas where severe pumping has damaged subgrade conditions, the area should be

allowed to dry until soils are workable without pumping or the wetted areas removed and replaced with drier site soils.

Pavement Design Recommendations

Asphalt Concrete Pavements:

The following asphalt concrete pavement section is based on anticipated traffic types and frequencies, site soil conditions, percent passing the Number 200 sieve and plasticity index laboratory test results, Maricopa County Department of Transportation (MCDOT) Pavement Design Chart for Local Roads, existing Oak Street pavement section measurements of 2.5 inches to 3.0 inches of asphalt concrete on 8 inches of base material and a 20-year design life. Therefore, any material imported to the site and placed in pavement areas should have support characteristics the same as or better than the site soils.

	Pavement Section	
Area of Use	Asphalt Concrete	Base Material
Oak Street	2.5 inches	8.0 inches

This section is minimal and will require periodic maintenance (seal coats, overlays or patching) where proper drainage is provided and maintained. Should moisture penetrate to the subgrade soils or ponding occur on or adjacent to the pavement section, a significant reduction in pavement life could occur along with increased maintenance. Therefore, good surface drainage on and adjacent to the pavement is essential to achieving the desired pavement life.

Material Suitability and Requirements

Site Soils, Excavated Granite Bedrock, Milled Asphalt Concrete and Base Materials:

The site soils, excavated granite bedrock, milled asphalt pavement and base materials may be used as fill beneath drainage structure and retaining wall foundations and in paved roadways and unpaved roadways and as backfill for structures not designed to retain earth loads. These materials may not be used as backfill for earth retaining structures. All materials should be free of organics, debris, rubble and material greater than 4 inches in size.

Imported Soils:

Fill required beyond that available from site sources and for use in roadways, below structure foundations and as backfill for structures not designed to retain earth loads should be imported soils meeting MAG Specifications for Imported Borrow, Section 210.2. Materials used as backfill for earth retaining structures should be imported soils meeting MAG Specifications for Structure Backfill, Section 206.4.2

Site Preparation and Grading Procedures

Structure and Pavement Areas:

Recommendations presented in the previous sections of this supplement are based upon the following site preparation and grading procedures. Therefore, all earthwork should be accomplished with observation and testing by a qualified technician under the direction of a registered geotechnical/materials engineer. The following apply to structure and pavement areas.

1. Clear and grub the site by removing and disposing of all vegetation, any trash and debris, and any rubble, in accordance with MAG Section 201.

2. Moisture condition and place all fill and backfill materials required to achieve specified grades. Fill materials should be moisture conditioned, placed and compacted in horizontal lifts of thicknesses compatible with the compaction equipment being used.

3. Compact subgrade, fill, backfill, subbase fill or base material to the requirements presented in MAG Sections 200 and 300, as supplemented by MCDOT.

4. Place asphalt concrete in accordance with MAG Specifications Section 310 and 702, as applicable, using materials which comply with Section 321 of the MAG Specifications, as supplemented by MCDOT.

If you have any questions, please do not hesitate to call. This letter should be attached to and made a part of our original report dated July 26, 2010.

Respectfully submitted,
RICKER, ATKINSON, McBEE, MORMAN & ASSOCIATES, INC.

Expires – 9/30/2012
By: Kip E. Reese, P.E.

Expires – 3/31/2013
Reviewed By: Kenneth L. Ricker, P.E.

/bc
Copies to: Addressee (5 mgavan@gavanbarker.com)

Supplement No. 2 – Berm Evaluation

Geotechnical Engineering Report
Oak Street Basin and Storm Drain Design
Berm Evaluation
Oak Street and Hawes Road
Maricopa County, Arizona
RAMM Project No. G17544 Supplement No. 2
FCD Contract No. 2009C021

For:
Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

◆◆◆

By:
Ricker-Atkinson-McBee-Morman & Associates, Inc.
2105 South Hardy Drive, Suite 13
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R·A·M·M

RICKER • ATKINSON • McBEE • MORMAN & ASSOCIATES, INC.

Geotechnical Engineering • Construction Materials Testing

Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

March 22, 2011

Attention: Mark T. Gavan, P.E.

Subject: Geotechnical Engineering Report
Oak Street Basin and Storm Drain Design
Berm Evaluation
Oak Street and Hawes Road
Maricopa County, Arizona
FCD Contract No. 2009C021

RAMM Project No. G17544
Supplement No. 2

Attached to this letter is the Supplement No. 2 Geotechnical Engineering Report for the Oak Street Basin and Storm Drain Design-Berm Evaluation, to be located in Maricopa County, Arizona.

The proposed development will include construction of a berm/levee along the southwest quarter of the proposed basin. The berm will generally be 8 to 12 feet high, will be 100 feet to 125 feet wide at the base and 10 to 12 feet wide at the top. A 42-inch diameter reinforced concrete pipe (RCP), 237 feet long, will extend from the southwest corner of the basin, southwest beneath the berm. The results of our field explorations; laboratory testing; and engineering analysis, evaluation and recommendations are presented in the report.

The attached report was prepared based on project and site data available at this time and was prepared in a manner and to the standards of local geotechnical engineering practice. Our services did not include evaluations for the presence of hazardous materials, for corrosion potential/durability of on-site soils and water, or for area subsidence resulting from groundwater withdrawal or other geologic hazards.

If you have any questions, please do not hesitate to call. This report should be attached to and made a part of our original report dated July 26, 2010.

Respectfully submitted,
RICKER, ATKINSON, McBEE, MORMAN & ASSOCIATES, INC.

Expires – 9/30/2012

By: Kip E. Reese, P.E.

Expires – 3/31/2013

Reviewed By: Kenneth L. Ricker, P.E.

/dcw
Copies to: Addressee (5) (mgavan@gavanbarker.com)

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APPENDIX A - FIELD EXPLORATION

Site Plan A1

Soil Legend A2

Test Boring Logs A3

Registered Professional Engineer (Civil)
CERTIFICATE NO.
49948
KIP E.
REESE
Date Signed 3-22-11
ARIZONA U.S.A.
Expires 9/30/2012

APPENDIX B - LABORATORY ANALYSIS

Compression B1

Direct Shear B2

Sieve Analysis, Percent Passing No. 200 Sieve, Atterberg Limits B4

APPENDIX C – CALCULATIONS

Settlement Analysis

Slope Stability Analysis

Seepage Analysis

REPORT

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INTRODUCTION

This Supplement No. 2 report presents the results of our geotechnical engineering services for the Oak Street Basin and Storm Drain Design – Berm Evaluation, to be located in Maricopa County, Arizona. The scope of our services included performing a field exploration program, laboratory analysis and geotechnical engineering evaluation, analysis and recommendations. The geotechnical recommendations presented herein include those for berm construction, site development, and material use and requirements. We would be pleased to discuss with you any additional recommendations you may require. In addition, we are available to review project specifications and plans for conformance with our recommendations at no charge to you.

This firm should be notified for additional evaluation and recommendations if the project design parameters (location, type, size, pipeline alignment, etc.), site use or conditions encountered during construction differ from those presented herein.

PROPOSED CONSTRUCTION

The proposed development will include construction of a berm/levee along the west and south sides of the proposed basin. The berm will generally be 8 to 12 feet high, will be approximately 100 to 125 feet wide at the base, and 10 to 12 feet wide at the top of the berm to provide maintenance access. A 237–foot long, 42-inch diameter, reinforced concrete pipe (RCP) will provide basin drainage from the southwest corner of the basin and extend southwest beneath the berm. The bottom of the RCP will be approximately 8 to 10 feet below existing site grades within the footprint of the berm. The RCP will have a basin side concrete headwall, cutoff wall and rip-rap erosion protection.

SITE CONDITIONS

The proposed basin site is located at the northeast corner Oak Street and Hawes Road, in Maricopa County, Arizona. Site conditions at the time of our field exploration for the berm evaluation were relatively unchanged from our previous field exploration. The proposed berm and storm drain area was a gently rolling, undeveloped parcel which sloped gently downward to the southwest. Overall topographic relief in the berm area was approximately 13 feet. The berm and storm drain site was crossed by a relatively shallow, wide wash which was generally along

the alignment of the proposed storm drain. The site surface conditions in the berm and storm drain area consisted predominantly of granite-derived and stream deposit silty sand type soils. Vegetation on-site consisted of a slight to moderate coverage of native grasses, shrubs, cacti and trees.

FIELD EXPLORATIONS

Subsurface conditions in the proposed berm and storm drain area were explored by drilling three new test borings (10, 11 and 12) to depths of 15 to 16 feet, as shown on the Site Plan in Appendix A. The test borings were drilled with a CME 75 drill rig using 7-inch diameter, hollow-stem augers. The drilling equipment and crew were provided by D&S Drilling, Inc. The test boring locations were determined in the field by a field technician from our firm, who also directed the drilling. During the field explorations, representative disturbed and undisturbed samples were obtained, the test borings logged and soils and bedrock field classified by our field technician. The relatively undisturbed samples (ring samples) and driven disturbed samples (standard penetration tests (SPT)) were obtained by driving a 3-inch diameter, ring-lined, open-end sampler into the soil with a 140-pound hammer dropping 30 inches. The results of the field explorations are presented on the attached Test Boring Logs.

LABORATORY ANALYSIS

Representative samples obtained during the field explorations were subjected to the following tests in our laboratory.

Type of Test	Type of Sample	Number of Samples Tested
Compression	Undisturbed	1
Direct Shear	Undisturbed	2
Sieve Analysis, Percent Passing No. 200 Sieve & Atterberg Limits	Representative	3
Moisture Content/Dry Density *	Undisturbed	6

* Reported in the test boring logs

The results of the laboratory analysis are presented in Appendix B.

SUBSURFACE CONDITIONS

Proposed Berm and Storm Drain:

The results of the test borings are presented in Appendix A in the Test Boring Logs. In general, the surface and near surface soils encountered in the test borings, extending to depths of 3.5 to 10 feet, consisted of sand containing some gravel and trace to some silt. These soils were medium dense, had non-plastic to low plasticity fines, were interbedded with silty sand with some gravel and sand with some gravel and a trace to some clay. Underlying the surface soils, and extending to the maximum depths of our exploration (15 to 16 feet), decomposed to slightly weathered granite bedrock was encountered. The weathering decreased with depth. The granite bedrock was moderately soft to hard, hardness increasing with depth. The soil moisture contents were described as slightly damp. No groundwater was observed in the test borings.

DISCUSSION OF TEST RESULTS

Undisturbed samples of the near surface soils from the site exhibited moderate to high shear strength when tested in direct shear under submerged conditions.

ANALYSIS AND CONCLUSIONS

Engineering Analysis:

Engineering analyses were performed to evaluate a levee type berm for settlement, slope stability and seepage from base flood conditions. The berm was designed for a 500-year flood event and an emergency overflow elevation of 1814.0 feet. Peak flood elevation conditions are anticipated to exist for less than 24 hours. Our analysis is based on a review of basin outlet plan and profile and basin grading plans presented in 90% preliminary plans titled 'Oak Street Basin and Storm Drain Project', prepared by Gavan & Barker, Inc. dated December, 2010, the results of our field exploration and laboratory testing, and design and analysis methodologies presented in the U.S. Army Corps of Engineers Engineering Manual (COE EM) EM 1110-2-1913 titled "Design and Construction of Levees", dated 30 April, 2000.

Settlement:

Our settlement analyses are based on a soil profile from Test Boring No. 11, a generalized berm profile with a height of 10 feet above existing ground surface and soil properties based on and estimated from our laboratory test data for this report and our original report. Our calculations are presented in Appendix C. A summary of analyses values are presented below:

Compression Index, C_c: 0.004
Moist Density, in-situ soils: 110 pcf; Thickness: 10 feet
Moist Density, berm soils (compacted existing site soils): 135 pcf; Thickness: 10 feet
Calculated Settlement at the base of the embankment: $^3/_{16}$ inches

The soils are granular and relatively well drained. A majority of the settlement will likely occur during construction.

Slope Stability:

A typical cross-section was selected for slope stability analysis. The selection was based on our review of the referenced plans and criteria presented in the COE EM. Cross-section conditions identified in our review were: berm geometry (height, width, side slopes and maintenance road), berm fill and subsurface soil profiles and soil parameters. A detailed description of the cross section is presented below. Our supporting documentation is presented in Appendix C.
Note: γ = density; c = cohesion, Ø = phi, angle of internal friction.

Generalized Configuration:
Berm Basin Side slope – 4H:1V Berm Land Side slope – 5H:1V
Berm Base Width – 100 feet; Berm Top Width – 12 feet.
Head Wall Slope – 2H:1V
Maximum water depth (design storm) – 14 feet
Soil Profile:

Soil ID	Berm Fill	Native	Granite Bedrock
Depth Below Top of Berm (feet)	10	10 to 20	20+

Design Parameters:

Soil Type	Silty Sand, Some Gravel	Sand to Silty Sand,	Bedrock
Moist γ (pcf)	133	110	165
Soil ID	1	2	3
Moist Ø (°)	25	22	25
Moist c (psf)	0	300	165
Saturated γ (pcf)	140	130	165
Saturated Ø (°)	20	18	25
Saturated c (psf)	0	18	0

Slope stability analyses were performed on the cross section presented above. The section was analyzed using the Geosystem Software SB-Slope Versions 2.0 and 3.0. The program analyzed the section using the simplified Bishop Method. The following loading conditions were analyzed: End of construction (Case I); sudden drawdown (Case II); steady seepage at flood stage (Case III); and earthquake (Case IV, Case I conditions). The results of our analysis for the berm and COE EM minimum factors of safety are presented below.

Berm – Factors of Safety

Case	Calculated	COE EM Minimum
I (End of construction)	2.15	1.3
II (Sudden drawdown)	1.71	1.0
III (Steady seepage)	1.60	1.4
IV (Earthquake)	1.92	1.0

* Cross-section, soil profile, soil parameters and printout of computer analysis results are presented in Appendix C.

** A pseudo seismic analysis was performed using a 0.028g acceleration as the horizontal seismic force. A 100 year flood event occurring at the same time as a seismic event is not realistic. In addition, design parameters used in the analysis are conservative.

The above calculated factors of safety exceeded the COE EM minimum required values for all cases.

Seepage:

Our evaluation of the levee type berm due to seepage was based on assumptions and factors involved in seepage analysis presented in the COE EM Appendix B. The results of our seepage analysis are presented in Appendix C. A primary factor in the development of seepage is sustained high water and is assumed as part of our analyses. Seepage into or through levee type berm and foundation soils is not likely given the relatively short length of time of sustained high water. The parameters and results of our analyses, based on a simplified flow net and estimated permeability of the site soils, are summarized below.

Coefficient of permeability of native soils (k): 80×10^{-4} feet/sec.
Flow quality (q): 1.3 feet/sec
Exit gradient (i_e): 0.21; less than 0.5 which is below COE EM criteria.
Critical gradient (i_c):1.08; greater than 1 (piping will not occur).

Conclusions:

Estimated settlement was based on conservative soil parameters and saturation of the foundation soils. The actual settlement is likely to be significantly less due to a portion of the settlement occurring during construction and saturation of the foundation soils is unlikely due to the relatively short duration of water retention in the basin and short duration of sustained water level.

The slope stability of the levee type berm is above the COE EM minimums for the cases analyzed. Surface slides are not likely to occur due to the non-plastic to low plasticity of the anticipated berm fill soils and generally low slope angles.

Based on our seepage analysis and due to the relatively short duration of a sustained water level seepage into or through the berm and supporting soils seepage would likely not occur to an extent to have detrimental effects on the berm.

SITE DEVELOPMENT RECOMMENDATIONS

Construction Excavation and Temporary Construction Cut Slopes:

The proposed storm drain within the berm footprint will extend approximately 8 to 10 feet below

existing site grades. The following criteria are presented to aid in development of excavation plans.

1. Dewatering should not be required.

2. Unbraced temporary slopes in the sand and silty sand native soils and decomposed to highly weathered granite bedrock should be constructed no steeper than 1.5H:1V. Unbraced temporary slopes in the moderately weathered to unweathered granite bedrock should be constructed no steeper than near vertical. As an alternative to shoring, trench boxes or other forms of slope stabilization may be used for this project.

3. Surface areas behind the crest of excavations should be graded so that surface waters do not pond within 10 feet of the crest or drain into the excavation.

4. Heavy material stockpiles should not be placed within 10 feet of the crest. Similarly, heavy construction equipment should not pass or be parked within 10 feet of the crest.

5. The sides and crest of excavations and slopes should be monitored daily for evidence of movement or potential problems.

The design of any bracing or shoring systems should be reviewed by a qualified geotechnical engineer. Also, observations should be made by the geotechnical engineer during excavating to evaluate site conditions and determine if modifications are necessary in excavation procedures. If unbraced slopes are utilized, some surface raveling, erosion, and spalling should be expected unless measures are taken to stabilize exposed cut surfaces.

Storm Drain Excavation, Bedding and Backfill:

The storm drain should be constructed to include backfilling prior to construction of the berm. The storm drain excavation within the footprint of the berm should have a minimum clearance between the side and bottom of the pipe and the side and bottom of the excavation of 12 inches and should expose undisturbed native soils and/or granite bedrock. Any loose, disturbed and/or

unstable soils and granite bedrock should be removed from the exposed surfaces. The storm drain bedding and backfill should consist of one-sack of Portland cement per cubic yard of slurry mix (Controlled Low Strength Material, CLSM). The slurry should fill the excavation to include any voids up to the ground surface elevation. The CLSM should be placed in accordance with Maricopa Association of Government (MAG) Specifications, Section 604.

Berm Construction:

Berm construction should be performed in general accordance with COE EM 1110-2-1913 Category I construction methods and as presented in this report. The berm foundation preparation and berm construction should be done in accordance with the Site Preparation and Grading Procedures section of this report. Placement of berm fill soils should be in the direction of the long axis of the berm.

MATERIALS SUITABILITY AND REQUIREMENTS

Site Soils and Excavated Granite Bedrock:

Along the berm and storm drain alignment with the berm footprint, the site soils and excavated granite bedrock exhibit no to low plasticity. These soils and excavated bedrock may be used as fill in berm areas. All materials should be free of organics, debris, rubble and material greater than 4 inches in size and should be placed such that nesting and voids do not occur.

Imported Soils:

Fill required beyond that available from site sources and for use as berm should be imported soils meeting the following requirements:

Minimum Percent Passing No. 40 Sieve------ 30
Maximum Particle Size ---------- 4 inches
Maximum Swell Potential ---------- 1.5%*

* Based on a sample which is remolded to 95% of the ASTM D698 maximum dry density at a moisture content of 2 percent below optimum, placed under a surcharge load of 100 psf and wetted.

Placement of imported soils should be within the central one third of the berm footprint or within the basin side one third footprint to provide reduced permeability within the central portion of the berm or along the basin side of the berm.

Controlled Low Strength Material:

Controlled low strength material (CLSM) used as storm drain trench backfill within the footprint of the berm should conform to the requirements of Maricopa Association of Governments (MAG) Specifications (Section 728) for 1 sack CLSM.

Riprap:

Riprap used as erosion control material should conform to the requirements of MAG specifications for riprap (Section 703).

SITE PREPARATION AND GRADING PROCEDURES

Berm and Storm Drain Trench Excavation Areas:

Recommendations presented in the previous sections of this report are based upon the following site preparation and grading procedures. Therefore, all earthwork should be accomplished with observation and testing by a qualified technician under the direction of a registered geotechnical/materials engineer. The following apply to the berm areas and that portion of the storm drain trench excavation beyond the slurry backfill areas.

1. Clear and grub the site by removing and disposing of all vegetation, any trash and debris, and any rubble and remnants of former developments.

2. Moisture condition and place all fill and backfill materials required to achieve specified grades. Fill materials should be moisture conditioned, placed and compacted in horizontal lifts of thicknesses compatible with the compaction equipment being used.

3. Compact berm subgrade and fill, to the following minimum percent compaction of the ASTM D698 maximum dry density for each lift.

Material	Minimum Percent Compaction
Soil:	
Berm subgrade and fill	95

4. The moisture content of soil and base materials at the time of compaction should be:

Type	Area of Use	Moisture Content
On-site	Berm	Optimum plus or minus 3%
Imported	Berm	Optimum plus or minus 3%

5. Any soils which are disturbed or overexcavated by the contractor outside the limits of the plans or specifications should be replaced with materials compacted as specified above.

APPENDIX A
FIELD EXPLORATIONS

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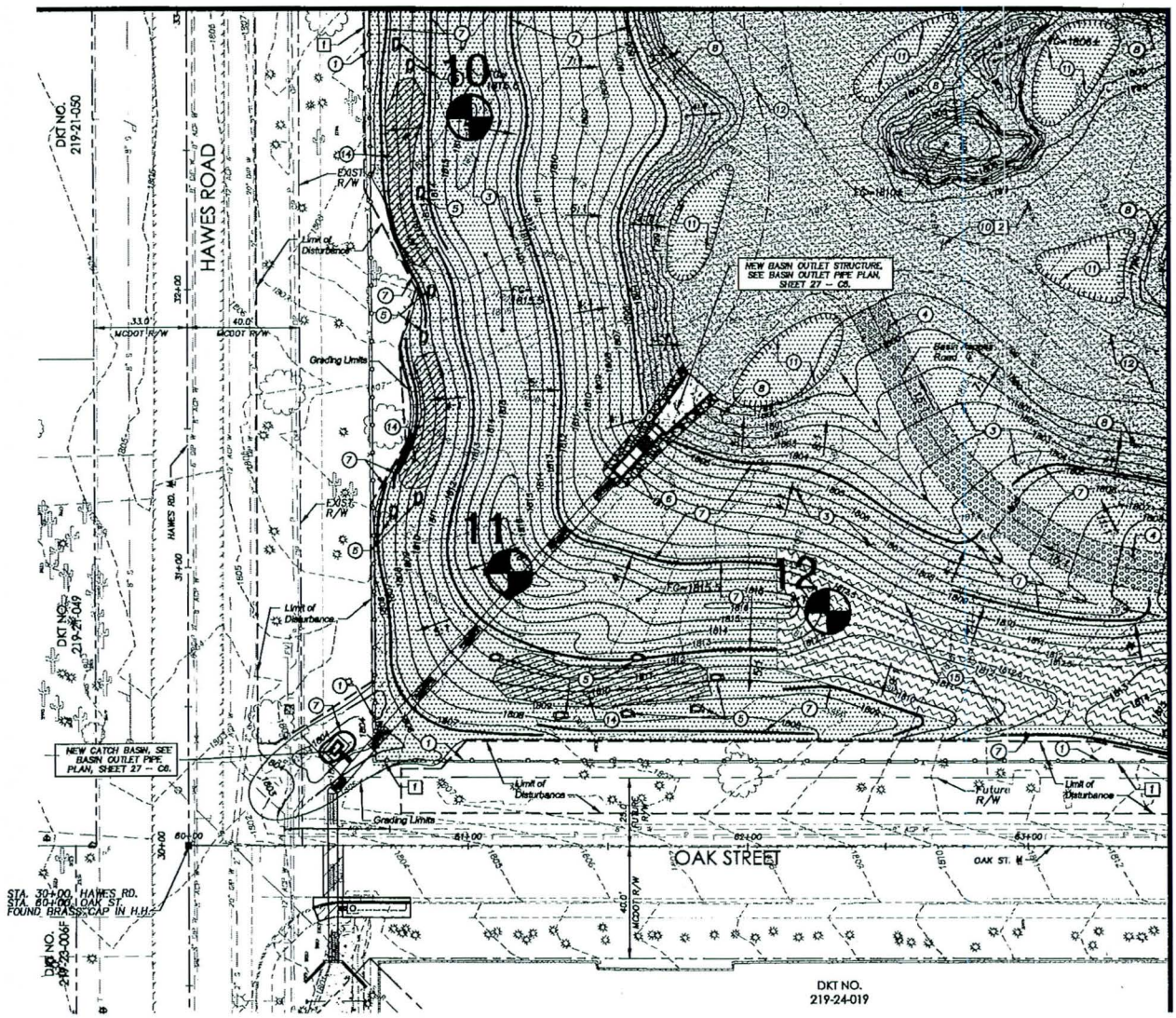


Test Boring Location

Not To Scale

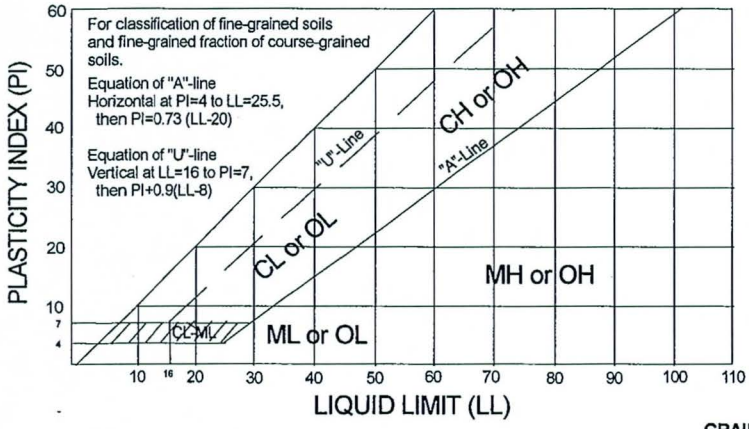
SITE PLAN

LEGEND

CLASSIFICATION OF SOILS

ASTM Designation: D2487-00
(Based on Unified Soil Classification System)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests					Soil Classification: Group Symbol	Soil Classification: Name
COARSE-GRAINED SOILS More than 50% retained on No. 200 Sieve	Gravels More than 50% coarse fraction retained on No. 4 Sieve	Clean Gravels Less than 5% fines	Cu > 4 and 1 <Cc < 3		GW	Well graded gravel
			Cu<4 and/or 1>Cc>3		GP	Poorly graded gravel
		Gravels with Fines More than 12% fines	Fines classify as ML or MH		GM	Silty gravel
			Fines classify as CL or CH		GC	Clayey gravel
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines	Cu > 6 and 1 <Cc < 3		SW	Well-graded sand
			Cu<6 and/or 1>Cc>3		SP	Poorly graded sand
		Sands with Fines More than 12% fines	Fines classify as ML or MH		SM	Silty sand
			Fines classify as CL or CH		SC	Clayey sand
FINE-GRAINED SOILS 50% or more passes the No. 200 Sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI>7 and plots on or above "A" line		CL	Lean clay
			PI<4 or plots below "A" line		ML	Silt
		Organic	Liquid Limit - oven dried / Liquid limit - not dried	<0.75	OL	Organic clay / Organic silt
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line		CH	Fat clay
			PI plots below "A" line		MH	Elastic silt
		Organic	Liquid limit - oven dried / Liquid limit - not dried	<0.75	OH	Organic clay / Organic silt
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor				PT	Peat



TEST BORING LOG DEFINITIONS

Blows per foot using 140 pound hammer with 30 inch free-fall.

Depth, feet	Blows/Foot: C	Blows/Foot: N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description

C = Continuous Penetration Resistance (2 inch diameter rod)
N = Standard Penetration Resistance (ASTM D1586)
R = Penetration Resistance (3 inch diameter ring line sampler)

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200	40	10	4	3/4"	3"	12"

SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

MOISTURE CONDITION (INCREASING MOISTURE ⟶)

DRY — SLIGHTLY DAMP — DAMP — MOIST (Plastic Limit) — VERY MOIST — WET (SATURATED) (Liquid Limit)

CONSISTENCY CORRELATION		RELATIVE DENSITY CORRELATION	
CLAYS & SILTS	BLOWS/FOOT*	SANDS & GRAVELS	BLOWS/FOOT*
VERY SOFT	0-2	VERY LOOSE	0-4
SOFT	2-4	LOOSE	4-10
FIRM	4-8	MEDIUM DENSE	10-30
STIFF	8-16	DENSE	30-50
VERY STIFF	16-32	VERY DENSE	OVER 50
HARD	OVER 32		

*Number of blows of 140 lb hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split-spoon sampler (ASTM D1586).

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 10

Elevation: Not Determined Sta: Date: 2-21-11

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		14	R	110	1	SW/ SM	Sand, Some Gravel, Trace to Some Silt; brown, slightly damp, medium dense, non-plastic to low plasticity fines, interbedded with silty sand, some gravel and sand, some gravel, trace to some clay.
5		100/6"	R	NR			
		100/4"	R	NR			Granite Bedrock, mottled grey/brown, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
10							
15							Stopped drilling at 15 feet. No groundwater observed. NR=No Recovery
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 11

Elevation: Not Determined Sta: Date: 2-21-11

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SW/ SM	Sand, Some Gravel, Trace to Some Silt; brown, slightly damp, medium dense, non-plastic to low plasticity fines, interbedded with silty sand, some gravel and sand, some gravel, trace to some clay.
		12	R	106	1		
5							5
		17	R	108	2		
		50/11"	R	116	1		
10							10
		66/10"	R	121	1		Granite Bedrock, mottled grey/brown, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
		100/2"	R	NR			
15							15
		100/2"	R	NR			
							Stopped drilling at 16 feet. No groundwater observed. NR=No Recovery
20							20
25							25

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 12

Elevation: Not Determined Sta: Date: 2-21-11

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		8	R	104	2	SW/ SM	Sand, Some Gravel, Trace to Some Silt; brown, slightly damp, medium dense, non-plastic to low plasticity fines, interbedded with silty sand, some gravel and sand, some gravel, trace to some clay.
5		100/6"	R	NR			Granite Bedrock, mottled grey/brown, moderately soft to hard, hardness increasing with depth, decomposed to slightly weathered, weathering decreasing with depth.
		100/2"	R	NR			
10							
15							Stopped drilling at 15 feet. No groundwater observed. NR=No Recovery
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

APPENDIX B
LABORATORY ANALYSIS

R·A·M·M

LABORATORY TEST RESULTS

Date: 22-Mar-11

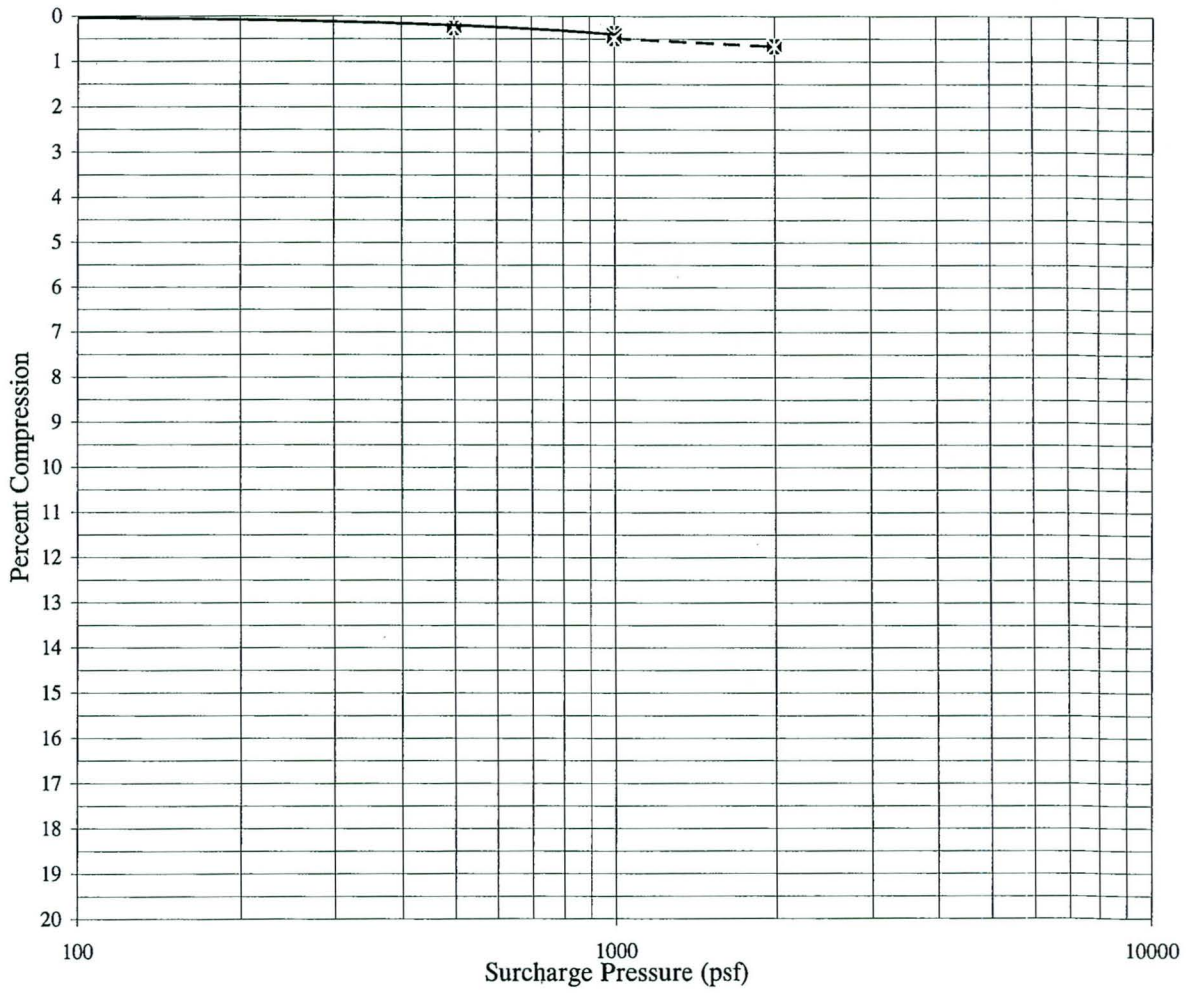
SAMPLE SOURCE: 10 @ 2'-3'

TESTING PERFORMED: Compression (ASTM D2435) - Driven Ring Sample

SAMPLED BY: RAMM/Miller

RESULTS:

Dry Density (pcf): 110 Moisture Content (%): 1



REMARKS: Sample submerged at 1000 psf.

LABORATORY TEST RESULTS

Date: 22-Mar-11

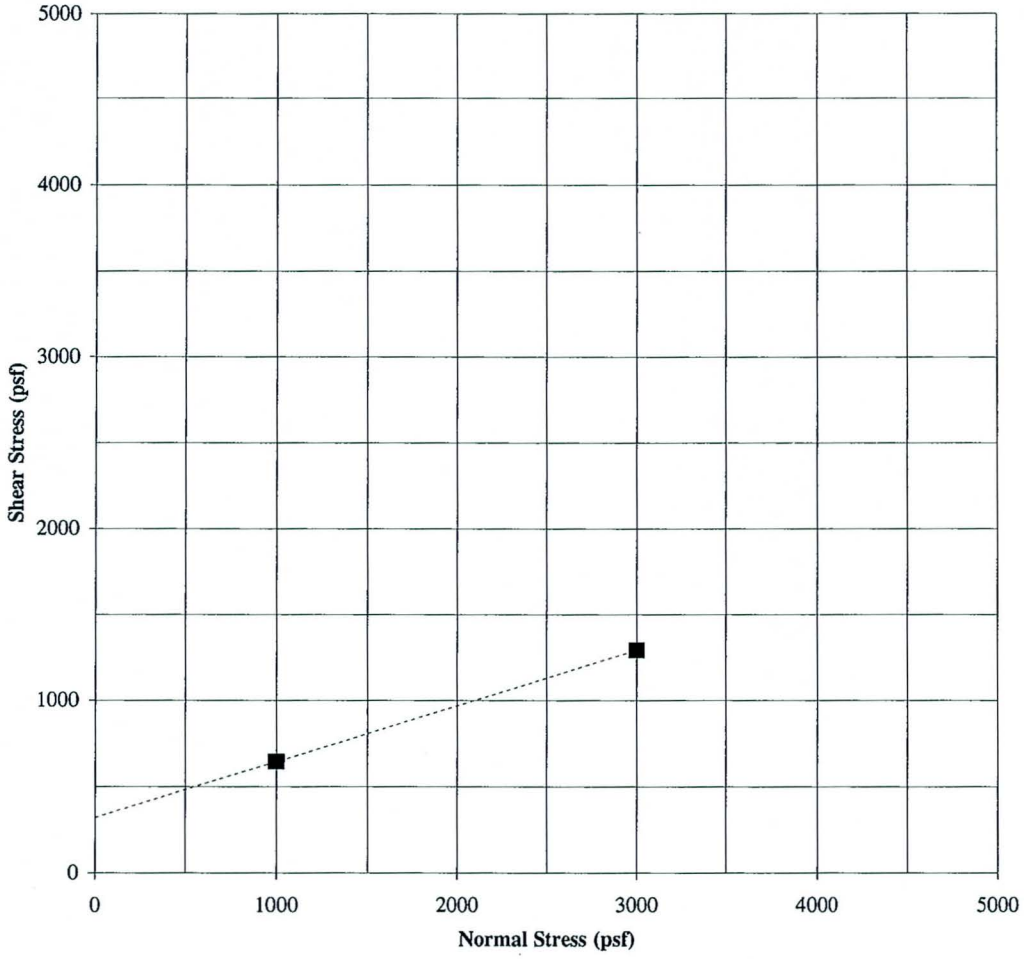
SAMPLE SOURCE: 11 @ 5'-6'

TESTING PERFORMED: Direct Shear (ASTM D3080) - Driven Ring Sample

SAMPLED BY: RAMM/Miller

RESULTS:

Dry Density (pcf):	108	Moisture Content (%):	2
Cohesion (psf):	300	Friction Angle (phi):	18



REMARKS: Samples submerged prior to testing.

LABORATORY TEST RESULTS

Date: 22-Mar-11

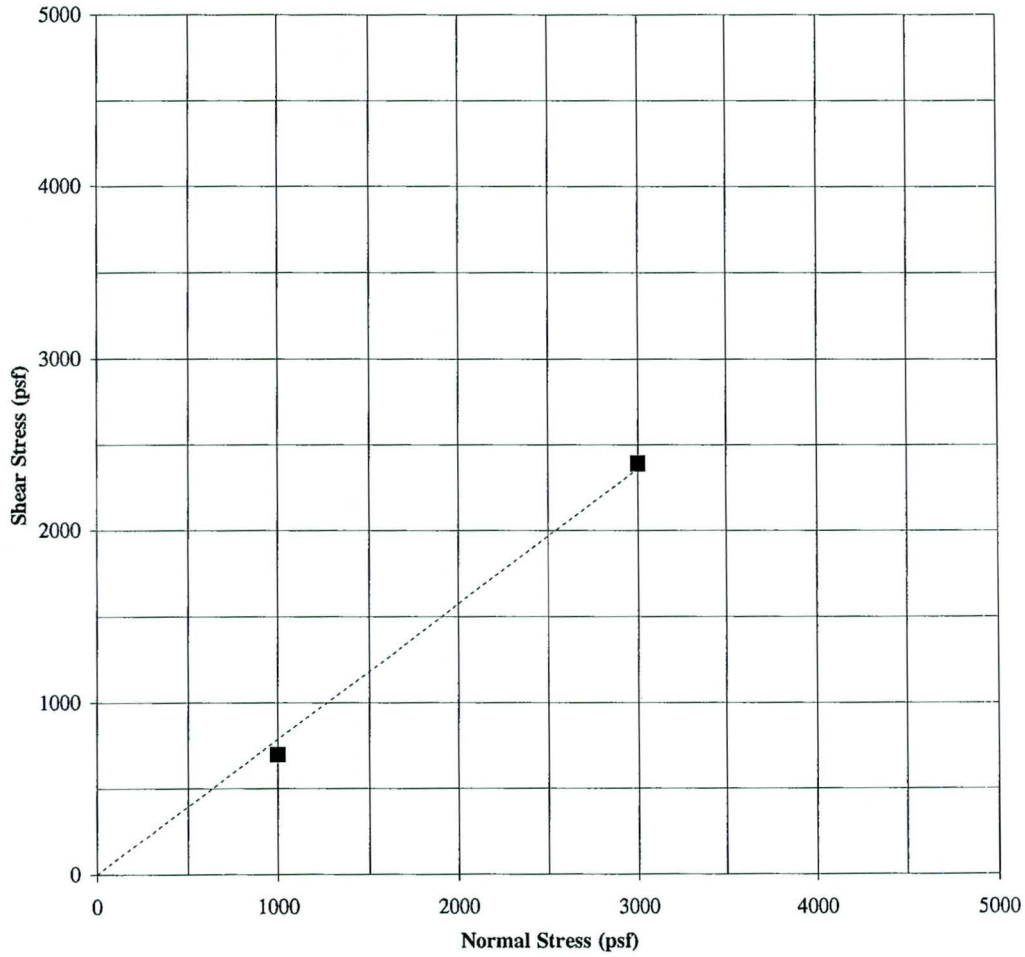
SAMPLE SOURCE: 12 @ 2'-3'

TESTING PERFORMED: Direct Shear (ASTM D3080) - Driven Ring Sample

SAMPLED BY: RAMM/Miller

RESULTS:

Dry Density (pcf):	104	Moisture Content (%):	2
Cohesion (psf):	0	Friction Angle (phi):	40



REMARKS: Samples submerged prior to testing.

LABORATORY TEST RESULTS

Date: 22-Mar-11

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Sieve Analysis, Percent Passing No. 200 Sieve, Atterberg Limits (ASTM C136, D1140, D4318)

SAMPLED BY: RAMM/Miller

RESULTS:

Sample Source	Atterberg Limits		Sieve Size - Accumulative Percent Passing											Soil Class.*
	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	
10 @ 0'-5'	N/A	NP	5.7	8	12	19	30	49	73	100				SW-SM
11 @ 5'-10'	26	4	12	15	18	22	29	46	69	97	100			SC-SM
12 @ 0'-5'	24	3	19	24	29	36	47	65	81	100				SM

NP = Non-Plastic

* Unified Soil Classification System

APPENDIX C
CALCULATIONS

R·A·M·M

Oak Street Basin — G17544
Berm Evaluation — 16 Mar 2011
per USACOE — ICER
EM 1110-2-1913

(90% Prelim) Project Plans titled "Oak Street Basin and Storm Drain Project PCN 420.04.31", prepared by Gavan & Barker, Inc., dated 12/10
Basin Outlet Pipe, plan & profile, sheet 27/39
Basin Grading Plan, sheets 39 thru 42

Use general cross-section based on Sheet 27 & 39.
Landside slope @ 5:1, Basin side slope @ 4:1 & 2:1
Base width berm 125' ±; Top width berm = 12' ±
Basin Elev = 1800; Top of Berm Elev = 1816
Land side toe of slope elev = 1806 = natural ground
Δ elev = say 10'; Ground elevations daylight 1808
Average Ground Elev = 1807

Soils: Berm Fill using on-site borrow
Use TB-6 @ 0-5 Max γ = 132.1 @ 8.3%
1. 95% @ -2% OMC = 125 @ 6
2. Native:
3. Granite

Use TB-11 for profile (thickest soil strata; @ RCP)

Oak Street Basin Berm Evaluation G-17544

17 Mar 11

Berm Fill @ 95% = 125 pcf γ_{dry} 1<E

W_s = 125 lbs For Vol = 1 Ft³ sg G_s = 2.65

$$V_v = 1\,Ft^3 - \frac{125\,lbs}{(2.65)(62.4\,pcf)} = 0.24\,ft^3$$

$$\gamma_{sat} = \frac{125\,lbs + (0.24\,ft^3)(62.4\,pcf)}{1\,Ft^3} = 140\,pcF$$

Native 108 pcf

$$V_v = 1\,Ft^3 - \frac{108\,lbs}{(2.65)(62.4\,pcf)} = 0.35\,Ft3$$

$$\gamma_{sat} = \frac{108\,lbs + (0,35\,Ft3)(62.4\,pcf)}{1\,Ft^3} = 130\,pcF$$

Oak Street Basin
Berm Evaluation

G17544
17 Mar 11
LVEA

Settlement Analysis

Ref Compression Results 10e2-3

$$C'_c = \frac{\Delta \varepsilon}{\log P_2/P_1} = \frac{0.01\delta_0 - 0.006\delta_0}{10{,}000/1{,}000} = 0.004$$

$\gamma_{\text{moist in-situ}} \cong 110$ pcf For $H = 5'$ say $p_o' = 550$ psf

$\gamma_{\text{berm moist}} = 135$ psf for $H = 10'$ $\Delta p = 1350$

$$\Delta H = C'_c H \log \frac{p_o' + \Delta p}{p_o'}$$

$$= (0.004)(5\text{ft} \cdot {}^{12\text{in}}/_{\text{ft}}) \log \frac{(550^{\text{psf}} + 1350)^{\text{psf}}}{550_{\text{psf}}}$$

$= 0.13$ inches $= {}^{1}/_{8}$ inch

Soils are granular. Most of settlement will occur during construction

Correct for 10' of soil @ TB-11, $H = 10'$, $p_o = 1110$ psf

$$\Delta H = (0.004)(10\text{ft} \cdot {}^{12\text{in}}/_{\text{ft}}) \log \frac{(1110_{\text{psf}} + 1350_{\text{psf}})}{1110\text{ psf}}$$

$\Delta H = 0.17 \cong {}^{3}/_{16}$ inch

LABORATORY TEST RESULTS

Date: 27-Feb-11

SAMPLE SOURCE: 10 @ 2'-3'

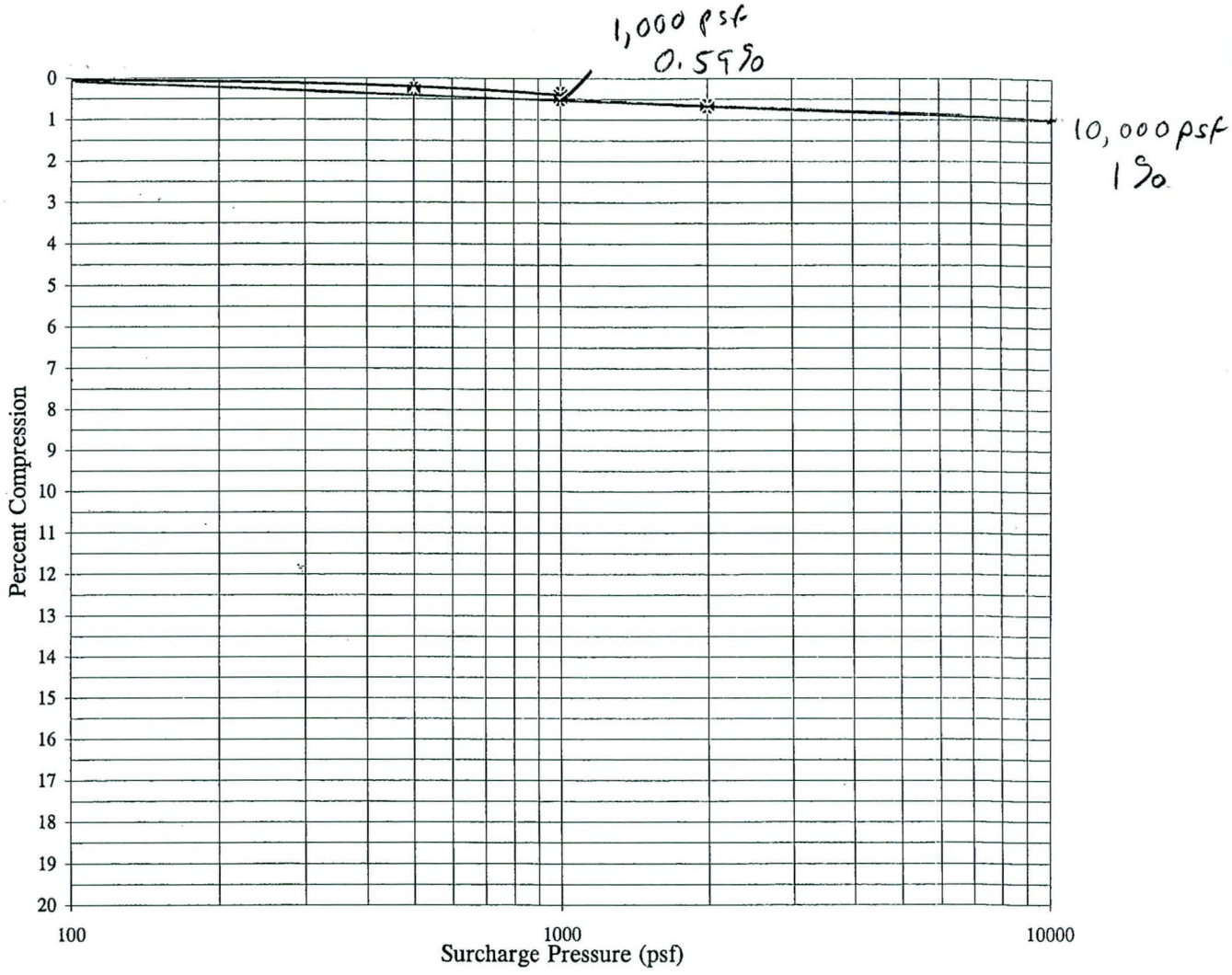
TESTING PERFORMED: Compression (ASTM D2435) - Driven Ring Sample

SAMPLED BY: RAMM/Miller

Settlement Calcs 17 Mar 11 JGER

RESULTS:

Dry Density (pcf): 110 Moisture Content (%): 0



REMARKS: Sample submerged at 1000 psf.

Oak Street [illegible]
Generalized Profile

Soil/Rock:

Direct Shear @ Sat

△1 125pcf @ 6% γ_{moist} = 133pcf; ϕ = 25°; C = 0psf (Berm Fill)

△2 108pcf @ 2% γ_{moist} = 110pcf; ϕ = 22°, C = 300 psf (Native)

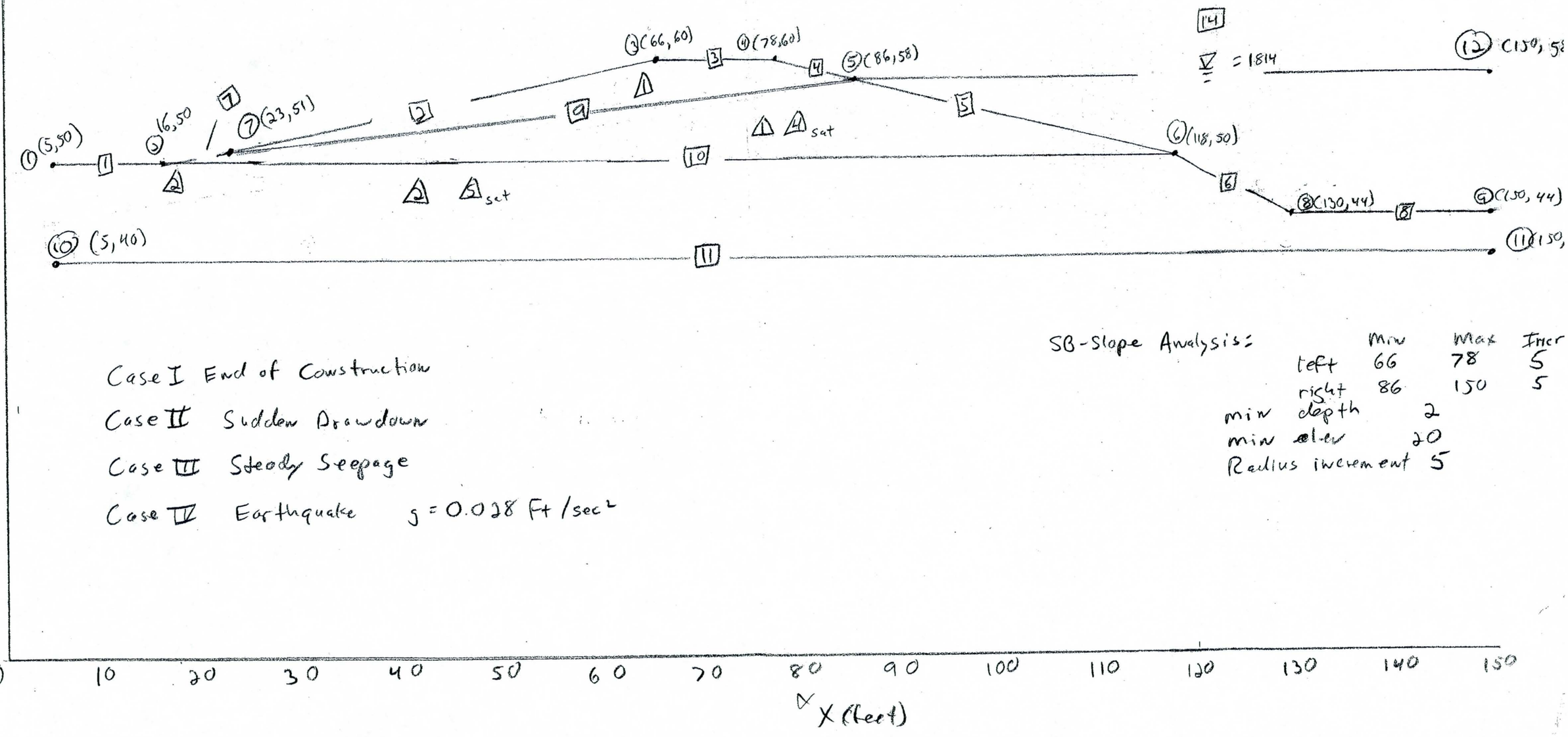
△3 γ = 165pcf; ϕ = 25°; C = 0psf (Granite Bedrock)

△4 γ_{sat} = 140pcf; ϕ = 20°, C = 0

△5 γ_{sat} = 130pcf; ϕ = 18°, C = 300psf

γ_{sat} Granite Bedrock no change

G17544
16 Mar 11
ICER



Case I End of Construction

Case II Sudden Drawdown

Case III Steady Seepage

Case IV Earthquake g = 0.028 Ft/sec²

SB-Slope Analysis:

	Min	Max	Incr
left	66	78	5
right	86	150	5

min depth 2
min elev 20
Radius increment 5

End of Construction

Range search; initial parameters:

	min	max	increment
left x	66.0	78.0	5.0
right x	86.0	150.0	5.0

radius increment is 5.0
minimum perpendicular depth is 2.0
limit at elevation 20.0

MIN FOR ENDPOINTS: x = 76.8, y = 66.6, r = 12.6, FS = 7.743
MIN FOR ENDPOINTS: x = 80.2, y = 71.7, r = 18.4, FS = 4.589
MIN FOR ENDPOINTS: x = 84.0, y = 78.0, r = 25.5, FS = 3.520
MIN FOR ENDPOINTS: x = 88.2, y = 85.7, r = 34.0, FS = 3.007
MIN FOR ENDPOINTS: x = 92.7, y = 94.6, r = 43.7, FS = 2.719
MIN FOR ENDPOINTS: x = 97.5, y = 104.9, r = 54.8, FS = 2.534
MIN FOR ENDPOINTS: x = 102.6, y = 116.4, r = 67.3, FS = 3.382
MIN FOR ENDPOINTS: x = 109.1, y = 129.1, r = 81.4, FS = 3.368
MIN FOR ENDPOINTS: x = 105.6, y = 94.3, r = 52.4, FS = 3.069
MIN FOR ENDPOINTS: x = 105.6, y = 81.0, r = 44.8, FS = 2.792
MIN FOR ENDPOINTS: x = 108.9, y = 86.6, r = 50.5, FS = 2.641
MIN FOR ENDPOINTS: x = 112.2, y = 92.6, r = 56.5, FS = 2.579
MIN FOR ENDPOINTS: x = 115.4, y = 99.0, r = 63.0, FS = 2.569
MIN FOR ENDPOINTS: x = 78.9, y = 62.2, r = 8.2, FS = 6.422
MIN FOR ENDPOINTS: x = 82.2, y = 66.0, r = 12.8, FS = 3.864
MIN FOR ENDPOINTS: x = 85.9, y = 71.2, r = 18.6, FS = 3.031
MIN FOR ENDPOINTS: x = 89.9, y = 77.6, r = 25.8, FS = 2.645
MIN FOR ENDPOINTS: x = 94.3, y = 85.3, r = 34.3, FS = 2.430
MIN FOR ENDPOINTS: x = 98.9, y = 94.3, r = 44.2, FS = 2.297
MIN FOR ENDPOINTS: x = 103.9, y = 104.5, r = 55.4, FS = 3.203
MIN FOR ENDPOINTS: x = 110.2, y = 115.9, r = 68.3, FS = 3.243
MIN FOR ENDPOINTS: x = 117.7, y = 128.6, r = 83.0, FS = 2.991
MIN FOR ENDPOINTS: x = 107.3, y = 75.8, r = 39.6, FS = 2.752
MIN FOR ENDPOINTS: x = 110.6, y = 81.0, r = 44.8, FS = 2.591
MIN FOR ENDPOINTS: x = 113.9, y = 86.6, r = 50.5, FS = 2.532
MIN FOR ENDPOINTS: x = 117.2, y = 92.6, r = 56.5, FS = 2.530
MIN FOR ENDPOINTS: x = 84.2, y = 61.7, r = 8.4, FS = 3.669
MIN FOR ENDPOINTS: x = 87.8, y = 65.6, r = 13.0, FS = 2.803
MIN FOR ENDPOINTS: x = 91.6, y = 70.7, r = 19.0, FS = 2.443
MIN FOR ENDPOINTS: x = 95.8, y = 77.2, r = 26.2, FS = 2.259
MIN FOR ENDPOINTS: x = 100.3, y = 84.9, r = 34.8, FS = 2.153
MIN FOR ENDPOINTS: x = 105.2, y = 93.9, r = 44.8, FS = 3.162
MIN FOR ENDPOINTS: x = 111.2, y = 104.1, r = 56.4, FS = 3.241
MIN FOR ENDPOINTS: x = 118.5, y = 115.5, r = 69.9, FS = 2.988
MIN FOR ENDPOINTS: x = 113.6, y = 86.6, r = 46.0, FS = 2.746
MIN FOR ENDPOINTS: x = 112.3, y = 75.8, r = 39.6, FS = 2.598
MIN FOR ENDPOINTS: x = 115.6, y = 81.0, r = 44.8, FS = 2.538
MIN FOR ENDPOINTS: x = 118.9, y = 86.6, r = 50.5, FS = 2.545
OVERALL MINIMUM: x = 100.3, y = 84.9, r = 34.8, FS = 2.153

```
Sudden Drawdown

Range search;  initial parameters:
             min      max  increment
 left x     66.0    78.0      5.0
 right x    86.0   150.0      5.0
 radius increment is    5.0
 minimum perpendicular depth is    2.0
 limit at elevation  20.0
MIN FOR ENDPOINTS: x =  76.8, y =  66.6, r =  12.6, FS = 6.567
MIN FOR ENDPOINTS: x =  80.2, y =  71.7, r =  18.4, FS = 3.822
MIN FOR ENDPOINTS: x =  84.0, y =  78.0, r =  25.5, FS = 2.886
MIN FOR ENDPOINTS: x =  88.2, y =  85.7, r =  34.0, FS = 2.440
MIN FOR ENDPOINTS: x =  92.7, y =  94.6, r =  43.7, FS = 2.191
MIN FOR ENDPOINTS: x =  97.5, y = 104.9, r =  54.8, FS = 2.032
MIN FOR ENDPOINTS: x = 102.6, y = 116.4, r =  67.3, FS = 2.922
MIN FOR ENDPOINTS: x = 109.1, y = 129.1, r =  81.4, FS = 2.953
MIN FOR ENDPOINTS: x = 117.0, y = 143.0, r =  97.4, FS = 2.697
MIN FOR ENDPOINTS: x = 110.7, y = 101.5, r =  61.0, FS = 2.474
MIN FOR ENDPOINTS: x = 114.2, y = 110.0, r =  69.5, FS = 2.457
MIN FOR ENDPOINTS: x = 112.2, y =  92.6, r =  56.5, FS = 2.471
MIN FOR ENDPOINTS: x = 115.4, y =  99.0, r =  63.0, FS = 2.448
MIN FOR ENDPOINTS: x =  78.9, y =  62.2, r =   8.2, FS = 5.394
MIN FOR ENDPOINTS: x =  82.2, y =  66.0, r =  12.8, FS = 3.194
MIN FOR ENDPOINTS: x =  85.9, y =  71.2, r =  18.6, FS = 2.468
MIN FOR ENDPOINTS: x =  89.9, y =  77.6, r =  25.8, FS = 2.134
MIN FOR ENDPOINTS: x =  94.3, y =  85.3, r =  34.3, FS = 1.948
MIN FOR ENDPOINTS: x =  98.9, y =  94.3, r =  44.2, FS = 1.833
MIN FOR ENDPOINTS: x = 103.9, y = 104.5, r =  55.4, FS = 2.769
MIN FOR ENDPOINTS: x = 110.2, y = 115.9, r =  68.3, FS = 2.847
MIN FOR ENDPOINTS: x = 117.7, y = 128.6, r =  83.0, FS = 2.618
MIN FOR ENDPOINTS: x = 112.1, y =  93.7, r =  53.2, FS = 2.410
MIN FOR ENDPOINTS: x = 115.7, y = 101.5, r =  61.0, FS = 2.398
MIN FOR ENDPOINTS: x = 113.9, y =  86.6, r =  50.5, FS = 2.418
MIN FOR ENDPOINTS: x = 117.2, y =  92.6, r =  56.5, FS = 2.402
MIN FOR ENDPOINTS: x =  84.2, y =  61.7, r =   8.4, FS = 3.007
MIN FOR ENDPOINTS: x =  87.8, y =  65.6, r =  13.0, FS = 2.263
MIN FOR ENDPOINTS: x =  91.6, y =  70.7, r =  19.0, FS = 1.956
MIN FOR ENDPOINTS: x =  95.8, y =  77.2, r =  26.2, FS = 1.800
MIN FOR ENDPOINTS: x = 100.3, y =  84.9, r =  34.8, FS = 1.708
MIN FOR ENDPOINTS: x = 105.2, y =  93.9, r =  44.8, FS = 2.735
MIN FOR ENDPOINTS: x = 111.2, y = 104.1, r =  56.4, FS = 2.849
MIN FOR ENDPOINTS: x = 118.5, y = 115.5, r =  69.9, FS = 2.617
MIN FOR ENDPOINTS: x = 113.6, y =  86.6, r =  46.0, FS = 2.399
MIN FOR ENDPOINTS: x = 117.1, y =  93.7, r =  53.2, FS = 2.391
MIN FOR ENDPOINTS: x = 115.6, y =  81.0, r =  44.8, FS = 2.415
MIN FOR ENDPOINTS: x = 118.9, y =  86.6, r =  50.5, FS = 2.408
OVERALL MINIMUM: x = 100.3, y =  84.9, r =  34.8, FS = 1.708
```

```
Steady Seepage at Flood Stage

Range search;  initial parameters:
            min      max  increment
 left x    66.0    78.0      5.0
 right x   86.0   150.0      5.0
 radius increment is   5.0
 minimum perpendicular depth is   2.0
 limit at elevation  20.0
MIN FOR ENDPOINTS: x =  76.8, y =  66.6, r =  12.6, FS = 5.347
MIN FOR ENDPOINTS: x =  80.2, y =  71.7, r =  18.4, FS = 3.139
MIN FOR ENDPOINTS: x =  84.0, y =  78.0, r =  25.5, FS = 2.493
MIN FOR ENDPOINTS: x =  88.2, y =  85.7, r =  34.0, FS = 2.190
MIN FOR ENDPOINTS: x =  92.7, y =  94.6, r =  43.7, FS = 2.021
MIN FOR ENDPOINTS: x =  97.5, y = 104.9, r =  54.8, FS = 1.910
MIN FOR ENDPOINTS: x = 102.6, y = 116.4, r =  67.3, FS = 3.388
MIN FOR ENDPOINTS: x = 100.6, y =  88.0, r =  44.5, FS = 3.582
MIN FOR ENDPOINTS: x = 101.5, y =  76.5, r =  39.1, FS = 3.157
MIN FOR ENDPOINTS: x = 105.6, y =  81.0, r =  44.8, FS = 2.822
MIN FOR ENDPOINTS: x = 108.9, y =  86.6, r =  50.5, FS = 2.689
MIN FOR ENDPOINTS: x = 112.2, y =  92.6, r =  56.5, FS = 2.637
MIN FOR ENDPOINTS: x = 115.4, y =  99.0, r =  63.0, FS = 2.638
MIN FOR ENDPOINTS: x =  78.9, y =  62.2, r =   8.2, FS = 4.231
MIN FOR ENDPOINTS: x =  82.2, y =  66.0, r =  12.8, FS = 2.545
MIN FOR ENDPOINTS: x =  85.9, y =  71.2, r =  18.6, FS = 2.088
MIN FOR ENDPOINTS: x =  89.9, y =  77.6, r =  25.8, FS = 1.887
MIN FOR ENDPOINTS: x =  94.3, y =  85.3, r =  34.3, FS = 1.776
MIN FOR ENDPOINTS: x =  98.9, y =  94.3, r =  44.2, FS = 1.709
MIN FOR ENDPOINTS: x = 103.9, y = 104.5, r =  55.4, FS = 3.269
MIN FOR ENDPOINTS: x = 110.2, y = 115.9, r =  68.3, FS = 3.537
MIN FOR ENDPOINTS: x = 103.3, y =  71.7, r =  34.3, FS = 3.178
MIN FOR ENDPOINTS: x = 107.3, y =  75.8, r =  39.6, FS = 2.813
MIN FOR ENDPOINTS: x = 110.6, y =  81.0, r =  44.8, FS = 2.667
MIN FOR ENDPOINTS: x = 113.9, y =  86.6, r =  50.5, FS = 2.617
MIN FOR ENDPOINTS: x = 117.2, y =  92.6, r =  56.5, FS = 2.622
MIN FOR ENDPOINTS: x =  84.2, y =  61.7, r =   8.4, FS = 2.338
MIN FOR ENDPOINTS: x =  87.8, y =  65.6, r =  13.0, FS = 1.907
MIN FOR ENDPOINTS: x =  91.6, y =  70.7, r =  19.0, FS = 1.735
MIN FOR ENDPOINTS: x =  95.8, y =  77.2, r =  26.2, FS = 1.651
MIN FOR ENDPOINTS: x = 100.3, y =  84.9, r =  34.8, FS = 1.603
MIN FOR ENDPOINTS: x = 105.2, y =  93.9, r =  44.8, FS = 3.367
MIN FOR ENDPOINTS: x = 111.2, y = 104.1, r =  56.4, FS = 3.687
MIN FOR ENDPOINTS: x = 105.0, y =  67.4, r =  30.0, FS = 3.316
MIN FOR ENDPOINTS: x = 109.0, y =  71.1, r =  34.8, FS = 2.893
MIN FOR ENDPOINTS: x = 112.3, y =  75.8, r =  39.6, FS = 2.727
MIN FOR ENDPOINTS: x = 115.6, y =  81.0, r =  44.8, FS = 2.675
MIN FOR ENDPOINTS: x = 118.9, y =  86.6, r =  50.5, FS = 2.692
OVERALL MINIMUM: x = 100.3, y =  84.9, r =  34.8, FS = 1.603
```

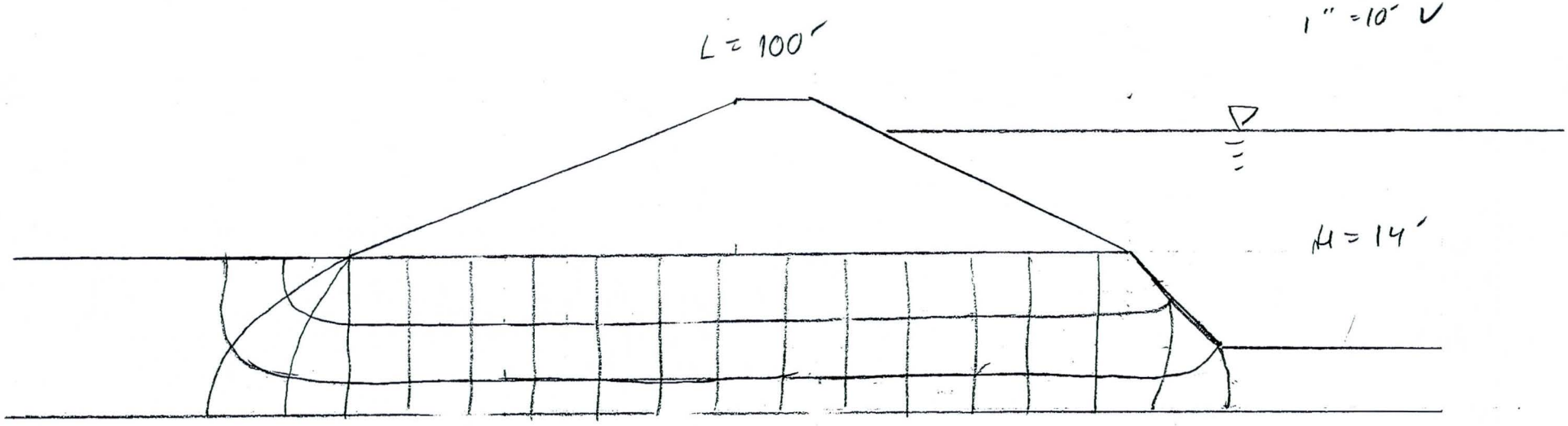
```
Earthquake

Range search;  initial parameters:
            min      max  increment
 left x    66.0    78.0      5.0
 right x   86.0   150.0      5.0
 radius increment is   5.0
 minimum perpendicular depth is   2.0
 limit at elevation  20.0
MIN FOR ENDPOINTS: x =  76.8, y =  66.6, r =  12.6, FS = 5.887
MIN FOR ENDPOINTS: x =  80.2, y =  71.7, r =  18.4, FS = 3.780
MIN FOR ENDPOINTS: x =  84.0, y =  78.0, r =  25.5, FS = 2.988
MIN FOR ENDPOINTS: x =  88.2, y =  85.7, r =  34.0, FS = 2.591
MIN FOR ENDPOINTS: x =  92.7, y =  94.6, r =  43.7, FS = 2.363
MIN FOR ENDPOINTS: x =  97.5, y = 104.9, r =  54.8, FS = 2.214
MIN FOR ENDPOINTS: x = 102.6, y = 116.4, r =  67.3, FS = 2.977
MIN FOR ENDPOINTS: x = 109.1, y = 129.1, r =  81.4, FS = 2.988
MIN FOR ENDPOINTS: x = 105.6, y =  94.3, r =  52.4, FS = 2.758
MIN FOR ENDPOINTS: x = 105.6, y =  81.0, r =  44.8, FS = 2.515
MIN FOR ENDPOINTS: x = 108.9, y =  86.6, r =  50.5, FS = 2.380
MIN FOR ENDPOINTS: x = 112.2, y =  92.6, r =  56.5, FS = 2.321
MIN FOR ENDPOINTS: x = 115.4, y =  99.0, r =  63.0, FS = 2.307
MIN FOR ENDPOINTS: x =  78.9, y =  62.2, r =   8.2, FS = 5.325
MIN FOR ENDPOINTS: x =  82.2, y =  66.0, r =  12.8, FS = 3.331
MIN FOR ENDPOINTS: x =  85.9, y =  71.2, r =  18.6, FS = 2.649
MIN FOR ENDPOINTS: x =  89.9, y =  77.6, r =  25.8, FS = 2.327
MIN FOR ENDPOINTS: x =  94.3, y =  85.3, r =  34.3, FS = 2.146
MIN FOR ENDPOINTS: x =  98.9, y =  94.3, r =  44.2, FS = 2.033
MIN FOR ENDPOINTS: x = 103.9, y = 104.5, r =  55.4, FS = 2.851
MIN FOR ENDPOINTS: x = 110.2, y = 115.9, r =  68.3, FS = 2.904
MIN FOR ENDPOINTS: x = 117.7, y = 128.6, r =  83.0, FS = 2.697
MIN FOR ENDPOINTS: x = 107.3, y =  75.8, r =  39.6, FS = 2.493
MIN FOR ENDPOINTS: x = 110.6, y =  81.0, r =  44.8, FS = 2.348
MIN FOR ENDPOINTS: x = 113.9, y =  86.6, r =  50.5, FS = 2.290
MIN FOR ENDPOINTS: x = 117.2, y =  92.6, r =  56.5, FS = 2.281
MIN FOR ENDPOINTS: x =  84.2, y =  61.7, r =   8.4, FS = 3.267
MIN FOR ENDPOINTS: x =  87.8, y =  65.6, r =  13.0, FS = 2.501
MIN FOR ENDPOINTS: x =  91.6, y =  70.7, r =  19.0, FS = 2.182
MIN FOR ENDPOINTS: x =  95.8, y =  77.2, r =  26.2, FS = 2.018
MIN FOR ENDPOINTS: x = 100.3, y =  84.9, r =  34.8, FS = 1.923
MIN FOR ENDPOINTS: x = 105.2, y =  93.9, r =  44.8, FS = 2.836
MIN FOR ENDPOINTS: x = 111.2, y = 104.1, r =  56.4, FS = 2.920
MIN FOR ENDPOINTS: x = 118.5, y = 115.5, r =  69.9, FS = 2.710
MIN FOR ENDPOINTS: x = 113.6, y =  86.6, r =  46.0, FS = 2.509
MIN FOR ENDPOINTS: x = 112.3, y =  75.8, r =  39.6, FS = 2.363
MIN FOR ENDPOINTS: x = 115.6, y =  81.0, r =  44.8, FS = 2.303
MIN FOR ENDPOINTS: x = 118.9, y =  86.6, r =  50.5, FS = 2.301
OVERALL MINIMUM: x = 100.3, y =  84.9, r =  34.8, FS = 1.923
```

Oak Street Basin
Seepage Analysis

G17544
17/18 Mar 11 1 of 2

1" = 20' H
1" = 10' V



N_f = No of Flow paths = 2

N_d = No of Equipotential drops = 17

$k = 80 \times 10^{-4}$ feet/sec EM pg 3-10 Fig 3-5

$$q = k \frac{N_f}{N_d} H_L L = (80 \times 10^{-4}\ ft/sec)\left(\frac{2}{17}\right)(14\ feet)(100\ Ft) = 1.3\ Ft^3/sec$$

i_E = exit gradient $= \frac{\Delta h_L}{L} = \frac{0.82\ Ft}{14\ Ft} = 0.21$ ok ✓ per EM Appendix B, B-14

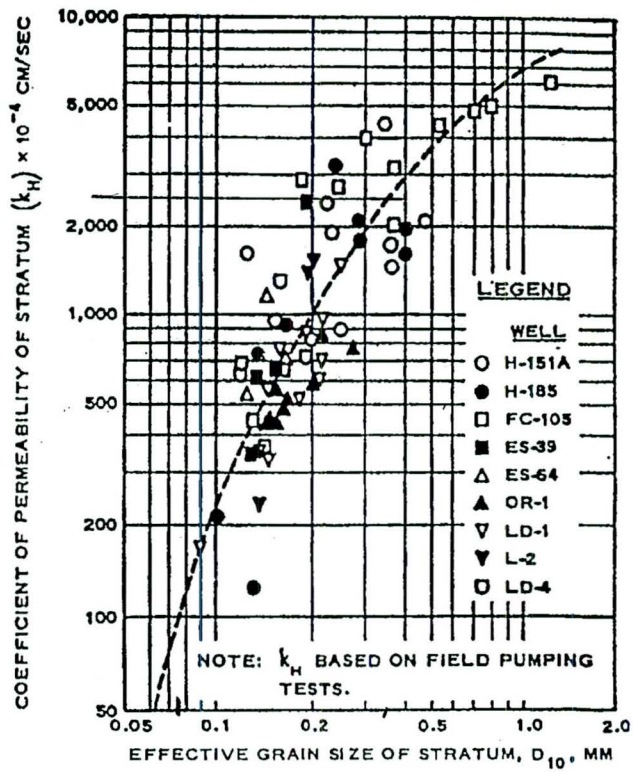
$i_E \leq 0.5$

$\Delta h_L = \frac{h_L}{N_d} = \frac{14\ Ft}{17} = 0.82\ Ft$

i_c = critical gradient $= \frac{\gamma_{sub}}{\gamma_{H_2O}} = \frac{\gamma_{sat} - \gamma_{H_2O}}{\gamma_{H_2O}} = \frac{130\ pcf - 62.4\ pcf}{62.4\ pcf} = 1.08$ ✓ ok > 1

G17544

1 > mar
1 < FER



b. Effective grain size, D_{10} versus coefficient of permeability, k_h (from WES TM No. 3-424, ref. A-1)

Figure 3-5. (Concluded)

K_H X10⁻

10 e 0 to 5' $D_{10} \cong$ #50 sieve = .35 mm 2500

11 e 5 to 10 $D_{10} \cong$ #200 sieve = 0.074 mm 100

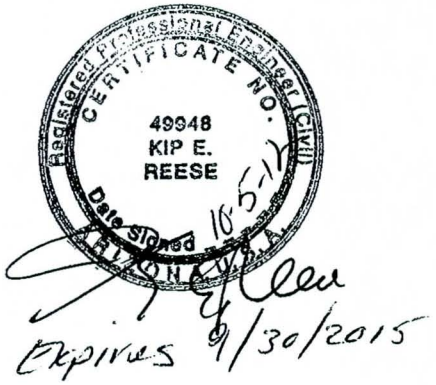
12 e 0-5 $D_{10} \cong$ 5? 0.016 mm

2500 X 10^{-4} cm/sec = 985 in/sec = 82' X 10^{-4}/sec

100 X 10^{-4} cm/sec = 40 inches X 10^{-4}/sec = 3 ft X 10^{-4}/sec

Supplement No. 3 – RCC Dam

Geotechnical Engineering Report
Oak Street Basin and Storm Drain Design
RCC Dam
Oak Street and Hawes Road
Maricopa County, Arizona
RAMM Project No. G17544 Supplement No. 3
FCD Contract No. 2009C021



For:
Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

◆◆◆

By:
Ricker-Atkinson-McBee-Morman & Associates, Inc.
2105 South Hardy Drive, Suite 13
Tempe, Arizona 85282

R·A·M·M

RICKER • ATKINSON • McBEE • MORMAN & ASSOCIATES, INC.

Geotechnical Engineering • Construction Materials Testing

Gavan & Barker, Inc.
3030 North Central Avenue, Suite 1530
Phoenix, Arizona 85012

October 5, 2012

Attention: Mark T. Gavan, P.E.

Subject: Geotechnical Engineering Report
Oak Street Basin and Storm Drain Design
RCC Dam
Oak Street and Hawes Road
Maricopa County, Arizona
FCD Contract No. 2009C021

RAMM Project No. G17544
Supplement No. 3

Attached to this letter is the Supplement No. 3 Geotechnical Engineering Report for the Oak Street Basin and Storm Drain Design-RCC Dam, to be located in Maricopa County, Arizona.

The proposed revised development will include construction of a roller compacted concrete (RCC) dam along the northwest, west, southwest and south sides of the proposed basin. The RCC dam replaces the original project design for an earthen berm. The RCC dam will be approximately 825 feet long, will be approximately 3 to 18 feet high, will be approximately 12 feet to 34 feet wide at the base and 8 feet wide at the top. A reinforced concrete inlet pipe (RCP) will be located in the northwest corner of the basin and a RCP outlet pipe will be located along the south side. The RCP pipes will be within the footprint of the RCC dam. An RCP inlet pipe will also be located in the southeast corner of the basin, outside the footprint of the RCC dam. The results of our field explorations; laboratory testing; and engineering analysis, evaluation and recommendations are presented in the report.

The attached report was prepared based on project and site data available at this time and was prepared in a manner and to the standards of local geotechnical engineering practice. Our services did not include evaluations for the presence of hazardous materials; for corrosion potential with respect to site soils; for corrosion potential and concrete durability with respect to site use water, or for area subsidence resulting from groundwater withdrawal or other geologic hazards.

If you have any questions, please do not hesitate to call. This report should be attached to and made a part of our original report presenting final recommendations for the basin and storm drains, dated July 26, 2010.

Respectfully submitted,
RICKER, ATKINSON, McBEE, MORMAN & ASSOCIATES, INC.

Expires – 9/30/2015

By: Kip E. Reese, P.E.

Expires – 3/31/2013

Reviewed By: Kenneth L. Ricker, P.E.

/dh
Copies to: Addressee (4) (mgavan@gavanbarker.com)

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REPORT

R·A·M·M

INTRODUCTION

This Supplement No. 3 report presents the results of our geotechnical engineering services for the Oak Street Basin and Storm Drain Design – RCC Dam, to be located in Maricopa County, Arizona. The scope of our services included attending a Failure Modes and Effects Analysis (FMEA) workshop, performing a field exploration program, laboratory analysis and geotechnical engineering evaluation, analysis and recommendations. The geotechnical recommendations presented herein include those for RCC design parameters, site development, and material use and requirements. We would be pleased to discuss with you any additional recommendations you may require. In addition, we are available to review project specifications and plans for conformance with our recommendations at no charge to you.

Preliminary recommendations for the basin were presented in a letter report dated March 10, 2010. Final recommendations for the basin and storm drains were presented in a report dated July 26, 2010. Foundation design recommendations for drop structures and pavement design for Oak Street were presented in our Supplement No. 1 letter, dated December 16, 2010. The results of our evaluation for a proposed earthen berm were presented in a report dated March 22, 2011.

This firm should be notified for additional evaluation and recommendations if the project design parameters (location, type, size, alignments, etc.), site use or conditions encountered during construction differ from those presented herein.

PROPOSED CONSTRUCTION

The proposed revised basin will include construction of a roller compacted concrete (RCC) dam along the northwest, west, southwest and south sides of the proposed basin. The RCC dam replaces the original project design for an earthen berm. The RCC dam will be approximately 825 feet long, will be approximately 3 to 18 feet high and will be approximately 12 feet to 34 feet wide at the base and 8 feet wide at the top. A reinforced concrete inlet pipe (RCP) will be located in the northwest corner of the basin and a RCP outlet pipe will be located along the south side. The final location of the outlet pipe had not been determined at the time of this report. The RCP inlet and outlet pipes will be within the footprint of the RCC dam. Another RCP inlet pipe will be located in the southeast corner of the basin, outside the footprint of the RCC dam. The base of the RCC dam will be founded at depths of approximately 4 to 12 feet below existing site grades. The RCC dam will have structural backfill placed along the upstream and downstream

faces of the RCC dam and extend from the base of the dam to basin bottom elevation or to near original ground elevation. Landscape fill will be placed adjacent to the downstream face of the RCC dam and on top of the downstream structural backfill. The proposed RCC dam alignment and centerline stationing and proposed inlet and outlet pipe locations are presented in a site plan titled Oak Street Basin RCC Dam Alignment, prepared by Gavan & Barker, Inc., undated. An alternate outlet pipe plan/sketch was also prepared by Gavan & Barker.

SITE CONDITIONS

The proposed basin site is located at the northeast corner Oak Street and Hawes Road, in Maricopa County, Arizona. Site conditions at the time of our field exploration for the RCC dam were relatively unchanged from our previous field exploration. The proposed RCC dam area was a gently rolling, undeveloped parcel which sloped gently downward to the southwest. Overall topographic relief in the RCC dam area was approximately 8 feet. The RCC dam alignment was crossed by a relatively shallow, wide wash near the southwest corner of the site. The site surface conditions in the RCC dam area consisted predominantly of granite-derived and stream deposit silty sand type soils. Granite outcrops were apparent at and near the east end of the south leg of the alignment and east of the east end of the north leg. Vegetation on-site consisted of a slight to moderate coverage of native grasses, shrubs, cacti and trees.

FIELD EXPLORATIONS

Subsurface conditions in the proposed RCC dam area, the RCP outlet pipe areas and in the RCP northwest basin inlet pipe area were explored. Continuous penetration testing (CPT) was performed at 19 locations (CPT 1 through 18 and CPT 22) along the RCC dam alignment and at four locations (CPT 19, CPT 20, CPT 21 and CPT 23) along the proposed outlet and inlet pipe alignments to depths of 1.1 to 8 feet; four test borings (R-1 through R-4) were drilled to depths of 10 to 27 feet (a test boring was not performed in Test Area 5), four test borings/coring (RC-2 through RC-5) were drilled/cored to depths of 20 to 30 feet (coring was not performed in Test Area 1); six test pits were excavated to depths of 1.9 to 12 feet and five test borings/percolation tests were drilled to depths of 5 to 25 feet at five test area locations along the proposed RCC dam alignment, as shown on the Site Plans in Appendix A. The CPT was performed using a 2-inch diameter rod with a tapered tip and a 140-pound hammer dropping 30 inches. The CPTs were performed and the test borings were drilled with a CME 75 drill rig using 7-inch diameter,

hollow-stem augers. The test borings/coring were drilled/cored with a Diedrich D-120 using 4-inch diameter, solid-stem auger, temporary surface casing and NX, 2 7/8" inside diameter, wire line rock coring equipment. The solid stem auger was used to advance the test borings to depths of 10 to 15 feet, temporary surface casing was installed and the NX rock coring equipment used for the remaining depths of exploration (20 to 30 feet). The test pits were excavated with a John Deere 310, four-wheel drive, rubber-tired backhoe equipped with a two-foot wide bucket. The front end loader of the backhoe was used to excavate RTP-6. The drilling and coring equipment, crew and backhoe operator were provided by D&S Drilling, Inc. The backhoe was provided by SunState Equipment Company. The RCC dam centerline was surveyed and staked by Gavan & Barker with station and elevation stakes/hubs at approximately 50-foot intervals. The CPT, test boring, test boring/coring, test pit and test boring/percolation test locations were determined in the field by a field technician from our firm and at the direction of a project engineer from our firm. During the field explorations, and at the direction of our field technician, representative disturbed and undisturbed samples were obtained, the CPT, test borings, test borings/coring, test pits and test boring/percolation tests logged and soils and bedrock field classified by our field technician. The relatively undisturbed samples (ring samples) and driven disturbed samples (standard penetration tests (SPT)) were obtained by driving a 3-inch diameter, ring-lined, open-end sampler and a 2-inch diameter, split spoon sampler, respectively, into the soil with a 140-pound hammer dropping 30 inches. Rock core and solid core were obtained using the NX-size double tube wire line coring equipment. Intact rock samples were obtained from RTP-4. The results of the field explorations are presented on the attached Test Boring Logs, Test Pit Profiles, CPT Profile and Core Photos in Appendix A.

The test borings/percolation tests were drilled with the CME 75 drill rig using 10-inch diameter, hollow-stem auger. Water was added to the test hole through 4-inch diameter, solid PVC pipe with a 10-foot long, perforated PVC pipe at the bottom. The PVC pipe was omitted at Test Boring/Percolation Test RP-1 (shallow depth). The percolation tests were performed using falling-head test procedures. The percolation test holes were filled to water heights of 4 to 12 feet and presoaked for 2 to 4 hours by refilling the test hole. After presoaking, the percolation test holes were filled to the initial height of water and the time for the water to fall was recorded at intervals of 5 to 30 minutes, depending on the rate of fall, until the height of the water was less than 1 to 2 feet. The holes were refilled and the process repeated until a relatively stabilized

percolation rate was obtained. After presoaking each percolation test hole to the initial water height presented below, the following ranges of unsaturated hydraulic conductivities (K) were calculated:

Location	Total Depth (feet)	Initial Water Height (feet)	K (cm/sec)
RP-1	5	4	3 to 5 X 10^{-4}
RP-2	20	12	1 to 2 X 10^{-3}
RP-3	25	8	1 to 2 X 10^{-3}
RP-4	14.5	4	2 to 5 X 10^{-4}
RP-5	15	4	1 X 10^{-3}

LABORATORY ANALYSIS

Representative samples obtained during the field explorations were subjected to the following tests in our laboratory.

Type of Test	Type of Sample	Number of Samples Tested
Direct Shear	Remolded	3
Sieve Analysis, Percent Passing No. 200 Sieve & Atterberg Limits	Representative	3
Maximum Density-Optimum Moisture Determination	Representative	4
Unit Weight & Unconfined Compressive Strength	Rock and Solid Cores	3/2**
Unit Weight & Unconfined Compressive Strength	Rock Hand Sample	1***
Hydraulic Conductivity	Solid Cores	2**
Hydraulic Conductivity	Remolded	2**
Moisture Content/Dry Density *	Undisturbed	6
Moisture Content*	Split Spoon	21
Alkali-Silica Reaction	Representative	2**

* Reported in the test boring logs.

** Performed by AMEC. Due to undersized diameter of solid core specimens unconfined compressive strength tests were performed instead of direct shear tests.

*** Test specimen sawcut from sample.

The results of the laboratory analysis are presented in Appendix B.

SUBSURFACE CONDITIONS

Proposed RCC Dam Alignment:

The results of the continuous penetration tests, test borings, test borings/coring and test pits are presented in Appendix A in the Test Boring Logs, Test Pit Profiles, CPT Profile and Core Photos. In general, the surface and near surface soils encountered in the field exploration, extending to depths of 4.5 to 30 feet, consisted of silty sand containing some gravel. These soils were medium dense to dense, had non-plastic to low plasticity fines, had moderate to heavy cementation below depths of 2 to 5 feet and had occasional zones of light to moderate cementation below depths of 18 feet. Sand to silty sand containing some gravel was encountered at the surface in the Test Area 3 test boring/test pit locations and extended to depths of 5 to 10 feet. These soils were loose to medium dense, had non-plastic to low plasticity fines, were interbedded with silty sand and were underlain by moderate to heavy cemented silty sand soils. Granite bedrock was encountered at the surface and to depths of 4.5 feet in the Test Area 4 test boring/test pit locations. The granite bedrock was decomposed to slightly weathered, weathering decreasing with depth, was moderately soft to hard, hardness increasing with depth and had moderate to widely spaced, smooth to slightly rough, clean joints measuring 50 to 70 degrees from the horizontal. The maximum depths of our exploration were 1.9 to 30 feet. A generalized subsurface profile along the centerline of the RCC dam alignment was developed based on the results of our field exploration and is presented in Appendix A. The soil moisture contents were described as slightly damp. No groundwater was observed in the test borings and test pits.

Revised Test Boring and Test Pit Logs:

The results of our RCC Dam field exploration, the proximity of the RCC dam alignment to some of our previous test borings and test pits and re-evaluation of the results of previous test borings and test pits indicate that material previously described as granite should be revised to describe the silty sand soils with moderate to heavy cementation. The revised test boring and test pit logs are attached in Appendix C.

DISCUSSION OF TEST RESULTS

Remolded samples of the near surface soils from the site exhibited moderate to high shear strength when tested in direct shear under submerged conditions. Representative samples were non-reactive when tested for alkali-silica reactivity. A summary of direct shear, maximum

density-optimum moisture determination, unit weight, unconfined compressive strength and hydraulic conductivity (laboratory and field) test results is presented in Appendix B.

ANALYSIS AND RECOMMENDATIONS

Engineering Analysis:

Engineering analyses were performed to provide design parameters for the RCC dam. The RCC dam design will be provided by AMEC. The RCC dam will be approximately 3 to 18 feet high and approximately 12 feet to 34 feet wide at the base and will be 8 feet wide at the top. The RCC dam will be founded on moderately to heavily cemented silty sand soils (SMC) and on granite bedrock (Granite) at depths of approximately 4 to 12 feet below existing site grades, based on the results of our field exploration, laboratory testing, discussions with AMEC and preliminary foundation elevations provided by AMEC.

Design parameters, presented below, are based on field and laboratory test results, calculations and estimation.

Property	Foundation		Backfill	In-Situ
	SMC	Granite	SMB	SM
γ	120	155	114	105
MC	2	NA	10	2
SG	2.65	2.65	2.65	2.65
ε	0.38	NA	0.45	0.57
Φ	45	50	38	25
C	10	25	0	0
k	350	350	NA	NA
μ	0.45	0.60	NA	NA
K	$1X10^{-3}$	$1X10^{-3}$	$1X10^{-4}$	$1X10^{-2}$

γ = dry density (pcf); MC = moisture content (%); SG = specific gravity; ε = void ratio; Φ = friction angle (Phi, degrees); C = cohesion (ksf); k = modulus of subgrade reaction (pci); μ = friction coefficient, concrete; K = permeability (cm/sec); SMC = Silty Sand Cemented; SMB = Silty Sand Blend (Proctor: 95%, -2% OMC); SM = Silty Sand, un-cemented (Use foundation SMC values for in-situ)

Foundation Recommendations:

The proposed RCC dam should be founded on moderately to heavily cemented silty sand soils and undisturbed moderately to slightly weathered granite bedrock. The dam thus founded and based on the proposed RCC dam profile may be designed using an allowable bearing pressure of

10 ksf provided the depth of embedment is at least 2 feet below finish grade. Finished grade is defined as the lowest adjacent finished grade within 5 feet of the perimeter of the structure. All footing excavations should be reviewed by a representative of the RCC dam design engineer prior to placing concrete. Any loose, disturbed or unstable soils should be removed from the bearing surface and replaced as otherwise directed by the RCC dam design engineer.

The allowable bearing capacity should be applied to maximum, design dead plus live loads and may be increased by one-third when considering temporary loads such as transient wind or seismic loads. A one-third increase may also be used for toe pressures due to eccentric or lateral loadings, assuming the entire footing bearing surface remains in compression. The weight of the footing concrete below grade may be neglected in dead load computations. A Site Class designation of B should be used for the site per the 2006 and 2009 International Building Code (IBC). The soil profile and site class designations are based on the general geology of the area in which predominantly granite bedrock and/or moderately to heavily cemented soils are known to exist to depths over 100 feet in the immediate vicinity of the site.

The estimated total and differential footing settlements, for anticipated loading conditions and anticipated short-term presence of water at or near footing level, are less than ¼ inch for foundations bearing on moderately to heavily cemented silty sand soils and undisturbed moderately to slightly weathered granite bedrock.

SITE DEVELOPMENT RECOMMENDATIONS

Excavatability:

The excavatability of site materials is difficult to evaluate based only on the exploration equipment used during this design report. Therefore, we recommend that the contractor evaluate the excavatability of site materials by performing test excavations with the size and type of equipment the contractor plans on using at the site. For design purposes the following paragraph presents our best analysis as to the excavatability of site soils.

The near surface soils and underlying decomposed to highly weathered granite bedrock to depths of 2 to 10 feet can probably be removed with conventional excavating equipment. Excavations will likely be slower and more difficult to accomplish with depth due the presence of moderate to heavy cemented soils and increasing hardness of the granite bedrock and may require heavy

excavating equipment and/or rock excavation techniques. Caving should be expected in the surface and near surface silty sand and sand to silty sand soils. OSHA requires all excavations over five feet in depth, in which personnel are to enter, be either braced or sloped in accordance with OSHA regulations. Excavations in the moderate to heavy cemented soils and decomposed to highly weathered granite bedrock will likely generate gravel-sized material and smaller. Excavations in the moderate to heavy cemented soils and moderately weathered granite bedrock may generate gravel- and cobble-sized material depending on the excavation methods employed. Excavations in the slightly weathered to unweathered granite bedrock may generate cobble- and boulder-sized material, depending on the excavation methods employed.

Construction Excavation and Temporary Construction Cut Slopes:

The proposed storm drain within the berm footprint will extend approximately 8 to 10 feet below existing site grades. The following criteria are presented to aid in development of excavation plans.

1. Dewatering should not be required.
2. Unbraced temporary slopes in the sand and silty sand native soils and decomposed to highly weathered granite bedrock should be constructed no steeper than 1.5H:1V. Unbraced temporary slopes in the moderately to heavily cemented silty sand soils should be constructed no steeper than near vertical to 0.25H:1V. Unbraced temporary slopes in the moderately weathered to unweathered granite bedrock should be constructed no steeper than near vertical. Slopes may need to be flattened where less cemented soils and/ or adverse jointing or fracturing is encountered. As an alternative to shoring, trench boxes or other forms of slope stabilization may be used for this project.
3. Surface areas behind the crest of excavations should be graded so that surface waters do not pond within 10 feet of the crest or drain into the excavation.
4. Heavy material stockpiles should not be placed within 10 feet of the crest. Similarly, heavy construction equipment should not pass or be parked within 10 feet of the crest.
5. The sides and crest of excavations and slopes should be monitored daily for evidence of movement or potential problems.

The design of any bracing or shoring systems should be reviewed by a qualified geotechnical engineer. Also, observations should be made by the geotechnical engineer during excavating to evaluate site conditions and determine if modifications are necessary in excavation procedures. If unbraced slopes are utilized, some surface raveling, erosion, and spalling should be expected unless measures are taken to stabilize exposed cut surfaces.

MATERIALS SUITABILITY AND REQUIREMENTS

Site Soils and Excavated Granite Bedrock:

Along the proposed RCC dam, outlet and inlet alignments, the site soils and excavated granite bedrock exhibit no to low plasticity. These soils and excavated bedrock may be used as fill in structural backfill areas. All materials should be free of organics, debris, rubble and material greater than 4 inches in size and should be placed such that nesting and voids do not occur.

Imported Soils:

Fill required beyond that available from site sources and for use as structural backfill should be imported soils meeting the following requirements:

Minimum Percent Passing No. 40 Sieve ---------- 30
Maximum Particle Size ---------- 4 inches
Maximum Swell Potential ---------- 1.5%*

* Based on a sample which is remolded to 95% of the ASTM D698 maximum dry density at a moisture content of 2 percent below optimum, placed under a surcharge load of 100 psf and wetted.

Imported soils should have a low corrosion potential as determined by a corrosion expert and/or material supplier, should meet ACI 318 negligible sulfate exposure durability requirements for concrete and be non-reactive when tested for alkali-silica reactivity when tested in accordance with ASTM C-1260.

SITE PREPARATION AND GRADING PROCEDURES

RCC Dam Structural Backfill

Recommendations presented in the previous sections of this report are based upon the following site preparation and grading procedures. Therefore, all earthwork should be accomplished with observation and testing by a qualified technician under the direction of a registered geotechnical/ materials engineer. Recommendations for site preparation and grading for the RCC Dam will be

provided by AMEC. The following apply to the structural backfill and that portion of the storm drain trench excavation beyond the slurry backfill areas.

1. Clear and grub the site by removing and disposing of all vegetation, any trash and debris, and any rubble and remnants of former developments.

2. Moisture condition and place all fill and backfill materials required to achieve specified grades. Fill materials should be moisture conditioned, placed and compacted in horizontal lifts of thicknesses compatible with the compaction equipment being used.

3. Compact structural backfill subgrade and fill to the following minimum percent compaction of the ASTM D698 maximum dry density for each lift.

Material	Minimum Percent Compaction
Soil:	
Structural backfill subgrade and fill	95

4. The moisture content of soil and base materials at the time of compaction should be:

Type	Area of Use	Moisture Content
On-site	Structural backfill	Optimum plus or minus 3%
Imported	Structural backfill	Optimum plus or minus 3%

5. Any soils which are disturbed or overexcavated by the contractor outside the limits of the plans or specifications should be replaced with materials compacted as specified above.

APPENDIX A
FIELD EXPLORATIONS

R·A·M·M

Test Area Location
This Report

R-# = Test Boring
RTP-# = Test Pit
RC-# = Coring
RP-# = Percolation Test
Note: Rock coring not performed at Test Area 1.

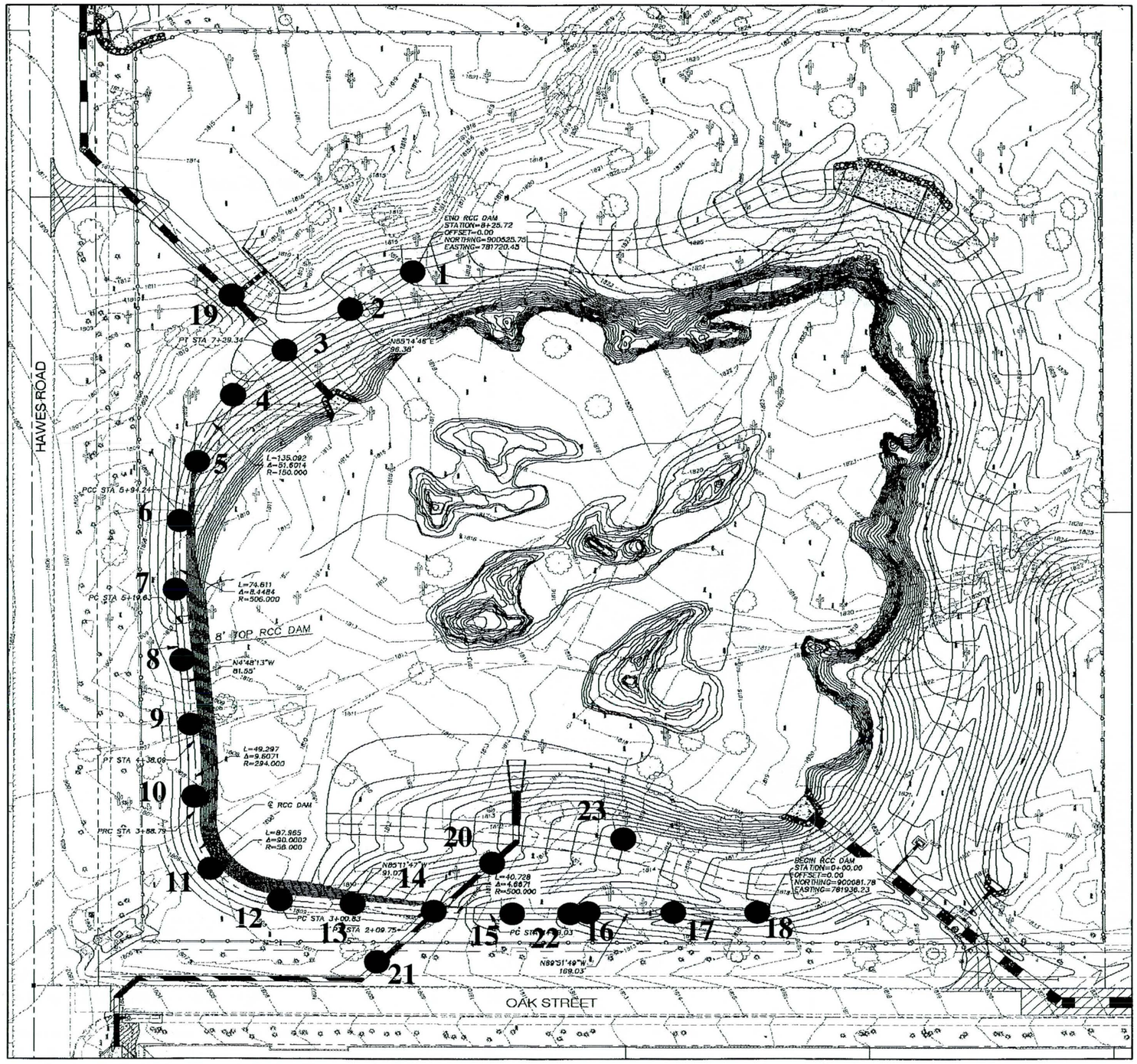
CPT: See continuous penetration test location Site Plan A-B.

Test Area Location
Previous Reports

= Test Boring
TP-# = Test Pit

Ref: RCC Dam Alignment Site Plan prepared by Gavan & Barker, Inc. undated. Not Shown: Alternate Outlet Pipe location approximate Sta 1+10

TEST AREA SITE PLAN



Continuous Penetration Test (CPT) Location ●

N

Ref: RCC Dam Alignment Site Plan prepared by Gavan & Barker, Inc. undated. Not Shown: Alternate Outlet Pipe location approximate Sta 1+10

Not To Scale

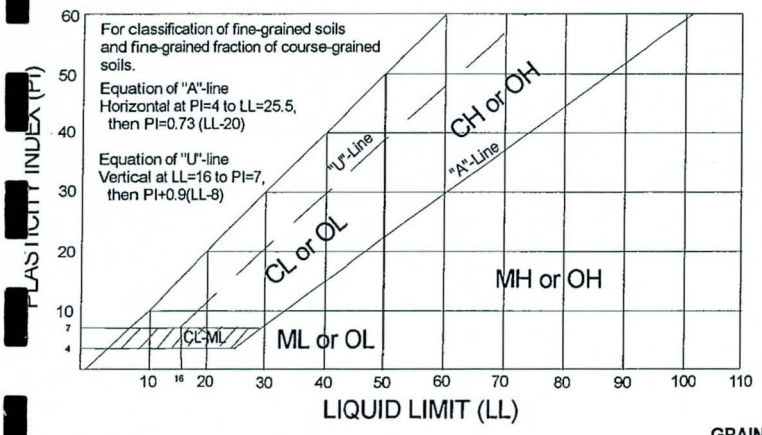
CPT SITE PLAN

LEGEND

CLASSIFICATION OF SOILS

ASTM Designation: D2487-10
(Based on Unified Soil Classification System)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests					Soil Classification: Group Symbol	Soil Classification: Name
COARSE-GRAINED SOILS More than 50% retained on No. 200 Sieve	Gravels More than 50% coarse fraction retained on No. 4 Sieve	Clean Gravels Less than 5% fines	Cu > 4 and 1 < Cc < 3		GW	Well graded gravel
			Cu<4 and/or 1>Cc>3		GP	Poorly graded gravel
		Gravels with Fines More than 12% fines	Fines classify as ML or MH		GM	Silty gravel
			Fines classify as CL or CH		GC	Clayey gravel
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines	Cu > 6 and 1 < Cc < 3		SW	Well-graded sand
			Cu<6 and/or 1>Cc>3		SP	Poorly graded sand
		Sands with Fines More than 12% fines	Fines classify as ML or MH		SM	Silty sand
			Fines classify as CL or CH		SC	Clayey sand
FINE-GRAINED SOILS 50% or more passes the No. 200 Sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI>7 and plots on or above "A" line		CL	Lean clay
			PI<4 or plots below "A" line		ML	Silt
		Organic	Liquid Limit - oven dried / Liquid limit - not dried	<0.75	OL	Organic clay / Organic silt
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line		CH	Fat clay
			PI plots below "A" line		MH	Elastic silt
		Organic	Liquid limit - oven dried / Liquid limit - not dried	<0.75	OH	Organic clay / Organic silt
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor				PT	Peat



TEST BORING LOG & TEST PIT LOG DEFINITIONS

Blows per foot using 140 pound hammer with 30 inch free-fall.

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description

C = Continuous Penetration Resistance (2 inch diameter rod)
N = Standard Penetration Resistance (ASTM D1586)
R = Penetration Resistance (3 inch diameter ring line sampler)

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200	40	10	4	3/4"	3"	12"

SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

MOISTURE CONDITION (INCREASING MOISTURE ⟶)

DRY — SLIGHTLY DAMP — DAMP — MOIST (Plastic Limit) — VERY MOIST — WET (SATURATED) (Liquid Limit)

CONSISTENCY CORRELATION		RELATIVE DENSITY CORRELATION	
CLAYS & SILTS	BLOWS/FOOT*	SANDS & GRAVELS	BLOWS/FOOT*
VERY SOFT	0-2	VERY LOOSE	0-4
SOFT	2-4	LOOSE	4-10
FIRM	4-8	MEDIUM DENSE	10-30
STIFF	8-16	DENSE	30-50
VERY STIFF	16-32	VERY DENSE	OVER 50
HARD	OVER 32		

*Number of blows of 140 lb hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split-spoon sampler (ASTM D1586).

TEST BORING LOG

BORING NO. R1 **DATE:** 4-17-12 **BY:** DM
PROJECT NAME: Oak Street Basin and Storm Drain Design **SHEET NO.** 1 **OF** 4
STA: 7+39, Center Line **GROUND ELEV.** 1,813' AMSL
DRILL EQUIPMENT: CME 55 **W.T. ELEV.** None Encountered
DRILLER: D & S Drilling

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		50/8"	R	100	3	SM	Silty Sand, Some Gravel; brown, slightly damp, medium dense to dense, non-plastic to low plasticity fines, gray, moderate to heavy cementation below depths of 2 to 4 feet.
		50/8"	R	100	3		
		50/8"	R	108	2		
		50/6"	R	NR			
5		50/3"	N		2		
		81/11"	N		3		
		50/7"	R	110	2		
		50/2"	R	NR			
		50/2"	N		1		
10		50/2"	N	NR			
							Auger and Sampler Refusal at 10 feet. No groundwater observed. NR = No Recovery. AMSL=Above Mean Sea Level.
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. R2
PROJECT NAME: Oak Street Basin and Storm Drain Design
STA: 5+95, Center Line
DRILL EQUIPMENT: CME 55

DATE: 4-17-12 **BY:** DM
SHEET NO. 2 **OF** 4
GROUND ELEV. 1,808' AMSL
W.T. ELEV. None Encountered
DRILLER: D & S Drilling

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		16	R	113	2	SM	Silty Sand, Some Gravel; brown, slightly damp, medium dense to dense, non-plastic to low plasticity fines, gray, moderate to heavy cementation below depths of 2 to 4 feet.
		44	R	113	2		
		50/11"	R	113	2		
		46	R	NR			
5		50/7"	R	NR			
		50/8"	R	NR			
		63	N		1		
		32	N		1		
10		31	R	NR			
		50/11"	R	NR			
		39	N		1		
		50/4"	N	NR			
15		75/4"	N		5		
		27	N		2		Occasional zones of light to moderate cementation.
20		50/1"	N	NR			
		50/2"	N	NR			
25		50/2"	N		2		Auger Refusal at 25 feet. Sampler Refusal at 25.2 feet. No groundwater observed. NR = No Recovery AMSL=Above Mean Sea Level.

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. R3 **DATE:** 4-17-12 **BY:** DM
PROJECT NAME: Oak Street Basin and Storm Drain Design **SHEET NO.** 3 **OF** 4
GROUND ELEV. 1,807' AMSL
STA: 4+01, Center Line **W.T. ELEV.** None Encountered
DRILL EQUIPMENT: CME 55 **DRILLER:** D & S Drilling

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		9	R	112	2	SW/	Sand to Silty Sand, Some Gravel; brown,
		8	R	119	1	SM	slightly damp, loose to medium dense, non-
		12	R	121	1		plastic to low plasticity fines, interbedded
		11	R	NR			silty sand.
5		9	R	NR			
		10	R	NR			
		16	R	114	1		
		32	R	112	1		
		50/6"	R	NR			
10		50/7"	R	NR		SM	Silty Sand, Some Gravel; gray, slightly
		50/5"	N		1		damp, dense, non-plastic to low plasticity
		50/4"	N		2		fines, moderate to heavy cementation.
		50/4"	N		3		
		50/4"	N		3		
15		50/2"	N	NR			
		50/5"	N		2		
		50/5"	N		2		Occasional zones of light to
20							moderate cementation.
		65	N		2		
		48	N		2		
							Auger and Sampler Refusal at 27 feet.
25							No groundwater observed.
		50/3"	N		2		NR = No Recovery
							AMSL = Above Mean Sea Level.
		50/0"	N	NR			This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. R4 **DATE:** 4-17-12 **BY:** DM
PROJECT NAME: Oak Street Basin and Storm Drain Design **SHEET NO.** 4 **OF** 4
STA: 1+10, 3' Right **GROUND ELEV.** 1,813' AMSL
DRILL EQUIPMENT: CME 55 **W.T. ELEV.** None Encountered
DRILLER: D & S Drilling

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		7	R	102	2	SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
		11	R	94	4		
		31	R	115	2		
		38	R	NT	NT		
5		25	N		1		
		50/1"	N	NR			Granite, mottled gray/brown decomposed to slightly weathered, weathering decreasing with depth, moderately soft to hard, hardness increasing with depth.
		50/2"	N		2		
10		50/1"	N		1		
							Auger Refusal at 10 feet, Sampler Refusal at 10.1 feet.
							No groundwater observed.
							NR=No Recovery.
							NT=Not Tested: 6-inch long brass tube.
15							AMSL=Above Mean Sea Level.
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. RC2
PROJECT NO. G17544
PROJECT NAME: Oak Street Basin and Storm Drain Design
STA. 5+90, Center Line
DRILL EQUIPMENT: Diedrich D-120

DATE: 4-24-12 **BY:** DM
SHEET NO. 1 **OF** 6
GROUND ELEV. 1808' AMSL
DRILLER: D & S Drilling

Depth, feet	Blows/Foot (Core Data) C (Run)	Blows/Foot (Core Data) N/R (%Rec./ %SCR)	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description (From auger cuttings and core description on field logs)
						SM	Silty Sand, Some Gravel; brown, slightly damp, medium dense to dense, non-plastic to low plasticity fines; gray, moderate to heavy cementation below depths of 2 to 4 feet.
5							
							Advanced test boring with 4-inch diameter solid stem, single flight auger to 9 feet. Set casing to 9 feet. Begin coring at 9 feet.
10							Drill Rate: 0.7 feet/min. Fluid Recovery: 100%
	1	54/49	NX	126	NA		
15							
	2	0/0	NX				Drill Rate: 0.7 feet/min. Fluid Recovery: 100%
20							
	3	0/0	NX				Drill Rate: 0.7 feet/min. Fluid Recovery: 100%
25							Stopped coring at 25 feet. No ground water observed. AMSL = Above Mean Sea Level. SCR = Solid Core Recovery.

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. RC3
PROJECT NO. G17544
PROJECT NAME: Oak Street Basin and Storm Drain Design
STA. 4+04, Center Line
DRILL EQUIPMENT: Diedrich D-120

DATE: 4-24-12 **BY:** DM
SHEET NO. 2 **OF** 6
GROUND ELEV. 1807' AMSL
DRILLER: D & S Drilling

Depth, feet	Blows/Foot (Core Data) C (Run)	Blows/Foot (Core Data) N/R (%Rec./ %SCR)	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description (From auger cuttings and core description on field logs)
						SW/ SM	Sand to Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines, interbedded silty sand.
5							
10							
						SM	Silty Sand, Some Gravel; gray, slightly damp, dense, non-plastic to low plasticity fines, moderate to heavy cementation.
							Advanced test boring with 4-inch diameter solid stem, single flight auger to 15 feet. Set casing to 15 feet. Begin coring at 15 feet.
15							
	1	20/0	NX				Drill Rate: 0.6 feet/min. Fluid Recovery: 100%
20							
	2	70/65	NX	113	NA		Drill Rate: 0.5 feet/min. Fluid Recovery: 90%
25							

- continued -

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. RC3 (continued)
PROJECT NO. G17544
PROJECT NAME: Oak Street Basin and Storm Drain Design
STA. 4+04, Center Line
DRILL EQUIPMENT: Diedrich D-120

DATE: 4-24-12 **BY:** DM
SHEET NO. 3 **OF** 6
GROUND ELEV. 1807' AMSL
DRILLER: D & S Drilling

Depth, feet	Blows/Foot (Core Data) C (Run)	Blows/Foot (Core Data) N/R (%Rec./ %SCR)	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description (From auger cuttings and core description on field logs)
	3	10	NX			SM	Silty Sand, Some Gravel; - continued - Drill Rate: 0.6 feet/min. Fluid Recovery: 80%
30							Stopped coring at 30 feet. No ground water observed. AMSL = Above Mean Sea Level. SCR = Solid Core Recovery.
35							
40							
45							
50							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. RC4
PROJECT NO. G17544
PROJECT NAME: Oak Street Basin and Storm Drain Design
STA. 1+10, 3' Right
DRILL EQUIPMENT: Diedrich D-120

DATE: 4-24-12 **BY:** DM
SHEET NO. 4 **OF** 6
GROUND ELEV. 1812' AMSL
DRILLER: D & S Drilling

Depth, feet	Blows/Foot (Core Data) C (Run)	Blows/Foot (Core Data) N/R (%Rec./ %SCR)	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description (From auger cuttings and core description on field logs)
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
5							
							Granite, mottled gray/brown, decomposed to slightly weathered, weathering decreasing with depth, moderately soft to hard, hardness increasing with depth, widely spaced, high angle, clean joints.
10							
							Advanced test boring with 4-inch diameter solid stem, single flight auger to 14 feet. Set casing to 14 feet. Begin coring at 14 feet.
15							Drill Rate: 0.5 feet/min. Fluid Recovery: 100%
	1	0/0	NX				
20							Drill Rate: 0.5 feet/min. Fluid Recovery: 100%
	2	90/80	NX	156	NA		70° clean joint at 22 feet.
25							50° clean joint at 24.5 feet.

- continued -

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

BORING NO. RC4 (continued)
PROJECT NO. G17544
PROJECT NAME: Oak Street Basin and Storm Drain Design
STA. 1+10, 3' Right
DRILL EQUIPMENT: Diedrich D-120

DATE: 4-24-12 **BY:** DM
SHEET NO. 5 **OF** 6
GROUND ELEV. 1812' AMSL
DRILLER: D & S Drilling

Depth, feet	Blows/Foot (Core Data) C (Run)	Blows/Foot (Core Data) N/R (%Rec./ %SCR)	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description (From auger cuttings and core description on field logs)
	3	100/90	NX				Granite, - continued - Drill Rate: 0.4 feet/min. Fluid Recovery: 100%
30							Stopped coring at 30 feet. No ground water observed. AMSL = Above Mean Sea Level. SCR = Solid Core Recovery.
35							
40							
45							
50							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

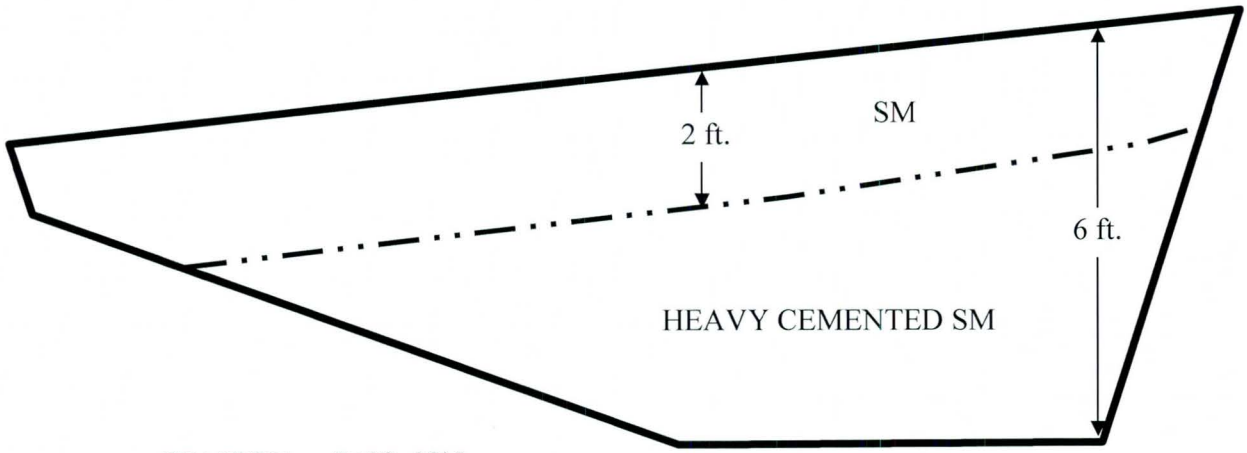
BORING NO. RC5
PROJECT NO. G17544
PROJECT NAME: Oak Street Basin and Storm Drain Design
STA. 2+72, Center Line
DRILL EQUIPMENT: Diedrich D-120

DATE: 4-24-12 **BY:** DM
SHEET NO. 6 **OF** 6
GROUND ELEV. 1808' AMSL
DRILLER: D & S Drilling

Depth, feet	Blows/Foot (Core Data) C (Run)	Blows/Foot (Core Data) N/R (%Rec./ %SCR)	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description (From auger cuttings and core description on field logs)
						SM	Silty Sand, Some Gravel; brown, slightly damp, medium dense to dense, non-plastic to low plasticity fines; gray, moderate to heavy cementation below depths of 2 to 4 feet.
5							
							Advanced test boring with 4-inch diameter solid stem, single flight auger to 10 feet. Set casing to 10 feet. Begin coring at 10 feet.
10							
	1	0/0	NX				Drill Rate: 0.6 feet/min. Fluid Recovery: 100%
15							
	2	68/45	NX	133	NA		Drill Rate: 0.6 feet/min. Fluid Recovery: 100%
20							
							Stopped coring at 20 feet. No ground water observed. AMSL = Above Mean Sea Level. SCR = Solid Core Recovery.
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

RTP-1

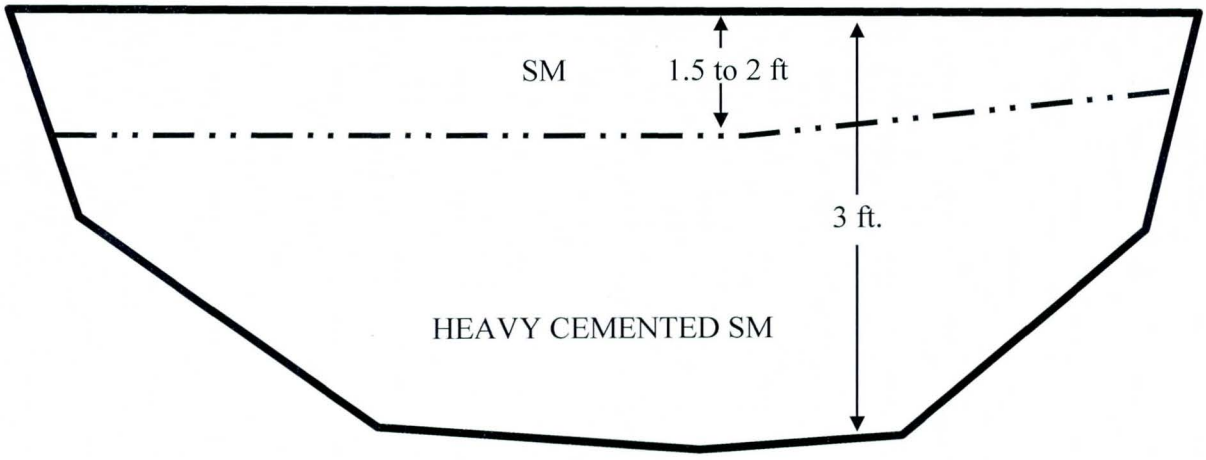


STATION: 7+29, 33' L
ELEVATION: 1812 AMSL
NORTH FACE

Not To Scale

CROSS SECTIONS

RTP 2

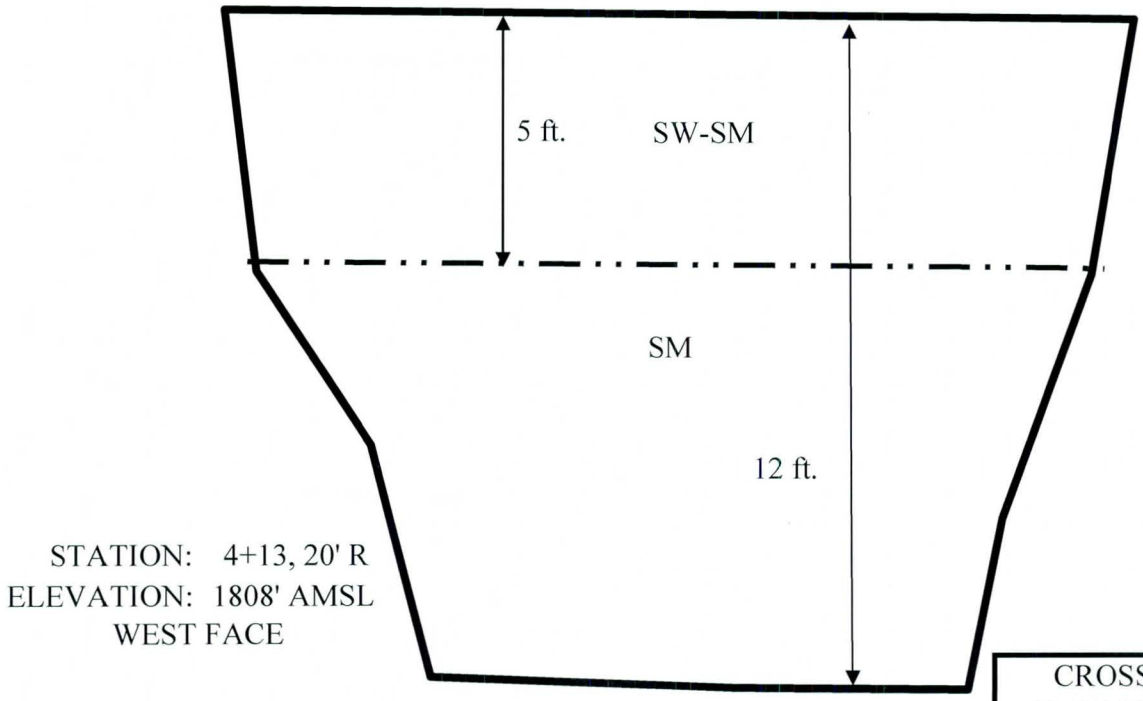


STATION: 5+94, 22' R
ELEVATION: 1809 AMSL
EAST FACE

Not To Scale

CROSS SECTIONS

RTP 3

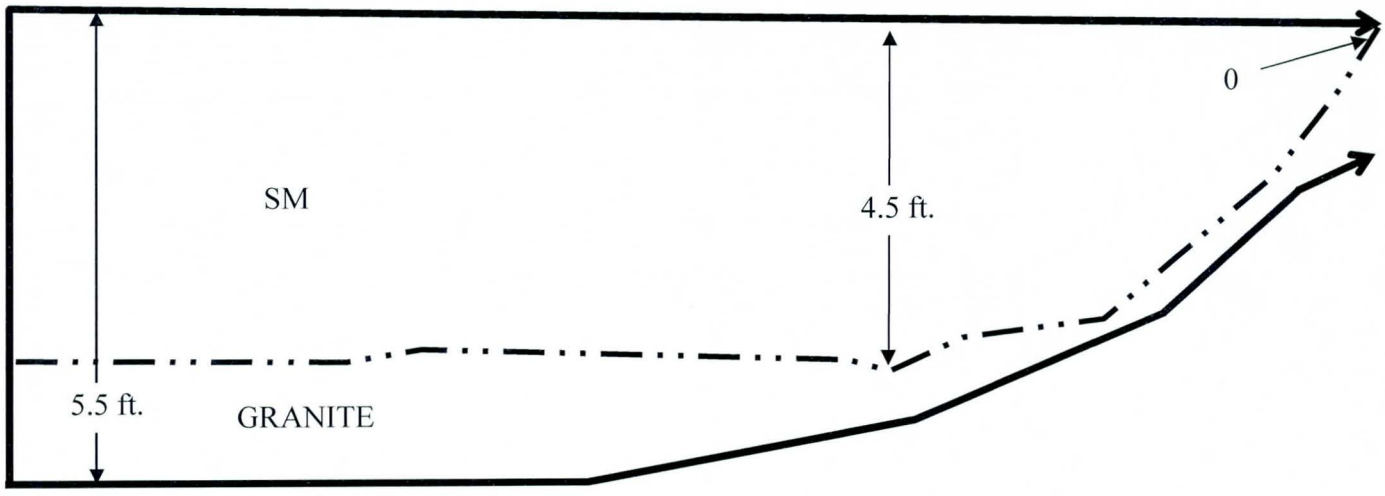


STATION: 4+13, 20' R
ELEVATION: 1808' AMSL
WEST FACE

Not To Scale

CROSS SECTIONS

RTP 4

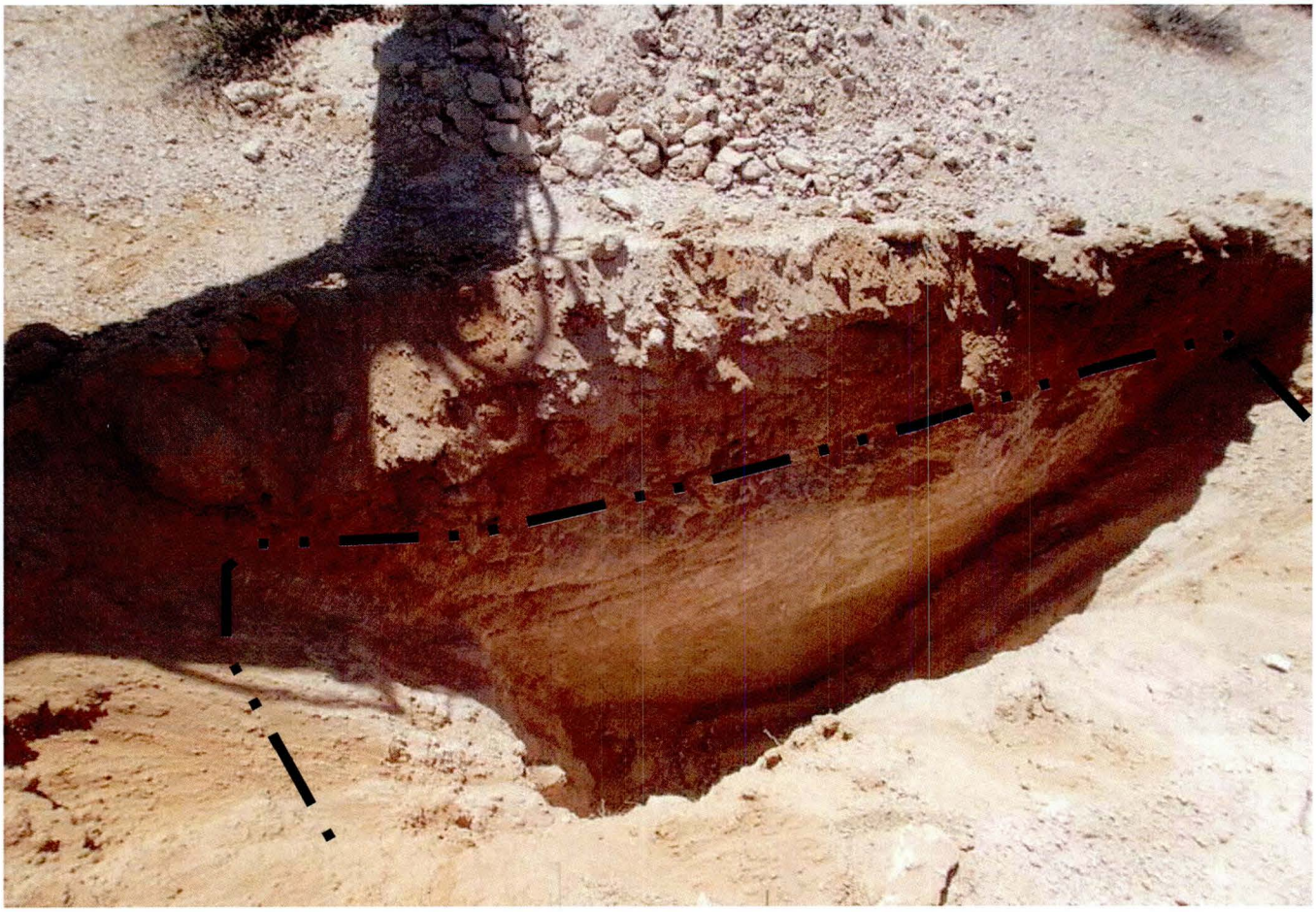


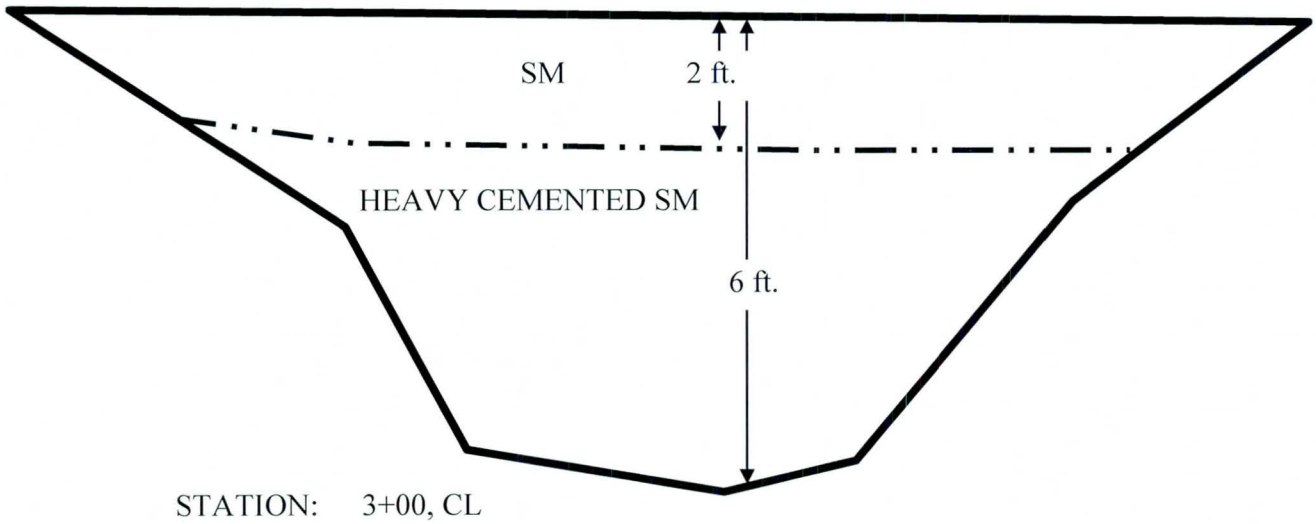
STATION: 1+00, 5' R
ELEVATION: 1813' AMSL
WEST FACE

Not To Scale

CROSS SECTIONS

RTP 5



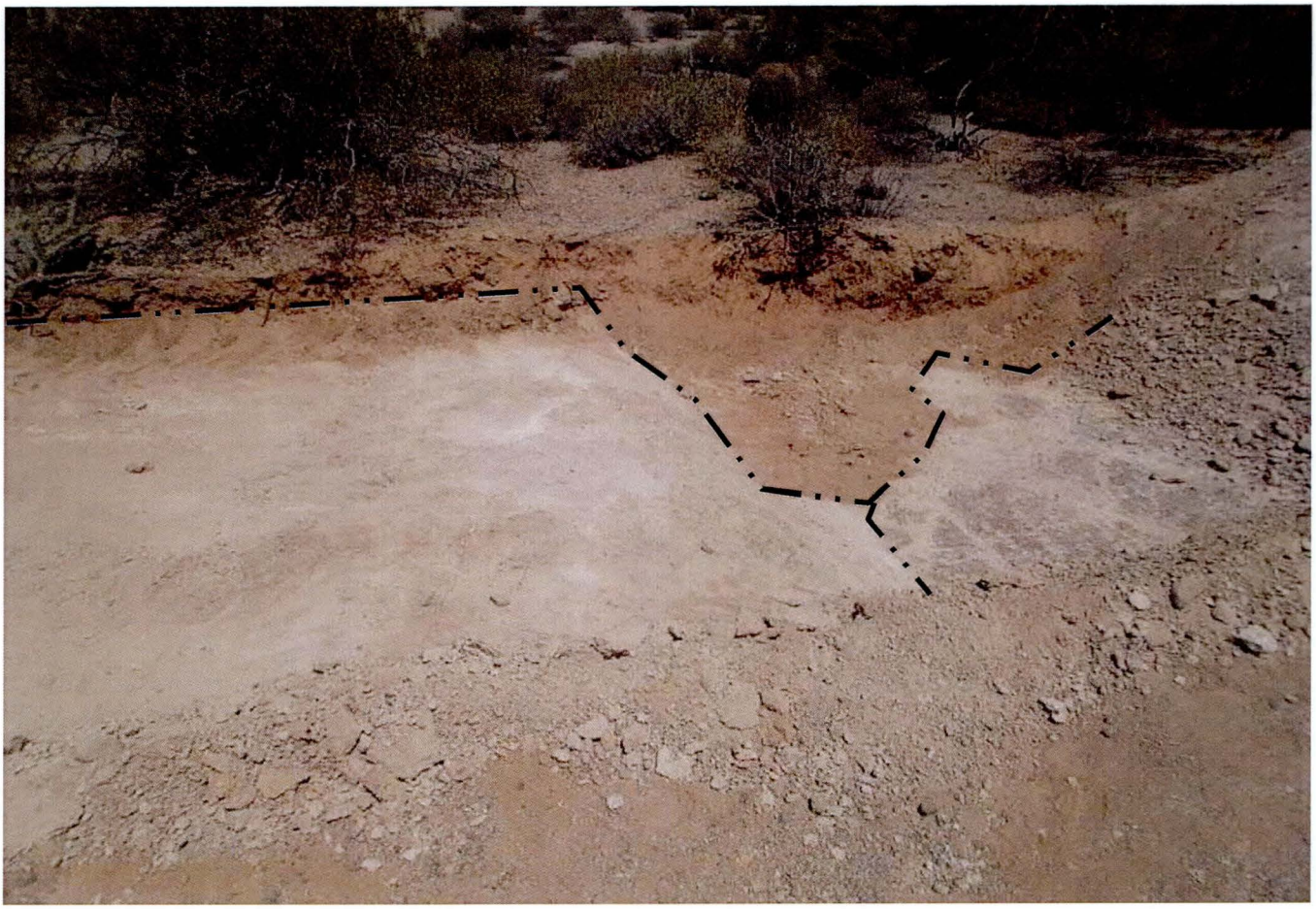


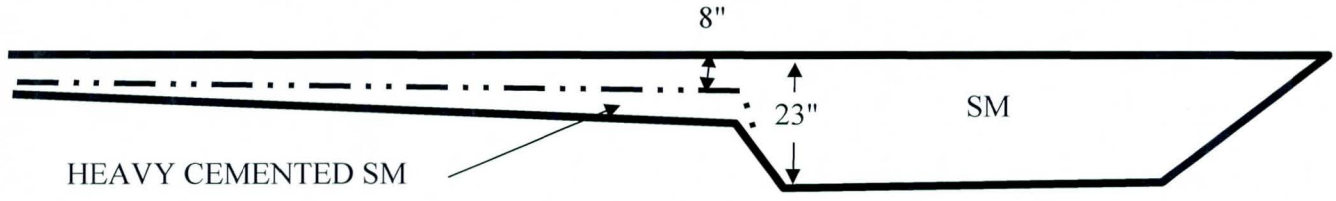
STATION: 3+00, CL
ELEVATION: 1808' AMSL
NORTH FACE

Not To Scale

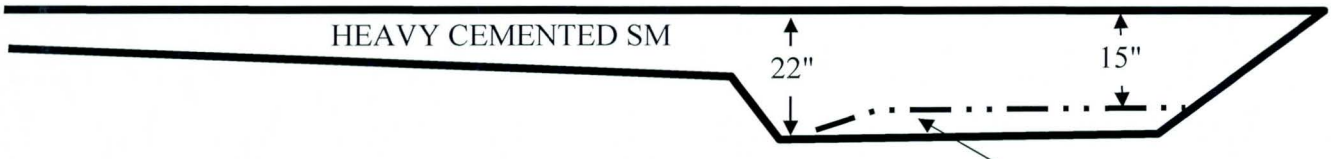
CROSS SECTIONS

RTP 6





NORTH FACE



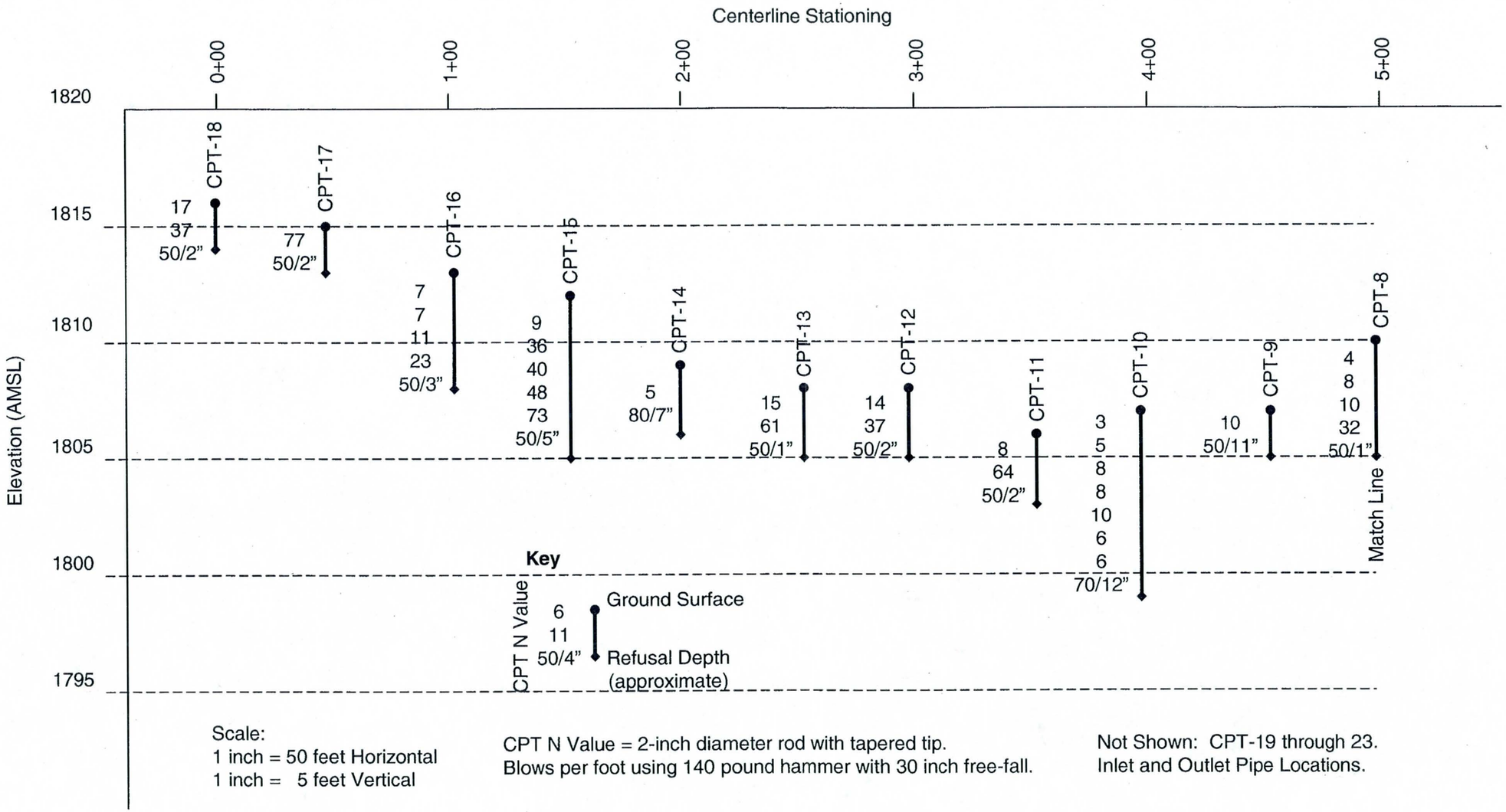
STATION: 0+85, 35' R
ELEVATION: 1815 AMSL

SOUTH FACE

Not To Scale

CROSS SECTIONS

OAK STREET BASIN
RCC DAM ALIGNMENT
CONTINUOUS PENETRATION TESTS PROFILE

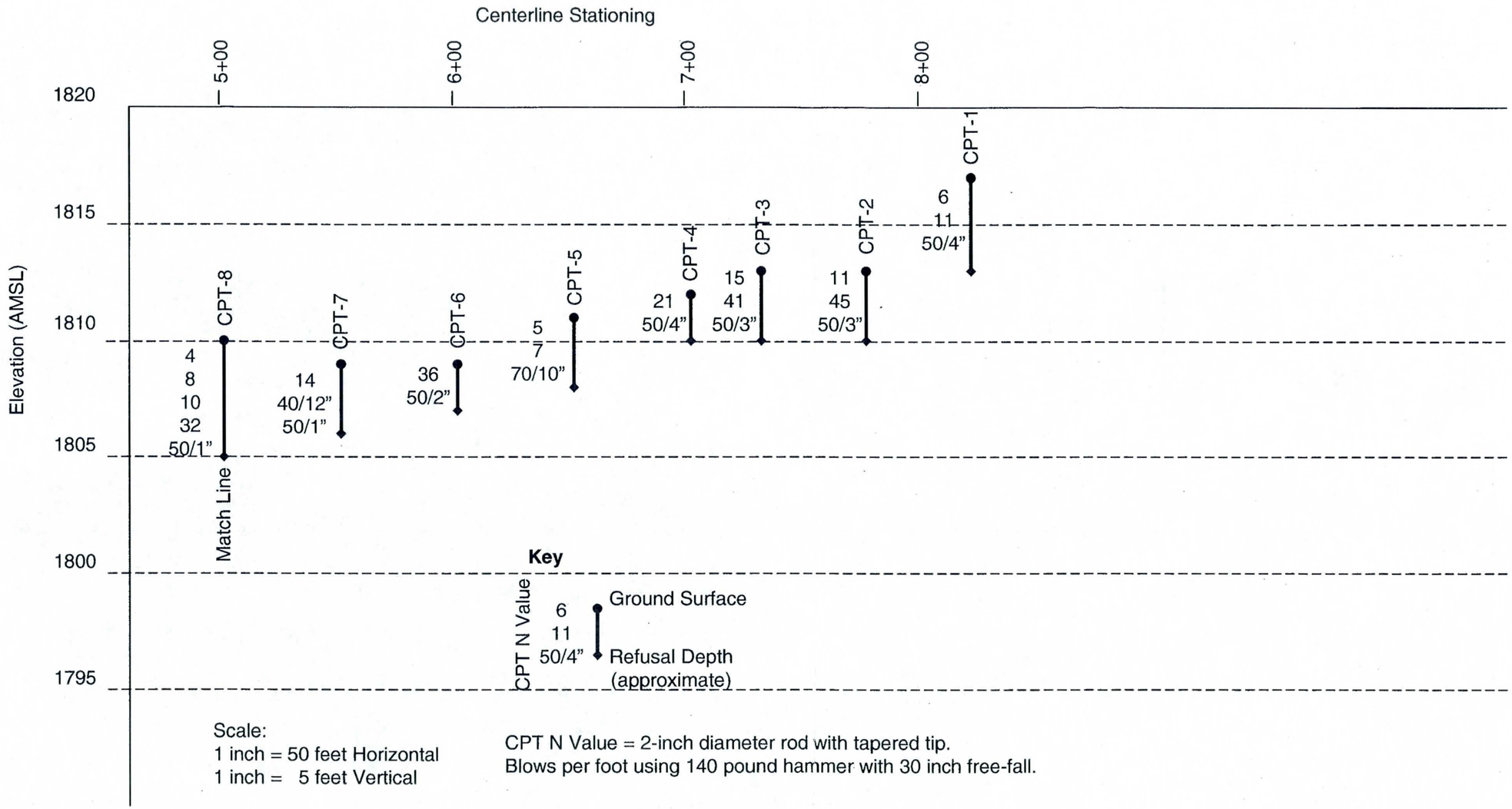


Scale:
1 inch = 50 feet Horizontal
1 inch = 5 feet Vertical

CPT N Value = 2-inch diameter rod with tapered tip.
Blows per foot using 140 pound hammer with 30 inch free-fall.

Not Shown: CPT-19 through 23.
Inlet and Outlet Pipe Locations.

OAK STREET BASIN
RCC DAM ALIGNMENT
CONTINUOUS PENETRATION TESTS PROFILE

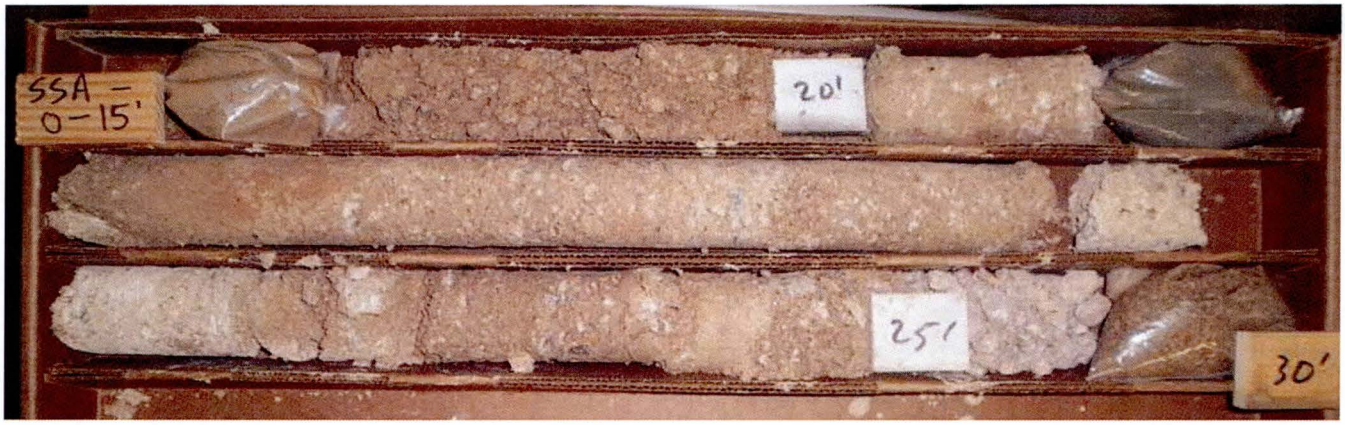


Scale:
1 inch = 50 feet Horizontal
1 inch = 5 feet Vertical

CPT N Value = 2-inch diameter rod with tapered tip.
Blows per foot using 140 pound hammer with 30 inch free-fall.

RC-2: Station 5+90, Centerline; Elevation 1808 ft. AMSL

RC-3: Station 4+04, Centerline; Elevation 1807 ft. AMSL



Not To Scale

CORE PHOTOS

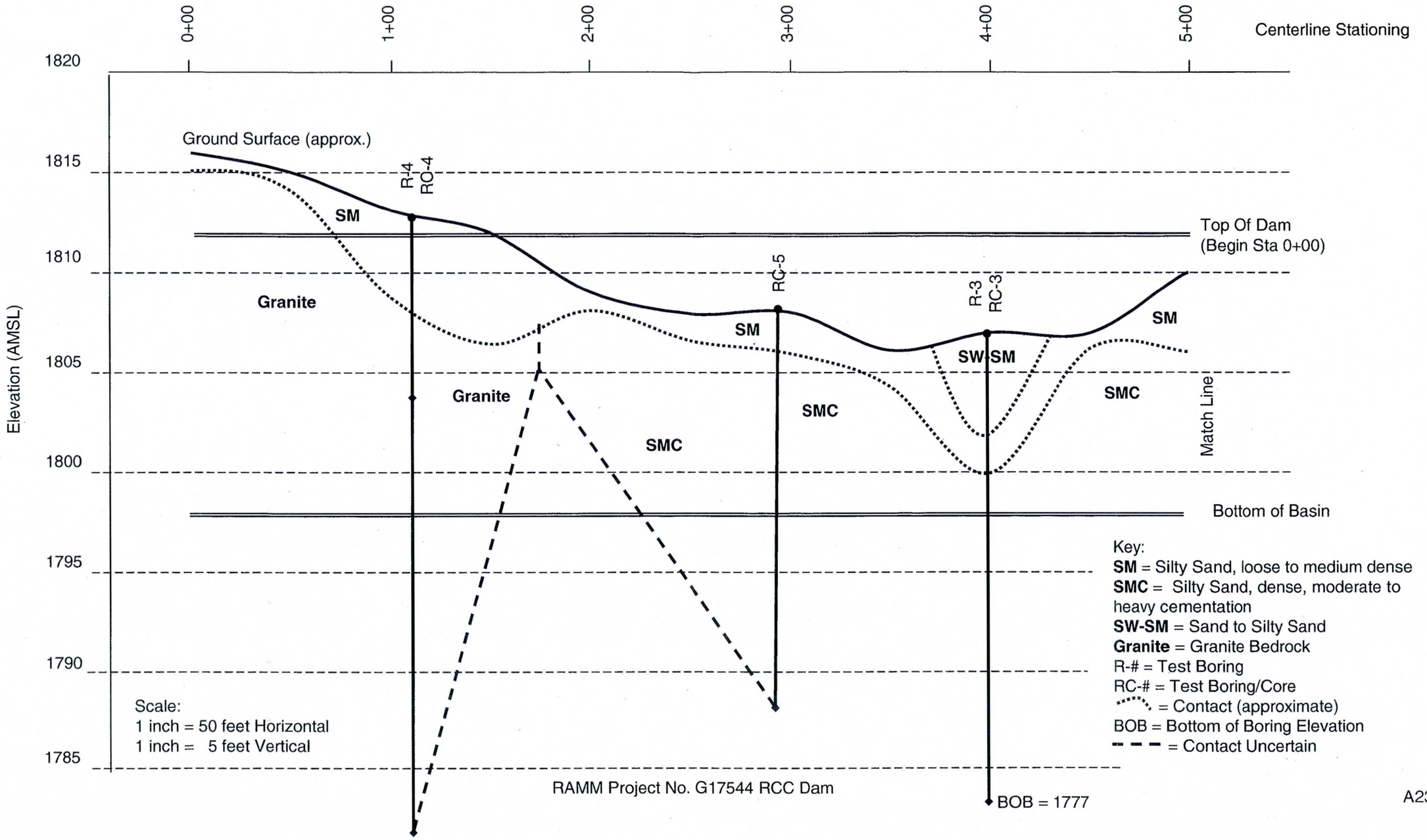
RC-4: Station 1+10, 3' Right; Elevation 1812 ft. AMSL

RC-5: Station 2+72, Centerline; Elevation 1808 ft. AMSL

Not To Scale

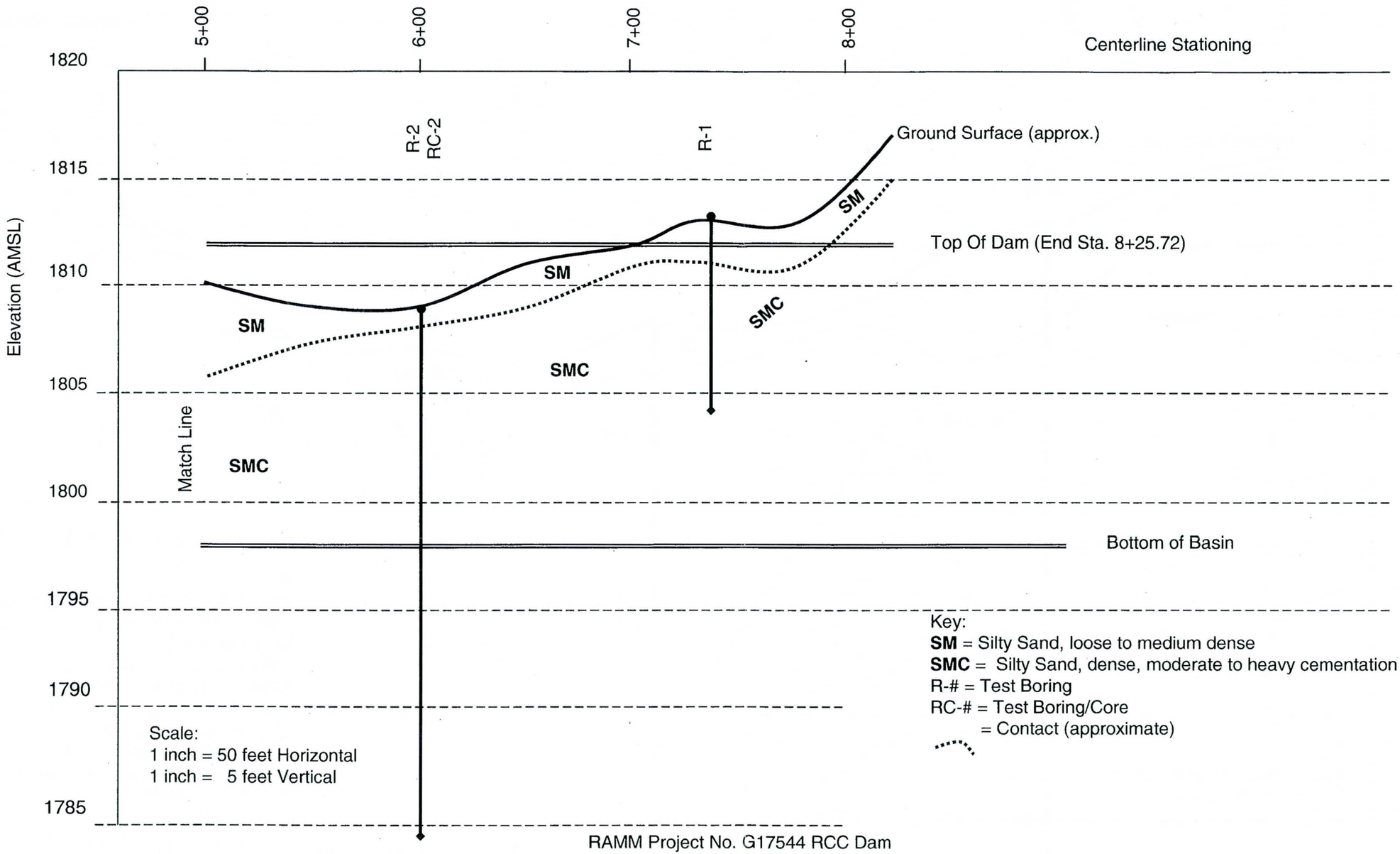
CORE PHOTOS

OAK STREET BASIN
RCC DAM ALIGNMENT
GENERALIZED SUBSURFACE PROFILE



RAMM Project No. G17544 RCC Dam

OAK STREET BASIN
RCC DAM ALIGNMENT
GENERALIZED SUBSURFACE PROFILE



APPENDIX B
LABORATORY ANALYSIS

R·A·M·M

LABORATORY TEST RESULTS

Date: 25-May-12

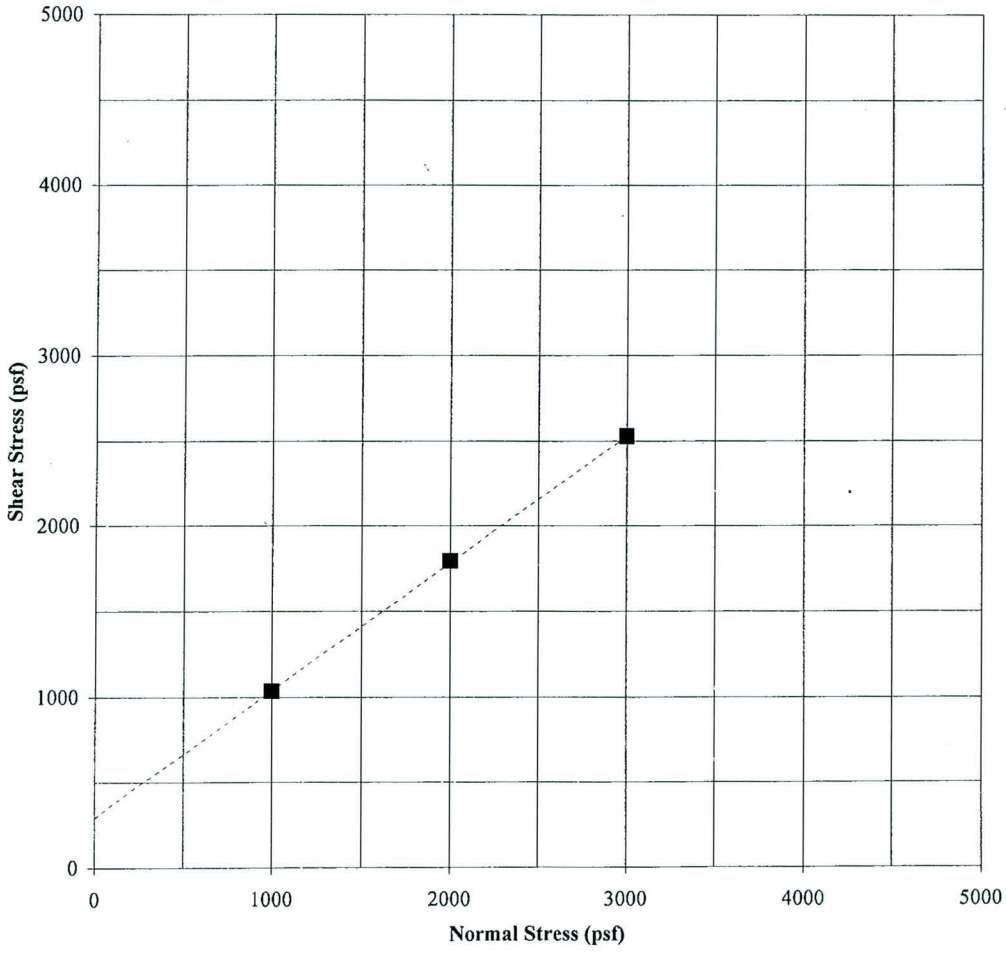
SAMPLE SOURCE: RTP-2 @ 1.5'-5'

TESTING PERFORMED: Direct Shear (ASTM D3080) - Remolded

SAMPLED BY: RAMM/Miller

RESULTS:

Remolded Dry Density (pcf):	112	Remolded Moisture Content (%):	11
Cohesion (psf):	300	Friction Angle (phi):	37



REMARKS: Samples submerged prior to testing.

LABORATORY TEST RESULTS

Date: 25-May-12

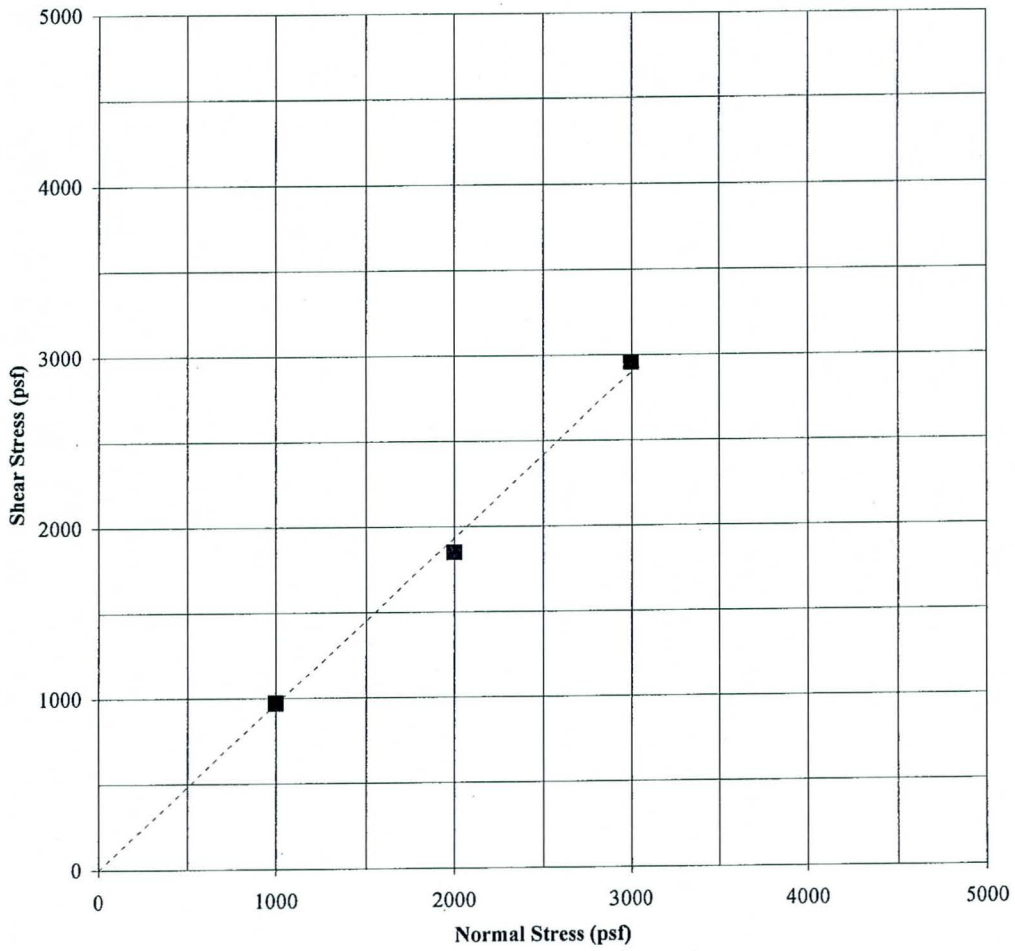
SAMPLE SOURCE: RTP-5 @ 0'-5'

TESTING PERFORMED: Direct Shear (ASTM D3080) - Remolded

SAMPLED BY: RAMM/Miller

RESULTS:

Remolded Dry Density (pcf):	112	Remolded Moisture Content (%):	11
Cohesion (psf):	0	Friction Angle (phi):	45



REMARKS: Samples submerged prior to testing.

LABORATORY TEST RESULTS

Date: 25-May-12

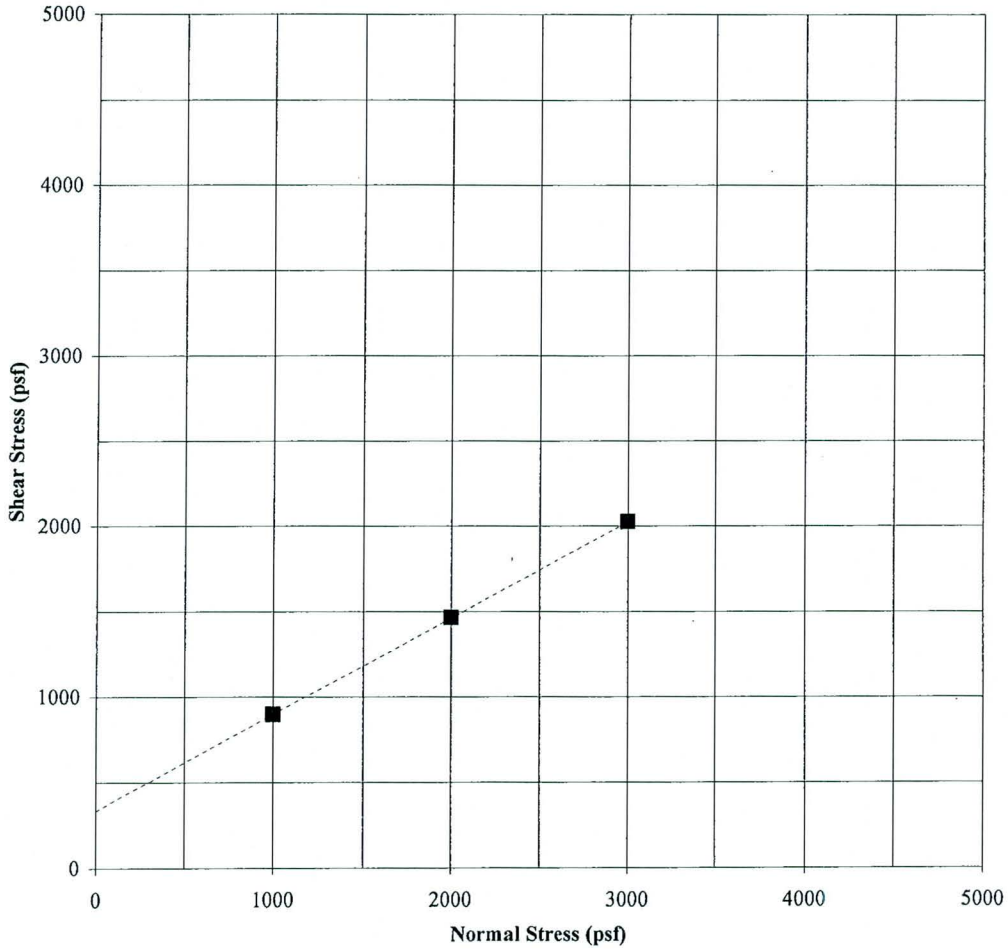
SAMPLE SOURCE: TP-2 @ 0'-2'

TESTING PERFORMED: Direct Shear (ASTM D3080) - Remolded

SAMPLED BY: RAMM/Miller

RESULTS:

Remolded Dry Density (pcf):	126	Remolded Moisture Content (%):	6
Cohesion (psf):	300	Friction Angle (phi):	29



REMARKS: Samples submerged prior to testing.

LABORATORY TEST RESULTS

Date: 25-May-12

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Sieve Analysis, Percent Passing No. 200 Sieve, Atterberg Limits (ASTM C136, D1140, D4318)

SAMPLED BY: RAMM/Miller

RESULTS:

Sample	Atterberg Limits		Sieve Size - Accumulative Percent Passing											Soil
Source	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.*
RTP-2 @ 1.5'-5'	32	2	16	21	26	32	42	57	78	98	100			SM
RTP-3 @ 0'-5'	N/A	NP	6	9	13	19	30	49	74	100				SW-SM
RTP-5 @ 0'-5'	30	2	22	27	33	41	53	70	86	100				SM
TP-2 @ 0'-2'	21	3	20	24	29	34	44	61	78	98	100			SM
TP-3 @ 0'-5'	N/A	NP	4	6	9	15	25	43	69	100				SP

NP = Non-Plastic

* Unified Soil Classification System

LABORATORY TEST RESULTS

Date: 17-May-12

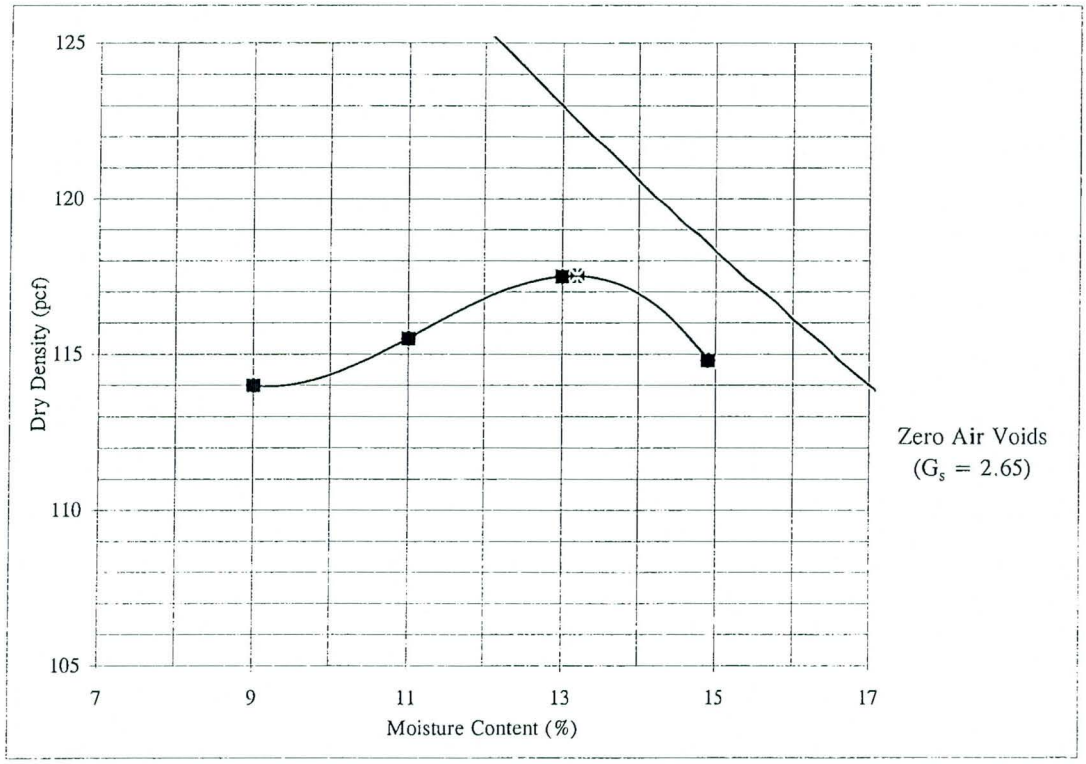
SAMPLE SOURCE: RTP-2 @ 1.5'-5'

TESTING PERFORMED Maximum Density-Optimum Moisture Determination (ASTM D698 Method A)

SAMPLED BY: RAMM/Miller

RESULTS:

Maximum Density (pcf) = 117.5 Optimum Moisture (%) = 13.2



LABORATORY TEST RESULTS

Date: 17-May-12

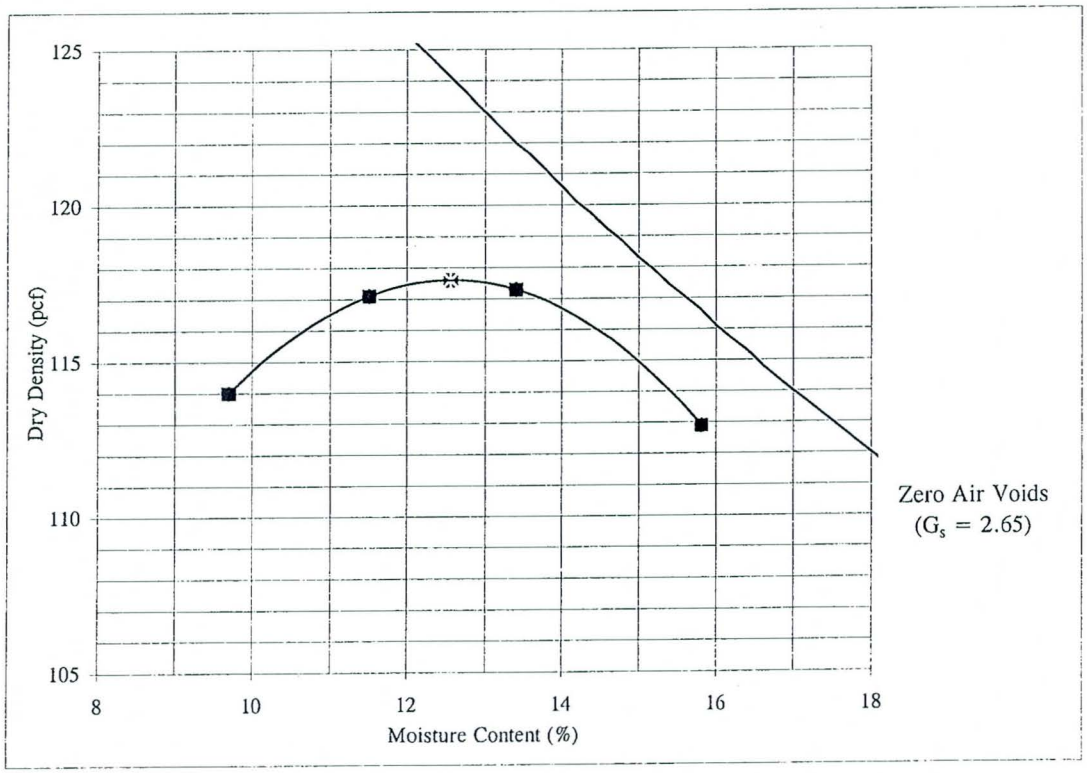
SAMPLE SOURCE: RTP-5 @ 0'-5'

TESTING PERFORMED Maximum Density-Optimum Moisture Determination (ASTM D698 Method A)

SAMPLED BY: RAMM/Miller

RESULTS:

Maximum Density (pcf) = 117.6 Optimum Moisture (%) = 12.6



LABORATORY TEST RESULTS

Date: 17-May-12

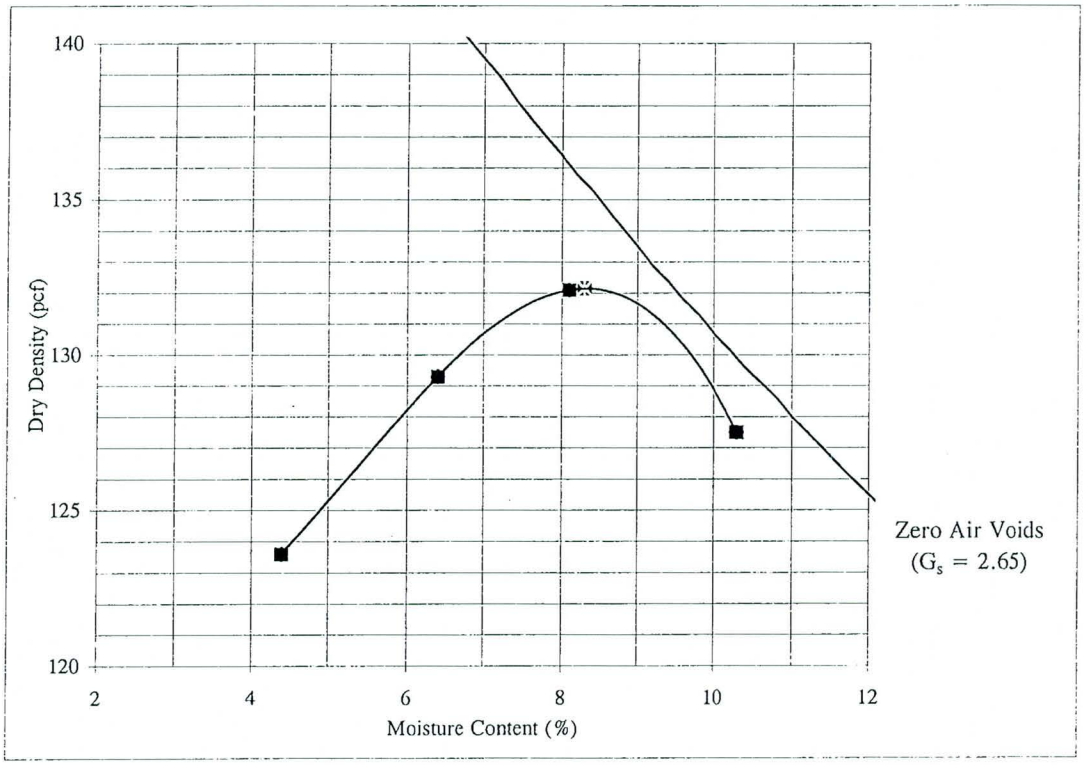
SAMPLE SOURCE: TP-2 @ 0'-2'

TESTING PERFORMED Maximum Density-Optimum Moisture Determination (ASTM D698 Method A)

SAMPLED BY: RAMM/Miller

RESULTS:

Maximum Density (pcf) = 132.2 Optimum Moisture (%) = 8.3



LABORATORY TEST RESULTS

Date: 17-May-12

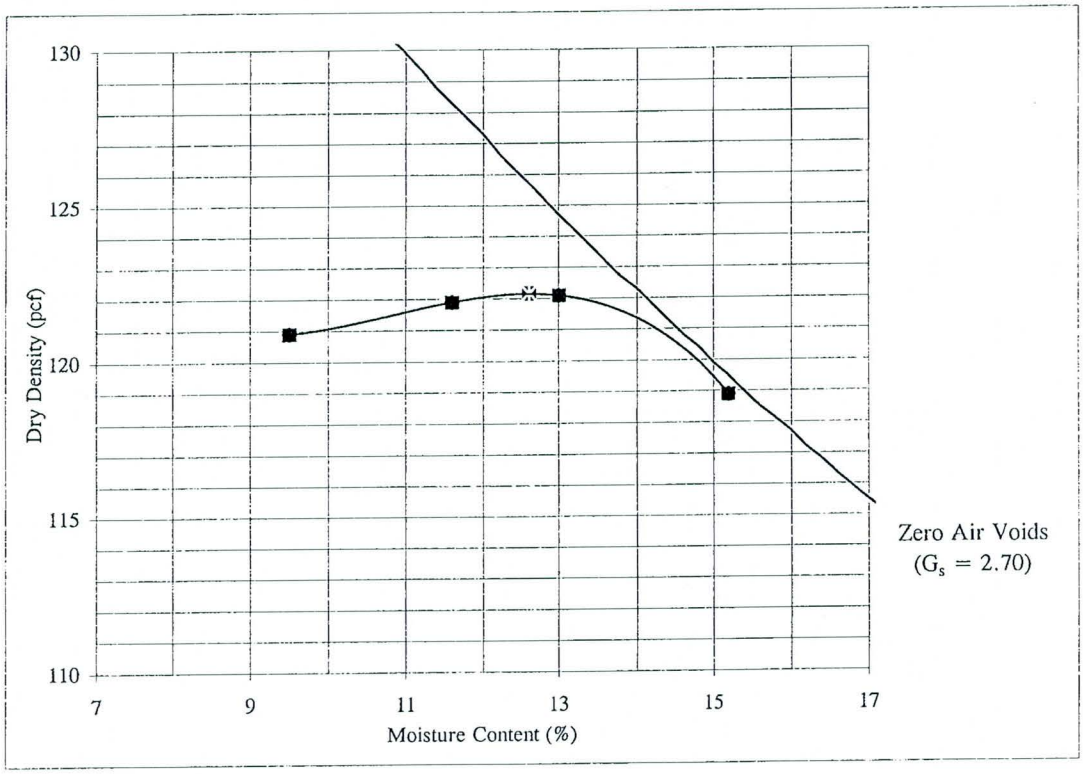
SAMPLE SOURCE: TP-3 @ 0'-5'

TESTING PERFORMED Maximum Density-Optimum Moisture Determination (ASTM D698 Method A)

SAMPLED BY: RAMM/Miller

RESULTS:

Maximum Density (pcf) = 122.2 Optimum Moisture (%) = 12.6



LABORATORY TEST RESULTS

Date: 25-May-12

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Unit Weight, Unconfined Compressive Strength of Rock Cores (ASTM D2938)

SAMPLED BY: RAMM/Miller

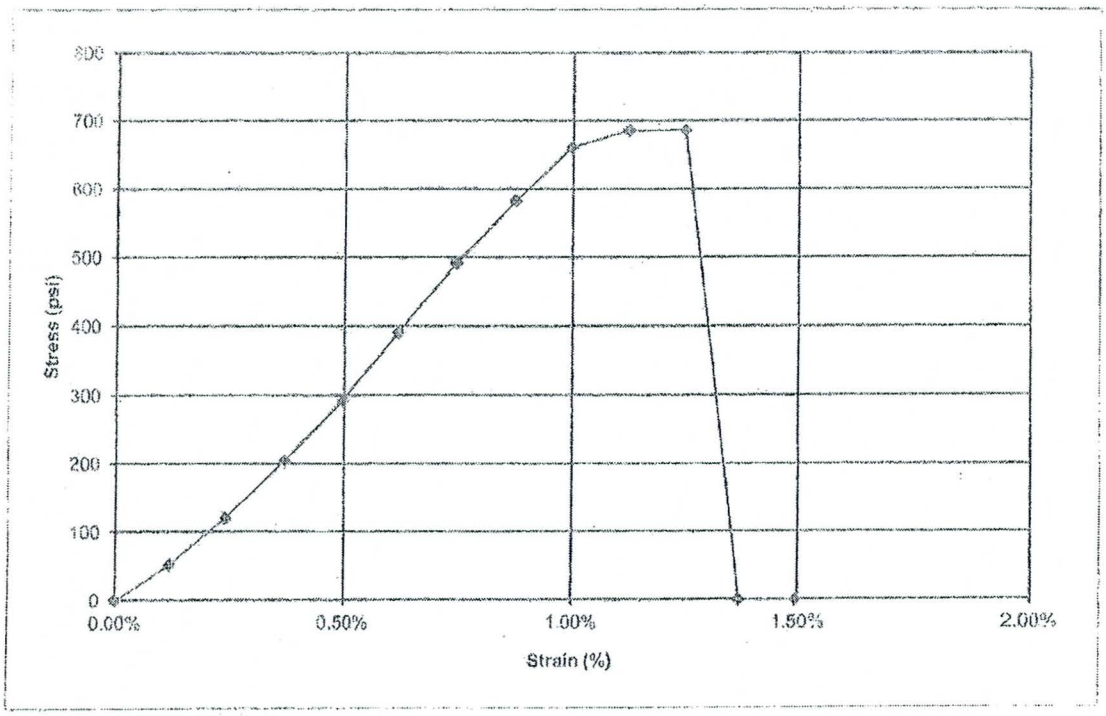
RESULTS:

Sample Source	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
RC-2 @ 11'-11.5'	126	1310
RC-3 @ 21'-22.5'	113	750
RC-4 @ 20.3'-20.6'	156	4060
RC-5 @ 16'-16.6'	133	1978
RTP-4 Granite Hand Sample	121	1120

PROJECT:	Oak Street Basin RCC Dam (G17544)	JOB NO:	17-2011-4050.(
LOCATION:	Oak St & Hanes Road, Mesa, Az	WORK ORDER NO:	2
MATERIAL:	Rock Core	LAB NO:	3
SAMPLE SOURCE:	RC 2 @ 13.7'-15'	DATE ASSIGNED:	05/07/12
SAMPLE PREP:	INSITU		

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS (ASTM D2166)

DIAMETER (D):	1.84 in	STRAIN RATE:	.222 inches/min.
HEIGHT (L):	4.01 in	TOTAL STRAIN:	1.25%
L/D (2.0-2.5 REQ.):	2.18		
DRY DENSITY:	131.0 lb/cu ft	UNCONFINED COMPRESSIVE STRENGTH	686 (psi)
MOISTURE CONTENT	0.9%		



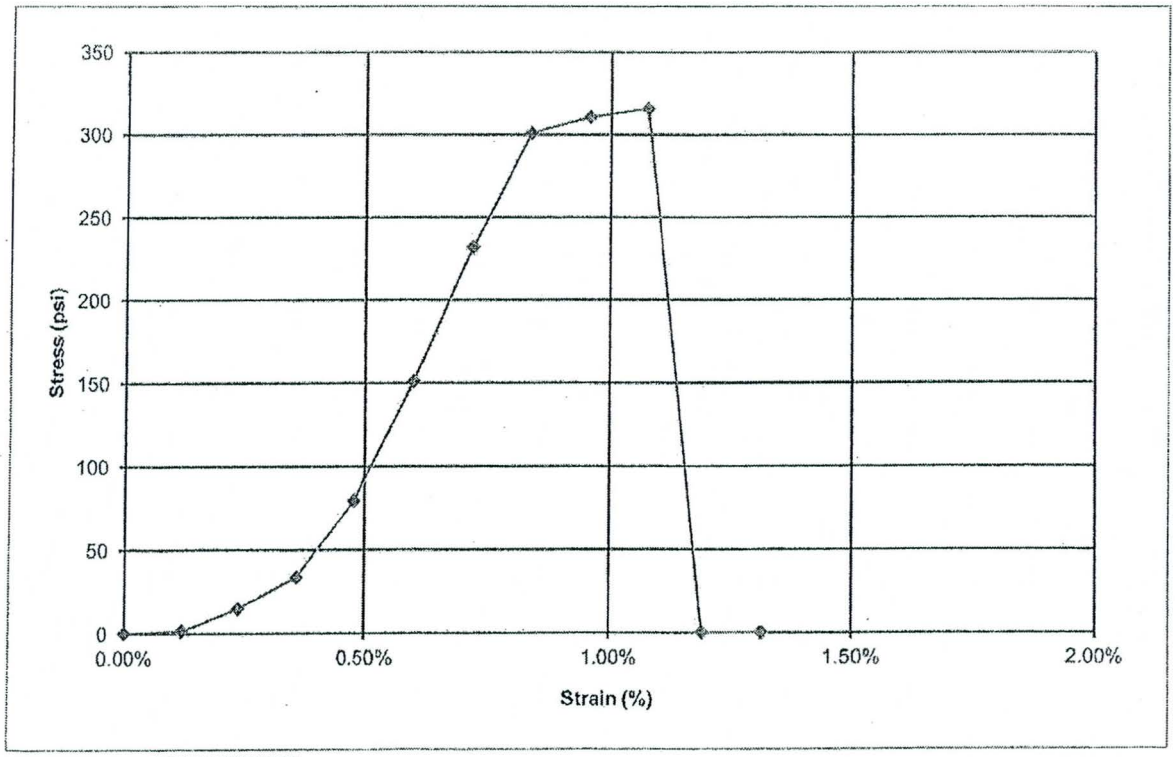
REVIEWED BY

PROJECT:	Oak Street Basin RCC Dam (G17544)	**JOB NO:**	17-2011-4050.(
LOCATION:	Oak St & Hanes Road, Mesa, Az	**WORK ORDER NO:**	2
MATERIAL:	Rock Core	**LAB NO:**	4
SAMPLE SOURCE:	RC 3 @ 21'-22.5'	**DATE ASSIGNED:**	05/07/12
SAMPLE PREP:	INSITU		

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS (ASTM D2166)

DIAMETER (D):	1.87 in	STRAIN RATE:	.222 inches/min.
HEIGHT (L):	4.18 in	TOTAL STRAIN:	1.08%
L/D (2.0-2.5 REQ.):	2.24		
DRY DENSITY:	111.9 lb/cu.ft	**UNCONFINED COMPRESSIVE STRENGTH**	316 (psi)
MOISTURE CONTENT:	1.9%		

Notes: As received, sample was in poor natural condition. Ends were gypsum capped to provide uniform surfaces for testing.



REVIEWED BY

PROJECT:	Oak Street Basin RCC Dam (G17544)	**JOB NO:**	17-2011-4050
LOCATION:	Oak St & Hanes Rd, Mesa, AZ	**WORK ORDER NO:**	2
MATERIAL:	Rock Core	**LAB NO:**	5
SAMPLE SOURCE:	RC 2 @ 13.7'-15'	**DATE SAMPLED:**	05/07/12
SAMPLE PREP:	INSITU		

MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER (ASTM D5084-00) METHOD "C"

AVERAGE PERMEABILITY			1.59E-03	cm/sec
INITIAL LENGTH OF SPECIMEN			5.22	cm
INITIAL DIAMETER OF SPECIMEN			4.88	cm
INITIAL WATER CONTENT			4.1	%
INITIAL DRY UNIT WEIGHT			104.4	pcf
INITIAL VOLUME			97.6	cu.cm
PERMEANT LIQUID			TAP WATER	
MAGNITUDE OF TOTAL BACK PRESSURE			60.0	psi
EFFECTIVE CONSOLIDATION STRESS			5.0	psi
RANGE OF HYDRAULIC GRADIENT USED	4.6	to	3.6	
FINAL LENGTH OF SPECIMEN			5.20	cm
FINAL DIAMETER OF SPECIMEN			4.74	cm
FINAL WATER CONTENT			13.6	%
FINAL DRY UNIT WEIGHT			111.1	pcf
FINAL VOLUME			91.8	cu.cm
DEGREE OF SATURATION (BEFORE AND AFTER TEST)	23%	and		95%
SPECIFIC GRAVITY USED IN CALCULATIONS OF SATURATION			2.397	

TIME INTERVAL (sec)	K (cm/sec)	K (ft/yr)
52	1.58E-03	1633
57	1.57E-03	1625
62	1.61E-03	1665
70	1.58E-03	1640

AASHTO R18

REVIEWED BY

PROJECT:	Oak Street Basin RCC Dam (G17544)	**JOB NO:**	17-2011-4050
LOCATION:	Oak St & Hanes Rd, Mesa, AZ	**WORK ORDER NO:**	2
MATERIAL:	Rock Core	**LAB NO:**	6
SAMPLE SOURCE:	RC 3 @ 21'-22.5'	**DATE SAMPLED:**	05/07/12
SAMPLE PREP:	INSITU		

MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER (ASTM D5084-00) METHOD "C"

AVERAGE PERMEABILITY			9.16E-04	cm/sec
INITIAL LENGTH OF SPECIMEN			5.19	cm
INITIAL DIAMETER OF SPECIMEN			4.78	cm
INITIAL WATER CONTENT			2.4	%
INITIAL DRY UNIT WEIGHT			113.4	pcf
INITIAL VOLUME			93.0	cu.cm
PERMEANT LIQUID			TAP WATER	
MAGNITUDE OF TOTAL BACK PRESSURE			60.0	psi
EFFECTIVE CONSOLIDATION STRESS			5.0	psi
RANGE OF HYDRAULIC GRADIENT USED	4.7	to	3.9	
FINAL LENGTH OF SPECIMEN			5.07	cm
FINAL DIAMETER OF SPECIMEN			4.77	cm
FINAL WATER CONTENT			13.9	%
FINAL DRY UNIT WEIGHT			116.7	pcf
FINAL VOLUME			90.4	cu.cm
DEGREE OF SATURATION (BEFORE AND AFTER TEST)	15%	and		95%
SPECIFIC GRAVITY USED IN CALCULATIONS OF SATURATION			2.578	

TIME INTERVAL (sec)	K (cm/sec)	K (ft/yr)
56	9.18E-04	950
64	9.20E-04	952
73	9.15E-04	947
80	9.11E-04	942

AASHTO R10

REVIEWED BY

amec

PROJECT:	Oak Street Basin RCC Dam (G17544)	**JOB NO:**	17-2011-4050
LOCATION:	Oak St & Hanes Rd, Mesa, AZ	**WORK ORDER NO:**	2
MATERIAL:	Brown Soil	**LAB NO:**	7
SAMPLE SOURCE:	RTP-2 @ 1.5' - 5'	**DATE SAMPLED:**	05/09/12
SAMPLE PREP:	Remolded to 95% Max Dry Density and -2% Opt. Moisture		
	Max Dry Density D698A 117.5 pcf @ 13.2% Opt. Moisture		
TARGET:	Dry Density 111.6 pcf @ 11.2% Moisture		

MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER (ASTM D5084-00) METHOD "C"

AVERAGE PERMEABILITY			1.28E-04	cm/sec
INITIAL LENGTH OF SPECIMEN			7.17	cm
INITIAL DIAMETER OF SPECIMEN			7.12	cm
INITIAL WATER CONTENT			11.2	%
INITIAL DRY UNIT WEIGHT			112.2	pcf
INITIAL VOLUME			285.5	cu.cm
PERMEANT LIQUID			TAP WATER	
MAGNITUDE OF TOTAL BACK PRESSURE			60.0	psi
EFFECTIVE CONSOLIDATION STRESS			5.0	psi
RANGE OF HYDRAULIC GRADIENT USED	3.3	to	3.0	
FINAL LENGTH OF SPECIMEN			7.03	cm
FINAL DIAMETER OF SPECIMEN			7.26	cm
FINAL WATER CONTENT			17.5	%
FINAL DRY UNIT WEIGHT			110.1	pcf
FINAL VOLUME			291.0	cu.cm
DEGREE OF SATURATION (BEFORE AND AFTER TEST)	60%	and	89%	
SPECIFIC GRAVITY USED IN CALCULATIONS OF SATURATION			2.701	

TIME INTERVAL (sec)	K (cm/sec)	K (ft/yr)
164	1.28E-04	132
187	1.28E-04	133
203	1.28E-04	133
220	1.28E-04	132

AASHTO R18

REVIEWED BY

PROJECT:	Oak Street Basin RCC Dam	**JOB NO:**	17-2011-4050
LOCATION:	Oak St & Hanes Rd, Mesa, AZ	**WORK ORDER NO:**	2
MATERIAL:	Brown Soil	**LAB NO:**	8
SAMPLE SOURCE:	RTP-5 @ 0' - 5'	**DATE SAMPLED:**	05/09/12
SAMPLE PREP:	Remolded to 95% Max Dry Density and -2% Opt. Moisture Max Dry Density D698A 117.6 pcf @ 12.6% Opt. Moisture		
TARGET:	Dry Density 111.7 pcf @ 10.6% Moisture		

MEASUREMENT OF HYDRAULIC CONDUCTIVITY OF SATURATED POROUS MATERIALS USING A FLEXIBLE WALL PERMEAMETER (ASTM D5084-00) METHOD "C"

AVERAGE PERMEABILITY			2.67E-04	cm/sec
INITIAL LENGTH OF SPECIMEN			7.57	cm
INITIAL DIAMETER OF SPECIMEN			7.14	cm
INITIAL WATER CONTENT			9.7	%
INITIAL DRY UNIT WEIGHT			106.3	pcf
INITIAL VOLUME			303.1	cu.cm
PERMEANT LIQUID			TAP WATER	
MAGNITUDE OF TOTAL BACK PRESSURE			60.0	psi
EFFECTIVE CONSOLIDATION STRESS			5.0	psi
RANGE OF HYDRAULIC GRADIENT USED	3.1	to	2.6	
FINAL LENGTH OF SPECIMEN			7.54	cm
FINAL DIAMETER OF SPECIMEN			7.29	cm
FINAL WATER CONTENT			21.7	%
FINAL DRY UNIT WEIGHT			102.4	pcf
FINAL VOLUME			314.7	cu.cm
DEGREE OF SATURATION (BEFORE AND AFTER TEST)	45%	and	91%	
SPECIFIC GRAVITY USED IN CALCULATIONS OF SATURATION			2.701	

TIME INTERVAL (sec)	K (cm/sec)	K (ft/yr)
127	2.69E-04	278
140	2.69E-04	278
160	2.67E-04	276
171	2.65E-04	275

AASHTO R18

REVIEWED BY

amec

Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali -Silica Reaction (ASTM C 1260)

Project: Oak Street Basin RCC Dam
Client: Amec Geotech
Project No: 17-2011-4050.0002

Cast Date: 5/8/2012
Report Date:
Work Order: 1
Lab No.: 2

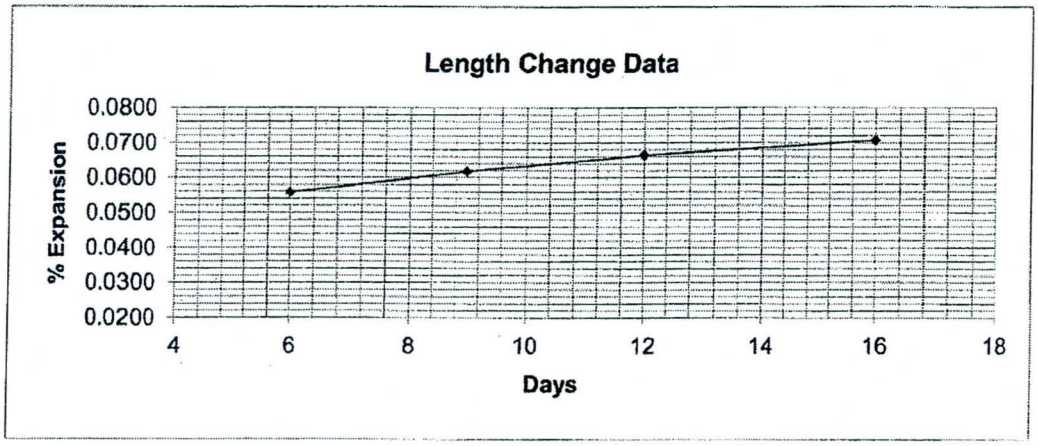
Type of Aggregate: RPT-1/RPT-2 Blend
Source of Aggregate: Field Investigation
Water/Cement Ratio: 0.47
Gage Length: 10.0

Type of Portland Cement: Type II
Source of Portland Cement: APC
Fly Ash Type and Source: N/A
% Fly Ash / Cement N/A
Silica Fume Type and Source: N/A
% Silica Fume / Cement N/A

Longitudinal Measurment Data

RPT-1/RPT-2 Blend

	Intial	Zero	6-Day	6-Day	9-Day	9-Day	12-Day	12-Day	16-Day	16-Day
	(24hr)	(24hr)	Reading	%Expansion	Reading	%Expansion	Reading	%Expansion	Reading	%Expansion
Date:	5/9/2012	5/10/2012	5/13/2012		5/17/2012		5/20/2012		5/24/2012	
Time	1:40	2:10	7:40		6:40		7:10		6:10	
Days	24 hrs	48 hrs	6		9		12		16	
Ref. Bar:	10.0028	10.0019	10.0022		10.0024		10.0024		10.0024	
Bar #1:	9.9997	10.0095	10.0157	0.0590	10.0164	0.0640	10.0169	0.0690	10.0173	0.0730
Bar #2:	9.9844	9.9905	9.9959	0.0510	9.9967	0.0570	9.9972	0.0620	9.9977	0.0670
Bar #3:	9.9959	10.0034	10.0094	0.0570	10.0103	0.0640	10.0107	0.0680	10.0111	0.0720
Average % Expansion:				0.0557		0.0617		0.0663		0.0707



Respectfully Submitted,

amec

Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali -Silica Reaction (ASTM C 1260)

Project: Oak Street Basin RCC Dam
Client: Amec Geotech
Project No: 17-2011-4050.0002

Cast Date: 5/8/2012
Report Date:
Work Order: 1
Lab No.: 1

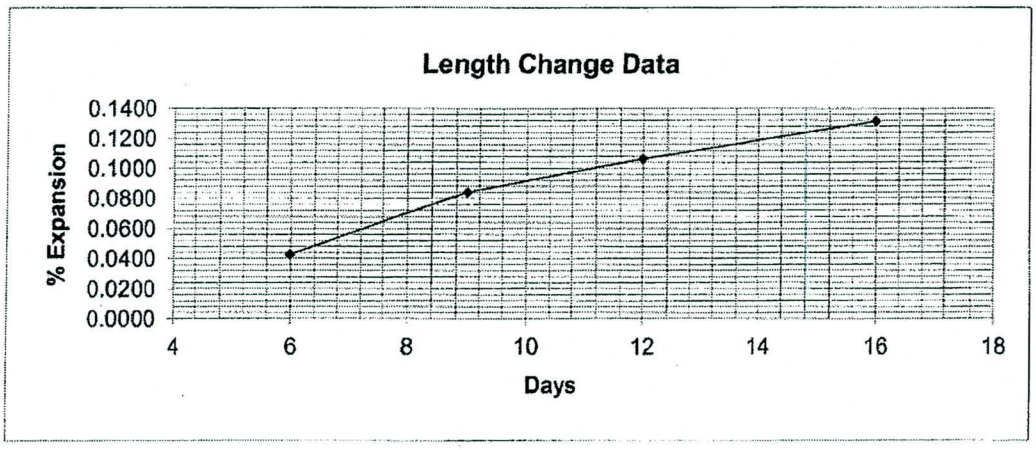
Type of Aggregate: RTP-4 @ 4-6'
Source of Aggregate: Field Investigation
Water/Cement Ratio: 0.47
Gage Length: 10.0

Type of Portland Cement: Type II
Source of Portland Cement: APC
Fly Ash Type and Source: N/A
% Fly Ash / Cement N/A
Silica Fume Type and Source: N/A
% Silica Fume / Cement N/A

Longitudinal Measurment Data

RTP-4 @ 4-6'

	Intial	Zero	6-Day	6-Day	9-Day	9-Day	12-Day	12-Day	16-Day	16-Day
	(24hr)	(24hr)	Reading	%Expansion	Reading	%Expansion	Reading	%Expansion	Reading	%Expansion
Date:	5/9/2012	5/10/2012	5/13/2012		5/17/2012		5/20/2012		5/24/2012	
Time	1:30	2:00	7:30		6:30		7:00		6:00	
Days	24 hrs	48 hrs	6		9		12		16	
Ref. Bar:	10.0028	10.0019	10.0022		10.0022		10.0023		10.0024	
Bar #1:	9.9862	9.9912	9.9957	0.0420	9.9998	0.0830	10.0019	0.1030	10.0040	0.1230
Bar #2:	10.0025	10.0074	10.0120	0.0430	10.0161	0.0840	10.0187	0.1090	10.0219	0.1400
Bar #3:	9.9854	9.9905	9.9951	0.0430	9.9993	0.0850	10.0016	0.1070	10.0041	0.1310
Average % Expansion:				0.0427		0.0840		0.1063		0.1313



Respectfully Submitted,

LABORATORY TEST RESULTS SUMMARY
OAK STREET BASIN RCC DAM
RAMM Project No. G17544

Direct Shear:

Location	Depth (feet)	Soil Type	Cohesion (psf)	Friction Angle (degrees)	Notes
RTP-2	1.5 to 5	SM (cemented)	300	37	remolded
RTP-5	0 to 5	SM (blend)	0	45	remolded
TP-2	0 to 2	SM	300	29	remolded
Note: Direct Shear Tests on solid core and rock core specimens not performed due to undersized diameter.					

Maximum Density-Optimum Moisture Determination:

Location	Depth (feet)	Soil Type	Maximum Density (pcf)	Optimum Moisture (percent)
RTP-2	1.5 to 5	SM (cemented)	117.5	13.2
RTP-5	0 to 5	SM (blend)	117.6	12.6
TP-2	0 to 2	SM	132.2	8.3
TP-3	0 to 5	SM (wash area)	122.2	12.6

Unit Weight, Unconfined Compressive Strength, Cohesion - solid core and rock core:

Location	Depth (feet)	Soil Type	Unit Weight (pcf)	q_u (psi)	C (ksf)	Shear Angle (degrees)
RC-2	11 to 11.5	SM (cemented)	126	1310	95	40 to 70
RC-3	21 to 22.5	SM (cemented)	113	750	54	70
RC-4	20.3 to 20.6	Granite (slightly weathered)	156	4060	293	60
RC-5	16 to 16.6	SM (cemented)	133	1978	143	50 to 70
RTP-4	3	Granite (weathered)	121	1120	81	Crumble
RC-2	13.7 to 15	SM (cemented)	131	686	49	AMEC
RC-3	21 to 22.5	SM (cemented)	316	316	23	AMEC
Notes: q_u = Unconfined Compressive Strength; C = Cohesion; Shear Angle measured from horizontal, at failure; AMEC performed in lieu of direct shear at direction of RAMM..						

LABORATORY TEST RESULTS SUMMARY (continued)

Hydraulic Conductivity (laboratory and field):

Location	Depth (feet)	Soil Type	K (cm/sec)	Notes
RC-2	13.7 to 15	SM (cemented)	1.59×10^{-3}	AMEC; solid core
RC-3	21 to 22.5	SM (cemented)	9.16×10^{-4}	AMEC; solid core
RTP-2	1.5 to 5	SM (cemented)	1.28×10^{-4}	AMEC; remolded
RTP-5	0 to 5	SM (blend)	2.67×10^{-4}	AMEC; remolded
RP-1	2 to 5	SM (cemented)	$3 \text{ to } 5 \times 10^{-4}$	RAMM; field; falling head
RP-2	2 to 15	SM (cemented)	2×10^{-3}	RAMM; field; falling head
RP-2	15 to 20	SM (cemented)	1×10^{-3}	RAMM; field; falling head
RP-3	12 to 25	SM (cemented)	$1 \text{ to } 2 \times 10^{-3}$	RAMM; field; falling head
RP-4	4 to 8	Granite	2×10^{-4}	RAMM; field; falling head
RP-4	8 to 14.5	Granite	5×10^{-4}	RAMM; field; falling head
RP-5	4 to 15	Granite	1×10^{-3}	RAMM; field; falling head

APPENDIX C
REVISED TEST BORING & TEST PIT LOGS

R·A·M·M

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 1

Elevation: Not Determined Datum: --- Date: 2-24-10 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines, gray, moderate to heavy cementation below depths of 2 to 4 feet.
		12	R	NR			
5		48	R	NR			
10							
15							
20							
							Stopped drilling at 20 feet. No groundwater observed. NR=No Recovery
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 2

Elevation: Not Determined Datum: --- Date: 2-24-10 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines, gray, moderate to heavy cementation below depths of 2 to 4 feet.
		50/8"	R	*	5		
5		50/4"	N		2		
10							
15							
							Auger refusal at 17.5 feet. No groundwater observed.
20							* Sample too disturbed to determine density.
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 3

Elevation: Not Determined Datum: --- Date: 2-24-10 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		5	R	104	6	SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines.
		10	R	110	2		
5		20	R	118	1		
10						SM	Silty Sand, Some Gravel; gray, slightly damp, non-plastic to low plasticity fines, moderate to heavy cementation.
15							Stopped drilling at 14 feet. No groundwater observed. NR=No Recovery
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 10

Elevation: Not Determined Sta: Date: 2-21-11 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		14	R	110	1	SW/ SM	Sand, Some Gravel, Trace to Some Silt; brown, slightly damp, medium dense, non-plastic to low plasticity fines, interbedded with silty sand, some gravel and sand, some gravel, trace to some clay.
5		100/6"	R	NR			
		100/4"	R	NR		SM	Silty Sand, Some Gravel; gray, slightly damp, loose to medium dense, non-plastic to low plasticity fines, moderate to heavy cementation below.
10							
15							Stopped drilling at 15 feet. No groundwater observed. NR=No Recovery
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 11

Elevation: Not Determined Sta: Date: 2-21-11 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SW/ SM	Sand, Some Gravel, Trace to Some Silt; brown, slightly damp, medium dense, non-plastic to low plasticity fines, interbedded with silty sand, some gravel and sand, some gravel, trace to some clay.
		12	R	106	1		
5							
		17	R	108	2		
		50/11"	R	116	1		
10							
		66/10"	R	121	1	SM	Silty Sand, Some Gravel; gray, slightly damp, loose to medium dense, non-plastic to low plasticity fines, moderate to heavy cementation.
		100/2"	R	NR			
15							
		100/2"	R	NR			
							Stopped drilling at 16 feet. No groundwater observed. NR=No Recovery
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST BORING LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST BORING: 12

Elevation: Not Determined Sta: Date: 2-21-11 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
		8	R	104	2	SW/SM	Sand, Some Gravel, Trace to Some Silt; brown, slightly damp, medium dense, non-plastic to low plasticity fines, interbedded with silty sand, some gravel and sand, some gravel, trace to some clay.
5		100/6"	R	NR		SM	Silty Sand, Some Gravel; gray, slightly damp, loose to medium dense, non-plastic to low plasticity fines, moderate to heavy cementation.
		100/2"	R	NR			
10							
15							Stopped drilling at 15 feet. No groundwater observed. NR=No Recovery
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: TP 1

Elevation: Not Determined Datum: Date: 6-3-10 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines, gray, moderate to heavy cementation below depths of 2 to 4 feet.
5							
10							
15							
							Stopped excavation at 18 feet. No groundwater observed.
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

TEST PIT LOG

Project: Oak Street Basin and Storm Drain Design-Maricopa County, AZ TEST PIT: TP 3

Elevation: Not Determined Datum: Date: 6-3-10 Revised 6-13-12

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density, pcf	Water Content, %	Unified Classification	Description
5						SM	Silty Sand, Some Gravel; brown, slightly damp, loose to medium dense, non-plastic to low plasticity fines, gray, moderate to heavy cementation below depths of 2 to 4 feet.
10							Stopped excavation at 10 feet. No groundwater observed.
15							
20							
25							

This boring log represents the conditions encountered on the date of drilling at this particular location. No other warranty is expressed or implied to the actual conditions which may exist within the vicinity of this boring location.

RCC DAM DESIGN

Final Design Report
Roller Compacted Concrete Dam
Oak Street Detention Basin & Storm Drain Project
Mesa, Arizona
FCD Contract 2009C021

Submitted to:

Gavan & Barker, Inc.
Phoenix, Arizona

Submitted by:

AMEC Environment & Infrastructure, Inc.
Phoenix, Arizona

December 4, 2012

AMEC Project No. 17-2011-4050

December 4, 2012
AMEC Project No. 17-2011-4050

Gavan & Barker, Inc.
3030 North Central Avenue, #1530
Phoenix, Arizona 85012-2748

Attn: Mark Gavan, PE

Re: Final Design Report
Roller Compacted Concrete Dam
Oak Street Detention Basin & Storm Drain Project
Flood Control District of Maricopa County
FCD Contract 2009C021
Mesa, Arizona

The final design report for the above-referenced project is submitted here within. This report presents the final design for the elements of a new roller compacted concrete dam for the Oak Street Detentions Basin & Storm Drain Project.

Should you have any questions concerning the data design elements discussed in this report, please do not hesitate to contact the undersigned.

Respectfully submitted,

AMEC Environment & Infrastructure, Inc.

Naida Causevic, EIT
Staff Professional

Reviewed by:

Registered Professional Engineer (Civil) CERTIFICATE NO. 40220 BRETT A. HOWEY Date Signed 12/4/12 ARIZONA, U.S.A. Expires 12/31/12

Brett A. Howey, PE
Arizona Operations Manager

c: Addressee (PDF)

G:\Geotechnical\2011 Projects\17-2011-4050 Oak Street Basin RCC Dam Design\100% Design Report\100% Design Report_Oaks Street Basin RCC Design_Final3.docx

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1.0 INTRODUCTION

This report presents the final design developed by AMEC Environment & Infrastructure, Inc. (AMEC) for the elements of a new roller compacted concrete dam for the Oak Street Detention Basin & Storm Drain Project (Oak Street). Oak Street will be operated and maintained by the Flood Control District of Maricopa County (District). This design report summarizes the primary elements of the roller compacted concrete dam (RCC), and presents engineering analyses in support of the design of the new dam. This task was authorized by the District in April 2012 under the terms and conditions of Contract No. FCD 2009C021.

2.0 PROJECT DESCRIPTION

Gavan & Barker, Inc. (G&B) is the District's prime design consultant for this project. It has been recognized that the embankment that forms the southwest section of the basin is considered a dam structure. The dam and storage capacity are not of sufficient size to be a jurisdictional dam as defined by Arizona Administrative Code Title 12 – Natural Resource, Chapter 15 – Natural Resources, Article 12 – Dam Safety Procedures, however, the District is treating the design of the embankment as a dam structure. After completion of a Failure Modes and Effects Analysis (FMEA) workshop which analyzed three potential alternatives (incised basin, embankment dam, and a RCC dam), the District has decided to move ahead with the RCC dam alternative per the outcome of the FMEA workshop (AMEC 2011) whereas the RCC alternative addressed all potential failure mdoes. The RCC dam will be approximately 825 feet long and less than about 14 feet in height.

Oak Street Detention Basin is located in northeast Mesa by Oak Street and Hawes Road, Maricopa County, Arizona, as part of the 35-square-mile area identified in the Spook Hill Area Drainage Master Plan (ADMP) as requiring necessary regional flood control infrastructure. The ADMP watershed extends from the Usery Mountains on the north and the Apache Trail on the east, to the Buckhorn-Mesa structures on the west and south. The Oak Street Detention Basin and Storm Drain project is the third scheduled project in support of this ADMP and involves construction of a basin at Oak Street and Hawes Road, and storm drains east along Oak Street and north along Hawes Road. The project will provide protection in conjunction with drainage infrastructure constructed by the Hermosa Vista/Hawes Road and McDowell Road projects.

AMEC has been retained by G&B to work with the project design team and provide design recommendations related to the typical dam structure cross section, dam foundation criteria, and RCC mix design.

3.0 GEOLOGIC & GEOTECHNICAL SITE CHARACTERIZATION

A subsurface exploration was conducted by Ricker-Atkinson-McBee-Morman & Associates, Inc. (RAMM) in April 2012 in support of the construction of the RCC dam, which borders the proposed basin. The investigation included 19 continuous penetration tests (CPT), eight borings (including auger and coring), six test pits, and five percolation tests. As part of AMEC's design efforts, on April 18, 2012, Mr. Brett Howey, PE, with AMEC, accompanied RAMM during the field investigation to observe test pit excavation and inspection of the subsurface conditions.

RAMM (2012) describes the near surface soils encountered at the site, above the bedrock and cemented soil horizons, as silty sands with gravel with a plasticity index ranging from nonplastic to low plasticity. The silty sand soils were generally moderate to heavy cemented, interbedded with low to medium cemented zones. The bedrock was classified as granite. The granite was decomposed to slightly weathered, moderately soft to hard, and moderate to widely spaced, smooth to slightly rough, clean joints, which measured 50 to 70 degrease from the horizontal. The condition of the rock improved with depth, with weathering decreasing and hardness increasing. The maximum depth of the geotechnical investigation was 30 feet below ground surface.

A summary of RAMM's engineering parameters is shown below in Table 1.

Table 1

RAMM (2012) Recommended Soil Parameters

Soil Type	Unit Weight [pcf]	Phi [degrees]	Cohesion [ksf]	Permeability [cm/sec]
Cemented Soil	120	45	10	$1x10^{-3}$
Bedrock (Granite)	155	50	25	$1x10^{-3}$
Structural Backfill	114	38	0	$1x10^{-4}$
Native Alluvium	105	25	0	$1x10^{-2}$

Note: pcf = pounds per cubic foot; ksf = kips per square foot

4.0 VERIFICATION OF DAM TYPE SUITABILITY

As part of the design effort an FMEA was held on September 14 and 15, 2011 to evaluate failure modes associated with the dam type alternatives (AMEC 2011). Prior to the FMEA, the project team had developed and recommended three concept alternatives consisting of an earthen embankment with central filter, a RCC structure and an incised basin. The goals of the FMEA were to achieve an understanding of the most significant site-specific failure modes and the potential consequences of a system failure for each of the three alternatives and to develop potential mitigation strategies to eliminate any identified failure modes and/or reduce the potential consequences.

After completion of the workshop the RCC dam type was selected as the preferred alternative. During the FMEA it was assumed the dam would be founded on a granitic bedrock foundation for its entire length. As a result of further investigation by RAMM (2012) the foundation conditions were determined to be primarily alluvium overlying cemented soils with only a small portion of the dam founded on a granitic bedrock foundation. AMEC reviewed the potential failure modes assessed during the FMEA against the changed foundation conditions, from bedrock to cemented soils, and maintains that use of an RCC dam structure remains suitable for this site without comprimising or creating any failure modes.

5.0 FOUNDATION EXCAVATION AND PREPARATION CRITERIA

5.1 Foundation Excavation

Prior to the construction of the RCC dam, soils within the foundation footprint should be over excavated and removed to sufficient depth to remove the loose, near surface alluvial soils. The RCC dam should be founded on cemented soils or rock. AMEC has reviewed and concurs with RAMM's recommendation to found the dam on moderately to heavily cemented soils or on undisturbed moderately to slightly weathered granite bedrock (RAMM 2012). RAMM presents a Generalized Subsurface Profile in their report which identified the approximate contact between the near surface soils and the cemented/granite materials. AMEC recommends foundation excavation to a minimum depth of the contact plus 2 foot of additional excavation. This foundation line is approximate and may require adjustment during construction. Where zones or pockets of soil or rock that is uncemented, erodible, weak, unstable, compressible, loose, pervious, highly weathered, or highly fractured exist, it may be necessary to remove these soils to a depth where suitable cemented soil or rock foundation is encountered. A maximum depth of no more than 1x the dam height is the recommended criteria to be used by the field engineer when assessing the maximium depth to remove uncement or non-rock foundation. The intent of the foundation excavation is to create a relatively smooth and evenly sloped surface on which to place the RCC dam materials.

5.2 Foundation Preparation

When the foundation excavation has reached the approximate limits recommended in Section 4.1 or when the on-site engineer/geologist determines that a satisfactory foundation is reached, the foundation footprint should be cleaned to sufficient depth to remove all debris, loose rock, sand, silt, and other objectionable material. Foundations surfaces should be maintained in a clean and undisturbed condition until the placement of RCC. A two-stage foundation preparation process is recommended. The first stage should include a preliminary clean-up consisting of removal of all unacceptable material by hand tools followed by air, water, air/water jets or combinations thereof. Following the preliminary clean-up, a final foundation cleaning should be completed depending on the type of foundation encountered as discussed below.

5.2.1 Cemented Soil Foundation

After removal of all unsuitable material all grade transitions within cemented foundation soils should be no steeper than 1 horizontal to 1 vertical (1:1). Filling of larger areas may be accomplished with RCC. Isolated areas of foundation may be filled with dental concrete having a 28-day compressive strength of 2,500 pounds per square inch (psi). The final surface should be prepared to allow adequate placement and compaction of mass quantity RCC dam materials so that they may be bonded to the foundation without damaging the foundation during placement.

5.2.2 Rock Foundation

Rock foundation preparation should consist of filling or excavating all areas to achieve a uniform foundation surface where all grade transitions within rock shall be no steeper than 0.5 horizontal to 1 vertical (0.5:1). Filling of larger areas may be accomplished with RCC. Isolated areas of foundation may be filled with dental concrete having a 28-day compressive strength of 2,500 psi. The final surface should be prepared to allow adequate placement and compaction of RCC dam materials so that they may be bonded to the foundation without damaging the foundation during placement.

6.0 TYPICAL SECTION

The proposed RCC embankment cross section, as depicted in Figure 1 and 2, includes upstream and downstream slopes of 0.7H:1V, with a 8-foot-wide crest. The crest of the RCC main dam section will be at elevation 1,812.3 feet (NAVD 88) and the crest of the emergency spillway overflow section will be 1,810.3 feet (NAVD 88). AMEC selected a "Partial" height and "Full" height section at the emergency spillway for analysis positioned at Stations 3+00 and 4+00 respectively (Figures 1 and 2).

To economize the design by reducing the overall height of the dam we are recommending inclusion of an upstream foundation bench. This bench will allow the designer to found the dam at a higher elevation as long as acceptable foundation materials have been reached. The upstream foundation bench provides a buffer between the basin reservoir and direct contact with the dam foundation. The additional foundation protection supports our engineering judgement that allows us to confidently recommend a reduction in the need for over excavation of the dam foundation to an elevation less than the invert of the basin bottom. The bench also provides an added benefit for a longer seepage path beneath the structure, however, our analyis has not quantified this added benefit. The criteria for the bench top width was developed from our engineering judgement and should be as follows:

- Minimum bench width at any location should not be less than 12 foot in width at the top (shown as dimension "B") on Figure 3.
- Calculate "B" as: $\boldsymbol{B = (2 \times H) - (0.7 \times H)}$, where H is the height of dam above the foundation (see Figure 3)

The criteria was established by limiting the slope projected from a point on the upstream crest (*Point A* – Figure 3) to the crest of bench catch point with the basin excavation (*Point B* – Figure 3) to no steeper than 2H: 1V with a minimum bench width of 12 feet.

6.1 Partial Height - Emergency Spillway

A typical RCC dam section was utilized for the seepage and stability analysis for a partial height emergency spillway overflow section. The overall maximum representative RCC dam cross section is 9 feet high measured from the foundation footprint. The section analyzed by AMEC is positioned at dam Station 3+00.

6.2 Full Height - Emergency Spillway

A typical RCC dam section was utilized for the seepage and stability analysis for a full height emergency spillway overflow section. The overall maximum representative RCC cross section is 13 feet high measured from the foundation footprint. The section analyzed by AMEC is positioned at dam Station 4+00.

7.0 DESIGN ANALYSIS

7.1 Design Methodology

The analysis and design of the RCC dam was based on the procedure outlined in the *PCA Design Manual for Small RCC Dams* (PCA, 2003). The dam was analyzed using a two-dimensional stability and stress analysis with anticipated field loading conditions. The cross section was analyzed for sliding, overturning and bearing pressure.

7.2 Loads

Loads considered for the RCC dam deign as recommended in the Design Manual for Small RCC Dams (PCA, 2003) include: dead load, external and internal water pressure, silt and earth pressures, and earthquake loading. The basic loading condition combinations considered in the analysis are as follows:

Loading Condition No. 1: Normal Operating (Usual)

- Headwater elevation at top of emergency spillway crest (elevation 1,810.3 feet)
- Minimum tailwater
- Uplift
- Silt, if applicable

Loading Condition No. 2: Flood Discharge (Unusual)

- Headwater at flood level (elevation 1,811.3 feet)
- Increased tailwater pressure, if applicable
- Uplift
- Silt, if applicable

Loading Condition No. 3: Normal Operating with earthquake (Extreme)

- Horizontal earthquake acceleration in downstream direction
- Headwater elevation at top of emergency spillway crest (elevation 1,810.3 feet)
- Minimum tailwater
- Uplift at pre-earthquake level
- Silt, if applicable

Dead loads were based on the weight of the RCC, which was assumed to be 150 pounds per cubic foot (pcf). The external water pressure component was divided into two segments, the headwater and tailwater component. For all loading conditions, only the headwater was taken into account since no tailwater is expected. In the stability analysis, the overflow from the emergency spillway section was neglected (USACE, 1995). The internal water pressure for uplift was taken as a linear distribution of the pressure from the headwater on the upstream side to the pressure of the tailwater on the downstream side.

Over the life of the structure, sediment accumulation is expected to be minimal due to the Districts on-going Operation and Maintenance, therefore, sediment loading was not considered in the calculations.

The seismic force was selected was based on the District's Seismic Exposure Evaluation Report (AMEC 2002). A sesimc probability coefficient of 0.13 was utilized in the analysis. Since the seismic probability is minor, a dynamic analysis to evaluate transient internal stresses was not performed. The earthquake load was analyzed as the equivalent of a static inertial force that was broken down into two components, the inertial force of the dam due to its own weight, and the inertial force of the reservoir water against the dam.

7.3 Stability

7.3.1 External Static

External static stability analyses of the two typical RCC sections was performed. Soil parameters utilized in the external stability analysis were based on laboratory testing and AMEC's experience with similar soils. AMEC reviewed strength parameters for cemented soils from adjacent projects and the strength parameters reported by RAMM (2012) and selected the more conservative values. Conservative values were selected since cemented soils may exhibit strength loss upon prolonged wetting periods and based on our experioence that use of conservative values would not result in an insufficient design. Therefore, AMEC used a friction angle of 30 degrees and a cohesion of 200 psf for stability analyses, which are consistent with parameters utilized in the nearby Spook Hill Flood Retarding Structure dam rehabilitation design project (AMEC 2006). Three failure criteria were analyzed in accordance with the *Design Manual for Small RCC Dams* (PCA 2003): overturning, sliding, and allowable bearing pressure.

Failure criteria used to analyze the stability of the RCC dam is shown below in Table 2.

Table 2

Stability Criteria (USACE, 1995)

Loading Condition	Minimum Sliding FOS	Resultant Location at Base (Overturning)	Foundation Bearing Pressure
Usual	2.0	Middle $^1/_3$	≤allowable
Unusual	1.7	Middle $^1/_2$	≤allowable
Extreme	1.3	Within Base	≤1.33×allowable

Two typical sections were chosen for analysis as described in Section 6.0. For the Partial height section (Sta. 3+00) two Cases were analyzed; Loading Condition 1 (Case 1) and Loading Condition 3 (Case 2). For the Full height section (Sta. 4+00) two Cases were analyzed; Loading Condition 2 (Case 3) and Loading Condition 3 (Case 4).

A simplified overall geometry of the typical sections was used in external stability calculations, which was more conservative and representative of a worst case scenario. The assumptions made included no structural or landscape fill on the downstream side (assumes eroded away during spillway discharge event) with the RCC structure positioned flat on the cemented foundation and a full basin reservoir pool to the invert of the emergency spillway crest.

Analysis results of each Case and Loading Condition considered are presented in Appendix A, and are tabulated in Table 3. For foundation bearing pressure calculations, allowable bearing pressure was calculated using a factor of safety (FOS) acceptability criteria of greater than 3.0.

Table 3

Results of Stability Analyses

Case	Loading Condition	Description	Computed Sliding FOS	Overturning	Foundation Bearing Pressure
1	1	Partial Height Station 3+00	20.4	OK	OK
2	3	Partial Height Station 3+00	8.1	OK	OK
3	2	Full Height Station 4+00	7.7	OK	OK
4	3	Full Height Station 4+00	4.3	OK	OK

The reults show that all cases meet the required minimum FOS against sliding, location of the resultant force for overturning, and allowable foundation bearing pressure set forth in *Gravity Dam Design* (USACE 1995).

7.3.2 Gravity Retaining Wall

Analysis results of the RCC as a gravity retaining wall with lateral loads exerted by the downstream structural and landscape fills are also presented in Appendix A. For the loading condition a flat homogeneous backfill with a friction angle of 25 degrees, cohesion of zero psf and a unit weight of 110 pcf was assumed. Passive loads were negligible and therefore not considered in the analysis. The wall was analyzed for sliding and overturning considering both active and at rest lateral earth pressures. The gravity retaining wall movement is anticipated to be minimal to none and at rest lateral earth pressures would apply; however, if wall movement is expected, active earth pressures could be relied upon. The results of the analysis are tabulated in Table 4.

Table 4

Results of Retaining Wall Analysis

Case	Description	Computed Sliding FOS	Computed Overturning FOS
At-Rest	Full Height Station 4+00	4.0	21.0
Active	Full Height Station 4+00	5.6	28.9

7.4 Seepage Analysis

Seepage analysis of the two typical RCC embankment sections was completed to assess both steady-state and transient seepage conditions. The seepage analyses were completed using the two-dimensional finite element computer program SEEP/W (GeoSlope International, Inc. 2012) and assigned saturated hydraulic conductivity values for the various elements of the dam and the foundation soils. SEEP/W is a computer code used to model the saturated and unsaturated flow of water within porous materials. Analyses were completed using quadrilateral elements to develop the finite element mesh, and solutions were obtained using four-point integration techniques.

Hydraulic conductivity values were based on the results of laboratory testing reported by RAMM (2012). Hydraulic conductivity values for the RCC dam were selected based on AMEC's experience with similar materials. Considering the short duration that the facility impounds a large storm event and the conservatively estimated hydraulic conductivity values acquired from RAMM's invetigation, the approach is appropriate and acceptable.

Summarized in the Table 4 below are the horizontal hydraulic conductivities that were used in the analyses. The ratio of the horizontal to vertical hydraulic conductivity was assumed to be 10 for all materials except the native alluvium/landscape fill.

Table 5

Saturated Hydraulic Conductivity

Material	Hydraulic Conductivity (ft/day)
RCC Embankment	1.42×10^{-3}
Native Alluvium/Landscape Fill	28.3
Structural Backfill	2.8×10^{-1}
Cemented Foundation	2.83

The results of steady-state and transient seepage analyses, assuming the upstream water surface is at elevation 1810.3 feet (maximum pool level), are shown in Appendix B.

7.4.1 Steady State Analysis

Steady state seepage analysis and exit gradient calculations were completed for both typical sections (Partial Height and Full Height) for a "normal" case that assumed a full basin reservoir pool to the invert of the emrgency spillway and the presence of the downstream structural fill, landscape fill, and native alluvium. At the request of the District an "extreme" steady state case was also analyzed which assumed a full basin reservoir pool to the invert of the emergency spillway and no material whatsoever downstream of the RCC dam. The following subsections discuss the results of these analysis.

7.4.1.1 Partial Height (Sta. 3+00)

Normal Case

As indicated by the steady state seepage analysis, shown in Figure B-1 (Appendix B), a phreatic surface could develop and exit at the downstream toe of the dam (coincident with ground line on downstream side) should a very prolonged impoundment event occur as a result of an obstructed principal outlet.

The computed exit gradient for the Partial height normal case is 0.07, which is less than unity, generally considered to result in piping, representing a safety factor of about 14. Typically a safety factor against piping of greater than 3.0 is desired due to variability with the behavior of soil particles and their resistance to movement. Figure B-2 shows the location of the exit gradient along with displaying flow and equipotential lines. The flow and equipotential lines were used to independently calculate the exit gradient which was compared to the exit gradient computed by SEEP/W. The hand check calculations are included in Appendix B.

Extreme Case

The same steady state seepage and exit gradient calculations were performed, shown in Figures B-3 and B-4 (Appendix B) with supporting calculations also in Appendix B, for the extreme case. The computed exit gradient for the Partial height extreme case is 0.47, representing a safety factor of about 2.1.

7.4.1.2 Full Height (Sta. 4+00)

Normal Case

As indicated by the steady state seepage analysis, shown in Figure B-5 (Appendix B), a phreatic surface could develop and exit at the downstream toe of the dam (coincident with ground line on downstream side) should a very prolonged impoundment event occur as a result of an obstructed principal outlet.

The computed exit gradient for the Full height normal case is 0.03, representing a safety factor of about 33. Figure B-6 shows the location of the exit gradient along with displaying flow and equipotential lines. The flow and equipotential lines were used to independently calculate the exit gradient which was compared to the exit gradient computed by SEEP/W. The hand check calculations are included in Appendix B.

Extreme Case

The same steady state seepage and exit gradient calculations were performed, shown in Figures B-7 and B-8 (Appendix B) with supporting calculations also in Appendix B, for the extreme case. The computed exit gradient for the Full height extreme case is 0.63, representing a safety factor of about 1.6.

7.4.2 Transient Analysis

As shown by the transient seepage analysis, a full phreatic surface is not anticipated to develop through the RCC dam; however, water may reach the downstream side of the dam in less than 10 days for both Typical section. As indicated by Figure B-9 and B-10, for the Partial height it takes about 6 days to reach the downstream side and about 8 days for the Full height section.

7.5 Settlement

AMEC's review of RAMM's data indicates that the foundation soils have a very limited collapse potential. The SPT blow counts support the lack of collapse potential in the foundation soils. SPT values were typically greater than 30, often with refusal blow counts. Typically SPT N-values greater than 30 indicate soils of sufficient integrity to exhibit much less collapse during wetting. As a result, the cemented and rock foundation soils appear competent and resistant to significant volume changes during loading and/or wetting. Minor differential settlement may occur as a result of periodic impoundments and associated foundation wetting events. As per RAMM's conclusions, the estimated total and differential footing settlement anticipated is less than 0.25 inches (RAMM 2012).

7.6 RCC Compressive Strength

The objective of selecting the design compressive strength is to produce a RCC material that satisfies performance requirements and is economical. For small RCC dams the desired physical properties of the mix depend largely on the design selected for the structure and its geographic location. For small RCC dams the main consideration is whether the RCC will be exposed to the weather or covered with conventional air-entrained concrete. Furthermore, the size of the structure and therefore the volume of RCC required also plays an important role in the selection of materials and mixture proportions. For small RCC dams more consideration needs to be given to reducing the cost of aggregate and minimizing equipment requirements that reducing the amount of Portland cement in the mix (PCA 2003). Most RCC projects utilize mixtures that yield an average compressive strength between 2,000 to 3,000 pounds per square inch (psi) at 90-days to 1-year (USACE 2000).

The Oak Stream dam will not be subject to repeated freeze-thaw cycles and will not be exposed to moisture from basin storage on a frequent basis. Therefore, durability of the RCC is not a concern. Furthermore, the anticipated flow over the emergency spillway section has a peak discharge of 830 cubic feet per second (cfs) for a 24-hour duration (G&B 2012). However, for much of the rising and receding limbs of the hydrograph there is nominal flow over the emergency spillway. As a result of the limited flow and limited need for durability, AMEC recommends the use of an RCC mix that provides a 90-day compressive strength of 2,500 psi.

7.7 Shrinkage & Cracking

PCA (2003) states that RCC dams will develop transverse cracks as there is a volume reduction due to a drop in temperature of the structure coupled with restraint, usually provided at the base of the structure. The cracks tend to start at the restrained base and proceed upward to the crest resulting in a full section transverse crack. Some surface cracking may also occur, due to loss of moisture from the concrete to the atmosphere. Transverse cracks do not pose any threat to the stability or safety of the dam. Cracks can, however, pose a problem if they are wide enough to pass water. Considering the purpose of the RCC dam is for flood control leakage through any transverse crack that may develop would not be of concern. Therefore, special treatment or design considerations related to shrinkage cracking are not required.

7.8 Lift Treatments

Shear resistance at lift lines is rarely a controlling factor in low head RCC dams (PCA 2003). Therefore, special treatment of individual lifts is not required. However, should there be a delay between placements of successive lifts a "cold-joint" could occur resulting in loss of shear resistance between lifts. "Cold-joints" may occur; as a result of exposure time prior to coverage by the next lift, RCC surface temperature, and placement temperatures. The realization that time and temperature effect shear resistance at the lift line has produced a concept termed joint-maturity. Joint-maturity is expressed in Fahrenheit degree-hours and is the product of the surface temperature and time of exposure until the next lift is placed. Small RCC dams commonly are built quickly, have lesser joint shear strength requirements, and incorporate higher cement content mix designs than larger RCC dams. Therefore, the design engineer can

in most cases simplify joint maturity requirements to exposure hours (PCA 2003). For Oak Street Basin three lift (joint) types are defined, the recommended time of exposure before joint treatment is recommended, and lift treatments are as discussed in the following table.

Joint Type	Exposure Time (ET)	Recommended Lift Treatment[1]
Fresh Joint	ET < 4 hrs	Method I
Intermediate Joint	4 hrs < ET < 16 hrs	Method II
Cold Joint	>16 hrs	Method III

[1]Treatment Definitions

Method I: Loose materials and contaminants should removed from the lift joint surface. The lift joint surface shall be cleaned using moderate-pressure air immediately before spreading the next RCC lift. Maintain the surface in a moistened condition.

Method II: Perform Treatment Method I. Before the placement of RCC on the surface, uniformly distribute a layer of dry Portland cement over the surface applied at a rate of 0.5 to 1 pound per square foot of surface followed by application of a sufficient quantity of water to dampen all of the cement. The cement paste should not be exposed on the surface for more than 10 minutes before being covered with RCC.

Method III: Perform Treatment Method I. Before the placement of RCC, the joint surface should be covered with a layer of the bonding mortar with a thickness of 0.25 to 0.5 inch. The bonding mortar shall be covered with the next layer or lift of RCC while the mortar is still fluid. In no case shall the bonding mortar remain uncovered for more than 30 minutes.

8.0 RCC MIX DESIGN

Appendix C includes calculations and laboratory test data in support of trial RCC mix designs.

9.0 CONSTRUCTION SPECIFICATIONS

Recommended specifications for Foundation Excavation, Preparation and Cleaning and a specification for Roller Compacted Concrete are included in Appendix D.

10.0 CONCLUSIONS

Exit gradients at the downstream toe for the "extreme" steady state seepage condition were less than the commonly accepted safety factor of 3.0. Nevertheless, the factors of safety for both the Partial height and Full height extreme cases were greater than 1.0 where factors of safety less than 1.0 would indicated soil piping concern. Its is AMEC's opinion that the design as recommended is acceptable and does not create any credible potential failure modes associated with piping during the "extreme" case for the following reasons:

- Piping is of most concern in the presence of uncemented fine grained soil foundation materials. The RCC dam at this site will be founded on an engineer approved cemented soil foundation where loose, poor foundation soils will be removed during construction which results in a foundation much less susceptable to piping.

- The "extreme" case assumes all materials have eroded from downstream of the dam to an elevation equivalent to the downstream RCC toe. Due to the adjacent existing ground elevations directly downstream of the dam (~8 feet above the toe elevation) the opportunity for a scour hole to develop to a significant depth is remote.

- The "extreme" case assumes the principal outlet has been obstructed and the reservoir is full to the invert of the emergency spillway. Should the outlet become obstructed the District's Operation & Maintence (O&M) Division would likely mobilize to the site to clear the obstruction and/or begin draining the reservoir by pumping. Transient seepage analysis indicate migration of a wetting front to the downstream toe would not occur prior to 4-days. This timeframe is sufficient to allow mobilization and mitigation of a full basin reservoir pool.

11.0 RECOMMENDATIONS

Based on AMEC's conclusions discussed in Section 10.0 we recommend the following be included in the project O&M manual:

- The area immediately downstream of the emergency spillway sections should be inspected and repaired immediately after a discharge event.

- The reservoir storage level and operation of the principal outlet should be monitored during flood events and actions taken as necessary to clear any outlet obstructions.

- Planning for the mobilization and use of a pump capable of draining the reservoir should be considered and excersized periodically.

12.0 REFERENCES

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FIGURES

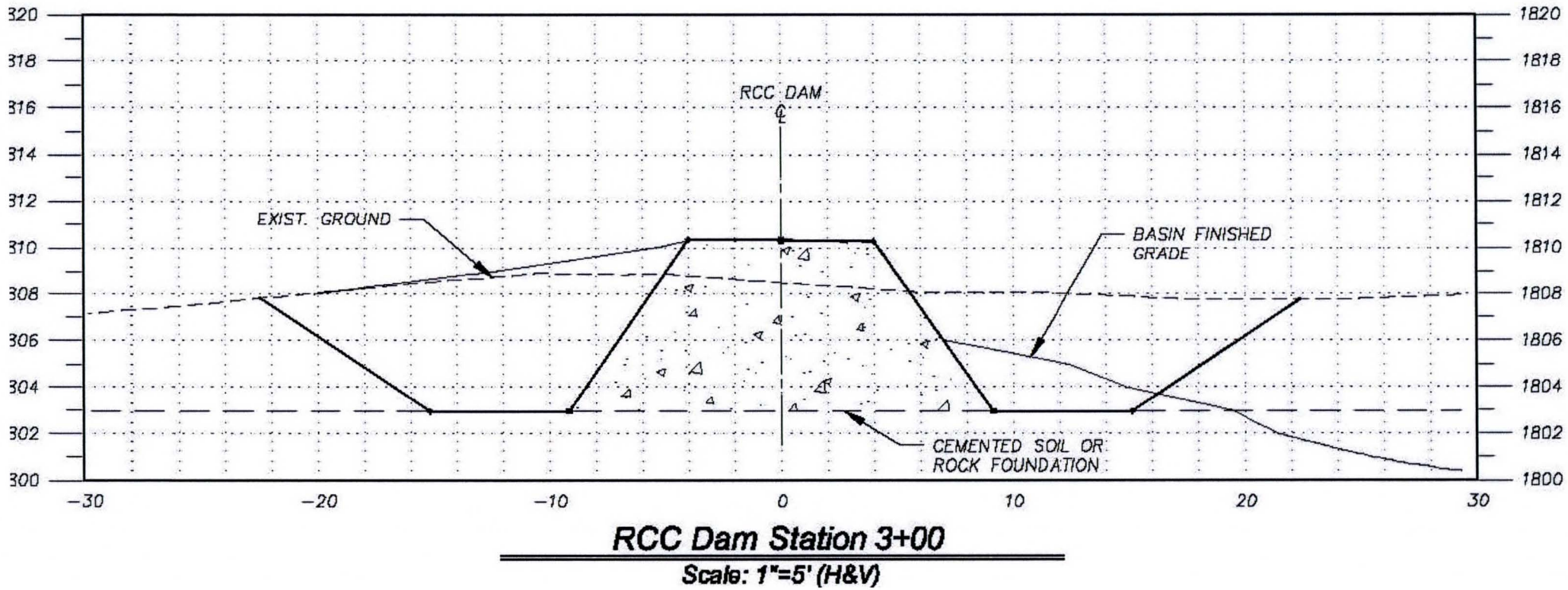


Figure 1 - Partial Height - Emergency Spillway Section

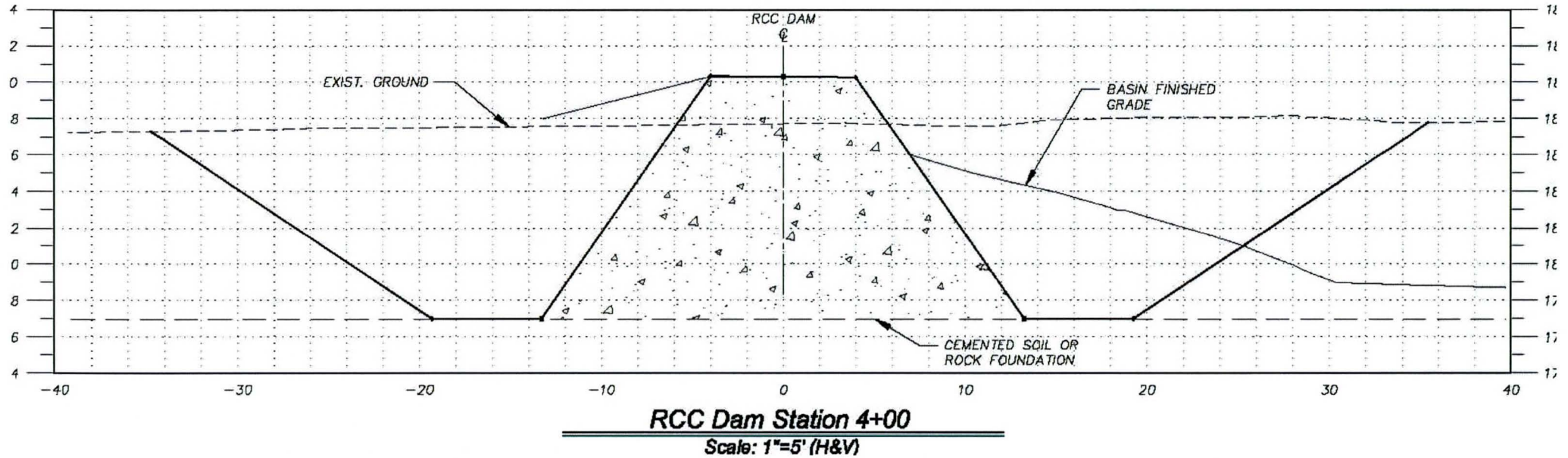
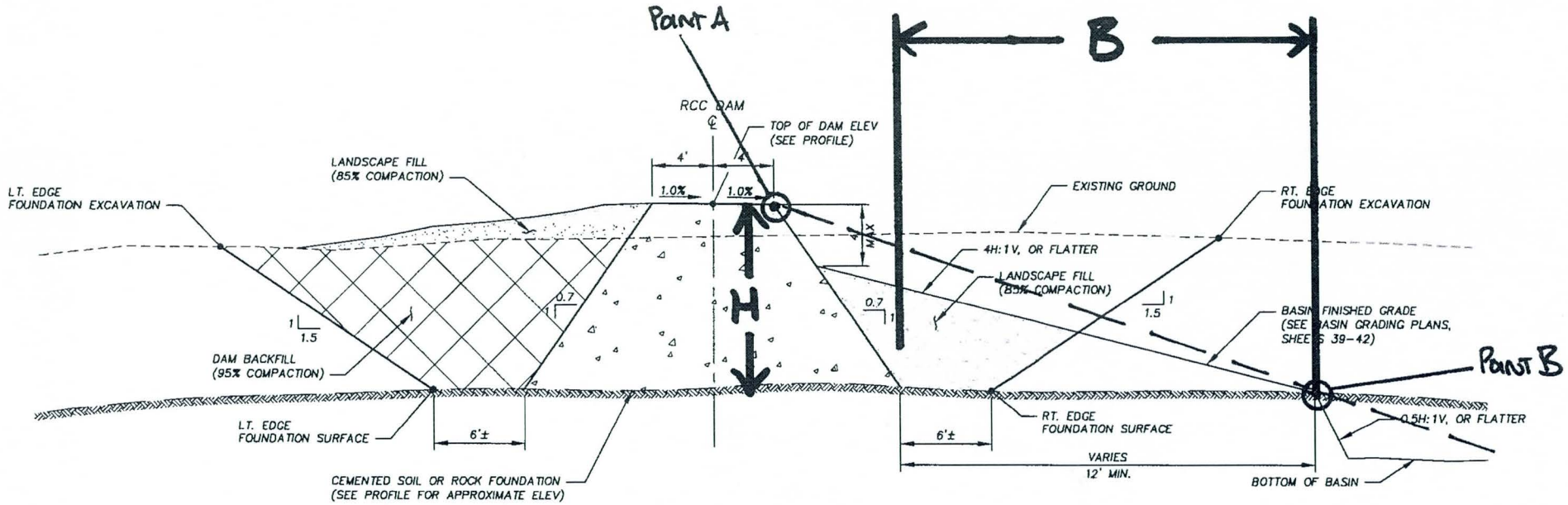


Figure 2 - Full Height - Emergency Spillway Section



Typical Section - RCC Dam
Sta 1+90 to Sta 7+50

$B = 2H - 0.7H$

AND MIN 12'

Figure 3 - Bench Width Criteria

APPENDIX A

EXTERNAL STABILITY ANALYSIS

CALCULATIONS/DOCUMENT REVIEW COVER SHEET

Subject: Oak St. Basin RCC Dam

Project No. 17-2011-4050
Sheet 1 of 13

External Stability Analysis
(title of calculations/document)

Calculations/Document By: NAIDA CAUSEVIC (print name)
(signature) 9/25/12 (date)

Assumptions Checked by Consultant/Senior Personnel: BRETT HAWEY (print name)
(signature) 9/26/12 (date)

Calculations/Document Checked by: BRETT HAWEY (print name)
(signature) 9/26/12 (date)

Approved by: BRETT HAWEY (print name)
(signature) 9/26/12 (date)

Approval Notice: If Calculations/Document are only spot checked, do not require checking, or are assumed to be correct by experience of engineering judgment, it should be noted here.

Revision Number	Date	By	Checked By (Initials)	Approval

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC

Input Parameters

RCC Dam Section Station 3+00
Loading Condition 1

RCC Dam Geometry

E_T	1810.3	[ft]	Elevation (top of dam)
E_B	1803	[ft]	Elevation (bottom of dam)
H_D	7.3	[ft]	Height of Dam
b	18.22	[ft]	Base of Dam
tc	8	[ft]	Crest Width
S	0.70	[dim]	Slope - (SH:1V) - downstream
$\gamma_{concrete}$	150	[pcf]	Unit Weight of Concrete (140-160 pcf)
$\phi_{concrete}$	45	[degrees]	Angle of internal friction of concrete
$c_{concrete}$	10	(psi)	Cohesion of concrete

External Water Pressure

γ_{water}	62.4	[pcf]	Unit Weight of Water
E_{HW}	1810.3	[ft]	Elevation of Reservoir Water Surface
h	7	[ft]	Headwater at the heel (upstream end of the dam)
E_{TW}	1803	[ft]	Elevation of Tailwater Surface
h_1	0	[ft]	Tailwater at the toe (downstream end of the dam)

Silt and Earth Pressure

α		[ft]	Height of Silt Deposit
K_0		[dim]	Coefficient of Lateral Earth Pressure at Rest
γ_{silt}		[pcf]	Buoyant Unit Weight of Silt
ϕ_{silt}		[degrees]	Angle of internal friction of silt (not used for calculations)

Seismic Forces

a		[dim]	Seismic Coefficient (see Figure 4.3)
y		[ft]	
C_{eq}		[dim]	factor depending generally on water depth and earthquake vibration period

Foundation Parameters

Soil

$\gamma_{foundation}$	120	(pcf)	Unit weight of foundation soils
$\phi_{foundation}$	30	(degrees)	Angle of internal friction of foundation soils
$c_{foundation}$	200	(psf)	Cohesion of foundation soils
D_f	0	(ft)	embedment of foundation below original grade
γ'	58	(pcf)	Affect of water
q_{fdn}	0	(psf)	overburden pressure for bearing capacity
N_q	18.40	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_c	30.14	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_γ	22.40	(dim)	Bearing Capacity Factor Vesic (1973, 1975)

Rock

q_{all}	10	(tsf)	Allowable Contact Stress
q_{ult}	use qult (soil)	(tsf)	Ultimate Bearing Capacity
N_{ms}	use q_{ult} (soil)	(dim)	Coefficinet
C_o	10	(ksf)	Uniaxial Compressive Strength

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 3+00
Loading Condition: 1

Section Shape: d Use a, b, c or d - See Table 5.1

Summation of Forces

Loading	Horizontal Forces		Vertical Forces		Stabilizing Moments	Overturning Moments
	Force [kips]	Arm [ft]	Force [kips]	Arm [ft]	[ft-k]	[ft-k]
Weight of Dam (W_c)			14.35545	see weight of dam calculations	131	
Uplift (U)			-1.2	see uplift calculations		14.1
Headwater (H)	1.7	2.4				4.0
Silt (S)	0.0	0.0				0.0
Earthquake Interia Force (P_{eq})	0.0	see seismic calculations				0
Earthquake Hydrodynamic Force (Pw_{eq})	0.0	3.285				0.0
Sum	1.7		13.2		131	18.2
					Net Moment	112.6

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 3+00
Loading Condition: 1

Sliding:

Failure Criteria

Load Condition	USACE	FERC/USBR	
		High Hazard Dam	Low Hazard Dam
Usual	2.00	3.00	2.00
Unusual	1.70	2.00	1.25
Extreme	1.30	1.00	1.00
F.S = 20.4	**OK**	**OK**	**OK**

Overturning:

Eccentricity	0.57	[ft]	**OK**
Base length	18.22	[ft]	

Bearing Pressure:

F.S = 3

q_{ult}	590	ksf	Toe (q_{max})	0.86 [ksf]	**OK**
$q_{allowable}$	196.8	[ksf]	Heel (q_{min})	0.59 [ksf]	

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC

Input Parameters

RCC Dam Section Station 3+00
Loading Condition 3

RCC Dam Geometry

E_T	1810.3	[ft]	Elevation (top of dam)
E_B	1803	[ft]	Elevation (bottom of dam)
H_D	7.3	[ft]	Height of Dam
b	18.22	[ft]	Base of Dam
tc	8	[ft]	Crest Width
S	0.70	[dim]	Slope - (SH:1V) - downstream
$\gamma_{concrete}$	150	[pcf]	Unit Weight of Concrete (140-160 pcf)
$\phi_{concrete}$	45	[degrees]	Angle of internal friction of concrete
$c_{concrete}$	10	(psi)	Cohesion of concrete

External Water Pressure

γ_{water}	62.4	[pcf]	Unit Weight of Water
E_{HW}	1810.3	[ft]	Elevation of Reservoir Water Surface
h	7	[ft]	Headwater at the heel (upstream end of the dam)
E_{TW}	1803	[ft]	Elevation of Tailwater Surface
h_1	0	[ft]	Tailwater at the toe (downstream end of the dam)

Silt and Earth Pressure

α		[ft]	Height of Silt Deposit
K_0		[dim]	Coefficient of Lateral Earth Pressure at Rest
γ_{silt}		[pcf]	Buoyant Unit Weight of Silt
ϕ_{silt}		[degrees]	Angle of internal friction of silt (not used for calculations)

Seismic Forces

a	0.130	[dim]	Seismic Coefficient (see Figure 4.3)
y	7	[ft]	
C_{eq}	0.150	[dim]	factor depending generally on water depth and earthquake vibration period

Foundation Parameters

Soil

$\gamma_{foundation}$	120	(pcf)	Unit weight of foundation soils
$\phi_{foundation}$	30	(degrees)	Angle of internal friction of foundation soils
$c_{foundation}$	200	(psf)	Cohesion of foundation soils
D_f	0	(ft)	embedment of foundation below original grade
γ'	58	(pcf)	Affect of water
q_{fdn}	0	(psf)	overburden pressure for bearing capacity
N_q	18.40	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_c	30.14	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_γ	22.40	(dim)	Bearing Capacity Factor Vesic (1973, 1975)

Rock

q_{all}	10	(tsf)	Allowable Contact Stress
q_{ult}	use qult (soil)	(tsf)	Ultimate Bearing Capacity
N_{ms}	use q_{ult} (soil)	(dim)	Coefficinet
C_o	10	(ksf)	Uniaxial Compressive Strength

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 3+00
Loading Condition: 3

Section Shape: d Use a, b, c or d - See Table 5.1

Summation of Forces

Loading	Horizontal Forces		Vertical Forces		Stabilizing Moments	Overturning Moments
	Force [kips]	Arm [ft]	Force [kips]	Arm [ft]	[ft-k]	[ft-k]
Weight of Dam (W_c)			14.35545	see weight of dam calculations	131	
Uplift (U)			-1.2	see uplift calculations		14.1
Headwater (H)	1.7	2.4				4.0
Silt (S)	0.0	0.0				0.0
Earthquake Interia Force (P_{eq})	1.9	see seismic calculations				6
Earthquake Hydrodynamic Force (Pw_{eq})	0.7	3.285				2.1
Sum	4.2		13.2		131	26.2
					Net Moment	104.5

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 3+00
Loading Condition: 3

Sliding:

Failure Criteria

Load Condition	USACE	FERC/USBR	
		High Hazard Dam	Low Hazard Dam
Usual	2.00	3.00	2.00
Unusual	1.70	2.00	1.25
Extreme	1.30	1.00	1.00
F.S = 8.1	OK	OK	OK

Overturning:

Eccentricity	1.19	[ft]	OK
Base length	18.22	[ft]	

Bearing Pressure:

F.S = 3

q_{ult}	590	ksf	Toe (q_{max})	1.01 [ksf]
$q_{allowable}$	196.8	[ksf]	Heel (q_{min})	0.44 [ksf]

OK

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC

Input Parameters

RCC Dam Section Station 4+00
Loading Condition 2

RCC Dam Geometry

E_T	1810.3	[ft]	Elevation (top of dam)
E_B	1795	[ft]	Elevation (bottom of dam)
H_D	15.3	[ft]	Height of Dam
b	29.42	[ft]	Base of Dam
tc	8	[ft]	Crest Width
S	0.70	[dim]	Slope - (SH:1V) - downstream
$\gamma_{concrete}$	150	[pcf]	Unit Weight of Concrete (140-160 pcf)
$\phi_{concrete}$	45	[degrees]	Angle of internal friction of concrete
$c_{concrete}$	10	(psi)	Cohesion of concrete

External Water Pressure

γ_{water}	62.4	[pcf]	Unit Weight of Water
E_{HW}	1811.3	[ft]	Elevation of Reservoir Water Surface
h	16	[ft]	Headwater at the heel (upstream end of the dam)
E_{TW}	1796	[ft]	Elevation of Tailwater Surface
h_1	1	[ft]	Tailwater at the toe (downstream end of the dam)

Silt and Earth Pressure

α		[ft]	Height of Silt Deposit
K_0		[dim]	Coefficient of Lateral Earth Pressure at Rest
γ_{silt}		[pcf]	Buoyant Unit Weight of Silt
ϕ_{silt}		[degrees]	Angle of internal friction of silt (not used for calculations)

Seismic Forces

a		[dim]	Seismic Coefficient (see Figure 4.3)
y		[ft]	
C_{eq}		[dim]	factor depending generally on water depth and earthquake vibration period

Foundation Parameters

Soil

$\gamma_{foundation}$	120	(pcf)	Unit weight of foundation soils
$\phi_{foundation}$	30	(degrees)	Angle of internal friction of foundation soils
$c_{foundation}$	200	(psf)	Cohesion of foundation soils
D_f	0	(ft)	embedment of foundation below original grade
γ'	58	(pcf)	Affect of water
q_{fdn}	0	(psf)	overburden pressure for bearing capacity
N_q	18.40	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_c	30.14	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_γ	22.40	(dim)	Bearing Capacity Factor Vesic (1973, 1975)

Rock

q_{all}	10	(tsf)	Allowable Contact Stress
q_{ult}	use qult (soil)	(tsf)	Ultimate Bearing Capacity
N_{ms}	use q_{ult} (soil)	(dim)	Coefficinet
C_o	10	(ksf)	Uniaxial Compressive Strength

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 4+00
Loading Condition: 2

Section Shape: d Use a, b, c or d - See Table 5.1

Summation of Forces

Loading	Horizontal Forces		Vertical Forces		Stabilizing Moments	Overturning Moments
	Force [kips]	Arm [ft]	Force [kips]	Arm [ft]	[ft-k]	[ft-k]
Weight of Dam (W_c)			42.93945	see weight of dam calculations	632	
Uplift (U)			-5.8	see uplift calculations		90.4
Headwater (H)	8.3	5.4				45.0
Silt (S)	0.0	0.0				0.0
Earthquake Interia Force (P_{eq})	0.0	see seismic calculations				0
Earthquake Hydrodynamic Force (Pw_{eq})	0.0	7.335				0.0
Sum	8.3		37.2		632	135.5
					Net Moment	496.2

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 4+00
Loading Condition: 2

Sliding:

Failure Criteria

Load Condition	USACE	FERC/USBR	
		High Hazard Dam	Low Hazard Dam
Usual	2.00	3.00	2.00
Unusual	1.70	2.00	1.25
Extreme	1.30	1.00	1.00
F.S = 7.7	OK	OK	OK

Overturning:

Eccentricity	1.36	[ft]	OK
Base length	29.42	[ft]	

Bearing Pressure:

F.S = 3

q_{ult}	598	ksf	Toe (q_{max})	1.61	[ksf]
$q_{allowable}$	199.2	[ksf]	Heel (q_{min})	0.91	[ksf]

OK

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC

Input Parameters

RCC Dam Section Station 4+00
Loading Condition 3

RCC Dam Geometry

E_T	1810.3	[ft]	Elevation (top of dam)
E_B	1795	[ft]	Elevation (bottom of dam)
H_D	15.3	[ft]	Height of Dam
b	29.42	[ft]	Base of Dam
tc	8	[ft]	Crest Width
S	0.70	[dim]	Slope - (SH:1V) - downstream
$\gamma_{concrete}$	150	[pcf]	Unit Weight of Concrete (140-160 pcf)
$\phi_{concrete}$	45	[degrees]	Angle of internal friction of concrete
$c_{concrete}$	10	(psi)	Cohesion of concrete

External Water Pressure

γ_{water}	62.4	[pcf]	Unit Weight of Water
E_{HW}	1811.3	[ft]	Elevation of Reservoir Water Surface
h	16	[ft]	Headwater at the heel (upstream end of the dam)
E_{TW}	1796	[ft]	Elevation of Tailwater Surface
h_1	1	[ft]	Tailwater at the toe (downstream end of the dam)

Silt and Earth Pressure

α		[ft]	Height of Silt Deposit
K_0		[dim]	Coefficient of Lateral Earth Pressure at Rest
γ_{silt}		[pcf]	Buoyant Unit Weight of Silt
ϕ_{silt}		[degrees]	Angle of internal friction of silt (not used for calculations)

Seismic Forces

a	0.130	[dim]	Seismic Coefficient (see Figure 4.3)
y	16	[ft]	
C_{eq}	0.051	[dim]	factor depending generally on water depth and earthquake vibration period

Foundation Parameters

Soil

$\gamma_{foundation}$	120	(pcf)	Unit weight of foundation soils
$\phi_{foundation}$	30	(degrees)	Angle of internal friction of foundation soils
$c_{foundation}$	200	(psf)	Cohesion of foundation soils
D_f	0	(ft)	embedment of foundation below original grade
γ'	58	(pcf)	Affect of water
q_{fdn}	0	(psf)	overburden pressure for bearing capacity
N_q	18.40	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_c	30.14	(dim)	Bearing Capacity Factor Meyerhof (1963)
N_γ	22.40	(dim)	Bearing Capacity Factor Vesic (1973, 1975)

Rock

q_{all}	10	(tsf)	Allowable Contact Stress
q_{ult}	use qult (soil)	(tsf)	Ultimate Bearing Capacity
N_{ms}	use q_{ult} (soil)	(dim)	Coefficinet
C_o	10	(ksf)	Uniaxial Compressive Strength

External Stability - Sliding, Overturning and Bearing Capacity
Developed By: Naida Causevic
Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 4+00
Loading Condition: 3

Section Shape: d Use a, b, c or d - See Table 5.1

Summation of Forces

Loading	Horizonatal Forces		Vertical Forces		Stabilizing Moments	Overturning Moments
	Force [kips]	Arm [ft]	Force [kips]	Arm [ft]	[ft-k]	[ft-k]
Weight of Dam (W_c)			42.93945	see weight of dam calculations	632	
Uplift (U)			-5.8	see uplift calculations		90.4
Headwater (H)	8.3	5.4				45.0
Silt (S)	0.0	0.0				0.0
Earthquake Interia Force (P_{eq})	5.6	see seismic calculations				35
Earthquake Hydrodynamic Force (Pw_{eq})	1.1	7.335				8.4
Sum	15.0		37.2		632	178.4
					Net Moment	453.2

Project Name: Oak Street Detention Basin and Storm Drain Project
Project No.: 17-2011-4050
Date: 9/25/2012
User: NC
Station: 4+00
Loading Condition: 3

Sliding:

Failure Criteria

Load Condition	USACE	FERC/USBR	
		High Hazard Dam	Low Hazard Dam
Usual	2.00	3.00	2.00
Unusual	1.70	2.00	1.25
Extreme	1.30	1.00	1.00
F.S = 4.3	OK	OK	OK

Overturning:

Eccentricity	2.51	[ft]	OK
Base length	29.42	[ft]	

Bearing Pressure:

F.S = 3

q_{ult}	598	ksf	Toe (q_{max})	1.91 [ksf]
$q_{allowable}$	199.2	[ksf]	Heel (q_{min})	0.62 [ksf]

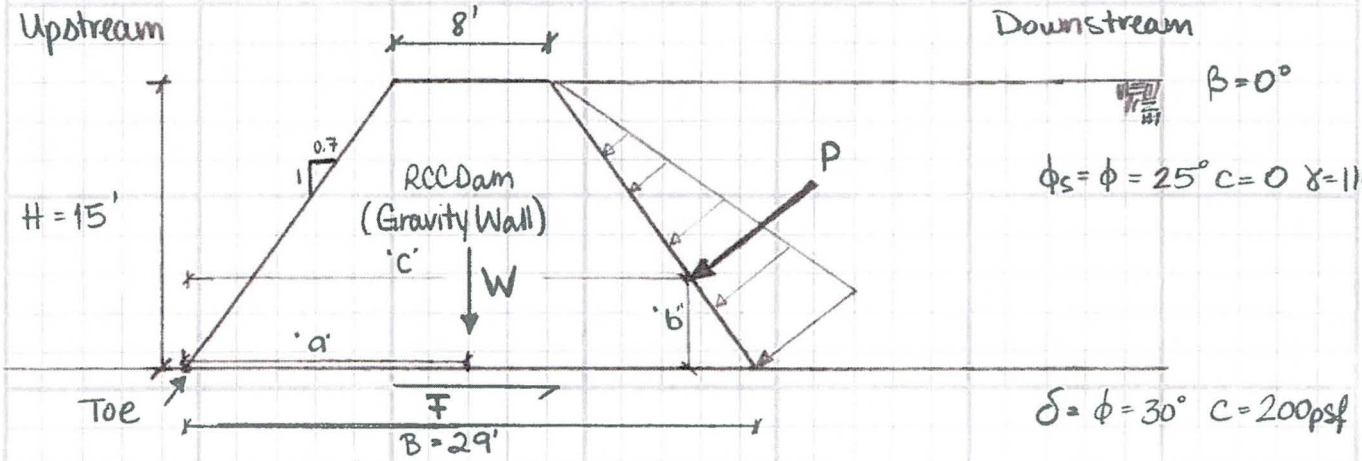
OK

Data Source: AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

* Considered both at rest and active earth pressure conditions *

Gravity Wall Section at 4+00

Assumptions :- Structural Fill / Native Alluvium horizontal and at the same height as gravity wall
- Use soil parameters of Native Alluvium only ($\phi = 25$, $c = 0$)
- Ignore Passive Force



Resistance to Sliding

$F = (W + P_v)\tan\delta + \vec{P_p}^{0}$ $\quad P_v = P\sin\phi_s$

→ At rest Earth Pressure

$K_o = 1 - \sin\phi_s = 1 - \sin 25 = 0.577$ ✓

$P_o = \frac{1}{2} K_o \gamma H^2 = \frac{1}{2}(0.577)(110\ lb/ft^3)(15\ ft)^2 = 7140.38\ lb/ft = 7.14\ kips/ft$ ✓

→ Active Earth Pressure

$K_a = \tan^2(45 - \phi_s/2) = \tan^2(45 - 25/2) = 0.406$ ✓

$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2}(0.406)(110\ lb/ft^3)(15\ ft)^2 = 5024.25\ lb/ft = 5.02\ kips/ft$ ✓

amec

Oak Street Detention Basin and Storm Drain

RCC Dam Design

Weight of Wall $= [2(\frac{1}{2}(15' \times 0.7)(15)) + (15' \times 8')] \times 150\ lb/ft^3$

$= 41{,}625\ lb/ft = 41.63\ kips/ft$ ✓

$F_o = (W + P_o \sin\phi_s)\tan\delta$

$= (41.63\ kips/ft + 7.14\ kips/ft \times \sin 25)\tan 30$

$= 25.78\ kips/ft$ ✓

$F_a = (W + P_a \sin\phi_s)\tan\delta$

$= (41.63\ kips/ft + 5.02\ kips/ft \times \sin 25)\tan 30$

$= 25.26\ kips/ft$ ✓

$FS = \frac{F}{P_H} \geq 1.5$

$P_H = P\cos\phi_s$

→ At Rest F.S

$F.S = \frac{25.78\ kips/ft}{(7.14\ kips/ft)(\cos 25)} = 4.0 \geq 1.5$ ✓ OK

→ Active F.S

$F.S = \frac{25.26\ kips/ft}{(5.02\ kips/ft)(\cos 25)} = 5.6 \geq 1.5$ ✓ OK

Oak Street Detention Basin and Storm Drain

RCC Dam Design

Resistance to Overturning

$$FS = \frac{W_R a + P_V \times c}{P_H \times b} \geq 2.0$$

Sum moments about toe:

→ At rest F.S.

$$F.S = \frac{(4.63^{kips/ft})\left(\frac{29'}{2}\right) + (7.14^{kips/ft} \times \sin 25^\circ)\left(29' - \left(\frac{15'}{3}\right)\cdot 0.7\right)}{(7.14^{kips/ft} \times \cos 25^\circ)\left(\frac{15'}{3}\right)}$$

$$= \frac{680.58^{kips\cdot ft/ft}}{32.36^{kips\cdot ft/ft}} = 21.0 \geq 2.0 \qquad \underline{OK}$$

→ Active F.S

$$F.S = \frac{(41.63^{kips/ft})(29'/2) + (5.02^{kips/ft} \times \sin 25^\circ)\left(29' - \left(\frac{15'}{3}\right)\cdot 0.7\right)}{(5.02^{kips/ft} \times \cos 25^\circ)\left(\frac{15'}{3}\right)}$$

$$= \frac{657.73^{kips\,ft/ft}}{22.75^{kips\,ft/ft}} = 28.9 \geq 2.0 \qquad \underline{OK}$$

Designed By: Naida Causevic

MODIFICATION HISTORY

Version No.	Date	Changes

References

Design Manual for Small RCC Gravity Dams - Portland Cement Association (PCA), 2003, Report EB225.01, Portland Cement Association, Skokie, Illinois.
AASHTO LRFD Bridge Desing Specifications - American Association of State Highway and Transportation Officials (AASHTO), Fifth Edition, 2010, Part II: Section 6 - Index, American Association of State Highway and Transportation Officials, Washington D.C.
AASHTO Standard Specifications for Highway Bridges - American Association of State Highway and Transportation Officials (AASHTO), 17th Edition, 2002, American Association of State Highway and Transportation Officials, Washington D.C.

This section shows the calculation procedure followed for the RCC Dam External Stability Desgin

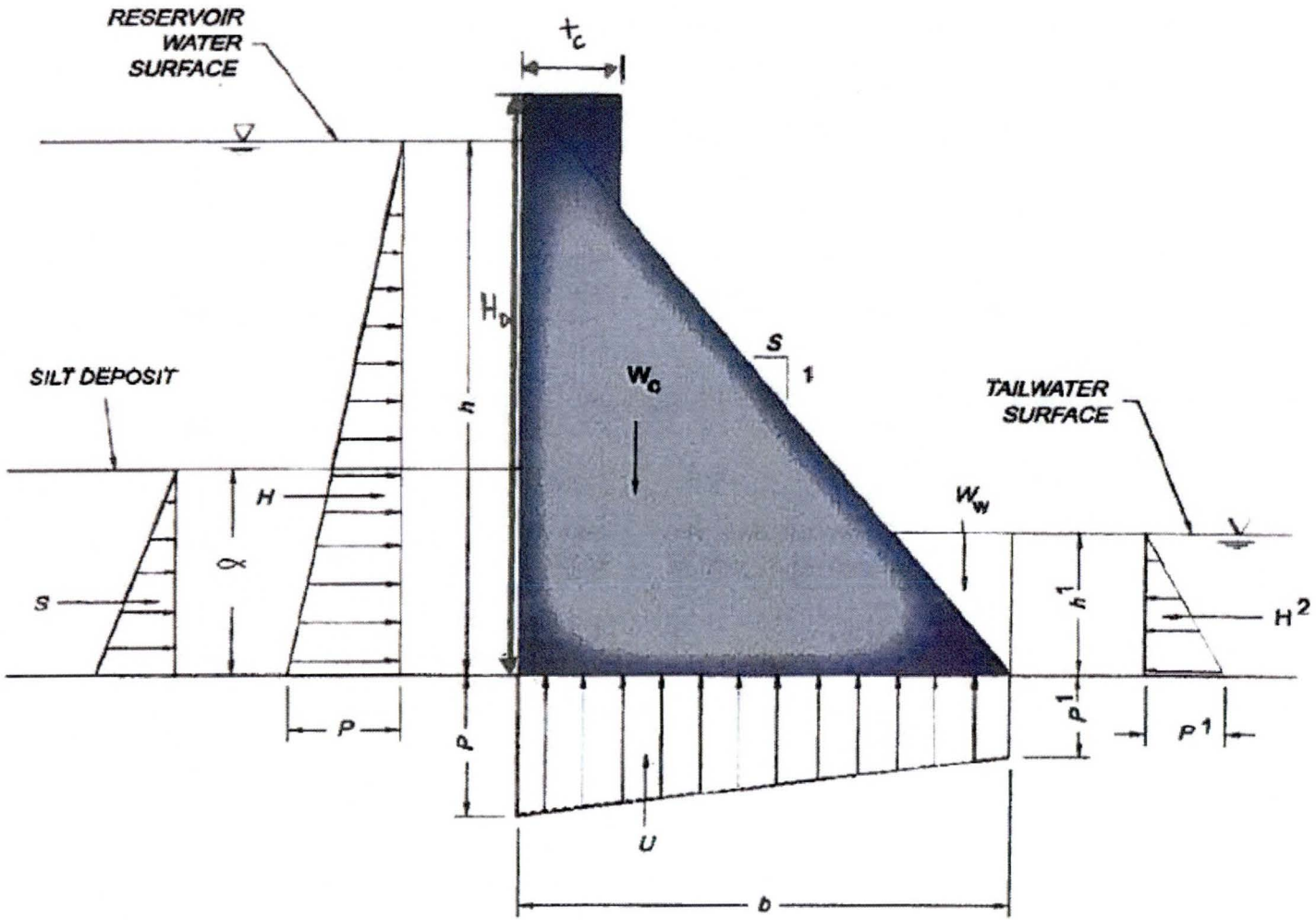


Figure 4.1. Loads on typical gravity section

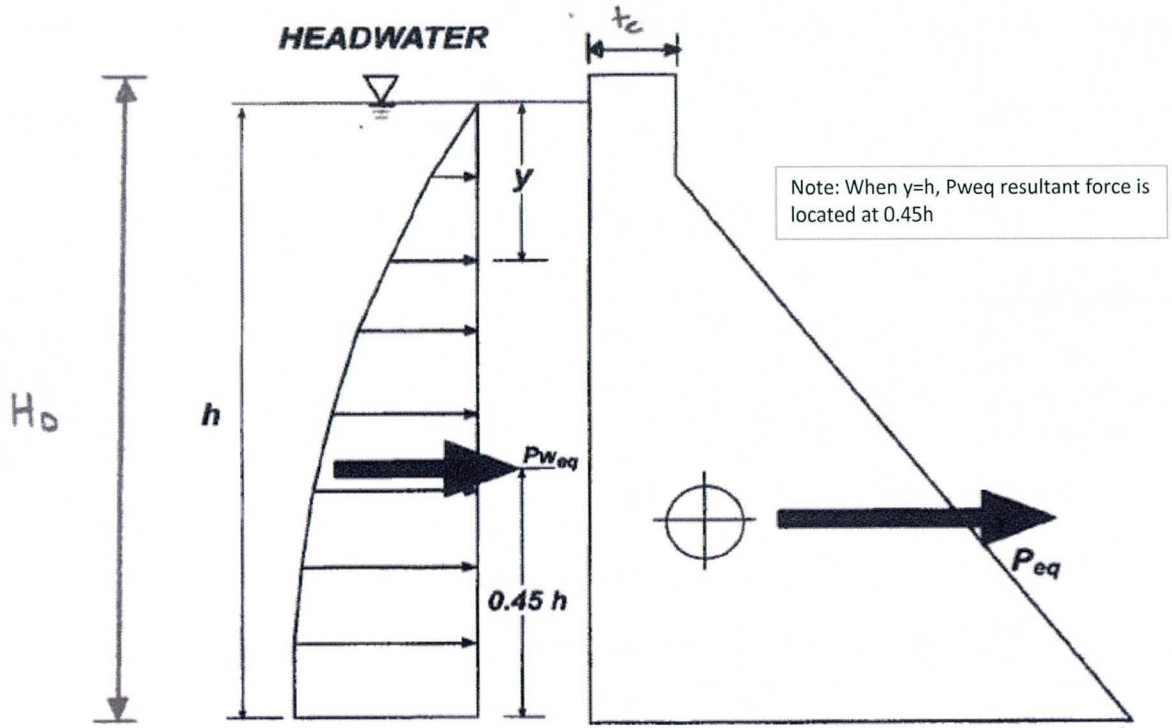


Figure 4.2. Seismically loaded gravity dam, non-overflow monolith

	A	B	C	D	E
1	External Stability - Sliding, Overturning and Bearing Capacity				
2	Developed By: Naida Causevic				
3	Created:	4/25/2012			
4					
5					
6	**Project Name:**	Oak Street Detention Basin and Storm Drain Project			
7	**Project No.:**	17-2011-4050			
8	**Date:**	6/25/2012			
9	**User:**	NC			
10					
11	**Input Parameters**				
12		**RCC Dam Section Station**			4+00
13		**Loading Condition**			3
14					
15		**RCC Dam Geometry**			
16		E_T	1810.3	[ft]	Elevation (top of dam)
17		E_B	1795	[ft]	Elevation (bottom of dam)
18		H_D	=$C$16-$C$17	[ft]	Height of Dam
19		b	=IF(Calculations!B13="b", ('Input Parameters'!C18*'Input Parameters'!C21)+'Input Parameters'!C20, IF(Calculations!B13="d", (2*('Input Parameters'!C18*'Input Parameters'!C21)+'Input Parameters'!C20), 'Input Parameters'!C18*'Input Parameters'!C21))	[ft]	Base of Dam
20		tc	8	[ft]	Crest Width
21		S	0.70	[dim]	Slope - (SH:1V) - downstream
22		$\gamma_{concrete}$	150	[pcf]	Unit Weight of Concrete (140-160 pcf)
23		$\phi_{concrete}$	45	[degrees]	Angle of internal friction of concrete
24		$c_{concrete}$	10	[psi]	Cohesion of concrete
25					
26		**External Water Pressure**			
27		γ_{water}	62.4	[pcf]	Unit Weight of Water
28		E_{HW}	1811.3	[ft]	Elevation of Reservoir Water Surface
29		h	=$C$28-$C$17	[ft]	Headwater at the heel (upstream end of the dam)
30		E_{TW}	1796	[ft]	Elevation of Tailwater Surface
31		h_1	=$C$30-$C$17	[ft]	Tailwater at the toe (downstream end of the dam)
32					
33					
34		**Silt and Earth Pressure**			
35		α		[ft]	Height of Silt Deposit
36		K_0		[dim]	Coefficient of Lateral Earth Pressure at Rest
37		γ_{silt}		[pcf]	Buoyant Unit Weight of Silt
38		ϕ_{silt}		[degrees]	Angle of internal friction of silt (not used for calculations)
39					
40					
41		**Seismic Forces**			
42		a	0.130	[dim]	Seismic Coefficient (see Figure 4.3)
43		y	16	[ft]	
44		C_{eq}	0.051	[dim]	factor depending generally on water depth and earthquake vibration period
45					

	A	B	C	D	E
46					
47		**Foundation Parameters**			
48				**Soil**	
49		$\gamma_{foundation}$	120	(pcf)	Unit weight of foundation soils
50		$\phi_{foundation}$	30	(degrees)	Angle of internal friction of foundation soils
51		$c_{foundation}$	200	(psf)	Cohesion of foundation soils
52		D_f	0	(ft)	embedment of foundation below original grade
53		γ'	='Input Parameters'!C52-'Input Parameters'!C27	(pcf)	Affect of water
54					
55		q_{fdn}	=C55*(C52-C27)	(psf)	overburden pressure for bearing capacity
56		N_q	=(EXP(PI()*TAN(RADIANS(C53)))*(TAN(RADIANS(45+(C53/2))))^2)	(dim)	Bearing Capacity Factor Meyerhof (1963)
57		N_c	=(C59-1)*(1/TAN(RADIANS(C53)))	(dim)	Bearing Capacity Factor Meyerhof (1963)
58		N_γ	=2*(C59+1)*TAN(RADIANS(C53))	(dim)	Bearing Capacity Factor Vesic (1973, 1975)
59					
60				**Rock**	
61		q_{all}	10	(tsf)	Allowable Contact Stress (Figure 4.4.8.1.1A)
62		q_{ult}	=IF(C68="use qult (soil)", $C$68, $C$68*$C$69)	(tsf)	Ultimate Bearing Capacity
63		N_{ms}	use q_{ult} (soil)	(dim)	Coefficinet (Figure 4.4.8.1.2A)
64		C_o	10	(ksf)	Uniaxial Compressive Strength (Figure 4.4.8.1.2B)

External Stability - Sliding, Overturning and Bearing Capacity

Developed By: Naida Causevic

Created: 4/25/2012

Project Name: Oak Street Detention Basin and Storm Drain Project

Project No.: 17-2011-4050

Date: 6/25/2012

User: NC

Station: 4+00

Loading Condition: 3

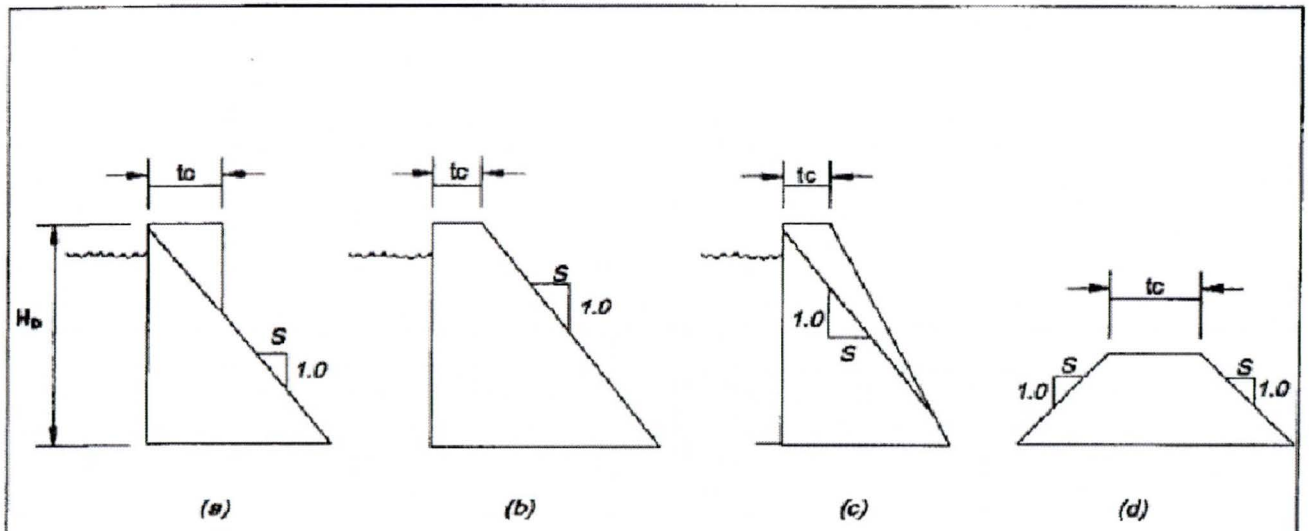
Section Shape: d Use a, b, c or d - See Table 5.1

Summation of Forces

Loading	Horizonatal Forces		Vertical Forces		Stabilizing Moments	Overturning Moments
	Force [kips]	Arm [ft]	Force [kips]	Arm [ft]	[ft-k]	[ft-k]
Weight of Dam (W_c)			=IF($B$13="a",SUM('Weight of Dam (Wc)'!P4:P5),IF($B$13="b",SUM('Weight of Dam (Wc)'!P8:P9),IF($B$13="c",SUM('Weight of Dam (Wc)'!P12:P13),SUM('Weight of Dam (Wc)'!P16:P18))))	see weight of dam calculations	=IF($B$13="a",SUM('Weight of Dam (Wc)'!R4:R5),IF($B$13="b",SUM('Weight of Dam (Wc)'!R8:R9),IF($B$13="c",SUM('Weight of Dam (Wc)'!R12:R13),SUM('Weight of Dam (Wc)'!R16:R18))))	
Uplift (U)			=SUM('Uplift (U)'!B10:B11)	see uplift calculations		=SUM('Uplift (U)'!D10:D11)
Headwater (H)	=(0.5*'Input Parameters'!C27*'Input Parameters'!C29*'Input Parameters'!C29)/1000	=(1/3)*'Input Parameters'!C29				=D21*E21
Silt (S)	=(0.5*'Input Parameters'!C36*'Input Parameters'!C37*'Input Parameters'!C35*'Input Parameters'!C35)/1000	=(1/3)*'Input Parameters'!C35				=D22*E22
Earthquake Interia Force (P_{eq})	=IF(B13="a", (SUM('Weight of Dam (Wc)'!P4:P5)*'Input Parameters'!C42), IF(B13="b", (SUM('Weight of Dam (Wc)'!P8:P9)*'Input Parameters'!C42), IF(B13="c", (SUM('Weight of Dam (Wc)'!P12:P13)*'Input Parameters'!C42), (SUM('Weight of Dam (Wc)'!P16:P18)*'Input Parameters'!C42))))	see seismic calculations				=IF($B$13="a",SUM('Seismic (Peq)'!R4:R5),IF($B$13="b",SUM('Seismic (Peq)'!R8:R9),IF($B$13="c",SUM('Seismic (Peq)'!R12:R13),SUM('Seismic (Peq)'!R16:R18))))
Earthquake Hydrodynamic Force (Pw_{eq})	=(2/3)*'Input Parameters'!C44*'Input Parameters'!C42*'Input Parameters'!C43*('Input Parameters'!C29*'Input Parameters'!C43)^0.5	=0.45*'Input Parameters'!C29				=D24*E24
Sum	=SUM(D19:D24)		=SUM(F19:F24)		=SUM(H19:H24)	=SUM(I19:I24)
					Net Moment	=H25-I25

	A	B	C	D	E	F	G	H	I
6	**Project Name:**	Oak Street Detention Basin and Storm Drain Project							
7	**Project No.:**	17-2011-4050							
8	**Date:**	6/25/2012							
9	**User:**	NC							
10	**Station:**	4+00							
11	**Loading Condition:**	3							
12									
28									
29	**Sliding:**								
30				**Failure Criteria**					
31				Load Condition	USACE	FERC/USBR			
32						High Hazard Dam	Low Hazard Dam		
33				Usual	2.00	3.00	2.00		
34				Unusual	1.70	2.00	1.25		
35				Extreme	1.30	1.00	1.00		
36			F.S =	=((F25*TAN(RADIANS(MIN('Input Parameters'!C23,'Input Parameters'!C53))))+(('Input Parameters'!C19*1*144)*(MIN('Input Parameters'!C24,'Input Parameters'!C54)/1000)))/Calculations!D25	**=IF($C$35>D32,"OK",IF($C$35>D33,"OK - Unusual Load Condition",IF($C$35>D34, "OK - Extreme Load Condition","FAIL")))**	**=IF($C$35>E32,"OK",IF($C$35>E33,"OK - Unusual Load Condition",IF($C$35>E34, "OK - Extreme Load Condition","FAIL")))**	**=IF($C$35>F32,"OK",IF($C$35>F33,"OK - Unusual Load Condition",IF($C$35>F34, "OK - Extreme Load Condition","FAIL")))**		
37									
38	**Overturning:**								
40									
41		Eccentricity	=('Input Parameters'!C19/2)-(Calculations!I26/Calculations!F25)	[ft]					
42						**=IF(C42/6>C40,"OK",IF(C42/4>C40,"OK - Unusuall loading condition",IF(C42>C40,"OK - Extreame loading condition","FAIL")))**				
43		Base length	='Input Parameters'!C19	[ft]					
44									
45	**Bearing Pressure:**								
46									
47									
48			F.S =	3					
49									
50			q_{ult}	=(((2/3)*'Input Parameters'!C54*144)*'Input Parameters'!C60+'Input Parameters'!C58*'Input Parameters'!C59+1/2*('Input Parameters'!C56*Calculations!C42*'Input Parameters'!C61))/1000	ksf	Toe (q_{max})	=(F25/'Input Parameters'!C19)*(1+(6*(Calculations!C40/'Input Parameters'!C19)))	[ksf]	
51								**=IF(AND(F49<C51,F51<C51), "OK", "FAIL")**	
52			$q_{allowable}$	=C49/C47	[ksf]	Heel (q_{min})	=(F25/'Input Parameters'!C19)*(1-(6*(Calculations!C40/'Input Parameters'!C19)))	[ksf]	
53									

Table 5.1. Possible Sections and Dimensions for Small RCC Dams

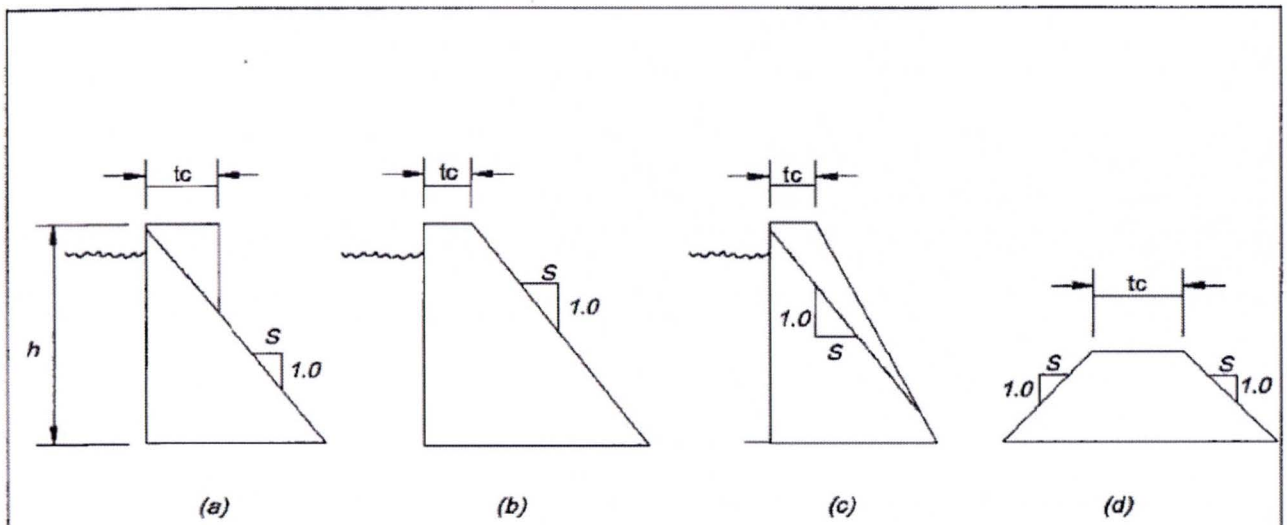


Section	Usual Height Range (H_D)	Usual Crest Width (tc)	Usual Slope (S)	Downstream Facing Treatment
(a)	40'-50' (12-15m)	10' to 16' (3.0-4.9m)	0.8 to 0.9	Formed or Exposed
(b)	10'-50' (3-15m)	10' to 12' (3.0-3.7m)	0.8 to 0.9	Formed or Exposed
(c)	10'-50' (3-15m)	10' to 12' (3.0-3.7m)	0.7 to 0.9	Formed - less volume than (b)
(d)	5'-20' (1.5-4.6m)	8' to 10' (2.4-3.0m)	0.8 to 1.0	Usually Exposed

	M	N	O	P	Q	R
1			Weight of Dam Calculations			
2						
3			Area	Weight	Moment Arm	Moment
4			[sf]	[kips]	[ft]	[ft-kp]
5						
6				Shape "a"		
7		Piece 1	=(0.5*'Input Parameters'!C18*'Input Parameters'!C21*'Input Parameters'!C18)	=(O4*'Input Parameters'!$C$22)/1000	=(2/3)*'Input Parameters'!C18*'Input Parameters'!C21	=P4*Q4
8		Piece 2	=(0.5*'Input Parameters'!C20*('Input Parameters'!C20/'Input Parameters'!C21))	=(O5*'Input Parameters'!$C$22)/1000	='Input Parameters'!C18*'Input Parameters'!C21)-((2/3)*'Input Parameters'!C20)	=P5*Q5
9		Total	=SUM(O4:O5)	=SUM(P4:P5)		=SUM(R4:R5)
10						
11				Shape "b"		
12		Piece 1	='Input Parameters'!C20*'Input Parameters'!C18)	=(O8*'Input Parameters'!$C$22)/1000	='Input Parameters'!C18*'Input Parameters'!C21)+(0.5*'Input Parameters'!C20)	=P8*Q8
13		Piece 2	=(0.5*'Input Parameters'!C18*'Input Parameters'!C21*'Input Parameters'!C18)	=(O9*'Input Parameters'!$C$22)/1000	=(2/3)*'Input Parameters'!C18*'Input Parameters'!C21	=P9*Q9
14		Total	=SUM(O8:O9)	=SUM(P8:P9)		=SUM(R8:R9)
15						
16				Shape "c"		
17		Piece 1	='Input Parameters'!C20*'Input Parameters'!C18)	=(O12*'Input Parameters'!$C$22)/1000	='Input Parameters'!C18*'Input Parameters'!C21)-(0.5*'Input Parameters'!C20)	=P12*Q12
18		Piece 2	=(0.5*(('Input Parameters'!C18*'Input Parameters'!C21)-'Input Parameters'!C20)*'Input Parameters'!C18)	=(O13*'Input Parameters'!$C$22)/1000	=(2/3)*(('Input Parameters'!C18*'Input Parameters'!C21)-'Input Parameters'!C20)	=P13*Q13
19		Total	=SUM(O12:O13)	=SUM(P12:P13)		=SUM(R12:R13)
20						
21				Shape "d"		
22		Piece 1	=(0.5*('Input Parameters'!C18*'Input Parameters'!C21)*'Input Parameters'!C18)	=(O16*'Input Parameters'!$C$22)/1000	=((1/3)*'Input Parameters'!C18*'Input Parameters'!C21)+'Input Parameters'!C20+('Input Parameters'!C18*'Input Parameters'!C21)	=P16*Q16
23		Piece 2	='Input Parameters'!C20*'Input Parameters'!C18)	=(O17*'Input Parameters'!$C$22)/1000	=0.5*'Input Parameters'!C20+('Input Parameters'!C18*'Input Parameters'!C21)	=P17*Q17
24		Piece 3	=(0.5*('Input Parameters'!C18*'Input Parameters'!C21)*'Input Parameters'!C18)	=(O18*'Input Parameters'!$C$22)/1000	=(2/3)*'Input Parameters'!C18*'Input Parameters'!C21	=P18*Q18
25		Total	=SUM(O16:O18)	=SUM(P16:P18)		=SUM(R16:R18)

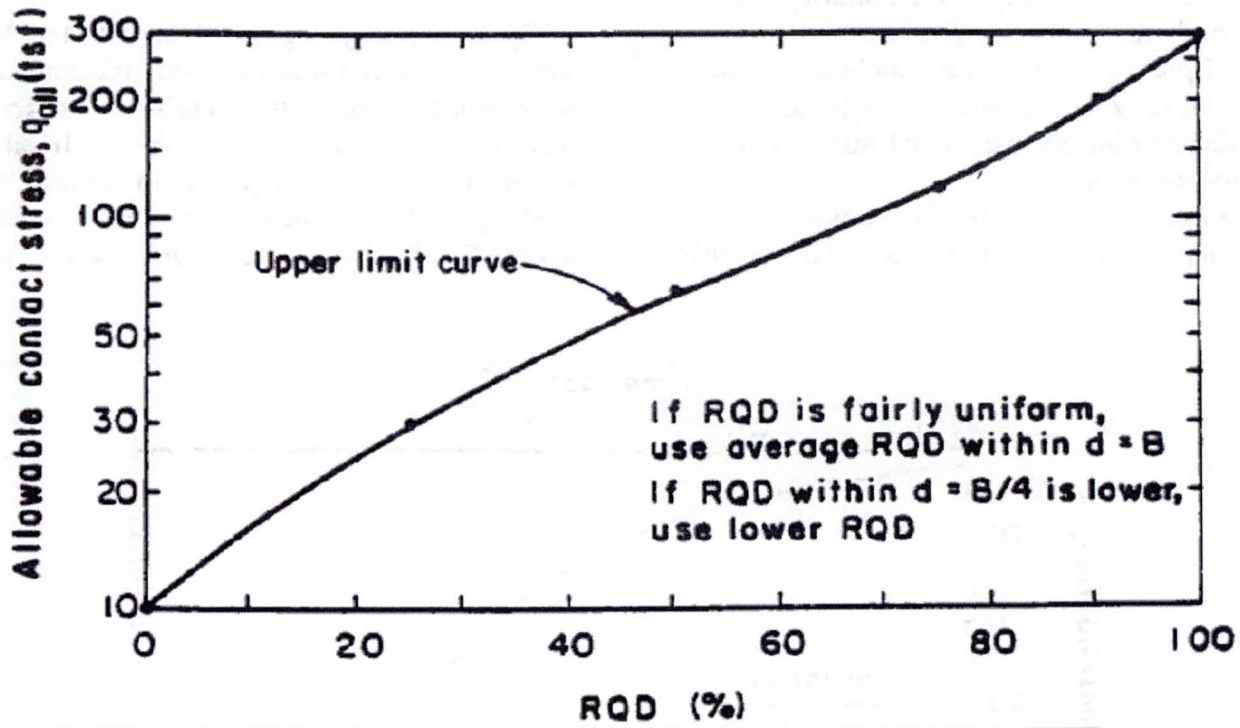
	A	B	C	D
1		Uplift Calculations		
2				
3	Pressure at			
4	Headwater			
5	Heel **(P)**	='Input Parameters'!C27*'Input Parameters'!C29	psf	
6	Tailwater			
7	Toe **(P1)**	='Input Parameters'!C27*'Input Parameters'!C31	psf	
8				
9		Uplift Force	Moment Arm	Moment
10		[kips]	[ft]	[ft-k]
11	Part 1	=-(B6*'Input Parameters'!C18*'Input Parameters'!C21)/1000	=0.5*'Input Parameters'!C19	=B10*C10
12	Part 2	=-(0.5*(B4-B6)*'Input Parameters'!C18*'Input Parameters'!C21)/1000	=(2/3)*'Input Parameters'!C19	=ABS(B11*C11)
13	Total	=SUM(B10:B11)		=SUM(D10:D11)

Table 5.1. Possible Sections and Dimensions for Small RCC Dams



Section	Usual Height Range (h)	Usual Crest Width (tc)	Usual Slope (S)	Downstream Facing Treatment
(a)	40'-50' (12-15m)	10' to 16' (3.0-4.9m)	0.8 to 0.9	Formed or Exposed
(b)	10'-50' (3-15m)	10' to 12' (3.0-3.7m)	0.8 to 0.9	Formed or Exposed
(c)	10'-50' (3-15m)	10' to 12' (3.0-3.7m)	0.7 to 0.9	Formed - less volume than (b)
(d)	5'-20' (1.5-4.6m)	8' to 10' (2.4-3.0m)	0.8 to 1.0	Usually Exposed

	M	N	O	P	Q	R
1			Seismic Calculations			
2						
3			Area	Weight	Moment Arm	Moment
4			[sf]	[kips]	[ft]	[ft-kp]
5						
6				Shape "a"		
7		Piece 1	=(0.5*'Input Parameters'!C18*'Input Parameters'!C21*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O4*'Input Parameters'!$C$22)/1000	=(1/3)*'Input Parameters'!C18	=P4*Q4
8		Piece 2	=(0.5*'Input Parameters'!C20*('Input Parameters'!C20/'Input Parameters'!C21))	=('Input Parameters'!$C$42)*(O5*'Input Parameters'!$C$22)/1000	='Input Parameters'!C18-((1/3)*('Input Parameters'!C20/'Input Parameters'!C21))	=P5*Q5
9		Total	=SUM(O4:O5)	=SUM(P4:P5)		=SUM(R4:R5)
10						
11				Shape "b"		
12		Piece 1	=('Input Parameters'!C20*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O8*'Input Parameters'!$C$22)/1000	=0.5*'Input Parameters'!C18	=P8*Q8
13		Piece 2	=(0.5*'Input Parameters'!C18*'Input Parameters'!C21*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O9*'Input Parameters'!$C$22)/1000	=(1/3)*'Input Parameters'!C18	=P9*Q9
14		Total	=SUM(O8:O9)	=SUM(P8:P9)		=SUM(R8:R9)
15						
16				Shape "c"		
17		Piece 1	=('Input Parameters'!C20*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O12*'Input Parameters'!$C$22)/1000	=0.5*'Input Parameters'!C18	=P12*Q12
18		Piece 2	=(0.5*(('Input Parameters'!C18*'Input Parameters'!C21)-'Input Parameters'!C20)*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O13*'Input Parameters'!$C$22)/1000	=(1/3)*'Input Parameters'!C18	=P13*Q13
19		Total	=SUM(O12:O13)	=SUM(P12:P13)		=SUM(R12:R13)
20						
21				Shape "d"		
22		Piece 1	=(0.5*('Input Parameters'!C18*'Input Parameters'!C21)*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O16*'Input Parameters'!$C$22)/1000	=(1/3)*'Input Parameters'!C18	=P16*Q16
23		Piece 2	=('Input Parameters'!C20*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O17*'Input Parameters'!$C$22)/1000	=0.5*'Input Parameters'!C18	=P17*Q17
24		Piece 3	=(0.5*('Input Parameters'!C18*'Input Parameters'!C21)*'Input Parameters'!C18)	=('Input Parameters'!$C$42)*(O18*'Input Parameters'!$C$22)/1000	=(1/3)*'Input Parameters'!C18	=P18*Q18
25		Total	=SUM(O16:O18)	=SUM(P16:P18)		=SUM(R16:R18)



Note:
q_{all} shall not exceed the unconfined compressive strength of the rock or 0.595 f'c of the concrete.

FIGURE 4.4.8.1.1A Allowable Contact Stress for Footings on Rock with Tight Discontinuities
Peck, et al. (1974)

TABLE 4.4.8.1.2A Values of Coefficient N_{ms} for Estimation of the Ultimate Bearing Capacity of Footings on Broken or Jointed Rock (Modified after Hoek, (1983))

Rock Mass Quality	General Description	RMR[1] Rating	NGI[2] Rating	RQD[3] (%)	N_{ms}[4] A	B	C	D	E
Excellent	Intact rock with joints spaced > 10 feet apart	100	500	95-100	3.8	4.3	5.0	5.2	6.1
Very good	Tightly interlocking, undisturbed rock with rough unweathered joints spaced 3 to 10 feet apart	85	100	90-95	1.4	1.6	1.9	2.0	2.3
Good	Fresh to slightly weathered rock, slightly disturbed with joints spaced 3 to 10 feet apart	65	10	75-90	0.28	0.32	0.38	0.40	0.46
Fair	Rock with several sets of moderately weathered joints spaced 1 to 3 feet apart	44	1	50-75	0.049	0.056	0.066	0.069	0.081
Poor	Rock with numerous weathered joints spaced 1 to 20 inches apart with some gouge	23	0.1	25-50	0.015	0.016	0.019	0.020	0.024
Very poor	Rock with numerous highly weathered joints spaced < 2 inches apart	3	0.01	<25	Use q_{ult} for an equivalent soil mass				

If rock mass quality is very poor, type "use q_{ult} (soil)"

[1]Geomechanics Rock Mass Rating (RMQ) System—Bieniawski, 1988.
[2]Norwegian Geotechnical Institute (NGI) Rock Mass Classification System, Barton, et al., 1974.
[3]Range of RQD values provided for general guidance only; actual determination of rock mass quality should be based on RMR or NGI rating systems.
[4]Value of N_{ms} as a function of rock type; refer to Table 4.4.8.1.2B for typical range of values of C_o for different rock type in each category.

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[1] (ksf)	C_o[1] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale[2]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

[1]Range of Uniaxial Compressive Strength values reported by various investigations.
[2]Not including oil shale.

This figure can be used for preliminary analysis, Uniaxial Compressive Strength should be determined from lab testing.

APPENDIX B

SEEPAGE ANALYSIS

Figure B-1

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 3+00 (Emergency Spillway - Partial Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1

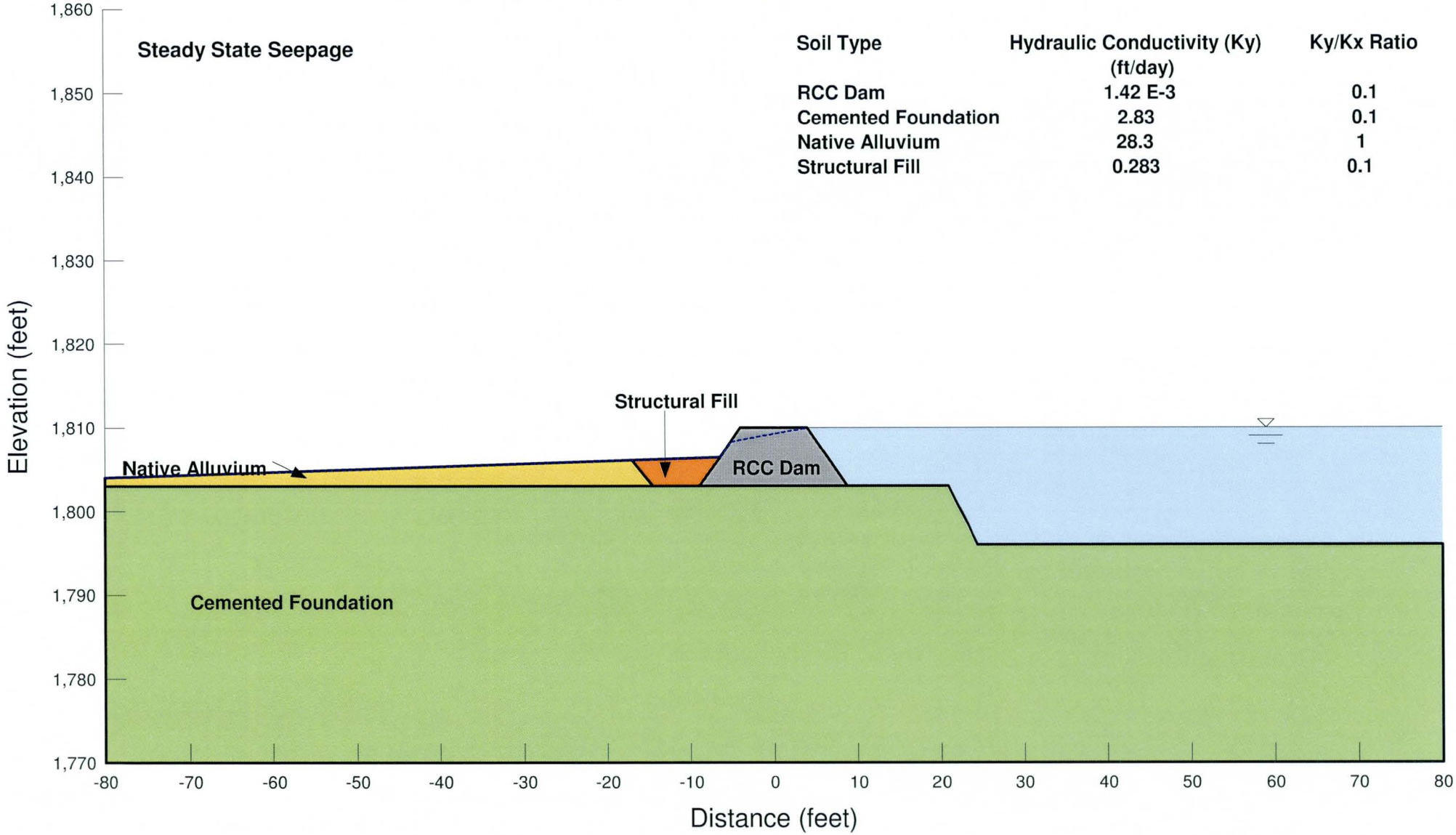


Figure B-2

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 3+00 (Emergency Spillway - Partial Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1

Exit Gradient = 0.07

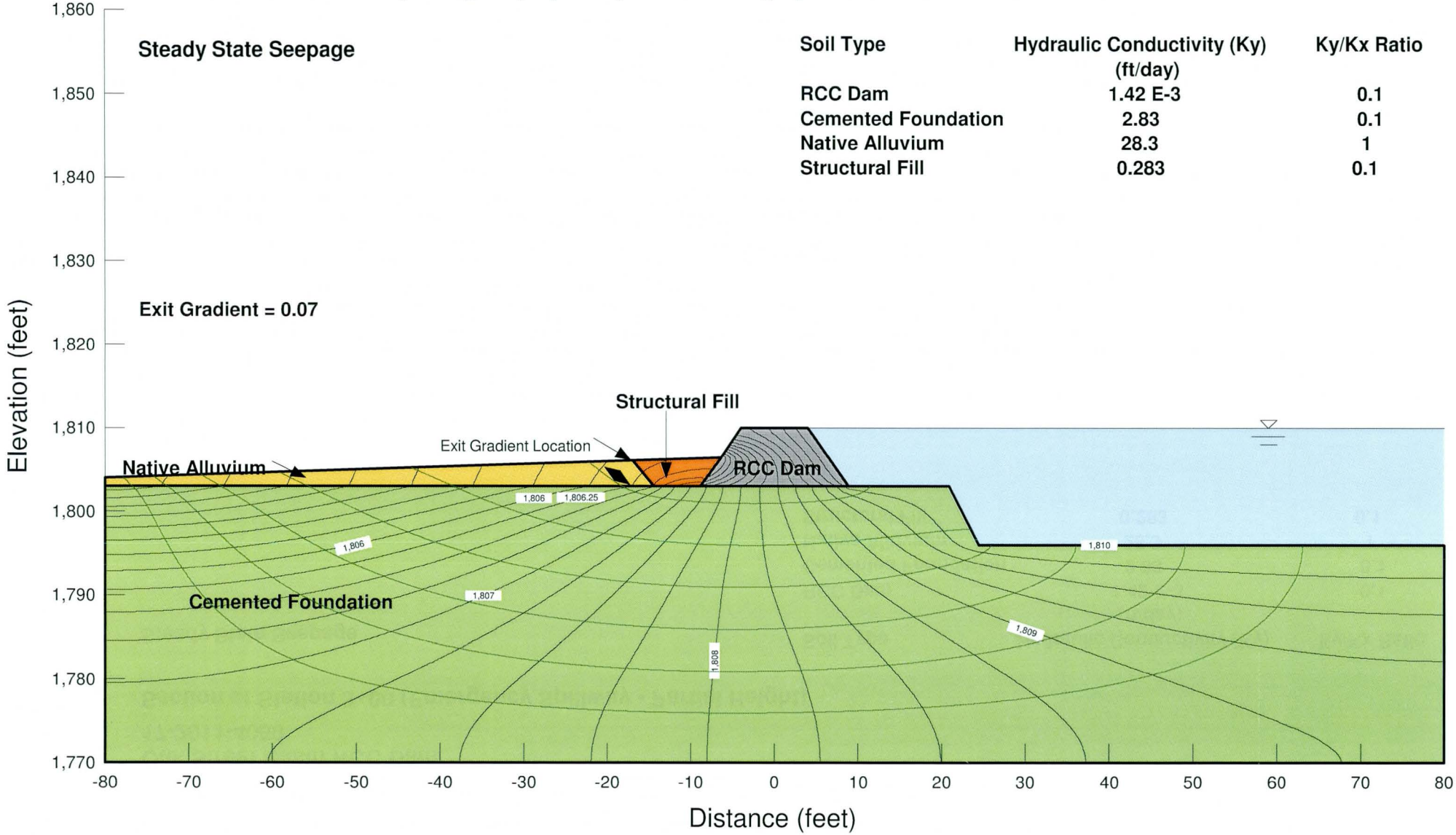


Figure B-3

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 3+00 (Emergency Spillway - Partial Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1

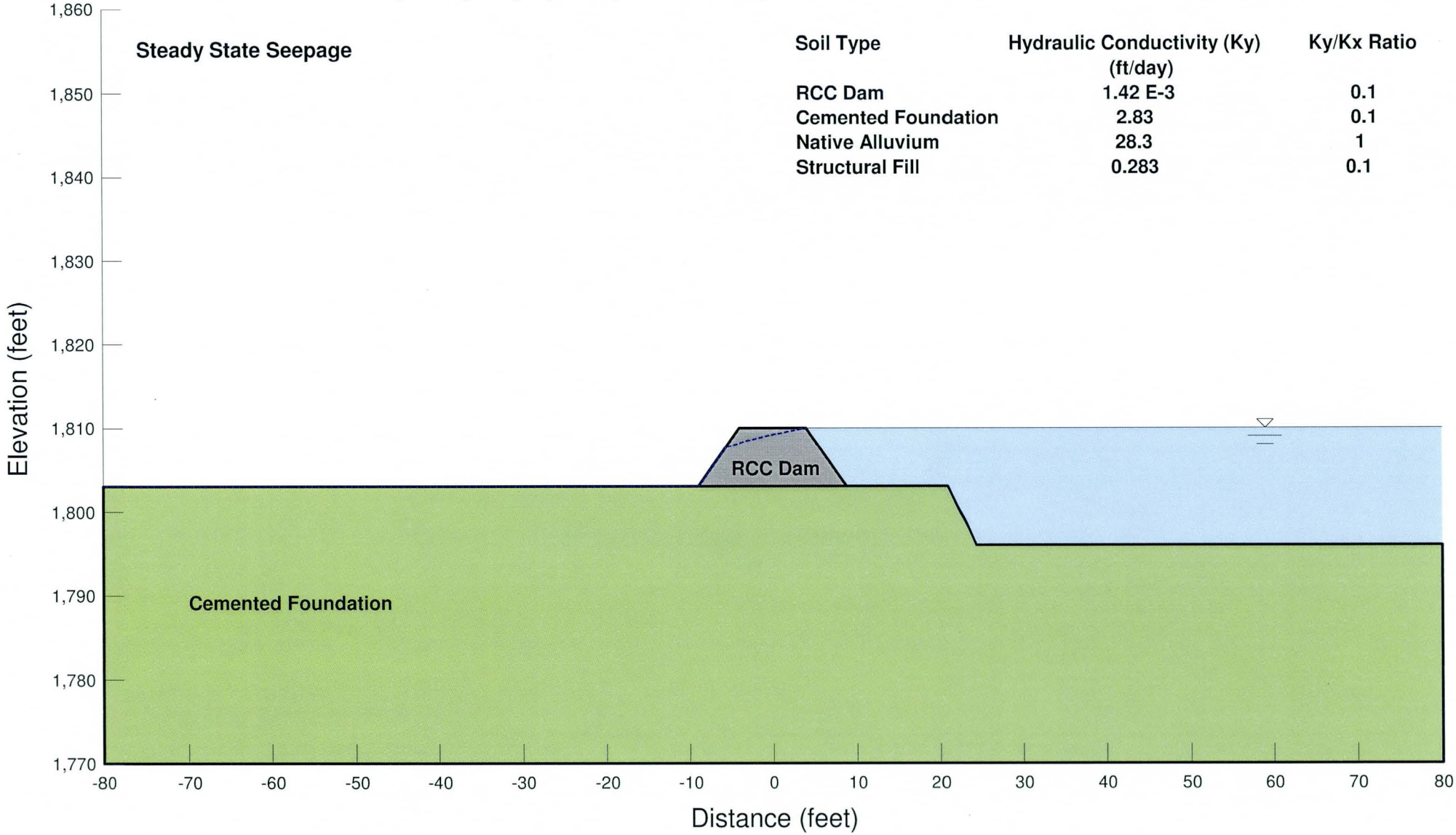


Figure B-4

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 3+00 (Emergency Spillway - Partial Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1

Exit Gradient = 0.47

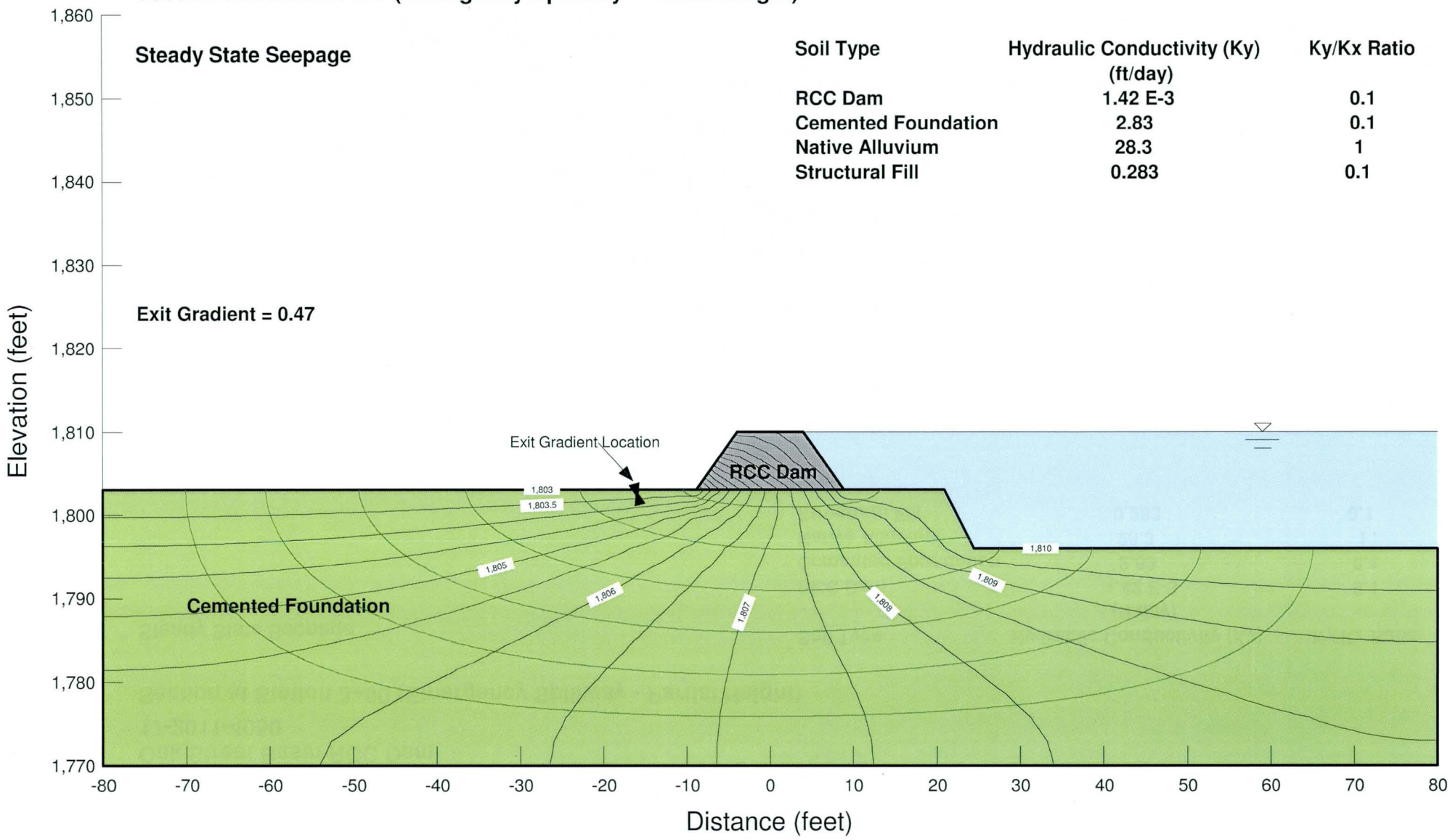


Figure B-5

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 4+00 (Emergency Spillway - Full Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1
Landscape Fill	28.3	1

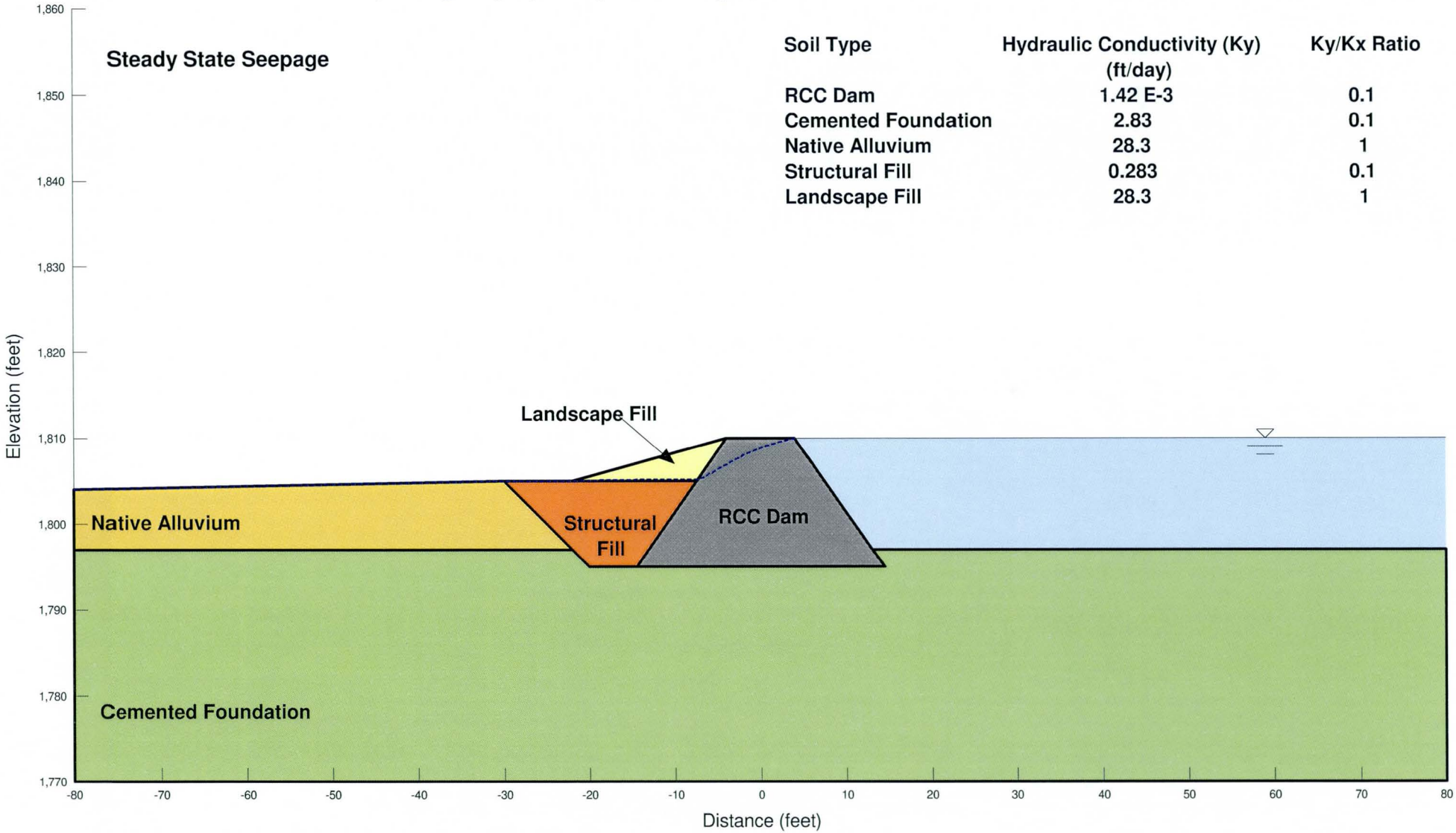


Figure B-6

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 4+00 (Emergency Spillway - Full Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1
Landscape Fill	28.3	1

Exit Gradient = 0.03

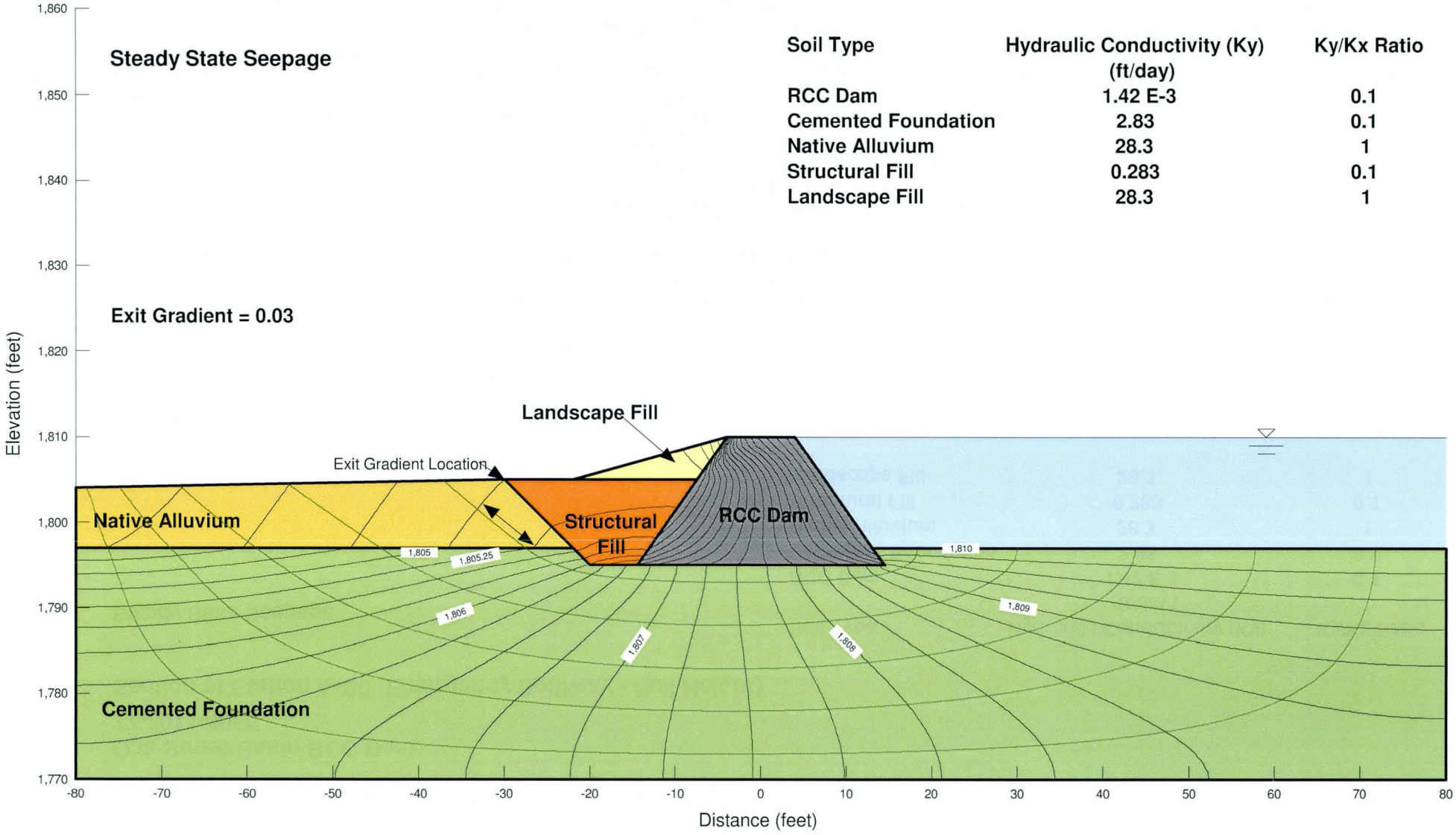


Figure B-7

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 4+00 (Emergency Spillway - Full Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1

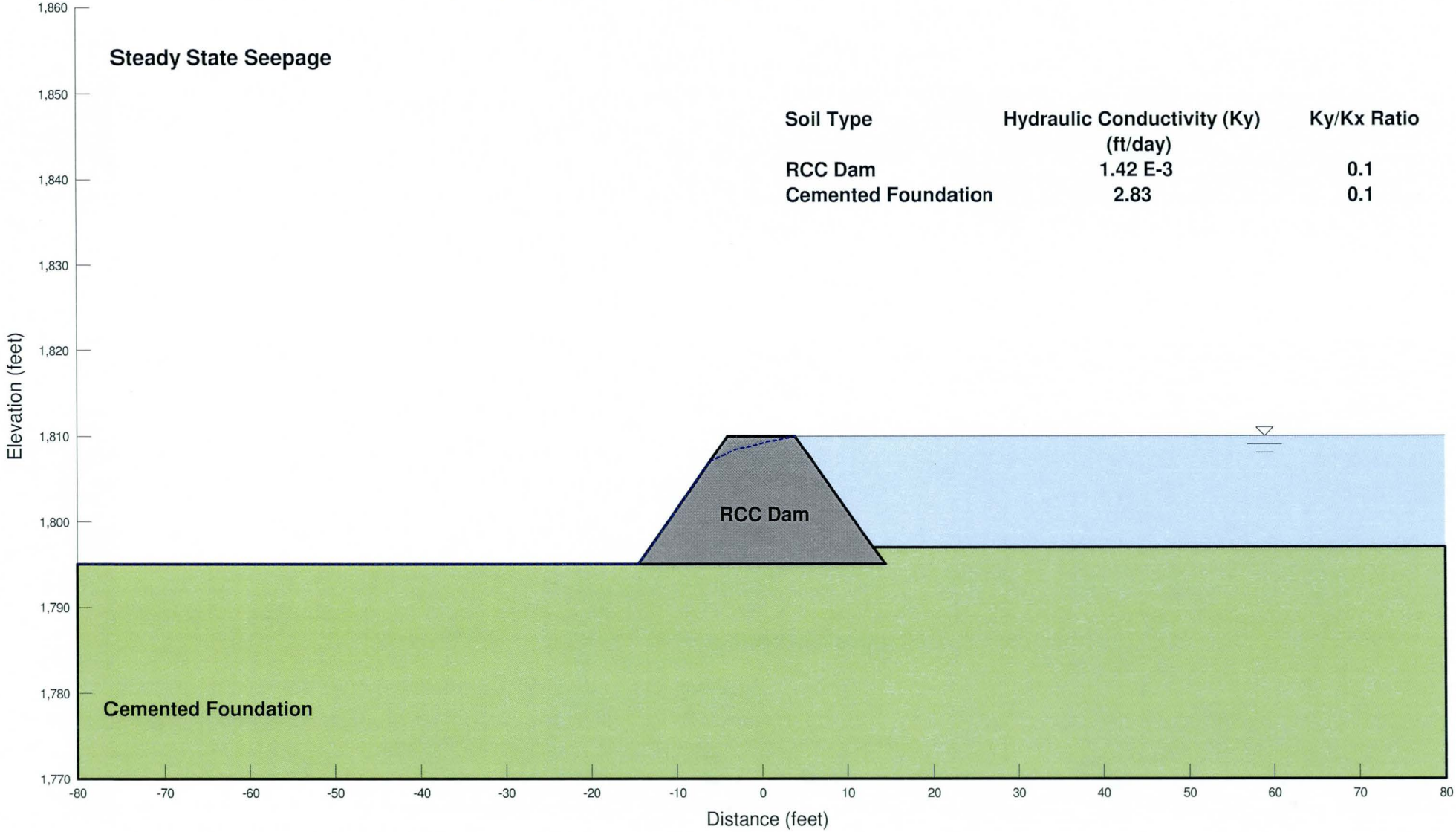


Figure B-8

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 4+00 (Emergency Spillway - Full Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1

Exit Gradient = 0.63

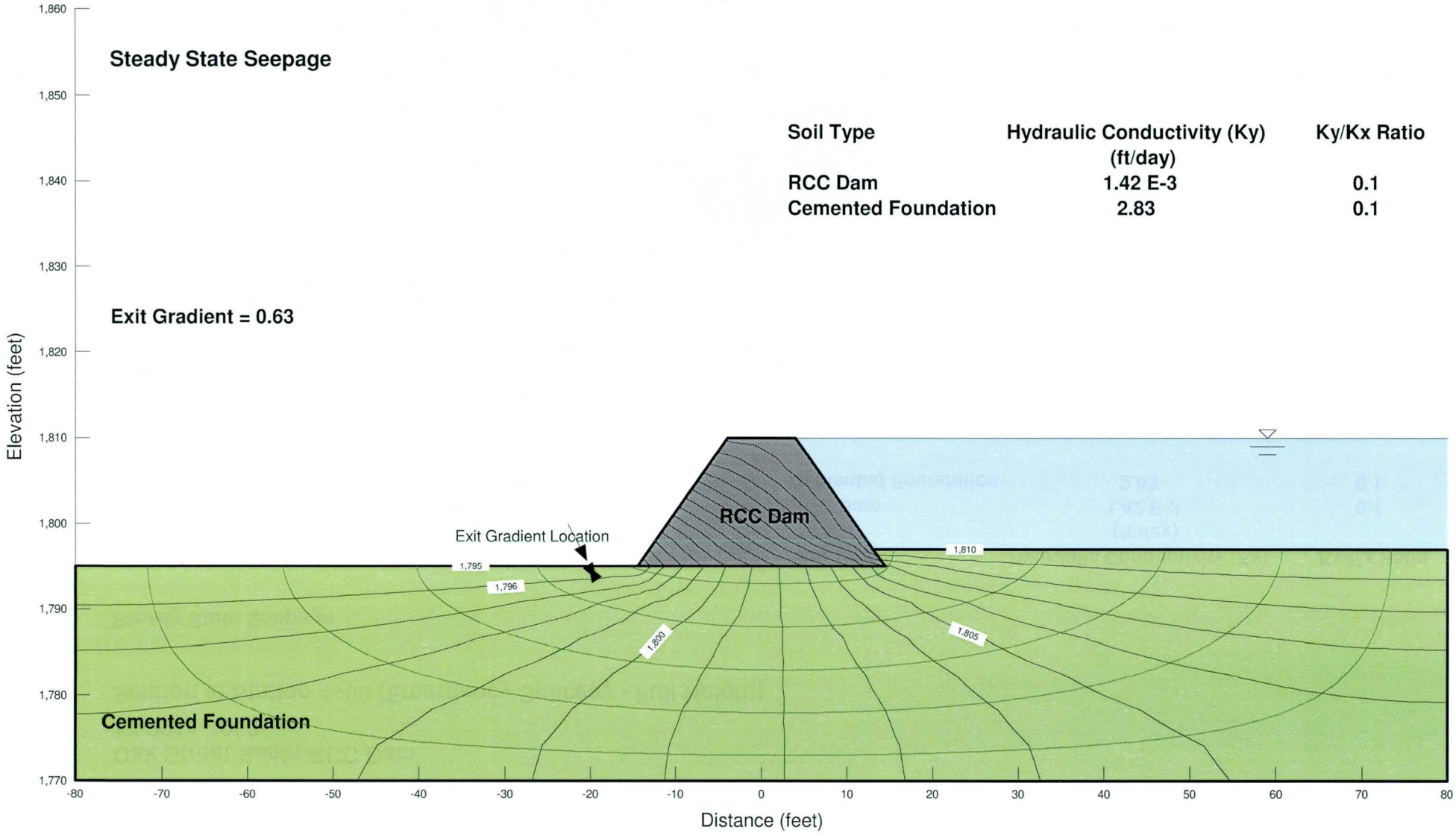


Figure B-9

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 3+00 (Emergency Spillway - Partial Height)

Transient Seepage (30 days)

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1

Note: Water reaches downstream side after 6 days

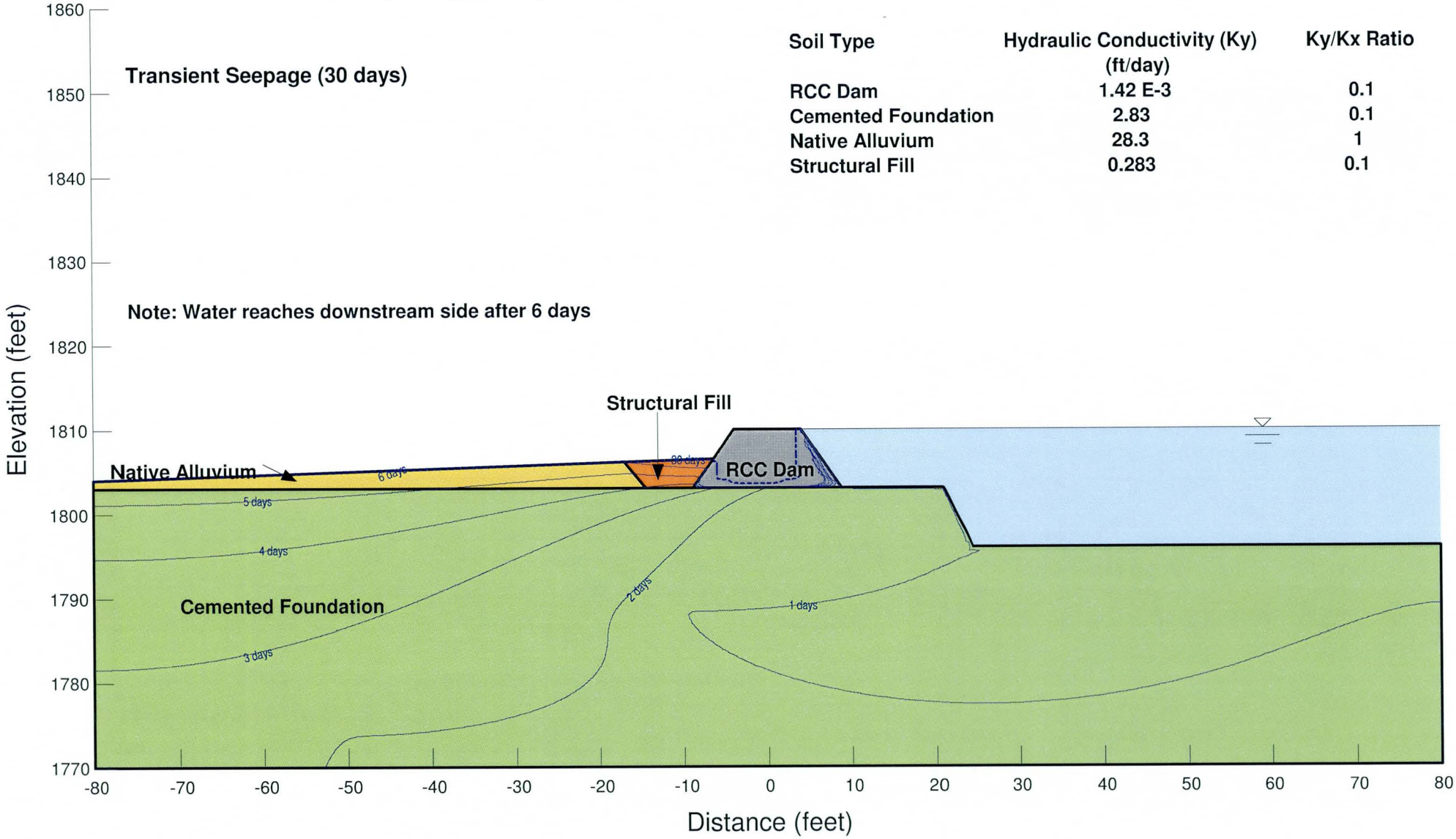


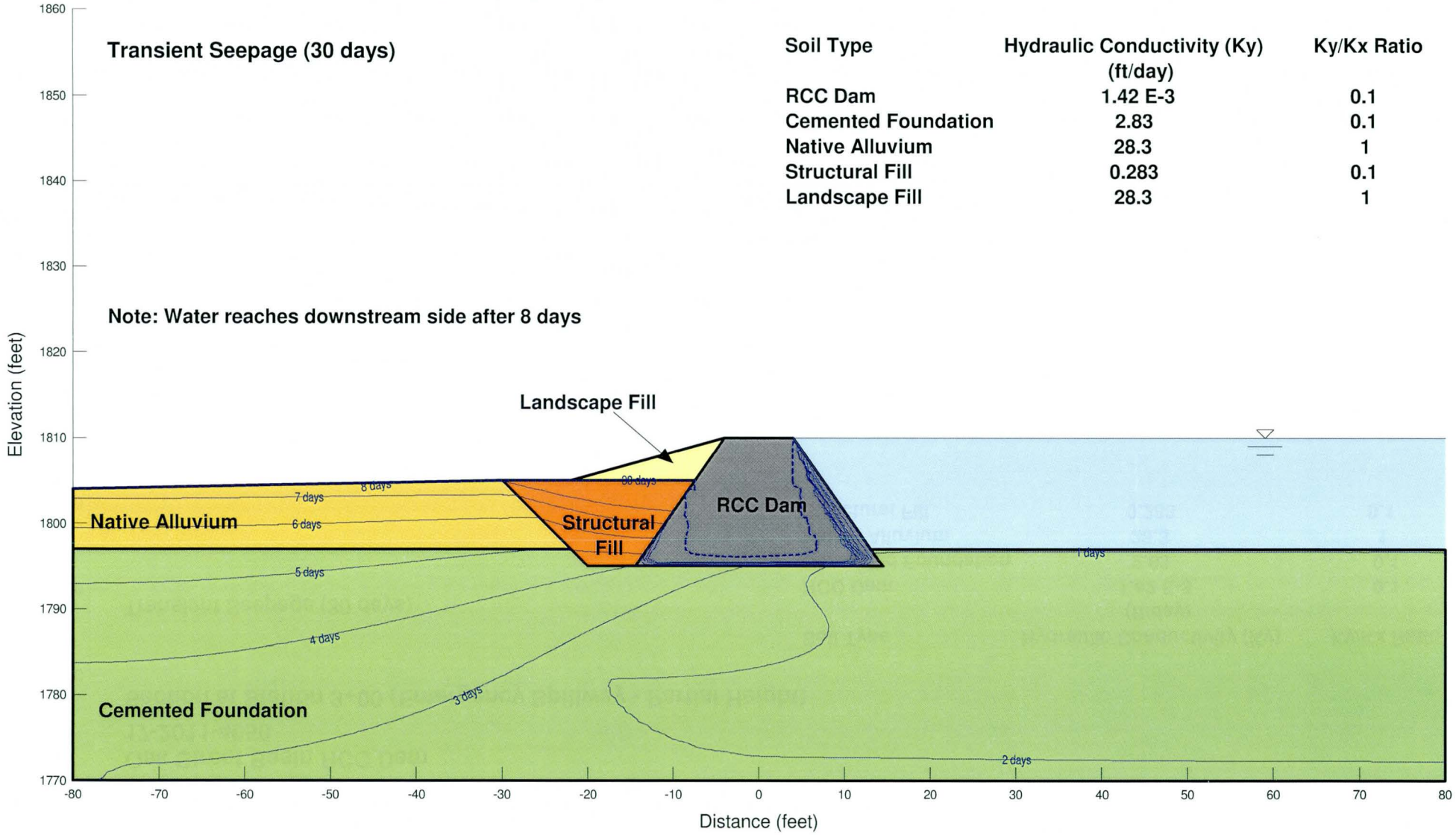
Figure B-10

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 4+00 (Emergency Spillway - Full Height)

Transient Seepage (30 days)

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1
Landscape Fill	28.3	1

Note: Water reaches downstream side after 8 days



Exit Gradient

$$\text{exit gradient} = i = \frac{\Delta h}{\ell} = \frac{\text{change in head}}{\text{length of flowpath}}$$

Section at Station 4+00

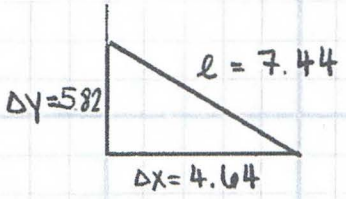
→ using SEEP/W solution (Figure B-6)

The total head loss between the last two equipotential lines on the downstream side is

$$1805.25' - 1805' = 0.25' \checkmark$$

The length of the flow path in the direction of the flow line within the element is

points: (-27.34, 1796.90)
(-33.16, 1801.54)



$$\sqrt{5.82^2 + 4.64^2} = 7.44' \checkmark$$

Therefore $$i = \frac{0.25'}{7.44'} = 0.03 \checkmark$$

The calculated exit gradient of 0.03, matches the SEEP/W calculated one of 0.03. (Figure B-6)

* Note: due to non homogeneous soil conditions flow lines are curved at surface boundry and therefore XY gradient used.

amec

Section at Station 3+00

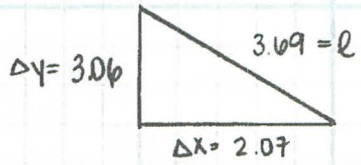
→ using SEEP/W solution (Figure B-2)

The total head loss between the last two equipotential lines on the downstream side is

$$1806.25' - 1806' = 0.25'$$ ✓

The length of the flow path in the direction of the flow line within the element

points: (-17.9, 1802.85)
(-20.96, 1804.92)



$$\sqrt{3.06^2 + 2.07^2} = 3.69$$ ✓

Therefore $$i = \frac{0.25'}{3.69'} = \underline{\underline{0.07}}$$ ✓

The calculated exit gradient of 0.07, matches the SEEP/W calculated one of 0.07. (Figure B-2)

* Note: Due to non homogeneous soil conditions flow lines are curved at surface boundries and therefore X-Y gradient used.

Oak Street Detention Basin and Storm Drain

RCC Dam Design

Section at Station 4+00 (without Structural Fill)

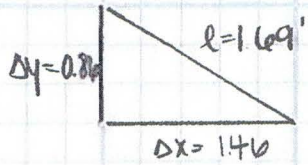
→ using SEEP/W solution (Figure B-8)

The total head loss between the last two equipotential lines on the downstream side is

$$1796' - 1795' = 1'$$

The length of the flow path in the direction of the flow line within the element is

points (-19.31, 1793.54)
(-20.17, 1795)



$$\sqrt{0.86^2 + 1.46^2} = 1.69' \checkmark$$

Therefore $i = \frac{1'}{1.69'} = 0.59 \checkmark$

The calculated exit gradient of 0.59, closely mathes the SEEP/W calculated one of 0.63. (Figure B-8)

* Note: due to non homogeneous soil conditions flow lines are curved at surface boundry and therefore XY gradient used

Section at Station 3+00 (without Structural Fill)

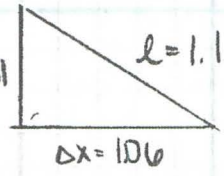
→ using SEEP/W solution (Figure B-4)

The total head loss between the last two equipotential lines on the downstream side is

$$1803.5' - 1803' = 0.5'$$

The length of the flow path in the direction of the flow line within the element is:

points : (-15.84, 1801.94)
(-16.15, 1803)



$$\sqrt{0.31^2 + 1.06^2} = 1.1$$

Therefore $i = \frac{0.5'}{1.1} = 0.45$

The calculated exit gradient of 0.45, closely matches the SEEP/W calculated one of 0.47. (Figure B-4)

* Note: due to non homogeneous soil conditions flow lines are curved at surface boundry and therefore XY gradient used

Figure B-6

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 4+00 (Emergency Spillway - Full Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1
Landscape Fill	28.3	1

Exit Gradient = 0.03

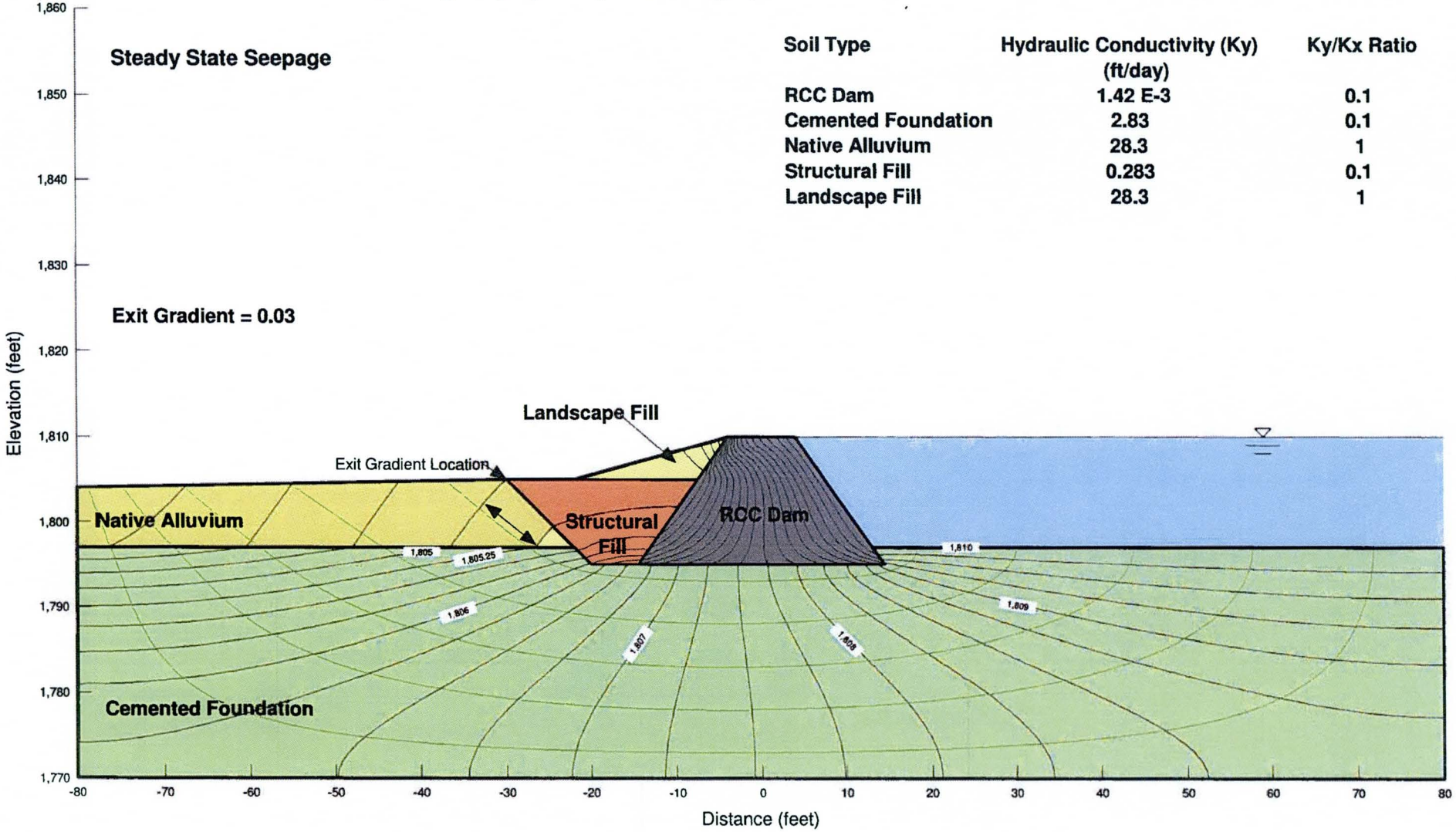


Figure B-2

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 3+00 (Emergency Spillway - Partial Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1

Exit Gradient = 0.07

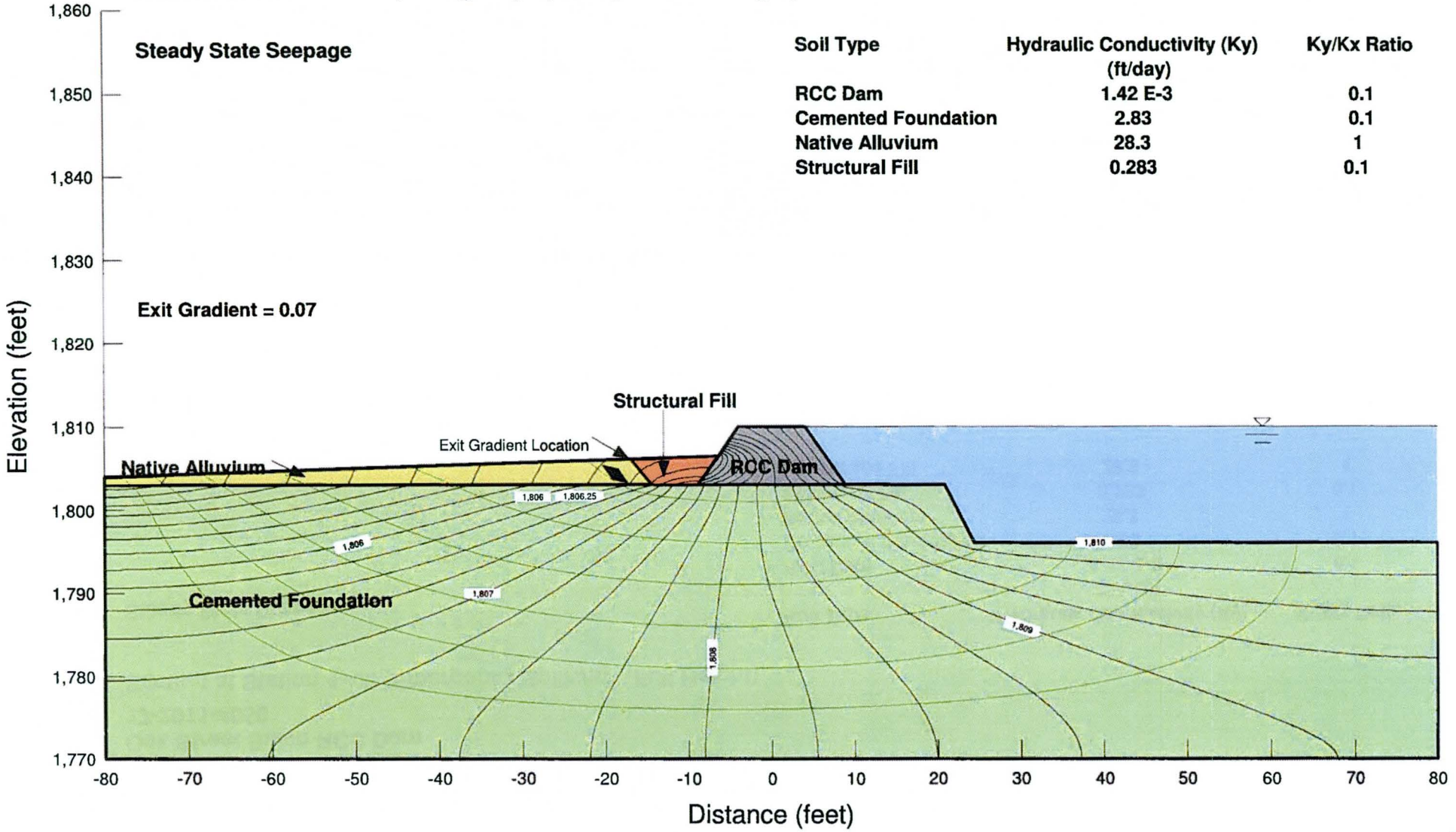


Figure B-8

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 4+00 (Emergency Spillway - Full Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1

Exit Gradient = 0.63

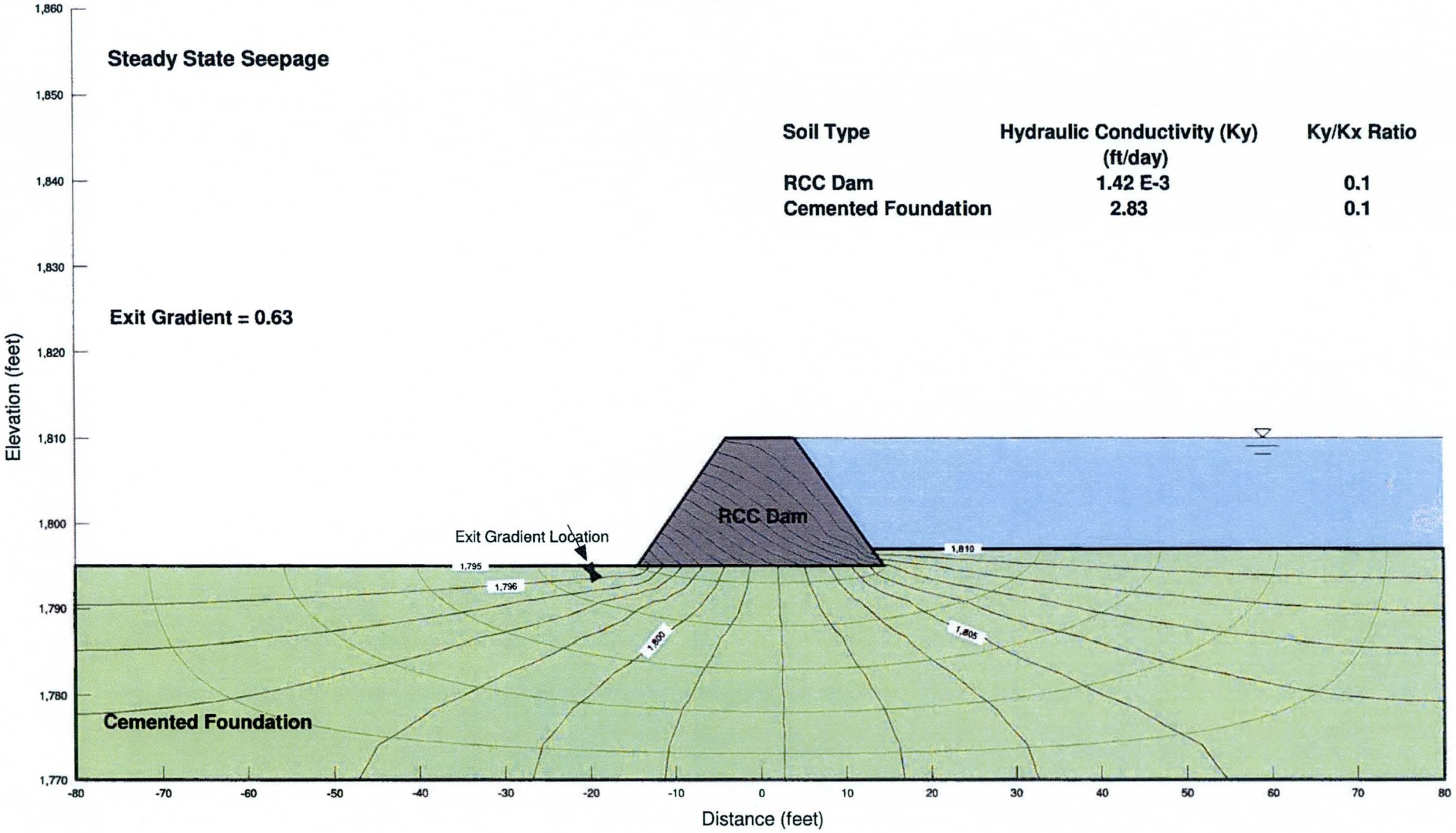


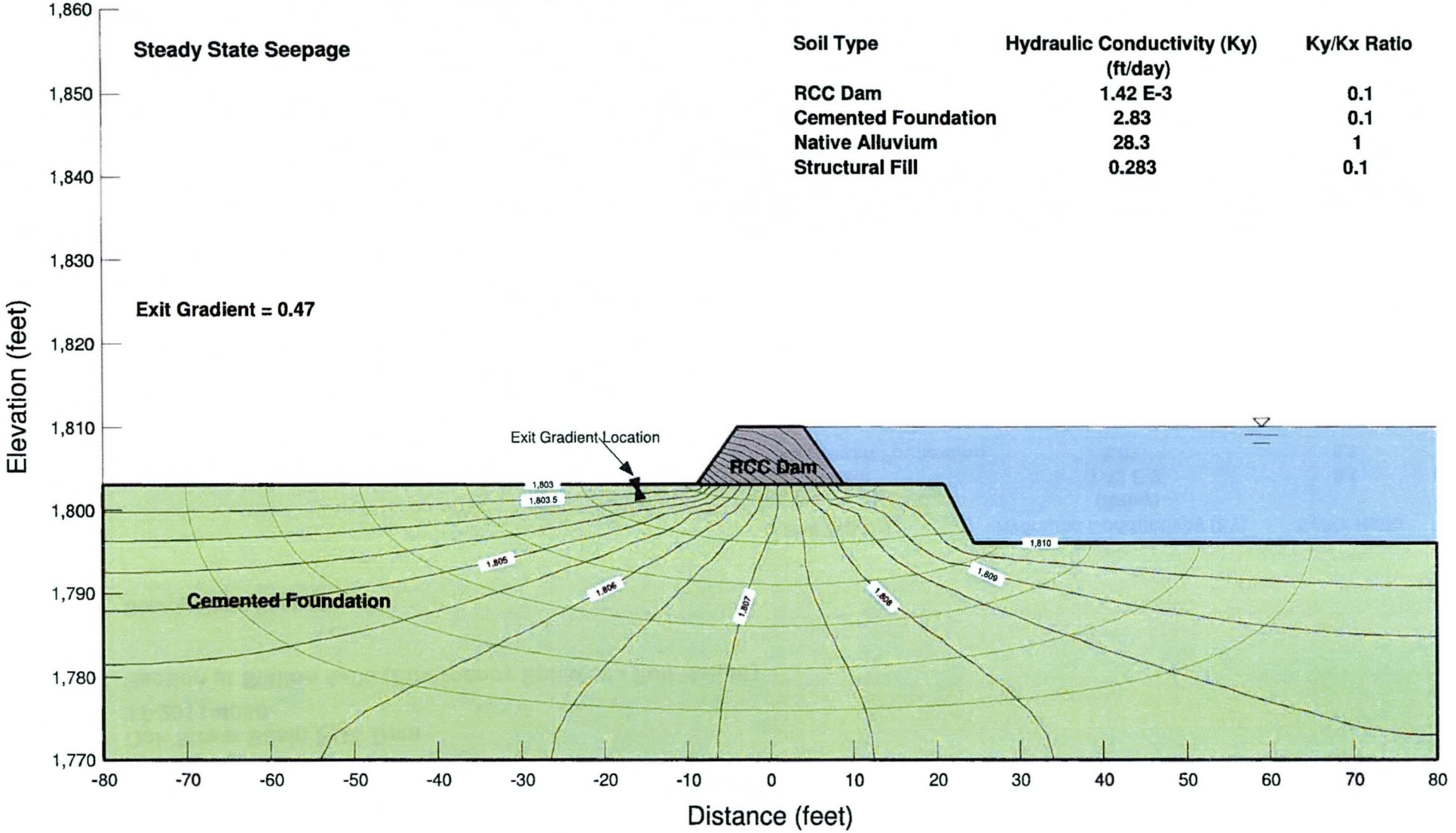
Figure B-4

Oak Street Basin RCC Dam
17-2011-4050
Section at Station 3+00 (Emergency Spillway - Partial Height)

Steady State Seepage

Soil Type	Hydraulic Conductivity (Ky) (ft/day)	Ky/Kx Ratio
RCC Dam	1.42 E-3	0.1
Cemented Foundation	2.83	0.1
Native Alluvium	28.3	1
Structural Fill	0.283	0.1

Exit Gradient = 0.47



8/8

Seepage Exit Gradients

1 Introduction

Most Soil Mechanics text books present and discuss the concept of seepage exit gradients and state that the exit gradients should not be greater than 1.0 Applying this criteria to two-dimensional finite element seepage analyses requires an understanding as to the conditions for which the criteria was developed and the physical meaning of an exit gradient greater than unity (1.0).

This document discusses the background as to how the exit gradient criterion was developed and how this criterion should be viewed when interpreting 2D finite element seepage analyses.

2 One-dimensional upward flow

The concept of exit gradients was developed primarily from pure upward flow in a column of soil. Consider the column in Figure 1. Water will flow upward through the column if the applied total hydraulic head H at the bottom of the column is greater than the surface elevation of the column.

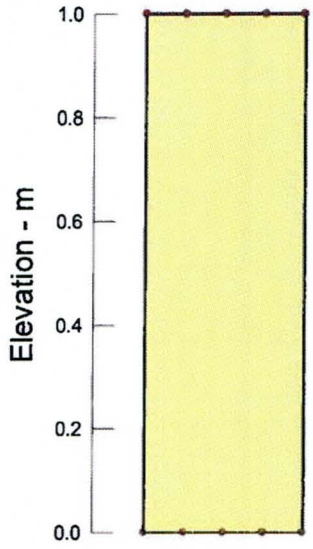


Figure 1 One-dimensional soil column

If for example H at the base is specified as 1.2 m water will flow upward through the column. The total head loss is 1.2 minus 1.0 which equals 0.2 m of head. The gradient is the total head loss divided by the height (length) of the column which in this case is 0.2. In equation form,

$$i = \frac{\left(H_{base} - H_{top}\right)}{L} = \frac{\left(1.2 - 1.0\right)}{1.0} = 0.2$$

For discussion purposes, let us assume that the total unit weight of the soil is 20 kN/m^3 and that the unit weight of the water is 10 kN/m^3.

Now if we apply a total head at the base of the column of equal to 2 m (equals 2 m of pressure head since the elevation at the base is zero), the upward gradient will be 1.0 and the effective stress throughout the column will be zero.

$$\sigma' = (\sigma - u) = (\gamma h - \gamma_w H) = (20*1 - 10*2) = 0$$

When the effective stress is zero, the gradient is sometimes referred to as the critical gradient, and since the zero effective stress conditions occur when the gradient is 1.0, the critical gradient is 1.0.

The zero effective stress condition under upward flow conditions is also referred to as a "quick" condition. It is this condition that is referred to colloquially sometimes as "quick sand" or "boiling."

Figure 2 shows the SEEP/W computed gradients when H at the bottom of the column is 2.0 and 1.0 m at the top of the column in Figure 1. This matches the earlier hand calculations.

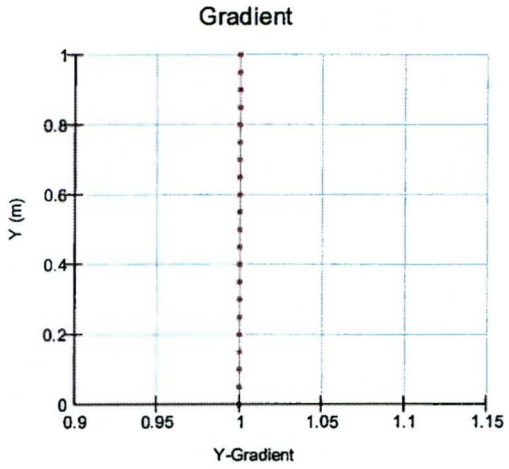


Figure 2 SEEP/W computed gradients

An important observation in the context of the later 2D flow discussions is that the gradient is a constant throughout the column.

The criterion that the exit gradient should not exceed 1.0 comes from this type of 1D upward flow analysis.

3 Two-dimensional flow under at hydraulic structure

Figure 3 shows a typical flow net seepage solution for flow under a hydraulic structure. The example is taken from the text book *Soil Mechanics, SI Version* by T.W. Lambe and R.V. Whitman, published by John Wiley & Sons.

This example illustrates how the exit gradient is computed from a flow net. The upward gradient is computed in the area marked with an X. The total head loss H between the last two equipotential lines is 0.62 m. The distance between the two equipotential lines on the downstream end in the X area is 3.3 m. The exit gradient is then computed as 0.62 divided by 3.3 making the upward gradient 0.19.

Of significance for later discussions is that the gradient is computed for a head loss over a fairly long distance of 3.3 m. The computed gradient is in essence an average gradient over this distance. Recognizing this as an average over a significant distance is important when we later discuss finite element results.

▶ **Example 18.2**

Given: Flow net in Fig. E18.2

Find: Pressure heads at points A to H; quantity of seepage; gradient in X

Solution: The pressure heads are shown in Fig. E18.2.

Seepage:

$$n_f = 4 \qquad n_d = 12.6 \qquad k = 0.05 \text{ cm/sec} \qquad \S = \frac{n_f}{n_d} = 0.317$$

$$\frac{Q}{L} = kH\S = 0.05 \times 10^{-2} \text{ m/sec} \times 7.8 \text{ m} \times 0.317 = 0.00124 \text{ (m}^3\text{/sec)(m)}$$

Gradient in X:

$$i_X = \frac{\Delta h}{l} = \frac{0.62}{3.3} = 0.19$$

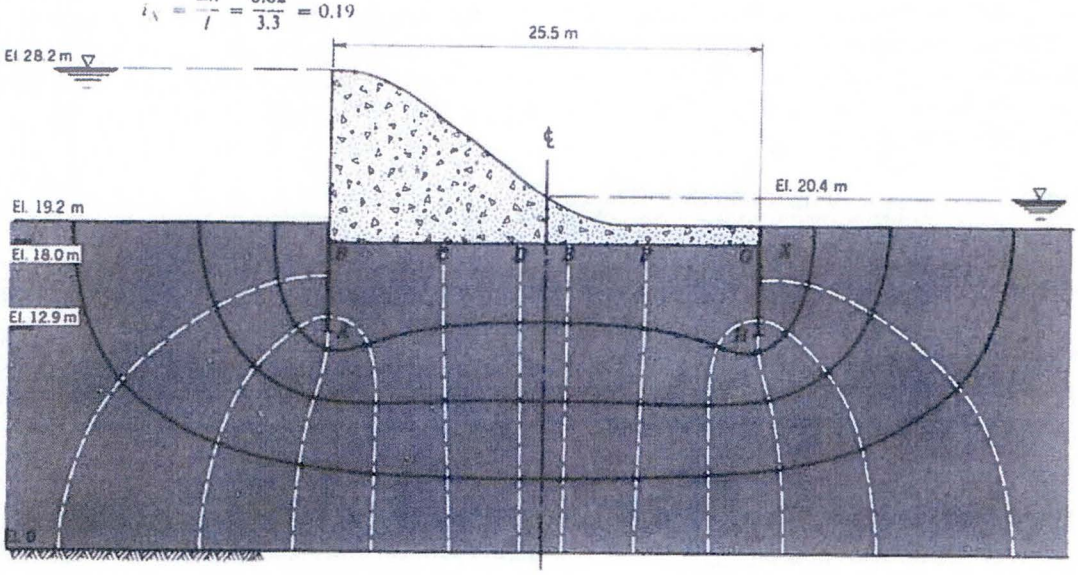


Figure 3 Example flow net from the Lambe and Whitman text book, page 271

Figure 4 shows the SEEP/W solution for the above text book example.

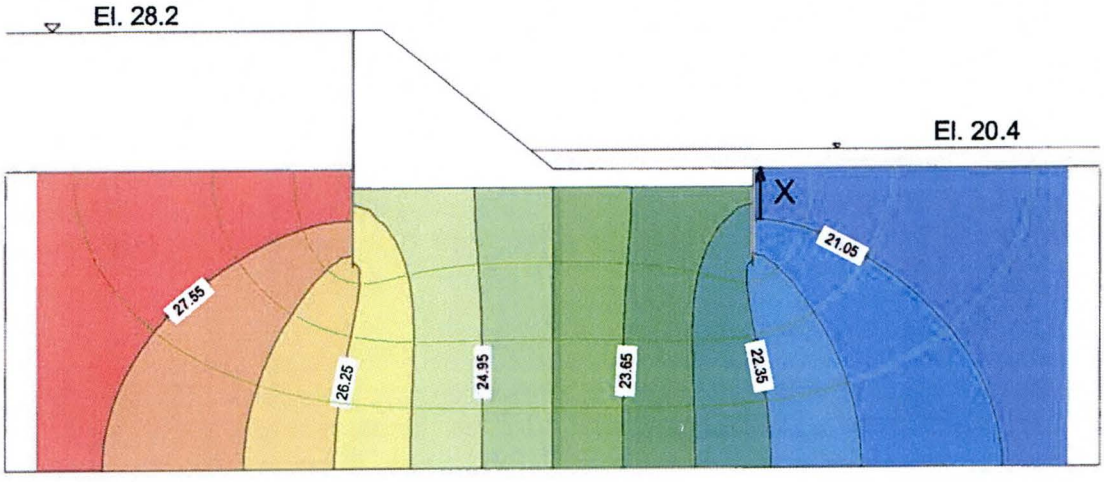


Figure 4 SEEP/W solution for the text book example

Now when we compute the upward gradient in the X area, the total head loss is between the last two equipotential lines is 21.05 - 20.4 = 0.65 (the equipotential increments are not all even in the text book example – there is about a half increment around the center-line. SEEP/W cannot accommodate this; the increments must all be the same and consequently the increment is 0.65 instead of 0.62 in Figure 3). The vertical distance at the location of the upward arrow in Figure 4 is about El 19.2 – 15.9 = 3.3 m. The upward gradient is therefore 0.65/3.3 = 0.20 which is in essence the same as for the flow net case.

In a finite element formulation the gradients are computed at what are known as Gauss integration points. In the context of local element coordinates which are ±1.0 at the corners of the element and (0, 0) at the center of a quadrilateral element, the Gauss integration points for a 4-noded quadrilateral are at local coordinates equal to ± 0.577. In SEEP/W we shade Gauss integration areas as shown in Figure 5. The actual Gauss integration point is about in the middle of the Gauss area. The Gauss area is shown primarily for convenient reference. It is important however to recognize that the calculations are done at the Gauss integration point. This is discussed in more detail in the theory chapters of the related GeoStudio Engineering Books.

Results can be inspected at the Gauss integration points with the View Results Information command in CONTOUR by clicking on a Gauss region.

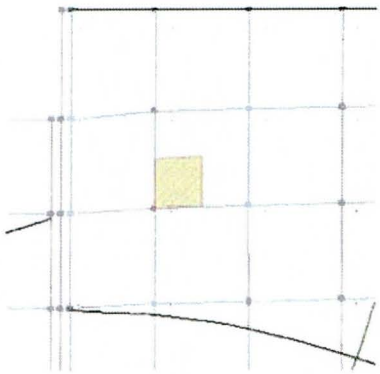


Figure 5 Illustration of a SEEP/W Gauss region

If we now click on the Gauss regions beside the up-arrow in the X area in Figure 4, the y-gradients vary between 0.188 and 0.206. The variation is small because the flow is predominantly upward and represents the situation discussed earlier for 1D upward flow in a column. However while the variation is small, it nonetheless demonstrates that the flow net method of computing exit gradients is an average over a meaningful distance.

Gradients can also be inspected at nodes. The gradients at nodes are however not computed as part of the finite element solution. They are computed as the average of the gradients in all the Gauss regions common to a node or that touch a node. Computing gradients at the nodes is done primarily for contouring purposes.

4 Homogeneous isotropic case

The exit gradient is independent of the hydraulic conductivity if the soil is homogeneous and isotropic ($K_x = K_y$). This is evident by examining the partial differential equation for steady-state seepage flow.

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} = 0$$

If K_x is equal to Ky we can divide both sides of the equation by K_x and get,

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

This shows that the pressure distribution is independent of the hydraulic conductivity in this special case and therefore the exit gradient is independent of the hydraulic conductivity.

The specific discharge or Darcian velocity (called Liquid Velocity in SEEP/W) is however directly related to the hydraulic conductivity. For fairly permeable sands the rate of upward flow will be much higher than for less permeable silty materials even though the exit gradient is the same. The point of significance here is that when we consider the possibility of piping where seepage exits the flow system it is necessary to look at more than just exit gradients – flow quantities and velocities also need to be considered.

5 Two-dimensional flow

Now let us look at the case of a simple 1:1 slope as illustrated in Figure 6. The ground surface profile has a sharp corner at the slope crest and at the slope toe. The purpose here is to examine the exit gradient at the slope toe.

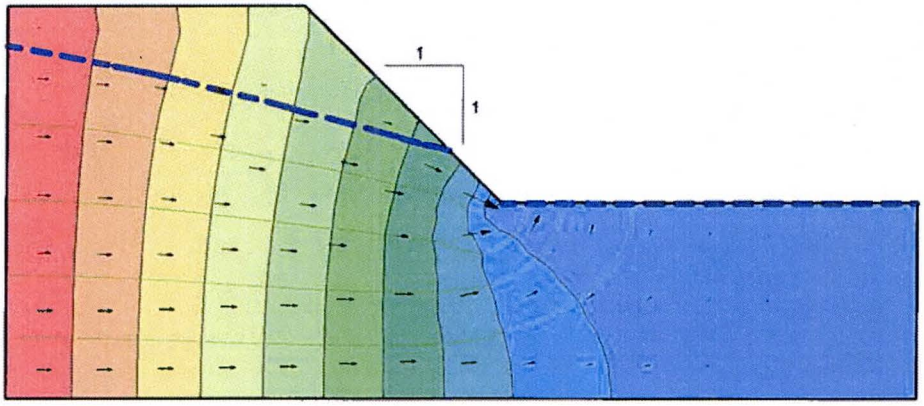


Figure 6

In mathematical terminology a sharp corner in the ground surface like at the slope toe is known as a point of singularity. Fundamentally, this means that the solution to the partial differential equation describing the flow within the system is undefined at the point of singularity. In this case it means that the derivatives of the flow equations are discontinuous at the slope toe. The consequence is that the gradients (derivatives) of the flow equations tend towards infinite at the point of singularity.

These tendencies for the gradients to become ever greater as the computations move closer to the singularity point can be illustrated by looking at the results for various element sizes. The Gauss integration point moves closer to the point of singularity as the element size gets smaller.

Figure 7 shows the mesh in the toe area when the elements are about 1 m in size. The computed x-y gradient at the toe node is 0.982. This is the average of the three Gauss regions common to this node.

When the element size is reduced to about 0.4 m as in Figure 8, the x-y gradient at the toe is 1.66. Reducing the element size to about 0.1 as in Figure 9 increases the x-y gradient at the toe to 3.443.

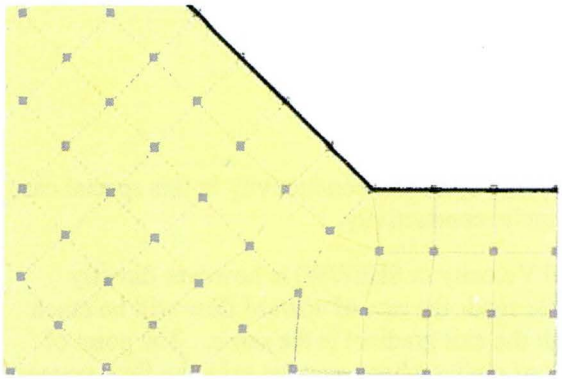


Figure 7 Mesh at toe with elements about 1 m in size

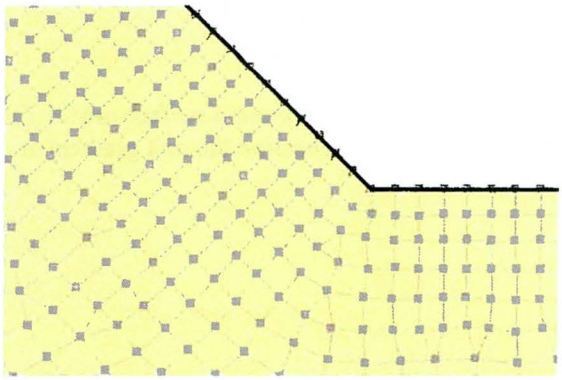


Figure 8 Mesh at the toe with elements about 0.4 m in size

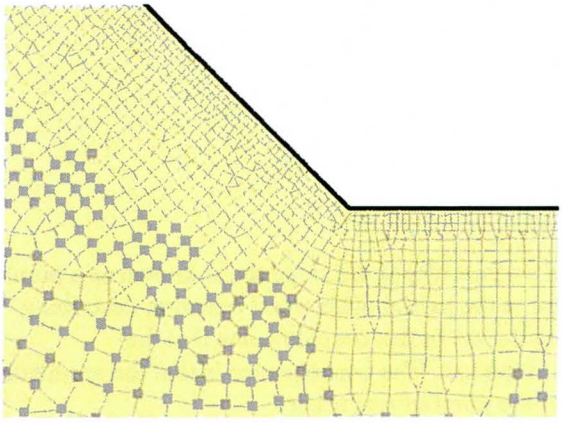


Figure 9 Mesh at the toe with elements about 0.1 m in size

Another way to inspect what happens at the point of singularity is to look at the rate of change of the gradient with distance around the toe. Figure 10 shows contours of the x-y gradients around the toe. The last (highest) contour shown is 1.0 – the value at the toe is 3.44. Note how the distance between each contour interval rapidly decreases towards the slope toe which is reflective of the rapid change in the gradient as the solution moves towards the point of singularity.

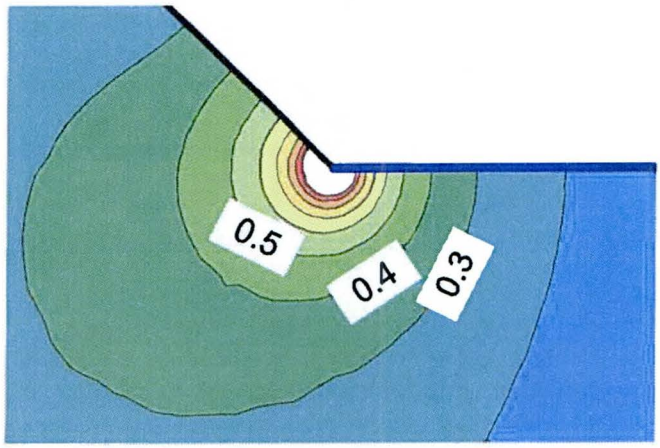


Figure 10 Contours of the x-y gradient – last contour shown is at 1.0

6 Effect of a curved transition

Theoretically as noted earlier the solution to the partial differential flow equation is undefined at points of singularity. The practical implication is that the computed gradients at points of singularity have no physical significance.

Moreover, points of singularity seldom if ever exist in real field situations (except maybe at the corners of concrete structures). If all likelihood the toe area of the slope is curved maybe something like what is illustrated in Figure 11. Now the maximum computed gradient is about 0.7 as opposed to the peak gradient of 3.44 in Figure 10. This is more realistic and further demonstrates that computed gradients at sharp points in the ground surface profile likely do not represent field conditions.

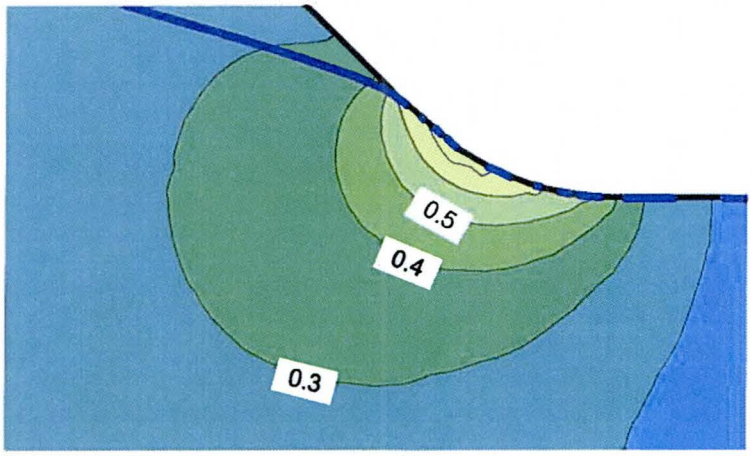


Figure 11 Gradient contours with a curved transition between the slope and the flat ground

In a large field problem it will not always be easy to make nice curves at the transition points. Using straight line segments make the model definition much easier. From a practical modeling perspective it is better to use straight line segments and ignore the exit gradients at sharp breaks in the ground surface profile then attempting to create a curve at the break points. Creating too much geometric complexity at the break points can sometimes obscure the interpretation of the overall global flow system.

7 Interpretation of SEEP/W computed exit gradients

Interpreting the exit gradients computed in a SEEP/W finite element analysis requires some judgment and understanding by the analyst. The computed results cannot always just be accepted at face value as being representative of field conditions. The follow are some issues that the analyst needs to understand and judge when interpreting the results.

1. The common belief that exit gradients should not exceed 1.0 was developed for one-dimensional upward flow which represents a "quick" condition.

2. Upward flow conditions that result in an exit gradient of 1.0 represent a zero effective stress condition. The effective stress state condition is a much more important issue than the gradient. Internal to an earth structure the gradient maybe greater than 1.0 but the effective stress may be fairly high. A gradient greater than 1.0 in such a case does not represent a "quick" condition and will therefore not necessarily be damaging to the structure.

3. Exit gradients are not necessarily the only governing issue – flow quantities also need to be considered in the context of the erodibility of the soil.

4. A seepage face is always an undesirable characteristic of engineered earth structures regardless of the exit gradients computed from a SEEP/W analysis. Generally some form of a granular protection layer is ideally required. Examining granular filter criteria is beyond the scope of this discussion.

5. When examining the exit gradients from an analysis such as the 45-degree slope illustrated earlier it is better to look at the gradients some distance away from sharp breaks in the ground surface profile, say a meter or two away. As noted earlier, the computed gradients at points of singularity are in all likelihood not reprehensive of actual field conditions anyway.

6. If you want to interpret SEEP/W exit gradients in the context of what has traditionally been done with flow nets, then you should look at the head loss between two equipotential lines over a significant distance and compute an average exit gradient of sorts.

7. Attempting to curve the ground profile with too much geometric complexity is not recommended as noted in the previous section. From a modeling prescribe it is better to simply ignore the computed gradients at sharp breaks in the ground surface profile.

8 Concluding remarks

The exit gradients computed with a SEEP/W analysis cannot always be taken at face value as being representative of the actual field conditions. Considerable understanding and judgment is vital to interpreting and using the computed results.

APPENDIX C

RCC MIX DESIGN

MEMO

To:	**Report Appendix** **90% Design Report** **Roller Compacted Concrete Dam** **Oak Street Detention Basin & Storm Drain Project** **Mesa, Arizona** **FCD Contract 2009C021**	**File No.:**	**17-2011-4050**
From:	**Richard L. Jenkins**	**Reviewed:**	**Ken W. Wylie**
Tel:	**(505) 821-1801**		**Brett A. Howey, PE**
Fax:	**(505) 821-7371**		
Date:	**September 11, 2012**		

Subject: Roller-Compacted Concrete Mix Design

Included within this Technical Memorandum is AMEC Environment & Infrastructure, Inc's (AMEC) documentation of the "informational" Roller-Compacted Concrete, (RCC), Mix Design for the Oak Street Detention Basin and Storm Drain Project. Materials were selected from locally available sources and proportions were selected using several published technical "guides" and AMEC's experience on similar projects.

Published "guides" used were:

- American Concrete Institute's, ACI 211.3R, Guide for Selecting Proportions for No-Slump Concrete.
- National Concrete Pavement Technology Center's, Guide for Roller-Compacted Concrete Pavements.
- Portland Cement Association's, Design Manual for Small RCC Dams.

The Design Process included the following steps:

- Decide on design approach. (*Soil Compaction Method* was chosen)
    - Choose well-graded aggregates and determine percentages of each that will meet the selected gradation band.
        - Gather data on the materials such as gradations, specific gravities, and absorptions.
    - Select a mid-range cementitious content. (8% was selected)
    - Develop moisture-density relationship plots to determine estimated optimum moisture content. (Proctor in general accordance with ASTM D-1557 method D was ran and an optimum moisture of 5.6% was determined. A moisture content of 6.6% was selected for the trial batches.)

AMEC Environment & Infrastructure, Inc.
8519 Jefferson NE
Albuquerque, New Mexico 87113
Tel. (505) 821-1801
Fax. (505) 821-7371
www.amec.com

- Batch lab trials at three (3) different cementitious contents. (choose a low-range, a mid-range, and a high-range of cementitious content)
- Compressive Strength Test specimens were cast using the vibrating hammer method of ASTM C 1435.
  - Two (2) Specimens are to be tested at 7-day and at 28-day age.
    - Also, one (1) Specimen was cast from each trial and was tested at 56-day age for later age strength projection.
- Test specimens and select required cementitious content.
- Calculate mixture proportions.

The lab trial batches did not include Fly Ash as part of the cementitious material. The lab trials were performed to estimate the constituent proportions that a contractor might use. For RCC production, potential use of Fly Ash should be based on the aggregate source and whether or not that aggregate has an Alkali-Silica reaction that results in expansion that exceeds the maximum allowed. The recommended test method to determine the reactivity is ASTM C 1260 mortar bars. Should the test data indicate an expansion that exceeds the maximum allowed (normally 0.10% maximum @ 16-day age), the aggregate should be rejected or additional testing using ASTM C 1567 should be performed using the proposed cement in combination with a proposed pozzolan to determine the quantities necessary to meet the specification requirements. The local agency specifications address the issue of Alkali-Silica Reactivity and an overview of their specifications is listed below.

- Maricopa Association of Governments (MAG) Specifications allow up to 25% fly ash, but do not require its use.
- The City of Phoenix recommends up to 15% fly ash replacement for most mixes, but that has to be added to the project specifications by the designer.
- ADOT standard spec's allow up to 20% fly ash with a 1.2:1 replacement ratio. In the past couple of years, ADOT has been adding ASTM C1260 and ASTM C1567 testing to project specifications. Because of that, many suppliers are doing this testing as part of their annual source qualifications, so they have the test results ready in case they want to bid one of those projects.

Using these as a guide, we have included the ASTM C 1260 and ASTM C 1567 testing and expansion limits in the Subsection 222.2.2 of the specifications along with adding requirements and limits to the Class F fly ash specifications in Subsection 222.2.5. As stated above, many of the suppliers are already performing this testing and already have the results.

Supporting attachments to this Technical Memorandum include:

- "DRAFT" of "informational" RCC Mix Design proportions, (US Customary System of Units).
  - Two (2) different sets of attachments are included with different cementious contents and proportions in each. One has proportions that target 2500-psi compressive strength @ 28-day age and the other targets 3500-psi compressive strength @ 28-day age. The 3500-psi is including a 1000-psi over-design for the 2500-psi specified strength.
- Combined aggregate blend grading with grading chart.
- Proctor results with plot.
- RCC mix design laboratory trial batch summary
  - Trial mix proportions, unit weights, and compressive strengths.

  - Including an "overview" of the procedure used for fabrication of RCC compressive strength specimens.
- Materials information for the various RCC constituent components.
  - Coarse aggregate grading and specific gravity and absorption.
  - Fine aggregate grading and specific gravity and absorption.
  - Cement properties test report. (currently not available – to be provided with 100% design report)

Roller-Compacted Concrete Mix Design

Informational Proportions-US Customary System of Units

Project: Oak Street Detention Basin and Storm Drain Project
Job No: 17-2011-4050.003
Lab No:
Supplier:
Date: 08/01/12
Amended: 09/11/12
Mix Code:
Mix Description: f 'c=2500-psi / NON-Air-Entrained NO Slump Roller-Compacted Concrete
Mix Use: Roller-Compacted Concrete

Mix Proportions per Cubic Yard

Material	Weight		Specific Gravity	Absorption	Volume
: : Portland Cement Type I-II Low Alkali	264	lbs.	3.15		1.34 cf
Water	203	lbs.	1.00		3.25 cf
	24.4	*gal.*		%	
CEMEX Plant No. 1451 : Concrete Sand (SSD)	1673	lbs.	2.627	1.83	10.21 cf
CEMEX Plant No. 1451 Sz. 57 Coarse Agg. (SSD)	1963	lbs.	2.638	1.09	11.93 cf
Air	1.0	%			0.27 cf
Totals		4103	lbs.		27.00 cf

	Target Plastic Properties	Specification
Slump (ASTM C 143)	0.00 in.	0.5 inch max
Unit Weight from Recommended Proportions.	152.0 pcf	
Air Content (ASTM C 231)	1.0 %	

Mix Characteristics

Cement Content @ Air Content of	1.0 %	264 lbs.
Total Cementitious Content		264 lbs.
Water/Cementitious Ratio		0.769
Fine-to-Total Aggregate		46.0 %

Compressive Strength Results (ASTM C 1435, C39, & C 42)

	From Original Mix Design	From 2012 Field Performance Data
@ 7-days	1750 -psi (1)	-psi
@ 28-days	2500 -psi (1)	-psi
@ 56-days	2860 -psi (1)	-psi

Specified Compressive Strength @ 28-days f 'c= 2500 -psi

PE: _____

(1) Interpolated Strength Results from "Strength vs. Water-Cementitious Ratio Graphs" from the Concrete Laboratory Batch Trials actual strength data.

(2) During Design Process: Concrete Specimens were compacted in general accordance with ASTM C -1435 using a vibrating hammer with an attached Tamping Head.

Roller Compacted Concrete, PCA Suggested Grading Limits
Aggregate Blend Grading

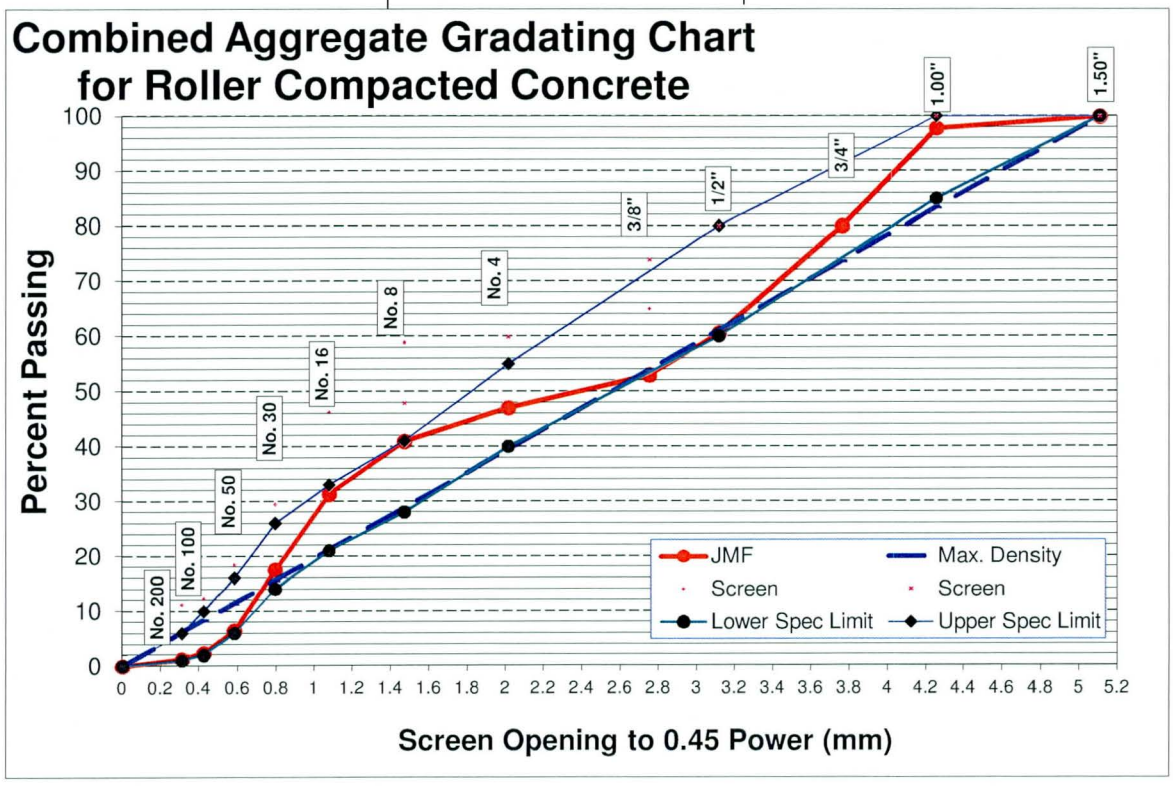
Mix: Phoenix RCC
Mix Code:
Project: Oak Street Basin RCC Dam

Date: 7/5/2012
AMEC Lab ID:
AMEC Project No.: 17-2011-4050.003

Trial: Trl.-1 Basis: Gradations from Phoenix AMEC

	Material									
		Sz. 57-				Concrete Sand-		Total Percentage	AMEC Suggested RCC	PCA Suggested RCC
Percent Used		54.0				46.0		100.0	Grading Limits	Grading Limits
Sieve								Composite		small Dam
1-1/2"		100				100		100	100	100
1"		96				100		98	85 - 100	85 - 100
3/4"		63				100		80		
1/2"		27				100		61	60 - 80	60 - 80
3/8"		13				100		53		
No. 4		2				100		47	40 - 55	40 - 55
No. 8		1				88		41	28 - 41	28 - 41
No.10		1				84		39		
No.16		1				67		31	21 - 33	21 - 33
No. 30		1				37		18	14 - 26	15 - 27
No. 40		1				24		12		
No. 50		1				13		7	6 - 16	10 - 20
No. 80		1				8		4		
No. 100		1				4		2	2 - 10	5 - 13
No. 200		0.6				2.0		1.2	1.0 - 6.0	2.0 - 7.0

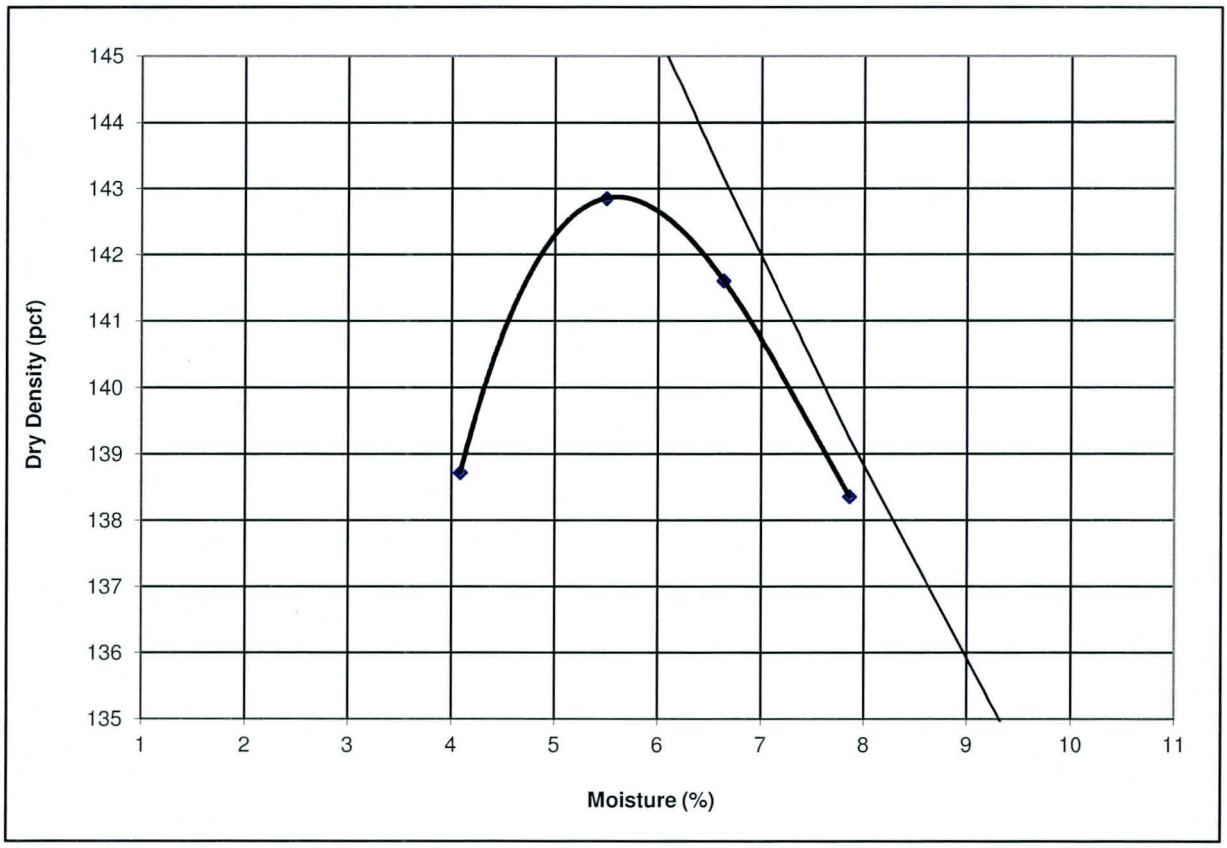
Material	Product Name and Product Co	Source
Sz. 57-	Cemex Sz. 57	Cemex, Plant No. 1451 @ South 19th Avenue
Concrete Sand-	Cemex Concrete Sand	Cemex, Plant No. 1451 @ South 19th Avenue



PROJECT:	Oak Street Basin RCC Dam	**JOB NO:**	17-2011-4050.003
CLIENT:	AMEC Engineering-Phoenix	**WORK ORDER NO:**	3
COLOR/TYPE MATERIAL:	Roller Compacted Concrete-Soil Cement Method Proctor	**LAB NO:**	10
SAMPLE SOURCE:	CEMEX: Plant 1451 @ South 19th Avenue, Phoenix	**DATE Sampled:**	6/19/2012
		SAMPLED by:	Dave Christensen / CEMEX

LABORATORY COMPACTION CHARACTERISTICS OF SOILS USING MODIFIED EFFORTS (56,000ft-lb-ft/cu.ft) (ASTMD1557D)

Type of Rammer: Mechanical



Point No:	#1	#2	#3	#4	
Dry Density(lbs/ft3):	138.7	142.9	141.6	138.4	
Moisture Content (%):	4.1	5.5	6.6	7.9	

MAXIMUM DRY DENSITY (pcf):	**142.9**
OPTIMUM MOISTURE (%):	**5.6**

NOTE: THE ZERO AIR VOIDS CURVE REPRESENTS AN ASSUMED SPECIFIC GRAVITY OF: 2.71 ASSUMED.

Remarks: Cementitious Materials %: 8% = Wt. of C' / (Wt. C' = Oven Dry Agg. Wt.), Solving results in 8.7-lbs Cement for each 100-lbs of Dry Aggregate.

Cement: Unknown Producer / Plant / Type, Fly Ash: None

DATA provided by AMEC's Phoenix Office.

Reviewed By: *Richard L. Jenkins*

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Albuquerque, NM 87113
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Overview of Procedure for fabrication of RCC Compressive Strength Specimens.

(ASTM C 1435-08 "Standard Practice for Molding Roller-Compacted Concrete in Cylinder Molds using a Vibrating Hammer")

Apparatus:

1. A single-use plastic cylindrical mold 6-inch diameter and 12-inch height shall be used for fabricating the compressive strength test specimens.
2. A Rigid Cylindrical Sleeve (It shall be made of steel or other hard metal resistant to cement paste corrosion.). The sleeve shall be split and hinged on one side so that it can be opened to remove the plastic mold; adjustable clamps shall be provided on the other side for tightening the sleeve around the mold.
3. A Vibrating Hammer. (Such as a Makita Model: HM1203C, or a DeWalt Model: D25900)
    - The vibrating hammer shall have a mass of 22-lbs. (+/-) 3-lbs.
    - It also shall have a minimum power input of 900 W and be capable of providing 2000 impacts per minute (+/-) 200 impacts per minute
4. Tamping Plate. A Circular steel plate attached to a steel shaft, which is inserted into the vibrating hammer chuck.
5. Various small tools to facilitate the specimen's fabrication.

Cylindrical Specimens are fabricated in four (4) lifts using the vibrating hammer with the attached tamping plate head. Insert a plastic mold into the rigid cylindrical sleeve and adjust the clamps. Place enough RCC into the mold so that the mold will be filled to approximately one-fourth of its volume after consolidation. Use a tamping rod to distribute the loose concrete as it is added. Use a square-faced scoop or similar device for adding the concrete. Place the vibrating hammer with attached tamping plate head onto the concrete. Start the vibrating hammer and allow the concrete to consolidate around the tamping plate. Observe the concrete in the annular space between the edge of the tamping plated and the inside of the mold. Mortar should fill the annular space. Once the mortar forms a ring around the total perimeter of the tamping plate, stop the vibrating hammer and remove from cylinder. Repeat the procedure for the second and third lifts filling the mold to approximately one-half and three-fourths of its volume respectively. For the fourth lift, overfill the mold mounding the concrete above the top of the mold. If the concrete is consolidated below the top of the mold, stop the hammer and add additional concrete into the mold so that when consolidated, the concrete can be finished smooth and level with the top of the mold.

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Roller-Compacted Concrete Mix Design
Laboratory Trial Batch Summary

Project: Oak Street Detention Basin and Storm Drain Project
Client: AMEC- Phoenix Engineering
Job No: 17-2011-4050.003
Mix Description: NON-Air-Entrained, Compressive Strength @ 28-day Age
Roller-Compacted Concrete, 1.0" Nominal Maximum Size Aggregate

Lab No:
Date: 7/5/2012
Report Date: 7/9/2012
Amended: *9/10/2012*
By: Richard L. Jenkins & Ken Wylie

Report Summary

Laboratory Mix Batches	Trial Mix ID: / Title: / Batch Date:	**Trl. "A-5" RCC-NA** 7/2/2012		**Trl. "A-8" RCC-NA** 7/2/2012		**Trl. "A-11" RCC-NA** 7/2/2012	
Material Proportions per CY	Specific Gravity	Weight lbs. (1) (2)	Volume cf.	Weight lbs. (1) (2)	Volume cf.	Weight lbs. (1) (2)	Volume cf.
: : Type I-II Low-Alkali Cement	3.15	191	0.97	309	1.57	422	2.15
Water	1.00	205	3.29	208	3.33	210	3.37
	H_2O in Gallons	*24.61 gallons*		*24.97 gallons*		*25.21 gallons*	
CEMEX Plant No.1451, Concrete Sand (SSD)	2.627	1702	10.38	1653	10.08	1602	9.77
CEMEX Plant No. 1451 , Sz. 57_1.00" Coarse Agg (SSD)	2.638	1999	12.14	1941	11.79	1880	11.42
Gravimetric Air Content, % (5)	-	0.8	0.22	0.8	0.22	1.1	0.30
			27.0		27.0		27.0
Plastic Properties							
Slump, in. (4)		N/A		N/A		N/A	
Ambient Temperature, deg. F (4)							
Concrete Temperature, deg. F (4)							
Cementitious Content, pcy		191		309		422	
Water/Cementitious Ratio (3)		1.073		0.673		0.498	
Fine-to-Total Aggregate, %(wt.) (3)		46.0		46.0		46.0	
Target Air Content, %		2.0		2.0		2.0	
Gravimetric Air Content, % (5)		0.80		0.81		1.12	
Measured Unit Weight, pcf (4)		151.76		152.26		152.35	
Theoretical **"WET"** Unit Weight, pcf (2) (3)		151.7		152.3		152.3	
Theoretical Maximum Unit Weight, pcf (3)		153.0		153.5		154.0	
Theoretical **"Fresh Dry"** Unit Weight (if Batched Proportions contained 6.6% moisture), pcf (2) (3)		**142.3**		**142.8**		**142.9**	
	Lab Specimen #	**212646**		**212647**		**212648**	
Compressive Strength, psi	**ASTM C 192, C 1435, & C 39**	psi		psi		psi	
@ 7 days (6)(9)	Normal Cure	830		2530		4540	
@ 28 days (6)(9)	Normal Cure	1160		3330		5320	
@ 56 days (7)(9)	Normal Cure	1240		3690		5580	
	7-day to 28-day ratio:	*1.398*		*1.316*		*1.172*	

Notes:
(1) units in lbs. unless stated otherwise
(2) proportions based on Gravimetric Air Content
(3) as calculated
(4) as measured
(5) as CALCULATED from Unit Weight
(6) average of two (2) specimens
(7) one (1) specimen
(8) 4" X 8" Cylinders
(9) 6" X 12" Cylinders

Concrete Laboratory Trial Batch
Compressive Strength Data Summary

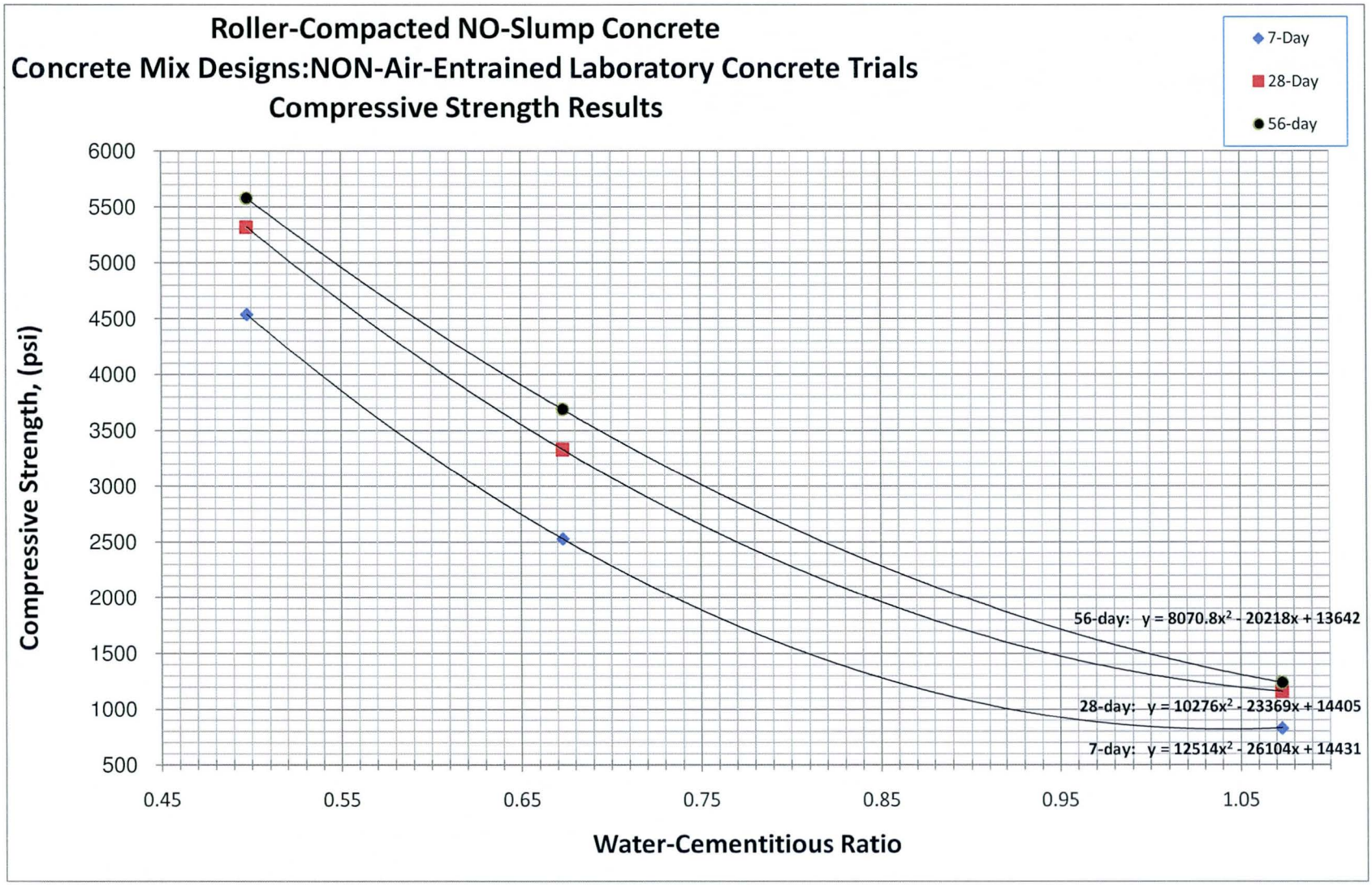
Report Summary

Project: Oak Street Detention Basin and Storm
Client: AMEC- Phoenix Engineering
Job No: 17-2011-4050.003
Lab No:

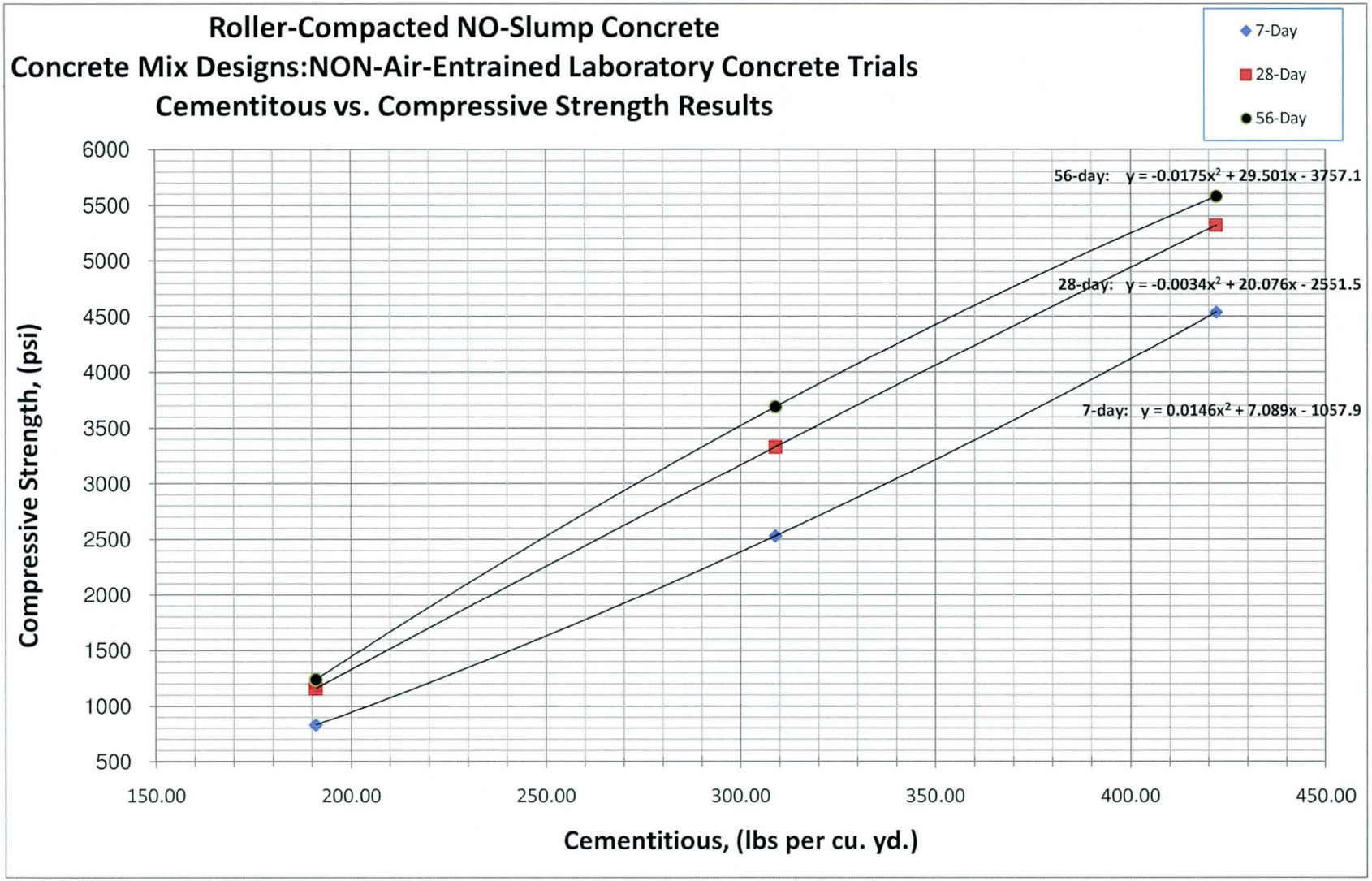
Date: 7/5/2012
Report Date: 7/9/2012
Amended: 9/10/2012
By: Richard L. Jenkins & Ken Wylie

Trial Mix ID:	**Trl. "A-5"**	**Trl. "A-8"**	**Trl. "A-11"**
Title:	**RCC-NA**	**RCC-NA**	**RCC-NA**
Batch Date:	7/2/2012	7/2/2012	7/2/2012
Water-Cementitious Ratio	**1.073**	**0.673**	**0.498**
7-Day Compressive Strength Data			
Lab Specimen #	**212646**	**212647**	**212648**
Test Date	7/9/2012	7/9/2012	7/9/2012
Specimen 1, psi	790	2520	4510
Specimen 2, psi	870	2540	4560
Specimen 3, psi			
Average, psi	**830**	**2530**	**4535**
28-Day Compressive Strength Data			
Test Date	7/30/2012	7/30/2012	7/30/2012
Specimen 1, psi	1160	3340	5210
Specimen 2, psi	1160	3320	5420
Specimen 3, psi			
Average, psi	**1160**	**3330**	**5315**
56 Day Compressive Strength Data			
Test Date	8/27/2012	8/27/2012	8/27/2012
Specimen 1, psi	1240	3690	5580
Specimen 2, psi			
Specimen 3, psi			
Average, psi	**1240**	**3690**	**5580**

Water-Cementitious vs. Compressive Strength Curve



Cementitious vs. Compressive Strength Curve



REPORT of AGGREGATE
PHYSICAL PROPERTIES

Project: Oak Street Detention Basin & Storm Drain Project
Project No: 17-2011-4050.003
Client: Phoenix-AMEC Engineering
Lab No.:
Material Tested: Sz. 57
Sample Date: 6/19/2012
Source: CEMEX: Plant No. 1451
Report Date: **Amended:**
Product Code:
Specifications:
Reviewed by:

SIEVE ANALYSIS

ASTM C136 AASHTO T27

Sieve Size	Accum. % Passing	Specifications
300 mm (12")		
250 mm (10")		
225 mm (9")		ASTM C-33
200 mm (8")		Sz. 57
150 mm (6")		Coarse Aggregate
125 mm (5")		Grading Limit
100 mm (4")		
75.0 mm (3")		
63.0 mm (2-1/2")		
50.0 mm (2")		
37.5 mm (1-1/2")	100	100
31.5 mm (1-1/4")		
25.0 mm (1")	96	95 - 100
19.0 mm (3/4")	63	
12.5 mm (1/2")	27	25 - 60
9.5 mm (3/8")	13	
6.3 mm (1/4")	4	
4.75mm (No. 4)	2	0 - 10
2.36 mm (No. 8)	1	0 - 5
2.00 mm (No. 10)	1	
1.18mm (No.16)	1	
0.600 mm (No.30)	1	
0.425mm (No. 40)	1	
0.300mm (No. 50)	1	
0.180mm (No. 80)		
0.150mm (No.100)	1	
0.075mm (No. 200)	0.6	0.0 - 2.0
ASTM C117 / AASHTO T11		
Soil Classification		
Moisture Content, %		
ASTM C566 AASHTO T255		
Fract. 1 Face, %		
Fract. 2 Face, %		
Fineness Modulus		
ASTM C136 AASHTO T27		

TEST RESULTS

Standard	PHYSICAL PROPERTIES		Results	Specification
ASTM C 29	Unit Weight	Unit Weight, lbs./cu.ft=		
AASHTO T 19	& Voids	Voids, %=		
		☐ Jigged ☐ Loose ☐ Rodded		
	Coarse	Bulk Specific Gravity (dry)=	2.609	
ASTM C 127	Specific	Bulk Specific Gravity, SSD=	2.638	
AASHTO T 85	Gravity &	Apparent Specific Gravity=	2.686	
	Absorption	Absorption, %=	1.09	
	Fine	Bulk Specific Gravity (dry)=		
ASTM C 128	Specific	Bulk Specific Gravity, SSD=		
AASHTO T 84	Gravity &	Apparent Specific Gravity=		
	Absorption	Absorption, %=		
ASTM D 2419	Sand	Sand Equivalent, %=		
AASHTO T 176	Equivalent			
ASTM D 4791	Flat &	Flat & Elongated, %=		
	Elongated	Ratio=		
ASTM C 131	L. A.	Small Coarse Loss, %=		
AASHTO T 96	Abrasion	Grading/ Revs.=		
ASTM C 535	L. A.	Large Coarse Loss, %=		
	Abrasion	Grading/ Revs.=		
	Soundness	Coarse Soundness Loss, %=		
ASTM C 88		Magnesium No. of Cycles=		
AASHTO T 104	Soundness	Fine Soundness Loss, %=		
		Magnesium No. of Cycles=		
ASTM C 142	Clay/Friable	Coarse Aggregate, %=		
AASHTO T 112	Particles	Fine Aggregate, %=		
ASTM 2974	Organic	Organic Matter % =		
AASHTO T267	Matter	Ash Content % =:		
ASTM C 123	Lightweight	Coarse Aggregate, %=		
AASHTO T 113	Pieces(Sp.Gr.2.0)	Fine Aggregate, %=		
ASTM C 1252	Fine Aggregate	Uncompacted Voids,%=		
AASHTO T 304	Angularity	Method C (as received material)		
	Compaction	Optimum Moisture, %=		
		Max. Density, lbs./cu.ft.=		
ASTM D 4318	Liquid Limit,	Liquid Limit=		
AASHTO T 89/90	Plastic Limit &	Plastic Limit=		
	Plasticity Index	Plasticity Index=		

NOTE: Data provided by AMEC's Phoenix office.

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Fax (505) 821-7371
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Project: Oak Street Detention Basin & Storm Drain Project
Project No: 17-2011-4050.003
Client: Phoenix-AMEC Engineering
Lab No.:
Material Tested: Concrete Sand
Sample Date: 6/19/2012
Source: CEMEX: Plant No. 1451
Report Date: **Amended:**
Product Code:
Specifications:
Reviewed by:

SIEVE ANALYSIS

ASTM C136 AASHTO T27

Sieve Size	Accum. % Passing	Specifications
300 mm (12")		
250 mm (10")		
225 mm (9")		
200 mm (8")		
150 mm (6")		ASTM C-33
125 mm (5")		
100 mm (4")		Fine Aggregate
75.0 mm (3")		Grading Limit
63.0 mm (2-1/2")		
50.0 mm (2")		
37.5 mm (1-1/2")		
31.5 mm (1-1/4")		
25.0 mm (1")		
19.0 mm (3/4")		
12.5 mm (1/2")		
9.5 mm (3/8")	100	100
6.3 mm (1/4")		
4.75mm (No. 4)	100	95 - 100
2.36 mm (No. 8)	88	80 - 100
2.00 mm (No. 10)	83	
1.18mm (No.16)	67	50 - 85
0.600 mm (No.30)	37	25 - 60
0.425mm (No. 40)	24	
0.300mm (No. 50)	13	5 - 30
0.180mm (No. 80)		
0.150mm (No.100)	4	0 - 10
0.075mm (No. 200)	2.0	0.0 - 3.0
ASTM C117 / AASHTO T11		
Soil Classification		
Moisture Content, %		
ASTM C566 AASHTO T255		
Fractured Face, %		
2 faces		
Fineness Modulus (FM)	2.92	
ASTM C136 AASHTO T27		

TEST RESULTS

Standard	PHYSICAL PROPERTIES			Results	Specification
ASTM C 29	Unit Weight	Unit Weight, lbs./cu.ft=			
AASHTO T 19	& Voids	Voids, %=			
		☐ Jigged ☐ Loose ☐ Rodded			
	Coarse	Bulk Specific Gravity (dry)=			
ASTM C 127	Specific	Bulk Specific Gravity, SSD=			
AASHTO T 85	Gravity &	Apparent Specific Gravity=			
	Absorption	Absorption, %=			
	Fine	Bulk Specific Gravity (dry)=		2.580	
ASTM C 128	Specific	Bulk Specific Gravity, SSD=		2.627	
AASHTO T 84	Gravity &	Apparent Specific Gravity=		2.708	
	Absorption	Absorption, %=		1.83	
ASTM D 2419	Sand	Sand Equivalent, %=			
AASHTO T 176	Equivalent				
ASTM D 4791	Flat &	Flat & Elongated, %=			
	Elongated	Ratio=			
ASTM C 131	L. A.	Small Coarse Loss, %=			
AASHTO T 96	Abrasion	Grading/ Revs.=			
ASTM C 535	L. A.	Large Coarse Loss, %=			
	Abrasion	Grading/ Revs.=			
	Soundness	Coarse Soundness Loss, %=			
ASTM C 88		Magnesium No. of Cycles=			
AASHTO T 104	Soundness	Fine Soundness Loss, %=			
		Magnesium No. of Cycles=			
ASTM C 142	Clay/Friable	Coarse Aggregate, %=			
AASHTO T 112	Particles	Fine Aggregate, %=			
ASTM C 123	Lightweight	Coarse Aggregate, %=			
AASHTO T 113	Pieces(Sp.Gr. 2.0)	Fine Aggregate, %=			
ASTM C 40	Organic	Lighter Than Standard	LTS		LTS or ETS
AASHTO T-21	Impurites	Equal to Standard	ETS		
		Darker Than Standard	DTS		
	Compaction	Optimum Moisture, %=			
		Max. Density, lbs./cu.ft.=			
ASTM D 4318	Liquid Limit,	Liquid Limit=			
AASHTO T 89/90	Plastic Limit &	Plastic Limit=			
	Plasticity Index	Plasticity Index=			

note: Data provided by AMEC's Phoenix office.

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Roller-Compacted Concrete Mix Design

Informational Proportions-US Customary System of Units

Project: Oak Street Detention Basin and Storm Drain Project
Job No: 17-2011-4050.003
Lab No:
Supplier:
Date: 07/31/12
Amended: 09/11/12
Mix Code:
Mix Description: f 'c=2500-psi / NON-Air-Entrained
NO Slump Roller-Compacted Concrete with 1000-psi Over-Design
Mix Use: Roller-Compacted Concrete

Mix Proportions per Cubic Yard

Material	Weight	Specific Gravity	Absorption	Volume
: : Portland Cement Type I-II Low Alkali	320 lbs.	3.15		1.63 cf
Water	210 lbs.	1.00		3.37 cf
	25.2 gal.		%	
CEMEX Plant No. 1451 : Concrete Sand (SSD)	1642 lbs.	2.627	1.83	10.02 cf
CEMEX Plant No. 1451 Sz. 57 Coarse Agg. (SSD)	1929 lbs.	2.638	1.09	11.72 cf
Air	1.0 %			0.27 cf
Totals	4101 lbs.			27.00 cf

Target Plastic Properties		Specification
Slump (ASTM C 143)	0.00 in.	0.5 inch max
Unit Weight from Recommended Proportions.	151.9 pcf	
Air Content (ASTM C 231)	1.0 %	

Mix Characteristics

Cement Content @ Air Content of	1.0 %	320 lbs.
Total Cementitious Content		320 lbs.
Water/Cementitious Ratio		0.656
Fine-to-Total Aggregate		46.0 %

Compressive Strength Results (ASTM C 1435, C39, & C 42)

	From Original Mix Design	From 2012 Field Performance Data
@ 7-days	2750 -psi (1)	-psi
@ 28-days	3560 -psi (1)	-psi
@ 56-days	3910 -psi (1)	-psi

Specified Compressive Strength @ 28-days f 'c= 2500 -psi
Compressive Strength with 1000-psi Over-design @ 28-days = 3500 -psi

PE: _____

(1) Interpolated Strength Results from "Strength vs. Water-Cementitious Ratio Graphs" from the Concrete Laboratory Batch Trials actual strength data.

(2) During Design Process: Concrete Specimens were compacted in general accordance with ASTM C -1435 using a vibrating hammer with an attached Tamping Head.

Roller Compacted Concrete, PCA Suggested Grading Limits
Aggregate Blend Grading

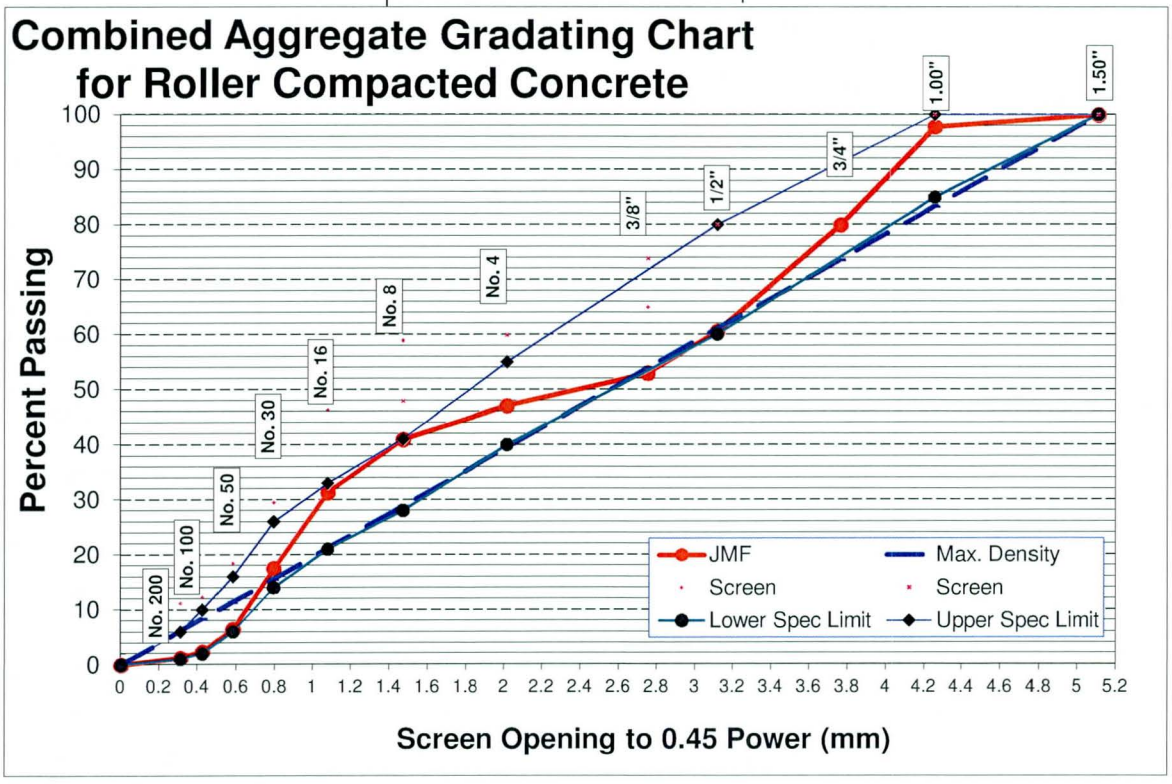
Mix: Phoenix RCC
Mix Code:
Project: Oak Street Basin RCC Dam

Date: 7/5/2012
AMEC Lab ID:
AMEC Project No.: 17-2011-4050.003

Trial: Trl.-1 Basis: Gradations from Phoenix AMEC

	Material									
		Sz. 57-				Concrete Sand-		Total Percentage	AMEC Suggested RCC	PCA Suggested RCC
Percent Used		54.0				46.0		100.0	Grading Limits	Grading Limits
Sieve								Composite		small Dam
1-1/2"		100				100		100	100	100
1"		96				100		98	85 - 100	85 - 100
3/4"		63				100		80		
1/2"		27				100		61	60 - 80	60 - 80
3/8"		13				100		53		
No. 4		2				100		47	40 - 55	40 - 55
No. 8		1				88		41	28 - 41	28 - 41
No.10		1				84		39		
No.16		1				67		31	21 - 33	21 - 33
No. 30		1				37		18	14 - 26	15 - 27
No. 40		1				24		12		
No. 50		1				13		7	6 - 16	10 - 20
No. 80		1				8		4		
No. 100		1				4		2	2 - 10	5 - 13
No. 200		0.6				2.0		1.2	1.0 - 6.0	2.0 - 7.0

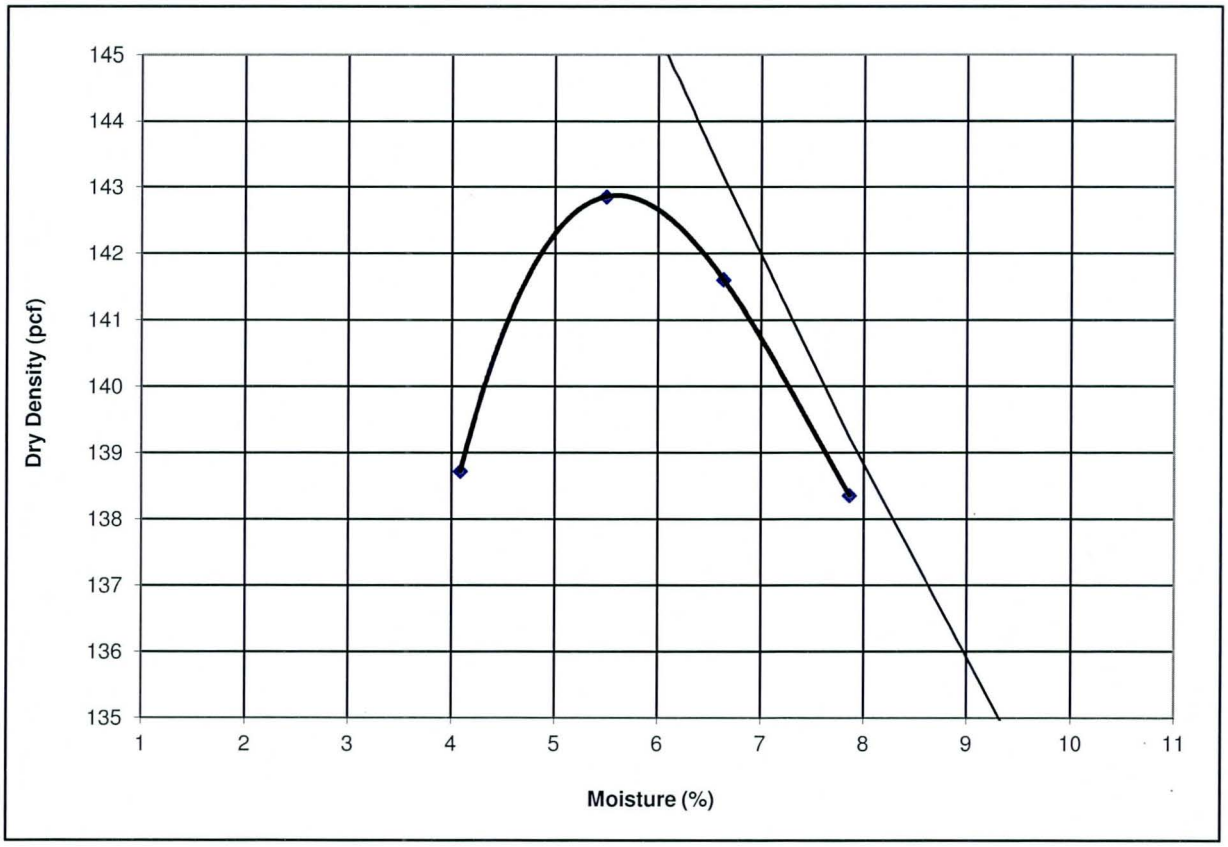
Material	Product Name and Product Co	Source
Sz. 57-	Cemex Sz. 57	Cemex, Plant No. 1451 @ South 19th Avenue
Concrete Sand-	Cemex Concrete Sand	Cemex, Plant No. 1451 @ South 19th Avenue



PROJECT:	Oak Street Basin RCC Dam	**JOB NO:**	17-2011-4050.003
CLIENT:	AMEC Engineering-Phoenix	**WORK ORDER NO:**	3
COLOR/TYPE MATERIAL:	Roller Compacted Concrete-Soil Cement Method Proctor	**LAB NO:**	10
SAMPLE SOURCE:	CEMEX: Plant 1451 @ South 19th Avenue, Phoenix	**DATE Sampled:**	6/19/2012
		SAMPLED by:	Dave Christensen / CEMEX

LABORATORY COMPACTION CHARACTERISTICS OF SOILS USING MODIFIED EFFORTS (56,000ft-lb-ft/cu.ft) (ASTMD1557D)

Type of Rammer: Mechanical



Point No:	#1	#2	#3	#4	
Dry Density(lbs/ft3):	138.7	142.9	141.6	138.4	
Moisture Content (%):	4.1	5.5	6.6	7.9	

MAXIMUM DRY DENSITY (pcf):	**142.9**
OPTIMUM MOISTURE (%):	**5.6**

NOTE: THE ZERO AIR VOIDS CURVE REPRESENTS AN ASSUMED SPECIFIC GRAVITY OF: 2.71 ASSUMED.

Remarks: Cementitious Materials %: 8% = Wt. of C' / (Wt. C' = Oven Dry Agg. Wt.), Solving results in 8.7-lbs Cement for each 100-lbs of Dry Aggregate.

Cement: Unknown Producer / Plant / Type, Fly Ash: None

DATA provided by AMEC's Phoenix Office.

Reviewed By: *Richard L. Jenkins*

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Overview of Procedure for fabrication of RCC Compressive Strength Specimens.

(ASTM C 1435-08 "Standard Practice for Molding Roller-Compacted Concrete in Cylinder Molds using a Vibrating Hammer")

Apparatus:

1. A single-use plastic cylindrical mold 6-inch diameter and 12-inch height shall be used for fabricating the compressive strength test specimens.
2. A Rigid Cylindrical Sleeve (It shall be made of steel or other hard metal resistant to cement paste corrosion.). The sleeve shall be split and hinged on one side so that it can be opened to remove the plastic mold; adjustable clamps shall be provided on the other side for tightening the sleeve around the mold.
3. A Vibrating Hammer. (Such as a Makita Model: HM1203C, or a DeWalt Model: D25900)
    - The vibrating hammer shall have a mass of 22-lbs. (+/-) 3-lbs.
    - It also shall have a minimum power input of 900 W and be capable of providing 2000 impacts per minute (+/-) 200 impacts per minute
4. Tamping Plate. A Circular steel plate attached to a steel shaft, which is inserted into the vibrating hammer chuck.
5. Various small tools to facilitate the specimen's fabrication.

Cylindrical Specimens are fabricated in four (4) lifts using the vibrating hammer with the attached tamping plate head. Insert a plastic mold into the rigid cylindrical sleeve and adjust the clamps. Place enough RCC into the mold so that the mold will be filled to approximately one-fourth of its volume after consolidation. Use a tamping rod to distribute the loose concrete as it is added. Use a square-faced scoop or similar device for adding the concrete. Place the vibrating hammer with attached tamping plate head onto the concrete. Start the vibrating hammer and allow the concrete to consolidate around the tamping plate. Observe the concrete in the annular space between the edge of the tamping plated and the inside of the mold. Mortar should fill the annular space. Once the mortar forms a ring around the total perimeter of the tamping plate, stop the vibrating hammer and remove from cylinder. Repeat the procedure for the second and third lifts filling the mold to approximately one-half and three-fourths of its volume respectively. For the fourth lift, overfill the mold mounding the concrete above the top of the mold. If the concrete is consolidated below the top of the mold, stop the hammer and add additional concrete into the mold so that when consolidated, the concrete can be finished smooth and level with the top of the mold.

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Roller-Compacted Concrete Mix Design
Laboratory Trial Batch Summary

Project: Oak Street Detention Basin and Storm Drain Project
Client: AMEC- Phoenix Engineering
Job No: 17-2011-4050.003
Mix Description: NON-Air-Entrained, Compressive Strength @ 28-day Age
Roller-Compacted Concrete, 1.0" Nominal Maximum Size Aggregate

Lab No:
Date: 7/5/2012
Report Date: 7/9/2012
Amended: *9/10/2012*
By: Richard L. Jenkins & Ken Wylie

Report Summary

Laboratory Mix Batches	Trial Mix ID: / Title: / Batch Date:	**Trl. "A-5" RCC-NA** 7/2/2012		**Trl. "A-8" RCC-NA** 7/2/2012		**Trl. "A-11" RCC-NA** 7/2/2012	
Material Proportions per CY	Specific Gravity	Weight lbs. (1) (2)	Volume cf.	Weight lbs. (1) (2)	Volume cf.	Weight lbs. (1) (2)	Volume cf.
: : Type I-II Low-Alkali Cement	3.15	191	0.97	309	1.57	422	2.15
Water	1.00	205	3.29	208	3.33	210	3.37
	H_2O in Gallons	*24.61 gallons*		*24.97 gallons*		*25.21 gallons*	
CEMEX Plant No.1451, Concrete Sand (SSD)	2.627	1702	10.38	1653	10.08	1602	9.77
CEMEX Plant No. 1451 , Sz. 57_1.00" Coarse Agg (SSD)	2.638	1999	12.14	1941	11.79	1880	11.42
Gravimetric Air Content, % (5)	-	0.8	0.22	0.8	0.22	1.1	0.30
			27.0		27.0		27.0
Plastic Properties							
Slump, in. (4)		N/A		N/A		N/A	
Ambient Temperature, deg. F (4)							
Concrete Temperature, deg. F (4)							
Cementitious Content, pcy		191		309		422	
Water/Cementitious Ratio (3)		1.073		0.673		0.498	
Fine-to-Total Aggregate, %(wt.) (3)		46.0		46.0		46.0	
Target Air Content, %		2.0		2.0		2.0	
Gravimetric Air Content, % (5)		0.80		0.81		1.12	
Measured Unit Weight, pcf (4)		151.76		152.26		152.35	
Theoretical **"WET"** Unit Weight, pcf (2) (3)		151.7		152.3		152.3	
Theoretical Maximum Unit Weight, pcf (3)		153.0		153.5		154.0	
Theoretical **"Fresh Dry"** Unit Weight (if Batched Proportions contained 6.6% moisture), pcf (2) (3)		**142.3**		**142.8**		**142.9**	
	Lab Specimen #	**212646**		**212647**		**212648**	
Compressive Strength, psi	**ASTM C 192, C 1435, & C 39**	psi		psi		psi	
@ 7 days (6)(9)	Normal Cure	830		2530		4540	
@ 28 days (6)(9)	Normal Cure	1160		3330		5320	
@ 56 days (7)(9)	Normal Cure	1240		3690		5580	
	7-day to 28-day ratio:	*1.398*		*1.316*		*1.172*	

Notes:
(1) units in lbs. unless stated otherwise
(2) proportions based on Gravimetric Air Content
(3) as calculated
(4) as measured
(5) as CALCULATED from Unit Weight
(6) average of two (2) specimens
(7) one (1) specimen
(8) 4" X 8" Cylinders
(9) 6" X 12" Cylinders

Concrete Laboratory Trial Batch
Compressive Strength Data Summary

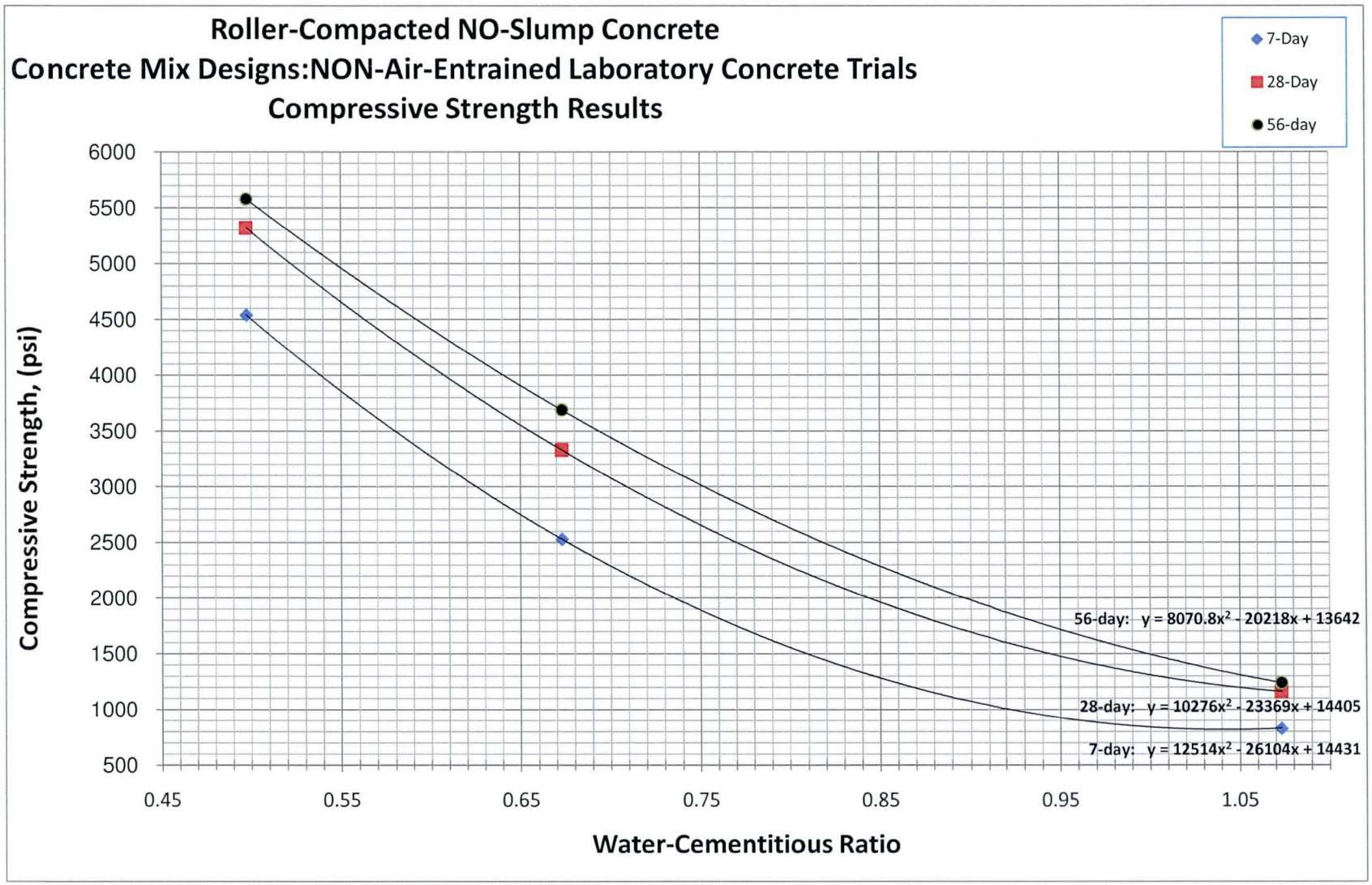
Report Summary

Project: Oak Street Detention Basin and Storn
Client: AMEC- Phoenix Engineering
Job No: 17-2011-4050.003
Lab No:

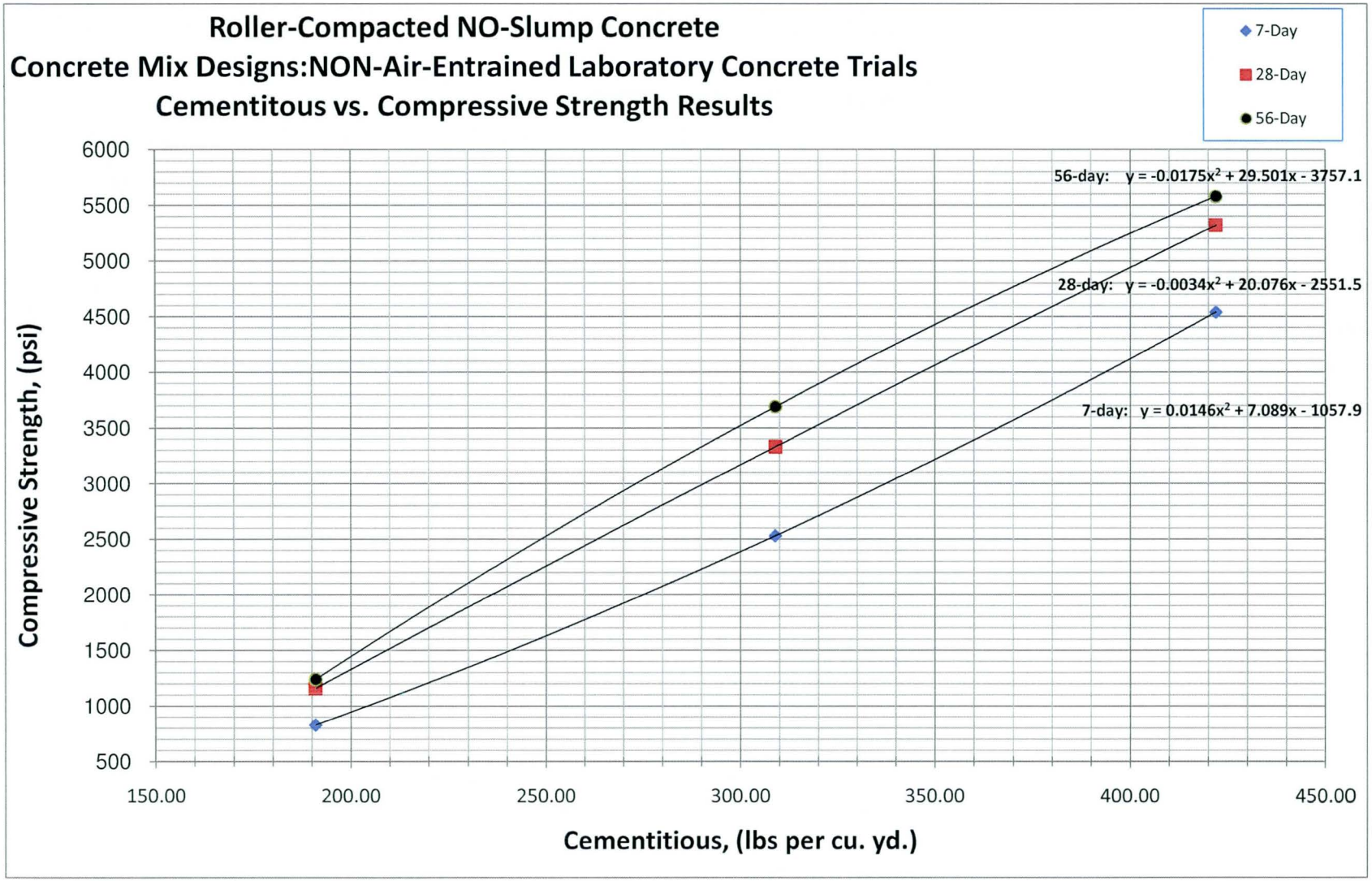
Date: 7/5/2012
Report Date: 7/9/2012
Amended: 9/10/2012
By: Richard L. Jenkins & Ken Wylie

Trial Mix ID:	**Trl. "A-5"**	**Trl. "A-8"**	**Trl. "A-11"**
Title:	**RCC-NA**	**RCC-NA**	**RCC-NA**
Batch Date:	7/2/2012	7/2/2012	7/2/2012
Water-Cementitious Ratio	**1.073**	**0.673**	**0.498**
7-Day Compressive Strength Data			
Lab Specimen #	**212646**	**212647**	**212648**
Test Date	7/9/2012	7/9/2012	7/9/2012
Specimen 1, psi	790	2520	4510
Specimen 2, psi	870	2540	4560
Specimen 3, psi			
Average, psi	**830**	**2530**	**4535**
28-Day Compressive Strength Data			
Test Date	7/30/2012	7/30/2012	7/30/2012
Specimen 1, psi	1160	3340	5210
Specimen 2, psi	1160	3320	5420
Specimen 3, psi			
Average, psi	**1160**	**3330**	**5315**
56 Day Compressive Strength Data			
Test Date	8/27/2012	8/27/2012	8/27/2012
Specimen 1, psi	1240	3690	5580
Specimen 2, psi			
Specimen 3, psi			
Average, psi	**1240**	**3690**	**5580**

Water-Cementitious vs. Compressive Strength Curve



Cementitious vs. Compressive Strength Curve



REPORT of AGGREGATE PHYSICAL PROPERTIES

Project: Oak Street Detention Basin & Storm Drain Project
Project No: 17-2011-4050.003
Client: Phoenix-AMEC Engineering
Lab No.:
Material Tested: Sz. 57
Sample Date: 6/19/2012
Source: CEMEX: Plant No. 1451
Report Date: **Amended:**
Product Code:
Specifications:
Reviewed by:

SIEVE ANALYSIS

ASTM C136	AASHTO T27	
Sieve Size	**Accum. % Passing**	**Specifications**
300 mm (12")		
250 mm (10")		
225 mm (9")		ASTM C-33
200 mm (8")		Sz. 57
150 mm (6")		Coarse Aggregate
125 mm (5")		Grading Limit
100 mm (4")		
75.0 mm (3")		
63.0 mm (2-1/2")		
50.0 mm (2")		
37.5 mm (1-1/2")	100	100
31.5 mm (1-1/4")		
25.0 mm (1")	96	95 - 100
19.0 mm (3/4")	63	
12.5 mm (1/2")	27	25 - 60
9.5 mm (3/8")	13	
6.3 mm (1/4")	4	
4.75mm (No. 4)	2	0 - 10
2.36 mm (No. 8)	1	0 - 5
2.00 mm (No. 10)	1	
1.18mm (No.16)	1	
0.600 mm (No.30)	1	
0.425mm (No. 40)	1	
0.300mm (No. 50)	1	
0.180mm (No. 80)		
0.150mm (No.100)	1	
0.075mm (No. 200)	0.6	0.0 - 2.0
ASTM C117 / AASHTO T11		
Soil Classification		
Moisture Content, %		
ASTM C566 AASHTO T255		
Fract. 1 Face, %		
Fract. 2 Face, %		
Fineness Modulus		
ASTM C136 AASHTO T27		

TEST RESULTS

Standard	PHYSICAL PROPERTIES		Results	Specification
ASTM C 29	Unit Weight	Unit Weight, lbs./cu.ft=		
AASHTO T 19	& Voids	Voids, %=		
		☐ Jigged ☐ Loose ☐ Rodded		
	Coarse	Bulk Specific Gravity (dry)=	2.609	
ASTM C 127	Specific	Bulk Specific Gravity, SSD=	2.638	
AASHTO T 85	Gravity &	Apparent Specific Gravity=	2.686	
	Absorption	Absorption, %=	1.09	
	Fine	Bulk Specific Gravity (dry)=		
ASTM C 128	Specific	Bulk Specific Gravity, SSD=		
AASHTO T 84	Gravity &	Apparent Specific Gravity=		
	Absorption	Absorption, %=		
ASTM D 2419	Sand	Sand Equivalent, %=		
AASHTO T 176	Equivalent			
ASTM D 4791	Flat &	Flat & Elongated, %=		
	Elongated	Ratio=		
ASTM C 131	L. A.	Small Coarse Loss, %=		
AASHTO T 96	Abrasion	Grading/ Revs.=		
ASTM C 535	L. A.	Large Coarse Loss, %=		
	Abrasion	Grading/ Revs.=		
	Soundness	Coarse Soundness Loss, %=		
ASTM C 88		Magnesium No. of Cycles=		
AASHTO T 104	Soundness	Fine Soundness Loss, %=		
		Magnesium No. of Cycles=		
ASTM C 142	Clay/Friable	Coarse Aggregate, %=		
AASHTO T 112	Particles	Fine Aggregate, %=		
ASTM 2974	Organic	Organic Matter % =		
AASHTO T267	Matter	Ash Content % =:		
ASTM C 123	Lightweight	Coarse Aggregate, %=		
AASHTO T 113	Pieces(Sp.Gr.2.0)	Fine Aggregate, %=		
ASTM C 1252	Fine Aggregate	Uncompacted Voids,%=		
AASHTO T 304	Angularity	Method C (as received material)		
	Compaction	Optimum Moisture, %=		
		Max. Density, lbs./cu.ft.=		
ASTM D 4318	Liquid Limit,	Liquid Limit=		
AASHTO T 89/90	Plastic Limit &	Plastic Limit=		
	Plasticity Index	Plasticity Index=		

NOTE: Data provided by AMEC's Phoenix office.

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Project: Oak Street Detention Basin & Storm Drain Project
Project No: 17-2011-4050.003
Client: Phoenix-AMEC Engineering
Lab No.:
Material Tested: Concrete Sand
Sample Date: 6/19/2012
Source: CEMEX: Plant No. 1451
Report Date: **Amended:**
Product Code:
Specifications:
Reviewed by:

SIEVE ANALYSIS

ASTM C136 AASHTO T27

Sieve Size	Accum. % Passing	Specifications
300 mm (12")		
250 mm (10")		
225 mm (9")		
200 mm (8")		
150 mm (6")		ASTM C-33
125 mm (5")		
100 mm (4")		Fine Aggregate
75.0 mm (3")		Grading Limit
63.0 mm (2-1/2")		
50.0 mm (2")		
37.5 mm (1-1/2")		
31.5 mm (1-1/4")		
25.0 mm (1")		
19.0 mm (3/4")		
12.5 mm (1/2")		
9.5 mm (3/8")	100	100
6.3 mm (1/4")		
4.75mm (No. 4)	100	95 - 100
2.36 mm (No. 8)	88	80 - 100
2.00 mm (No. 10)	83	
1.18mm (No.16)	67	50 - 85
0.600 mm (No.30)	37	25 - 60
0.425mm (No. 40)	24	
0.300mm (No. 50)	13	5 - 30
0.180mm (No. 80)		
0.150mm (No.100)	4	0 - 10
0.075mm (No. 200)	2.0	0.0 - 3.0
ASTM C117 / AASHTO T11		
Soil Classification		
Moisture Content, %		
ASTM C566 AASHTO T255		
Fractured Face, %		
2 faces		
Fineness Modulus (FM)	2.92	
ASTM C136 AASHTO T27		

TEST RESULTS

Standard	PHYSICAL PROPERTIES			Results	Specification
ASTM C 29	Unit Weight	Unit Weight, lbs./cu.ft=			
AASHTO T 19	& Voids	Voids, %=			
		☐ Jigged ☐ Loose ☐ Rodded			
	Coarse	Bulk Specific Gravity (dry)=			
ASTM C 127	Specific	Bulk Specific Gravity, SSD=			
AASHTO T 85	Gravity &	Apparent Specific Gravity=			
	Absorption	Absorption, %=			
	Fine	Bulk Specific Gravity (dry)=		2.580	
ASTM C 128	Specific	Bulk Specific Gravity, SSD=		2.627	
AASHTO T 84	Gravity &	Apparent Specific Gravity=		2.708	
	Absorption	Absorption, %=		1.83	
ASTM D 2419	Sand	Sand Equivalent, %=			
AASHTO T 176	Equivalent				
ASTM D 4791	Flat &	Flat & Elongated, %=			
	Elongated	Ratio=			
ASTM C 131	L. A.	Small Coarse Loss, %=			
AASHTO T 96	Abrasion	Grading/ Revs.=			
ASTM C 535	L. A.	Large Coarse Loss, %=			
	Abrasion	Grading/ Revs.=			
	Soundness	Coarse Soundness Loss, %=			
ASTM C 88		Magnesium No. of Cycles=			
AASHTO T 104	Soundness	Fine Soundness Loss, %=			
		Magnesium No. of Cycles=			
ASTM C 142	Clay/Friable	Coarse Aggregate, %=			
AASHTO T 112	Particles	Fine Aggregate, %=			
ASTM C 123	Lightweight	Coarse Aggregate, %=			
AASHTO T 113	Pieces(Sp.Gr. 2.0)	Fine Aggregate, %=			
ASTM C 40	Organic	Lighter Than Standard	LTS		LTS or ETS
AASHTO T-21	Impurites	Equal to Standard	ETS		
		Darker Than Standard	DTS		
	Compaction	Optimum Moisture, %=			
		Max. Density, lbs./cu.ft.=			
ASTM D 4318	Liquid Limit,	Liquid Limit=			
AASHTO T 89/90	Plastic Limit &	Plastic Limit=			
	Plasticity Index	Plasticity Index=			

note: Data provided by AMEC's Phoenix office.

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APPENDIX D

CONSTRUCTION SPECIFICATIONS

SECTION 208 – RCC DAM FOUNDATION EXCAVATION & PREPARATION

SUBSECTION 208.1 DESCRIPTION:

Foundation excavation and preparation shall consist of the excavation and preparation of cemented soil and rock within the foundation limits for the construction of the RCC dam and all outlet conduits that penetrate the RCC dam foundation as designated on the Plans or in these Special Provisions.

Foundation excavation and preparation shall include the furnishing of all materials and equipment and the providing of other facilities which may be necessary to perform the excavations and foundation preparation. Foundation construction activities will consist of the following general activities:

- Excavation to the lines, grades and elevations as shown on the Plans;
- Foundation over-excavation, if required; and
- Foundation surface preparation.

The Contractor's approach will require methodical planning, sequencing, and cooperation with the Engineer in completing the required foundation excavation and preparation. A pre-activity meeting shall be held with the Engineer prior to mobilization where the contractor will outline all processes, equipment, and the proposed construction approach for all activities outlined in this section of the Special Provisions.

Guide Specification

SUBSECTION 208.2 METHODS:

SUBSECTION 208.2.1 FOUNDATION EXCAVATION:

The Contractor shall excavate the RCC dam foundation area and as shown on the Plans. Typically, cemented soils or rock will be exposed during excavations for the RCC dam. The RCC dam is to be founded on cemented soils, rock, or a foundation approved by the Engineer. Non-cemented soils, unacceptable rock or soils not approved by the Engineer are to be over excavated and removed. The limits of the proposed foundation work are approximately as indicated on the Plans. The intent of the indicated foundation excavation depth is to achieve acceptable bearing cemented soil or rock and to remove unacceptable soil or rock materials. Unacceptable materials are generally defined as soil or rock that are uncemented, erodible, weak, unstable, compressible, loose, pervious, highly weathered, or highly fractured.
The required foundation excavation will be approximately four (4) to thirteen (13) feet deep, typically, except for localized deeper zones that will be identified by the Engineer. The Engineer reserves the right to increase the excavated depth or width of the foundation if unacceptable conditions are encountered. Shallower excavation depths may also be required depending on the specific types of materials encountered. All excavation shall be open cut unless otherwise shown on the Project Plans or approved by the Engineer.

Where cemented foundation soils are present it will be necessary to carefully excavate to the surface of the cemented soils but not significantly into these soils. Where pockets or zones of less cemented soils are encountered, or where deeper zones of non-cemented or less cemented soils are encountered, outside the excavation limits shown on the Plans, it may be necessary to remove these soils to a depth to be determined during construction by the Engineer. The intent of foundation excavation is to create a relatively smooth and evenly sloped surface on which to place the RCC dam materials. All grade transitions within cemented foundation soils shall be no steeper than 1 horizontal to 1 vertical (1:1) and all transitions within rock shall be no steeper than 0.5 horizontal to 1 vertical (0.5:1).

The excavated RCC dam foundation surface shall be visually inspected by the Engineer as specified in Subsection 208.3.2. To ensure that unacceptable materials have been thoroughly removed. If additional localized deeper zones of unacceptable materials are encountered, they will be quantified by the Contractor and removed in accordance with Subsection 208.2.2 of this Special Provisions. It will be important for the Engineer to closely observe this construction activity, and for the Contractor to cooperate with the Engineer in completing the required foundation excavation and preparation.

Hand tools, where required or permitted by these Special Provisions include, but are not limited to air, water, air/water jets, shovels, bars, picks, wedges, brooms, and hand brushes. Light power tools and power brooms may be used in lieu of

hand tools only when the Engineer approves such use. An air jet should consist of a nozzle with a supply hose connected to a suitable source of compressed air. The compressed air should have a pressure between 90 and 110 psi. The compressed air shall be controllable at the nozzle. An air/water jet shall consist of a nozzle with associated controls and supply hoses connected to suitable sources of compressed air and water. Water should be introduced into the air stream at the nozzle when needed, at a rate of up to 30 gpm. The air and water shall be separately controllable at the nozzle. A water jet shall consist of a nozzle with a supply hose connected to a suitable source of water. The system should be capable of delivering up to 200 gpm. The flow rate shall be controllable at the nozzle.

After Foundation Excavation acceptance by the Engineer, at the direction of the Engineer, the Contractor shall proceed with Foundation Over-Excavation or the Foundation Surface Preparation as specified in Subsections 208.2.2 and 208.2.3 of this Special Provision after Foundation Excavation.

SUBSECTION 208.2.2 FOUNDATION OVER-EXCAVATION:

The Contractor shall be responsible for foundation over-excavation of localized deeper zones of unacceptable materials as directed by the Engineer to the limits of the proposed foundation work which are approximately indicated on the Plans. When over-excavation is required that exceeds the approximate limits on the Plans, the foundation limits shall become coincident with the projection of the upstream and downstream RCC structure slopes to the point of connection with the foundation at the newly over-excavated foundation elevation. The foundation excavation will be visually inspected by the Engineer to ensure localized deeper zones of unacceptable soil materials have been thoroughly removed within the foundation area. Unacceptable materials may include, but not be limited to, loose soils, pervious zones, previously buried granular materials forming washes, areas of intensely to very intensely fractured or weathered rock, and other similar soils as identified by the Engineer. After Foundation Over-Excavation acceptance by the Engineer, at the direction of the Engineer, the Contractor shall proceed with the Foundation Surface Preparation as specified in Subsection 208.2.3 of this Special Provision after Foundation Over-Excavation.

SUBSECTION 208.2.3 FOUNDATION SURFACE PREPARATION:

When the excavation has reached the approximate limits shown or when the Engineer determines that a satisfactory foundation may have been reached, the Engineer may direct that a surface preparation be performed on all or any part of the foundation surface. This preparation shall consist of removing all debris, loose rock, sand, silt, and other objectionable material by hand tools followed by air, water, air/water jets or any combination of additional methods approved or directed. Prepared foundation surfaces shall be cleared of all unconsolidated or highly weathered or fractured portions of the foundation material as directed by the Engineer. Only the minimum amount of cleaning required to effectively prepare the foundation surfaces is allowed. The Engineer may require that the excavation be continued and the surface preparation procedure repeated until a satisfactory foundation surface is reached.

All areas of the RCC dam foundation slated to receive RCC materials shall be prepared in accordance with this subsection. The Contractor shall prepare the RCC dam foundation surface areas at the direction of the Engineer depending on the foundation material type encountered and in accordance with Subsections 208.2.3.1 and 208.2.3.2 prior to RCC material placement activities. The Contractor shall coordinate the foundation surface preparation activities with other construction activities to prevent damaging previously approved foundations and all other constructed components of the overall dam. Foundations surfaces shall be maintained in a clean and undisturbed condition until the placement of RCC. Costs associated with additional foundation surface preparation, repair or reconstruction of foundation previously approved by the Engineer or dam components already constructed shall be borne by the Contractor at no additional cost to the Owner.

Subsection 208.2.3.1 Cemented Soil Surfaces

All excavated cemented soil surfaces shall be prepared by removing all loose material, pockets of fines, sand, gravel and other unacceptable material by hand tools followed by air, water, air/water jets or any combination of additional methods as approved or directed by the Engineer from the foundation surface including areas of depressions and open cracks. The loose material need not be removed where the width of the crack is less than ½ inches. Picking, barring, and hand excavation may be necessary to obtain a foundation surface free from loose or shattered materials. This surface preparation shall consist of filling or excavating all areas of the foundation to achieve a uniform foundation surface where all grade transitions within cemented soil shall be no steeper than 1 horizontal to 1 vertical (1:1). Filling of larger areas may be accomplished with RCC as long it meets all requirements of Special Provision 222 and the surface to receive RCC has been accepted by the Engineer. Isolated areas of foundation may be filled with dental concrete having a 28-day

Guide Specification

compressive strength of 2,500 psi and meeting all requirements of Section 725 and the surface to receive RCC has been accepted by the Engineer.

Subsection 208.2.3.2 Rock Surfaces

All excavated rock surfaces shall be prepared by removing all debris, loose rock, sand, silt, and other unacceptable material by hand tools followed by air, water, air/water jets or any combination of additional methods as approved or directed by the Engineer. This surface preparation shall consist of filling or excavating all areas of the foundation to achieve a uniform foundation surface where all grade transitions within rock shall be no steeper than 0.5 horizontal to 1 vertical (0.5:1). Filling of larger areas may be accomplished with RCC as long it meets all requirements of Special Provision 222 and the surface to receive RCC has been accepted by the Engineer. Isolated areas of foundation may be filled with dental concrete having a 28-day compressive strength of 2,500 psi and meeting all requirements of Section 725 and the surface to receive the dental concrete has been accepted by the Engineer.

SUBSECTION 208.3 FOUNDATION INSPECTION:

After the Contractor has completed the Foundation Surface Preparation in accordance with Subsection 208.2.4, the Contractor shall notify the Engineer who will make an inspection of the foundation excavation and surface. Inspection and acceptance shall be given by the Engineer prior to placing the dam RCC materials. The Contractor shall perform any additional foundation excavation, over-excavation, or surface preparation as directed by the Engineer. Upon inspection the Engineer may require Over-excavation in Subsection 208.2.2 and Foundation Surface Preparation in Subsection 208.2.3, until acceptance of the foundation surface is achieved.

Foundation approved for RCC placement by the Engineer shall be covered with RCC as soon as possible. No additional compensation will be provided to the Contractor for Foundation Surface Preparation should the foundation surface become compromised by rain, surface runoff, flooding, wind, excessive drying or any other occurrence. The Contractor shall be responsible for all clean-up and surface preparation necessary to achieve the foundation preparation requirements as outlined in these Special Provisions. Costs associated with additional foundation clean-up and preparation, repair or reconstruction of foundation previously approved by the Engineer shall be borne by the Contractor at no additional cost to the Owner.

After acceptance of the Foundation Surface Preparation by the Engineer, but prior to RCC placement, the Contractor's surveyor shall collect as-built data to establish the elevation of all accepted foundation surfaces. Any over-excavated areas shall also be surveyed to quantify the volume of over-excavation for pay purposes and for documentation on the red-line as-built drawings.

Guide Specification

SUBSECTION 208.4 TESTS:

Construction quality control testing shall be conducted by the Contractor on all RCC and dental concrete used to prepare the foundation surface. Testing shall be completed in accordance with the test methods and minimum testing frequencies stated within Section 725 and Section 222 of these Special Provisions.

SUBSECTION 208.5 MEASUREMENT:

No separate measurement shall be made for Foundation Excavation. The quantity of material in cubic yards to be excavated from natural ground to the surface of the RCC Dam foundation shall be included in the measurement of BASIN EXCAVATION.

In the event that the Engineer determines that Foundation Over-Excavation is required, the following shall apply:

Measurement for payment of Foundation Over-Excavation as directed by the Engineer shall be per in-place cubic yard.

Measurement for payment of Foundation Surface Preparation shall be per square yard in plan view.

Measurement for payment of RCC material placed during Foundation Surface Preparation shall be measured in accordance with Special Provision Section 222.

Measurement for payment of dental concrete placed during Foundation Surface Preparation shall be measured in accordance with Special Provision Section 222.

SUBSECTION 208.6 PAYMENT:

No separate payment shall be made for Foundation Excavation. The cost thereof shall be included in the price bid for BASIN EXCAVATION.

Payment for any additional Foundation Over-Excavation as directed by the Engineer will be made on the basis of the unit price bid per in-place cubic yard. The cost shall include all labor, materials, testing, survey as required, dust control and equipment necessary to remove the existing unsuitable material from the foundations as directed by the Engineer. This allowance item also includes the cost of the survey documentation required for all over-excavated areas. Use of this allowance shall be approved by the Engineer.

BID ITEM 208-1 FOUNDATION OVER-EXCAVATION ALLOWANCE

Payment for all Foundation Surface Preparation shall be made on the basis of the unit price bid per square yard in plan view. The cost shall include all labor, materials, testing, survey as required, dust control, and equipment necessary to prepare all foundation surface areas as the directed by the Engineer.

BID ITEM 208-2 FOUNDATION SURFACE PREPARATION

Payment for additional Foundation Surface Preparation shall be made on the basis of the unit price bid per square yard in plan view. This allowance item shall include all labor, materials, testing, survey as required, dust control, and equipment necessary to prepare all foundation surface areas as the directed by the Engineer. Use of this allowance shall be approved by the Engineer and only allowed when additional Foundation Over-excavation has been directed by the Engineer.

BID ITEM 208-3 FOUNDATION SURFACE PREPARATION ALLOWANCE

Payment for the RCC material placed during Foundation Surface Preparation will be made on the basis of the unit price bid per cubic yard in-place, and shall include full compensation for furnishing all labor, materials, tools and equipment, placing of RCC, testing, dust control, equipment necessary to prepare all foundation surface areas as directed by the Engineer. Use of this allowance shall be approved by the Engineer.

BID ITEM 208-4 FOUNDATION SURFACE PREPARATION WITH RCC ALLOWANCE

Payment for the dental concrete material placed during Foundation Surface Preparation will be made on the basis of the unit price bid per cubic yard in-place, and shall include full compensation for furnishing all labor, materials, tools and equipment, placing of dental concrete, testing, dust control, equipment necessary to prepare all foundation surface areas as directed by the Engineer. Use of this allowance shall be approved by the Engineer.

BID ITEM 208-5 FOUNDATION SURFACE PREPARATION WITH DENTAL CONCRETE ALLOWANCE

No payment shall be made for incidental excavation and surface preparation beyond the specified excavation payment limits or otherwise in addition to the excavation quantified for payment. The cost thereof shall be included in the price bid for construction of the items to which such excavation is incidental or appurtenant.

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SECTION 222 – ROLLER COMPACTED CONCRETE

SUBSECTION 222.1 DESCRIPTION:

The work shall consist of furnishing all materials, tools, equipment, and mixing plant; and performing all labor for the mixing, transporting, forming, placing, compacting, and curing of roller compacted concrete (RCC) as required to install the structure(s) as shown on the drawings and as described within these Special Provisions.

SUBSECTION 222.2 MATERIALS:

SUBSECTION 222.2.1 Portland Cement:

Portland Cement shall comply with the latest Specifications as approved by the Engineer for Portland Cement (ASTM C 150, Type II [low alkali]), and shall conform to the requirements of Section 725.

SUBSECTION 222.2.2 Aggregate:

RCC aggregate shall be obtained from a commercial aggregate supplier. Aggregate for RCC shall contain no deleterious material and shall meet the requirements of ASTM C33 for concrete aggregates. Fine and coarse aggregates to be used in RCC shall be evaluated and tested for alkali-silica reactivity. The fine and coarse aggregates shall be evaluated separately in accordance with ASTM C 1260 utilizing the Contractor's proposed low alkali portland cement. Test results shall have a measured expansion equal to or less than 0.10 percent after 14-days of immersion in a (1N) NaOH solution. Should the test data indicate an expansion of greater than 0.10 percent, the aggregate(s) shall be rejected or additional testing shall be performed utilizing the Contractor's proposed low alkali portland cement and proposed Class F fly ash in accordance with ASTM C 1567. Determine the quantity that will meet the requirements of these specifications and that will lower the expansion equal to or less than 0.10 percent after 14-days of immersion in a (1N) NaOH solution. Evaluate the fine and coarse aggregates separately in accordance with ASTM C 1567. Should the test data indicate that the expansion still exceeds the 0.10 percent after 14-days of immersion in a (1N) NaOH solution, the aggregate(s) shall be rejected.

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The combined aggregate shall be well-graded without gradation gaps and conform to the gradation in Table 222-1:

Table 222-1 Aggregate Gradation	
Sieve Size	**Percent passing by weight**
1-1/2" (37.5 mm)	100
1" (25 mm)	85-100
1/2" (12.5 mm)	60-80
No. 4 (4.75 mm)	40-55
No. 8 (1.18 mm)	28-41
No. 16 (1.18 mm)	21-33
No. 30 (600 µm)	14-26
No. 50 (300 µm)	6-16
No. 100 (150 µm)	2-10
No. 200 (75 µm)	1-6

SUBSECTION 222.2.3 Water:
Water incorporated into the mix or used for curing RCC shall be clean and free from injurious amounts of oil, salt, acid, alkali, organic matter, turbidity, or other deleterious substances. Water shall conform to the requirements of ASTM C 94 except that wash water shall not be used for mixing RCC.

SUBSECTION 222.2.4 Water Reducing Agents:
Water-reducing, set-retarding admixture shall conform to ASTM C 494, type D.

SUBSECTION 222.2.5 Fly Ash:
Fly ash shall be class F unless otherwise specified and shall conform to ASTM C 618, Class F, and shall have a loss on ignition not exceeding 3 percent and shall have a Calcium Oxide (CaO) content of less than 13 percent and a total equivalent alkali content less than 3 percent. The source of pozzolan shall consistently supply material with similar chemical and physical properties.

SUBSECTION 222.2.6 Bonding Mortar:
Bonding mortar shall consist of cement, sand, water, and a water-reducing, set-retarding admixture to retard the set and control the consistency of the mortar. The cement, water, and admixture shall be as specified. Sand shall comply with ASTM C 33 for fine aggregate. The mortar shall be mixed in the proportions 1 part cement to 2.5 parts sand, by weight. Water content shall be sufficient to provide a spreadable consistency. In combination with the admixture, the maximum water-to-cementitious materials ratio shall be 0.45. The mortar slump shall be 7 to 9 inches when tested in accordance with ASTM C 143. The admixture shall be included at the manufacturer's recommended dosage so that the initial set time is retarded at least 3 hours when the ambient air temperature is 95 degrees Fahrenheit.

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SUBSECTION 222.3 MATERIAL TESTING & SAMPLING:

SUBSECTION 222.3.1 Material Testing:

The contractor shall test materials or provide certified test results to ensure all materials conform to the specified requirements. All nonconforming materials shall be promptly removed from the job site, including those that have been incorporated into the work.

SUBSECTION 222.3.2 Material Sampling:

The contractor shall provide suitable platforms, tools, equipment, and labor necessary for obtaining representative samples of materials to be used for the contractor's quality control testing and for the Government's quality assurance testing. Samples may be taken from stockpiles, aggregate bins and feed belts, entrance to the mixer, mixer discharge, gob hopper discharge, points in transit, or the placement area.

Aggregate sampling shall be in accordance with ASTM D 75.

SUBSECTION 222.4 MATERIAL HANDLING & PROCESSING:

Transportation of cement and pozzolan to the batching plant shall be accomplished in weather-tight trucks, conveyors, or other means that will completely and thoroughly protect the cementitious materials from exposure to moisture and contaminants.

The temperature of cement and pozzolan when delivered to the job site shall not exceed 160 degrees Fahrenheit.

Aggregates shall be transported to the plant site in two or more components that will be combined in the mix plant to meet the overall aggregate gradation. Aggregate components shall be of a gradation that will minimize segregation prior to introduction into the mixer. Aggregates shall be stored in stockpile(s) in the designated contractor use area. Aggregate components shall be stockpiled separately.

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SUBSECTION 222.5 SUBMITTALS:

Manufacturer's certifications and test reports shall in no way relieve the contractor of the responsibility for furnishing materials that meet the specified requirements. Manufacturer's certifications and test reports shall be produced and dated within the 6 months preceding the delivery of the submittal.

SUBSECTION 222.5.1 RCC Pre-production Submittals:

The submittals listed below shall be provided, in writing for approval, to the engineer no less than 30 days prior to planned test section construction.

Job mix

a. A certified statement of materials, mix proportions (reported for saturated surface dry aggregate), theoretical air-free density (TAFD), moisture/density curves (wet density only), unit weight of mix as determined from the weight of compressive strength cylinders, and all compressive strength test results for each of the three mixes required in the development of the RCC job mix.

b. Gradation of each of the aggregate component and combined aggregates used in each mix developed in the mix design program.
c. The compacted bulk density and voids in each of the aggregate components and combined aggregates used in each mix developed in the mix design program.
d. A statement of materials and mix proportions used in each mix developed in the mix design program.
e. A statement of materials and mix proportions proposed to be used in manufacturing the RCC job mix.

Bonding mortar

a. A statement of materials and mix proportions to be used in manufacturing the bonding mortar.

Test section plan

a. The contractor's proposed location for the test section and equipment, materials, personnel, and methods to construct the test section as specified. The submittal shall include plans for the pre-and post-test section briefings.

SUBSECTION 222.5.2 RCC Test Section Submittals:
Within 24-hours of completing the tests or test section, the following information shall be transmitted to the engineer in writing.

a. Results of moisture and density tests used to compute the apparent maximum density (AMD). Include the results of all density tests made of RCC in the test section.
b. Lift maps of the test section.
c. Results of compressive strength tests.
d. Unit weight of RCC.
e. Production plan that includes RCC production methods, materials, plant, equipment, and personnel as modified based on the results from the performance of the test section that have been documented to produce RCC that meets the requirements of the specification.

Unless otherwise specified, 7 days prior to beginning RCC production, submit a final written plan for RCC production methods, materials, plant, equipment, and personnel that will produce RCC that meets the requirements of this specification.

SUBSECTION 222.5.3 RCC Production Submittals:
The following submittals shall be provided in writing within 24 hours after delivery tickets, records, or test results are produced, unless otherwise specified.

a. Delivery tickets for cement and pozzolan shall include the source, date manufactured or produced, type or class, contractor's name, project name, and a certification that the material meets the specification requirements.
b. Delivery tickets for aggregates shall include the source, material description, date, and certification that the material meets the specification requirements.
c. Delivery tickets for bonding mortar shall include name and location of batch plant, ticket number, load and truck number, date, destination, class of cementitious materials, mix

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proportions, quantity of bonding mortar, time mixer drum charged with cement, and recording of revolution counter (transit-mixed concrete). If bonding mortar is produced on site, the above required information shall be provided as applicable.

d. Records of climatic conditions shall be collected on a daily basis and reported on a weekly basis.
e. Mix plant production records and summary of daily material use and RCC produced shall be submitted before the start of the next production shift. Production records shall include a comparison of actual materials used to the approved job mix.
f. Results of RCC moisture and wet density tests.
g. Results of uniformity tests.
h. Results of compressive strength tests.
i. Results of RCC temperature tests.
j. Lift maps shall be submitted before the start of the next production shift.

SUBSECTION 222.6 RCC MIX DESIGN:

The contractor shall be responsible for the mix design and selection of all materials to be used in the design mix. The materials and proportions so stated, when approved, shall constitute the job mix. The job mix(s) shall be prepared to meet the quality, consistency, and strength of the RCC specified.

The contractor shall conduct the mix design program at a materials testing laboratory staffed by American Concrete Institute (ACI) Certified Grade II Concrete Laboratory Testing Technicians.

SUBSECTION 222.6.1 Trial Mix Design Parameters:

The aggregate gradation shall be as specified. The bulk density and voids in each of the aggregate components and the combined aggregate shall be determined according to ASTM C 29.

The density, relative density, and absorption shall be determined according to ASTM C 127 for coarse aggregate and ASTM C 128 for fine aggregate.

The density of the RCC shall be determined on compressive strength cylinders in accordance with ASTM C39.

The minimum compressive strength for the RCC job mix shall be 2,500 pounds per square inch (psi) at 28 days.

Unless otherwise approved by the Engineer:

- A minimum of three separate mixes shall be developed in the laboratory.
- Pozzolan(s) shall comprise at least 20 percent (by volume), but shall not exceed 50 percent (by volume) of the cementitious materials.
- The remainder of the cementitious materials shall be comprised of Portland cement.

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- A compaction curve (wet density only) shall be developed for each mix to determine the water content that corresponds to the maximum wet density of each mix. Compaction tests shall be performed in accordance with ASTM D 1557, adapted as follows:
  - The mold specified for Method C shall be used.
  - All mix components shall be included.
  - When the maximum size aggregate in the mix is larger than 3/4 inch, place the material in three layers into the mold and compact each layer with 94 blows of the hammer.

SUBSECTION 222.6.2 Theoretical Air Free Density:
The theoretical air free density (TAFD) shall be computed for each of the three laboratory mixes in the mix design program. The TAFD is the maximum wet density that can be attained for a specific mix assuming there is no air (entrapped or entrained) in the mix. The TAFD shall be computed by dividing the sum of the individual weights of the mix components by the sum of the individual absolute volumes of the mix components. The absolute volume is the volume of the solid matter in the particles, exclusive of the volume of voids between the particles as determined per ACI 211. The saturated surface dry weight and density of the aggregate shall be used when computing the TAFD.

SUBSECTION 222.6.3 Laboratory Compressive Strength:
A minimum of nine compressive strength cylinders from each RCC mixture shall be prepared in accordance with ASTM C 1176 or C 1435 and weighed to determine the density of the RCC within each cylinder. Any cylinder that weighs less than 98 percent of the weight of the heaviest cylinder shall be discarded and another cylinder prepared and weighed until all 9 cylinders have a weight that is at least 98 percent of that of the heaviest cylinder. The water content of each mixture, from which cylinders are made, shall be within 0.5 percent of the water content that corresponds to the maximum wet density determined in accordance with ASTM D 1557. Three cylinders from each RCC mixture shall be tested at 7, 28, and 56 days for compressive strength in accordance with ASTM C 39.

The average of the two closest 28-day strength values shall represent the 28-day compressive strength of the mix. The 28-day compressive strength of at least one of the mix designs shall be 75 percent to 100 percent of the specified strength. The 28-day compressive strength of at least one of the mix designs shall be 100 percent to 125 percent of the specified strength. The 28-day compressive strength of the remaining mix design shall approximate the specified strength.

The 28-day compressive strengths of each of the three laboratory mix designs will be plotted to form a curve showing the relationship of the cementitious materials content to the 28-day compressive strength of the laboratory mix designs. A cementitious material content shall be selected from this curve corresponding to the 28-day compressive strength specified in Subsection 222.7.1. The proposed job mix shall be proportioned to contain the selected cementitious materials content and shall be submitted for approval.

After the job mix has been approved, neither the source, character, or grading of the aggregates; nor the source mill, type, brand, or quantity of the cement; nor the source, type, nor the type, or quantity of the poizzolan; no the type, brand, or quantity of the chemical admixture(s) used shall

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be changed without approval. Changes to the approved job mix will require submittal and approval of a new job mix that complies with the requirements of this specification.

SUBSECTION 222.7 RCC TEST SECTION:

Prior to RCC production the contractor shall construct a test section as part of the RCC placement operations. The test section shall be installed at an approved location proposed by the contractor. If the test section will be incorporated into the RCC structure, it shall be located in a noncritical part of the structure, and it shall be removed if it fails to meet the requirements of this specification. If the test section will not be incorporated into the structure, the contractor shall remove and dispose of the test section upon completion of the testing requirements unless otherwise specified.

All RCC that is incorporated into the structure and placed prior to determining the AMD shall be compacted to a density that is at least 96 percent of the TAFD. All RCC incorporated into the structure after the AMD is determined shall be compacted to specification requirements.

The test section shall be used to demonstrate all techniques, materials, plant and equipment, and personnel to be used for RCC construction and quality control.
The test section shall be of sufficient size to allow the complete RCC placement and compaction operation to be conducted with the equipment operating at normal operating speeds.

When the test section is placed, a minimum of two 12-inch lifts shall be placed, and tests for determining the AMD shall be conducted on the uppermost lift.

A section shall be constructed to determine the adequacy of the procedures implemented to construct vertical surfaces. Any surface that is not horizontal is, within this specification, considered to be a vertical surface. The tolerance of vertical surfaces shall conform to the requirements of Subsection 222.19. The finish and appearance of formed and unformed vertical surfaces shall comply with the requirements specified in Subsection 222.18.

The contractor and engineer shall conduct a pre-test section briefing to review the field status related to the preparedness, capability, and readiness of the contractor to construct the test section according to the approved plan. After test section construction and before RCC production, the contractor and engineer shall conduct a post-test section briefing to discuss adjustments to the techniques, materials, plant, equipment, and personnel that will be used in RCC production.

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SUBSECTION 222.7.1 Apparent Maximum Density:

AMD is the maximum RCC density of the approved job mix that can be attained by compacting the RCC with the production roller as defined with this Special Provision. The AMD shall be greater than or equal to 98 percent of the TAFD. The AMD of the RCC shall be determined from the test section.

To determine the AMD, the RCC lift shall be compacted by successive passes of the production roller over the entire lift surface. Beginning with the second pass of the production roller and after each pass thereafter, in-place wet density tests (ASTM C 1040) shall be made at a minimum

of two locations and each at a depth of 12 inches. Successive passes of the production roller, followed by density tests at the 12-inch depth, shall be made until the density of the lift no longer increases. Once it has been determined that the density measurement at the 12-inch depth is no longer increasing, density tests shall be taken in six locations at a depth of 10 inches.

If more than one of these density measurements results in values less than 96 percent of the TAFD, the contractor shall modify operations and repeat the process for determining the AMD. The AMD shall be the average of the in-place density test values that are greater than or equal to 96 percent of the TAFD of the job mix. If the AMD is less than 98 percent of the TAFD of the job mix, the contractor shall modify operations to attain an AMD that is at least 98 percent of the TAFD.

SUBSECTION 222.7.2 Compressive Strength Tests:
A minimum of nine compressive strength cylinders shall be prepared from the mix in accordance with ASTM C 1176 or C 1435 and weighed to determine the density of the RCC within each cylinder. Any cylinder that weighs less than 98 percent of the weight of the heaviest cylinder shall be discarded and another cylinder prepared and weighed until all 9 cylinders have a weight that is at least 98 percent of that of the heaviest cylinder. Three cylinders shall be tested at 7, 28, and 56 days, respectively, for compressive strength in accordance with ASTM C 39.

SUBSECTION 222.8 RCC MIX PLANT:
The plant shall either be a batch-type pugmill, a continuous-flow pugmill, or a central mix drum plant. The pugmill shall be a twin shaft paddle-type mixer and shall have adequate capacity to produce a uniform RCC mix at a rate that will conform to the production schedule. The plant shall have a minimum capacity of 100 tons per hour. The central mix drum plant shall conform to the requirements of ASTM C94 and be capable of adequately mixing no-slump concrete

The plant shall have demonstrated satisfactory reliable performance on similar mixes on other RCC projects with little or no down time because of mixer breakdown or other production-related problems, excluding normal maintenance. Satisfactory reliable performance of the proposed plant shall be documented by mixer uniformity tests from recent production work showing that the plant produced a mix of similar proportions that met the requirements for production rate and uniformity set forth in this specification.

The results of uniformity tests that are conducted after the plant is set up and calibrated may be substituted for documentation of reliable past plant performance. RCC produced during uniformity tests required herein shall not be incorporated into the test section or any permanent structure.

The contractor shall perform trial runs of the mixing and proportioning equipment, including uniformity tests if required.

Accuracy. Facilities shall be provided for the accurate measurement and control of each of the materials entering the RCC mix. Delivery of materials as they are discharged from the mixer and from any gob hoppers shall be within the tolerances shown in Table 222–2.

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Table 222-2 Tolerances in Proportioning the Various Ingredients	
Material	**Tolerance**
Pozzolan, mass	± 2%
Cement, mass	± 2%
Aggregate, mass	± 3%
Water, mass or volume	± 2%
Chemical admixture, mass or volume	± 3%

Component monitoring systems. The systems that meter individual mix components shall be interlocked with the plant control and shall warn the operator and shut down the plant if any component is not feeding into the mixing chamber.

Aggregate bins. A separate bin shall be provided for each gradation of aggregate supplied for the RCC job mix. The bins and associated conveyors shall be capable of discharging and conveying the aggregates at a uniform rate without clogging, under all conditions.

Portland cement and pozzolan silos. All onsite storage facilities and connection hoses shall be properly labeled with readily visible signage. The storage silo(s) shall be weather tight to prevent moisture and contaminants from accessing the portland cement and pozzolan. Blended cement/pozzolan products mixed by the cement manufacturer are not permitted. Silos shall be capable of dispensing at a uniform rate without clogging or bridging of the materials.

Portland cement, pozzolan, and aggregate feed. For a continuous-flow pugmill, the portland cement , pozzolan, and aggregates shall be uniformly, continuously, and simultaneously fed into the mixing mechanism at the appropriate ratios. Each bin opening shall be provided with a gate that can be maintained at the necessary opening size to consistently provide the correct feed rate. The bins shall be of sufficient size to assure a uniform flow of aggregate at a constant rate. Portland cement and pozzolan shall be fed continuously by a feed device that is adjustable to ensure a uniform flow of cement pozzolan at a constant rate for proportions established by the approved job mix. Feed device(s) shall be capable of gradual adjustment while in operation. For a central mix drum plant, materials batching shall be in accordance with ASTM C94.

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Water dispenser. A suitable water facility shall be provided that is capable of metering and dispensing the mix water within the specified tolerances. The mechanism for delivering water to the mixers shall be free from leakage. The meter shall measure the weight of water being added in pounds per unit time for continuous-flow pugmills, and weight per batch for batch-type pugmills. The valve shall be capable of gradual adjustment during the mixing process to compensate for varying moisture contents in the aggregates.

Admixture dispenser. The liquid admixture dispensing system shall be capable of metering and dispensing within the specified tolerances. The dispenser shall be designed and installed in such a manner that will permit convenient checking of its accuracy and will assure uniform distribution of the liquid admixture with water to the materials entering the mixer. The system shall be leak-free and designed and installed to prevent backflow or siphoning.

Mixing mechanism. The mixing mechanism shall be capable of combining the materials into a uniform mixture and discharging this mixture without segregation. The mixing mechanism shall produce a mix that meets the uniformity requirements listed in Table 222-3.

Uniformity tests. When a continuous-flow pugmill is used, the three samples for obtaining uniformity tests shall be taken from RCC produced near the beginning, the middle, and the end of a production run lasting a minimum of 1 minute. When a batch-type pugmill is used, the samples shall be taken from RCC produced from three separate batches. Each batch shall be similar in size, be produced by charging the mixer in a similar manner, be mixed at the same mixing speed and mix retention time as the other two batches, and be representative of a normal production run. When a central mix drum plant is used, samples shall be obtained from the first, middle, and last third of the batch as it is discharged from the mixer following the sampling method prescribed in USACE CRD C 55-92.

Table 222-3
Requirements for Uniformity

Test	Allowable max. difference 1/	ASTM Standard
Water content of full mix (% by weight) (Select one of the ASTM standard tests listed)	10%	C 566 D 2216 D 3017 D 4643 D4959
Coarse aggregate content (% by weight)	10%	C 685 (annex)
Density (lb/ft^3) of full mix	2%	C 1170, C 1176, C 1435
Compressive strength at 14 days (lb/in^2)	15%	C 39 2/

1/ The allowable maximum difference = 100 multiplied by the (maximum value – minimum value) divided by the average of three tests.
2/ Cylinders shall be made in accordance with ASTM Standard C 1176 or C 1435

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Mix record. The mix plant shall be capable of continually producing an RCC mix record. For pugmill mixing, the record shall show the weight of portland cement, pozzolan, water, aggregate, and weight or volume of chemical admixture that is processed through the plant during a specific time interval. The time interval shall not exceed 30 minutes. For central drum mix production, the time interval shall be per batch. The aggregate shall be reported in terms of saturated surface dry weight processed through the plant during a specific time interval. The RCC mix record shall be produced at all times when the plant is producing a mix. The aggregate moisture shall be tested daily whenever RCC is being produced. The mix plant record shall include the aggregate moisture content tests upon which the batch proportions are based.

A summary shift record shall be provided. The record shall include the total quantity of each constituent in the mix, total quantity of RCC produced, and a comparison of the quantity of each constituent mixed per cubic yard to that of the job mix.

Noise pollution. The plant shall be operated to comply with all applicable regulations pertaining to noise pollution.

Pollution control. The plant shall be operated to comply with all applicable regulations pertaining to air and water quality.

SUBSECTION 222.9 RCC MIXING:

The plant shall be operated according to the manufacturer's recommendations. The mixing mechanism shall be maintained in satisfactory operating condition and shall be cleaned after each production run. All supply bins and silos shall be kept sufficiently full to ensure a uniform and constant flow of all materials.

All RCC produced from the beginning of startup shall be disposed of offsite until a uniform mix of the required proportions is consistently being discharged from the mixer.

After a batch-type pugmill has been calibrated and has produced the job mix with the specified uniformity, the mix retention time shall not be reduced.

SUBSECTION 222.9.1 Uniformity:

RCC uniformity shall be monitored by continuous visual inspection by the plant operator and by periodic visual inspection by contractor quality control personnel. The mix shall be visually inspected for uniformity by contractor quality control personnel at the beginning of each production run and at least once each hour during the production run. If it becomes apparent that the mixer is not producing a uniform mix of the proportions specified, RCC production shall be promptly discontinued until the problem(s) that caused the uniformity problem are discovered and corrected.

If a uniformity problem is suspected, the contractor shall conduct the first three tests listed in Table 222-3, determine the maximum difference and compare to the allowable maximum difference, and take appropriate corrective measure(s). Adjustments shall be made to the mixing plant as necessary to obtain the required uniformity and consistency of the RCC mix when uniformity test results indicate that the requirements for uniformity are not being met.

SUBSECTION 222.10 RCC CONVEYING:

The RCC mix shall be conveyed from mixer to placement area as rapidly as practicable by methods that prevent segregation, contamination, and loss of water. The total length of time from the end of mixing until the RCC has been placed, spread, and compacted shall not exceed 45 minutes.

The contractor shall provide baffles to limit free fall of mixed RCC to a maximum of 5 feet at the discharge end of conveyors, within hoppers, and at other locations where the potential for segregation may occur. Chutes that tend to cause segregation, such as an inclined chute, will not be permitted.

SUBSECTION 222.10.1 Communications:

Telephone, radio, or other voice communication shall be provided between all interim storage hoppers, the batch plant control, and the placement locations. The contractor shall provide the Government inspector the same form of communication.

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SUBSECTION 222.10.2 Temporary Storage Containers:
Gob hoppers shall be used for storage wherever the mixed RCC is temporarily accumulated prior to being loaded into hauling equipment and when direct conveyor systems do not otherwise provide continuous delivery to the final placement location. Gob hoppers shall be configured to allow free flow of RCC without segregation or choking.

SUBSECTION 222.10.3 Conveyor Belts:
The conveyor system design and layout shall provide for adequate capacity, speed, reach, and pivot points to convey RCC to all placement areas.

Conveyor belts shall be designed, erected, operated, and maintained in a manner that meets production requirements and does not segregate materials. RCC shall not be exposed on any belt for a period exceeding 5 minutes without being protected from the drying elements of wind and sun. RCC shall not be exposed on any belt during rainfall unless it is protected from the rain.

SUBSECTION 222.10.4 Hauling Equipment:
Equipment shall be maintained in good operating condition and shall not be permitted onto the RCC surface when vehicle fluids are leaking or when there is a potential for contamination to the RCC.

RCC may be hauled using trucks, front-end loaders, or scrapers. Hauling equipment that rut, score, mar, or indent the RCC surface shall not be operated directly on previously compacted RCC surfaces. Hauling equipment shall not track mud or other contaminants onto previously placed RCC and shall not be operated directly on uncompacted RCC. All hauling vehicles shall be operated in a manner that prevents tight turns, sudden stops, or other actions that cause damage or displacement to previously compacted RCC. The contractor shall implement necessary measures to prevent contamination or damage of the previously compacted RCC.

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SUBSECTION 222.11 WEATHER:

SUBSECTION 222.11.1 Adverse Weather:
In adverse weather (heavy rain, severe cold, heavy snow, and hot temperature) where the conditions specified herein for RCC construction cannot be maintained, an interruption in placing operations shall be required.

SUBSECTION 222.11.2 Wet Weather Placement:
Wet weather placement—RCC shall be protected from rainfall induced erosion and shall not be placed during rainfall events at a rate equal to or greater than 0.1 inch in 20 minutes. Placement during a light mist may continue when covered belt conveyors rather than hauling vehicles are used to convey the mix. Placement of RCC will not be permitted when rainwater accumulates on compacted RCC surfaces.

SUBSECTION 222.11.3 Cold Weather Placement:
RCC shall not be placed when the ambient air temperature drops below 35 degrees Fahrenheit or the temperature of the RCC mix is less than 40 degrees Fahrenheit.

SUBSECTION 222.11.4 Hot Weather Placement:
The maximum temperature of the RCC at time of placement shall not exceed 90 degrees Fahrenheit. After placing, but prior to compaction, the temperature of the mix shall be determined according to ASTM C 1064. Introduction of chilled water and/or ice, shading and/or cooling of the aggregates, or other measures may be required in the production of RCC to maintain the RCC temperature within the specified requirements.

SUBSECTION 222.12 PLACING & SPREADING:

The placement of RCC into the structure shall be performed in a virtually continuous non-stop operation.

SUBSECTION 222.12.1 Layout of the Placement Area:
The RCC shall be spread in level lifts across the entire area of the structure. The direction of RCC placement shall be parallel to the long axis of the structure so that the number of lanes and the number of edge joints are minimized. Transverse joints of adjacent lanes shall be offset by no less than 20 feet - horizontally.

SUBSECTION 222.12.2 Placing:
RCC mix, bonding mortar, Portland cement, neat cement grout, or concrete shall not be placed on previously placed layers that have not been compacted to the specified density.

RCC mix shall be deposited as near to its final location as possible. When haul vehicle delivery is used, depositing shall be accomplished with a dump-spread action while the placing vehicle is moving. Belt placement shall discharge with a spreading action that does not segregate the material. RCC shall not be deposited directly against formwork or other vertical surfaces. Piles that form when placing RCC shall not contact forms or vertical surfaces. Neither the discharge height nor the pile height of RCC shall exceed 5 feet.

SUBSECTION 222.12.3 Spreading:
Spreading of the RCC shall be completed within 10 minutes following depositing. RCC shall be spread into an uncompacted, uniform lift thickness that can be compacted to produce a lift of the specified thickness and density. In areas requiring special compaction, it may be necessary to deposit, spread, and compact the RCC in several layers to produce a lift of the specified thickness and density.

Equipment shall not operate directly on any surface that has been cleaned and prepared to receive a subsequent lift of RCC.

Spreading of RCC material shall be performed with a track-type bulldozer in a manner that will not cause damage to previously compacted RCC.

With RCC placement at or near its final location, spreading will typically be limited to leveling the RCC into uniform lifts. In isolated or confined placement locations, the RCC may be deposited and spread up to a maximum distance of 50 feet provided segregation does not occur, specified spreading time is not exceeded and the time specified between mixing and a completion of compaction is not exceeded. The contractor shall conduct placing and spreading operations in a manner that will prevent segregation of RCC.

Guide Specification

If segregation of RCC occurs during the spreading operation, it shall be corrected by immediately reworking the RCC. Reworking shall be performed using techniques that do not damage previously compacted RCC.

SUBSECTION 222.13COMPACTION:

The entire surface area of each RCC lift shall be compacted to the specified density, as determined from the apparent maximum density (AMD) specified in section 6, with adequate compaction overlap to assure complete compaction of the RCC. The in-place wet density of the compacted RCC measured at a depth of 10 inches shall not be less than 98 percent of the AMD.

The wet density of the compacted RCC shall be tested in accordance with ASTM C 1040. The moisture content of the compacted RCC shall be tested in accordance with ASTM D 3017. A minimum of three moisture and density tests will be performed on each lift of RCC with no less than one density measurement and moisture measurement for each 100 cubic yards of RCC compacted.

Production compaction shall be performed with production rollers as defined in this section. RCC compacted with production rollers shall be compacted in single lifts that are 12 inches thick (plus/minus one inch) after the specified density has been achieved.

Production rollers shall consist of single or double smooth drum, large, self-propelled vibratory rollers. Production rollers shall impart a centrifugal force of at least 450 pounds per inch of drum width at the operating frequency during compaction.
Production rollers shall be used in open areas where they can compact RCC to the specified density within the specified time and without damage to the structure, forms, foundation, or appurtenances.

Rollers shall be only operated in the vibratory mode while actively compacting the RCC. Compaction shall be completed with the roller operated in the static mode, as necessary, to achieve the specified density near the surface of the lift.

Placement and compaction of RCC shall be completed without damaging the structure, other structures, forms, foundation, or other embedded appurtenances. Any appurtenance damaged by the compaction process will be repaired or removed and replaced at the contractor's expense.

Special compaction techniques shall be performed using special compaction rollers and power tampers in areas where production rollers cannot maneuver or will cause damage to the structure, forms, foundation, or appurtenances. RCC shall be deposited, spread, and compacted in 4 or 6 inch thick layers as required to obtain uniform specified density throughout the 12-inch lift with limited compaction to avoid drying the surface.

The individual layers that compose one lift shall be deposited, spread, and compacted within 1 hour from the time the first layer within that lift is placed.

Special compaction rollers shall be vibratory rollers that are capable of operating in confined areas and shall produce a centrifugal force of at least 150 pounds per linear inch of drum width

Guide Specification

for each drum of a double drum unit and 300 pounds per linear inch of drum width for a single drum unit.

Power tampers shall develop a force per blow of at least 3,500 pounds per square foot. The maximum layer thickness to be compacted by power tampers shall be 6 +/- inches after the specified density has been achieved. Manually directed vibratory plate compactors may be used to shape, smooth, and level the surface, but shall not be used as a substitute for vibratory rollers and power tampers.

SUBSECTION 222.14 RECORD TESTING:

One set of RCC cylinders for compressive strength tests shall be obtained for each 500 cubic yards of RCC placed to be used for compressive strength testing.

Each set shall consist of 9 compressive strength cylinders prepared in accordance with ASTM C 1176 or C 1435 and weighed to determine the density of the RCC within each cylinder. Any cylinder that weighs less than 98 percent of the weight of the heaviest cylinder shall be discarded and another cylinder prepared and weighed until all 9 cylinders have a weight that is at least 98 percent of that of the heaviest cylinder. Three cylinders shall be tested for compressive strength in accordance with ASTM C 39 at 7, 28, and 56 days, respectively, after the specimens are molded.

SUBSECTION 222.15 LIFT JOINTS:

The term joint, as used in this specification, applies to all surfaces that will eventually be covered by RCC mix, bonding mortar, Portland cement, neat cement grout, or conventional concrete.

SUBSECTION 222.15.1 Joint Condition:

Three potential joint conditions will exist during construction: fresh joint, intermediate joint, and cold joint. The condition of a joint shall be defined on the basis of joint maturity or time of exposure. Joint maturity is defined as the product of the average RCC surface temperature (AST) in degrees Fahrenheit and the time of exposure (TE) in hours. Joint maturity is expressed in degree-hours (deg F-hr) and is calculated as:

Joint maturity in deg F-hr = (AST) x (TE).

The TE shall be the period, expressed to the nearest quarter hour, beginning when the compaction of RCC is completed and ending when covered by the subsequent placement of RCC.

Whenever the joint condition is defined on the basis of joint maturity, the AST shall be determined hourly during the exposure period by measuring the RCC surface temperature at various locations with a surface thermometer. The temperature shall be measured in degrees Fahrenheit and the temperature readings averaged to determine the AST.

A fresh joint is defined as a joint having maturity of 400 deg F-hr or less. In lieu of determining the joint maturity, a fresh joint may be defined as a joint with a TE of 4 hours or less.

Guide Specification

An intermediate joint is defined as a joint having a maturity greater than 400 deg F-hr, but less than or equal to 1,600 deg F-hr. In lieu of determining the joint maturity, an intermediate joint may be defined as a joint with a TE of more than 4 hours, but less than 16 hours.

A cold joint is defined as a joint having a maturity of over 1,600 deg F-hr. In lieu of determining the joint maturity, a cold joint may be defined as a joint with a TE of 16 hours or more.

SUBSECTION 222.15.2 Joint Treatment:
All joint surfaces shall be kept continuously moist, clean, and uncontaminated until placement of succeeding RCC lifts. Water that ponds on a finished surface shall be removed prior to placing RCC.

The surface of previously placed RCC shall be free of soil, dust, or other contaminants prior to being covered with joint treatment material or another layer or lift of RCC. Cleaning of previously placed RCC lifts shall be accomplished by pressurized water and/or air or other methods provided that the surface of the in-place layer is not damaged by the cleaning operation.

The contractor shall have a pressure washer and a blowpipe onsite capable of delivering a combined air-water mixture, with the ability to adjust the pressure, volume, and proportion of air and water.

Edge joints that are exposed for more than 30 minutes shall be trimmed back no less than 9 inches to an RCC surface, that has been compacted to the specified density and beveled at a slope of one horizontal to one vertical. Immediately before placing RCC against a trimmed surface, the joint condition shall be determined and the specified joint treatment shall be applied.

Any surface to be covered with RCC, a bonding material or conventional concrete that is damaged to the extent that over 25 percent of the exposed coarse aggregate is undercut shall be treated as a cold joint. Coarse aggregate with less than half of it surface area remaining embedded and bonded to the RCC is considered undercut.

All RCC materials removed by cleaning, brooming, smoothing, beveling, or trimming layers shall be collected and removed from the structure.

Treatment Method I. Remove any loose materials and contaminants from the lift joint surface. The lift joint surface shall be cleaned using moderate-pressure air immediately before spreading the next RCC lift. Maintain the surface in a moistened condition.

Treatment Method II. Perform Treatment Method I. Before the placement of RCC on the joint surface, uniformly distribute a layer of dry Portland cement over the surface. The Portland cement shall be applied at a rate of 0.5 to 1 pound per square foot of surface. The amount of water applied to the lift surface before, during, or after distributing the cement shall be of sufficient quantity to dampen all of the cement. The ratio of water to cement shall be limited to that which will produce a tacky paste. Water applied after the cement is distributed shall be applied in a fine mist to prevent the displacement of cement. The cement shall be applied immediately ahead of placing the next layer or lift of RCC. The cement shall not be exposed on

Guide Specification

the surface more than 10 minutes before being covered with RCC. Portland cement paste that does not meet these requirements shall be removed from the structure and disposed of, and the treatment method shall be repeated.

Treatment Method III. Perform Treatment Method I. Before the placement of RCC, the joint surface shall be covered with a layer of the bonding mortar specified in Subsection 222.2.6. The thickness of the bonding mortar shall be 0.25 to 0.5 inch. The bonding mortar shall be covered with the next layer or lift of RCC while the mortar is still fluid. In no case shall the bonding mortar remain uncovered for more than 30 minutes. Bonding mortar shall be placed in a manner that will avoid segregation. Bonding mortar that does not meet these requirements shall be removed from the structure and disposed of, and the treatment method shall be repeated.

SUBSECTION 222.16 CURING & PROTECTION:

SUBSECTION 222.16.1 Curing:

Curing of RCC shall begin immediately after compaction. All exposed and completed RCC surfaces shall be cured for a minimum of 14 days at or above 40 degrees Fahrenheit following placement. All repairs including that required to fill holes associated with form anchorages and coring shall be cured for a minimum of 7 days at or above 40 degrees Fahrenheit following repair.

Curing activities shall not be discontinued or interrupted until the RCC has remained at or above 40 degrees Fahrenheit for a total of 14 days. This will require extending the curing period by the number of days that the RCC temperature drops below 40 degrees Fahrenheit during the curing period.

SUBSECTION 222.16.2 Protection:

The temperature of RCC shall be maintained at or above 35 degrees Fahrenheit from the time the RCC is placed until 7 days after the curing period. When ambient temperatures are expected to be below 32 degrees Fahrenheit, measures shall be implemented to protect the RCC from freezing. The protection shall remain in place until ambient temperatures remain continuously above 35 degrees Fahrenheit for 24 hours. Protective measures shall not hinder the specified curing of the RCC.

RCC shall be protected from damage by precipitation, vehicular traffic, or other causes.

SUBSECTION 222.17 VERTICAL SURFACES:

All RCC vertical surfaces that are subject to exposure shall be finished to ensure a minimum of 80 percent of the surface area is free from honeycomb or other voids and is uniform in appearance.

Forming is not required for vertical RCC surfaces that are not subject to permanent exposure.

Guide Specification

SUBSECTION 222.18 TOLERANCES:

Any variation in the face or surface of the finished RCC shall be within the tolerances stated herein.

The structure(s) shall be constructed to the lines and grades depicted on the drawings.

The thickness of compacted lifts of RCC shall be 12 inches plus or minus 1 inch.

Limit gradual overbuild of exposed unformed RCC faces to 0.5 foot. Under build will not be allowed.

Do not exceed 0.1 foot in 10 feet variation in an unformed, exposed RCC face as measured in a straight line along the length and height of the face, or 0.5 foot over the entire length of the structure. Abrupt changes shall be less than 0.1 foot.

The elevation of any horizontal RCC surface shall be plus or minus 0.1 foot of the specified grade except that the elevation of a finished top of dam or spillway crest shall be no more than 0.1 foot above the specified elevation.

SUBSECTION 222.19 RCC REPAIR:

SUBSECTION 222.19.1 Repair or Replacement:
The contractor shall repair or replace RCC that does not meet the requirements of this specification. Before starting any repair or replacement work, the contractor shall prepare a written plan for the repair or replacement. The primary reference for material and repair methods for the plan shall be the appropriate sections of the American Concrete Institute's Manual of Concrete Inspection. The repair plan shall be submitted for review and approval at least 10 days before any repair or replacement work.

Guide Specification

SUBSECTION 222.19.2 Patching:
All form bolts, metal ties, and similar forming restraints shall be removed to a depth of 1 inch below the surface of the RCC and their cavities repaired unless otherwise specifically permitted or approved by the Engineer. Small cavities large air holes, minor honeycombed areas, holes created from test coring, and other superficial imperfections that require patching to meet the specified finish requirement shall be thoroughly cleaned and filled. Holes left by bolts or straps that passes through the RCC shall be filled solid with a dense, well-bonded nonshrink patching material. Dry-pack mortar and replacement concrete shall follow the appropriate procedure detailed in the Repair and Maintenance chapter of the Concrete Manual, Bureau of Reclamation, U.S. Department of the Interior. Proprietary patching material shall be appropriate for the type of repair used within the manufacturer's recommended limits and applied according to the manufacturer's recommendations.

When proprietary patching material is proposed in the plan, the manufacturer's data sheets and written recommendations shall be included in the plan.

Repair material or replacement concrete shall have properties, color, and texture similar to and compatible with the concrete being repaired or replaced.

SUBSECTION 222.20 CLEANUP OF SPILLAGE:

All loose gravel and uncompacted RCC material shall be removed from the structure for disposal in approved location(s) and shall not be recycled into the RCC mix.

SUBSECTION 222.21 MEASUREMENT:

For items of work for which specific unit prices are established in the contract, the volume of RCC is measured and computed to the nearest cubic yard by the method of average cross-sectional end areas. No deduction in volume is made for embedded items, such as, but not limited to conduits, inlet structures, outlet structures, and their appurtenances.

The volume of RCC shall be determined by measuring from the surface of the foundation when approved for RCC placement to the specified neatlines of the completed RCC structures.

If the test section is not incorporated into the RCC structure, the volume of RCC placed into the construction of the test section will be added to the volume computation of the completed RCC structure to determine the total volume of RCC for payment. If the test section is incorporated into the RCC structure, no addition volume of RCC shall be included for payment.

SUBSECTION 222.22 PAYMENT:

Payment for the RCC, for which a specific unit price is established in the contract, will be computed to the nearest cubic yard. Payment shall constitute full compensation for furnishing all labor, materials (except cementitious materials), equipment, tools transportation, and all other items necessary and incidental to the construction and removal of the test section and construction of the RCC structure, including joint treatment, trimming and removal, repair, replacement, patching, curing, protection, site clean up, and disposal of spillage and waste materials. Payment will not be made for RCC material that is wasted or rejected for failure to comply with this specification.

BID ITEM 222-01 RCC

Payment for each cementitious component of the RCC, for which a separate bid item is included in the contract, shall include the quantity incorporated into the RCC structure and test section. This quantity shall be computed based on statement of delivery tickets. Payment will not be made for any cementitious materials not incorporated into the structure(s) or test section. Payment for the cementitious materials for which specific unit prices are established in the contract will be to the nearest 0.1 ton of cementitious materials.

BID ITEM 222-02 RCC CEMENT

Guide Specification

STRUCTURAL

OAK STREET BASIN
AND STORM DRAIN PROJECT

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
PCN 420.04.31
FCD CONTRACT NO. 2010C038
CITY OF MESA PROJECT NO. 02-239-001

STRUCTURAL CALCULATIONS

October 9, 2012

Premier Project No. 2009034

Prepared By:

Premier Engineering Corporation
6437 West Chandler Blvd
Suite 1
Chandler, AZ 85226
Phone: (480) 829-6000

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1.0

FOUNDATION DESIGN

Rajesh Dholakia

From: Kip Reese [kreese@rammeng.com]

Sent: Friday, November 12, 2010 2:44 PM

To: Rajesh Dholakia; Sergio Oliden

Cc: Mark Gavan; Omer Karovic

Subject: G17544 - Preliminary Geotechnical Engineering Recommendations - OAK STREET BASIN Retaining Walls and Drainage Structures

Raj:

Based on our telephone conversations, review of 60% drawings for the project, estimated and measured soil and bedrock properties and soil and bedrock encountered in our test borings and test pits, our Preliminary Geotechnical Engineering Recommendations for retaining walls and drainage structures are presented below.

Retaining walls and drainage structures may be supported on spread and mat-type footings bearing on undisturbed soils, undisturbed decomposed to slightly weathered granite bedrock and/or compacted fill. The retaining walls and drainage structures will likely be founded on granite bedrock. Please refer to Drawing Numbers SB1 through SB5 (Sheets 51 through 55) of the 60% drawings for depths of soil/bedrock. The following values may be used for design:

	Foundation Type				
	Spread		Mat		
Bearing Material	Depth of Embedment (feet below finish grade)	Bearing Pressure (psf)	Depth of Embedment (feet below finish grade)	Bearing Pressure (psf)	Modulus of Subgrade Reaction (pci)
Soil	1.5	1500	1.0	1000	250
	2.0	2000	1.5	1500	300
Bedrock	1.5	3000	0.0	2000	NA
	2.0	4000	0.5	2500	NA

All footing excavations should be reviewed by the geotechnical engineer, prior to placing reinforcing steel or concrete. Any loose, disturbed or unstable soils and/or bedrock should be removed from the bearing surface and replaced with MAG cement/AB slurry or as directed by the geotechnical engineer. The estimated total and differential footing settlements for anticipated loading conditions and anticipated presence of water at or near footing level are less than 3/4 inch for foundations bearing on undisturbed site soils and and/or compacted fill and less than ¼ inch for foundations bearing on undisturbed decomposed to slightly weathered granite bedrock. Continuous footings and retaining walls should be reinforced and mat foundations constructed with properly designed reinforcement and with frequent expansion/contraction joints. A Site Class designation of B should be used for the site per the 2000, 2003 and 2006 International Building Code (IBC).

	Material Properties/Parameters		
	Soil	Bedrock	Backfill
Unit Weight (pcf, dry)	110	150	125
Angle of Internal Friction (degrees)	32	45	35
Cohesion (psf)	0	0	0

The site soils may be used as fill and backfill in all areas of the site provided these soils are free of organic materials, debris, rubble and material greater than 3 inches in size. Import material used as structure backfill should conform to the requirements of MAG Specifications for Structure Excavation and Backfill (Section 206).

The following tabulation presents the recommended lateral earth pressures and base friction values which should be used in the lateral design of footings, keyways, below grade structures and earth retaining systems. The lateral pressures are equivalent fluid pressures for average anticipated conditions.

Backfill Pressures (Site Soils and Import):
- Unrestrained walls---------- 32 psf/ft
- Unrestrained walls (saturated)---------- 80 psf/ft
- Restrained walls---------- 55 psf/ft
- Restrained walls (saturated)---------- 90 psf/ft

Passive Pressures
- Continuous (Site Soils)---------- 350 psf/ft
- Continuous (Site Soils, saturated)---------- 235 psf/ft
- Continuous (Granite Bedrock)---------- 500 psf/ft
- Continuous (Granite Bedrock, saturated)---------- 400 psf/ft

Coefficient of Base Friction:
- Concrete to soil (dry and saturated)---------- 0.45
- Concrete to granite bedrock (dry and saturated)---------- 0.60

The above equivalent fluid pressures are for vertical walls with horizontal backfills and do not include temporary loads imposed by compaction equipment or permanent loads resulting from backfill swell pressures, or surcharge loads. Any retaining walls should contain weep holes to reduce the potential for the buildup of hydrostatic pressures.

Please call/email if you have any questions.

Regards,

Kip

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From: Rajesh Dholakia [mailto:rdholakia@pre-engr.com]
Sent: Friday, November 05, 2010 1:25 PM
To: Kip Reese; Sergio Oliden
Cc: Mark Gavan; Omer Karovic
Subject: OAK STREET BASIN -Geotech Data

Kip,

We are designing retaining walls and drainage structures for this site. We need the following information for foundation design.
I do have your Geotechnical Report but it does not have the following data.

Allowable soil bearing capacity for a spread footing @ each structure location
Coefficient of friction

Depth of each layer of soil and its following characteristics
Unit weight of foundation soil
Angle of internal friction
Modulus of subgrade reaction
Cohesion coefficient if any

Backfill material characteristics such as
Unit weight
Angle of internal friction
Active and At-rest earth pressure (Dry)
Active and At-rest earth pressure (Saturated)

Whatever additional information that can be useful in structure design, please provide us.

Thanks.

Rajesh Dholakia, PE

15 years PREMIER ENGINEERING CORPORATION

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2.0

INLET STRUCTURE DESIGN

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 20091034

TITLE Oak Street Basin & Storm Drain

SUBJECT Inlet Structures

MADE BY JPH DATE 11-24-10 CHECKED BY Raj D DATE 12/26/10

Design Assumptions for Inlet Structures

- Wall Drains will not be provided.
- Design for saturated soil below the top of the lowest wall.
- There may be some construction surcharge loads on the walls, however it is assumed that the soil will not be saturated during construction, Therefore, no surcharges will be considered as the saturated soil case will govern.
- Walls are assumed to be fixed at all joints
- The Inlet Structures are not assumed to store water and do need to be designed for such requirements (ie. stringent crack control).
- The analysis is done following the book "Rectangular Concrete Tanks", revised fifth edition, by Javeed A. Munshi. The book provides moment and shear coefficients for different loading conditions, fixity conditions, and size ratios. The results provided are from finite element analyses of two-dimensional plates subject to out of plane loads using SAP.

2009034 Oak Street Basin & Storm Drain

Inlet Structure Dimensions:

Struct. No.	I.D. (in)	W (ft)	L (ft)	E (ft)	F (ft)	Min Inside Width (ft)	Max Inside Width (ft)	Top of Wall	Top of Wier	Invert	Inside Length (ft)
1	36	8.33	4.00	2.33	1.33	8.33	13.00	1813.83	1812.00	1806.00	4.00
2	48	9.67	4.50	2.58	1.50	9.67	14.83	1830.80	1828.80	1822.00	4.50
3	24	6.00	3.50	2.00	1.17	6.00	10.00	1820.17	1819.00	1814.00	3.50
5	72	18.00	6.00	3.50	2.00	18.00	25.00	1849.8	1847.3	1837	6.00
6	72	21.00	6.00	3.50	2.00	21.00	28.00	1868.70	1866.20	1855.04	6.00

Struct. No.	Pipe Thick. (in)	Wall Thick. (in)	Bot. Slab Elev.	Max Wall Ht (ft)	Min Wall Ht (ft)	Avg Wall Ht (ft)	Min Outside Width (ft)	Max Outside Width (ft)	Ave Outside Width (ft)	Outside Length (ft)	Side Wall Length (ft)
1	3	10	1805.25	8.58	6.75	7.66	10.00	14.67	12.33	5.67	6.54
2	4	10	1821.17	9.63	7.63	8.63	11.33	16.50	13.92	6.17	7.12
3	2	10	1813.33	6.84	5.67	6.25	7.67	11.67	9.67	5.17	5.97
5	6	12	1836.00	13.80	11.30	12.55	20.00	27.00	23.50	8.00	9.24
6	6	12	1854.04	14.66	12.16	13.41	23.00	30.00	26.50	8.00	9.24

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

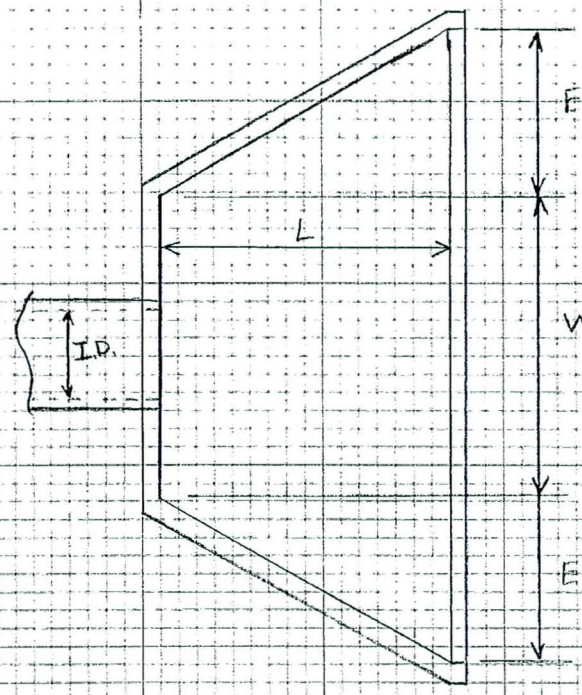
JOB NUMBER 2009034

TITLE Oak St Basin

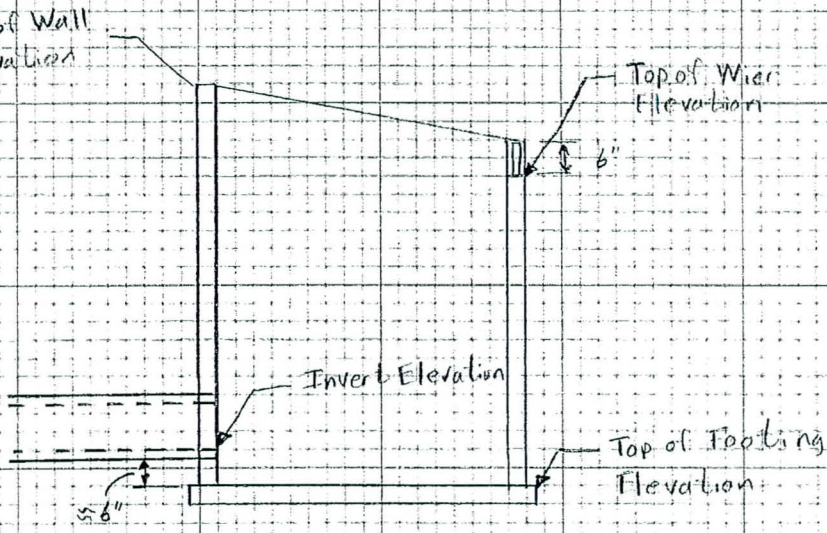
SUBJECT Inlet Structures

MADE BY JPH DATE 11-30-10 CHECKED BY Raj D DATE 12-26-10

Dimensions



Plan



2.1

INLET STRUCTURE #1

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

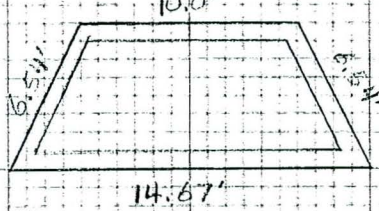
JOB NUMBER 2009034

TITLE _____

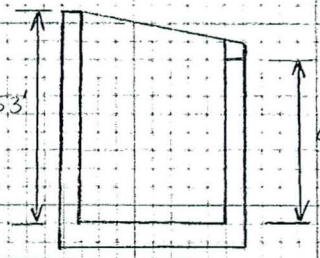
SUBJECT Inlet Structure #1

MADE BY JPH DATE 11-18-10 CHECKED BY Raj D DATE 12/26/10

Rectangular Cracked Tank Analysis



Plan



Section

High Wall

$a = 8.53'$
$b = 10.0'$
$c = 6.54'$

$\frac{b}{a} = 1.17$
$\frac{c}{a} = 0.77$

→ Use $\frac{b}{a} = 1.0$ $\frac{c}{a} = 0.5$

$q = (60\ psf/ft)(8.53' - 6.75') + (97\ psf/ft)(6.75')$
$= 106.8 + 668 = 775\ psf$

$q_u = \gamma(\beta_E q) = 1.3(1.15)(775\ psf) = 1159\ psf$

Short Wall

$a = 6.75'$
$b = 14.67'$
$c = 6.54'$

$\frac{b}{a} = 2.17$
$\frac{c}{a} = 0.97$

→ Use $\frac{b}{a} = 2.0$ $\frac{c}{a} = 1.0$

$q = (90\ psf/ft)(6.75') = 608\ psf$

$q_u = \gamma(\beta_E q) = 1.3(1.15)(608\ psf) = 909\ psf$

- Assume Free Top and Fixed Base
- Side walls (short side) will be designed for worst case moments from tables for High Wall and Short Wall cases.

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2009034

TITLE Oak Street Basin & Storm Drain

SUBJECT Inlet Structure #1

MADE BY JPH DATE 11-24-10 CHECKED BY RJD DATE 12/26/10

Bottom Slab

Negative Moment for bottom slab will be equal to negative moments in the bottom of the wall.

Positive moments will be determined from a rectangular tank analysis of the slab assuming hinged connections to the walls

b = Avg width of slab = 12.33'

a = Length of Slab = 5.67'

b/a = 2.17 → use b/a = 2.5

$W_{shortwall} = (150)(14.67')(6.75')(10/12) = 12.38^K$

$W_{highwall} = (150)(10.90')(2.58')(10/12) = 10.73^K$

$W_{sidewalls} = (150)(6.54')(7.66')(10/12)(2) = 12.52^K$

35.6^K

$A_{slab} = b \cdot a = 69.9\ ft^2$

$q = \frac{W_{walls}}{A_{slab}} = 509\ psf$

$q_u = \gamma B_D q = (1.3)(1.0)(509) = 662\ psf$

- See Excell Spreadsheet for Moment Calculations

Inlet Structure #1:

Shear Design, Vu = Cs*qu*a

High Wall - b/a = 1.0, c/a = 0.5

qu =	1159	psf
a =	8.53	ft

Shear Coefficients, Cs, for b/a = 1.0	
Bottom Edge - Midpoint =	0.32
Side Edge - Maximum =	0.24
Side Edge - Midpoint =	0.23

Shear Coefficients for c/a = 0.5	
Bottom Edge - Midpoint =	0.19
Side Edge - Maximum =	0.17
Side Edge - Midpoint =	0.13

Max Bottom Shear Coefficient =	0.32
Max Side Shear Coefficient =	0.24

Bottom Shear Force =	3164	lb
Side Shear Force =	2373	lb

Short Wall - b/a = 2.0, c/a = 1.0

qu =	1159	psf
a =	8.53	ft

Shear Coefficients, Cs, for b/a = 2.0	
Bottom Edge - Midpoint =	0.45
Side Edge - Maximum =	0.27
Side Edge - Midpoint =	0.26

Shear Coefficients for c/a = 1.0	
Bottom Edge - Midpoint =	0.32
Side Edge - Maximum =	0.24
Side Edge - Midpoint =	0.23

Max Bottom Shear Coefficient =	0.45
Max Side Shear Coefficient =	0.27

Bottom Shear Force =	4449	lb
Side Shear Force =	2669	lb

Max Shear Force, Vu = 4449 lb

Check walls for Shear

f'c =	3000	psi
t =	10	in
d =	7.625	in
b =	12	in

Vc = 2*sqrt(f'c)*b*d

Φ = 0.85

ΦVc = 8520 lb > Vu = 4449 lb OK

Inlet Structure #1:

High Wall - b/a = 1.0, c/a = 0.5
(p 3-34)

qu = 1159 psf
a = 8.53 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-3	-4	3	9	13	14
0.9a	-13	-4	4	9	13	14
0.8a	-15	-4	4	10	14	15
0.7a	-17	-5	4	11	14	16
0.6a	-19	-5	5	11	15	16
0.5a	-21	-5	5	11	14	15
0.4a	-21	-4	5	10	13	14
0.3a	-19	-4	4	8	10	10
0.2a	-14	2	3	4	5	5
0.1a	-6	-1	0	-1	-1	-1
Bottom	0	-2	-5	-7	-8	-8

My Coefficients

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-3	-11	-10	-10	-9	-9
0.9a	-13	-11	-10	-9	-8	-8
0.8a	-15	-12	-10	-8	-7	-7
0.7a	-17	-13	-10	-7	-6	-6
0.6a	-19	-14	-9	-6	-5	-4
0.5a	-21	-14	-8	-5	-3	-2
0.4a	-21	-13	-7	-3	-1	0
0.3a	-19	-10	-5	-1	1	2
0.2a	-14	-7	-2	1	2	3
0.1a	-6	-2	0	1	1	2
Bottom	0	0	0	-1	-1	-1

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.25	-0.34	0.25	0.76	1.10	1.18
0.9a	-1.10	-0.34	0.34	0.76	1.10	1.18
0.8a	-1.26	-0.34	0.34	0.84	1.18	1.26
0.7a	-1.43	-0.42	0.34	0.93	1.18	1.35
0.6a	-1.60	-0.42	0.42	0.93	1.26	1.35
0.5a	-1.77	-0.42	0.42	0.93	1.18	1.26
0.4a	-1.77	-0.34	0.42	0.84	1.10	1.18
0.3a	-1.60	-0.34	0.34	0.67	0.84	0.84
0.2a	-1.18	0.17	0.25	0.34	0.42	0.42
0.1a	-0.51	-0.08	0.00	-0.08	-0.08	-0.08
Bottom	0.00	-0.17	-0.42	-0.59	-0.67	-0.67

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.25	-0.93	-0.84	-0.84	-0.76	-0.76
0.9a	-1.10	-0.93	-0.84	-0.76	-0.67	-0.67
0.8a	-1.26	-1.01	-0.84	-0.67	-0.59	-0.59
0.7a	-1.43	-1.10	-0.84	-0.59	-0.51	-0.51
0.6a	-1.60	-1.18	-0.76	-0.51	-0.42	-0.34
0.5a	-1.77	-1.18	-0.67	-0.42	-0.25	-0.17
0.4a	-1.77	-1.10	-0.59	-0.25	-0.08	0.00
0.3a	-1.60	-0.84	-0.42	-0.08	0.08	0.17
0.2a	-1.18	-0.59	-0.17	0.08	0.17	0.25
0.1a	-0.51	-0.17	0.00	0.08	0.08	0.17
Bottom	0.00	0.00	0.00	-0.08	-0.08	-0.08

Short Wall - b/a = 2.0, c/a = 1.0
(p 3-32)

qu = 1159 psf
a = 8.53 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-22	-13	8	20	27	29
0.9a	-42	-12	8	19	25	27
0.8a	-41	-11	8	19	24	25
0.7a	-41	-10	8	18	22	23
0.6a	-41	-8	8	16	19	20
0.5a	-39	-6	8	14	16	16
0.4a	-35	-4	7	10	11	11
0.3a	-28	-3	4	6	5	5
0.2a	-18	-2	0	0	-1	-2
0.1a	-6	-3	-5	-8	-9	-10
Bottom	0	-5	-11	-16	-18	-19

My Coefficients

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-22	-29	-18	-9	-4	-2
0.9a	-42	-27	-16	-7	-2	0
0.8a	-41	-26	-14	-5	0	2
0.7a	-41	-24	-11	-3	2	4
0.6a	-41	-22	-9	0	5	6
0.5a	-39	-19	-6	2	7	8
0.4a	-35	-15	-3	4	7	9
0.3a	-28	-11	-1	4	7	8
0.2a	-18	-6	0	3	5	5
0.1a	-6	-2	0	1	1	1
Bottom	0	0	-2	-3	-4	-5

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-1.86	-1.10	0.67	1.69	2.28	2.45
0.9a	-3.54	-1.01	0.67	1.60	2.11	2.28
0.8a	-3.46	-0.93	0.67	1.60	2.02	2.11
0.7a	-3.46	-0.84	0.67	1.52	1.86	1.94
0.6a	-3.46	-0.67	0.67	1.35	1.60	1.69
0.5a	-3.29	-0.51	0.67	1.18	1.35	1.35
0.4a	-2.95	-0.34	0.59	0.84	0.93	0.93
0.3a	-2.36	-0.25	0.34	0.51	0.42	0.42
0.2a	-1.52	-0.17	0.00	0.00	-0.08	-0.17
0.1a	-0.51	-0.25	-0.42	-0.67	-0.76	-0.84
Bottom	0.00	-0.42	-0.93	-1.35	-1.52	-1.60

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-1.86	-2.45	-1.52	-0.76	-0.34	-0.17
0.9a	-3.54	-2.28	-1.35	-0.59	-0.17	0.00
0.8a	-3.46	-2.19	-1.18	-0.42	0.00	0.17
0.7a	-3.46	-2.02	-0.93	-0.25	0.17	0.34
0.6a	-3.46	-1.86	-0.76	0.00	0.42	0.51
0.5a	-3.29	-1.60	-0.51	0.17	0.59	0.67
0.4a	-2.95	-1.26	-0.25	0.34	0.59	0.76
0.3a	-2.36	-0.93	-0.08	0.34	0.59	0.67
0.2a	-1.52	-0.51	0.00	0.25	0.42	0.42
0.1a	-0.51	-0.17	0.00	0.08	0.08	0.08
Bottom	0.00	0.00	-0.17	-0.25	-0.34	-0.42

Inlet Structure #1:

Maximum Positive Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	Maximum
Top	-0.08	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-0.25	-0.08	0.08	0.25	0.42	0.42	**0.42**
0.8a	-0.25	-0.08	0.34	0.59	0.76	0.84	**0.84**
0.7a	-0.25	0.00	0.51	0.84	1.10	1.10	**1.10**
Long Side 0.6a	-0.34	0.08	0.67	1.01	1.18	1.26	**1.26**
0.5a	-0.34	0.08	0.67	1.01	1.10	1.18	**1.18**
0.4a	-0.34	0.08	0.51	0.76	0.93	0.93	**0.93**
0.3a	-0.34	0.08	0.42	0.59	0.76	0.76	**0.76**
0.2a	-0.25	0.00	0.08	0.17	0.17	0.17	**0.17**
0.1a	-0.08	-0.25	-0.59	-0.84	-1.10	-1.18	**-0.08**
Bottom	0.00	-0.84	-1.94	-2.78	-3.29	-3.46	**0.00**

Maximum Positive Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	Maximum
Top	-0.08	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-0.25	-0.17	-0.17	-0.08	-0.08	-0.08	**-0.08**
0.8a	-0.25	-0.25	-0.17	-0.17	-0.08	0.00	**0.00**
0.7a	-0.25	-0.25	-0.17	0.00	0.08	0.17	**0.17**
Short Side 0.6a	-0.34	-0.25	0.00	0.25	0.34	0.42	**0.42**
0.5a	-0.34	-0.25	0.17	0.42	0.59	0.67	**0.67**
0.4a	0.59	-0.08	0.34	0.67	0.84	0.93	**0.93**
0.3a	-0.34	0.00	0.42	0.76	0.93	0.93	**0.93**
0.2a	-0.25	0.08	0.42	0.59	0.67	0.67	**0.67**
0.1a	-0.08	0.08	0.17	0.17	0.25	0.25	**0.25**
Bottom	0.00	0.08	-0.08	-0.25	-0.42	-0.42	**0.08**

Maximum Negative Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	Maximum
Top	-0.34	0.00	0.00	0.00	0.00	0.00	**-0.34**
0.9a	-0.67	-0.17	0.00	0.08	0.08	0.08	**-0.67**
0.8a	-0.67	-0.08	0.08	0.17	0.25	0.25	**-0.67**
0.7a	-0.67	0.00	0.17	0.34	0.51	0.51	**-0.67**
Long Side 0.6a	-0.67	0.00	0.34	0.59	0.67	0.76	**-0.67**
0.5a	-0.67	0.08	0.42	0.67	0.84	0.93	**-0.67**
0.4a	-0.59	0.08	0.42	0.67	0.67	0.67	**-0.59**
0.3a	-0.51	-0.08	0.00	-0.17	-0.34	-0.42	**-0.51**
0.2a	-0.34	-0.42	-0.93	-1.52	-1.94	-2.02	**-2.02**
0.1a	-0.08	-1.10	-2.45	-3.63	-4.30	-4.55	**-4.55**
Bottom	0.00	-2.28	-4.81	-6.66	-7.67	-8.01	**-8.01**

Maximum Negative Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	Maximum
Top	-0.34	0.00	0.00	0.00	0.00	0.00	**-0.34**
0.9a	-0.67	-0.42	-0.25	-0.17	-0.08	-0.08	**-0.67**
0.8a	-0.67	-0.51	-0.25	-0.17	-0.17	-0.17	**-0.67**
0.7a	-0.67	-0.42	-0.17	-0.17	-0.17	-0.17	**-0.67**
Short Side 0.6a	-0.67	-0.34	-0.25	-0.17	-0.17	-0.17	**-0.67**
0.5a	-0.67	-0.25	-0.17	-0.17	-0.08	-0.08	**-0.67**
0.4a	-0.34	-0.25	-0.17	-0.08	0.00	0.00	**-0.34**
0.3a	-0.51	-0.17	-0.08	0.08	0.08	0.17	**-0.51**
0.2a	-0.34	-0.08	0.08	0.17	0.25	0.25	**-0.34**
0.1a	-0.08	0.08	0.08	-0.08	-0.17	-0.17	**-0.17**
Bottom	0.00	-0.17	-0.84	-1.43	-1.86	-1.94	**-1.94**

Inlet Structure #1:

High Wall - b/a = 1.0, c/a = 0.5
(p 3-34)
qu = 1159 psf
a = 8.53 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-1	0	0	0	0	0
0.9a	-3	-1	0	1	1	1
0.8a	-3	-1	1	2	3	3
0.7a	-3	0	2	4	6	6
Long Side 0.6a	-4	0	4	7	8	9
0.5a	-4	1	5	8	10	11
0.4a	-4	1	5	9	11	11
0.3a	-4	1	5	7	9	9
0.2a	-3	0	1	2	2	2
0.1a	-1	-3	-7	-10	-13	-14
Bottom	0	-10	-23	-33	-39	-41

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-1	0	0	0	0	0
0.9a	-3	-2	-2	-1	-1	-1
0.8a	-3	-3	-2	-2	-2	-2
0.7a	-3	-3	-2	-2	-2	-2
Short Side 0.6a	-4	-3	-3	-2	-2	-2
0.5a	-4	-3	-2	-2	-1	-1
0.4a	-4	-3	-2	-1	0	0
0.3a	-4	-2	-1	1	1	2
0.2a	-3	-1	1	2	3	3
0.1a	-1	1	2	2	3	3
Bottom	0	1	-1	-3	-5	-5

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.08	0.00	0.00	0.00	0.00	0.00
0.9a	-0.25	-0.08	0.00	0.08	0.08	0.08
0.8a	-0.25	-0.08	0.08	0.17	0.25	0.25
0.7a	-0.25	0.00	0.17	0.34	0.51	0.51
Long Side 0.6a	-0.34	0.00	0.34	0.59	0.67	0.76
0.5a	-0.34	0.08	0.42	0.67	0.84	0.93
0.4a	-0.34	0.08	0.42	0.76	0.93	0.93
0.3a	-0.34	0.08	0.42	0.59	0.76	0.76
0.2a	-0.25	0.00	0.08	0.17	0.17	0.17
0.1a	-0.08	-0.25	-0.59	-0.84	-1.10	-1.18
Bottom	0.00	-0.84	-1.94	-2.78	-3.29	-3.46

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.08	0.00	0.00	0.00	0.00	0.00
0.9a	-0.25	-0.17	-0.17	-0.08	-0.08	-0.08
0.8a	-0.25	-0.25	-0.17	-0.17	-0.17	-0.17
0.7a	-0.25	-0.25	-0.17	-0.17	-0.17	-0.17
Short Side 0.6a	-0.34	-0.25	-0.25	-0.17	-0.17	-0.17
0.5a	-0.34	-0.25	-0.17	-0.17	-0.08	-0.08
0.4a	-0.34	-0.25	-0.17	-0.08	0.00	0.00
0.3a	-0.34	-0.17	-0.08	0.08	0.08	0.17
0.2a	-0.25	-0.08	0.08	0.17	0.25	0.25
0.1a	-0.08	0.08	0.17	0.17	0.25	0.25
Bottom	0.00	0.08	-0.08	-0.25	-0.42	-0.42

Short Wall - b/a = 2.0, c/a = 1.0
(p 3-32)
qu = 1159 psf
a = 8.53 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-4	0	0	0	0	0
0.9a	-8	-2	1	3	5	5
0.8a	-8	-1	4	7	9	10
0.7a	-8	0	6	10	13	13
Long Side 0.6a	-8	1	8	12	14	15
0.5a	-8	1	8	12	13	14
0.4a	-7	1	6	8	8	8
0.3a	-6	-1	0	-2	-4	-5
0.2a	-4	-5	-11	-18	-23	-24
0.1a	-1	-13	-29	-43	-51	-54
Bottom	0	-27	-57	-79	-91	-95

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-4	0	0	0	0	0
0.9a	-8	-5	-3	-2	-1	-1
0.8a	-8	-6	-3	-2	-1	0
0.7a	-8	-5	-2	0	1	2
Short Side 0.6a	-8	-4	0	3	4	5
0.5a	-8	-3	2	5	7	8
0.4a	7	-1	4	8	10	11
0.3a	-6	0	5	9	11	11
0.2a	-4	1	5	7	8	8
0.1a	-1	1	1	-1	-2	-2
Bottom	0	-2	-10	-17	-22	-23

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.34	0.00	0.00	0.00	0.00	0.00
0.9a	-0.67	-0.17	0.08	0.25	0.42	0.42
0.8a	-0.67	-0.08	0.34	0.59	0.76	0.84
0.7a	-0.67	0.00	0.51	0.84	1.10	1.10
Long Side 0.6a	-0.67	0.08	0.67	1.01	1.18	1.26
0.5a	-0.67	0.08	0.67	1.01	1.10	1.18
0.4a	-0.59	0.08	0.51	0.67	0.67	0.67
0.3a	-0.51	-0.08	0.00	-0.17	-0.34	-0.42
0.2a	-0.34	-0.42	-0.93	-1.52	-1.94	-2.02
0.1a	-0.08	-1.10	-2.45	-3.63	-4.30	-4.55
Bottom	0.00	-2.28	-4.81	-6.66	-7.67	-8.01

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.34	0.00	0.00	0.00	0.00	0.00
0.9a	-0.67	-0.42	-0.25	-0.17	-0.08	-0.08
0.8a	-0.67	-0.51	-0.25	-0.17	-0.08	0.00
0.7a	-0.67	-0.42	-0.17	0.00	0.08	0.17
Short Side 0.6a	-0.67	-0.34	0.00	0.25	0.34	0.42
0.5a	-0.67	-0.25	0.17	0.42	0.59	0.67
0.4a	0.59	-0.08	0.34	0.67	0.84	0.93
0.3a	-0.51	0.00	0.42	0.76	0.93	0.93
0.2a	-0.34	0.08	0.42	0.59	0.67	0.67
0.1a	-0.08	0.08	0.08	-0.08	-0.17	-0.17
Bottom	0.00	-0.17	-0.84	-1.43	-1.86	-1.94

Inlet Structure #1:

Maximum Positive Horizontal Bending Moments

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.25	-0.34	0.67	1.69	2.28	2.45
0.9a	-1.10	-0.34	0.67	1.60	2.11	2.28
0.8a	-1.26	-0.34	0.67	1.60	2.02	2.11
0.7a	-1.43	-0.42	0.67	1.52	1.86	1.94
0.6a	-1.60	-0.42	0.67	1.35	1.60	1.69
0.5a	-1.77	-0.42	0.67	1.18	1.35	1.35
0.4a	-1.77	-0.34	0.59	0.84	1.10	1.18
0.3a	-1.60	-0.25	0.34	0.67	0.84	0.84
0.2a	-1.18	0.17	0.25	0.34	0.42	0.42
0.1a	-0.51	-0.08	0.00	-0.08	-0.08	-0.08
Bottom	0.00	-0.17	-0.42	-0.59	-0.67	-0.67
Maximum	**0.00**	**0.17**	**0.67**	**1.69**	**2.28**	**2.45**

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.25	-0.93	-0.84	-0.76	-0.34	-0.17
0.9a	-1.10	-0.93	-0.84	-0.59	-0.17	0.00
0.8a	-1.26	-1.01	-0.84	-0.42	0.00	0.17
0.7a	-1.43	-1.10	-0.84	-0.25	0.17	0.34
0.6a	-1.60	-1.18	-0.76	0.00	0.42	0.51
0.5a	-1.77	-1.18	-0.51	0.17	0.59	0.67
0.4a	-1.77	-1.10	-0.25	0.34	0.59	0.76
0.3a	-1.60	-0.84	-0.08	0.34	0.59	0.67
0.2a	-1.18	-0.51	0.00	0.25	0.42	0.42
0.1a	-0.51	-0.17	0.00	0.08	0.08	0.17
Bottom	0.00	0.00	0.00	-0.08	-0.08	-0.08
Maximum	**0.00**	**0.00**	**0.00**	**0.34**	**0.59**	**0.76**

Maximum Negative Horizontal Bending Moments

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-1.86	-1.10	0.25	0.76	1.10	1.18
0.9a	-3.54	-1.01	0.34	0.76	1.10	1.18
0.8a	-3.46	-0.93	0.34	0.84	1.18	1.26
0.7a	-3.46	-0.84	0.34	0.93	1.18	1.35
0.6a	-3.46	-0.67	0.42	0.93	1.26	1.35
0.5a	-3.29	-0.51	0.42	0.93	1.18	1.26
0.4a	-2.95	-0.34	0.42	0.84	0.93	0.93
0.3a	-2.36	-0.34	0.34	0.51	0.42	0.42
0.2a	-1.52	-0.17	0.00	0.00	-0.08	-0.17
0.1a	-0.51	-0.25	-0.42	-0.67	-0.76	-0.84
Bottom	0.00	-0.42	-0.93	-1.35	-1.52	-1.60
Maximum	**-3.54**	**-1.10**	**-0.93**	**-1.35**	**-1.52**	**-1.60**

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-1.86	-2.45	-1.52	-0.84	-0.76	-0.76
0.9a	-3.54	-2.28	-1.35	-0.76	-0.67	-0.67
0.8a	-3.46	-2.19	-1.18	-0.67	-0.59	-0.59
0.7a	-3.46	-2.02	-0.93	-0.59	-0.51	-0.51
0.6a	-3.46	-1.86	-0.76	-0.51	-0.42	-0.34
0.5a	-3.29	-1.60	-0.67	-0.42	-0.25	-0.17
0.4a	-2.95	-1.26	-0.59	-0.25	-0.08	0.00
0.3a	-2.36	-0.93	-0.42	-0.08	0.08	0.17
0.2a	-1.52	-0.59	-0.17	0.08	0.17	0.25
0.1a	-0.51	-0.17	0.00	0.08	0.08	0.08
Bottom	0.00	0.00	-0.17	-0.25	-0.34	-0.42
Maximum	**-3.54**	**-2.45**	**-1.52**	**-0.84**	**-0.76**	**-0.76**

PREMIER ENGINEERING CORPORATION

TITLE Oak Street Basin Storm Drain
SUBJECT Inlet Structures
BY JPH DATE 11/22/10 CHKD BY RMD DATE 12/26/10

SHEET # ___ OF ___
JOB # 2009034

Inlet Structure #1:

Bottom Slab Positive Moments (tension on top face):

b = 12.33 ft
a = 5.67 ft
b/a = 2.1746032 -----> 2.5 (p 2-61)

qu = 662 psf

My Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0	0	0	0	0	0
0.9a	0	11	13	12	11	11
0.8a	0	20	23	22	21	20
0.7a	0	27	31	29	27	27
0.6a	0	30	35	34	31	31
0.5a	0	32	37	35	33	32
0.4a	0	30	35	34	31	31
0.3a	0	27	31	29	27	27
0.2a	0	20	23	22	21	20
0.1a	0	11	13	12	11	11
Bottom	0	0	0	0	0	0

Mx Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0	0	0	0	0	0
0.9a	0	20	31	37	40	41
0.8a	0	34	54	65	71	72
0.7a	0	42	69	84	92	94
0.6a	0	47	78	96	105	108
0.5a	0	48	81	100	109	112
0.4a	0	47	78	96	105	108
0.3a	0	42	69	84	92	94
0.2a	0	34	54	65	71	72
0.1a	0	20	31	37	40	41
Bottom	0	0	0	0	0	0

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	0.23	0.28	0.26	0.23	0.23
0.8a	0.00	0.43	0.49	0.47	0.45	0.43
0.7a	0.00	0.57	0.66	0.62	0.57	0.57
0.6a	0.00	0.64	0.74	0.72	0.66	0.66
0.5a	0.00	0.68	0.79	0.74	0.70	0.68
0.4a	0.00	0.64	0.74	0.72	0.66	0.66
0.3a	0.00	0.57	0.66	0.62	0.57	0.57
0.2a	0.00	0.43	0.49	0.47	0.45	0.43
0.1a	0.00	0.23	0.28	0.26	0.23	0.23
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Mux Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	0.43	0.66	0.79	0.85	0.00
0.8a	0.00	0.72	1.15	1.38	1.51	0.00
0.7a	0.00	0.89	1.47	1.79	1.96	0.00
0.6a	0.00	1.00	1.66	2.04	2.23	0.00
0.5a	0.00	1.02	1.72	2.13	2.32	0.00
0.4a	0.00	1.00	1.66	2.04	2.23	0.00
0.3a	0.00	0.89	1.47	1.79	1.96	0.00
0.2a	0.00	0.72	1.15	1.38	1.51	0.00
0.1a	0.00	0.43	0.66	0.79	0.85	0.00
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Muy,max =	**0.79 kip-ft**
Mux,max =	**2.32 kip-ft**

INLET STRUCTURE #1 - REINFORCEMENT DESIGN

Design Parameters:

$f_c := 3000psi$

$f_y := 60ksi$

$\beta_1 := 0.85$

$\Phi_b := 0.9$

$b := 12in$

$t_{wall} := 10in$ $\quad d_{wall} := t_{wall} - 2in - \frac{.75in}{2} = 7.625 \times in$

$t_{slab} := 11in$ $\quad d_{slab} := t_{slab} - 3in - \frac{.75in}{2} = 7.625 \times in$

Balanced Reinforcement ratio for rectangular sections with tension reinforcement only:

$$\rho_b := \left(\frac{0.85 \times \beta_1 \times f_c}{f_y}\right) \times \left(\frac{87ksi}{87ksi + f_y}\right) = 0.021$$

Maximum Reinforcement of Flexural Members: (AASHTO 8.16.3.1)

$\rho_{max} := 0.75 \times \rho_b = 0.016$

Minimum Reinforcement of Flexural Members: (AASHTO 8.17.1)

ΦMn >= 1.2Mcr unless Asprov > 4/3*Asreqd

$$M_{cr} = \frac{f_r \times I_g}{y_t}$$

$f_r = 7.5 \times \sqrt{f_c}$ $\quad f_r := 7.5 \times \sqrt{3000} = 410.792$ psi

$$I_{gwall} := \frac{12in \times t_{wall}^3}{12} = 0.048\,ft^4 \qquad I_{gslab} := \frac{12in \times t_{slab}^3}{12} = 0.064\,ft^4$$

$$y_{twall} := \frac{t_{wall}}{2} = 0.417\,ft \qquad y_{tslab} := \frac{t_{slab}}{2} = 0.458\,ft$$

$$M_{crwall} := \frac{f_r \times psi \times I_{gwall}}{y_{twall}} = 6.847 \times ft \times kip \quad \text{-----> } 1.2M_{crwall} = 8.216 \times ft \times kip$$

$$M_{crslab} := \frac{f_r \times psi \times I_{gslab}}{y_{tslab}} = 8.284 \times ft \times kip \quad \text{-----> } 1.2M_{crslab} = 9.941 \times ft \times kip$$

Temperature/Shrinkage Reinforcement (AASHTO 8.20.1)

$A_{s.TSreq} := 0.125in^2$ (Min area of steel per foot in each direction)

$S_{max1} := 3 \times t_{wall} = 30 \times in$ $\qquad S_{max2} := 18in$

$S_{max} := \min(S_{max1}, S_{max2}) = 18 \times in$

$A_{No4} := 0.2$ (Area of #4 bar)

$$A_{s.TS} := A_{No4} \times in^2 \times \frac{12in}{S_{max}} = 0.133 \times in^2 \quad > \quad A_{s.TSreq} = 0.125 \times in^2 \quad \text{OK}$$

Therefore use a minimum of #4 with a max spacing of 18" o.c.
- All layers, all directions

Vertical Reinforcement - Outside Face:

All Walls - Check worst case:

$M_{u.wall} := 8.01 \times kip \times ft$ (at bottom of wall)

Try #5 at 12" o.c.

$$A_{s.wall} := \frac{0.31in^2 \times 12}{12} = 0.31 \times in^2$$

$$\rho_{wall} := \frac{A_{s.wall}}{d_{wall} \times b} = 3.388 \times 10^{-3}$$

$check_{\rho wall} := if(\rho_{wall} < \rho_{max}, "OK", "NG") = "OK"$

$$\Phi M_{n.wall} := \Phi_b \times \left[A_{s.wall} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{wall} \times f_y}{f_c} \right) \right] = 10.204 \times kip \times ft$$

$check_{wall} := if(\Phi M_{n.wall} > M_{u.wall}, "OK", "NG") = "OK"$

$check2_{wall} := if(\Phi M_{n.wall} > 1.2M_{crwall}, "OK", "NG") = "OK"$

Therefore Use #5 at 12" o.c. outside face vertical rebar for all walls

#5 @ 9" Closed. Ok.

Vertical Reinforcement - Inside Face:

All Walls - Check worst case:

$M_{u.inwall} := 1.26 \times kip \times ft$

Try #4 at 12" o.c.

$A_{s.inwall} := \frac{0.2in^2 \times 12}{12} = 0.2 \times in^2$

$\rho_{inwall} := \frac{A_{s.inwall}}{d_{wall} \times b} = 2.186 \times 10^{-3}$

$check_{\rho inwall} := if(\rho_{inwall} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.inwall} := \Phi_b \times \left[A_{s.inwall} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{inwall} \times f_y}{f_c} \right) \right] = 6.682 \times kip \times ft$

$check_{inwall} := if(\Phi M_{n.inwall} > M_{u.inwall}, "OK", "NG") = "OK"$

$check2_{inwall} := if(\Phi M_{n.inwall} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.inwall}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.008$ -----> $\omega := 0.001$ (Design Aid)

$A_{s.inreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.005 \times in^2$ -----> $\frac{4}{3} \times A_{s.inreq} = 0.006 \times in^2$

$check3_{inwall} := if\left(A_{s.inwall} > \frac{4}{3} \times A_{s.inreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 12" o.c. inside face vertical rebar for all walls.

Horizontal Reinforcement - Outside Face:

All Walls - Check worst case:

$M_{u.hout} := 3.54 \times kip \times ft$

Try #4 at 12" o.c.

$A_{s.hout} := \frac{0.2in^2 \times 12}{12} = 0.2 \times in^2$

$\rho_{hout} := \frac{A_{s.hout}}{d_{wall} \times b} = 2.186 \times 10^{-3}$

$check_{\rho hout} := if(\rho_{hout} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.hout} := \Phi_b \times \left[A_{s.hout} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{hout} \times f_y}{f_c} \right) \right] = 6.682 \times kip \times ft$

$check_{hout} := if(\Phi M_{n.hout} > M_{u.hout}, "OK", "NG") = "OK"$

$check2_{hout} := if(\Phi M_{n.hout} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.hout}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0226$ -----> $\omega := 0.023$ (Design Aid)

$A_{s.houtreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.105 \times in^2$ -----> $\frac{4}{3} \times A_{s.houtreq} = 0.14 \times in^2$

$check3_{hout} := if\left(A_{s.hout} > \frac{4}{3} \times A_{s.houtreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 12" o.c. outside face horizontal rebar for all walls.

Horizontal Reinforcement - Inside Face:

All Walls - Check worst case:

$M_{u.hin} := 2.45 \times kip \times ft$

Try #4 at 18" o.c.

$A_{s.hin} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{hin} := \frac{A_{s.hin}}{d_{wall} \times b} = 1.457 \times 10^{-3}$

$check_{\rho hin} := if(\rho_{hin} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.hin} := \Phi_b \times \left[A_{s.hin} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{hin} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{hin} := if(\Phi M_{n.hin} > M_{u.hin}, "OK", "NG") = "OK"$

$check2_{hin} := if(\Phi M_{n.hin} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.hin}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0156$ -----> $\omega := 0.016$ (Design Aid)

$A_{s.hinreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.073 \times in^2$ -----> $\frac{4}{3} \times A_{s.hinreq} = 0.098 \times in^2$

$check3_{hin} := if\left(A_{s.hin} > \frac{4}{3} \times A_{s.hinreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. inside face horizontal rebar for all walls.

#4@ 12" Clear.

Slab Reinforcement - Top Face:

Check worst case moment in both directions:

$M_{u.slabt} := 2.32 \times kip \times ft$

Try #4 at 18" o.c.

$A_{s.slabt} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{slabt} := \frac{A_{s.slabt}}{d_{slab} \times b} = 1.457 \times 10^{-3}$

$check_{\rho slabt} := if(\rho_{slabt} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabt} := \Phi_b \times \left[A_{s.slabt} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabt} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{slabt} := if(\Phi M_{n.slabt} > M_{u.slabt}, "OK", "NG") = "OK"$

$check2_{slabt} := if(\Phi M_{n.slabt} > 1.2M_{crslab}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.slabt}}{\Phi_b \times f_c \times b \times d_{slab}^2} = 0.0148$ -----> $\omega := 0.015$ (Design Aid)

$A_{s.slabtreq} := \omega \times b \times d_{slab} \times \frac{f_c}{f_y} = 0.069 \times in^2$ -----> $\frac{4}{3} \times A_{s.slabtreq} = 0.091 \times in^2$

$check3_{slabt} := if\left(A_{s.slabt} > \frac{4}{3} \times A_{s.slabtreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. each way in top face of slab.

Slab Reinforcement - Bottom Face in Short Direction:

Check worst case moment from bottom of High Wall and Short Wall:

$M_{u.slabbs} := M_{u.wall} = 8.01\ kip \times ft$

Try #5 at 12" o.c.

$A_{s.slabbs} := \frac{0.31in^2 \times 12}{12} = 0.31 \times in^2$

$\rho_{slabbs} := \frac{A_{s.slabbs}}{d_{slab} \times b} = 3.388 \times 10^{-3}$

$check_{\rho slabbs} := if(\rho_{slabbs} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabbs} := \Phi_b \times \left[A_{s.slabbs} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbs} \times f_y}{f_c} \right) \right] = 10.204 \times kip \times ft$

$check_{slabbs} := if(\Phi M_{n.slabbs} > M_{u.slabbs}, "OK", "NG") = "OK"$

$check2_{slabbs} := if(\Phi M_{n.slabbs} > 1.2M_{crslab}, "OK", "NG") = "OK"$

Therefore Use #5 at 12" o.c. in short direction in bottom face of slab.

#5 @ 9" Closed.

Slab Reinforcement - Bottom Face in Long Direction:

Check worst case moment from bottom of Side Walls:

$M_{u.slabbl} := 1.94kip \times ft$

Try #4 at 18" o.c.

$A_{s.slabbl} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{slabbl} := \frac{A_{s.slabbl}}{d_{slab} \times b} = 1.457 \times 10^{-3}$

$check_{\rho slabbl} := if(\rho_{slabbl} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabbl} := \Phi_b \times \left[A_{s.slabbl} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbl} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{slabbl} := if(\Phi M_{n.slabbl} > M_{u.slabbl}, "OK", "NG") = "OK"$

$check2_{slabbl} := if(\Phi M_{n.slabbl} > 1.2M_{crslab}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.slabbl}}{\Phi_b \times f_c \times b \times d_{slab}^2} = 0.0124$ -----> $\omega := 0.013$ (Design Aid)

$A_{s.slabblreq} := \omega \times b \times d_{slab} \times \frac{f_c}{f_y} = 0.059 \times in^2$ -----> $\frac{4}{3} \times A_{s.slabblreq} = 0.079 \times in^2$

$check3_{slabbl} := if\left(A_{s.slabbl} > \frac{4}{3} \times A_{s.slabblreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. in long direction in bottom face of slab.

#4 @ 9" Clear.

2.2

INLET STRUCTURE #2

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

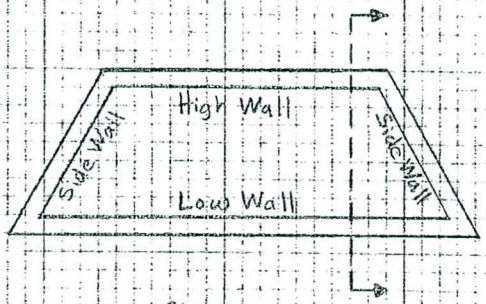
JOB NUMBER 200934

TITLE Oak St Basin

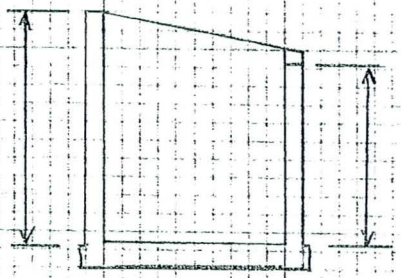
SUBJECT Inlet Structure #2

MADE BY JPH DATE 11-19-10 CHECKED BY Raj D DATE 12/26/10

Rectangular Concrete Tank Analysis



Plan



Section

High Wall

$a = 10.50'$

$b = 11.33'$ $\quad \frac{b}{a} = 1.0$ $\quad \rightarrow$ Use $\frac{b}{a} = 1.0$ $\quad \frac{c}{a} = 0.5$

$c = 7.12'$ $\quad \frac{c}{a} = 0.711$

$q = (60\ psf/ft)(10.5+8.5) + 90(8.50) = 875\ psf$

$q_u = \gamma(B_E q) = 1.3(1.15)(875\ psf) = 1308\ psf$

Short Wall

$a = 8.50'$

$b = 16.50'$ $\quad \frac{b}{a} = 2.16$ $\quad \rightarrow$ Use $\frac{b}{a} = 2.0$ $\quad \frac{c}{a} = 1.0$

$c = 7.12'$ $\quad \frac{c}{a} = 0.93$

$q = (90\ psf/ft)(7.63') = 687\ psf$

$q_u = \gamma(B_E q) = 1.3(1.15)(687\ psf) = 1027\ psf$

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2009034

TITLE Oak Street Basin & Storm Drain

SUBJECT Inlet Structure #2

MADE BY JPH DATE 11-24-10 CHECKED BY RMD DATE 12/26/10

Bottom Slab

Negative Moment for bottom slab will be equal to negative moments in the bottom of the wall.

Positive moments will be determined from a rectangular tank analysis of the slab assuming hinged connections to the walls

b = Avg width of slab = 13.92'

a = Length of Slab = 6.17'

b/a = 2.25 → use b/a = 2.5

$W_{shortwall} = (150)(7.63')(16.50')(10/12) = 15.74^k$

$W_{highwall} = (150)(9.63')(11.33')(10/12) = 13.64^k$

$W_{sidewalls} = (150)(8.63')(7.12')(10/12)(2) = 15.36^k$

44.7^k

$A_{slab} = b \cdot a = 85.9\ ft^2$

$q = \frac{W_{walls}}{A_{slab}} = 520\ psf$

$q_u = \gamma B_D q = (1.3)(1.0)(520) = 676\ psf$

- See Excell Spreadsheet for Moment Calculations

Inlet Structure #2:

Shear Design, Vu = Cs*qu*a

High Wall - b/a = 1.0, c/a = 0.5
(p 3-34)

qu =	1308	psf
a =	9.63	ft

Short Wall - b/a = 2.0, c/a = 1.0
(p 3-32)

qu =	1027	psf
a =	7.63	ft

Shear Coefficients, Cs, for b/a = 1.0	
Bottom Edge - Midpoint =	0.32
Side Edge - Maximum =	0.24
Side Edge - Midpoint =	0.23

Shear Coefficients for c/a = 0.5	
Bottom Edge - Midpoint =	0.19
Side Edge - Maximum =	0.17
Side Edge - Midpoint =	0.13

Max Bottom Shear Coefficient =	0.32
Max Side Shear Coefficient =	0.24

Bottom Shear Force =	4031	lb
Side Shear Force =	3023	lb

Shear Coefficients, Cs, for b/a = 2.0	
Bottom Edge - Midpoint =	0.45
Side Edge - Maximum =	0.27
Side Edge - Midpoint =	0.26

Shear Coefficients for c/a = 1.0	
Bottom Edge - Midpoint =	0.32
Side Edge - Maximum =	0.24
Side Edge - Midpoint =	0.23

Max Bottom Shear Coefficient =	0.45
Max Side Shear Coefficient =	0.27

Bottom Shear Force =	3526	lb
Side Shear Force =	2116	lb

Max Shear Force, Vu = 4031 lb

Check walls for Shear

f'c =	3000	psi
t =	10	in
d =	7.625	in
b =	12	in

Vc = 2*sqrt(f'c)*b*d

Φ = 0.85

ΦVc = 8520 lb > Vu = 4031 lb OK

Inlet Structure #2:

High Wall - b/a = 1.0, c/a = 0.5
(p 3-34)
qu = 1308 psf
a = 9.63 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-1	0	0	0	0	0
0.9a	-3	-1	0	1	1	1
0.8a	-3	-1	1	2	3	3
0.7a	-3	0	2	4	6	6
Long Side 0.6a	-4	0	4	7	8	9
0.5a	-4	1	5	8	10	11
0.4a	-4	1	5	9	11	11
0.3a	-4	1	5	7	9	9
0.2a	-3	0	1	2	2	2
0.1a	-1	-3	-7	-10	-13	-14
Bottom	0	-10	-23	-33	-39	-41

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.12	0.00	0.00	0.00	0.00	0.00
0.9a	-0.36	-0.12	0.00	0.12	0.12	0.12
0.8a	-0.36	-0.12	0.12	0.24	0.36	0.36
0.7a	-0.36	0.00	0.24	0.49	0.73	0.73
Long Side 0.6a	-0.49	0.00	0.49	0.85	0.97	1.09
0.5a	-0.49	0.12	0.61	0.97	1.21	1.33
0.4a	-0.49	0.12	0.61	1.09	1.33	1.33
0.3a	-0.49	0.12	0.61	0.85	1.09	1.09
0.2a	-0.36	0.00	0.12	0.24	0.24	0.24
0.1a	-0.12	-0.36	-0.85	-1.21	-1.58	-1.70
Bottom	0.00	-1.21	-2.79	-4.00	-4.73	-4.97

Vertical Bending Moments, Muz (in-kips)

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-1	0	0	0	0	0
0.9a	-3	-2	-2	-1	-1	-1
0.8a	-3	-3	-2	-2	-2	-2
0.7a	-3	-3	-2	-2	-2	-2
Short Side 0.6a	-4	-3	-3	-2	-2	-2
0.5a	-4	-3	-2	-2	-1	-1
0.4a	-4	-3	-2	-1	0	0
0.3a	-4	-2	-1	1	1	2
0.2a	-3	-1	1	2	3	3
0.1a	-1	1	2	2	3	3
Bottom	0	1	-1	-3	-5	-5

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.12	0.00	0.00	0.00	0.00	0.00
0.9a	-0.36	-0.24	-0.24	-0.12	-0.12	-0.12
0.8a	-0.36	-0.36	-0.24	-0.24	-0.24	-0.24
0.7a	-0.36	-0.36	-0.24	-0.24	-0.24	-0.24
Short Side 0.6a	-0.49	-0.36	-0.36	-0.24	-0.24	-0.24
0.5a	-0.49	-0.36	-0.24	-0.24	-0.12	-0.12
0.4a	-0.49	-0.36	-0.24	-0.12	0.00	0.00
0.3a	-0.49	-0.24	-0.12	0.12	0.12	0.24
0.2a	-0.36	-0.12	0.12	0.24	0.36	0.36
0.1a	-0.12	0.12	0.24	0.24	0.36	0.36
Bottom	0.00	0.12	-0.12	-0.36	-0.61	-0.61

Short Wall - b/a = 2.0, c/a = 1.0
(p 3-32)
qu = 1027 psf
a = 7.63 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-4	0	0	0	0	0
0.9a	-8	-2	1	3	5	5
0.8a	-8	-1	4	7	9	10
0.7a	-8	0	6	10	13	13
Long Side 0.6a	-8	1	8	12	14	15
0.5a	-8	1	8	12	13	14
0.4a	-7	1	6	8	8	8
0.3a	-6	-1	0	-2	-4	-5
0.2a	-4	-5	-11	-18	-23	-24
0.1a	-1	-13	-29	-43	-51	-54
Bottom	0	-27	-57	-79	-91	-95

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.49	0.00	0.00	0.00	0.00	0.00
0.9a	-0.97	-0.24	0.12	0.36	0.61	0.61
0.8a	-0.97	-0.12	0.49	0.85	1.09	1.21
0.7a	-0.97	0.00	0.73	1.21	1.58	1.58
Long Side 0.6a	-0.97	0.12	0.97	1.46	1.70	1.82
0.5a	-0.97	0.12	0.97	1.46	1.58	1.70
0.4a	-0.85	0.12	0.73	0.97	0.97	0.97
0.3a	-0.73	-0.12	0.00	-0.24	-0.49	-0.61
0.2a	-0.49	-0.61	-1.33	-2.18	-2.79	-2.91
0.1a	-0.12	-1.58	-3.52	-5.22	-6.19	-6.55
Bottom	0.00	-3.28	-6.91	-9.58	-11.04	-11.52

Vertical Bending Moments, Muz (in-kips)

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-4	0	0	0	0	0
0.9a	-8	-5	-3	-2	-1	-1
0.8a	-8	-6	-3	-2	-1	0
0.7a	-8	-5	-2	0	1	2
Short Side 0.6a	-8	-4	0	3	4	5
0.5a	-8	-3	2	5	7	8
0.4a	7	-1	4	8	10	11
0.3a	-6	0	5	9	11	11
0.2a	-4	1	5	7	8	8
0.1a	-1	1	1	-1	-2	-2
Bottom	0	-2	-10	-17	-22	-23

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.49	0.00	0.00	0.00	0.00	0.00
0.9a	-0.97	-0.61	-0.36	-0.24	-0.12	-0.12
0.8a	-0.97	-0.73	-0.36	-0.24	-0.12	0.00
0.7a	-0.97	-0.61	-0.24	0.00	0.12	0.24
Short Side 0.6a	-0.97	-0.49	0.00	0.36	0.49	0.61
0.5a	-0.97	-0.36	0.24	0.61	0.85	0.97
0.4a	0.85	-0.12	0.49	0.97	1.21	1.33
0.3a	-0.73	0.00	0.61	1.09	1.33	1.33
0.2a	-0.49	0.12	0.61	0.85	0.97	0.97
0.1a	-0.12	0.12	0.12	-0.12	-0.24	-0.24
Bottom	0.00	-0.24	-1.21	-2.06	-2.67	-2.79

Inlet Structure #2:

Maximum Positive Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	Maximum
Top	-0.12	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-0.36	-0.12	0.12	0.36	0.61	0.61	**0.61**
0.8a	-0.36	-0.12	0.49	0.85	1.09	1.21	**1.21**
0.7a	-0.36	0.00	0.73	1.21	1.58	1.58	**1.58**
Long Side 0.6a	-0.49	0.12	0.97	1.46	1.70	1.82	**1.82**
0.5a	-0.49	0.12	0.97	1.46	1.58	1.70	**1.70**
0.4a	-0.49	0.12	0.73	1.09	1.33	1.33	**1.33**
0.3a	-0.49	0.12	0.61	0.85	1.09	1.09	**1.09**
0.2a	-0.36	0.00	0.12	0.24	0.24	0.24	**0.24**
0.1a	-0.12	-0.36	-0.85	-1.21	-1.58	-1.70	**-0.12**
Bottom	0.00	-1.21	-2.79	-4.00	-4.73	-4.97	**0.00**

Maximum Positive Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	Maximum
Top	-0.12	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-0.36	-0.24	-0.24	-0.12	-0.12	-0.12	**-0.12**
0.8a	-0.36	-0.36	-0.24	-0.24	-0.12	0.00	**0.00**
0.7a	-0.36	-0.36	-0.24	0.00	0.12	0.24	**0.24**
Short Side 0.6a	-0.49	-0.36	0.00	0.36	0.49	0.61	**0.61**
0.5a	-0.49	-0.36	0.24	0.61	0.85	0.97	**0.97**
0.4a	0.85	-0.12	0.49	0.97	1.21	1.33	**1.33**
0.3a	-0.49	0.00	0.61	1.09	1.33	1.33	**1.33**
0.2a	-0.36	0.12	0.61	0.85	0.97	0.97	**0.97**
0.1a	-0.12	0.12	0.24	0.24	0.36	0.36	**0.36**
Bottom	0.00	0.12	-0.12	-0.36	-0.61	-0.61	**0.12**

Maximum Negative Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	Maximum
Top	-0.49	0.00	0.00	0.00	0.00	0.00	**-0.49**
0.9a	-0.97	-0.24	0.00	0.12	0.12	0.12	**-0.97**
0.8a	-0.97	-0.12	0.12	0.24	0.36	0.36	**-0.97**
0.7a	-0.97	0.00	0.24	0.49	0.73	0.73	**-0.97**
Long Side 0.6a	-0.97	0.00	0.49	0.85	0.97	1.09	**-0.97**
0.5a	-0.97	0.12	0.61	0.97	1.21	1.33	**-0.97**
0.4a	-0.85	0.12	0.61	0.97	0.97	0.97	**-0.85**
0.3a	-0.73	-0.12	0.00	-0.24	-0.49	-0.61	**-0.73**
0.2a	-0.49	-0.61	-1.33	-2.18	-2.79	-2.91	**-2.91**
0.1a	-0.12	-1.58	-3.52	-5.22	-6.19	-6.55	**-6.55**
Bottom	0.00	-3.28	-6.91	-9.58	-11.04	-11.52	**-11.52**

Maximum Negative Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	Maximum
Top	-0.49	0.00	0.00	0.00	0.00	0.00	**-0.49**
0.9a	-0.97	-0.61	-0.36	-0.24	-0.12	-0.12	**-0.97**
0.8a	-0.97	-0.73	-0.36	-0.24	-0.24	-0.24	**-0.97**
0.7a	-0.97	-0.61	-0.24	-0.24	-0.24	-0.24	**-0.97**
Short Side 0.6a	-0.97	-0.49	-0.36	-0.24	-0.24	-0.24	**-0.97**
0.5a	-0.97	-0.36	-0.24	-0.24	-0.12	-0.12	**-0.97**
0.4a	-0.49	-0.36	-0.24	-0.12	0.00	0.00	**-0.49**
0.3a	-0.73	-0.24	-0.12	0.12	0.12	0.24	**-0.73**
0.2a	-0.49	-0.12	0.12	0.24	0.36	0.36	**-0.49**
0.1a	-0.12	0.12	0.12	-0.12	-0.24	-0.24	**-0.24**
Bottom	0.00	-0.24	-1.21	-2.06	-2.67	-2.79	**-2.79**

Inlet Structure #2:

High Wall - b/a = 1.0, c/a = 0.5
(p 3-34)

qu = 1308 psf
a = 9.63 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-3	-4	3	9	13	14
0.9a	-13	-4	4	9	13	14
0.8a	-15	-4	4	10	14	15
0.7a	-17	-5	4	11	14	16
Long Side 0.6a	-19	-5	5	11	15	16
0.5a	-21	-5	5	11	14	15
0.4a	-21	-4	5	10	13	14
0.3a	-19	-4	4	8	10	10
0.2a	-14	2	3	4	5	5
0.1a	-6	-1	0	-1	-1	-1
Bottom	0	-2	-5	-7	-8	-8

My Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-3	-11	-10	-10	-9	-9
0.9a	-13	-11	-10	-9	-8	-8
0.8a	-15	-12	-10	-8	-7	-7
0.7a	-17	-13	-10	-7	-6	-6
Short Side 0.6a	-19	-14	-9	-6	-5	-4
0.5a	-21	-14	-8	-5	-3	-2
0.4a	-21	-13	-7	-3	-1	0
0.3a	-19	-10	-5	-1	1	2
0.2a	-14	-7	-2	1	2	3
0.1a	-6	-2	0	1	1	2
Bottom	0	0	0	-1	-1	-1

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.36	-0.49	0.36	1.09	1.58	1.70
0.9a	-1.58	-0.49	0.49	1.09	1.58	1.70
0.8a	-1.82	-0.49	0.49	1.21	1.70	1.82
0.7a	-2.06	-0.61	0.49	1.33	1.70	1.94
Long Side 0.6a	-2.30	-0.61	0.61	1.33	1.82	1.94
0.5a	-2.55	-0.61	0.61	1.33	1.70	1.82
0.4a	-2.55	-0.49	0.61	1.21	1.58	1.70
0.3a	-2.30	-0.49	0.49	0.97	1.21	1.21
0.2a	-1.70	0.24	0.36	0.49	0.61	0.61
0.1a	-0.73	-0.12	0.00	-0.12	-0.12	-0.12
Bottom	0.00	-0.24	-0.61	-0.85	-0.97	-0.97

Muy Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.36	-1.33	-1.21	-1.21	-1.09	-1.09
0.9a	-1.58	-1.33	-1.21	-1.09	-0.97	-0.97
0.8a	-1.82	-1.46	-1.21	-0.97	-0.85	-0.85
0.7a	-2.06	-1.58	-1.21	-0.85	-0.73	-0.73
Short Side 0.6a	-2.30	-1.70	-1.09	-0.73	-0.61	-0.49
0.5a	-2.55	-1.70	-0.97	-0.61	-0.36	-0.24
0.4a	-2.55	-1.58	-0.85	-0.36	-0.12	0.00
0.3a	-2.30	-1.21	-0.61	-0.12	0.12	0.24
0.2a	-1.70	-0.85	-0.24	0.12	0.24	0.36
0.1a	-0.73	-0.24	0.00	0.12	0.12	0.24
Bottom	0.00	0.00	0.00	-0.12	-0.12	-0.12

Short Wall - b/a = 2.0, c/a = 1.0
(p 3-32)

qu = 1027 psf
a = 7.63 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-22	-13	8	20	27	29
0.9a	-42	-12	8	19	25	27
0.8a	-41	-11	8	19	24	25
0.7a	-41	-10	8	18	22	23
Long Side 0.6a	-41	-8	8	16	19	20
0.5a	-39	-6	8	14	16	16
0.4a	-35	-4	7	10	11	11
0.3a	-28	-3	4	6	5	5
0.2a	-18	-2	0	0	-1	-2
0.1a	-6	-3	-5	-8	-9	-10
Bottom	0	-5	-11	-16	-18	-19

My Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-22	-29	-18	-9	-4	-2
0.9a	-42	-27	-16	-7	-2	0
0.8a	-41	-26	-14	-5	0	2
0.7a	-41	-24	-11	-3	2	4
Short Side 0.6a	-41	-22	-9	0	5	6
0.5a	-39	-19	-6	2	7	8
0.4a	-35	-15	-3	4	7	9
0.3a	-28	-11	-1	4	7	8
0.2a	-18	-6	0	3	5	5
0.1a	-6	-2	0	1	1	1
Bottom	0	0	-2	-3	-4	-5

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-2.67	-1.58	0.97	2.43	3.28	3.52
0.9a	-5.09	-1.46	0.97	2.30	3.03	3.28
0.8a	-4.97	-1.33	0.97	2.30	2.91	3.03
0.7a	-4.97	-1.21	0.97	2.18	2.67	2.79
Long Side 0.6a	-4.97	-0.97	0.97	1.94	2.30	2.43
0.5a	-4.73	-0.73	0.97	1.70	1.94	1.94
0.4a	-4.25	-0.49	0.85	1.21	1.33	1.33
0.3a	-3.40	-0.36	0.49	0.73	0.61	0.61
0.2a	-2.18	-0.24	0.00	0.00	-0.12	-0.24
0.1a	-0.73	-0.36	-0.61	-0.97	-1.09	-1.21
Bottom	0.00	-0.61	-1.33	-1.94	-2.18	-2.30

Muy Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-2.67	-3.52	-2.18	-1.09	-0.49	-0.24
0.9a	-5.09	-3.28	-1.94	-0.85	-0.24	0.00
0.8a	-4.97	-3.15	-1.70	-0.61	0.00	0.24
0.7a	-4.97	-2.91	-1.33	-0.36	0.24	0.49
Short Side 0.6a	-4.97	-2.67	-1.09	0.00	0.61	0.73
0.5a	-4.73	-2.30	-0.73	0.24	0.85	0.97
0.4a	-4.25	-1.82	-0.36	0.49	0.85	1.09
0.3a	-3.40	-1.33	-0.12	0.49	0.85	0.97
0.2a	-2.18	-0.73	0.00	0.36	0.61	0.61
0.1a	-0.73	-0.24	0.00	0.12	0.12	0.12
Bottom	0.00	0.00	-0.24	-0.36	-0.49	-0.61

Inlet Structure #2:

Maximum Positive Horizontal Bending Moments

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.36	-0.49	0.97	2.43	3.28	3.52
0.9a	-1.58	-0.49	0.97	2.30	3.03	3.28
0.8a	-1.82	-0.49	0.97	2.30	2.91	3.03
0.7a	-2.06	-0.61	0.97	2.18	2.67	2.79
Long Side 0.6a	-2.30	-0.61	0.97	1.94	2.30	2.43
0.5a	-2.55	-0.61	0.97	1.70	1.94	1.94
0.4a	-2.55	-0.49	0.85	1.21	1.58	1.70
0.3a	-2.30	-0.36	0.49	0.97	1.21	1.21
0.2a	-1.70	0.24	0.36	0.49	0.61	0.61
0.1a	-0.73	-0.12	0.00	-0.12	-0.12	-0.12
Bottom	0.00	-0.24	-0.61	-0.85	-0.97	-0.97
Maximum	**0.00**	**0.24**	**0.97**	**2.43**	**3.28**	**3.52**

Muy Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.36	-1.33	-1.21	-1.09	-0.49	-0.24
0.9a	-1.58	-1.33	-1.21	-0.85	-0.24	0.00
0.8a	-1.82	-1.46	-1.21	-0.61	0.00	0.24
0.7a	-2.06	-1.58	-1.21	-0.36	0.24	0.49
Short Side 0.6a	-2.30	-1.70	-1.09	0.00	0.61	0.73
0.5a	-2.55	-1.70	-0.73	0.24	0.85	0.97
0.4a	-2.55	-1.58	-0.36	0.49	0.85	1.09
0.3a	-2.30	-1.21	-0.12	0.49	0.85	0.97
0.2a	-1.70	-0.73	0.00	0.36	0.61	0.61
0.1a	-0.73	-0.24	0.00	0.12	0.12	0.24
Bottom	0.00	0.00	0.00	-0.12	-0.12	-0.12
Maximum	**0.00**	**0.00**	**0.00**	**0.49**	**0.85**	**1.09**

Maximum Negative Horizontal Bending Moments

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-2.67	-1.58	0.36	1.09	1.58	1.70
0.9a	-5.09	-1.46	0.49	1.09	1.58	1.70
0.8a	-4.97	-1.33	0.49	1.21	1.70	1.82
0.7a	-4.97	-1.21	0.49	1.33	1.70	1.94
Long Side 0.6a	-4.97	-0.97	0.61	1.33	1.82	1.94
0.5a	-4.73	-0.73	0.61	1.33	1.70	1.82
0.4a	-4.25	-0.49	0.61	1.21	1.33	1.33
0.3a	-3.40	-0.49	0.49	0.73	0.61	0.61
0.2a	-2.18	-0.24	0.00	0.00	-0.12	-0.24
0.1a	-0.73	-0.36	-0.61	-0.97	-1.09	-1.21
Bottom	0.00	-0.61	-1.33	-1.94	-2.18	-2.30
Maximum	**-5.09**	**-1.58**	**-1.33**	**-1.94**	**-2.18**	**-2.30**

Muy Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-2.67	-3.52	-2.18	-1.21	-1.09	-1.09
0.9a	-5.09	-3.28	-1.94	-1.09	-0.97	-0.97
0.8a	-4.97	-3.15	-1.70	-0.97	-0.85	-0.85
0.7a	-4.97	-2.91	-1.33	-0.85	-0.73	-0.73
Short Side 0.6a	-4.97	-2.67	-1.09	-0.73	-0.61	-0.49
0.5a	-4.73	-2.30	-0.97	-0.61	-0.36	-0.24
0.4a	-4.25	-1.82	-0.85	-0.36	-0.12	0.00
0.3a	-3.40	-1.33	-0.61	-0.12	0.12	0.24
0.2a	-2.18	-0.85	-0.24	0.12	0.24	0.36
0.1a	-0.73	-0.24	0.00	0.12	0.12	0.12
Bottom	0.00	0.00	-0.24	-0.36	-0.49	-0.61
Maximum	**-5.09**	**-3.52**	**-2.18**	**-1.21**	**-1.09**	**-1.09**

Inlet Structure #2:

Bottom Slab Positive Moments (tension on top face):

b = 13.92 ft
a = 6.17 ft
b/a = 2.2560778 -----> 2.5 (p 2-61)

qu = 676 psf

My Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0	0	0	0	0	0
0.9a	0	11	13	12	11	11
0.8a	0	20	23	22	21	20
0.7a	0	27	31	29	27	27
0.6a	0	30	35	34	31	31
0.5a	0	32	37	35	33	32
0.4a	0	30	35	34	31	31
0.3a	0	27	31	29	27	27
0.2a	0	20	23	22	21	20
0.1a	0	11	13	12	11	11
Bottom	0	0	0	0	0	0

Mx Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0	0	0	0	0	0
0.9a	0	20	31	37	40	41
0.8a	0	34	54	65	71	72
0.7a	0	42	69	84	92	94
0.6a	0	47	78	96	105	108
0.5a	0	48	81	100	109	112
0.4a	0	47	78	96	105	108
0.3a	0	42	69	84	92	94
0.2a	0	34	54	65	71	72
0.1a	0	20	31	37	40	41
Bottom	0	0	0	0	0	0

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	0.28	0.33	0.31	0.28	0.28
0.8a	0.00	0.51	0.59	0.57	0.54	0.51
0.7a	0.00	0.69	0.80	0.75	0.69	0.69
0.6a	0.00	0.77	0.90	0.87	0.80	0.80
0.5a	0.00	0.82	0.95	0.90	0.85	0.82
0.4a	0.00	0.77	0.90	0.87	0.80	0.80
0.3a	0.00	0.69	0.80	0.75	0.69	0.69
0.2a	0.00	0.51	0.59	0.57	0.54	0.51
0.1a	0.00	0.28	0.33	0.31	0.28	0.28
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Mux Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	0.51	0.80	0.95	1.03	0.00
0.8a	0.00	0.87	1.39	1.67	1.83	0.00
0.7a	0.00	1.08	1.78	2.16	2.37	0.00
0.6a	0.00	1.21	2.01	2.47	2.70	0.00
0.5a	0.00	1.24	2.08	2.57	2.81	0.00
0.4a	0.00	1.21	2.01	2.47	2.70	0.00
0.3a	0.00	1.08	1.78	2.16	2.37	0.00
0.2a	0.00	0.87	1.39	1.67	1.83	0.00
0.1a	0.00	0.51	0.80	0.95	1.03	0.00
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Muy,max =	**0.95 kip-ft**
Mux,max =	**2.81 kip-ft**

INLET STRUCTURE #2 - REINFORCEMENT DESIGN

Design Parameters:

$f_c := 3000psi$

$f_y := 60ksi$

$\beta_1 := 0.85$

$\Phi_b := 0.9$

$b := 12in$

$t_{wall} := 10in$ $\quad d_{wall} := t_{wall} - 2in - \frac{.75in}{2} = 7.625 \times in$

$t_{slab} := 11in$ $\quad d_{slab} := t_{slab} - 3in - \frac{.75in}{2} = 7.625 \times in$

Balanced Reinforcement ratio for rectangular sections with tension reinforcement only:

$$\rho_b := \left(\frac{0.85 \times \beta_1 \times f_c}{f_y}\right) \times \left(\frac{87ksi}{87ksi + f_y}\right) = 0.021$$

Maximum Reinforcement of Flexural Members: (AASHTO 8.16.3.1)

$\rho_{max} := 0.75 \times \rho_b = 0.016$

Minimum Reinforcement of Flexural Members: (AASHTO 8.17.1)

ΦMn >= 1.2Mcr unless Asprov > 4/3*Asreqd

$$M_{cr} = \frac{f_r \times I_g}{y_t}$$

$f_r = 7.5 \times \sqrt{f_c}$ $\quad f_r := 7.5 \times \sqrt{3000} = 410.792$ psi

$$I_{gwall} := \frac{12in \times t_{wall}^3}{12} = 0.048\,ft^4 \qquad I_{gslab} := \frac{12in \times t_{slab}^3}{12} = 0.064\,ft^4$$

$$y_{twall} := \frac{t_{wall}}{2} = 0.417\,ft \qquad y_{tslab} := \frac{t_{slab}}{2} = 0.458\,ft$$

$$M_{crwall} := \frac{f_r \times psi \times I_{gwall}}{y_{twall}} = 6.847 \times ft \times kip \quad \text{-----> } 1.2M_{crwall} = 8.216 \times ft \times kip$$

$$M_{crslab} := \frac{f_r \times psi \times I_{gslab}}{y_{tslab}} = 8.284 \times ft \times kip \quad \text{-----> } 1.2M_{crslab} = 9.941 \times ft \times kip$$

Temperature/Shrinkage Reinforcement (AASHTO 8.20.1)

$A_{s.TSreq} := 0.125in^2$ (Min area of steel per foot in each direction)

$S_{max1} := 3 \times t_{wall} = 30 \times in$ $\qquad S_{max2} := 18in$

$S_{max} := \min(S_{max1}, S_{max2}) = 18 \times in$

$A_{No4} := 0.2$ (Area of #4 bar)

$$A_{s.TS} := A_{No4} \times in^2 \times \frac{12in}{S_{max}} = 0.133 \times in^2 \quad > \quad A_{s.TSreq} = 0.125 \times in^2 \quad \textbf{OK}$$

Therefore use a minimum of #4 with a max spacing of 18" o.c.
- All layers, all directions

Vertical Reinforcement - Outside Face:

All Walls - Check worst case:

$M_{u.wall} := 11.52 \times kip \times ft$

Try #5 at 9" o.c.

$A_{s.wall} := \frac{0.31in^2 \times 12}{9} = 0.413 \times in^2$

$\rho_{wall} := \frac{A_{s.wall}}{d_{wall} \times b} = 4.517 \times 10^{-3}$

$check_{\rho wall} := if(\rho_{wall} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.wall} := \Phi_b \times \left[A_{s.wall} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{wall} \times f_y}{f_c} \right) \right] = 13.414 \times kip \times ft$

$check_{wall} := if(\Phi M_{n.wall} > M_{u.wall}, "OK", "NG") = "OK"$

$check2_{wall} := if(\Phi M_{n.wall} > 1.2M_{crwall}, "OK", "NG") = "OK"$

Therefore Use #5 at 9" o.c. outside face vertical rebar for all walls.

Vertical Reinforcement - Inside Face:

All Walls - Check worst case:

$M_{u.inwall} := 1.82 \times kip \times ft$

Try #4 at 18" o.c.

$$A_{s.inwall} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$$

$$\rho_{inwall} := \frac{A_{s.inwall}}{d_{wall} \times b} = 1.457 \times 10^{-3}$$

$check_{\rho inwall} := if(\rho_{inwall} < \rho_{max}, "OK", "NG") = "OK"$

$$\Phi M_{n.inwall} := \Phi_b \times \left[A_{s.inwall} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{inwall} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$$

$check_{inwall} := if(\Phi M_{n.inwall} > M_{u.inwall}, "OK", "NG") = "OK"$

$check2_{inwall} := if(\Phi M_{n.inwall} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$$\frac{M_{u.inwall}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0116 \quad \text{-----> } \omega := 0.012 \quad \text{(Design Aid)}$$

$$A_{s.inreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.055 \times in^2 \quad \text{-----> } \frac{4}{3} \times A_{s.inreq} = 0.073 \times in^2$$

$check3_{inwall} := if\left(A_{s.inwall} > \frac{4}{3} \times A_{s.inreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. inside face vertical rebar for all walls.

#4 @ 18" Clsd.

Horizontal Reinforcement - Outside Face:

All Walls - Check worst case:

$M_{u.hout} := 5.09 \times kip \times ft$

Try #4 at 12" o.c.

$A_{s.hout} := \frac{0.2in^2 \times 12}{12} = 0.2 \times in^2$

$\rho_{hout} := \frac{A_{s.hout}}{d_{wall} \times b} = 2.186 \times 10^{-3}$

$check_{\rho hout} := if(\rho_{hout} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.hout} := \Phi_b \times \left[A_{s.hout} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{hout} \times f_y}{f_c} \right) \right] = 6.682 \times kip \times ft$

$check_{hout} := if(\Phi M_{n.hout} > M_{u.hout}, "OK", "NG") = "OK"$

$check2_{hout} := if(\Phi M_{n.hout} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.hout}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0324$ -----> $\omega := 0.033$ (Design Aid)

$A_{s.houtreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.151 \times in^2$ -----> $\frac{4}{3} \times A_{s.houtreq} = 0.201 \times in^2$

$check3_{hout} := if\left(A_{s.hout} > \frac{4}{3} \times A_{s.houtreq}, "OK", "NG" \right) = "NG"$

As is close enough to 4/3 As,req ------> OK

Therefore Use #4 at 12" o.c. outside face horizontal rebar for all walls.

Horizontal Reinforcement - Inside Face:

All Walls - Check worst case:

$M_{u.hin} := 3.52 \times kip \times ft$

Try #4 at 12" o.c.

$A_{s.hin} := \frac{0.2in^2 \times 12}{12} = 0.2 \times in^2$

$\rho_{hin} := \frac{A_{s.hin}}{d_{wall} \times b} = 2.186 \times 10^{-3}$

$check_{\rho hin} := if(\rho_{hin} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.hin} := \Phi_b \times \left[A_{s.hin} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{hin} \times f_y}{f_c} \right) \right] = 6.682 \times kip \times ft$

$check_{hin} := if(\Phi M_{n.hin} > M_{u.hin}, "OK", "NG") = "OK"$

$check2_{hin} := if(\Phi M_{n.hin} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.hin}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0224$ -----> $\omega := 0.023$ (Design Aid)

$A_{s.hinreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.105 \times in^2$ -----> $\frac{4}{3} \times A_{s.hinreq} = 0.14 \times in^2$

$check3_{hin} := if\left(A_{s.hin} > \frac{4}{3} \times A_{s.hinreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 12" o.c. inside face horizontal rebar for all walls.

Slab Reinforcement - Top Face:

Check worst case moment in both directions:

$M_{u.slabt} := 2.81 \times kip \times ft$

Try #4 at 18" o.c.

$A_{s.slabt} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{slabt} := \frac{A_{s.slabt}}{d_{slab} \times b} = 1.457 \times 10^{-3}$

$check_{\rho slabt} := if(\rho_{slabt} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabt} := \Phi_b \times \left[A_{s.slabt} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabt} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{slabt} := if(\Phi M_{n.slabt} > M_{u.slabt}, "OK", "NG") = "OK"$

$check2_{slabt} := if(\Phi M_{n.slabt} > 1.2M_{crslab}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.slabt}}{\Phi_b \times f_c \times b \times d_{slab}^2} = 0.0179$ -----> $\omega := 0.019$ (Design Aid)

$A_{s.slabtreq} := \omega \times b \times d_{slab} \times \frac{f_c}{f_y} = 0.087 \times in^2$ -----> $\frac{4}{3} \times A_{s.slabtreq} = 0.116 \times in^2$

$check3_{slabt} := if\left(A_{s.slabt} > \frac{4}{3} \times A_{s.slabtreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. each way in top face of slab.

Slab Reinforcement - Bottom Face in Short Direction:

Check worst case moment from bottom of High Wall and Short Wall:

$$M_{u.slabbs} := M_{u.wall} = 11.52 \times kip \times ft$$

Try #5 at 9" o.c.

$$A_{s.slabbs} := \frac{0.31in^2 \times 12}{9} = 0.413 \times in^2$$

$$\rho_{slabbs} := \frac{A_{s.slabbs}}{d_{slab} \times b} = 4.517 \times 10^{-3}$$

$$check_{\rho slabbs} := if(\rho_{slabbs} < \rho_{max}, "OK", "NG") = "OK"$$

$$\Phi M_{n.slabbs} := \Phi_b \times \left[A_{s.slabbs} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbs} \times f_y}{f_c} \right) \right] = 13.414 \times kip \times ft$$

$$check_{slabbs} := if(\Phi M_{n.slabbs} > M_{u.slabbs}, "OK", "NG") = "OK"$$

$$check2_{slabbs} := if(\Phi M_{n.slabbs} > 1.2M_{crslab}, "OK", "NG") = "OK"$$

Therefore Use #5 at 9" o.c. in short direction in bottom face of slab.

Slab Reinforcement - Bottom Face in Long Direction:

Check worst case moment from bottom of Side Walls:

$M_{u.slabbl} := 2.79kip \times ft$

Try #4 at 18" o.c.

$A_{s.slabbl} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{slabbl} := \frac{A_{s.slabbl}}{d_{slab} \times b} = 1.457 \times 10^{-3}$

$check_{\rho slabbl} := if(\rho_{slabbl} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabbl} := \Phi_b \times \left[A_{s.slabbl} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbl} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{slabbl} := if(\Phi M_{n.slabbl} > M_{u.slabbl}, "OK", "NG") = "OK"$

$check2_{slabbl} := if(\Phi M_{n.slabbl} > 1.2M_{crslab}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.slabbl}}{\Phi_b \times f_c \times b \times d_{slab}^2} = 0.0178$ -----> $\omega := 0.018$ (Design Aid)

$A_{s.slabblreq} := \omega \times b \times d_{slab} \times \frac{f_c}{f_y} = 0.082 \times in^2$ -----> $\frac{4}{3} \times A_{s.slabblreq} = 0.11 \times in^2$

$check3_{slabbl} := if\left(A_{s.slabbl} > \frac{4}{3} \times A_{s.slabblreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. in long direction in bottom face of slab.

#4 @ 9" Used

2.3

INLET STRUCTURE #3

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

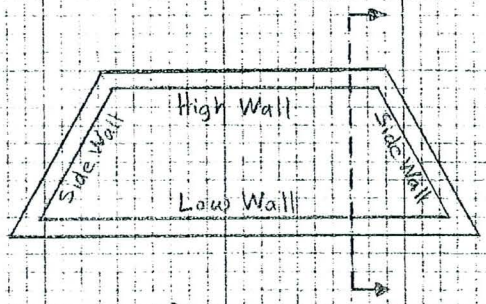
JOB NUMBER 2009034

TITLE Oak St Basin

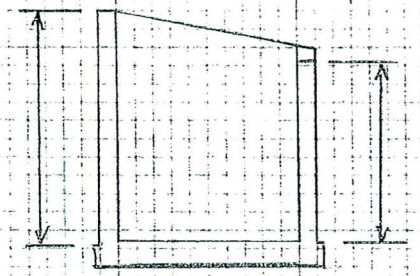
SUBJECT Inlet Structure #3

MADE BY JPH DATE 11-19-10 CHECKED BY RAJ D DATE 12/26/10

Rectangular Concrete Tank Analysis



Plan



Section

High Wall

$a = 6.84'$
$b = 7.67'$
$c = 5.97'$

$\frac{b}{a} = 1.12$ → Use $\frac{b}{a} = 1.0$ $\quad \frac{c}{a} = 0.87$ → Use $\frac{c}{a} = 0.5$

$q = (60\ psf/ft)(6.84 - 5.67') + (90\ psf/ft)(5.67') = 632\ psf$

$q_u = \gamma(P_E q) = 1.3(1.15)(632\ psf) = 945\ psf$

Short Wall

$a = 5.67'$
$b = 11.67'$
$c = 5.97'$

$\frac{b}{a} = 2.05$ → Use $\frac{b}{a} = 2.0$ $\quad \frac{c}{a} = 1.05$ → Use $\frac{c}{a} = 1.0$

$q = (90\ psf/ft)(5.67') = 510\ psf$

$q_u = \gamma(P_E q) = 1.3(1.15)(510\ psf) = 762\ psf$

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2009024

TITLE Oak Street Basin & Storm Drain

SUBJECT Inlet Structure #3

MADE BY JPH DATE 11-24-10 CHECKED BY Raj D DATE 12/26/10

Bottom Slab

Negative Moment for bottom slab will be equal to negative moments in the bottom of the wall.

Positive moments will be determined from a rectangular tank analysis of the slab assuming hinged connections to the walls

b = Avg width of slab = 9.67'

a = Length of slab = 5.17'

b/a = 1.87 → use b/a = 2.0

$W_{shortwall} = (150)(5.67')(11.67')(10/12) = 8.27^K$

$W_{highwall} = (150)(6.84')(7.67')(10/12) = 6.56^K$

$W_{sidewalls} = (150)(6.25')(5.97')(10/12)(2) = 9.38^K$

24.2^K

$A_{slab} = b \cdot a = 50.0\ ft^2$

$q = \frac{W_{walls}}{A_{slab}} = 484\ psf$

$q_u = \gamma B_D q = (1.3)(1.0)(484) = 629\ psf$

– See Excell Spreadsheet for Moment Calculations

Inlet Structure #3:

Shear Design, Vu = Cs*qu*a

High Wall - b/a = 1.0, c/a = 0.5

qu =	945	psf
a =	6.84	ft

Shear Coefficients, Cs, for b/a = 1.0	
Bottom Edge - Midpoint =	0.32
Side Edge - Maximum =	0.24
Side Edge - Midpoint =	0.23

Shear Coefficients for c/a = 0.5	
Bottom Edge - Midpoint =	0.19
Side Edge - Maximum =	0.17
Side Edge - Midpoint =	0.13

Max Bottom Shear Coefficient =	0.32
Max Side Shear Coefficient =	0.24

Bottom Shear Force =	2068	lb
Side Shear Force =	1551	lb

Short Wall - b/a = 2.0, c/a = 1.0

qu =	762	psf
a =	5.67	ft

Shear Coefficients, Cs, for b/a = 2.0	
Bottom Edge - Midpoint =	0.45
Side Edge - Maximum =	0.27
Side Edge - Midpoint =	0.26

Shear Coefficients for c/a = 1.0	
Bottom Edge - Midpoint =	0.32
Side Edge - Maximum =	0.24
Side Edge - Midpoint =	0.23

Max Bottom Shear Coefficient =	0.45
Max Side Shear Coefficient =	0.27

Bottom Shear Force =	1944	lb
Side Shear Force =	1167	lb

Max Shear Force, Vu = 2068 lb

Check walls for Shear

f'c =	3000	psi
t =	10	in
d =	7.625	in
b =	12	in

Vc = 2*sqrt(f'c)*b*d

Φ = 0.85

ΦVc = 8520 lb > Vu = 2068 lb OK

Inlet Structure #3:

High Wall - b/a = 1.0, c/a = 0.5
(p 3-34)
qu = 945 psf
a = 6.84 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-1	0	0	0	0	0
0.9a	-3	-1	0	1	1	1
0.8a	-3	-1	1	2	3	3
0.7a	-3	0	2	4	6	6
Long Side 0.6a	-4	0	4	7	8	9
0.5a	-4	1	5	8	10	11
0.4a	-4	1	5	9	11	11
0.3a	-4	1	5	7	9	9
0.2a	-3	0	1	2	2	2
0.1a	-1	-3	-7	-10	-13	-14
Bottom	0	-10	-23	-33	-39	-41

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.04	0.00	0.00	0.00	0.00	0.00
0.9a	-0.13	-0.04	0.00	0.04	0.04	0.04
0.8a	-0.13	-0.04	0.04	0.09	0.13	0.13
0.7a	-0.13	0.00	0.09	0.18	0.27	0.27
Long Side 0.6a	-0.18	0.00	0.18	0.31	0.35	0.40
0.5a	-0.18	0.04	0.22	0.35	0.44	0.49
0.4a	-0.18	0.04	0.22	0.40	0.49	0.49
0.3a	-0.18	0.04	0.22	0.31	0.40	0.40
0.2a	-0.13	0.00	0.04	0.09	0.09	0.09
0.1a	-0.04	-0.13	-0.31	-0.44	-0.57	-0.62
Bottom	0.00	-0.44	-1.02	-1.46	-1.72	-1.81

Vertical Bending Moments, Muz (in-kips)

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-1	0	0	0	0	0
0.9a	-3	-2	-2	-1	-1	-1
0.8a	-3	-3	-2	-2	-2	-2
0.7a	-3	-3	-2	-2	-2	-2
Short Side 0.6a	-4	-3	-3	-2	-2	-2
0.5a	-4	-3	-2	-2	-1	-1
0.4a	-4	-3	-2	-1	0	0
0.3a	-4	-2	-1	1	1	2
0.2a	-3	-1	1	2	3	3
0.1a	-1	1	2	2	3	3
Bottom	0	1	-1	-3	-5	-5

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.04	0.00	0.00	0.00	0.00	0.00
0.9a	-0.13	-0.09	-0.09	-0.04	-0.04	-0.04
0.8a	-0.13	-0.13	-0.09	-0.09	-0.09	-0.09
0.7a	-0.13	-0.13	-0.09	-0.09	-0.09	-0.09
Short Side 0.6a	-0.18	-0.13	-0.13	-0.09	-0.09	-0.09
0.5a	-0.18	-0.13	-0.09	-0.09	-0.04	-0.04
0.4a	-0.18	-0.13	-0.09	-0.04	0.00	0.00
0.3a	-0.18	-0.09	-0.04	0.04	0.04	0.09
0.2a	-0.13	-0.04	0.04	0.09	0.13	0.13
0.1a	-0.04	0.04	0.09	0.09	0.13	0.13
Bottom	0.00	0.04	-0.04	-0.13	-0.22	-0.22

Short Wall - b/a = 2.0, c/a = 1.0
(p 3-32)
qu = 762 psf
a = 5.67 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-4	0	0	0	0	0
0.9a	-8	-2	1	3	5	5
0.8a	-8	-1	4	7	9	10
0.7a	-8	0	6	10	13	13
Long Side 0.6a	-8	1	8	12	14	15
0.5a	-8	1	8	12	13	14
0.4a	-7	1	6	8	8	8
0.3a	-6	-1	0	-2	-4	-5
0.2a	-4	-5	-11	-18	-23	-24
0.1a	-1	-13	-29	-43	-51	-54
Bottom	0	-27	-57	-79	-91	-95

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.18	0.00	0.00	0.00	0.00	0.00
0.9a	-0.35	-0.09	0.04	0.13	0.22	0.22
0.8a	-0.35	-0.04	0.18	0.31	0.40	0.44
0.7a	-0.35	0.00	0.27	0.44	0.57	0.57
Long Side 0.6a	-0.35	0.04	0.35	0.53	0.62	0.66
0.5a	-0.35	0.04	0.35	0.53	0.57	0.62
0.4a	-0.31	0.04	0.27	0.35	0.35	0.35
0.3a	-0.27	-0.04	0.00	-0.09	-0.18	-0.22
0.2a	-0.18	-0.22	-0.49	-0.80	-1.02	-1.06
0.1a	-0.04	-0.57	-1.28	-1.90	-2.25	-2.39
Bottom	0.00	-1.19	-2.52	-3.49	-4.02	-4.20

Vertical Bending Moments, Muz (in-kips)

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-4	0	0	0	0	0
0.9a	-8	-5	-3	-2	-1	-1
0.8a	-8	-6	-3	-2	-1	0
0.7a	-8	-5	-2	0	1	2
Short Side 0.6a	-8	-4	0	3	4	5
0.5a	-8	-3	2	5	7	8
0.4a	7	-1	4	8	10	11
0.3a	-6	0	5	9	11	11
0.2a	-4	1	5	7	8	8
0.1a	-1	1	1	-1	-2	-2
Bottom	0	-2	-10	-17	-22	-23

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.18	0.00	0.00	0.00	0.00	0.00
0.9a	-0.35	-0.22	-0.13	-0.09	-0.04	-0.04
0.8a	-0.35	-0.27	-0.13	-0.09	-0.04	0.00
0.7a	-0.35	-0.22	-0.09	0.00	0.04	0.09
Short Side 0.6a	-0.35	-0.18	0.00	0.13	0.18	0.22
0.5a	-0.35	-0.13	0.09	0.22	0.31	0.35
0.4a	0.31	-0.04	0.18	0.35	0.44	0.49
0.3a	-0.27	0.00	0.22	0.40	0.49	0.49
0.2a	-0.18	0.04	0.22	0.31	0.35	0.35
0.1a	-0.04	0.04	0.04	-0.04	-0.09	-0.09
Bottom	0.00	-0.09	-0.44	-0.75	-0.97	-1.02

Inlet Structure #3:

Maximum Positive Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	Maximum
Top	-0.04	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-0.13	-0.04	0.04	0.13	0.22	0.22	**0.22**
0.8a	-0.13	-0.04	0.18	0.31	0.40	0.44	**0.44**
0.7a	-0.13	0.00	0.27	0.44	0.57	0.57	**0.57**
Long Side 0.6a	-0.18	0.04	0.35	0.53	0.62	0.66	**0.66**
0.5a	-0.18	0.04	0.35	0.53	0.57	0.62	**0.62**
0.4a	-0.18	0.04	0.27	0.40	0.49	0.49	**0.49**
0.3a	-0.18	0.04	0.22	0.31	0.40	0.40	**0.40**
0.2a	-0.13	0.00	0.04	0.09	0.09	0.09	**0.09**
0.1a	-0.04	-0.13	-0.31	-0.44	-0.57	-0.62	**-0.04**
Bottom	0.00	-0.44	-1.02	-1.46	-1.72	-1.81	**0.00**

Maximum Positive Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	Maximum
Top	-0.04	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-0.13	-0.09	-0.09	-0.04	-0.04	-0.04	**-0.04**
0.8a	-0.13	-0.13	-0.09	-0.09	-0.04	0.00	**0.00**
0.7a	-0.13	-0.13	-0.09	0.00	0.04	0.09	**0.09**
Short Side 0.6a	-0.18	-0.13	0.00	0.13	0.18	0.22	**0.22**
0.5a	-0.18	-0.13	0.09	0.22	0.31	0.35	**0.35**
0.4a	0.31	-0.04	0.18	0.35	0.44	0.49	**0.49**
0.3a	-0.18	0.00	0.22	0.40	0.49	0.49	**0.49**
0.2a	-0.13	0.04	0.22	0.31	0.35	0.35	**0.35**
0.1a	-0.04	0.04	0.09	0.09	0.13	0.13	**0.13**
Bottom	0.00	0.04	-0.04	-0.13	-0.22	-0.22	**0.04**

Maximum Negative Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	Maximum
Top	-0.18	0.00	0.00	0.00	0.00	0.00	**-0.18**
0.9a	-0.35	-0.09	0.00	0.04	0.04	0.04	**-0.35**
0.8a	-0.35	-0.04	0.04	0.09	0.13	0.13	**-0.35**
0.7a	-0.35	0.00	0.09	0.18	0.27	0.27	**-0.35**
Long Side 0.6a	-0.35	0.00	0.18	0.31	0.35	0.40	**-0.35**
0.5a	-0.35	0.04	0.22	0.35	0.44	0.49	**-0.35**
0.4a	-0.31	0.04	0.22	0.35	0.35	0.35	**-0.31**
0.3a	-0.27	-0.04	0.00	-0.09	-0.18	-0.22	**-0.27**
0.2a	-0.18	-0.22	-0.49	-0.80	-1.02	-1.06	**-1.06**
0.1a	-0.04	-0.57	-1.28	-1.90	-2.25	-2.39	**-2.39**
Bottom	0.00	-1.19	-2.52	-3.49	-4.02	-4.20	**-4.20**

Maximum Negative Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	Maximum
Top	-0.18	0.00	0.00	0.00	0.00	0.00	**-0.18**
0.9a	-0.35	-0.22	-0.13	-0.09	-0.04	-0.04	**-0.35**
0.8a	-0.35	-0.27	-0.13	-0.09	-0.09	-0.09	**-0.35**
0.7a	-0.35	-0.22	-0.09	-0.09	-0.09	-0.09	**-0.35**
Short Side 0.6a	-0.35	-0.18	-0.13	-0.09	-0.09	-0.09	**-0.35**
0.5a	-0.35	-0.13	-0.09	-0.09	-0.04	-0.04	**-0.35**
0.4a	-0.18	-0.13	-0.09	-0.04	0.00	0.00	**-0.18**
0.3a	-0.27	-0.09	-0.04	0.04	0.04	0.09	**-0.27**
0.2a	-0.18	-0.04	0.04	0.09	0.13	0.13	**-0.18**
0.1a	-0.04	0.04	0.04	-0.04	-0.09	-0.09	**-0.09**
Bottom	0.00	-0.09	-0.44	-0.75	-0.97	-1.02	**-1.02**

Inlet Structure #3:

High Wall - b/a = 1.0, c/a = 0.5
(p 3-34)
qu = 945 psf
a = 6.84 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-3	-4	3	9	13	14
0.9a	-13	-4	4	9	13	14
0.8a	-15	-4	4	10	14	15
0.7a	-17	-5	4	11	14	16
0.6a	-19	-5	5	11	15	16
0.5a	-21	-5	5	11	14	15
0.4a	-21	-4	5	10	13	14
0.3a	-19	-4	4	8	10	10
0.2a	-14	2	3	4	5	5
0.1a	-6	-1	0	-1	-1	-1
Bottom	0	-2	-5	-7	-8	-8

My Coefficients

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-3	-11	-10	-10	-9	-9
0.9a	-13	-11	-10	-9	-8	-8
0.8a	-15	-12	-10	-8	-7	-7
0.7a	-17	-13	-10	-7	-6	-6
0.6a	-19	-14	-9	-6	-5	-4
0.5a	-21	-14	-8	-5	-3	-2
0.4a	-21	-13	-7	-3	-1	0
0.3a	-19	-10	-5	-1	1	2
0.2a	-14	-7	-2	1	2	3
0.1a	-6	-2	0	1	1	2
Bottom	0	0	0	-1	-1	-1

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.13	-0.18	0.13	0.40	0.57	0.62
0.9a	-0.57	-0.18	0.18	0.40	0.57	0.62
0.8a	-0.66	-0.18	0.18	0.44	0.62	0.66
0.7a	-0.75	-0.22	0.18	0.49	0.62	0.71
0.6a	-0.84	-0.22	0.22	0.49	0.66	0.71
0.5a	-0.93	-0.22	0.22	0.49	0.62	0.66
0.4a	-0.93	-0.18	0.22	0.44	0.57	0.62
0.3a	-0.84	-0.18	0.18	0.35	0.44	0.44
0.2a	-0.62	0.09	0.13	0.18	0.22	0.22
0.1a	-0.27	-0.04	0.00	-0.04	-0.04	-0.04
Bottom	0.00	-0.09	-0.22	-0.31	-0.35	-0.35

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.13	-0.49	-0.44	-0.44	-0.40	-0.40
0.9a	-0.57	-0.49	-0.44	-0.40	-0.35	-0.35
0.8a	-0.66	-0.53	-0.44	-0.35	-0.31	-0.31
0.7a	-0.75	-0.57	-0.44	-0.31	-0.27	-0.27
0.6a	-0.84	-0.62	-0.40	-0.27	-0.22	-0.18
0.5a	-0.93	-0.62	-0.35	-0.22	-0.13	-0.09
0.4a	-0.93	-0.57	-0.31	-0.13	-0.04	0.00
0.3a	-0.84	-0.44	-0.22	-0.04	0.04	0.09
0.2a	-0.62	-0.31	-0.09	0.04	0.09	0.13
0.1a	-0.27	-0.09	0.00	0.04	0.04	0.09
Bottom	0.00	0.00	0.00	-0.04	-0.04	-0.04

Short Wall - b/a = 2.0, c/a = 1.0
(p 3-32)
qu = 762 psf
a = 5.67 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-22	-13	8	20	27	29
0.9a	-42	-12	8	19	25	27
0.8a	-41	-11	8	19	24	25
0.7a	-41	-10	8	18	22	23
0.6a	-41	-8	8	16	19	20
0.5a	-39	-6	8	14	16	16
0.4a	-35	-4	7	10	11	11
0.3a	-28	-3	4	6	5	5
0.2a	-18	-2	0	0	-1	-2
0.1a	-6	-3	-5	-8	-9	-10
Bottom	0	-5	-11	-16	-18	-19

My Coefficients

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-22	-29	-18	-9	-4	-2
0.9a	-42	-27	-16	-7	-2	0
0.8a	-41	-26	-14	-5	0	2
0.7a	-41	-24	-11	-3	2	4
0.6a	-41	-22	-9	0	5	6
0.5a	-39	-19	-6	2	7	8
0.4a	-35	-15	-3	4	7	9
0.3a	-28	-11	-1	4	7	8
0.2a	-18	-6	0	3	5	5
0.1a	-6	-2	0	1	1	1
Bottom	0	0	-2	-3	-4	-5

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.97	-0.57	0.35	0.88	1.19	1.28
0.9a	-1.86	-0.53	0.35	0.84	1.11	1.19
0.8a	-1.81	-0.49	0.35	0.84	1.06	1.11
0.7a	-1.81	-0.44	0.35	0.80	0.97	1.02
0.6a	-1.81	-0.35	0.35	0.71	0.84	0.88
0.5a	-1.72	-0.27	0.35	0.62	0.71	0.71
0.4a	-1.55	-0.18	0.31	0.44	0.49	0.49
0.3a	-1.24	-0.13	0.18	0.27	0.22	0.22
0.2a	-0.80	-0.09	0.00	0.00	-0.04	-0.09
0.1a	-0.27	-0.13	-0.22	-0.35	-0.40	-0.44
Bottom	0.00	-0.22	-0.49	-0.71	-0.80	-0.84

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.97	-1.28	-0.80	-0.40	-0.18	-0.09
0.9a	-1.86	-1.19	-0.71	-0.31	-0.09	0.00
0.8a	-1.81	-1.15	-0.62	-0.22	0.00	0.09
0.7a	-1.81	-1.06	-0.49	-0.13	0.09	0.18
0.6a	-1.81	-0.97	-0.40	0.00	0.22	0.27
0.5a	-1.72	-0.84	-0.27	0.09	0.31	0.35
0.4a	-1.55	-0.66	-0.13	0.18	0.31	0.40
0.3a	-1.24	-0.49	-0.04	0.18	0.31	0.35
0.2a	-0.80	-0.27	0.00	0.13	0.22	0.22
0.1a	-0.27	-0.09	0.00	0.04	0.04	0.04
Bottom	0.00	0.00	-0.09	-0.13	-0.18	-0.22

Inlet Structure #3:

Maximum Positive Horizontal Bending Moments

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.13	-0.18	0.35	0.88	1.19	1.28
0.9a	-0.57	-0.18	0.35	0.84	1.11	1.19
0.8a	-0.66	-0.18	0.35	0.84	1.06	1.11
0.7a	-0.75	-0.22	0.35	0.80	0.97	1.02
Long Side 0.6a	-0.84	-0.22	0.35	0.71	0.84	0.88
0.5a	-0.93	-0.22	0.35	0.62	0.71	0.71
0.4a	-0.93	-0.18	0.31	0.44	0.57	0.62
0.3a	-0.84	-0.13	0.18	0.35	0.44	0.44
0.2a	-0.62	0.09	0.13	0.18	0.22	0.22
0.1a	-0.27	-0.04	0.00	-0.04	-0.04	-0.04
Bottom	0.00	-0.09	-0.22	-0.31	-0.35	-0.35
Maximum	**0.00**	**0.09**	**0.35**	**0.88**	**1.19**	**1.28**

Muy Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.13	-0.49	-0.44	-0.40	-0.18	-0.09
0.9a	-0.57	-0.49	-0.44	-0.31	-0.09	0.00
0.8a	-0.66	-0.53	-0.44	-0.22	0.00	0.09
0.7a	-0.75	-0.57	-0.44	-0.13	0.09	0.18
Short Side 0.6a	-0.84	-0.62	-0.40	0.00	0.22	0.27
0.5a	-0.93	-0.62	-0.27	0.09	0.31	0.35
0.4a	-0.93	-0.57	-0.13	0.18	0.31	0.40
0.3a	-0.84	-0.44	-0.04	0.18	0.31	0.35
0.2a	-0.62	-0.27	0.00	0.13	0.22	0.22
0.1a	-0.27	-0.09	0.00	0.04	0.04	0.09
Bottom	0.00	0.00	0.00	-0.04	-0.04	-0.04
Maximum	**0.00**	**0.00**	**0.00**	**0.18**	**0.31**	**0.40**

Maximum Negative Horizontal Bending Moments

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.97	-0.57	0.13	0.40	0.57	0.62
0.9a	-1.86	-0.53	0.18	0.40	0.57	0.62
0.8a	-1.81	-0.49	0.18	0.44	0.62	0.66
0.7a	-1.81	-0.44	0.18	0.49	0.62	0.71
Long Side 0.6a	-1.81	-0.35	0.22	0.49	0.66	0.71
0.5a	-1.72	-0.27	0.22	0.49	0.62	0.66
0.4a	-1.55	-0.18	0.22	0.44	0.49	0.49
0.3a	-1.24	-0.18	0.18	0.27	0.22	0.22
0.2a	-0.80	-0.09	0.00	0.00	-0.04	-0.09
0.1a	-0.27	-0.13	-0.22	-0.35	-0.40	-0.44
Bottom	0.00	-0.22	-0.49	-0.71	-0.80	-0.84
Maximum	**-1.86**	**-0.57**	**-0.49**	**-0.71**	**-0.80**	**-0.84**

Muy Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.97	-1.28	-0.80	-0.44	-0.40	-0.40
0.9a	-1.86	-1.19	-0.71	-0.40	-0.35	-0.35
0.8a	-1.81	-1.15	-0.62	-0.35	-0.31	-0.31
0.7a	-1.81	-1.06	-0.49	-0.31	-0.27	-0.27
Short Side 0.6a	-1.81	-0.97	-0.40	-0.27	-0.22	-0.18
0.5a	-1.72	-0.84	-0.35	-0.22	-0.13	-0.09
0.4a	-1.55	-0.66	-0.31	-0.13	-0.04	0.00
0.3a	-1.24	-0.49	-0.22	-0.04	0.04	0.09
0.2a	-0.80	-0.31	-0.09	0.04	0.09	0.13
0.1a	-0.27	-0.09	0.00	0.04	0.04	0.04
Bottom	0.00	0.00	-0.09	-0.13	-0.18	-0.22
Maximum	**-1.86**	**-1.28**	**-0.80**	**-0.44**	**-0.40**	**-0.40**

Inlet Structure #3:

Bottom Slab Positive Moments (tension on top face):

b = 9.67 ft
a = 5.17 ft
b/a = 1.8704062 -----> 2.0 (p 2-61)

qu = 629 psf

My Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0	0	0	0	0	0
0.9a	0	11	13	13	13	13
0.8a	0	19	23	24	23	23
0.7a	0	24	31	32	31	30
0.6a	0	28	36	37	36	35
0.5a	0	29	37	38	37	37
0.4a	0	28	36	37	36	35
0.3a	0	24	31	32	31	30
0.2a	0	19	23	24	23	23
0.1a	0	11	13	13	13	13
Bottom	0	0	0	0	0	0

Mx Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0	0	0	0	0	0
0.9a	0	17	27	33	36	37
0.8a	0	28	46	57	63	65
0.7a	0	34	58	74	82	85
0.6a	0	38	66	83	93	96
0.5a	0	39	68	86	97	100
0.4a	0	38	66	83	93	96
0.3a	0	34	58	74	82	85
0.2a	0	28	46	57	63	65
0.1a	0	17	27	33	36	37
Bottom	0	0	0	0	0	0

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	0.18	0.22	0.22	0.22	0.22
0.8a	0.00	0.32	0.39	0.40	0.39	0.39
0.7a	0.00	0.40	0.52	0.54	0.52	0.50
0.6a	0.00	0.47	0.61	0.62	0.61	0.59
0.5a	0.00	0.49	0.62	0.64	0.62	0.62
0.4a	0.00	0.47	0.61	0.62	0.61	0.59
0.3a	0.00	0.40	0.52	0.54	0.52	0.50
0.2a	0.00	0.32	0.39	0.40	0.39	0.39
0.1a	0.00	0.18	0.22	0.22	0.22	0.22
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Mux Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	0.29	0.45	0.55	0.61	0.00
0.8a	0.00	0.47	0.77	0.96	1.06	0.00
0.7a	0.00	0.57	0.98	1.24	1.38	0.00
0.6a	0.00	0.64	1.11	1.40	1.56	0.00
0.5a	0.00	0.66	1.14	1.45	1.63	0.00
0.4a	0.00	0.64	1.11	1.40	1.56	0.00
0.3a	0.00	0.57	0.98	1.24	1.38	0.00
0.2a	0.00	0.47	0.77	0.96	1.06	0.00
0.1a	0.00	0.29	0.45	0.55	0.61	0.00
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Muy,max =	**0.64 kip-ft**
Mux,max =	**1.63 kip-ft**

INLET STRUCTURE #3 - REINFORCEMENT DESIGN

Design Parameters:

$f_c := 3000psi$

$f_y := 60ksi$

$\beta_1 := 0.85$

$\Phi_b := 0.9$

$b := 12in$

$t_{wall} := 10in$ $\quad d_{wall} := t_{wall} - 2in - \frac{.75in}{2} = 7.625 \times in$

$t_{slab} := 11in$ $\quad d_{slab} := t_{slab} - 3in - \frac{.75in}{2} = 7.625 \times in$

Balanced Reinforcement ratio for rectangular sections with tension reinforcement only:

$$\rho_b := \left(\frac{0.85 \times \beta_1 \times f_c}{f_y}\right) \times \left(\frac{87ksi}{87ksi + f_y}\right) = 0.021$$

Maximum Reinforcement of Flexural Members: (AASHTO 8.16.3.1)

$\rho_{max} := 0.75 \times \rho_b = 0.016$

Minimum Reinforcement of Flexural Members: (AASHTO 8.17.1)

ΦMn >= 1.2Mcr unless Asprov > 4/3*Asreqd

$$M_{cr} = \frac{f_r \times I_g}{y_t}$$

$f_r = 7.5 \times \sqrt{f_c}$ $\quad f_r := 7.5 \times \sqrt{3000} = 410.792$ psi

$$I_{gwall} := \frac{12in \times t_{wall}^3}{12} = 0.048\,ft^4 \qquad I_{gslab} := \frac{12in \times t_{slab}^3}{12} = 0.064\,ft^4$$

$$y_{twall} := \frac{t_{wall}}{2} = 0.417\,ft \qquad y_{tslab} := \frac{t_{slab}}{2} = 0.458\,ft$$

$$M_{crwall} := \frac{f_r \times psi \times I_{gwall}}{y_{twall}} = 6.847 \times ft \times kip \quad \text{-----> } 1.2M_{crwall} = 8.216 \times ft \times kip$$

$$M_{crslab} := \frac{f_r \times psi \times I_{gslab}}{y_{tslab}} = 8.284 \times ft \times kip \quad \text{-----> } 1.2M_{crslab} = 9.941 \times ft \times kip$$

Temperature/Shrinkage Reinforcement (AASHTO 8.20.1)

$A_{s.TSreq} := 0.125in^2$ (Min area of steel per foot in each direction)

$S_{max1} := 3 \times t_{wall} = 30 \times in$ $\qquad S_{max2} := 18in$

$S_{max} := \min(S_{max1}, S_{max2}) = 18 \times in$

$A_{No4} := 0.2$ (Area of #4 bar)

$$A_{s.TS} := A_{No4} \times in^2 \times \frac{12in}{S_{max}} = 0.133 \times in^2 \quad > \quad A_{s.TSreq} = 0.125 \times in^2 \quad \text{OK}$$

Therefore use a minimum of #4 with a max spacing of 18" o.c.
- All layers, all directions

Vertical Reinforcement - Outside Face:

All Walls - Check worst case:

$M_{u.wall} := 4.2 \times kip \times ft$

Try #5 at 12" o.c.

$A_{s.wall} := \frac{0.31in^2 \times 12}{12} = 0.31 \times in^2$

$\rho_{wall} := \frac{A_{s.wall}}{d_{wall} \times b} = 3.388 \times 10^{-3}$

$check_{\rho wall} := if(\rho_{wall} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.wall} := \Phi_b \times \left[A_{s.wall} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{wall} \times f_y}{f_c} \right) \right] = 10.204 \times kip \times ft$

$check_{wall} := if(\Phi M_{n.wall} > M_{u.wall}, "OK", "NG") = "OK"$

$check2_{wall} := if(\Phi M_{n.wall} > 1.2M_{crwall}, "OK", "NG") = "OK"$

Therefore Use #5 at 12" o.c. outside face vertical rebar for all walls.

#5 @ 9" Closed ok

Vertical Reinforcement - Inside Face:

All Walls - Check worst case:

$M_{u.inwall} := 0.66 \times kip \times ft$

Try #4 at 18" o.c.

$$A_{s.inwall} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$$

$$\rho_{inwall} := \frac{A_{s.inwall}}{d_{wall} \times b} = 1.457 \times 10^{-3}$$

$check_{\rho inwall} := if(\rho_{inwall} < \rho_{max}, "OK", "NG") = "OK"$

$$\Phi M_{n.inwall} := \Phi_b \times \left[A_{s.inwall} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{inwall} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$$

$check_{inwall} := if(\Phi M_{n.inwall} > M_{u.inwall}, "OK", "NG") = "OK"$

$check2_{inwall} := if(\Phi M_{n.inwall} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$$\frac{M_{u.inwall}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0042 \quad \text{-----> } \omega := 0.005 \quad \text{(Design Aid)}$$

$$A_{s.inreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.023 \times in^2 \quad \text{-----> } \frac{4}{3} \times A_{s.inreq} = 0.031 \times in^2$$

$check3_{inwall} := if\left(A_{s.inwall} > \frac{4}{3} \times A_{s.inreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. inside face vertical rebar for all walls.

#4 @ 18" o/c ea face

Horizontal Reinforcement - Outside Face:

All Walls - Check worst case:

$M_{u.hout} := 1.86 \times kip \times ft$

Try #4 at 18" o.c.

$A_{s.hout} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{hout} := \frac{A_{s.hout}}{d_{wall} \times b} = 1.457 \times 10^{-3}$

$check_{\rho hout} := if(\rho_{hout} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.hout} := \Phi_b \times \left[A_{s.hout} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{hout} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{hout} := if(\Phi M_{n.hout} > M_{u.hout}, "OK", "NG") = "OK"$

$check2_{hout} := if(\Phi M_{n.hout} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.hout}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0118$ -----> $\omega := 0.012$ (Design Aid)

$A_{s.houtreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.055 \times in^2$ -----> $\frac{4}{3} \times A_{s.houtreq} = 0.073 \times in^2$

$check3_{hout} := if\left(A_{s.hout} > \frac{4}{3} \times A_{s.houtreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. outside face horizontal rebar for all walls.

#4 @ 12" Oc ef

Horizontal Reinforcement - Inside Face:

All Walls - Check worst case:

$M_{u.hin} := 1.28 \times kip \times ft$

Try #4 at 18" o.c.

$A_{s.hin} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{hin} := \frac{A_{s.hin}}{d_{wall} \times b} = 1.457 \times 10^{-3}$

$check_{\rho hin} := if(\rho_{hin} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.hin} := \Phi_b \times \left[A_{s.hin} \times f_y \times d_{wall} \times \left(1 - 0.6 \times \frac{\rho_{hin} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{hin} := if(\Phi M_{n.hin} > M_{u.hin}, "OK", "NG") = "OK"$

$check2_{hin} := if(\Phi M_{n.hin} > 1.2M_{crwall}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.hin}}{\Phi_b \times f_c \times b \times d_{wall}^2} = 0.0082$ -----> $\omega := 0.009$ (Design Aid)

$A_{s.hinreq} := \omega \times b \times d_{wall} \times \frac{f_c}{f_y} = 0.041 \times in^2$ -----> $\frac{4}{3} \times A_{s.hinreq} = 0.055 \times in^2$

$check3_{hin} := if\left(A_{s.hin} > \frac{4}{3} \times A_{s.hinreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. inside face horizontal rebar for all walls.

#4 @ 12" closed. OK

Slab Reinforcement - Top Face:

Check worst case moment in both directions:

$M_{u.slabt} := 1.63 \times kip \times ft$

Try #4 at 18" o.c.

$A_{s.slabt} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{slabt} := \frac{A_{s.slabt}}{d_{slab} \times b} = 1.457 \times 10^{-3}$

$check_{\rho slabt} := if(\rho_{slabt} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabt} := \Phi_b \times \left[A_{s.slabt} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabt} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{slabt} := if(\Phi M_{n.slabt} > M_{u.slabt}, "OK", "NG") = "OK"$

$check2_{slabt} := if(\Phi M_{n.slabt} > 1.2M_{crslab}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.slabt}}{\Phi_b \times f_c \times b \times d_{slab}^2} = 0.0104$ -----> $\omega := 0.011$ (Design Aid)

$A_{s.slabtreq} := \omega \times b \times d_{slab} \times \frac{f_c}{f_y} = 0.05 \times in^2$ -----> $\frac{4}{3} \times A_{s.slabtreq} = 0.067 \times in^2$

$check3_{slabt} := if\left(A_{s.slabt} > \frac{4}{3} \times A_{s.slabtreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. each way in top face of slab.

Slab Reinforcement - Bottom Face in Short Direction:

Check worst case moment from bottom of High Wall and Short Wall:

$$M_{u.slabbs} := M_{u.wall} = 4.2 \times kip \times ft$$

Try #5 at 12" o.c.

$$A_{s.slabbs} := \frac{0.31in^2 \times 12}{12} = 0.31 \times in^2$$

$$\rho_{slabbs} := \frac{A_{s.slabbs}}{d_{slab} \times b} = 3.388 \times 10^{-3}$$

$$check_{\rho slabbs} := if\left(\rho_{slabbs} < \rho_{max}, "OK", "NG"\right) = "OK"$$

$$\Phi M_{n.slabbs} := \Phi_b \times \left[A_{s.slabbs} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbs} \times f_y}{f_c}\right)\right] = 10.204 \times kip \times ft$$

$$check_{slabbs} := if\left(\Phi M_{n.slabbs} > M_{u.slabbs}, "OK", "NG"\right) = "OK"$$

$$check2_{slabbs} := if\left(\Phi M_{n.slabbs} > 1.2M_{crslab}, "OK", "NG"\right) = "OK"$$

Therefore Use #5 at 12" o.c. in short direction in bottom face of slab.

#5@ 9" Clsd

Slab Reinforcement - Bottom Face in Long Direction:

Check worst case moment from bottom of Side Walls:

$M_{u.slabbl} := 1.02kip \times ft$

Try #4 at 18" o.c.

$A_{s.slabbl} := \frac{0.2in^2 \times 12}{18} = 0.133 \times in^2$

$\rho_{slabbl} := \frac{A_{s.slabbl}}{d_{slab} \times b} = 1.457 \times 10^{-3}$

$check_{\rho slabbl} := if(\rho_{slabbl} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabbl} := \Phi_b \times \left[A_{s.slabbl} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbl} \times f_y}{f_c} \right) \right] = 4.495 \times kip \times ft$

$check_{slabbl} := if(\Phi M_{n.slabbl} > M_{u.slabbl}, "OK", "NG") = "OK"$

$check2_{slabbl} := if(\Phi M_{n.slabbl} > 1.2M_{crslab}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.slabbl}}{\Phi_b \times f_c \times b \times d_{slab}^2} = 0.0065$ -----> $\omega := 0.007$ (Design Aid)

$A_{s.slabblreq} := \omega \times b \times d_{slab} \times \frac{f_c}{f_y} = 0.032 \times in^2$ -----> $\frac{4}{3} \times A_{s.slabblreq} = 0.043 \times in^2$

$check3_{slabbl} := if\left(A_{s.slabbl} > \frac{4}{3} \times A_{s.slabblreq}, "OK", "NG" \right) = "OK"$

Therefore Use #4 at 18" o.c. in long direction in bottom face of slab.

2.4

INLET STRUCTURE #6

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

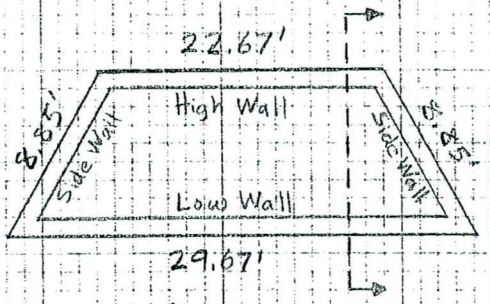
JOB NUMBER 2009034

TITLE Oak St Basin

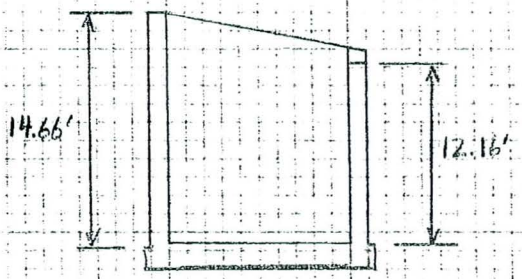
SUBJECT Inlet Structure #6

MADE BY JPH DATE 11-18-10 CHECKED BY Raj D DATE 12/26/10

Rectangular Concrete Tank Analysis



Plan



Section

High Wall

$a = 14.66'$

$b = 22.67'$ $\quad \frac{b}{a} = 1.55$ $\quad \rightarrow$ Use $\frac{b}{a} = 1.5$ $\quad \frac{c}{a} = 0.5$

$c = 8.85'$ $\quad \frac{c}{a} = 0.60$

$q_t = (60\ psf/ft)(14.66' - 12.16') + (99\ psf/ft)(12.16') = 1354\ psf$

$q_u = \gamma(P_E + q) = 1.3(1.15)(1354\ psf) = 2024\ psf$

Short Wall

$a = 12.16'$

$b = 29.67'$ $\quad \frac{b}{a} = 2.44$ $\quad \rightarrow$ Use $\frac{b}{a} = 3.0$ $\quad \frac{c}{a} = 0.5$

$c = 8.85'$ $\quad \frac{c}{a} = 0.73$

$q = (90\ psf/ft)(12.16') = 1106\ psf$

$q_u = \gamma(P_E + q) = 1.3(1.15)(1106\ psf) = 1654\ psf$

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2009034

TITLE Oak Street

SUBJECT Inlet Structure #6

MADE BY JPH DATE 11-23-10 CHECKED BY Raj D DATE 12/26/10

Bottom Slab

Negative moment for bottom slab will be equal to negative moments in the bottom of the wall.

Positive moments will be determined from an analysis of the slab assuming hinged connections to the wall.

Use $\frac{b}{a} = \frac{26.17}{7.67} = 3.4 \rightarrow 3.5$

b = Avg. width of slab

a = Length of slab

$W_{single\ wall} = (150\ pcf)(12.16')(29.67')(\frac{3}{12}) = 58.4^k$

$W_{high\ wall} = (150\ pcf)(14.66')(22.67')(\frac{3}{12}) = 14.9^k$

$W_{side\ walls} = (150\ pcf)(13.41')(8.85')(\frac{3}{12})(2) = 78.1^k$

151.2^k

$A_{slab} = (26.17')(7.67') = 201\ ft^2$

$q = \frac{151.2^k}{201\ ft^2} = 0.75\ Ksf = 752\ psf$

$q_u = \gamma\ P_b\ q = (1.3)(1.0)(752) = 978\ psf$

- See Excel Spreadsheet for Moment Calculations

Inlet Structure #6 (#5 similar):

Shear Design, Vu = Cs*qu*a
where Cs = Shear Coefficient

High Wall - b/a = 1.5, c/a = 0.5

qu = 2024 psf
a = 14.66 ft

Shear Coefficients for b/a = 1.5
Bottom Edge - Midpoint = 0.40
Side Edge - Maximum = 0.26
Side Edge - Midpoint = 0.26

Shear Coefficients for c/a = 0.5
Bottom Edge - Midpoint = 0.19
Side Edge - Maximum = 0.17
Side Edge - Midpoint = 0.13

Max Bottom Shear Coefficient = 0.40
Max Side Shear Coefficient = 0.26

Short Wall - b/a = 3.0, c/a = 0.5

qu = 1654 psf
a = 12.16 ft

Shear Coefficients for b/a = 3.0
Bottom Edge - Midpoint = 0.50
Side Edge - Maximum = 0.37
Side Edge - Midpoint = 0.24

Shear Coefficients for c/a = 0.5
Bottom Edge - Midpoint = 0.19
Side Edge - Maximum = 0.17
Side Edge - Midpoint = 0.13

Max Bottom Shear Coefficient = 0.50
Max Side Shear Coefficient = 0.37

Design Shears (lb)

	Bottom Edge	Side Edge (Max)	Side Edge (Mid)
High Wall:	**11869**	**7715**	**7715**
Side Wall	**5638**	**5044**	**3857**
Short Wall:	**10056**	**7442**	**4827**

Check walls for Shear
Side wall will be 10" thick for full height,
high wall and short wall will be 10" at top and 13" at bottom

Vc = 2*sqrt(f'c)*b*d
Φ = 0.85
f'c = 3000 psi
b = 12 in

	Top	Middle	Bottom
t (in) =	10	11.5	13
d (in) =	7.625	9.125	10.625
ΦVc (lb) =	8519.82438	10195.855	11871.886

Check shear for side wall:
Vu, max = 5638 lb
ΦVc = 8520 lb > Vu = 5638 lb OK

Check shear for high wall and short wall:
At side edge:
Vu,side = 7715 lb
ΦVc = 10196 lb > Vu = 7715 lb OK

At bottom edge:
Vu,bot = 11869 lb
ΦVc = 11872 lb > Vu = 11869 lb OK

Inlet Structure #6 (#5 similar):

High Wall - b/a = 1.5, c/a = 0.5
(p 3-33)

qu = 2024 psf
a = 14.66 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-2	0	0	0	0	0
0.9a	-6	-2	1	2	3	3
0.8a	-7	-2	2	5	7	7
0.7a	-7	-1	4	8	10	11
Long Side 0.6a	-7	0	6	11	13	14
0.5a	-7	0	7	12	14	15
0.4a	-7	1	7	10	12	13
0.3a	-5	0	4	5	6	6
0.2a	-4	-2	-3	-5	-7	-8
0.1a	-1	-7	-16	-24	-30	-32
Bottom	0	-16	-38	-54	-64	-67

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-0.87	0.00	0.00	0.00	0.00	0.00
0.9a	-2.61	-0.87	0.43	0.87	1.30	1.30
0.8a	-3.04	-0.87	0.87	2.17	3.04	3.04
0.7a	-3.04	-0.43	1.74	3.48	4.35	4.78
Long Side 0.6a	-3.04	0.00	2.61	4.78	5.65	6.09
0.5a	-3.04	0.00	3.04	5.22	6.09	6.52
0.4a	-3.04	0.43	3.04	4.35	5.22	5.65
0.3a	-2.17	0.00	1.74	2.17	2.61	2.61
0.2a	-1.74	-0.87	-1.30	-2.17	-3.04	-3.48
0.1a	-0.43	-3.04	-6.96	-10.44	-13.05	-13.92
Bottom	0.00	-6.96	-16.53	-23.49	-27.84	-29.14

Vertical Bending Moments, Muz (in-kips)

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-2	0	0	0	0	0
0.9a	-6	-3	-2	-1	-1	0
0.8a	-7	-4	-2	0	0	1
0.7a	-7	-4	-1	1	2	3
Short Side 0.6a	-7	-3	0	3	5	6
0.5a	-7	-2	2	6	8	8
0.4a	-7	-1	4	7	10	10
0.3a	-5	0	5	8	10	10
0.2a	-4	1	4	5	6	6
0.1a	-1	0	-1	-3	-5	-5
Bottom	0	-4	-13	-21	-26	-28

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-0.87	0.00	0.00	0.00	0.00	0.00
0.9a	-2.61	-1.30	-0.87	-0.43	-0.43	0.00
0.8a	-3.04	-1.74	-0.87	0.00	0.00	0.43
0.7a	-3.04	-1.74	-0.43	0.43	0.87	1.30
Short Side 0.6a	-3.04	-1.30	0.00	1.30	2.17	2.61
0.5a	-3.04	-0.87	0.87	2.61	3.48	3.48
0.4a	-3.04	-0.43	1.74	3.04	4.35	4.35
0.3a	-2.17	0.00	2.17	3.48	4.35	4.35
0.2a	-1.74	0.43	1.74	2.17	2.61	2.61
0.1a	-0.43	0.00	-0.43	-1.30	-2.17	-2.17
Bottom	0.00	-1.74	-5.65	-9.13	-11.31	-12.18

Short Wall - b/a = 3.0, c/a = 0.5
(p 3-31)

qu = 1654 psf
a = 12.16 ft

Vertical Bending Moments, Mux (in-kips)

Mx Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-7	0	0	0	0	0
0.9a	-11	-1	2	4	4	4
0.8a	-9	0	5	7	8	8
0.7a	-8	2	7	9	9	9
Long Side 0.6a	-8	3	8	9	8	8
0.5a	-7	3	6	5	3	2
0.4a	-6	1	0	-4	-8	-10
0.3a	-4	-3	-11	-19	-26	-28
0.2a	-3	-12	-28	-42	-51	-54
0.1a	-1	-26	-54	-74	-85	-89
Bottom	0	-49	-90	-116	-130	-134

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-3.04	0.00	0.00	0.00	0.00	0.00
0.9a	-4.78	-0.43	0.87	1.74	1.74	1.74
0.8a	-3.91	0.00	2.17	3.04	3.48	3.48
0.7a	-3.48	0.87	3.04	3.91	3.91	3.91
Long Side 0.6a	-3.48	1.30	3.48	3.91	3.48	3.48
0.5a	-3.04	1.30	2.61	2.17	1.30	0.87
0.4a	-2.61	0.43	0.00	-1.74	-3.48	-4.35
0.3a	-1.74	-1.30	-4.78	-8.26	-11.31	-12.18
0.2a	-1.30	-5.22	-12.18	-18.27	-22.18	-23.49
0.1a	-0.43	-11.31	-23.49	-32.19	-36.97	-38.71
Bottom	0.00	-21.31	-39.15	-50.46	-56.55	-58.29

Vertical Bending Moments, Muz (in-kips)

Mz Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-7	0	0	0	0	0
0.9a	-11	-9	-8	-7	-6	-6
0.8a	-9	-9	-8	-8	-8	-8
0.7a	-8	-8	-8	-7	-7	-7
Short Side 0.6a	-8	-7	-6	-6	-6	-6
0.5a	-7	-6	-5	-4	-4	-3
0.4a	-6	-4	-3	-2	-1	-1
0.3a	-4	-2	0	1	2	2
0.2a	-3	0	2	4	5	5
0.1a	-1	2	4	5	6	6
Bottom	0	3	4	3	3	3

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-3.04	0.00	0.00	0.00	0.00	0.00
0.9a	-4.78	-3.91	-3.48	-3.04	-2.61	-2.61
0.8a	-3.91	-3.91	-3.48	-3.48	-3.48	-3.48
0.7a	-3.48	-3.48	-3.48	-3.04	-3.04	-3.04
Short Side 0.6a	-3.48	-3.04	-2.61	-2.61	-2.61	-2.61
0.5a	-3.04	-2.61	-2.17	-1.74	-1.74	-1.30
0.4a	-2.61	-1.74	-1.30	-0.87	-0.43	-0.43
0.3a	-1.74	-0.87	0.00	0.43	0.87	0.87
0.2a	-1.30	0.00	0.87	1.74	2.17	2.17
0.1a	-0.43	0.87	1.74	2.17	2.61	2.61
Bottom	0.00	1.30	1.74	1.30	1.30	1.30

Inlet Structure #6 (#5 similar):

Maximum Positive Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	**Maximum**
Top	-0.87	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-2.61	-0.43	0.87	1.74	1.74	1.74	**1.74**
0.8a	-3.04	0.00	2.17	3.04	3.48	3.48	**3.48**
0.7a	-3.04	0.87	3.04	3.91	4.35	4.78	**4.78**
Long Side 0.6a	-3.04	1.30	3.48	4.78	5.65	6.09	**6.09**
0.5a	-3.04	1.30	3.04	5.22	6.09	6.52	**6.52**
0.4a	-2.61	0.43	3.04	4.35	5.22	5.65	**5.65**
0.3a	-1.74	0.00	1.74	2.17	2.61	2.61	**2.61**
0.2a	-1.30	-0.87	-1.30	-2.17	-3.04	-3.48	**-0.87**
0.1a	-0.43	-3.04	-6.96	-10.44	-13.05	-13.92	**-0.43**
Bottom	0.00	-6.96	-16.53	-23.49	-27.84	-29.14	**0.00**

Maximum Positive Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	**Maximum**
Top	-0.87	0.00	0.00	0.00	0.00	0.00	**0.00**
0.9a	-2.61	-1.30	-0.87	-0.43	-0.43	0.00	**0.00**
0.8a	-3.04	-1.74	-0.87	0.00	0.00	0.43	**0.43**
0.7a	-3.04	-1.74	-0.43	0.43	0.87	1.30	**1.30**
Short Side 0.6a	-3.04	-1.30	0.00	1.30	2.17	2.61	**2.61**
0.5a	-3.04	-0.87	0.87	2.61	3.48	3.48	**3.48**
0.4a	-2.61	-0.43	1.74	3.04	4.35	4.35	**4.35**
0.3a	-1.74	0.00	2.17	3.48	4.35	4.35	**4.35**
0.2a	-1.30	0.43	1.74	2.17	2.61	2.61	**2.61**
0.1a	-0.43	0.87	1.74	2.17	2.61	2.61	**2.61**
Bottom	0.00	1.30	1.74	1.30	1.30	1.30	**1.74**

Maximum Negative Vertical Bending Moments

Mux Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b		
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b	**Maximum**
Top	-3.04	0.00	0.00	0.00	0.00	0.00	**-3.04**
0.9a	-4.78	-0.87	0.43	0.87	1.30	1.30	**-4.78**
0.8a	-3.91	-0.87	0.87	2.17	3.04	3.04	**-3.91**
0.7a	-3.48	-0.43	1.74	3.48	3.91	3.91	**-3.48**
Long Side 0.6a	-3.48	0.00	2.61	3.91	3.48	3.48	**-3.48**
0.5a	-3.04	0.00	2.61	2.17	1.30	0.87	**-3.04**
0.4a	-3.04	0.43	0.00	-1.74	-3.48	-4.35	**-4.35**
0.3a	-2.17	-1.30	-4.78	-8.26	-11.31	-12.18	**-12.18**
0.2a	-1.74	-5.22	-12.18	-18.27	-22.18	-23.49	**-23.49**
0.1a	-0.43	-11.31	-23.49	-32.19	-36.97	-38.71	**-38.71**
Bottom	0.00	-21.31	-39.15	-50.46	-56.55	-58.29	**-58.29**

Maximum Negative Vertical Bending Moments

Muz Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c		
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c	**Maximum**
Top	-3.04	0.00	0.00	0.00	0.00	0.00	**-3.04**
0.9a	-4.78	-3.91	-3.48	-3.04	-2.61	-2.61	**-4.78**
0.8a	-3.91	-3.91	-3.48	-3.48	-3.48	-3.48	**-3.91**
0.7a	-3.48	-3.48	-3.48	-3.04	-3.04	-3.04	**-3.48**
Short Side 0.6a	-3.48	-3.04	-2.61	-2.61	-2.61	-2.61	**-3.48**
0.5a	-3.04	-2.61	-2.17	-1.74	-1.74	-1.30	**-3.04**
0.4a	-3.04	-1.74	-1.30	-0.87	-0.43	-0.43	**-3.04**
0.3a	-2.17	-0.87	0.00	0.43	0.87	0.87	**-2.17**
0.2a	-1.74	0.00	0.87	1.74	2.17	2.17	**-1.74**
0.1a	-0.43	0.00	-0.43	-1.30	-2.17	-2.17	**-2.17**
Bottom	0.00	-1.74	-5.65	-9.13	-11.31	-12.18	**-12.18**

Inlet Structure #6 (#5 similar):

High Wall - b/a = 1.5, c/a = 0.5
(p 3-33)
qu = 2024 psf
a = 14.66 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-12	-12	4	15	22	24
0.9a	-31	-11	4	15	21	23
0.8a	-33	-11	5	15	20	22
0.7a	-35	-11	5	15	20	21
0.6a	-36	-10	6	14	19	20
0.5a	-36	-9	6	13	16	17
0.4a	-33	-7	6	11	13	14
0.3a	-27	-5	4	7	8	9
0.2a	-18	-3	2	3	2	2
0.1a	-6	-2	-2	-4	-5	-5
Bottom	0	-3	-8	-11	-13	-13

My Coefficients

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-12	-20	-11	-3	1	3
0.9a	-31	-19	-10	-2	2	4
0.8a	-33	-19	-8	-1	4	6
0.7a	-35	-19	-7	1	6	8
0.6a	-36	-18	-5	3	8	9
0.5a	-36	-16	-3	5	9	10
0.4a	-33	-13	-1	5	9	10
0.3a	-27	-10	0	5	8	8
0.2a	-18	-5	1	4	5	5
0.1a	-6	-2	0	0	0	0
Bottom	0	-1	-3	-4	-5	-6

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-5.22	-5.22	1.74	6.52	9.57	10.44
0.9a	-13.48	-4.78	1.74	6.52	9.13	10.00
0.8a	-14.35	-4.78	2.17	6.52	8.70	9.57
0.7a	-15.22	-4.78	2.17	6.52	8.70	9.13
0.6a	-15.66	-4.35	2.61	6.09	8.26	8.70
0.5a	-15.66	-3.91	2.61	5.65	6.96	7.39
0.4a	-14.35	-3.04	2.61	4.78	5.65	6.09
0.3a	-11.74	-2.17	1.74	3.04	3.48	3.91
0.2a	-7.83	-1.30	0.87	1.30	0.87	0.87
0.1a	-2.61	-0.87	-0.87	-1.74	-2.17	-2.17
Bottom	0.00	-1.30	-3.48	-4.78	-5.65	-5.65

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-5.22	-8.70	-4.78	-1.30	0.43	1.30
0.9a	-13.48	-8.26	-4.35	-0.87	0.87	1.74
0.8a	-14.35	-8.26	-3.48	-0.43	1.74	2.61
0.7a	-15.22	-8.26	-3.04	0.43	2.61	3.48
0.6a	-15.66	-7.83	-2.17	1.30	3.48	3.91
0.5a	-15.66	-6.96	-1.30	2.17	3.91	4.35
0.4a	-14.35	-5.65	-0.43	2.17	3.91	4.35
0.3a	-11.74	-4.35	0.00	2.17	3.48	3.48
0.2a	-7.83	-2.17	0.43	1.74	2.17	2.17
0.1a	-2.61	-0.87	0.00	0.00	0.00	0.00
Bottom	0.00	-0.43	-1.30	-1.74	-2.17	-2.61

Short Wall - b/a = 3.0, c/a = 0.5
(p 3-31)
qu = 1654 psf
a = 12.16 ft

Horizontal Bending Moments, Muy (in-kips)

My Coefficients

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-37	-7	14	21	22	22
0.9a	-54	-6	13	19	20	20
0.8a	-47	-5	13	18	18	18
0.7a	-42	-3	12	16	16	16
0.6a	-38	-1	11	13	13	12
0.5a	-34	0	9	10	9	9
0.4a	-29	1	6	6	4	4
0.3a	-22	1	2	0	-1	-2
0.2a	-14	-1	-3	6	-8	-9
0.1a	-5	-5	-10	-14	-17	-17
Bottom	0	-10	-18	-23	-26	-27

My Coefficients

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-37	-51	-47	-44	-42	-41
0.9a	-54	-47	-43	-40	-38	-37
0.8a	-47	-42	-39	-36	-34	-33
0.7a	-42	-38	-34	-31	-29	-28
0.6a	-38	-33	-28	-25	-23	-23
0.5a	-34	-27	-23	-19	-17	-16
0.4a	-29	-22	-16	-13	-11	-10
0.3a	-22	-15	-10	-7	-5	-5
0.2a	-14	-8	-5	-2	-1	-1
0.1a	-5	-2	-1	0	1	1
Bottom	0	1	1	1	1	1

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-16.09	-3.04	6.09	9.13	9.57	9.57
0.9a	-23.49	-2.61	5.65	8.26	8.70	8.70
0.8a	-20.44	-2.17	5.65	7.83	7.83	7.83
0.7a	-18.27	-1.30	5.22	6.96	6.96	6.96
0.6a	-16.53	-0.43	4.78	5.65	5.65	5.22
0.5a	-14.79	0.00	3.91	4.35	3.91	3.91
0.4a	-12.61	0.43	2.61	2.61	1.74	1.74
0.3a	-9.57	0.43	0.87	0.00	-0.43	-0.87
0.2a	-6.09	-0.43	-1.30	2.61	-3.48	-3.91
0.1a	-2.17	-2.17	-4.35	-6.09	-7.39	-7.39
Bottom	0.00	-4.35	-7.83	-10.00	-11.31	-11.74

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-16.09	-22.18	-20.44	-19.14	-18.27	-17.83
0.9a	-23.49	-20.44	-18.70	-17.40	-16.53	-16.09
0.8a	-20.44	-18.27	-16.96	-15.66	-14.79	-14.35
0.7a	-18.27	-16.53	-14.79	-13.48	-12.61	-12.18
0.6a	-16.53	-14.35	-12.18	-10.87	-10.00	-10.00
0.5a	-14.79	-11.74	-10.00	-8.26	-7.39	-6.96
0.4a	-12.61	-9.57	-6.96	-5.65	-4.78	-4.35
0.3a	-9.57	-6.52	-4.35	-3.04	-2.17	-2.17
0.2a	-6.09	-3.48	-2.17	-0.87	-0.43	-0.43
0.1a	-2.17	-0.87	-0.43	0.00	0.43	0.43
Bottom	0.00	0.43	0.43	0.43	0.43	0.43

Inlet Structure #6 (#5 similar):

Maximum Positive Horizontal Bending Moments

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-5.22	-3.04	6.09	9.13	9.57	10.44
0.9a	-13.48	-2.61	5.65	8.26	9.13	10.00
0.8a	-14.35	-2.17	5.65	7.83	8.70	9.57
0.7a	-15.22	-1.30	5.22	6.96	8.70	9.13
0.6a	-15.66	-0.43	4.78	6.09	8.26	8.70
0.5a	-14.79	0.00	3.91	5.65	6.96	7.39
0.4a	-12.61	0.43	2.61	4.78	5.65	6.09
0.3a	-9.57	0.43	1.74	3.04	3.48	3.91
0.2a	-6.09	-0.43	0.87	2.61	0.87	0.87
0.1a	-2.17	-0.87	-0.87	-1.74	-2.17	-2.17
Bottom	0.00	-1.30	-3.48	-4.78	-5.65	-5.65
Maximum	**0.00**	**0.43**	**6.09**	**9.13**	**9.57**	**10.44**

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-5.22	-8.70	-4.78	-1.30	0.43	1.30
0.9a	-13.48	-8.26	-4.35	-0.87	0.87	1.74
0.8a	-14.35	-8.26	-3.48	-0.43	1.74	2.61
0.7a	-15.22	-8.26	-3.04	0.43	2.61	3.48
0.6a	-15.66	-7.83	-2.17	1.30	3.48	3.91
0.5a	-14.79	-6.96	-1.30	2.17	3.91	4.35
0.4a	-12.61	-5.65	-0.43	2.17	3.91	4.35
0.3a	-9.57	-4.35	0.00	2.17	3.48	3.48
0.2a	-6.09	-2.17	0.43	1.74	2.17	2.17
0.1a	-2.17	-0.87	0.00	0.00	0.43	0.43
Bottom	0.00	0.43	0.43	0.43	0.43	0.43
Maximum	**0.00**	**0.43**	**0.43**	**2.17**	**3.91**	**4.35**

Maximum Negative Horizontal Bending Moments

Muy Moments (kip-ft)

Long Side

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	-16.09	-5.22	1.74	6.52	9.57	9.57
0.9a	-23.49	-4.78	1.74	6.52	8.70	8.70
0.8a	-20.44	-4.78	2.17	6.52	7.83	7.83
0.7a	-18.27	-4.78	2.17	6.52	6.96	6.96
0.6a	-16.53	-4.35	2.61	5.65	5.65	5.22
0.5a	-15.66	-3.91	2.61	4.35	3.91	3.91
0.4a	-14.35	-3.04	2.61	2.61	1.74	1.74
0.3a	-11.74	-2.17	0.87	0.00	-0.43	-0.87
0.2a	-7.83	-1.30	-1.30	1.30	-3.48	-3.91
0.1a	-2.61	-2.17	-4.35	-6.09	-7.39	-7.39
Bottom	0.00	-4.35	-7.83	-10.00	-11.31	-11.74
Maximum	**-23.49**	**-5.22**	**-7.83**	**-10.00**	**-11.31**	**-11.74**

Muy Moments (kip-ft)

Short Side

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	-16.09	-22.18	-20.44	-19.14	-18.27	-17.83
0.9a	-23.49	-20.44	-18.70	-17.40	-16.53	-16.09
0.8a	-20.44	-18.27	-16.96	-15.66	-14.79	-14.35
0.7a	-18.27	-16.53	-14.79	-13.48	-12.61	-12.18
0.6a	-16.53	-14.35	-12.18	-10.87	-10.00	-10.00
0.5a	-15.66	-11.74	-10.00	-8.26	-7.39	-6.96
0.4a	-14.35	-9.57	-6.96	-5.65	-4.78	-4.35
0.3a	-11.74	-6.52	-4.35	-3.04	-2.17	-2.17
0.2a	-7.83	-3.48	-2.17	-0.87	-0.43	-0.43
0.1a	-2.61	-0.87	-0.43	0.00	0.00	0.00
Bottom	0.00	-0.43	-1.30	-1.74	-2.17	-2.61
Maximum	**-23.49**	**-22.18**	**-20.44**	**-19.14**	**-18.27**	**-17.83**

Inlet Structure #6 (#5 similar):

b = 26.17 ft
a = 7.67 ft
b/a = 3.4119948 -----> 4.0 (p 2-60)

qu = 978 psf

My Coefficients

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0	0	0	0	0	0
0.9a	0	13	12	10	10	9
0.8a	0	23	21	18	17	17
0.7a	0	30	28	24	22	22
0.6a	0	34	32	28	26	25
0.5a	0	36	33	29	27	26
0.4a	0	34	32	28	26	25
0.3a	0	30	28	24	22	22
0.2a	0	23	21	18	17	17
0.1a	0	13	12	10	10	9
Bottom	0	0	0	0	0	0

Mx Coefficients

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0	0	0	0	0	0
0.9a	0	28	39	43	44	44
0.8a	0	47	68	76	78	79
0.7a	0	60	88	99	103	104
0.6a	0	68	100	113	117	118
0.5a	0	70	104	117	122	123
0.4a	0	68	100	113	117	118
0.3a	0	60	88	99	103	104
0.2a	0	47	68	76	78	79
0.1a	0	28	39	43	44	44
Bottom	0	0	0	0	0	0

Muy Moments (kip-ft)

		0.1b	0.2b	0.3b	0.4b	
	Corner	0.9b	0.8b	0.7b	0.6b	0.5b
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	0.75	0.69	0.58	0.58	0.52
0.8a	0.00	1.32	1.21	1.04	0.98	0.98
0.7a	0.00	1.73	1.61	1.38	1.27	1.27
0.6a	0.00	1.96	1.84	1.61	1.50	1.44
0.5a	0.00	2.07	1.90	1.67	1.55	1.50
0.4a	0.00	1.96	1.84	1.61	1.50	1.44
0.3a	0.00	1.73	1.61	1.38	1.27	1.27
0.2a	0.00	1.32	1.21	1.04	0.98	0.98
0.1a	0.00	0.75	0.69	0.58	0.58	0.52
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Mux Moments (kip-ft)

		0.1c	0.2c	0.3c	0.4c	
	Corner	0.9c	0.8c	0.7c	0.6c	0.5c
Top	0.00	0.00	0.00	0.00	0.00	0.00
0.9a	0.00	1.61	2.24	2.47	2.53	2.53
0.8a	0.00	2.70	3.91	4.37	4.49	4.55
0.7a	0.00	3.45	5.06	5.70	5.93	5.98
0.6a	0.00	3.91	5.75	6.50	6.73	6.79
0.5a	0.00	4.03	5.98	6.73	7.02	7.08
0.4a	0.00	3.91	5.75	6.50	6.73	6.79
0.3a	0.00	3.45	5.06	5.70	5.93	5.98
0.2a	0.00	2.70	3.91	4.37	4.49	4.55
0.1a	0.00	1.61	2.24	2.47	2.53	2.53
Bottom	0.00	0.00	0.00	0.00	0.00	0.00

Muy,max = 2.07 kip-ft
Mux,max = 7.08 kip-ft

PREMIER ENGINEERING CORPORATION

Title: Oak Street Basin & Storm Drain
Subject: Inlet Structure #6
Job No. 2009034

Sheet No 1 of 10
Made By : JPH.
Date: 11/2010
Checked By: Raj D
Date: 12/2010

INLET STRUCTURE #6 (#5 Similar) - REINFORCEMENT DESIGN

Design Parameters:

$f_c := 3000psi$

$f_y := 60ksi$

$\beta_1 := 0.85$

$\Phi_b := 0.9$

$b := 12in$

Wall thickness varies linearly from top to bottom.

$t_{wallt} := 10in$ $\quad d_{wallt} := t_{wallt} - 2in - \frac{.75in}{2} = 7.625 \times in$

$t_{wallb} := 13in$ $\quad d_{wallb} := t_{wallb} - 2in - \frac{.75in}{2} = 10.625 \times in$

$t_{slab} := 14in$ $\quad d_{slab} := t_{slab} - 3in - \frac{.75in}{2} = 10.625 \times in$

Note:
Actual wall = 12"
Design Conservative
OK

Balanced Reinforcement ratio for rectangular sections with tension reinforcement only:

$$\rho_b := \left(\frac{0.85 \times \beta_1 \times f_c}{f_y}\right) \times \left(\frac{87ksi}{87ksi + f_y}\right) = 0.021$$

Maximum Reinforcement of Flexural Members: (AASHTO 8.16.3.1)

$\rho_{max} := 0.75 \times \rho_b = 0.016$

Minimum Reinforcement of Flexural Members: (AASHTO 8.17.1)

ΦMn >= 1.2Mcr unless Asprov > 4/3*Asreqd

$$M_{cr} = \frac{f_r \times I_g}{y_t}$$

$f_r = 7.5 \times \sqrt{f_c}$ $\quad f_r := 7.5 \times \sqrt{3000} = 410.792$ psi

Use wall thickness at bottom of wall for determining Mcr - Worst Case

$$I_{gwall} := \frac{12in \times t_{wallb}^3}{12} = 0.106\,ft^4 \qquad I_{gslab} := \frac{12in \times t_{slab}^3}{12} = 0.132\,ft^4$$

$$y_{twall} := \frac{t_{wallb}}{2} = 0.542\,ft \qquad y_{tslab} := \frac{t_{slab}}{2} = 0.583\,ft$$

$$M_{crwall} := \frac{f_r \times psi \times I_{gwall}}{y_{twall}} = 11.571 \times ft \times kip \quad \text{-----> } 1.2M_{crwall} = 13.885 \times ft \times kip$$

$$M_{crslab} := \frac{f_r \times psi \times I_{gslab}}{y_{tslab}} = 13.419 \times ft \times kip \quad \text{-----> } 1.2M_{crslab} = 16.103 \times ft \times kip$$

Temperature/Shrinkage Reinforcement (AASHTO 8.20.1)

$A_{s.TSreq} := 0.125in^2$ (Min area of steel per foot in each direction)

$S_{max1} := 3 \times t_{wallt} = 30 \times in$ $\qquad S_{max2} := 18in$

$S_{max} := \min(S_{max1}, S_{max2}) = 18 \times in$

$A_{No4} := 0.2$ (Area of #4 bar)

$$A_{s.TS} := A_{No4} \times in^2 \times \frac{12in}{S_{max}} = 0.133 \times in^2 \quad > \quad A_{s.TSreq} = 0.125 \times in^2 \quad \text{OK}$$

Therefore use a minimum of #4 with a max spacing of 18" o.c.
- All layers, all directions

Vertical Reinforcement - Outside Face:

Short Wall only:

$M_{u.swall} := 58.29 \times kip \times ft$ (at bottom of wall)

Try #8 at 6" o.c.

$$A_{s.swall} := \frac{0.79in^2 \times 12}{6} = 1.58 \times in^2$$

$$\rho_{swall} := \frac{A_{s.swall}}{d_{wallb} \times b} = 0.012$$

$check_{\rho swall} := if(\rho_{swall} < \rho_{max}, "OK", "NG") = "OK"$

$$\Phi M_{n.swall} := \Phi_b \times \left[A_{s.swall} \times f_y \times d_{wallb} \times \left(1 - 0.6 \times \frac{\rho_{swall} \times f_y}{f_c} \right) \right] = 64.31 \times kip \times ft$$

$check_{swall} := if(\Phi M_{n.swall} > M_{u.swall}, "OK", "NG") = "OK"$

$check2_{swall} := if(\Phi M_{n.swall} > 1.2M_{crwall}, "OK", "NG") = "OK"$

Therefore Use #8 at 6" o.c. outside face vertical rebar for the short wall.

#8@ 5" Clsd

Vertical Reinforcement - Outside Face:

High Wall (Use for Side Walls also - conservative):

$M_{u.hwall} := 29.14 \times kip \times ft$ (at bottom of wall)

Try #8 at 12" o.c.

$$A_{s.hwall} := \frac{0.79in^2 \times 12}{12} = 0.79 \times in^2$$

$$\rho_{hwall} := \frac{A_{s.hwall}}{d_{wallb} \times b} = 6.196 \times 10^{-3}$$

$check_{\rho shwall} := if(\rho_{hwall} < \rho_{max}, "OK", "NG") = "OK"$

$$\Phi M_{n.hwall} := \Phi_b \times \left[A_{s.hwall} \times f_y \times d_{wallb} \times \left(1 - 0.6 \times \frac{\rho_{hwall} \times f_y}{f_c} \right) \right] = 34.963 \times kip \times ft$$

$check_{hwall} := if(\Phi M_{n.hwall} > M_{u.hwall}, "OK", "NG") = "OK"$

$check2_{hwall} := if(\Phi M_{n.hwall} > 1.2M_{crwall}, "OK", "NG") = "OK"$

Therefore Use #8 at 12" o.c. outside face vertical rebar for the tall wall and side walls.

#8 @ 10" Clear

Vertical Reinforcement - Inside Face:

All Walls - Check worst case:

$M_{u.inwall} := 6.52 \times kip \times ft$ (at midheight of wall, use top of wall thickness)

Try #5 at 12" o.c.

$$A_{s.inwall} := \frac{0.31in^2 \times 12}{12} = 0.31 \times in^2$$

$$\rho_{inwall} := \frac{A_{s.inwall}}{d_{wallt} \times b} = 3.388 \times 10^{-3}$$

$$check_{\rho inwall} := if(\rho_{inwall} < \rho_{max}, "OK", "NG") = "OK"$$

$$\Phi M_{n.inwall} := \Phi_b \times \left[A_{s.inwall} \times f_y \times d_{wallt} \times \left(1 - 0.6 \times \frac{\rho_{inwall} \times f_y}{f_c} \right) \right] = 10.204 \times kip \times ft$$

$$check_{inwall} := if(\Phi M_{n.inwall} > M_{u.inwall}, "OK", "NG") = "OK"$$

$$check2_{inwall} := if(\Phi M_{n.inwall} > 1.2M_{crwall}, "OK", "NG") = "NG"$$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$$\frac{M_{u.inwall}}{\Phi_b \times f_c \times b \times d_{wallt}^2} = 0.042 \quad \text{-----> } \omega := 0.043 \quad \text{(Design Aid)}$$

$$A_{s.inreq} := \omega \times b \times d_{wallt} \times \frac{f_c}{f_y} = 0.197 \times in^2 \quad \text{-----> } \frac{4}{3} \times A_{s.inreq} = 0.262 \times in^2$$

$$check3_{inwall} := if\left(A_{s.inwall} > \frac{4}{3} \times A_{s.inreq}, "OK", "NG" \right) = "OK"$$

Therefore Use #5 at 12" o.c. inside face vertical rebar for all walls.

Horizontal Reinforcement - Outside Face:

Check worst case moment:

$M_{u.hout} := 23.49 \times kip \times ft$ (at corner near top of wall, use top of wall thickness)

Try #6 at 6" o.c.

$$A_{s.hout} := \frac{0.44in^2 \times 12}{6} = 0.88 \times in^2$$

$$\rho_{hout} := \frac{A_{s.hout}}{d_{wallt} \times b} = 9.617 \times 10^{-3}$$

$$check_{\rho hout} := if\left(\rho_{hout} < \rho_{max}, "OK", "NG"\right) = "OK"$$

$$\Phi M_{n.hout} := \Phi_b \times \left[A_{s.hout} \times f_y \times d_{wallt} \times \left(1 - 0.6 \times \frac{\rho_{hout} \times f_y}{f_c}\right)\right] = 26.71 \times kip \times ft$$

$$check_{hout} := if\left(\Phi M_{n.hout} > M_{u.hout}, "OK", "NG"\right) = "OK"$$

$$check2_{hout} := if\left(\Phi M_{n.hout} > 1.2M_{crwall}, "OK", "NG"\right) = "OK"$$

Therefore Use #6 at 6" o.c. outside face horizontal rebar for all walls.

Horizontal Reinforcement - Inside Face:

Check worst case moment:

$M_{u.hin} := 10.44 \times kip \times ft$ (at middle top of wall, use top of wall thickness)

Try #6 at 12" o.c.

$$A_{s.hin} := \frac{0.44in^2 \times 12}{12} = 0.44 \times in^2$$

$$\rho_{hin} := \frac{A_{s.hin}}{d_{wallt} \times b} = 4.809 \times 10^{-3}$$

$$check_{\rho hin} := if(\rho_{hin} < \rho_{max}, "OK", "NG") = "OK"$$

$$\Phi M_{n.hin} := \Phi_b \times \left[A_{s.hin} \times f_y \times d_{wallt} \times \left(1 - 0.6 \times \frac{\rho_{hin} \times f_y}{f_c} \right) \right] = 14.226 \times kip \times ft$$

$$check_{hin} := if(\Phi M_{n.hin} > M_{u.hin}, "OK", "NG") = "OK"$$

$$check2_{hin} := if(\Phi M_{n.hin} > 1.2M_{crwall}, "OK", "NG") = "OK"$$

Therefore Use #6 at 12" o.c. inside face horizontal rebar for all walls.

Slab Reinforcement - Top Face:

Check worst case moment in both directions:

$M_{u.slabt} := 7.08 \times kip \times ft$

Try #5 at 12" o.c.

$A_{s.slabt} := \frac{0.31 in^2 \times 12}{12} = 0.31 \times in^2$

$\rho_{slabt} := \frac{A_{s.slabt}}{d_{slab} \times b} = 2.431 \times 10^{-3}$

$check_{\rho slabt} := if(\rho_{slabt} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabt} := \Phi_b \times \left[A_{s.slabt} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabt} \times f_y}{f_c} \right) \right] = 14.389 \times kip \times ft$

$check_{slabt} := if(\Phi M_{n.slabt} > M_{u.slabt}, "OK", "NG") = "OK"$

$check2_{slabt} := if(\Phi M_{n.slabt} > 1.2 M_{crslab}, "OK", "NG") = "NG"$

Since ΦMn < 1.2Mcr, check if As,prov > 4/3 As,reqd

$\frac{M_{u.slabt}}{\Phi_b \times f_c \times b \times d_{slab}^2} = 0.023$ -----> $\omega := 0.024$ (Design Aid)

$A_{s.slabtreq} := \omega \times b \times d_{slab} \times \frac{f_c}{f_y} = 0.153 \times in^2$ -----> $\frac{4}{3} \times A_{s.slabtreq} = 0.204 \times in^2$

$check3_{slabt} := if\left(A_{s.slabt} > \frac{4}{3} \times A_{s.slabtreq}, "OK", "NG" \right) = "OK"$

Therefore Use #5 at 12" o.c. each way in top face of slab.

Slab Reinforcement - Bottom Face in Short Direction:

Check worst case moment from bottom of High Wall and Short Wall:

$$M_{u.slabbs} := \max\left(M_{u.hwall}, M_{u.swall}\right) = 58.29\,kip \times ft$$

Try #8 at 6" o.c.

$$A_{s.slabbs} := \frac{0.79in^2 \times 12}{6} = 1.58 \times in^2$$

$$\rho_{slabbs} := \frac{A_{s.slabbs}}{d_{slab} \times b} = 0.012$$

$$check_{\rho slabbs} := if\left(\rho_{slabbs} < \rho_{max}, "OK", "NG"\right) = "OK"$$

$$\Phi M_{n.slabbs} := \Phi_b \times \left[A_{s.slabbs} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbs} \times f_y}{f_c}\right)\right] = 64.31 \times kip \times ft$$

$$check_{slabbs} := if\left(\Phi M_{n.slabbs} > M_{u.slabbs}, "OK", "NG"\right) = "OK"$$

$$check2_{slabbs} := if\left(\Phi M_{n.slabbs} > 1.2M_{crslab}, "OK", "NG"\right) = "OK"$$

Therefore Use #8 at 6" o.c. in short direction in bottom face of slab.

#8@6" Clear OK

Slab Reinforcement - Bottom Face in Long Direction:

Check worst case moment from bottom of Side Walls:

$M_{u.slabbl} := 12.18kip \times ft$

Try #6 at 12" o.c.

$A_{s.slabbl} := \frac{0.44in^2 \times 12}{12} = 0.44 \times in^2$

$\rho_{slabbl} := \frac{A_{s.slabbl}}{d_{slab} \times b} = 3.451 \times 10^{-3}$

$check_{\rho slabbl} := if(\rho_{slabbl} < \rho_{max}, "OK", "NG") = "OK"$

$\Phi M_{n.slabbl} := \Phi_b \times \left[A_{s.slabbl} \times f_y \times d_{slab} \times \left(1 - 0.6 \times \frac{\rho_{slabbl} \times f_y}{f_c} \right) \right] = 20.166 \times kip \times ft$

$check_{slabbl} := if(\Phi M_{n.slabbl} > M_{u.slabbl}, "OK", "NG") = "OK"$

$check2_{slabbl} := if(\Phi M_{n.slabbl} > 1.2M_{crslab}, "OK", "NG") = "OK"$

Therefore Use #6 at 12" o.c. in long direction in bottom face of slab.

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2091024

TITLE Oak Hill Basin 2 Storm Drain

SUBJECT Inlet Structures

MADE BY JPH DATE 11-29-10 CHECKED BY Raj D DATE 12/26/10

Trash Rack Bars

Design trash rack for 50 psf live load

Max Span = 6'-0" (Inlet Structure 1 and 1b)

Try 2" x 1/2" bars at 4" o.c. Note: 3" x 3/8" member used with 1" x 1/4" Chase Bars

Deff = 2"

OK

$w_D = 3.4$ plf

$w_L = (50\ psf)(4/12) = 16.7$ plf

$w_T = 20.1$ plf

$M = \frac{wL^2}{8} = \frac{(20.1\ plf)(6')^2}{8} = 90.5$ ft-lb $= 1085$ in-lb

$I = \frac{bh^3}{12} = \frac{(0.5)(2")^3}{12} = 0.33\ in^4$

$S = \frac{I}{y} = \frac{0.33\ in^4}{1"} = 0.33\ in^3$

$f_b = \frac{M}{S} = \frac{1085\ in\text{-}lb}{0.33\ in^3} = 3288$ psi

Allowable Stress per AASHTO T&L 10.32.1A

$F_y = 30$ ksi

$$F_b = \frac{50 \times 10^6 C_b}{S_{xc}} \left(\frac{I_{yc}}{L}\right) \sqrt{0.772 \frac{D}{I_{yc}} + 9.87 \left(\frac{d}{L}\right)^2} \leq 0.55 F_y = 19.8\ ksi$$

$C_b = 1.75 + 1.05 (M_1/M_2) + 0.3 (M_1/M_2)^2 \leq 2.3$

Unbraced flat length

$M_1 = M_2 = 0$

$C_b = 1.75$

$I = \frac{1}{3} bh^3 = \frac{(0.5)(2")^3}{3} = 1.33\ in^4$

$$F_b = \frac{70 \times 10^6 (1.75)}{0.33\ in^3} \left(\frac{0.33\ in^4}{72"^2}\right) \sqrt{0.772 \times \frac{1.33\ in^4}{72"} + 9.87 \left(\frac{2"}{72"}\right)^2} = 18.0\ ksi$$

Use $F_b = 0.55 F_y = 19.8$ ksi

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2009034

TITLE Oak St. Basin & Storm Drain

SUBJECT Inlet Structure

MADE BY JPH DATE 11-29-10 CHECKED BY RJ DATE 12/26/10

Trash Rack Bars - cont.

$V = w_1\left(\frac{L}{2}\right) = (20.1\ plf)\left(\frac{6'}{2}\right) = 60.3\ lb$

$f_v = \frac{V}{bt} = \frac{60.3\ lb}{(0.5")(2")} = 60.3\ psi$

$F_v = 0.33\ F_y = 0.33(36\ ksi) = 11.88\ ksi > f_v = 60.3\ psi$ OK

∴ Use 3"x3/8" steel bars for trash rack

Check Deflection

$\Delta = \frac{wL^4}{384EI} = \frac{(20.1\ plf)(6)^4(1728)}{384(29\times10^6\ psi)(0.33\ in^4)} = 0.012" \leq \frac{L}{600}$ OK

3.0
SPECIAL CURB DESIGN

PREMIER
ENGINEERING CORPORATION

SHEET NO 1 OF _____

JOB NUMBER 2009034

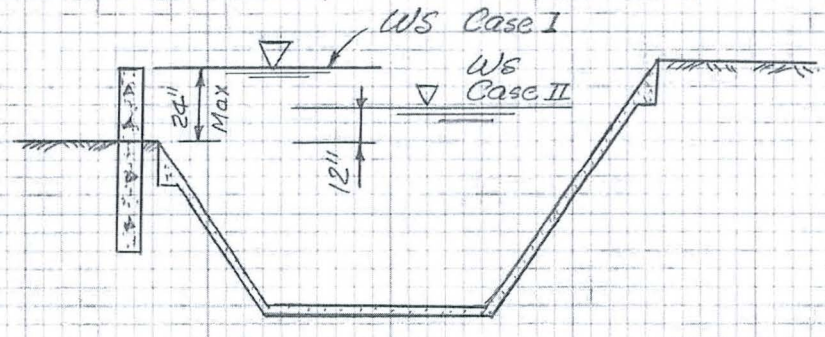
TITLE Oak Street Basin

SUBJECT Special Curb Wall

MADE BY RMD DATE 11/2/10 CHECKED BY SEO DATE 4/12/11

Special Curb Wall Data

- Wall height varies from 6" to 24" above finish ground @ Canal.



Case I: Wall height 24". Water depth = 24"
Water overtopping Special Curb.

Case II: Wall height 24". Water depth = 12".
Water 12" above Canal Bank.

Soil Data (Assumed)

Relatively poor soil.
γ = 105 pcf. = 0.06076 pci
Angle of internal friction = 23°
Modulus of subgrade reaction = 75 pci.

Water Data

Unit wt = 62.4 lbs/c.f.
Coefficient of active pressure = Ka = 1.0

Wind

Use 15 lbs/SFt.

Concrete

f_c' = 2500 psi.
No reinforcement.

Use AASHTO 17th edition 2002 design criteria.

PREMIER
ENGINEERING CORPORATION

SHEET NO 2 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

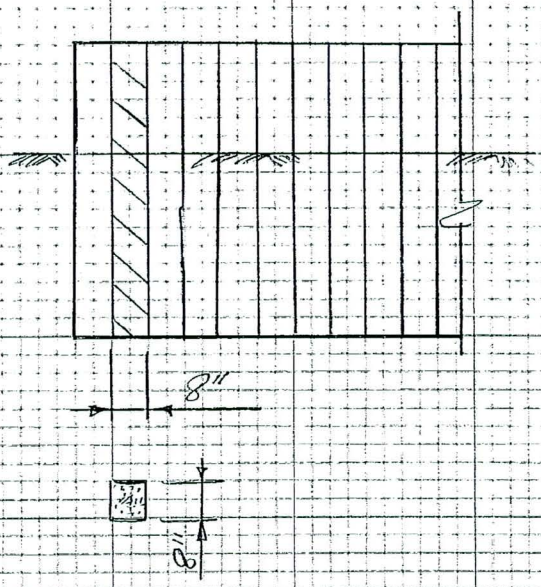
SUBJECT Special Curb. wall

MADE BY RMD DATE 11/2/10 CHECKED BY SEO DATE 12 APR 11

Case I Analysis (Water depth 24" above canal top)

Use 8" thick wall.

Assume 8" width for analysis.



Note: Neglect top 6" soil (Disturbed)

Section Properties

Area = 8x8 = 64 Sq. in.

Moment of Inertia

$I = \frac{1}{12}(8)(8)^3 = 341 \text{ in}^4$

Section Modulus

$S = \frac{I}{d/2} = \frac{341}{4}$

$S = 85.33 \text{ in}^3$

Modulus of Elasticity

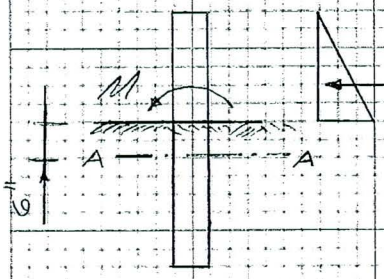
$E_c = 57000\sqrt{f_c'}$

$= 57000\sqrt{2500}$

$E_c = 2850000 \text{ psi}$.

Forces on wall

Maximum Water Pressure:



$h = 24" = 2'$

$F = \frac{1}{2}(62.4)(2)^2\left(\frac{8}{12}\right) = 83.2 \text{ lbs.}$

Moment @ A-A 6" below GL

$M = F \times a = 83.2 \times \left(\frac{1}{3} \times 24 + 6"\right)$

$M = 1165 \text{ lb.in.}$

Wind:

Uniform $p = 15 \text{ lbs/sf}$

On 8" width, $15 \times \frac{8}{12} = 10 \text{ lbs/ft.}$

Force $= 10 \times 2 = 20 \text{ lbs}$

Moment $= 20 \times \left(\frac{2}{2} \times 12 + 6"\right) = 360 \text{ lb.in.}$ @ A-A

Does not Control.

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ENGINEERING CORPORATION

SHEET NO 3 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

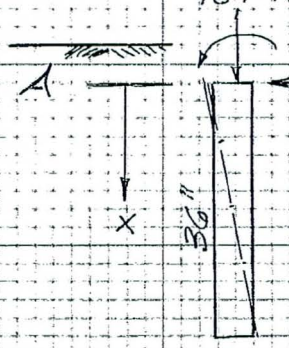
SUBJECT Special Curb wall

MADE BY LMD DATE 11/2/10 CHECKED BY SEO DATE 12 APRIL

Wall above A-A

$\frac{8" \times 8"}{144} \times 2.5 \times 150 = 167$ lbs

L-Pile Analysis Model:



Close spacing of 8"x8" units
Use capacity reduction factor
P-multiplier = 0.50
y-multiplier = 1
Stable @ depth 36" Use

Results

Max Pile head deflection = 0.50"
Maximum Moment = 2070 lb.in.
Maximum Shear = 179 lbs

Check tensile stress

M = 2070 lb.in
S = 85.33 in³
$f = M/S = \frac{2070}{85.33} = 25$ psi. Small
f_{cr} allowable $= 7.5\sqrt{f'_c}$
$= 7.5\sqrt{2500} = 375$ psi. > 25 psi.
Reinforcement not required.

Use 8" thick wall
Provide 3.5' depth below GL (Canal top)

8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpo

LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

(c) 1985-2007 by Ensoft, Inc.
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This program is licensed to:

XX
XX

Path to file locations: P:\2009034\Engineering\Calculations\Rajesh D\L-pile\
Name of input data file: 8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpd
Name of output file: 8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpo
Name of plot output file: 8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpp
Name of runtime file: 8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpr

Time and Date of Analysis

Date: November 4, 2010 Time: 11:51:11

Problem Title

8" thick special curb- Case I: 24" water depth above canal bank Neglect 6" soil

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 50
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 36.00 in

Depth of ground surface below top of pile = .00 in

8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpo

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	8.00000000	341.0000	64.0000	2850000.
2	36.0000	8.00000000	341.0000	64.0000	2850000.

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 36.000 in
p-y subgrade modulus k for top of soil layer = 75.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 75.000 lbs/in**3

(Depth of lowest layer extends .00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.06076
2	36.00	.06076

Shear Strength of Soils

Shear strength parameters with depth defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	23.00	------	------
2	36.000	.00000	23.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	.000	.5000	1.0000
2	36.000	.5000	1.0000

8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpo

Loading Type

Cyclic loading criteria was used for computation of p-y curves.

Number of cycles of loading = 10.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 83.200 lbs
Bending moment at pile head = 1165.000 in-lbs
Axial load at pile head = 167.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 83.200 lbs
Specified moment at pile head = 1165.000 in-lbs
Specified axial load at pile head = 167.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. p lbs/in	Es*h F/L lbs/in
0.000	.501147	1165.0000	83.2000	-.0185336	16.2751	0.0000	0.0000
.720	.487803	1227.1325	83.1046	-.0185327	17.0039	-.2649983	.3911387
1.440	.474460	1289.1274	82.8012	-.0185318	17.7311	-.5778889	.8769546
2.160	.461118	1350.8227	82.2541	-.0185308	18.4548	-.9417669	1.4705
2.880	.447776	1412.0296	81.4255	-.0185298	19.1728	-1.3597	2.1864
3.600	.434435	1472.5315	80.2755	-.0185287	19.8825	-1.8349	3.0410
4.320	.421094	1532.0821	78.7645	-.0185276	20.5810	-2.3622	4.0390
5.040	.407755	1590.4080	76.8566	-.0185265	21.2652	-2.9374	5.1868
5.760	.394416	1647.2109	74.5136	-.0185253	21.9315	-3.5709	6.5186
6.480	.381079	1702.1626	71.6927	-.0185240	22.5761	-4.2651	8.0583
7.200	.367742	1754.9030	68.3492	-.0185227	23.1947	-5.0223	9.8332
7.920	.354406	1805.0398	64.4370	-.0185214	23.7829	-5.8450	11.8746
8.640	.341071	1852.1463	59.9434	-.0185201	24.3354	-6.6370	14.0107
9.360	.327737	1895.8121	54.8676	-.0185187	24.8476	-7.4625	16.3942
10.080	.314404	1935.6091	49.1812	-.0185173	25.3145	-8.3330	19.0831
10.800	.301072	1971.0861	42.8516	-.0185158	25.7306	-9.2494	22.1195
11.520	.287741	2001.7680	35.9063	-.0185143	26.0905	-10.0431	25.1303
12.240	.274411	2027.2434	28.4150	-.0185128	26.3894	-10.7661	28.2480
12.960	.261083	2047.1375	20.4537	-.0185113	26.6227	-11.3485	31.2963
13.680	.247755	2061.1483	12.0773	-.0185098	26.7871	-11.9193	34.6387
14.400	.234429	2068.9800	3.2940	-.0185083	26.8789	-12.4787	38.3258
15.120	.221103	2070.3426	-5.8880	-.0185068	26.8949	-13.0269	42.4209
15.840	.207779	2064.9517	-15.4610	-.0185052	26.8317	-13.5647	47.0047
16.560	.194456	2052.5288	-25.4215	-.0185037	26.6860	-14.1031	52.2189
17.280	.181134	2032.7946	-35.7618	-.0185022	26.4545	-14.6202	58.1147
18.000	.167812	2005.4811	-46.4642	-.0185007	26.1341	-15.1087	64.8239
18.720	.154493	1970.3352	-57.5072	-.0184992	25.7218	-15.5663	72.5453
19.440	.141174	1927.1194	-68.8676	-.0184978	25.2149	-15.9904	81.5526
20.160	.127856	1875.6142	-80.5130	-.0184964	24.6107	-16.3581	92.1181
20.880	.114539	1815.6287	-92.3702	-.0184950	23.9071	-16.5785	104.2137
21.600	.101223	1747.0487	-104.3432	-.0184937	23.1026	-16.6799	118.6441
22.320	.087908	1669.8218	-116.3372	-.0184924	22.1967	-16.6368	136.2615
23.040	.074594	1583.9702	-128.2362	-.0184912	21.1897	-16.4158	158.4500

8 in uniform Floodwall 2 ft water neglect 6 in soil along Hawes Rd.lpo

23.760	.061281	1489.6085	-139.8954	-.0184901	20.0828	-15.9709	187.6463
24.480	.047968	1386.9673	-151.1033	-.0184890	18.8788	-15.1620	227.5814
25.200	.034656	1276.4660	-161.5566	-.0184880	17.5826	-13.8751	288.2592
25.920	.021345	1158.7718	-170.8340	-.0184871	16.2020	-11.8954	401.2415
26.640	.008035	1034.9109	-178.0060	-.0184863	14.7491	-8.0270	719.2800
27.360	-.005275	906.8887	-178.9474	-.0184856	13.2474	5.4120	738.7200
28.080	-.018584	781.6720	-173.0917	-.0184850	11.7785	10.8539	420.5093
28.800	-.031893	662.0819	-164.4367	-.0184844	10.3757	13.1876	297.7157
29.520	-.045202	549.3283	-154.2579	-.0184840	9.0531	15.0868	240.3107
30.240	-.058510	444.3955	-142.7793	-.0184836	7.8222	16.7983	206.7125
30.960	-.071818	348.1710	-130.0985	-.0184833	6.6935	18.4261	184.7279
31.680	-.085126	261.4985	-116.2554	-.0184831	5.6768	20.0271	169.3900
32.400	-.098434	185.2081	-101.1083	-.0184829	4.7819	22.0482	161.2725
33.120	-.111741	120.3474	-84.3811	-.0184828	4.0211	24.4161	157.3241
33.840	-.125049	68.1440	-65.9403	-.0184827	3.4087	26.8085	154.3564
34.560	-.138357	29.8382	-45.7642	-.0184827	2.9594	29.2362	152.1437
35.280	-.151664	6.6883	-23.8075	-.0184827	2.6878	31.7544	150.7485
36.000	-.164972	0.0000	0.0000	-.0184827	2.6094	34.3777	75.0188

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.50114730	in
Computed slope at pile head	=	-.01853362	
Maximum bending moment	=	2070.34255	lbs-in
Maximum shear force	=	-178.94742	lbs
Depth of maximum bending moment	=	15.12000000	in
Depth of maximum shear force	=	27.36000000	in
Number of iterations	=	38	
Number of zero deflection points	=	1	

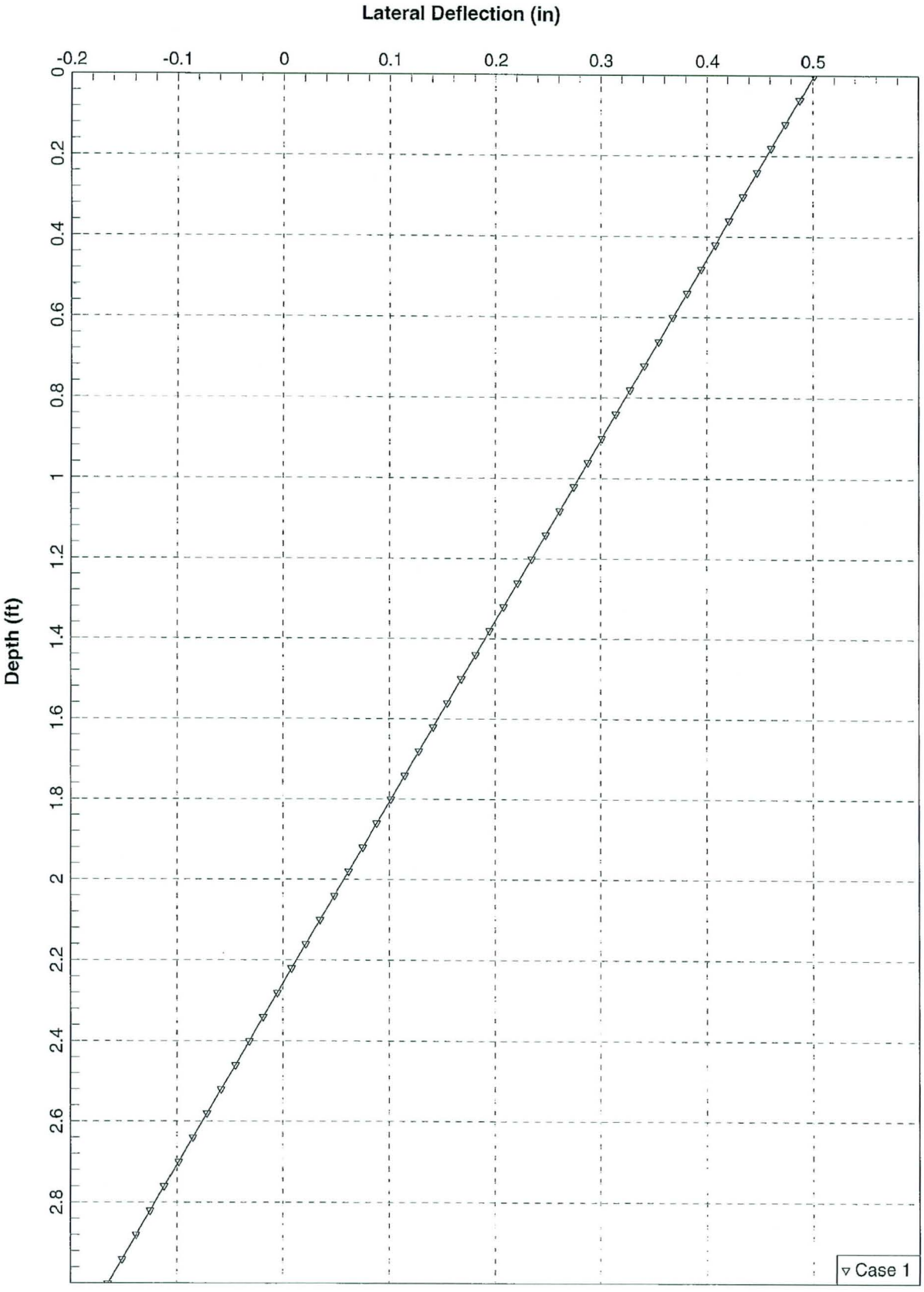
Summary of Pile Response(s)

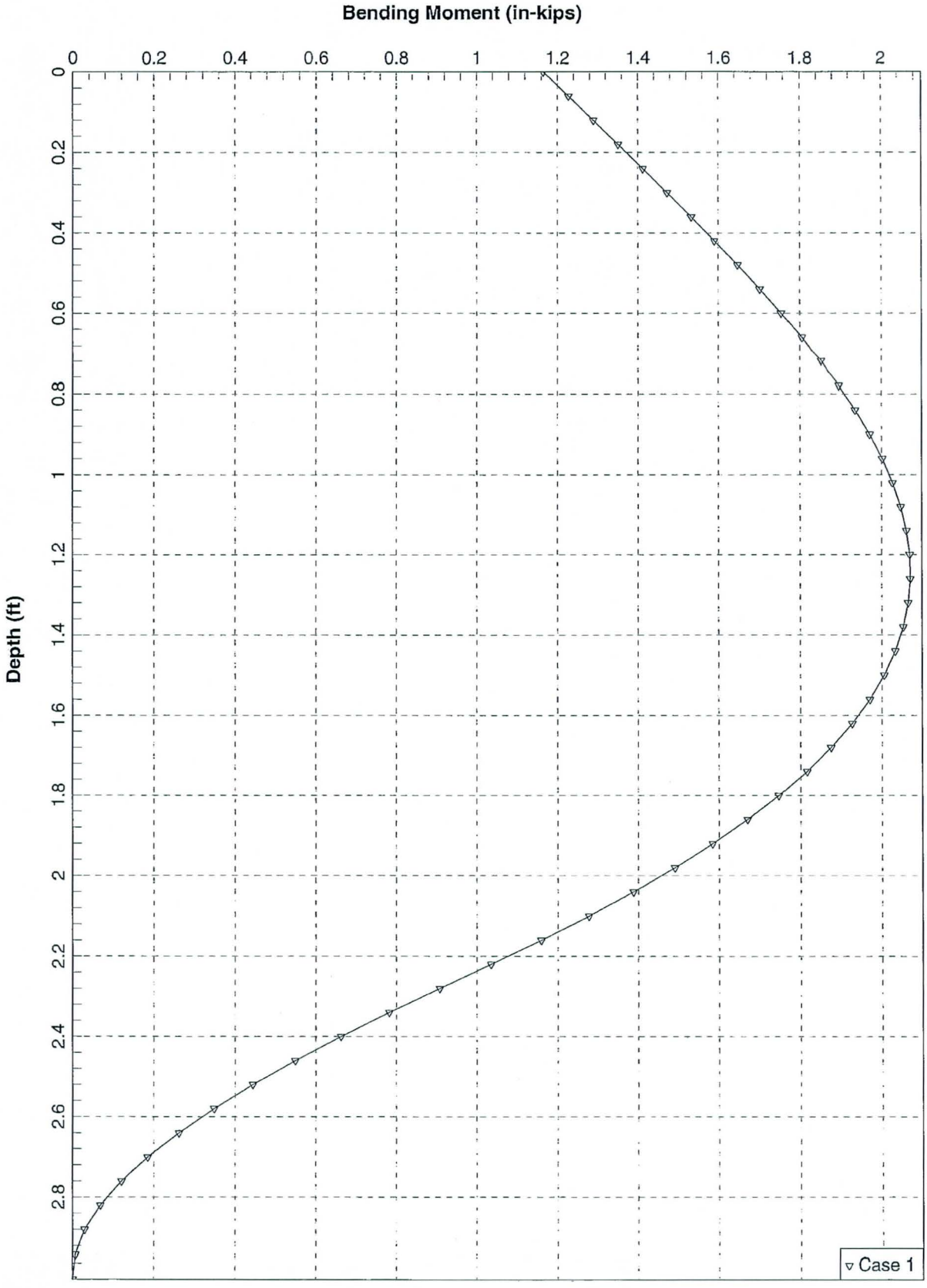
Definition of Symbols for Pile-Head Loading Conditions:

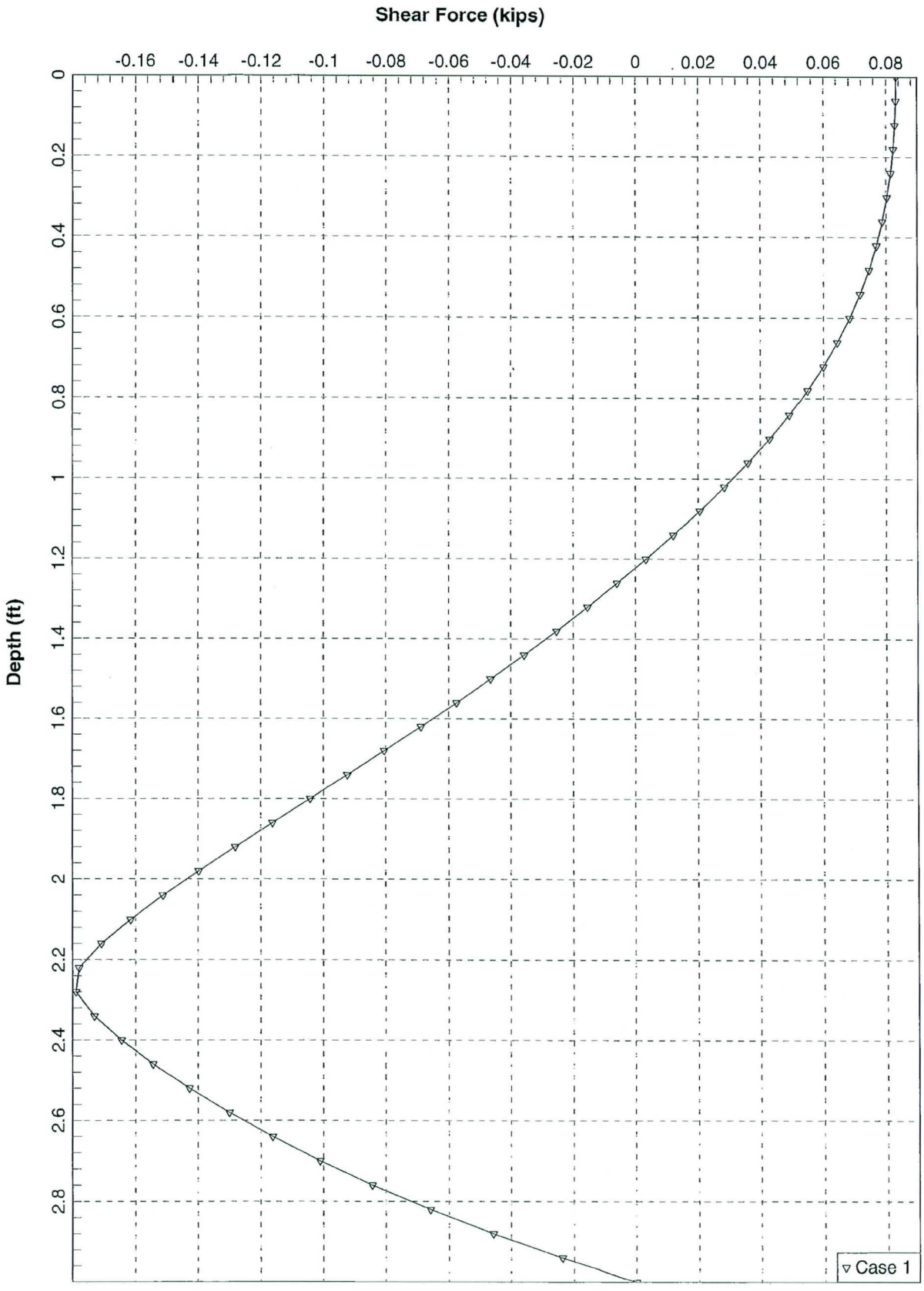
Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 83.200	M= 1165.000	167.0000	.5011473	2070.3426	-178.9474

The analysis ended normally.


Lateral Deflection (in)
-0.2
-0.1
0
0.1
0.2
0.3
0.4
0.5
Depth (ft)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
2.6
2.8
▽ Case 1


Bending Moment (in-kips)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Depth (ft)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
2.6
2.8
Case 1


Shear Force (kips)
-0.16
-0.14
-0.12
-0.1
-0.08
-0.06
-0.04
-0.02
0
0.02
0.04
0.06
0.08
Depth (ft)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2
2.4
2.6
2.8
Case 1

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

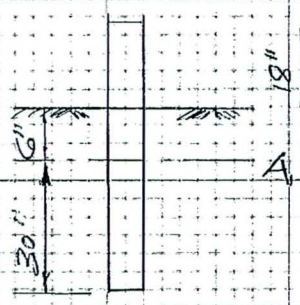
JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Special Curb

MADE BY Raj. D DATE 11/3/10 CHECKED BY SEO DATE 12 APR 11

Case II: Water depth 18" Wall 18"



Water:

$F = \frac{1}{2}(0.667 \times 0.0624)(1.5)^2 = 0.047$ k

$M = F\left(\frac{1.5}{3} + 0.5\right) = 0.047$ k-ft

F = 47 lbs, M = 561 lb-in.

Wind

Uniform p = 15 lbs/sf

on 8" wall $0.67 \times 15 = 10$ lbs/sf.

$F = 10 \times 1.5 = 15$ lbs

$M = F(0.5 \times 1.50 + 0.5)(12)$

M = 225 lb-in

Use F = 47 lbs M = 561 lb-in

L-pile results:

Max. Pile top deflection $\Delta = 0.294$

Max. Bending Moment = 990 lb-in

Max. Shear = 107 lbs.

Reinforcement is not reqd.

Use 8" wall.

Provide 3'-0 embedment in Board.

8 in uniform Floodwall 18 inches water neglect 6 in soil along Hawes Rd.lpo

LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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This program is licensed to:

XX
XX

Path to file locations: P:\2009034\Engineering\Calculations\Rajesh D\L-pile\
Name of input data file: 8 in uniform Floodwall 18 inches water neglect 6 in soil along Hawes Rd.lpd
Name of output file: 8 in uniform Floodwall 18 inches water neglect 6 in soil along Hawes Rd.lpo
Name of plot output file: 8 in uniform Floodwall 18 inches water neglect 6 in soil along Hawes Rd.lpp
Name of runtime file: 8 in uniform Floodwall 18 inches water neglect 6 in soil along Hawes Rd.lpr

Time and Date of Analysis

Date: December 15, 2010 Time: 13:59:07

Problem Title

8" uniform thickness special curb- Case II: 18" water depth above canal bank

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multiplers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 50
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 30.00 in

Depth of ground surface below top of pile = .00 in

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	8.00000000	341.0000	64.0000	2850000.
2	30.0000	8.00000000	341.0000	64.0000	2850000.

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 30.000 in
p-y subgrade modulus k for top of soil layer = 75.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 75.000 lbs/in**3

(Depth of lowest layer extends .00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.06076
2	30.00	.06076

Shear Strength of Soils

Shear strength parameters with depth defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	23.00	------	------
2	30.000	.00000	23.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	.000	.5000	1.0000
2	30.000	.5000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves.

Number of cycles of loading = 10.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 47.000 lbs
Bending moment at pile head = 561.000 in-lbs
Axial load at pile head = 134.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 47.000 lbs
Specified moment at pile head = 561.000 in-lbs
Specified axial load at pile head = 134.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. p lbs/in	Es*h F/L lbs/in
0.000	.283978	560.9999	47.0000	-.0125695	8.6744	0.0000	0.0000
.600	.276436	590.2106	46.9374	-.0125692	9.0170	-.2089379	.4534957
1.200	.268895	619.3459	46.7413	-.0125688	9.3588	-.4445004	.9918376
1.800	.261354	648.3212	46.3955	-.0125684	9.6987	-.7079646	1.6253
2.400	.253813	677.0416	45.8830	-.0125680	10.0356	-1.0006	2.3654
3.000	.246272	705.4017	45.1856	-.0125676	10.3683	-1.3238	3.2252
3.600	.238732	733.2852	44.2849	-.0125671	10.6953	-1.6788	4.2193
4.200	.231192	760.5643	43.1632	-.0125667	11.0153	-2.0602	5.3466
4.800	.223652	787.1017	41.8082	-.0125662	11.3266	-2.4564	6.5899
5.400	.216112	812.7548	40.2077	-.0125657	11.6275	-2.8787	7.9922
6.000	.208573	837.3715	38.3460	-.0125652	11.9163	-3.3270	9.5707
6.600	.201034	860.7904	36.2075	-.0125646	12.1910	-3.8013	11.3453
7.200	.193496	882.8409	33.7766	-.0125641	12.4497	-4.3016	13.3386
7.800	.185957	903.3427	31.0378	-.0125636	12.6901	-4.8277	15.5768
8.400	.178419	922.1065	27.9909	-.0125630	12.9102	-5.3288	17.9200
9.000	.170882	938.9519	24.6466	-.0125624	13.1078	-5.8188	20.4311
9.600	.163344	953.7024	21.0048	-.0125618	13.2809	-6.3204	23.2161
10.200	.155807	966.1776	17.0589	-.0125612	13.4272	-6.8327	26.3122
10.800	.148271	976.1930	12.8025	-.0125606	13.5447	-7.3552	29.7640
11.400	.140735	983.5604	8.2298	-.0125600	13.6311	-7.8872	33.6257
12.000	.133199	988.0884	3.3353	-.0125594	13.6842	-8.4279	37.9637
12.600	.125663	989.5823	-1.8531	-.0125588	13.7018	-8.8670	42.3368
13.200	.118128	987.8841	-7.3031	-.0125582	13.6818	-9.2997	47.2352
13.800	.110594	982.8379	-13.0104	-.0125576	13.6226	-9.7246	52.7584
14.400	.103059	974.2908	-18.9698	-.0125570	13.5224	-10.1400	59.0342
15.000	.095525	962.0933	-25.1751	-.0125564	13.3793	-10.5442	66.2291
15.600	.087991	946.0998	-31.6189	-.0125558	13.1917	-10.9351	74.5645
16.200	.080458	926.1696	-38.2867	-.0125552	12.9579	-11.2911	84.2013
16.800	.072925	902.1746	-45.1475	-.0125547	12.6764	-11.5781	95.2600
17.400	.065393	874.0114	-52.1671	-.0125541	12.3461	-11.8205	108.4577
18.000	.057860	841.5928	-59.3160	-.0125536	11.9658	-12.0091	124.5317
18.600	.050328	804.8509	-66.5581	-.0125531	11.5348	-12.1312	144.6248
19.200	.042797	763.7416	-73.8484	-.0125526	11.0526	-12.1698	170.6185

8 in uniform Floodwall 18 inches water neglect 6 in soil along Hawes Rd.lpo

19.800	.035265	718.2513	-81.1293	-.0125521	10.5190	-12.0999	205.8668
20.400	.027734	668.4049	-88.2937	-.0125517	9.9343	-11.7815	254.8827
21.000	.020203	614.3172	-95.1784	-.0125513	9.2998	-11.1674	331.6551
21.600	.012672	556.2091	-101.5718	-.0125510	8.6182	-10.1440	480.2847
22.200	.005142	494.4492	-105.8992	-.0125506	7.8937	-4.2807	499.5000
22.800	-.002388	431.1482	-106.5708	-.0125503	7.1512	2.0420	513.0000
23.400	-.009918	368.5823	-103.3472	-.0125501	6.4173	8.7034	526.5000
24.000	-.017448	309.1497	-97.3160	-.0125499	5.7201	11.4003	392.0226
24.600	-.024978	253.8211	-90.1208	-.0125497	5.0711	12.5838	302.2733
25.200	-.032508	203.0227	-82.2631	-.0125496	4.4752	13.6086	251.1726
25.800	-.040038	157.1234	-73.8160	-.0125495	3.9368	14.5484	218.0199
26.400	-.047567	116.4614	-64.8195	-.0125494	3.4599	15.4399	194.7533
27.000	-.055097	81.3579	-55.2963	-.0125493	3.0481	16.3041	177.5498
27.600	-.062627	52.1238	-45.2587	-.0125493	2.7052	17.1548	164.3527
28.200	-.070156	29.0654	-34.7118	-.0125492	2.4347	18.0013	153.9537
28.800	-.077686	12.4875	-23.6561	-.0125492	2.2402	18.8511	145.5951
29.400	-.085215	2.6959	-12.0878	-.0125492	2.1254	19.7099	138.7773
30.000	-.092745	0.0000	0.0000	-.0125492	2.0938	20.5828	66.5788

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.28397807 in
Computed slope at pile head	=	-.01256951
Maximum bending moment	=	989.58233 lbs-in
Maximum shear force	=	-106.57080 lbs
Depth of maximum bending moment	=	12.60000000 in
Depth of maximum shear force	=	22.80000000 in
Number of iterations	=	35
Number of zero deflection points	=	1

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 47.000	M= 561.000	134.0000	.2839781	989.5823	-106.5708

The analysis ended normally.

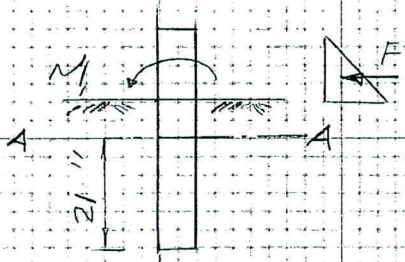
Case III: Wall h = 12"

Water depth = 12" above Bank.

$F = \frac{1}{2}(62.4)(1)^2(\frac{8}{12}) = 20.8$ lbs.

$M = 20.8 \times (\frac{1}{3} \times 12 + 6")$

$M = 208$ lb. in



Wind (Only)

Min Sim p = 15 lbs/sf

On 8" wall $\frac{15 \times 8}{12} = 10$ lbs/sf } Does not Controls

Force = 10 x 1 = 10 lbs

Moment = 10 x (6 + 6) = 120 lb.in)

Use

F = 21 lbs M = 208 lb. in.

L-pile analysis Results

Max. Pile head deflection = 0.48"

Max. Bending Moment = 358 lb. in

Max. Shear = 59 lbs.

Reinforcement is not required.

Use 8" wall

Provide 2 ft 3" embedment in ground.

8 in uniform Floodwall 12 inches water neglect 6 in soil along Hawes Rd.lpo

LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

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Path to file locations: P:\2009034\Engineering\Calculations\Rajesh D\L-pile\
Name of input data file: 8 in uniform Floodwall 12 inches water neglect 6 in soil along Hawes Rd.lpd
Name of output file: 8 in uniform Floodwall 12 inches water neglect 6 in soil along Hawes Rd.lpo
Name of plot output file: 8 in uniform Floodwall 12 inches water neglect 6 in soil along Hawes Rd.lpp
Name of runtime file: 8 in uniform Floodwall 12 inches water neglect 6 in soil along Hawes Rd.lpr

Time and Date of Analysis

Date: December 15, 2010 Time: 14:04:02

Problem Title

8" uniform thickness special curb- Case III: 12" water depth above canal bank

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 50
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 21.00 in

Depth of ground surface below top of pile = .00 in

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	8.00000000	341.0000	64.0000	2850000.
2	21.0000	8.00000000	341.0000	64.0000	2850000.

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 27.000 in
p-y subgrade modulus k for top of soil layer = 75.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 75.000 lbs/in**3

(Depth of lowest layer extends 6.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.06076
2	27.00	.06076

Shear Strength of Soils

Shear strength parameters with depth defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	23.00	------	------
2	27.000	.00000	23.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	.000	.5000	1.0000
2	21.000	.5000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves.

Number of cycles of loading = 10.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 21.000 lbs
Bending moment at pile head = 208.000 in-lbs
Axial load at pile head = 100.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 21.000 lbs
Specified moment at pile head = 208.000 in-lbs
Specified axial load at pile head = 100.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. p lbs/in	Es*h F/L lbs/in
0.000	.481661	207.9993	21.0000	-.0297043	4.0024	0.0000	0.0000
.420	.469185	218.0646	20.9660	-.0297042	4.1204	-.1489395	.1333261
.840	.456709	228.1059	20.8716	-.0297041	4.2382	-.3137370	.2885195
1.260	.444234	238.0918	20.7017	-.0297040	4.3554	-.4950066	.4680033
1.680	.431758	247.9905	20.4522	-.0297039	4.4715	-.6933627	.6744806
2.100	.419282	257.7668	20.1156	-.0297038	4.5862	-.9094198	.9109765
2.520	.406807	267.3827	19.6844	-.0297037	4.6990	-1.1438	1.1809
2.940	.394331	276.7968	19.1508	-.0297035	4.8094	-1.3971	1.4880
3.360	.381856	285.9645	18.5067	-.0297034	4.9169	-1.6699	1.8368
3.780	.369380	294.8375	17.7438	-.0297033	5.0210	-1.9629	2.2319
4.200	.356905	303.3644	16.8545	-.0297032	5.1210	-2.2719	2.6735
4.620	.344430	311.4904	15.8325	-.0297030	5.2163	-2.5950	3.1643
5.040	.331954	319.1587	14.6707	-.0297029	5.3063	-2.9374	3.7165
5.460	.319479	326.3088	13.3609	-.0297028	5.3902	-3.2997	4.3379
5.880	.307004	332.8769	11.8947	-.0297026	5.4672	-3.6823	5.0376
6.300	.294529	338.7954	10.2697	-.0297025	5.5366	-4.0555	5.7832
6.720	.282054	343.9984	8.4938	-.0297023	5.5977	-4.4012	6.5537
7.140	.269579	348.4251	6.5713	-.0297022	5.6496	-4.7535	7.4059
7.560	.257104	352.0133	4.4996	-.0297020	5.6917	-5.1119	8.3507
7.980	.244629	354.6998	2.2762	-.0297019	5.7232	-5.4757	9.4011
8.400	.232155	356.4203	-.0895309	-.0297017	5.7434	-5.7897	10.4744
8.820	.219680	357.1195	-2.5862	-.0297016	5.7516	-6.0992	11.6609
9.240	.207205	356.7428	-5.2123	-.0297014	5.7472	-6.4062	12.9852
9.660	.194731	355.2361	-7.9667	-.0297013	5.7295	-6.7099	14.4720
10.080	.182256	352.5457	-10.8478	-.0297011	5.6979	-7.0097	16.1534
10.500	.169782	348.6188	-13.8539	-.0297010	5.6519	-7.3048	18.0704
10.920	.157307	343.4033	-16.9828	-.0297008	5.5907	-7.5947	20.2773
11.340	.144833	336.8481	-20.2321	-.0297007	5.5138	-7.8785	22.8468
11.760	.132359	328.9032	-23.5993	-.0297005	5.4206	-8.1556	25.8792
12.180	.119885	319.5196	-27.0733	-.0297004	5.3105	-8.3872	29.3834
12.600	.107411	308.6565	-30.6310	-.0297002	5.1831	-8.5544	33.4498
13.020	.094937	296.2843	-34.2527	-.0297001	5.0380	-8.6918	38.4524
13.440	.082463	282.3790	-37.9243	-.0297000	4.8749	-8.7919	44.7792

13.860	.069989	266.9227	-41.6279	-.0296999	4.6936	-8.8445	53.0756
14.280	.057515	249.9063	-45.3403	-.0296997	4.4940	-8.8335	64.5067
14.700	.045041	231.3317	-49.0292	-.0296996	4.2761	-8.7326	81.4306
15.120	.032567	211.2166	-52.6464	-.0296995	4.0401	-8.4922	109.5201
15.540	.020093	189.6034	-56.1094	-.0296995	3.7866	-7.9980	167.1787
15.960	.007619	166.5794	-58.7466	-.0296994	3.5165	-4.5602	251.3700
16.380	-.004854	142.7511	-59.0780	-.0296993	3.2370	2.9818	257.9850
16.800	-.017328	119.4486	-56.6472	-.0296993	2.9637	8.5938	208.2976
17.220	-.029802	97.6622	-52.7626	-.0296992	2.7081	9.9041	139.5805
17.640	-.042275	77.6228	-48.3793	-.0296992	2.4730	10.9686	108.9713
18.060	-.054749	59.5183	-43.5732	-.0296991	2.2607	11.9176	91.4242
18.480	-.067223	43.5160	-38.3821	-.0296991	2.0730	12.8020	79.9856
18.900	-.079696	29.7720	-32.8278	-.0296991	1.9117	13.6471	71.9202
19.320	-.092170	18.4354	-26.9238	-.0296991	1.7788	14.4671	65.9238
19.740	-.104644	9.6507	-20.6788	-.0296991	1.6757	15.2711	61.2924
20.160	-.117117	3.5599	-14.1024	-.0296991	1.6043	16.0450	57.5400
20.580	-.129591	.2994465	-7.2079	-.0296991	1.5660	16.7859	54.4025
21.000	-.142064	0.0000	0.0000	-.0296991	1.5625	17.5375	25.9240

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.48166077 in
Computed slope at pile head	=	-.02970428
Maximum bending moment	=	357.11951 lbs-in
Maximum shear force	=	-59.07803645 lbs
Depth of maximum bending moment	=	8.82000000 in
Depth of maximum shear force	=	16.38000000 in
Number of iterations	=	64
Number of zero deflection points	=	1

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 21.000	M= 208.000	100.0000	.4816608	357.1195	-59.0780

The analysis ended normally.

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2009039

TITLE Oak Street Basin

SUBJECT Special Curb

MADE BY Raj D DATE 11/3/10 CHECKED BY SEO DATE 12 Apr 11

Case IV : Water depth 6" Wall 6"

Try pile depth 1'-3".

Water:

$F = \frac{1}{2}(0.67 \times 62.4)(0.5)^2 = 5.23$ lbs

$M = F(0.17 + 0.5)(12) = 42$ lb.in.

Wind

$F = 10 \times 0.5 = 5$ lbs

$M = 5(0.5 \times 0.6 + 0.6)(12) = 45$ lb in

Use $F = 5$ lbs $M = 45$ lb.in

L-pile Results

Pile top deflection = 0.05

Max. Moment = 68 lb in Small

Max. Shear = 13 lbs

Embedment = 15" + 6" = 21"

Use 8" wall.

LPILE Plus for Windows, Version 5.0 (5.0.39)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

(c) 1985-2007 by Ensoft, Inc.
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This program is licensed to:

XX
XX

Path to file locations: P:\2009034\Engineering\Calculations\Rajesh D\L-pile\
Name of input data file: 8 in uniform Floodwall 6 inches water neglect 6 in soil along Hawes Rd.lpd
Name of output file: 8 in uniform Floodwall 6 inches water neglect 6 in soil along Hawes Rd.lpo
Name of plot output file: 8 in uniform Floodwall 6 inches water neglect 6 in soil along Hawes Rd.lpp
Name of runtime file: 8 in uniform Floodwall 6 inches water neglect 6 in soil along Hawes Rd.lpr

Time and Date of Analysis

Date: December 15, 2010 Time: 14:13:18

Problem Title

8" uniform thickness special curb- Case IV: 6" water depth above canal bank

Program Options

Units Used in Computations - US Customary Units: Inches, Pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis uses p-y multipliers for group action
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 50
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 15.00 in

Depth of ground surface below top of pile = .00 in

Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	8.00000000	341.0000	64.0000	2850000.
2	15.0000	8.00000000	341.0000	64.0000	2850000.

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 24.000 in
p-y subgrade modulus k for top of soil layer = 75.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 75.000 lbs/in**3

(Depth of lowest layer extends 9.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Effective unit weight of soil with depth defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.06076
2	24.00	.06076

Shear Strength of Soils

Shear strength parameters with depth defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	23.00	------	------
2	24.000	.00000	23.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

p-y Modification Factors

Distribution of p-y multipliers with depth defined using 2 points

Point No.	Depth X in	p-mult	y-mult
1	.000	.5000	1.0000
2	15.000	.5000	1.0000

Loading Type

Cyclic loading criteria was used for computation of p-y curves.

Number of cycles of loading = 10.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 5.000 lbs
Bending moment at pile head = 45.000 in-lbs
Axial load at pile head = 67.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment) condition.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 5.000 lbs
Specified moment at pile head = 45.000 in-lbs
Specified axial load at pile head = 67.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res. p lbs/in	Es*h F/L lbs/in
0.000	.050616	44.9985	5.0000	-.0043821	1.5747	0.0000	0.0000
.300	.049301	46.5877	4.9937	-.0043820	1.5934	-.0542149	.3299011
.600	.047986	48.1709	4.9667	-.0043820	1.6119	-.1135996	.7101978
.900	.046672	49.7438	4.9229	-.0043820	1.6304	-.1783004	1.1461
1.200	.045357	51.3008	4.8589	-.0043820	1.6486	-.2484586	1.6433
1.500	.044043	52.8353	4.7730	-.0043820	1.6666	-.3242099	2.2084
1.800	.042728	54.3407	4.6635	-.0043820	1.6843	-.4056841	2.8484
2.100	.041414	55.8096	4.5287	-.0043819	1.7015	-.4930040	3.5713
2.400	.040099	57.2341	4.3668	-.0043819	1.7182	-.5862848	4.3863
2.700	.038784	58.6058	4.1760	-.0043819	1.7343	-.6856334	5.3034
3.000	.037470	59.9159	3.9545	-.0043819	1.7497	-.7911472	6.3343
3.300	.036155	61.1547	3.7004	-.0043819	1.7642	-.9029126	7.4920
3.600	.034841	62.3123	3.4118	-.0043819	1.7778	-1.0210	8.7915
3.900	.033526	63.3779	3.0868	-.0043818	1.7903	-1.1455	10.2501
4.200	.032212	64.3405	2.7255	-.0043818	1.8016	-1.2631	11.7643
4.500	.030897	65.1894	2.3295	-.0043818	1.8116	-1.3775	13.3751
4.800	.029582	65.9143	1.8987	-.0043818	1.8201	-1.4944	15.1545
5.100	.028268	66.5048	1.4325	-.0043818	1.8270	-1.6135	17.1234
5.400	.026953	66.9500	.9302861	-.0043817	1.8322	-1.7346	19.3066
5.700	.025639	67.2391	.3914869	-.0043817	1.8356	-1.8574	21.7334
6.000	.024324	67.3610	-.1843532	-.0043817	1.8370	-1.9815	24.4388
6.300	.023010	67.3046	-.7975713	-.0043817	1.8364	-2.1066	27.4654
6.600	.021695	67.0586	-1.4484	-.0043816	1.8335	-2.2321	30.8647
6.900	.020381	66.6117	-2.1368	-.0043816	1.8282	-2.3574	34.7007
7.200	.019066	65.9527	-2.8627	-.0043816	1.8205	-2.4820	39.0533
7.500	.017752	65.0702	-3.6258	-.0043816	1.8102	-2.6050	44.0241
7.800	.016437	63.9534	-4.4254	-.0043816	1.7971	-2.7256	49.7445
8.100	.015123	62.5912	-5.2595	-.0043815	1.7811	-2.8352	56.2429
8.400	.013809	60.9738	-6.1234	-.0043815	1.7621	-2.9243	63.5331
8.700	.012494	59.0932	-7.0129	-.0043815	1.7401	-3.0055	72.1660
9.000	.011180	56.9422	-7.9252	-.0043815	1.7148	-3.0767	82.5618
9.300	.009865	54.5142	-8.8570	-.0043815	1.6863	-3.1354	95.3467
9.600	.008551	51.8041	-9.7891	-.0043815	1.6545	-3.0783	108.0000

8 in uniform Floodwall 6 inches water neglect 6 in soil along Hawes Rd.lpo

9.900	.007236	48.8169	-10.6538	-.0043814	1.6195	-2.6865	111.3750
10.200	.005922	45.5880	-11.3965	-.0043814	1.5816	-2.2651	114.7500
10.500	.004607	42.1551	-12.0084	-.0043814	1.5414	-1.8142	118.1250
10.800	.003293	38.5590	-12.4806	-.0043814	1.4992	-1.3337	121.5000
11.100	.001979	34.8429	-12.8042	-.0043814	1.4556	-.8236028	124.8750
11.400	.000664	31.0526	-12.9703	-.0043814	1.4111	-.2839498	128.2500
11.700	-.000650	27.2368	-12.9701	-.0043814	1.3664	.2852764	131.6250
12.000	-.001965	23.4467	-12.7947	-.0043814	1.3219	.8840757	135.0000
12.300	-.003279	19.7361	-12.4353	-.0043814	1.2784	1.5124	138.3750
12.600	-.004593	16.1617	-11.8828	-.0043814	1.2365	2.1704	141.7500
12.900	-.005908	12.7825	-11.1286	-.0043813	1.1968	2.8579	145.1250
13.200	-.007222	9.6606	-10.1637	-.0043813	1.1602	3.5750	148.5000
13.500	-.008537	6.8605	-8.9792	-.0043813	1.1273	4.3217	151.8750
13.800	-.009851	4.4493	-7.5662	-.0043813	1.0991	5.0979	155.2500
14.100	-.011165	2.4969	-5.9160	-.0043813	1.0762	5.9037	158.6250
14.400	-.012480	1.0758	-4.0669	-.0043813	1.0595	6.4236	154.4161
14.700	-.013794	.2328833	-2.0866	-.0043813	1.0496	6.7783	147.4162
15.000	-.015109	0.0000	0.0000	-.0043813	1.0469	7.1322	70.8090

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.05061570 in
Computed slope at pile head	=	-.00438205
Maximum bending moment	=	67.36102096 lbs-in
Maximum shear force	=	-12.97034553 lbs
Depth of maximum bending moment	=	6.00000000 in
Depth of maximum shear force	=	11.40000000 in
Number of iterations	=	21
Number of zero deflection points	=	1

Summary of Pile Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment, y = pile-head displacment in
Type 2 = Shear and Slope, M = Pile-head Moment lbs-in
Type 3 = Shear and Rot. Stiffness, V = Pile-head Shear Force lbs
Type 4 = Deflection and Moment, S = Pile-head Slope, radians
Type 5 = Deflection and Slope, R = Rot. Stiffness of Pile-head in-lbs/rad

Load Type	Pile-Head Condition 1	Pile-Head Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 5.000	M= 45.000	67.0000	.0506157	67.3610	-12.9703

The analysis ended normally.

PREMIER
ENGINEERING CORPORATION

SHEET NO 19 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Special Curb wall

MADE BY RMD DATE 11/3/10 CHECKED BY SEO DATE 12 APR 11

Check Sliding

Angle of internal Suction $\Theta = 23°$

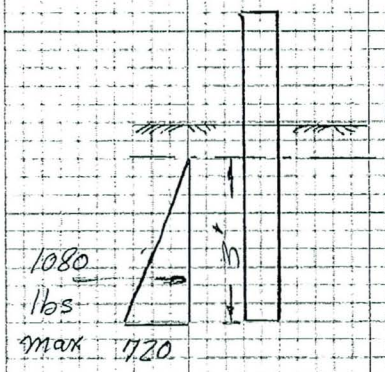
K_p Passive Resistance $= \frac{1+\sin 23}{1-\sin 23} = 2.28$

Passive Resistance $= K_p \gamma$

$K_p = 2.28 \times 105 = 240$ pcf

Case I

2' water above Bank.



Resistance (Max)

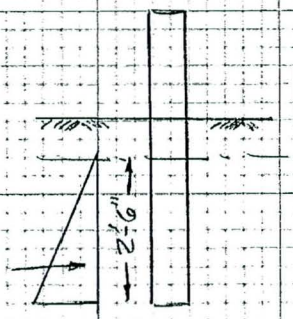
$\frac{1}{2}(240)(3)^2 = 1080$ lbs.

Resistance > Water force

No Sliding.

Case II

1' Water depth. 1' Wind



$\Sigma F = 30.8$ lbs

Resistance $= \frac{1}{2}(240)(2.5)^2 = 750$ lbs.

Resistance > ΣF

No Sliding

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

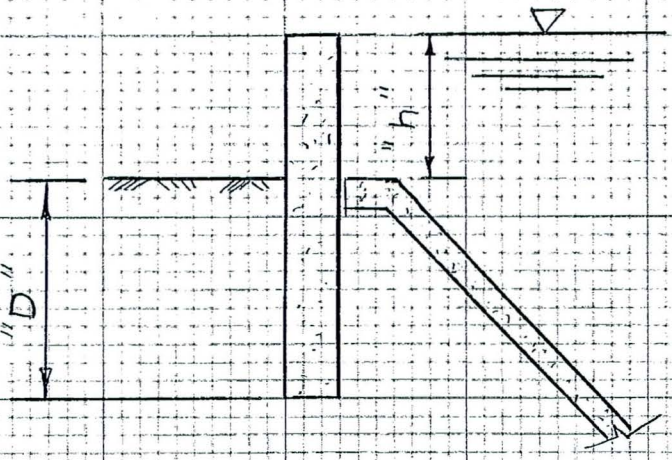
JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Special Curb Wall

MADE BY Raj D. DATE 12/2/10 CHECKED BY SEO DATE 12 APR 11

Results:



Water depth above bank "h" inches	Wall depth below ground "D" ft.
6"	1'-9"
12"	2'-3"
18"	3'-0"
24"	3'-6"

4.0

DROP STRUCTURE DESIGN

PREMIER
ENGINEERING CORPORATION

SHEET NO 1 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure.

MADE BY Raj D DATE 11/23/10 CHECKED BY SEO DATE 12 APR 11

Precast Drop Structure.

Pipe diameter = 84" I.D. 7" walls

Concrete unit wt. = 150 lbs/cft.

Backfill:

See Geotechnical Recommendations.

Unit wt. = 125 lbs/cft.

Use restrained unbalanced earth pressure = 55 lbs/ft²

Soil:

Allowable bearing pressure = 2000 psf. (2' below grade).

Fill on top of structure:

Unit wt 125 lbs/cft.

depth = 4' average.

Live load Surcharge: 2' of fill height.

Live loading: HS20+44 Neglect impact.

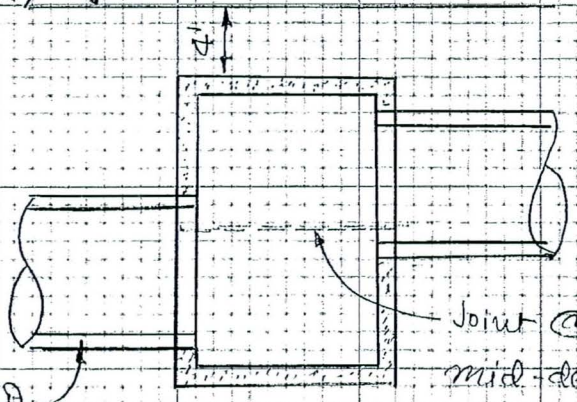
Two lane traffic.

Structure:

Inside 10'x10' in plan + 14' inside height.

Walls: 12" Top slab and Bottom slab = 12"

Reduce ↓



PREMIER
ENGINEERING CORPORATION

SHEET NO 2 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure -

MADE BY Raj D. DATE 4/23/10 CHECKED BY SEO DATE 12 APR 11

Bearing Pressure:

Total loads: DL + LL.

Live load: 2 Wheels HS20-44 no impact.

Dead loads: Drop Structure & Fill on top. 4'-0

Live loads	2 X 16	= 32 K
Fill on top	12X12X4X0.125	= 72 K
Top slab	12X12X1X0.150	= 21.6 K
Bottom Slab		= 21.6 K
Walls	4X14X10X1X0.150	= 84 K
Water	0.0624X11X10X10	= 69 K
Concrete	10X10X1.25X0.150	= 19 K
		319.2 K

Average Bearing Pressure: $\frac{319.2}{12X12} = 2.22$ KSF max. when flowing full + Live load.

Note: Blockout for pipes are not subtracted because pipes will rest on walls adding load.

Allowable bearing pressure = 3 KSF @ depth of 15' (Geotech Recommendation)

PREMIER
ENGINEERING CORPORATION

SHEET NO 3 OF _____
JOB NUMBER 2009034

TITLE Oak Street Basin
SUBJECT Drop Structure - Top Slab
MADE BY Raj D. DATE 11/24/10 CHECKED BY SED DATE 12 APR 11

Design load on top slab (Factored)

Dead loads:

Self wt. $12 \times 12 \times 1 \times 0.150 = 21.6^{k}$

Fill on top $12 \times 12 \times 4 \times 0.125 = 72^{k}$

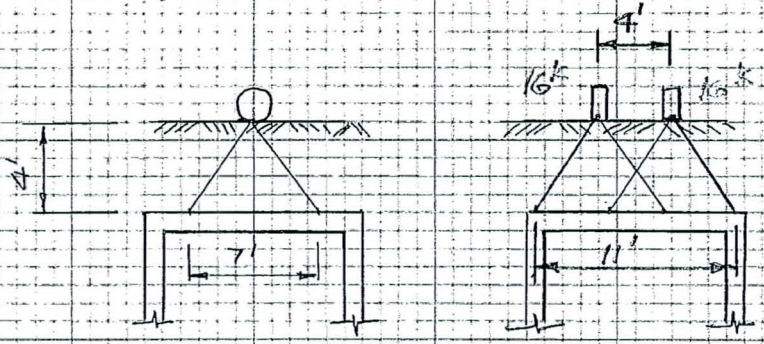
93.6^{k}

Uniform $W_{DL} = \frac{93.6}{12 \times 12} = 0.65\ k/SF$

Live loads:

Fill depth $\simeq 4'$ average. = No impact.

Wheel load will be distributed over top slab



$W_{LL} = \frac{2 \times 16}{7 \times 11} = 0.42\ k/SF$

Assume uniform LL on entire top slab = $W_{LL} = 0.42\ k/ft^2$ (Conservative)

Factored uniform load: q

$q = 1.3(W_{DL} + 1.67 W_{LL})$

$q = 1.3(0.65 + 1.67 \times 0.42) = 1.76\ k/SF$

Since Span lengths equal in both direction, uniform load in each direction will be.

$q_x = q_y = \frac{q}{2} = \frac{1.76}{2} = 0.88\ k/SF$

PREMIER
ENGINEERING CORPORATION

SHEET NO 4 OF

JOB NUMBER 2009034

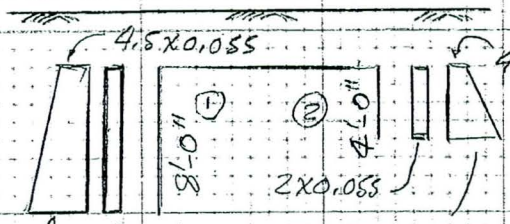
TITLE Oak Street Basin

SUBJECT Drop Structure — Top half

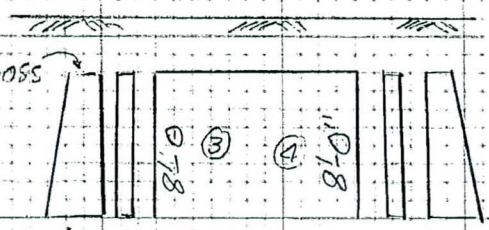
MADE BY Raj D. DATE 11/24/10 CHECKED BY SEO DATE 12 APR 11

Top half Structure (Walls)

Longitudinal Section:



Transverse Section



Equivalent uniform pressure (Factored)

Wall #1:

LL Surcharge $1.3(1.67 \times 2 \times 0.055) = 0.239$ k/SF

Earth Pressure $1.3(1.15 \times 4.5 \times 0.055) = 0.370$

$1.3(1.15 \times \frac{1}{2}(8.00)(0.055)) = 0.329$

Uniform P = 0.938 k/SF

Use 1 k/SF

Wall #2:

LL Surcharge = 0.239

Earth Pressure = 0.370

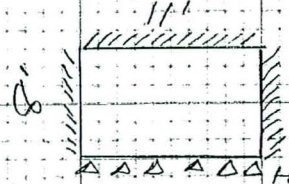
$1.3(1.15 \times \frac{1}{2}(4.00)(0.055)) = 0.164$

0.774 Use 0.8 k/SF

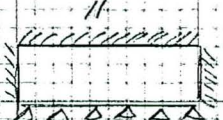
Walls #3 and #4:

Use 1 k/SF

Walls #1, #3 & #4



Wall #2



Friction @ St. creates fixed condition.

PREMIER
ENGINEERING CORPORATION

SHEET NO 5 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure - Top half

MADE BY Raj D DATE 11/29/10 CHECKED BY SED DATE 12 APR 11

Walls #1, #3, and #4 load distribution

Uniform load = $1\ k/ft^2$

Vertical: $l_x = 8'$

Horizontal: $l_y = 11'$

$$w_x = w \times \frac{l_y^4}{l_y^4 + l_x^4}$$

$$w_x = 1.0 \times \frac{(11)^4}{(11)^4 + (8)^4} = 1.0 \times \frac{14641}{14641 + 4096} = 0.78\ k/ft$$

Use vertically $w_x = 0.78\ k/ft$

$$w_y = w \times \frac{l_x^4}{l_x^4 + l_y^4} = 1 \times \frac{4096}{4096 + 14641} = 0.22\ k/ft$$

Use horizontally $w_y = 0.22\ k/ft$

Wall #2:

Uniform load = $w = 0.8\ k/sf$

Vertical $l_x = 4'$

Horizontal $l_y = 11'$

$$w_x = 0.8 \times \frac{(11)^4}{(11)^4 + (4)^4} = \frac{14641}{14641 + 256} = 0.98\ k/ft$$

$w_y = 0.02\ k/ft$

Use vertically $w_x = 0.98\ k/ft$

Use horizontally $w_y = 0.02\ k/ft$

PREMIER
ENGINEERING CORPORATION

SHEET NO 6 OF

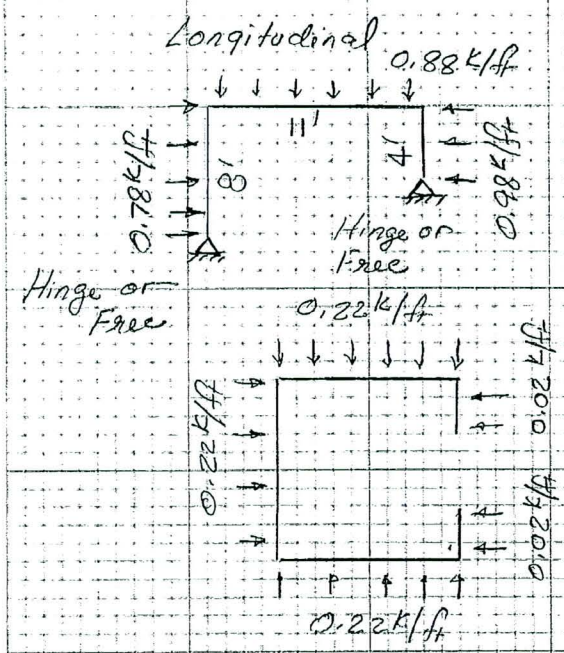
JOB NUMBER 2009034

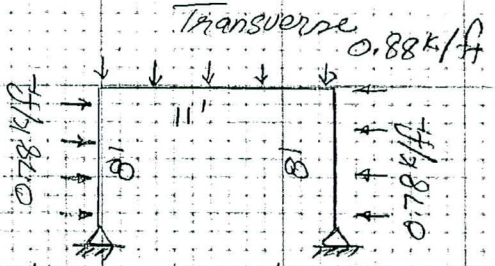
TITLE Oak Street Basin

SUBJECT Drop Structure - Top half

MADE BY Raj D. DATE 11/27/10 CHECKED BY SEO DATE 12APR11

loads for middle 1' Strip (Vertically)





Horizontal Direction

Use WINBDS for analysis.

Analyse using hinge condition for +Mmax in top slab

Analyse using cantilever walls for -Mmax @ joints

```
**************************************************************************
*                                WinBDS                                  *
*                                                                        *
*                          Bridge Design System                          *
*                             Version 5.0.3                              *
*                                                                        *
*                          Imbsen Software Systems                       *
*                              www.imbsen.com                            *
*                                                                        *
*                 Windows (GUI) By: CV-McBridge Software                 *
*                          Web Site: www.CV-McBridge.com                 *
*                                                                        *
*  Licensed  To: Premier Engineering Corporation                         *
*  License Date: Jun 05, 2008                                            *
**************************************************************************
```

Top Half
Longitudinal Section
Middle Strip

LISTING OF THE SORTED INPUT FILE

```
CARD          1         2         3         4         5         6         7         8
NUMBER 12345678901234567890123456789012345678901234567890123456789012345678901234567890
------ --------------------------------------------------------------------------------
     1 BdsInPutAssume hinge condition at joint                                      000
     2 BdsInPutDrop Structure                                                       000
     3 BdsInPutLongitudinal Section                                                 000
     4 BdsInPutOAK STREET BASIN PROJECT                                             000
     5 BdsInPutTop half structure                                                   000
     6 BdsInPutUS                                                                   001
     7 BdsInPut01 1 2P    80   008     4070                                         100
     8 BdsInPut02 2 3  H 110   008     4070                                         100
     9 BdsInPut03 4 3P    40   008     4070                                         100
    10 BdsInPut0001   0780U      80                                                 300
    11 BdsInPut0002   0880U     110                                                 300
    12 BdsInPut0003  -0980U      40                                                 300

--------------------------------------------------------------------------------
          1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume hinge condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
1	1	2	P			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	4	3	P			4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume hinge condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	1	2	P			8.0	0.3256E+03	0.0	4070.	0.500	0.000	0.000	0.508
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.492	0.327
3	4	3	P			4.0	0.3256E+03	0.0	4070.	0.500	0.000	0.000	0.673

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume hinge condition at joint

OAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	0.780	U	0.0	8.0	0.	0.		
	2	0.880	U	0.0	11.0	0.	0.		
	3	-0.980	U	0.0	4.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	0.	-6.	2	-9.	-9.	3	0.	2.

Assume hinge condition at joint

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	-8.	-3.	0.	3.	5.	6.	5.	4.	1.	-3.	-7.

WARN1.. - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	0.	1.	2.	3.	3.	2.	1.	-1.	-3.	-5.	-8.
3	0.	0.	0.	1.	1.	2.	2.	3.	5.	6.	7.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	4.9	4.0	3.0	2.0	1.1	0.1	-0.9	-1.8	-2.8	-3.8	-4.8

VERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	2.1	1.5	0.8	0.2	-0.4	-1.0	-1.7	-2.3	-2.9	-3.5	-4.1
3	-0.1	0.2	0.6	1.0	1.4	1.8	2.2	2.6	3.0	3.4	3.8

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	4.9	4.9	0.0
3	4.8	4.8	0.0

Assume hinge condition at joint

RIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	0.000121	0.000113	2	0.000113	-0.000151	3	0.000048	-0.000151

ORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2	E= 4070. KSI	0.000	0.001	0.001	0.001	0.000

ERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 3	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000

```
*****************************************************************************
*                                WinBDS                                     *
*                                                                           *
*                          Bridge Design System                             *
*                             Version 5.0.3                                 *
*                                                                           *
*                          Imbsen Software Systems                          *
*                               www.imbsen.com                              *
*                                                                           *
*                  Windows (GUI) By: CV-McBridge Software                   *
*                          Web Site: www.CV-McBridge.com                    *
*                                                                           *
*  Licensed  To: Premier Engineering Corporation                            *
*  License Date: Jun 05, 2008                                               *
*****************************************************************************
```

Top half

Transverse Section

Middle Strip

LISTING OF THE SORTED INPUT FILE

```
CARD                1         2         3         4         5         6         7         8
UMBER    12345678901234567890123456789012345678901234567890123456789012345678901234567890
-------------------------------------------------------------------------------------------
    1    BdsInPutAssume hinge condition at joint                                        000
    2    BdsInPutDrop Structure                                                         000
    3    BdsInPutOAK STREET BASIN PROJECT                                               000
    4    BdsInPutTop half structure                                                     000
    5    BdsInPutTransverse section                                                     000
    6    BdsInPutUS                                                                     001
    7    BdsInPut01 1 2P     80    008      4070                                        100
    8    BdsInPut02 2 3   H 110    008      4070                                        100
    9    BdsInPut03 4 3P     80    008      4070                                        100
   10    BdsInPut0001    0780U       80                                               300
   11    BdsInPut0002    0880U      110                                               300
   12    BdsInPut0003   -0780U       80                                               300

-------------------------------------------------------------------------------------------
                1         2         3         4         5         6         7         8
       12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume hinge condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
1	1	2	P			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	4	3	P			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume hinge condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	1	2	P			8.0	0.3256E+03	0.0	4070.	0.500	0.000	0.000	0.508
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.492	0.492
3	4	3	P			8.0	0.3256E+03	0.0	4070.	0.500	0.000	0.000	0.508

**** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
**** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume hinge condition at joint

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	0.780	U	0.0	8.0	0.	0.		
	2	0.880	U	0.0	11.0	0.	0.		
	3	-0.780	U	0.0	8.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	0.	-6.	2	-9.	-9.	3	0.	6.

Assume hinge condition at joint

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	-8.	-3.	1.	3.	5.	5.	5.	3.	1.	-3.	-8.

WARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	0.	1.	2.	3.	3.	2.	1.	0.	-2.	-5.	-8.
3	0.	-1.	-2.	-3.	-3.	-2.	-1.	0.	2.	5.	8.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	4.8	3.9	2.9	1.9	1.0	0.0	-1.0	-1.9	-2.9	-3.9	-4.8

VERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	2.1	1.5	0.9	0.2	-0.4	-1.0	-1.6	-2.2	-2.9	-3.5	-4.1
3	-2.1	-1.5	-0.9	-0.2	0.4	1.0	1.6	2.2	2.9	3.5	4.1

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	4.8	4.8	0.0
3	4.8	4.8	0.0

Assume hinge condition at joint

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	0.000127	0.000101	2	0.000101	-0.000101	3	-0.000127	-0.000101

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2 E= 4070. KSI	0.000	0.001	0.001	0.001	0.000

VERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1 E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 3 E= 4070. KSI	0.000	0.000	0.000	0.000	0.000

```
*****************************************************************************
*                                WinBDS                                     *
*                                                                           *
*                          Bridge Design System                             *
*                             Version 5.0.3                                 *
*                                                                           *
*                          Imbsen Software Systems                          *
*                               www.imbsen.com                              *
*                                                                           *
*                   Windows (GUI) By: CV-McBridge Software                  *
*                            Web Site: www.CV-McBridge.com                  *
*                                                                           *
*  Licensed  To: Premier Engineering Corporation                            *
*  License Date: Jun 05, 2008                                               *
*****************************************************************************
```

Top half

Walls

Horizontal Direction

LISTING OF THE SORTED INPUT FILE

```
CARD          1         2         3         4         5         6         7         8
NUMBER 12345678901234567890123456789012345678901234567890123456789012345678901234567890
------ --------------------------------------------------------------------------------
     1 BdsInPutDrop Structure                                                       000
     2 BdsInPutTop half structure                                                   000
     3 BdsInPutUS                                                                   001
     4 BdsInPutWalls - horizontal direction                                         000
     5 BdsInPut01 1 2C H  40   008     4070                                         100
     6 BdsInPut02 2 3  H 110   008     4070                                         100
     7 BdsInPut03 3 4  H 110   008     4070                                         100
     8 BdsInPut04 4 5  H 110   008     4070                                         100
     9 BdsInPut05 5 6 CH  40   008     4070                                         100
    10 BdsInPut0001  -0020U      40                                                300
    11 BdsInPut0002  -0220U     110                                                300
    12 BdsInPut0003  -0220U     110                                                300
    13 BdsInPut0004  -0220U     110                                                300
    14 BdsInPut0005  -0020U      40                                                300

          1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Drop Structure

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
	1	2	C		H	4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	3	4			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
4	4	5			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
5	5	6		C	H	4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Drop Structure

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	1	2	C		H	4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	1.000	0.500
3	3	4			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.500	0.500
4	4	5			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.500	1.000
5	5	6		C	H	4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Drop Structure

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	-0.020	U	0.0	4.0	0.	0.		
	2	-0.220	U	0.0	11.0	0.	0.		
	3	-0.220	U	0.0	11.0	0.	0.		
	4	-0.220	U	0.0	11.0	0.	0.		
	5	-0.020	U	0.0	4.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	0.	0.	2	2.	2.	3	2.	2.
4	2.	2.	5	0.	0.			

Drop Structure

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
2	0.	-1.	-1.	-2.	-2.	-2.	-2.	-1.	0.	1.	3.
3	3.	1.	1.	0.	-1.	-1.	-1.	0.	1.	1.	3.
4	3.	1.	0.	-1.	-2.	-2.	-2.	-2.	-1.	-1.	0.
5	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

WARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1
2	-1.0	-0.7	-0.5	-0.3	0.0	0.2	0.5	0.7	1.0	1.2	1.4
3	-1.2	-1.0	-0.7	-0.5	-0.2	0.0	0.2	0.5	0.7	1.0	1.2
4	-1.4	-1.2	-1.0	-0.7	-0.5	-0.2	0.0	0.3	0.5	0.7	1.0
5	-0.1	-0.1	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Drop Structure

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	-0.000140	-0.000145	2	-0.000145	0.000048	3	0.000048	-0.000048
*	-0.000048	0.000145	5	0.000145	0.000140			

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1	E= 4070. KSI	0.001	0.000	0.000	0.000	0.000
MEMBER 2	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 3	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 4	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 5	E= 4070. KSI	0.000	0.000	0.000	0.000	0.001

```
**************************************************************************
*                              WinBDS                                    *
*                                                                        *
*                         Bridge Design System                           *
*                            Version 5.0.3                               *
*                                                                        *
*                        Imbsen Software Systems                         *
*                             www.imbsen.com                             *
*                                                                        *
*                Windows (GUI) By: CV-McBridge Software                  *
*                        Web Site: www.CV-McBridge.com                   *
*                                                                        *
*  Licensed  To: Premier Engineering Corporation                         *
*  License Date: Jun 05, 2008                                            *
**************************************************************************
```

Top Half
Longitudinal Section
Free @ Ht.

LISTING OF THE SORTED INPUT FILE

```
CARD                1         2         3         4         5         6         7         8
NUMBER     12345678901234567890123456789012345678901234567890123456789012345678901234567890
-------    --------------------------------------------------------------------------------

    1      BdsInPutAssume free condition at joint                                        000
    2      BdsInPutDrop Structure                                                        000
    3      BdsInPutLongitudinal Section                                                  000
    4      BdsInPutOAK STREET BASIN PROJECT                                              000
    5      BdsInPutTop half structure                                                    000
    6      BdsInPutUS                                                                    001
    7      BdsInPut01 1 2C    80   008     4070                                          100
    8      BdsInPut02 2 3  H 110   008     4070                                          100
    9      BdsInPut03 4 3C    40   008     4070                                          100
   10      BdsInPut0001   0780U      80                                                 300
   11      BdsInPut0002   0880U     110                                                 300
   12      BdsInPut0003  -0980U      40                                                 300

                    1         2         3         4         5         6         7         8
           12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume free condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
1	1	2	C			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	4	3	C			4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume free condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	1	2	C		8.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000
2	2	3		H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	1.000	1.000
3	4	3	C		4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume free condition at joint

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	0.780	U	0.0	8.0	0.	0.		
	2	0.880	U	0.0	11.0	0.	0.		
	3	-0.980	U	0.0	4.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	0.	-25.	2	-9.	-9.	3	0.	8.

Assume free condition at joint

*** SIDESWAY NOT CONSIDERED. ***

ORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

EM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	-25.	-18.	-13.	-9.	-5.	-3.	-2.	-2.	-3.	-5.	-8.

ARN1. - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	0.	0.	-1.	-2.	-4.	-6.	-9.	-12.	-16.	-20.	-25.
3	0.	0.	0.	1.	1.	2.	3.	4.	5.	6.	8.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	6.4	5.4	4.5	3.5	2.5	1.6	0.6	-0.4	-1.3	-2.3	-3.3

ERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	0.0	-0.6	-1.2	-1.9	-2.5	-3.1	-3.7	-4.4	-5.0	-5.6	-6.2
3	0.0	0.4	0.8	1.2	1.6	2.0	2.4	2.7	3.1	3.5	3.9

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	6.4	6.4	0.0
3	3.3	3.3	0.0

Assume free condition at joint

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	-0.002637	-0.001218	2	-0.001218	0.000548	3	0.000771	0.000548

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2 E= 4070. KSI	0.000	-0.002	-0.002	-0.001	0.000

VERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1 E= 4070. KSI	0.018	0.013	0.008	0.003	0.000
MEMBER 3 E= 4070. KSI	-0.003	-0.002	-0.001	-0.001	0.000

```
**************************************************************************
*                                WinBDS                                  *
*                                                                        *
*                          Bridge Design System                          *
*                             Version 5.0.3                              *
*                                                                        *
*                          Imbsen Software Systems                       *
*                              www.imbsen.com                            *
*                                                                        *
*                  Windows (GUI) By: CV-McBridge Software                *
*                           Web Site: www.CV-McBridge.com                *
*                                                                        *
*  Licensed  To: Premier Engineering Corporation                         *
*  License Date: Jun 05, 2008                                            *
**************************************************************************
```

Top Half

Transverse Section

Free @ Joint

LISTING OF THE SORTED INPUT FILE

```
CARD          1         2         3         4         5         6         7         8
NUMBER 12345678901234567890123456789012345678901234567890123456789012345678901234567890
------ --------------------------------------------------------------------------------
     1 BdsInPutAssume free condition at joint                                       000
     2 BdsInPutDrop Structure                                                       000
     3 BdsInPutOAK STREET BASIN PROJECT                                             000
     4 BdsInPutTop half structure                                                   000
     5 BdsInPutTransverse section                                                   000
     6 BdsInPutUS                                                                   001
     7 BdsInPut01 1 2C    80   008      4070                                       100
     8 BdsInPut02 2 3  H 110   008      4070                                       100
     9 BdsInPut03 4 3C    80   008      4070                                       100
    10 BdsInPut0001   0780U      80                                                300
    11 BdsInPut0002   0880U     110                                                300
    12 BdsInPut0003  -0780U      80                                                300

--------------------------------------------------------------------------------
          1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume free condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
1	1	2	C			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	4	3	C			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume free condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	1	2	C			8.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	1.000	1.000
3	4	3	C			8.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume free condition at joint

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	0.780	U	0.0	8.0	0.	0.		
	2	0.880	U	0.0	11.0	0.	0.		
	3	-0.780	U	0.0	8.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	0.	-25.	2	-9.	-9.	3	0.	25.

Assume free condition at joint

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	-25.	-20.	-16.	-14.	-12.	-12.	-12.	-14.	-16.	-20.	-25.

WARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	0.	0.	-1.	-2.	-4.	-6.	-9.	-12.	-16.	-20.	-25.
3	0.	0.	1.	2.	4.	6.	9.	12.	16.	20.	25.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	4.8	3.9	2.9	1.9	1.0	0.0	-1.0	-1.9	-2.9	-3.9	-4.8

VERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	0.0	-0.6	-1.2	-1.9	-2.5	-3.1	-3.7	-4.4	-5.0	-5.6	-6.2
3	0.0	0.6	1.2	1.9	2.5	3.1	3.7	4.4	5.0	5.6	6.2

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	4.8	4.8	0.0
3	4.8	4.8	0.0

Assume free condition at joint

RIAL 0

ANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	-0.003307	-0.001887	2	-0.001887	0.001887	3	0.003307	0.001887

ORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2 E= 4070. KSI	0.000	-0.003	-0.004	-0.003	0.000

ERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1 E= 4070. KSI	0.024	0.017	0.011	0.005	0.000
MEMBER 3 E= 4070. KSI	-0.024	-0.017	-0.011	-0.005	0.000

TITLE Oak Street Basin

SUBJECT Drop Structure - Bottom half

MADE BY Raj D DATE 11/27/10 CHECKED BY SEO DATE 4/11/11

Design load on Bottom Slab (Factored)

Dead loads:

Fill on top	72 k/144	= 0.50 k/SF
Top slab	21.6/144	= 0.15
Walls	84/144	= 0.58
		1.23 k/SF

Live loads:

Use 0.42 k/SF

Factored uniform load

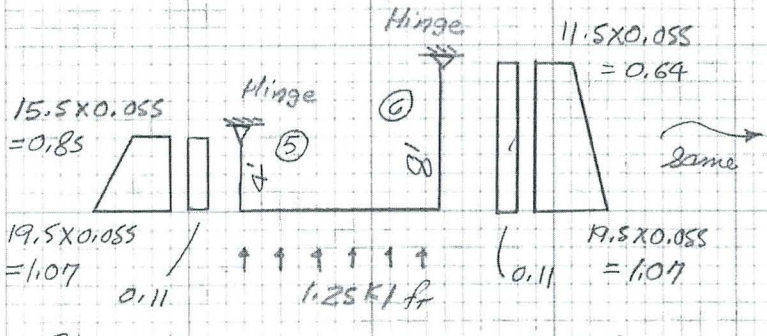
$q_f = 1.3(1.23 + 1.67 \times 0.42) = 2.5$ k/SF

Since floor is square,

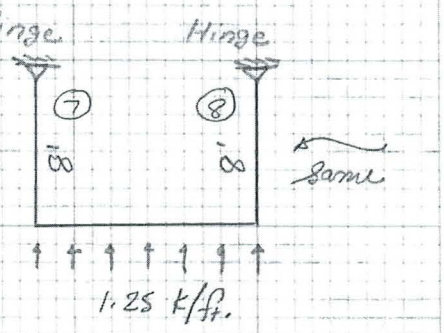
$w_x = w_y = 2.5/2 = 1.25$ k/Ft

Bottom Half Structure (Walls)

Longitudinal Section



Transverse Section



Equivalent uniform pressure (factored)

Wall #5:

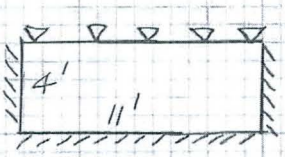
LL. Surcharge	1.3(1.67X0.11)	= 0.239 k/ft.
Earth Pressure	1.3(1.15 X 0.85)	= 1.271
	1.3(1.15 X ½(1.07-0.85)	= 0.165
		W = 1.675 k/ft.

PREMIER
ENGINEERING CORPORATION

SHEET NO 8 OF
JOB NUMBER 2009034

TITLE Oak Street Basin
SUBJECT Drop Structure – Bottom half
MADE BY Raj D DATE 11/27/10 CHECKED BY SEO DATE 4/11/11

$w = 1.675^{k}/_{SF}$



$$w_x = 1.675 \times \frac{(11)^4}{(11)^4+(4)^4} = 1.65\ ^{k}/_{ft}$$

$$w_y = 1.675 \times \frac{(4)^4}{(11)^4+(4)^4} = 0.03\ ^{k}/_{ft}$$

Walls #6, #7, #8

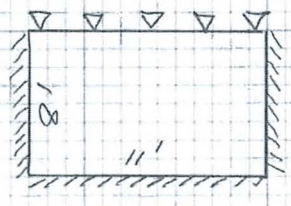
Equivalent uniform pressures (factored)

L.L. Surcharge $1.3(1.67 \times 0.11) = 0.239\ ^{k}/_{SF}$

Soil pressure $1.3(1.15)(0.64) = 0.957$

$1.3 \times 1.15 \times \frac{1}{2}(1.07-0.64) = 0.322$

$1.518\ ^{k}/_{SF}$

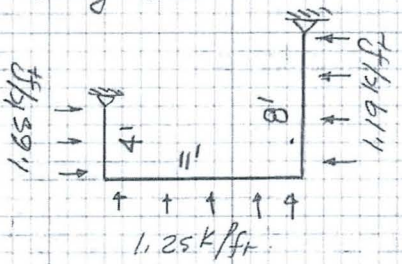


$$w_x = 1.518 \times \frac{(11)^4}{(11)^4+(8)^4} = 1.19\ ^{k}/_{ft}$$

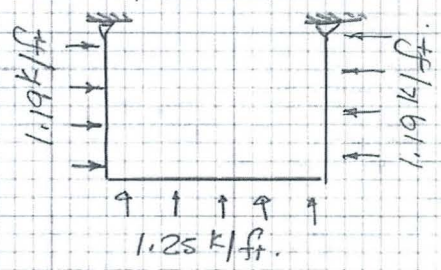
$$w_y = 1.518 \times \frac{(8)^4}{(11)^4+(8)^4} = 0.33\ ^{k}/_{ft}$$

Analysis of a middle 1' strip (Vertical)

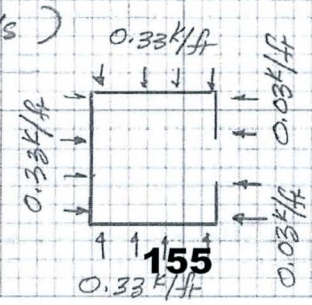
Longitudinal



Transverse



Horizontal (Walls)



```
*************************************************************************
*                              WinBDS                                   *
*                                                                       *
*                         Bridge Design System                          *
*                            Version 5.0.3                              *
*                                                                       *
*                         Imbsen Software Systems                       *
*                              www.imbsen.com                           *
*                                                                       *
*                  Windows (GUI) By: CV-McBridge Software               *
*                          Web Site: www.CV-McBridge.com                *
*                                                                       *
*  Licensed  To: Premier Engineering Corporation                        *
*  License Date: Jun 05, 2008                                           *
*************************************************************************
```

Bottom Half
Longitudinal Section
Hinge @ Jt.

LISTING OF THE SORTED INPUT FILE

```
CARD                1         2         3         4         5         6         7         8
NUMBER     12345678901234567890123456789012345678901234567890123456789012345678901234567890

    1      BdsInPutAssume hinge condition at joint                                        000
    2      BdsInPutBottom  half structure                                                 000
    3      BdsInPutDrop Structure                                                         000
    4      BdsInPutLongitudinal Section                                                   000
    5      BdsInPutOAK STREET BASIN PROJECT                                               000
    6      BdsInPutUS                                                                     001
    7      BdsInPut01 2 1 P   40   008      4070                                         100
    8      BdsInPut02 2 3  H 110   008      4070                                         100
    9      BdsInPut03 3 4 P   80   008      4070                                         100
   10      BdsInPut0001   1650U      40                                                  300
   11      BdsInPut0002  -1250U     110                                                  300
   12      BdsInPut0003  -1190U      80                                                  300

                    1         2         3         4         5         6         7         8
           12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume hinge condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
	2	1	P			4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	3	4	P			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume hinge condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	2	1		P	4.0	0.3256E+03	0.0	4070.	0.000	0.500	0.673	0.000
2	2	3		H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.327	0.492
3	3	4		P	8.0	0.3256E+03	0.0	4070.	0.000	0.500	0.508	0.000

**** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
**** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume hinge condition at joint

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	1.650	U	0.0	4.0	0.	0.		
	2	-1.250	U	0.0	11.0	0.	0.		
	3	-1.190	U	0.0	8.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	-3.	0.	2	13.	13.	3	10.	0.

Assume hinge condition at joint

*** SIDESWAY NOT CONSIDERED. ***

ORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	10.	4.	-1.	-5.	-7.	-8.	-7.	-4.	0.	5.	12.

ARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	-10.	-8.	-6.	-4.	-3.	-2.	-1.	0.	0.	0.	0.
3	12.	7.	3.	0.	-2.	-4.	-4.	-4.	-4.	-2.	0.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	-6.7	-5.4	-4.0	-2.6	-1.2	0.1	1.5	2.9	4.3	5.6	7.0

VERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	5.9	5.2	4.6	3.9	3.3	2.6	1.9	1.3	0.6	0.0	-0.7
3	-6.3	-5.3	-4.4	-3.4	-2.4	-1.5	-0.5	0.4	1.4	2.3	3.3

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	0.0	0.0	0.0
3	0.0	0.0	0.0

Assume hinge condition at joint

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	-0.000201	0.000053	2	-0.000201	0.000138	3	0.000138	0.000201

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2 E= 4070. KSI	0.000	-0.001	-0.001	-0.001	0.000

VERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1 E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 3 E= 4070. KSI	0.000	0.000	0.000	0.000	0.000

```
*****************************************************************************
*                                  WinBDS                                   *
*                                                                           *
*                            Bridge Design System                           *
*                               Version 5.0.3                               *
*                                                                           *
*                            Imbsen Software Systems                        *
*                                 www.imbsen.com                            *
*                                                                           *
*                     Windows (GUI) By: CV-McBridge Software                *
*                              Web Site: www.CV-McBridge.com                *
*                                                                           *
*  Licensed  To: Premier Engineering Corporation                            *
*  License Date: Jun 05, 2008                                               *
*****************************************************************************
```

Bottom Half

Transverse Section

Hinge @ Jt.

LISTING OF THE SORTED INPUT FILE

```
CARD          1         2         3         4         5         6         7         8
NUMBER 12345678901234567890123456789012345678901234567890123456789012345678901234567890
------ --------------------------------------------------------------------------------
     1 BdsInPutAssume hinge condition at joint                                      000
     2 BdsInPutBottom half structure                                                000
     3 BdsInPutDrop Structure                                                       000
     4 BdsInPutOAK STREET BASIN PROJECT                                             000
     5 BdsInPutTransverse section                                                   000
     6 BdsInPutUS                                                                   001
     7 BdsInPut01 2 1 P   80   008     4070                                         100
     8 BdsInPut02 2 3  H 110   008     4070                                         100
     9 BdsInPut03 3 4 P   80   008     4070                                         100
    10 BdsInPut0001   1190U      80                                                300
    11 BdsInPut0002  -1250U     110                                                300
    12 BdsInPut0003  -1190U      80                                                300

          1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume hinge condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
	2	1	P			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	3	4	P			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume hinge condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	2	1	P			8.0	0.3256E+03	0.0	4070.	0.000	0.500	0.508	0.000
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.492	0.492
3	3	4	P			8.0	0.3256E+03	0.0	4070.	0.000	0.500	0.508	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume hinge condition at joint

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	1.190	U	0.0	8.0	0.	0.		
	2	-1.250	U	0.0	11.0	0.	0.		
	3	-1.190	U	0.0	8.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	-10.	0.	2	13.	13.	3	10.	0.

Assume hinge condition at joint

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	12.	5.	-1.	-4.	-7.	-7.	-7.	-4.	-1.	5.	12.

WARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	-12.	-7.	-3.	0.	2.	4.	5.	5.	4.	2.	0.
3	12.	7.	3.	0.	-2.	-4.	-5.	-5.	-4.	-2.	0.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	-6.9	-5.5	-4.1	-2.8	-1.4	0.0	1.4	2.7	4.1	5.5	6.9

VERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	6.2	5.3	4.3	3.4	2.4	1.4	0.5	-0.5	-1.4	-2.4	-3.3
3	-6.2	-5.3	-4.3	-3.4	-2.4	-1.4	-0.5	0.5	1.4	2.4	3.3

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	0.0	0.0	0.0
3	0.0	0.0	0.0

Assume hinge condition at joint

RIAL 0

ANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	-0.000118	-0.000212	2	-0.000118	0.000118	3	0.000118	0.000212

ORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2	E= 4070. KSI	0.000	-0.001	-0.001	-0.001	0.000

ERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 3	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000

```
*****************************************************************************
*                                 WinBDS                                    *
*                                                                           *
*                          Bridge Design System                             *
*                             Version 5.0.3                                 *
*                                                                           *
*                          Imbsen Software Systems                          *
*                              www.imbsen.com                               *
*                                                                           *
*                  Windows (GUI) By: CV-McBridge Software                   *
*                          Web Site: www.CV-McBridge.com                    *
*                                                                           *
*  Licensed  To: Premier Engineering Corporation                            *
*  License Date: Jun 05, 2008                                               *
*****************************************************************************
```

Bottom Half
Horizontal Section
hinge @ Jt

LISTING OF THE SORTED INPUT FILE

```
CARD          1         2         3         4         5         6         7         8
NUMBER 12345678901234567890123456789012345678901234567890123456789012345678901234567890
------ --------------------------------------------------------------------------------
     1 BdsInPutBottom half structure                                              000
     2 BdsInPutDrop Structure                                                     000
     3 BdsInPutUS                                                                 001
     4 BdsInPutWalls - horizontal direction                                       000
     5 BdsInPut01 1 2C H  40   008      4070                                      100
     6 BdsInPut02 2 3  H 110   008      4070                                      100
     7 BdsInPut03 3 4  H 110   008      4070                                      100
     8 BdsInPut04 4 5  H 110   008      4070                                      100
     9 BdsInPut05 5 6 CH  40   008      4070                                      100
    10 BdsInPut0001  -0030U      40                                               300
    11 BdsInPut0002  -0330U     110                                               300
    12 BdsInPut0003  -0330U     110                                               300
    13 BdsInPut0004  -0330U     110                                               300
    14 BdsInPut0005  -0030U      40                                               300

               1         2         3         4         5         6         7         8
      12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Bottom half structure

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
1	1	2	C		H	4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	3	4			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
4	4	5			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
5	5	6		C	H	4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Bottom half structure

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	1	2	C		H	4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	1.000	0.500
3	3	4			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.500	0.500
4	4	5			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.500	1.000
5	5	6		C	H	4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Bottom half structure

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	-0.030	U	0.0	4.0	0.	0.		
	2	-0.330	U	0.0	11.0	0.	0.		
	3	-0.330	U	0.0	11.0	0.	0.		
	4	-0.330	U	0.0	11.0	0.	0.		
	5	-0.030	U	0.0	4.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	0.	0.	2	3.	3.	3	3.	3.
4	3.	3.	5	0.	0.			

Bottom half structure

*** SIDESWAY NOT CONSIDERED. ***

ORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

EM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
2	0.	-1.	-2.	-3.	-3.	-3.	-2.	-1.	0.	2.	4.
3	4.	2.	1.	0.	-1.	-1.	-1.	0.	1.	2.	4.
4	4.	2.	0.	-1.	-2.	-3.	-3.	-3.	-2.	-1.	0.
5	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

WARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

ORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

1	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.1
2	-1.5	-1.1	-0.8	-0.4	0.0	0.3	0.7	1.1	1.4	1.8	2.2
3	-1.8	-1.5	-1.1	-0.7	-0.4	0.0	0.4	0.7	1.1	1.5	1.8
4	-2.2	-1.8	-1.4	-1.1	-0.7	-0.3	0.0	0.4	0.8	1.1	1.5
5	-0.1	-0.1	-0.1	-0.1	-0.1	-0.1	0.0	0.0	0.0	0.0	0.0

Bottom half structure

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	-0.000210	-0.000217	2	-0.000217	0.000072	3	0.000072	-0.000072
4	-0.000072	0.000217	5	0.000217	0.000210			

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1	E= 4070. KSI	0.001	0.001	0.000	0.000	0.000
MEMBER 2	E= 4070. KSI	0.000	-0.001	-0.001	0.000	0.000
MEMBER 3	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 4	E= 4070. KSI	0.000	0.000	-0.001	-0.001	0.000
MEMBER 5	E= 4070. KSI	0.000	0.000	0.000	0.001	0.001

```
**************************************************************************
*                                WinBDS                                  *
*                                                                        *
*                          Bridge Design System                          *
*                             Version 5.0.3                              *
*                                                                        *
*                          Imbsen Software Systems                       *
*                              www.imbsen.com                            *
*                                                                        *
*                    Windows (GUI) By: CV-McBridge Software              *
*                             Web Site: www.CV-McBridge.com              *
*                                                                        *
*  Licensed  To: Premier Engineering Corporation                         *
*  License Date: Jun 05, 2008                                            *
**************************************************************************
```

Bottom Half
Longitudinal Section
Free @ Joint

LISTING OF THE SORTED INPUT FILE

```
CARD          1         2         3         4         5         6         7         8
NUMBER 12345678901234567890123456789012345678901234567890123456789012345678901234567890
------ --------------------------------------------------------------------------------
     1 BdsInPutAssume Free condition at joint                                       000
     2 BdsInPutBottom  half structure                                               000
     3 BdsInPutDrop Structure                                                       000
     4 BdsInPutLongitudinal Section                                                 000
     5 BdsInPutOAK STREET BASIN PROJECT                                             000
     6 BdsInPutUS                                                                   001
     7 BdsInPut01 2 1 C   40   008     4070                                         100
     8 BdsInPut02 2 3  H 110   008     4070                                         100
     9 BdsInPut03 3 4 C   80   008     4070                                         100
    10 BdsInPut0001   1650U      40                                                300
    11 BdsInPut0001   2100P  40                                                    300
    12 BdsInPut0002  -1250U     110                                                300
    13 BdsInPut0003  -0100P  80                                                    300
    14 BdsInPut0003  -1190U      80                                                300

           1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume Free condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
	2	1	C			4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	3	4	C			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume Free condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	2	1		C		4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	1.000	1.000
3	3	4		C		8.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume Free condition at joint

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	1.650	U	0.0	4.0	0.	0.		
	1	2.100	P	4.0	0.0	0.	0.		
	2	-1.250	U	0.0	11.0	0.	0.		
	3	-0.100	P	8.0	0.0	0.	0.		
	3	-1.190	U	0.0	8.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	-22.	0.	2	13.	13.	3	39.	0.

Assume Free condition at joint

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	22.	17.	13.	11.	10.	11.	14.	18.	23.	30.	39.

WARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	-22.	-18.	-15.	-12.	-10.	-7.	-5.	-4.	-2.	-1.	0.
3	39.	32.	25.	19.	14.	10.	6.	4.	2.	0.	0.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	-5.3	-3.9	-2.6	-1.2	0.2	1.6	2.9	4.3	5.7	7.1	8.4

VERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	8.7	6.0	7.4	6.7	6.1	5.4	4.7	4.1	3.4	2.8	2.1
3	-9.6	-8.7	-7.7	-6.8	-5.8	-4.9	-3.9	-3.0	-2.0	-1.1	-0.1

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	0.0	0.0	0.0
3	0.0	0.0	0.0

Assume Free condition at joint

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	0.001731	0.002465	2	0.001731	-0.002407	3	-0.002407	-0.004641

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2	E= 4070. KSI	0.000	0.003	0.005	0.004	0.000

VERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1	E= 4070. KSI	0.000	0.002	0.004	0.007	0.009
MEMBER 3	E= 4070. KSI	0.000	-0.006	-0.014	-0.023	-0.033

```
**************************************************************************
*                              WinBDS                                    *
*                                                                        *
*                         Bridge Design System                           *
*                            Version 5.0.3                               *
*                                                                        *
*                        Imbsen Software Systems                         *
*                             www.imbsen.com                             *
*                                                                        *
*                 Windows (GUI) By: CV-McBridge Software                 *
*                         Web Site: www.CV-McBridge.com                  *
*                                                                        *
*  Licensed  To: Premier Engineering Corporation                         *
*  License Date: Jun 05, 2008                                            *
**************************************************************************
```

Bottom Half

Transverse

Free @ H.

LISTING OF THE SORTED INPUT FILE

```
CARD                1         2         3         4         5         6         7         8
NUMBER     12345678901234567890123456789012345678901234567890123456789012345678901234567890
-------    --------------------------------------------------------------------------------
    1      BdsInPutAssume free condition at joint                                        000
    2      BdsInPutBottom half structure                                                 000
    3      BdsInPutDrop Structure                                                        000
    4      BdsInPutInclude reaction from top half structure                              000
    5      BdsInPutOAK STREET BASIN PROJECT                                              000
    6      BdsInPutTransverse section                                                    000
    7      BdsInPutUS                                                                    001
    8      BdsInPut01 2 1 C   80   008      4070                                         100
    9      BdsInPut02 2 3  H 110   008      4070                                         100
   10      BdsInPut03 3 4 C   80   008      4070                                         100
   11      BdsInPut0001   1190U       80                                               300
   12      BdsInPut0001   2100P  80                                                     300
   13      BdsInPut0002  -1250U      110                                               300
   14      BdsInPut0003  -1190U       80                                               300
   15      BdsInPut0003  -2100P  80                                                     300

                    1         2         3         4         5         6         7         8
           12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Assume free condition at joint

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
1	2	1	C			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	3	4	C			8.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Assume free condition at joint

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	2	1		C		8.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	1.000	1.000
3	3	4		C		8.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Assume free condition at joint

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	1.190	U	0.0	8.0	0.	0.		
	1	2.100	P	8.0	0.0	0.	0.		
	2	-1.250	U	0.0	11.0	0.	0.		
	3	-1.190	U	0.0	8.0	0.	0.		
	3	-2.100	P	8.0	0.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	-55.	0.	2	13.	13.	3	55.	0.

Assume free condition at joint

*** SIDESWAY NOT CONSIDERED. ***

ORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
2	55.	48.	43.	39.	37.	36.	37.	39.	43.	48.	55.

ARNIN - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

VERTICAL MEMBER MOMENTS(KIP-FT) TRIAL 0

1	-55.	-46.	-38.	-30.	-24.	-18.	-13.	-8.	-5.	-2.	0.
3	55.	46.	38.	30.	24.	18.	13.	8.	5.	2.	0.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

2	-6.9	-5.5	-4.1	-2.7	-1.4	0.0	1.4	2.8	4.1	5.5	6.9

ERTICAL MEMBER SHEARS(KIPS) TRIAL 0

1	11.6	10.7	9.7	8.8	7.8	6.9	5.9	5.0	4.0	3.1	2.1
3	-11.6	-10.7	-9.7	-8.8	-7.8	-6.9	-5.9	-5.0	-4.0	-3.1	-2.1

VERTICAL MEMBER REACTIONS(KIPS) TRIAL 0

MEM NO	LT REACTION	RT REACTION	MEMBER WEIGHT
1	0.0	0.0	0.0
3	0.0	0.0	0.0

Assume free condition at joint

TRIAL 0

TANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
1	0.004959	0.008558	2	0.004959	-0.004959	3	-0.004959	-0.008558

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 2	E= 4070. KSI	0.000	0.010	0.013	0.010	0.000

VERTICAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END.

MEMBER 1	E= 4070. KSI	0.000	0.012	0.027	0.043	0.060
MEMBER 3	E= 4070. KSI	0.000	-0.012	-0.027	-0.043	-0.060

```
**************************************************************************
*                                WinBDS                                  *
*                                                                        *
*                          Bridge Design System                          *
*                             Version 5.0.3                              *
*                                                                        *
*                          Imbsen Software Systems                       *
*                              www.imbsen.com                            *
*                                                                        *
*                  Windows (GUI) By: CV-McBridge Software                *
*                           Web Site: www.CV-McBridge.com                *
*                                                                        *
*  Licensed  To: Premier Engineering Corporation                         *
*  License Date: Jun 05, 2008                                            *
**************************************************************************
```

Edge Beam in Wall
Considering Shear at Jt.
Bottom half.
Free @ Jts (Vertically)

LISTING OF THE SORTED INPUT FILE

```
CARD          1         2         3         4         5         6         7         8
NUMBER 12345678901234567890123456789012345678901234567890123456789012345678901234567890
------ --------------------------------------------------------------------------------
     1 BdsInPutBottom half structure                                              000
     2 BdsInPutDrop Structure                                                     000
     3 BdsInPutEdge beam in walls near joint                                      000
     4 BdsInPutUS                                                                 001
     5 BdsInPutWalls - horizontal direction                                       000
     6 BdsInPutinclude shear + horizontal force                                   000
     7 BdsInPut01 1 2C H  40   008     4070                                       100
     8 BdsInPut02 2 3  H 110   008     4070                                       100
     9 BdsInPut03 3 4  H 110   008     4070                                       100
    10 BdsInPut04 4 5  H 110   008     4070                                       100
    11 BdsInPut05 5 6 CH  40   008     4070                                       100
    12 BdsInPut0001  -1190U      40                                              300
    13 BdsInPut0002  -1190U     110                                              300
    14 BdsInPut0003  -1190U     110                                              300
    15 BdsInPut0004  -1190U     110                                              300
    16 BdsInPut0005  -1190U      40                                              300

          1         2         3         4         5         6         7         8
12345678901234567890123456789012345678901234567890123456789012345678901234567890
```

Bottom half structure

FRAME DESCRIPTION

MEM NO	JT. LT	JT. RT	END COND LT	END COND RT	DIR	SPAN (FT)	I (FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	DEAD LOAD UNI (KIP/FT)	DEAD LOAD SEC (KCF)	K LT	K RT	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	RECALL MEM
1	1	2	C		H	4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
2	2	3			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
3	3	4			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
4	4	5			H	11.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	
5	5	6		C	H	4.0	0.08	0.0	4070.	0.000	0.000	0.00	0.00	0.00	0.00	

Bottom half structure

FRAME PROPERTIES

MEM NO	JT LT	JT RT	END COND LT	END COND RT	DIR	SPAN (FT)	MIN E*I (KSI-FT**4)	SUPPORT OR HINGE (FT)	E (KSI)	CARRY OVER FACTORS LT	CARRY OVER FACTORS RT	DISTRIBUTION FACTORS LT	DISTRIBUTION FACTORS RT
1	1	2	C		H	4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000
2	2	3			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	1.000	0.500
3	3	4			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.500	0.500
4	4	5			H	11.0	0.3256E+03	0.0	4070.	0.500	0.500	0.500	1.000
5	5	6		C	H	4.0	0.3256E+03	0.0	4070.	0.000	0.000	0.000	0.000

***** IF MEMBER IS HORIZONTAL SUPPORT OR HINGE FIELD EQUALS LOCATION OF HINGE FROM LEFT END OF MEMBER *****
***** IF MEMBER IS VERTICAL SUPPORT OR HINGE FIELD EQUALS SUPPORT WIDTH USED FOR MOMENT REDUCTION *****

Bottom half structure

LOAD DATA TRIAL 0

LINE	MEM	W OR P (KIP/FT) (KIP)	LOAD CODE	A (FT)	B (FT)	FIXED END MOMENTS LEFT (KIP-FT)	RIGHT (KIP-FT)	DEFLT	COMMENTS
	1	-1.190	U	0.0	4.0	0.	0.		
	2	-1.190	U	0.0	11.0	0.	0.		
	3	-1.190	U	0.0	11.0	0.	0.		
	4	-1.190	U	0.0	11.0	0.	0.		
	5	-1.190	U	0.0	4.0	0.	0.		

FIXED END MOMENTS TRIAL 0

MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)	MEM NO	FIXED END MOMENTS LT (KIP-FT)	RT (KIP-FT)
1	0.	10.	2	12.	12.	3	12.	12.
4	12.	12.	5	10.	0.			

Bottom half structure

*** SIDESWAY NOT CONSIDERED. ***

HORIZONTAL MEMBER MOMENTS (KIP-FT) TRIAL 0

MEM NO	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
1	0.	0.	0.	1.	2.	2.	3.	5.	6.	8.	10.
2	10.	3.	-1.	-5.	-7.	-7.	-6.	-4.	0.	6.	12.
3	12.	6.	1.	-3.	-5.	-6.	-5.	-3.	1.	6.	12.
4	12.	6.	0.	-4.	-6.	-7.	-7.	-5.	-1.	3.	10.
5	10.	8.	6.	5.	3.	2.	2.	1.	0.	0.	0.

WARNING - MEMBER DEPTHS MISSING SO MOMENT REDUCTION AND STRESSES ARE NOT CALCULATED.

HORIZONTAL MEMBER SHEARS(KIPS) TRIAL 0

1	0.0	0.5	1.0	1.4	1.9	2.4	2.9	3.3	3.8	4.3	4.8
2	-6.3	-5.0	-3.7	-2.3	-1.0	0.3	1.6	2.9	4.2	5.5	6.8
3	-6.5	-5.2	-3.9	-2.6	-1.3	0.0	1.3	2.6	3.9	5.2	6.5
4	-6.8	-5.5	-4.2	-2.9	-1.6	-0.3	1.0	2.3	3.7	5.0	6.3
5	-4.8	-4.3	-3.8	-3.3	-2.9	-2.4	-1.9	-1.4	-1.0	-0.5	0.0

Bottom half structure

RIAL 0

ANGENTIAL ROTATIONS - RADIANS - CLOCKWISE POSITIVE

SPAN	LT. END	RT. END	SPAN	LT. END	RT. END	SPAN	LT. END	RT. END
	0.000096	-0.000174	2	-0.000174	0.000058	3	0.000058	-0.000058
4	-0.000058	0.000174	5	0.000174	-0.000096			

HORIZONTAL MEMBER DEFLECTIONS IN FEET AT 1/ 4 POINTS FROM LEFT END - DOWNWARD POSITIVE

MEMBER 1	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000
MEMBER 2	E= 4070. KSI	0.000	-0.001	-0.001	-0.001	0.000
MEMBER 3	E= 4070. KSI	0.000	0.000	-0.001	0.000	0.000
MEMBER 4	E= 4070. KSI	0.000	-0.001	-0.001	-0.001	0.000
MEMBER 5	E= 4070. KSI	0.000	0.000	0.000	0.000	0.000

PREMIER
ENGINEERING CORPORATION

SHEET NO 9 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure

MADE BY Raj D DATE 11/28/10 CHECKED BY SEO DATE 12 April 11

Minimum Reinf. for a flexural member.

AASHTO 8.17.1

$\phi M_n > 1.2 M_{cr}$ $t = 12"$ $b = 12"$

$I_g = \frac{1}{12}(12)(12)^3 = 1728 \text{ in}^4$

$S = \frac{I_g}{y} = \frac{1728}{6} = 288 \text{ in}^3$

$f_r = 7.5\sqrt{f_c'} = 7.5\sqrt{5000} = 530 \text{ psi}$

$M_{cr} = f_r \times S = (530 \times 288)\frac{1}{12000} = 13 \text{ k-ft}$

$1.2 M_{cr} = 1.2 \times 13 = 15 \text{ k-ft}$

If $\phi M_n < 15$ k-ft, increase reinf. by 1/3.

Minimum Reinforcement as per ACI 318-95 10.5.

$A_{s\,min} = \frac{3\sqrt{f_c'}}{f_y} \times bd$ OR $\frac{200\,bd}{f_y}$

$= \frac{3\sqrt{5000}}{60000} \times 12 \times 9.375$ $= \frac{200 \times 12 \times 9.375}{60000}$

$= 0.43$ OR $= 0.40$

Use min. #4 @ 4" $A_s = 0.60$ Sq. in.

ACI 318-95 10.5.3 allows to increase reinf. by 1/3 instead 100.

Crack Control (ACI 318-95 10.6)

Use WSD $f_s = 24$ ksi $Z = 115$ k/in

$d_c = 1\frac{1}{2} + \frac{3}{8} = 1.875"$

Max. Spacing $= \frac{Z^3}{2 \times d_c^2 \times f_s^3}$

$S_{max} = \frac{(115)^3}{2 \times (1.875)^2 (24)^3} = 15"$ max.

Try #4 @ 4" $S \leq S_{max}$ Crack control ok

$A_s = 0.60$ Sq. in $A_s \geq 0.43$ Sq. in Min. Reinf. ok.

PREMIER
ENGINEERING CORPORATION

SHEET NO 10 OF _____
JOB NUMBER 2009034

TITLE Oak Street Basin
SUBJECT Drop Structure
MADE BY Raj D. DATE 11/28/10 CHECKED BY SED DATE 12/8/11

Strength of #4 @ 4" (LFD)

$\phi = 0.9$ $f_y = 60$ ksi $f_c' = 5$ ksi $b = 12"$ $d = 9.375"$ $A_s = 0.60$ Sq.in

$$\rho = \frac{A_s}{bd} = \frac{0.60}{12 \times 9.375} = 0.0053$$

$$\beta = 0.85 - 0.05\left(\frac{f_c' - 4000}{1000}\right) \geq 0.65$$

$\beta = 0.80$ use

Maximum Steel:

$$\rho_b = \frac{0.85 \beta f_c'}{f_y}\left(\frac{87}{87 + f_y}\right) = 0.0335$$

$0.75 \rho_b = 0.75 \times 0.0335 = 0.025 > \rho = 0.0053$ ok.

Moment Strength

$$M_n = A_s f_y d\left(1 - \frac{0.6 \rho f_y}{f_c}\right)$$

$$M_n = 0.60 \times 60 \times 9.375\left(1 - \frac{0.6 \times 0.0053 \times 60}{5}\right)\frac{1}{12}$$

$M_n = 27$ k-ft

$\phi M_n = 0.9 \times 27 = 24$ k-ft $> 1.2 M_{cr} = 15$ k-ft.

#4 @ 4" satisfies min. reinf. and crack control criteria.

If less than #4 @ 4" used, increase reinforcement by 1/3.

Temp. and Shrinkage Reinforcement

AASHTO 8.20

A_s req'd = 1/8 Sq.in/ft each direction

$Sp \leq 18"$ or $3x.t$

#4 @ 8" $A_s = 0.30$ Sq.in. ok

PREMIER
ENGINEERING CORPORATION

SHEET NO 11 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure - Top half

MADE BY Raj D DATE 11/27/10 CHECKED BY SEO DATE 12 [illegible]

Top half Structure: Moments

	Hinge	Free	Clse
Top Slab: Longitudinal Section			
Max Negative M @ Support	-8	-25	-25
Max Positive M mid-span	+6	-3	+6
Top Slab - Transverse direction			
Max -M @ Support	-8	-25	-25
Max +M @ mid span	+5	+0	+5
Walls: Longitudinal			
Max -M @ Support	-8	-25	-25
Max +M	+3	+0	3
Walls - Transverse			
Max -M @ Support	-8	-25	-25
Max +M	+3	+0	3
Walls - Horizontal			
Max -M	-3	—	-3
Max +M	+2	—	+2
Shear			
Vertical longitudinal			
Top Slab Vu	4.9	6.4	6.4
Walls Vu	4.1	6.2	6.2
Vertical Transverse			
Top Slab Vu	4.8	4.8	4.8
Walls Vu	4.1	6.2	6.2
Horizontal			
Walls	1.4	1.4	1.4

PREMIER
ENGINEERING CORPORATION

SHEET NO 12 OF _____

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure — Bottom half

MADE BY Raj D DATE 11/27/10 CHECKED BY S.E.O DATE 12 APR 11

Bottom Half Structure: Moments

	Hinge	Free	Clse
Bottom Slab - longitudinal			
Max -M @ Support	-12	-39	-39
Max +M mid span	+8	+0	+8
Bottom Slab - Transverse			
Max -M @ Support	-12	-55	-55
Max +M @ Support	+7	+0	+7
Walls - Longitudinal			
Max -M @ Support	-12	-39	-39
Max +M	+4	+0	+4
Walls - transverse			
Max -M @ Support	-12	-65	-55
Max +M	+5	+0	+5
Walls - Horizontal			
Max -M	-4	-12	-12
Max +M	+3	+7	+7

Shear:

Vertical longitudinal			
Bottom Slab Vu	7	8.4	8.4
Walls Vu	6.3	9.6	9.6
Vertical transverse			
Bottom Slab Vu	6.9	6.9	6.9
Walls Vu	6.2	11.6	11.6
Horizontal			
Walls	2.2	6.8	6.8

PREMIER
ENGINEERING CORPORATION

SHEET NO 13 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure - Top half

MADE BY Raj D DATE 11/30/10 CHECKED BY SEO DATE 12 APR 11

Flexural and Tension Reinforcement (Top Slab)

@ Support:

$-M_u$ max = −25 k-ft.

Tension (shear in wall) = 6.2 k

tension resisted by top & bottom reinforcement.

Steel reqd for direction (top) = $\frac{N_u}{2\phi f_y}$

$A_{s_{Nu}} = \frac{6.2}{2 \times 0.9 \times 60} = 0.06$ Sq. in.

Try #5 @ 4" in top of slab.

$A_s = 3 \times 0.31 = 0.93$ Sq. in

$A_{s_{Nu}} = -0.06$

for flexural 0.87 Sq. in.

$f_c' = 5$ ksi $f_y = 60$ ksi $b = 12"$ $d = 9.375"$

$\rho = \frac{A_s}{bd} = \frac{0.87}{12 \times 9.375} = 0.0077$

$\phi M_n = \phi A_s f_y d \left(1 - \frac{0.6 \rho f_y}{f_c'}\right)$

$\phi M_n = 0.9 \times 0.87 \times 60 \times 9.375 \left(1 - \frac{0.6 \times 0.0077 \times 60}{5}\right)\left(\frac{1}{12}\right)$

$\phi M_n = 35$ k-ft $> M_u$ 25 k-ft.

Use #5 @ 4" in top of top slab. (Middle ½ strip)

@ Mid-Span

$+M_u$ max = 6 k-ft.

Use minimum reinforcement #4 @ 4". (full width)

$\phi M_n = 25$ k-ft $> M_u$

Use #4 @ 4" in bottom of top slab (full width)

PREMIER
ENGINEERING CORPORATION

SHEET NO 14 OF _____
JOB NUMBER 2009034

TITLE Oak Street Basin
SUBJECT Drop Structure - top half
MADE BY Raj D. DATE 11/30/10 CHECKED BY SED DATE 12 APR 11

Walls

Max $-M_u$ @ Support = -25 k-ft.

Tension (Shear in Slab) = 6.4 k

Same as top slab

Use #5 @ 4" in outside face (middle 1/2).

Use #4 @ 4" in inside face. (Full width).

Shear

Max V_u = 6.4 k

Shear strength of concrete Section

$b = 12"$ $d = 9.375"$ $f'_c = 5$ KSI.

$\phi V_n = \phi 2 \sqrt{f'_c} b d$ Neglect tension

$\phi V_n = \frac{0.85 \times 2 \times \sqrt{5000} \times 12 \times 9.375}{1000}$

$\phi V_n = 13.5$ k $> V_u$ max Ok.

Alternate Reinforcement (Not used)

Top Slab:

In top: #5 @ 4" (middle half strip) $\phi M_n = 35$ k-ft

#4 @ 4" (outer 1/4 strips) min reinf

In bottom: #4 @ 4" (Full width) $\phi M_n = 25$ k-ft.

#4 @ 4" $A_s = 0.60$ sq in. minimum reinf

Walls:

Outside face

#5 @ 4" (middle half strip)

#4 @ 4" (outer quarter strip) min reinf

Inside face

#4 @ 4" min. reinf.

Wall horizontal

By Inspection #5 @ 4"

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SHEET NO 15 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure - Bottom half

MADE BY Raj D DATE 11/20/10 CHECKED BY SEO DATE 12 APR 11

Bottom Slab

Max $-M_u = -55$ k-ft

Tension (shown in wall) $= 11.6^k$

Steel req'd for tension (each face)

$$A_{snu} = \frac{1}{2} \times \frac{11.6}{0.9 \times 60} = 0.11 \text{ Sq. in.}$$

Try #7 @ 4"

$$A_s = \frac{0.60}{0.33} = 1.82 \text{ Sq in}$$

$$A_{snu} = -0.11 \text{ Sq in}$$

Available for flexural $= 1.71$ Sq in

$$\rho = \frac{1.71}{12 \times 9.375} = 0.0152 < 0.75 \rho_b = 0.025 \text{ ok.}$$

$$\phi M_n = 0.9 \times 1.71 \times 60 \times 9.375 \left(1 - \frac{0.6 \times 0.0152 \times 60}{5}\right)\left(\frac{1}{12}\right)$$

$\phi M_n = 64$ k-ft ≈ 55 k-ft M_u

Use #7 @ 4" because M_u max is @ ℄ Span & @ ℄ wall,

one single point @ face of wall $M_u \approx \phi M_n$.

Use #7 @ 4" in bottom of bottom slab (middle 1/2 strip)

Note:

Moments in bottom slab are large @ 1/4 length too.

Use #7 @ 4" in bottom of bottom slab (full width)

Max $+M_u = 8$ k-ft

Use minimum reinf #7 @ 4" $A_s = 0.60$ Sq. in.

Strength: $A_s = 0.60$ Sq. in

$- A_{snu} = 0.11$ Sq. in.

0.49 Sq. in.

Reduce 1/3 to satisfy min. reinf. criteria.

A_s available $= \frac{2}{3} \times 0.49 = 0.33$ Sq. in.

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SHEET NO 16 OF _____
JOB NUMBER 2009034

TITLE Oak Street Basin
SUBJECT Drop Structure – Bottom half
MADE BY Ray D DATE 11/30/10 CHECKED BY SED DATE 12 April 11

$$\phi M_n = 0.9 \times 0.33 \times 60 \times 9.375 \left(1 - \frac{0.6\left(\frac{0.33}{12 \times 9.375}\right) \times 60}{5}\right)\left(\frac{1}{12}\right)$$

$\phi M_n = 13.60$ k-ft $> M_u$ 8 k-ft ok.

Use #4 @ 4" in top of bottom slab (full width).

Walls

Outside face – M_u max = –5.5 k-ft.

Use #7 @ 4" (full width)

Inside face + M_u max = 5 k-ft.

Use #4 @ 4" (full width)

Shear

V_u max = 11.6k < ϕV_n of concrete 13.5 k

Shear ok

Design for wall horizontal bars

By inspection, use #5 @ 4" (both faces)

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SHEET NO 17 OF

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure - Shear Key

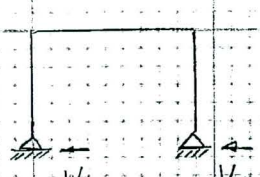
MADE BY Raj D DATE 11/30/10 CHECKED BY S.E.O DATE 12 APR 11

Shear key :

Maximum Reaction @ hinge from top structure .....

Longitudinal Section 3.8 k/ft. max.

Transverse Section 4.1 k/ft. max.



Design $V_u = 4.1^k/ft.$

Shear key width $5\frac{3}{4}"$ $d = 5.75" - 1\frac{1}{2}" - \frac{3}{16}" = 4"$

Strength $\phi V_c = 0.85 \times 2\sqrt{5000} \times 12 \times 4/1000$

$\phi V_c = 5.8^k > 4.1^k$ V_u OK.

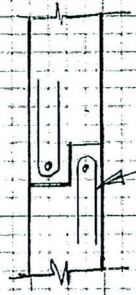
Shear friction

AASHTO 8.16.6.4.3

$\frac{V_u}{\phi} = \frac{4.1}{0.85} = 4.83^k$

Assume crack along Shear key.

Try #3 U Bars @ 12"



$A_{vf} = 2 \times 0.11 = 0.22$ Sq. in.

$V_n = A_{vf} f_y \mu$ $\mu = 1$ for crack

$= 0.22 \times 40 \times 1.0$

$V_n = 8.8^k > \frac{V_u}{\phi}$ OK.

Use #3 U Bars @ 12" (Grade 40)

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SHEET NO 18 OF

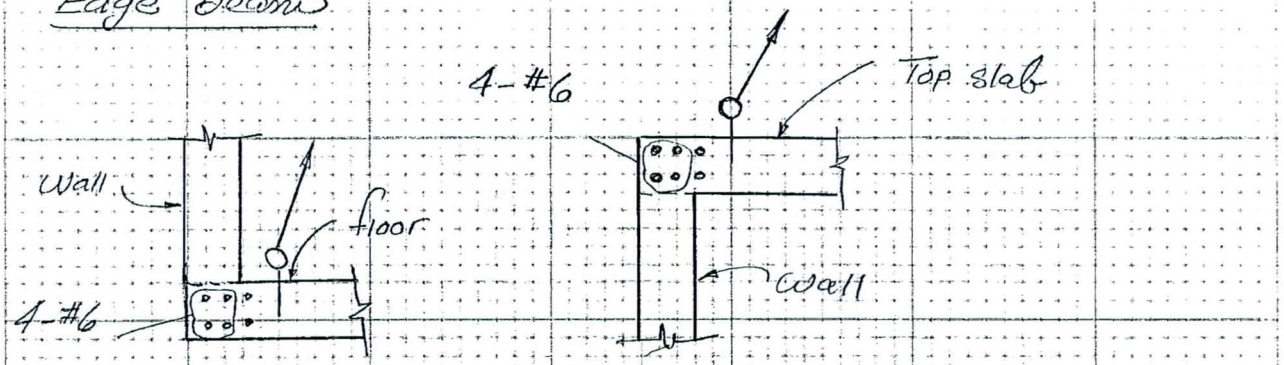
JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Drop Structure - Edge Beam

MADE BY Raj D. DATE 11/24/10 CHECKED BY SED DATE 12 APR 11

Edge Beam



DL of wall:

$7 \times 1 \times 0.150 \times 1.30 = 1.4^{k}/ft$ factored

$V_u = 1.4 \times 12/2 = 8.4^{k}$ $\quad V_u/\phi = \frac{8.4}{0.85} = 9.9^{k}$

$\pm M_u \approx \frac{wL^2}{8} = \frac{1.4(12)^2}{8} = 25.2\ k\text{-}ft.$

$\frac{M_u}{\phi} = \frac{25.2}{0.9} = 28\ k\text{-}ft.$

Shear:

Allowable Shear $= 2\sqrt{f_c'}\,bd$ $\quad b = 12''$ $d = 9.375''$

$V_n = 2\sqrt{5000} \times 12 \times 9.375$ $\quad f_c' = 5000\ psi.$

$V_n = 15.9^{k} > 9.9^{k}$ Ok. No Shear Reinf.

Moment:

Try 2-#6 $A_s = 0.88$ Sq. in.

$M_n = A_s f_y d \left(1 - \frac{0.6\rho f_y}{f_c}\right)$

$\rho = \frac{0.88}{12 \times 9.375} = 0.0078$

$M_n = 0.88 \times 60 \times 9.375 \left(1 - \frac{0.6 \times 0.0078 \times 60}{5}\right) \frac{1}{12}$

$M_n = 3.9\ k\text{-}ft \rightarrow M_u/\phi = 2.8^{k}$

Use 2-#6 $A_s = 0.88$ Sq. in Under the wall.
Top and Bottom.
Shear Reinforcement not Required.

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Analysis based on Rectangular Tank By Javeed A Munshi PCA Publication. (Finite element method).
- Use Chapter 2 for a square tank.

Upper Structure:

Top Slab:
- Uniform load $q = 1.76$ ksf.
- Analyse for $+M_{max}$ using simply Supported on edges.
- See Case 10 Page 2-59 $l/a = 1$

Shear

$V_{max} = C_s q a$ for l/a Page 2-59

$C_s = 0.34$

$V_{max} = 0.34 \times 1.76 \times 11 = 6.6^k$ (Factored)

Shear strength

$\phi V_n = (2\sqrt{f_c'}\, b d)(\phi)$ $\phi = 0.85$

$\phi V_n = (2\sqrt{5000})(12)(9.375)(0.85)$ $b = 12"$, $d = 9.375"$

$\phi V_n = 13.5^k > V_{max}$ Ok

Moments

Mid span $M_x = M_y = \text{Coeff} \times q (a)^2 / 1000$

$M_+ = 44 \times 1.76 (11)^2 / 1000$

$+M_{max} = 9.4$ k-ft.

Min Reinf. #4 @ 4" $A_s = 0.60$ Sq in. has moment Capacity $\phi M_n \geq 1.2 M_{cr}$

$\phi M_n = 24$ k-ft $> +M_{max}$ Ok.

Use #4 @ 4".

(Actual $+M$ will be less due to continuity @ edge).

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Walls

Transverse Section:

Wall length = 11'-0" b

Wall Height = 8'-0 a

$b/a = \frac{11}{8} \approx 1.5$

See page 2-47 Free @ one edge.
Fixed three sides.

$q = 1$ K/ft.

Shear: (Page 2-47) Case 8

$V_{max} = C_s q a$ $C_s = 0.95$ max.

$= 0.95 \times 1 \times 8$

$V_{ux} = 8$ K $< \phi V_n$ 13.5k Ok.

Moments

Vertical: (M_x) See Page 2-50

$b/a = 1.5$

$M_x = C q a^2/1000$

$-M_x = -124(1)(8)^2/1000$

Maximum Negative M @ Support (in wall & top Slab)

$-M_{x\,max} = 8$ k-ft.

Since Max $-M_x$ is less than 1.2 Mcr

Need min. Reinf. #4 @ 4"

Provided #5 @ 4" Ok.

Horizontal (M_y) $b/a = 1.5$

$-M_y = -18.1(1)(8)^2/1000$

Max $-M_y = 11.6$ k-ft.

Reinforcement provided #4 @ 4"

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SUBJECT Drop Structure
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Shear in top Slab = 6.6 k

Hence tension in wall = 6.6 k

$\frac{V_u}{\phi} = \frac{6.6}{0.85}$

$A_{s\,Nu\ req'd} = \frac{V_u}{\phi f_y} = \frac{6.6}{0.85 \times 60} = 0.13$ Sq.in

A_s used up for tension

$A_{sNu} = 0.13$ Sq.in

Net available for $= M_y$

$A_s' = A_s - A_{sNu} = 0.60 - 0.13 = 0.47$ Sq.in.

Strength:

$P = A_s/bd = \frac{0.47}{12 \times 9.375} = 0.0042$

$M_n = A_s f_y d \left(1 - \frac{0.6 f_y P}{f_c'}\right) \frac{1}{12}$

$M_n = 0.47 \times 60 \times 9.375 \left(1 - \frac{0.6 \times 60 \times 0.0042}{5}\right)\left(\frac{1}{12}\right)$

$M_n = 21.36$ k-ft

$\phi M_n = 0.9 \times 21.36 = 19.22$ k-ft

$\phi M_n > = M_y$ $\phi M_n > 1.2\ M_{cr}$ (15.0 k-ft)

Use #4 @ 4" Ok.

Longitudinal Section (Short wall)

$q = 0.8$ k/ft $\quad b = 11'$ $\quad a = 4'$ $\quad b/a \approx 3$

Shear

$C_s = 1.59$

$V_{max} = 1.59 \times 0.8 \times 4 = 5.1^k < \phi V_n\ 13.5^k$ Ok

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TITLE Oak Street Basin
SUBJECT Drop Structure
MADE BY Raj D DATE 12/27/10 CHECKED BY SED DATE 12 APR 11

Moments (Vertical) $\ell/a = 3$ Page 2-48

$C_s = -.351$

$-M_x$ Max. $= .351 \times 0.8 (4)^2 / 1000$

Max Negative moment in wall & Top Slab

$-M_x$ max $= 4.5$ k-ft Small.

#5 @ 4" provided Ok.

Horizontal (M_y)

$C_s = -.379$

Max $-M_y = .379 (0.8)(4)^2 / 1000$

$-M_y = 4.9$ k-ft Small

#4 @ 4" provided Strength $\phi M_n > M_y$ ok.

Else

Note: $+M_x$ and $+M_y$ in walls are small

#4 @ 4" ok.

Summary

Reinforcement based on Rectangular Tank.

Top Slab:

Top #5 @ 4"

Bottom #4 @ 4"

Walls:

Outside face Vertical #5 @ 4"

Inside face Vertical #4 @ 4"

Horizontal outside face #4 @ 4"

Horizontal inside face #4 @ 4"

Note: Hinge condition @ joint does not control.

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TITLE Oak Street Basin
SUBJECT Drop Structure
MADE BY Raj D DATE 12/27/10 CHECKED BY SED DATE 12 APR 11

Bottom half cont: (Walls free @ top ends)

Bottom Slab:
- Uniform load $q = 2.5$ k/ft.
- Analysis for + M Based on simply Supported.
- Case 10 Pg. 2-59 $l/a = 1$

Shear
$C_s = 0.34$ $a = 11'$
$V_{max} = V_u = 0.34 \times 2.5 \times 11 = 9.35$ k
$\phi V_n = 13.5$ k $> V_u$ Ok.

Moments
Mid-Span $M_x = M_y = 44(2.5)(11)^2/1000$
$= 13.4$ k-ft.
#4 @ 4" Okay
$\phi M_n = 24$ k-ft $> M_x$ or M_y Ok.
Use #4 @ 4" both ways in top of bottom slab.
(Actual + Mx and +My will be less due to continuity @ edge.)
Negative Moments same as walls.

Walls
Transverse Section (One free edge)
Wall $b = 11'$ $a = 8'$ $l/a = 1.5$ $q = 1.518$ k/ft
Shear
$V_{max} = 0.75 \times 1.518 \times 8 = 11.6$ k
$\phi V_n = 13.5$ k $> V_{max}$ Ok.
(Actual Shear can be 1% more due to hinge condition @ top and reaction from top slab)

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SHEET NO 6 OF _____
JOB NUMBER 2009039

TITLE Oak Street Basin
SUBJECT Drop Structure
MADE BY Raj D DATE 12/27/10 CHECKED BY SED DATE 12 APR 11

Moments:

Vertical (M_x) See page 2-50

$B = 11$ $a = 8$ $B/a = 1.5$

$q = 1.518$ k/sf

$-M_x = -124(1.518)(8)^2/1000 = -12$ kft

Reinforcement provided #7 @ 4" $\phi M_n = 64$ kft

$\phi M_n > -M_x$ ok.

Horizontal (M_y)

$-M_y = -181(1.518)(8)^2/1000 = -17.6$ kft

Reinf in horizontal direction #4 @ 4 may not be sufficient.

Horizontal Shear

$V_{max} = C_s q a$ $B/a = 1.5$ $q = 1.518$ k/sf

$= 1 \times 1.518 \times 8$ $C_s = 1$ $a = 8'$

$V_{max} = 12$ k

Hence tension in wall = 12 k

½ of tension taken by each side of wall reinf.

Hence $A_{s,req} = \frac{1}{2} \times 12 \times \frac{1}{60} = 0.10$ Sq in — Needed for tension

#4 @ 4" Provided

$A_s = 0.60$ Sq in

$A_{s,req} = 0.10$ Sq in

0.50 Available for bending

$\phi M_n = 0.9 \times 0.5 \times 60 \times 9.375 \left(1 - \frac{0.6 \times 60 \times \frac{0.5}{12}}{5}\right)\left(\frac{1}{12}\right)$

$\phi M_n = 22.68$ kft M_y ok.

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Longitudinal Section

Short wall:

$b/a = \frac{11}{4} = 3$ $q = 1.675$ k/ft

Vertical (M_x)

M_x Max $= -.351(1.675)(4)^2/1000$

$M_x = 9.4$ k-ft

#7@4" $\phi M_n = 69$ k-ft $> M_x$ Ok.

Horizontal (M_y)

Shear $= 1.59(1.675)(4) = 10.7^k < \phi V_n$

Tension in wall = Shear

$N_u = 10.7^k$

tension on each side, Reinf $\frac{N_u}{2} = \frac{10.7}{2} = 5.35^k$

$A_{s\,Nu} = \frac{5.35}{60} = 0.09$ Sq in.

$M_y = -.379(1.675)(4)^2/1000$

$M_y = -10.2$ k-ft

ϕM_n #4 @ 4" $= 22.68$ k-ft $> M_y$ Ok.

Use #4 @ 4"

Summary

Bottom Slab Top #4 @4" Both ways

Bottom #7@4" Both ways

Walls Outside face Vert. #7@4"

Horiz #4@4"

Inside face #4@4" Both ways

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SHEET NO 1 OF

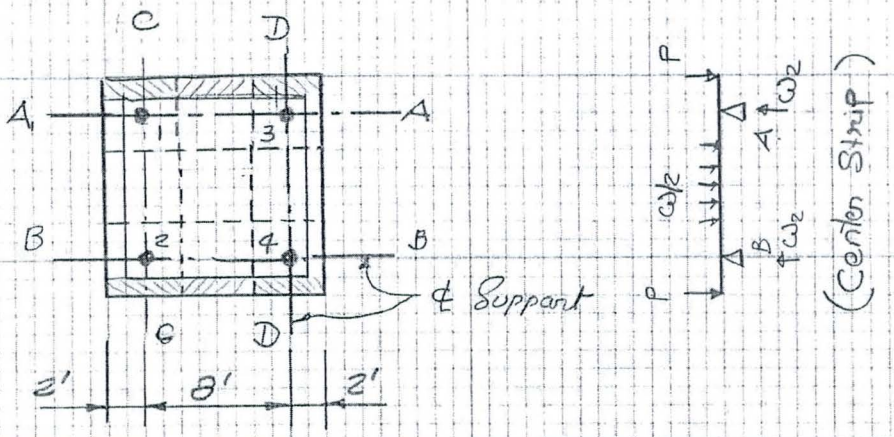
JOB NUMBER 2009034

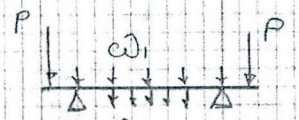
TITLE Oak Street Basin

SUBJECT Drop Structure

MADE BY Raj D. DATE 4/13/11 CHECKED BY SEO DATE 4/13/11

Lifting @ 4 Points





(Edge Strip)
A-A

ω_1 Self wt of edge strip

ω_2 Reaction @ Support of transverse middle strip

P Wall load.

Center Strip

Slab width/Length = 1

Distribute center strip slab load equal in both direction

Uniform load = $\omega/2$ = $0.150 \times 1.0 \times 1/2 = 0.075$ k/ft

Wall load:

height $\approx 7'$ (No deduction for Blockouts)

$P = 7 \times 1 \times 0.150 = 1.05^k \downarrow$

Therefore, reaction ω_2 @ A-A Support line

$\omega_2 = 0.075 \times 4 \times \frac{1}{2} + 1.05 = 1.2$ k/ft

Edge Strip (4' wide)

ω_2 reaction from center strip = 1.20 k/ft.

ω_1 Self wt $1.0 \times 4.0 \times 0.150 = 0.60$ k/ft.

P Wall wt $7 \times 4 \times 0.150 = 4.20^k$

TITLE Oak Street Basin

SUBJECT Drop Structure

MADE BY Raj D DATE 4/13/11 CHECKED BY SEO DATE 4/13/11

Reaction @ Cable lift pt. R

$R = P + \omega_1 L_1/2 + \omega_2 \times 4/2$

$R = 4.2 + 0.6 \times 12/2 + 1.20 \times 4/2$

$R_{AA} = 10.2^{k}$

Vertical load in lift point

Vertical force $= R_{AA} + R_{CC}$

$= 10.2 + 10.2$

$F = 20.4^{k}$

Moment (Edge Strip) (A-A) (4' width)

Moment in Cantilever:

$P \times 2 + \omega_1 (2)^2 (1/2)$

$M = 4.2 \times 2 + 0.60(4)(1/2) = 9.6^{k\text{-}ft}$

Moment/ft $= \frac{9.6}{4} = 2.4$ k-ft. (Small)

Slab reinf. is sufficient

Note:
Contractor shall design lifting mechanism and additional reinforcement required to resist punching shear at lift point

5.0

OUTLET STRUCTURE DESIGN

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SHEET NO 1 OF _____

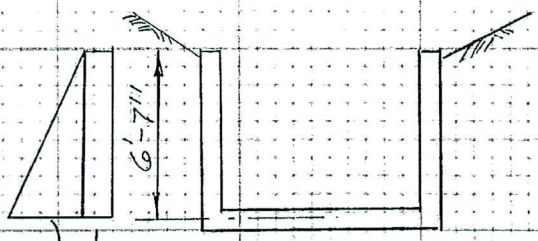
JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Outlet Structure

MADE BY Raj D. DATE 11/30/10 CHECKED BY SEO DATE 12 APR 11

Channel



Design height ≈ 6'-7"

Loads:

Earth Pressure = 55 #/SF

Surcharge load = 2' Fill

Construction Live load

Walls: 8"

Floor: 8"

Design fill slope: Level.

By Observation:

Bearing Pressure: Small.

Stability:

Overturning: Ok

Sliding: Ok

Walls:

Thickness: 8"

Width: 12"

Maximum factored loads @ base.

Design for h ≈ 6'-7"

Moment $M_u = 1.3\left(1.67 \times 2 \times 0.055 \times (6.58)^2 (1/2)\right) = 5.17$

$1.3\left(1.15 \times \frac{1}{6} \times (0.055)(6.58)^3\right) = 3.91$

$M_u = \overline{9.08}$ k-ft

Try #6 @ 9"

$A_s = 0.59$ Sq in. $b = 12"$ $d = 4.63"$ $f_c' = 3$ ksi. $f_y = 60$ ksi.

Moment Capacity M_n

$M_n = A_s f_y d \left(1 - \frac{0.6 p f_y}{f_c'}\right)$ $\quad p = \frac{0.41}{12 \times 4.63} = 0.0073$

$M_n = 0.59 \times 60 \times 4.63\left(1 - \frac{0.6 \times 0.0073 \times 60}{3}\right)\left(\frac{1}{12}\right)$

$M_n = 12.46$ k-ft

$\phi M_n = 0.9 \times 12.46 = 11.20$ k-ft. > 9.08 k-ft.

$\phi M_n > M_u$ Use #6 @ 9"

Shear

$V_u = 1.3(1.67 \times 2 \times 0.056 \times 6.58) = 1.58$ k

$1.3(1.15 \times \frac{1}{2}(0.056)(6.58)^2 = 1.78$ k

$V_u = 3.36$ k

$V_c = 2\sqrt{f'_c}\,bd$

$V_c = 2\sqrt{3000} \times 12 \times 4.63 / 1000$

$V_c = 6.10$ k

$\phi V_c = 0.85 \times 6.10 = 5.20$ k > 3.36 k ok.

$\phi V_c > V_u$ ok.

Use 8" wall.

Floor

Use #6 @ 9"

Distribution Reinforcement

Also Temp. & Shrinkage

Used #4 @ 9". $A_s = 0.27$ Sq. in.

Steel Rack

Eff. Span Length (between Supports) = 9'

loads:

Self weight 4 lbs/ft.

Debris &c. 50 lbs/SF

Uniform load $w = 4 + 50 \times \frac{4}{12} \approx 25$ lbs/ft.

Use WSD

Steel A36 $f_s = 19.8$ ksi. $f_v = 12$ ksi.

Try 3" x 3/8" Bars with 1" x 1/4" Cross Bars

$d_{eff} = 2$"

Moment = $0.025(9)^2(\frac{1}{8}) = 0.25$ k-ft. Small.

$f_s = \frac{M}{S}$ $S = \frac{1}{6}(\frac{3}{8})(2)^2 = 0.25$ in^3 $d_{eff} = 2''$

$f_s = \frac{0.25 \times 12}{0.25} \approx 12$ ksi < 19.8 ksi. Ok

Shear

$V = \frac{1}{2}(0.025)(9) = 0.12$ k Small

$v = \frac{0.12}{2 \times 3/8} = 0.16$ ksi < 12 ksi ok

Use $3'' \times 3/8''$ Bars @ 4" c/c

Small Channel

Try 6" walls and floor with $2'' \times 3/8''$ Rack bars.

Design $b = 2 \div 3''$

$f_c' = 3$ ksi $b = 12''$ $d = 2.69''$

Moment:

$M_u = 1.3(1.67 \times 2 \times 0.055(2.25)^2(1/2)) = 0.61$

$1.3(1.16 \times \frac{1}{6} \times 0.055(2.25)^3 = 0.16$

0.77 k-ft.

Small

Try #4 @ 9" $A_s = 0.27$ Sq in.

$P = \frac{A_s}{bd} = 0.0083$

$\phi M_n = 0.9 \times 0.27 \times 60 \times 2.69\left(1 - \frac{0.6 \times 0.0083 \times 60}{3}\right)\frac{1}{12}$

$\phi M_n = 3$ k-ft $> M_u$ ok.

Use #4 @ 9" with 6" wall.

Steel Rack

Use $2'' \times 3/8''$ Bars @ 5" c/c.

PREMIER
ENGINEERING CORPORATION

SHEET NO 4 OF _____

JOB NUMBER 2009034

TITLE Oak Street Basin

SUBJECT Outlet Structure

MADE BY Raj D DATE 12/1/10 CHECKED BY SEO DATE 12 APR 11

Cut-off wall

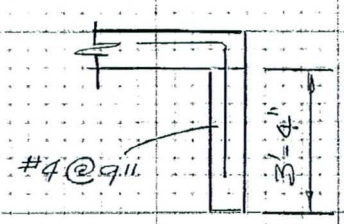
Assume 4' Scour in front of wall

Try 8" Wall.

Use Saturated soil condition

Earth pressure ≈ 90 lbs/SF

Moment @ base of floor



$$M_u = 1.3\left(1.15 \times \frac{1}{2}(0.090)(3.33)^3\left(\frac{2}{3}\right)\right) = 1.67 \text{ k-ft}$$

$\phi = 12"$ $d = 8 - 3 - 5/16 = 4.69"$ $f'_c = 3$ ksi $f_y = 60$ ksi

Use #4 @ 9"

Moment Capacity

$$\phi M_n = 0.27 \times 60 \times 4.69\left(1 - \frac{0.6 \times 60 \times \frac{0.27}{12 \times 4.69}}{3}\right)\left(\frac{1}{12}\right)$$

$\phi M_n = 6$ k-ft

$\phi M_n > M_u$ ok

Use #4 @ 9"

Shear ok.

6.0
CONTAINMENT WALL DESIGN

PREMIER
ENGINEERING CORPORATION

SHEET NO _____ OF _____

JOB NUMBER 2009034

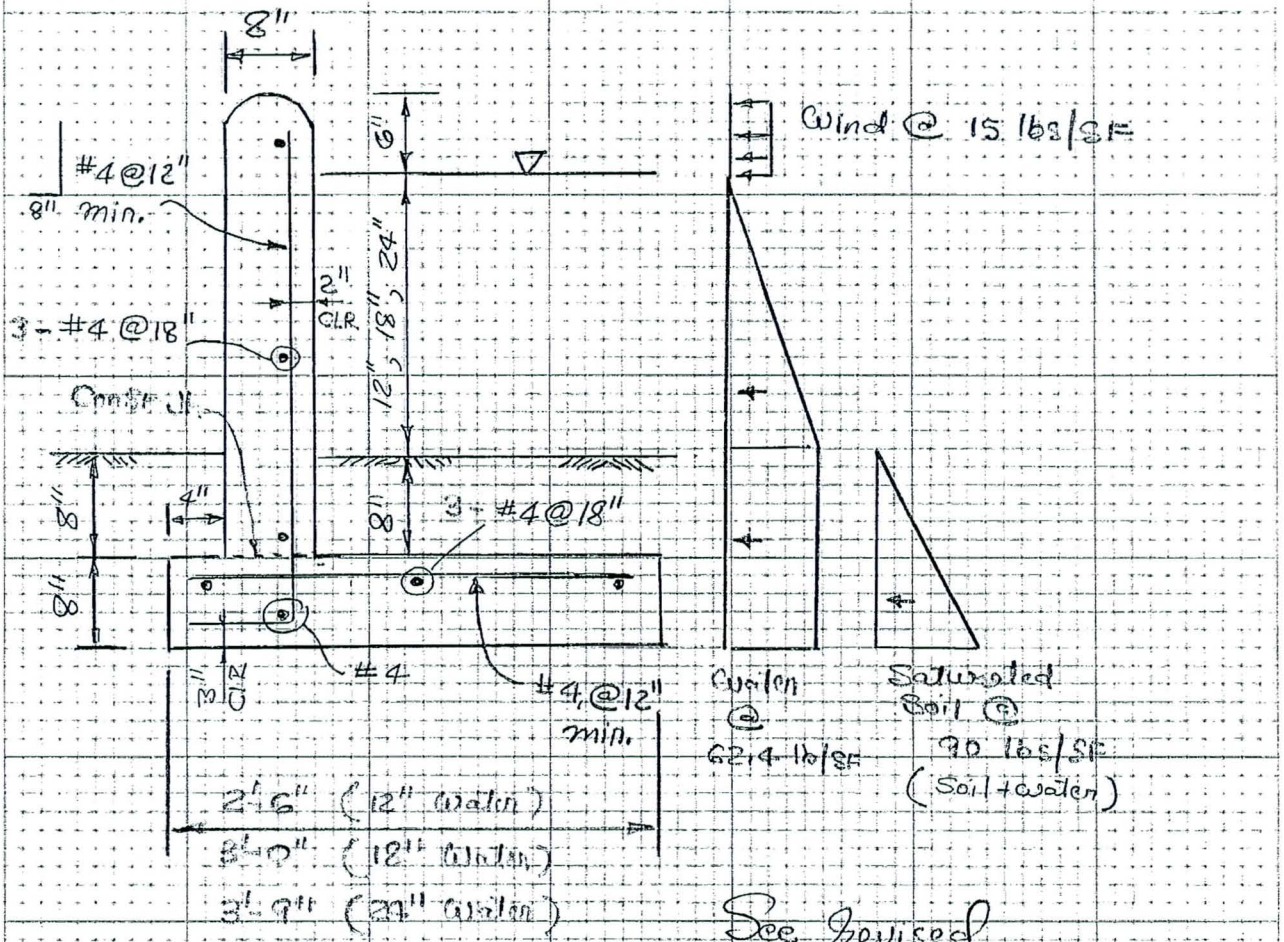
TITLE Oak Street Basin

SUBJECT Flood Containment Walls

MADE BY Raj D DATE 12/4/10 CHECKED BY SED DATE 12 APR 11

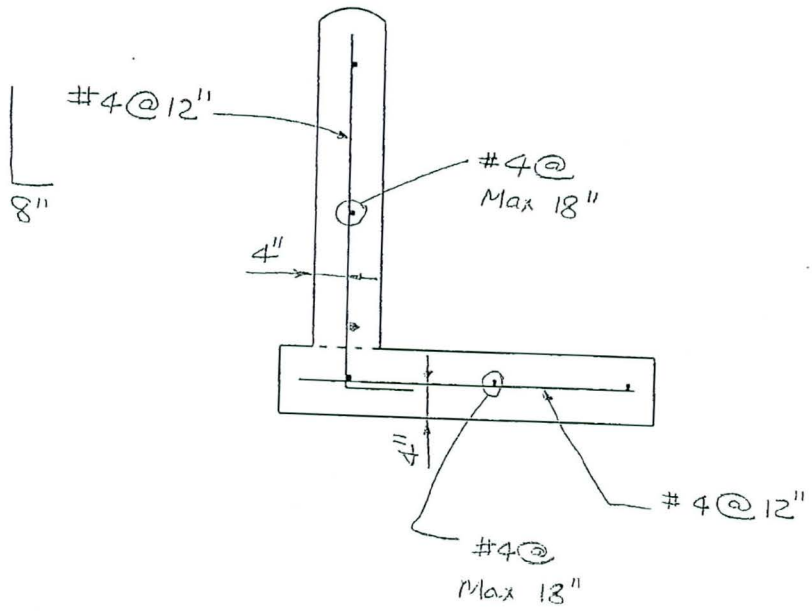
Flood Containment Wall

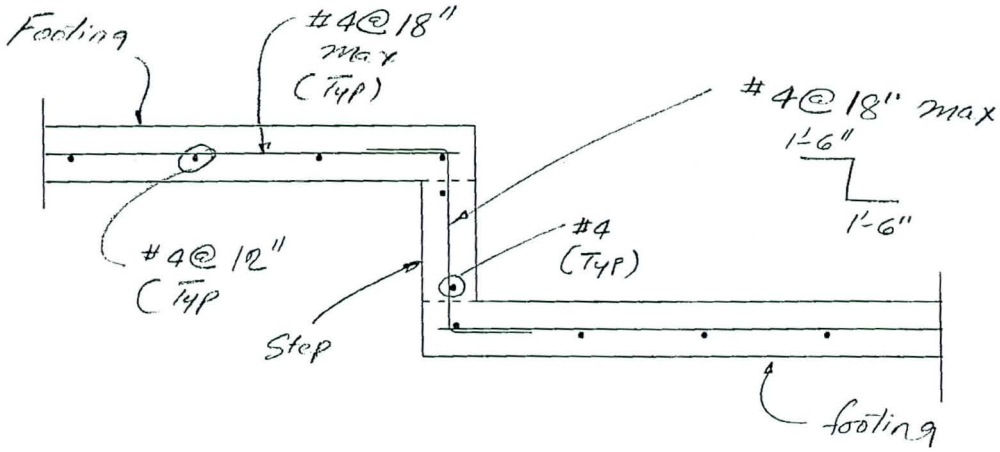
Design depth of water 12", 18", 24"



See revised reinforcement next page.

Containment Wall





STEP DETAIL

FLOOD CONTAINMENT WALL

Water depth 12"

Soil Data:

Depth of embedment 1'-6"
Allowable bearing pressure: 1500 PSF
Unit weight of dry soil: 110 lbs/cft
Unit weight of wet soil: 132 lbs/cft

$abp := 1.5 \quad ksf$
$wsoil := 0.110$ k/cft
$wwsoil := 0.132$ k/cft

Backfill pressure (site soil)
At rest - restrained wall (unsaturated): 55 lbs/sft
At rest - restrained wall (saturated): 90 lbs/sft

$psoil := 0.090$ k/sf

Coefficient of base friction:
Soil (dry or saturated): 0.45

$\mu := 0.45$

Water:
Unit pressure of water: 62.4 lbs/sft

$pwater := 0.0624$ k/sf

Wind:
Wind pressure: Use 15 lbs/sft

$pwind := 0.015$ k/sf

Concrete f'c = 3,000 PSI
Steel fy = 60,000 PSI
Use Ultimate strength

$fc := 3 \quad ksi$
$fy := 60 \quad ksi$

Wall:
Thickness = 8"
Height above water surface = 6"
Depth of water = 12"
Depth below bottom of canal = 8"

$wallt := 0.667 \quad ft$
$t1 := 0.5 \quad ft$
$t2 := 1 \quad ft$ **INPUT**
$t3 := 0.6667 \quad ft$

Footing:
Depth = 8"
Width = 30"
Toe width = 4"
Heel width

$ftgt := 0.6667 \quad ft$ **INPUT**
$ftgw := 2.5 \quad ft$ **INPUT**
$toe := 0.33333 \quad ft$ **INPUT**
$heel := ftgw - wallt - toe = 1.5 \quad ft$

Neglect passive resistance.

STABILITY ANALYSIS:

Stabilizing forces:

Footing $ftgwt := ftgw \times ftgt \times 0.150 = 0.25 \quad k$

Arm $ftgarm := \frac{ftgw}{2} = 1.25 \quad ft$

Moment $ftgm := ftgwt \times ftgarm = 0.313 \quad kft$

Wall

$wallwt := (t1 + t2 + t3) \times wallt \times 0.150 = 0.217 \quad k$

$wallarm := toe + \left(\frac{wallt}{2}\right) = 0.667 \quad ft$

$wallm := wallwt \times wallarm = 0.145 \quad kft$

Water

$waterwt := 0.0624 \times heel \times (t2 + t3) = 0.156 \quad k$

$waterarm := ftgw - \frac{heel}{2} = 1.75 \quad ft$

$waterm := waterwt \times waterarm = 0.273 \quad kft$

Soil

$soilwt := wwsoil \times heel \times t3 = 0.132 \quad k$

$soilarm := ftgw - \frac{heel}{2} = 1.75 \quad ft$

$soilm := soilwt \times soilarm = 0.231 \quad kft$

Fill on toe

$fillwt := toe \times t3 \times wwsoil = 0.029 \quad k$

$fillarm := \frac{toe}{2} = 0.167 \quad ft$

$fillm := fillwt \times fillarm = 4.889 \times 10^{-3} \quad kft$

Total Weight $wt := ftgwt + wallwt + waterwt + soilwt + fillwt = 0.784 \quad k$

Stabilizing moment $stams := ftgm + wallm + waterm + soilm + fillm = 0.966 \quad kft$

Overturning forces

Wind

$windf := pwind \times t1 = 7.5 \times 10^{-3} \quad k$

Water

$waterf1 := 0.5 \times 0.0624 \times (t2)^2 = 0.031 \quad k$

$waterf2 := 0.0624 \times t2 \times (t3 + ftgt) = 0.083 \quad k$

$waterf := waterf1 + waterf2 = 0.114 \quad k$

Soil

$soilf := 0.5 \times psoil \times (t3 + ftgt)^2 = 0.08 \quad k$

Horizontal force

$hf := windf + waterf + soilf = 0.202 \quad k$

Overturning moment

Wind

$windm := windf \times \left(ftgt + t3 + t2 + \frac{t1}{2}\right) = 0.019 \quad kft$

Water

Above soil $waterm1 := waterf1 \times \left[\left(\frac{t2}{3}\right) + t3 + ftgt\right] = 0.052 \quad kft$

in soil $waterm2 := waterf2 \times 0.5 \times (t3 + ftgt) = 0.055 \quad kft$

total $waterm12 := waterm1 + waterm2 = 0.107$ *kft*

Soil

Saturated $satm := soilf \times \left(\frac{1}{3}\right) \times (t3 + ftgt) = 0.036$ *kft*

Overturning moments $overtm := windm + waterm12 + satm = 0.162$ *kft*

Eccentricity

$$ecc := \left(\frac{ftgw}{2}\right) - \left[\frac{(stams - overtm)}{wt}\right] = 0.225 \quad ft$$

$$\left(\frac{1}{6}\right) \times ftgw = 0.417 \quad ft$$

ecc < (1/6)xftgw Hence no uplift footing width is acceptable. **Check**

Factor of safety against overturning

$$Ratio1 := \frac{stams}{overtm} = 5.947$$ > 2 OK **Check**

Factor of safety against sliding

$$Ratio2 := \mu \times \frac{wt}{hf} = 1.747$$ > 1.5 OK **Check**

Bearing pressure

At toe end maximum

$$fmax := \left(\frac{wt}{ftgw}\right) \times \left[1 + \left(6 \times \frac{ecc}{ftgw}\right)\right] = 0.483 \quad ksf$$

At heel end minimum

$$fmin := \left(\frac{wt}{ftgw}\right) \times \left[1 - \left(6 \times \frac{ecc}{ftgw}\right)\right] = 0.144 \quad ksf$$

Allowable soil bearing pressure is 1.5 ksf Hence OK **Check**

Minimum Reinforcement for crack control - AASHTO 8.17.1

Concrete strength $fcp := 3000 \quad psi$

Mn > 1.2Mcr

Section width $b := 12 \quad in$

Section depth $D := 8 \quad in$

Mcr = fr x Ig/yt

$fr := 7.5 \times \sqrt{(fcp)} = 410.792 \quad psi$

$Ig := \left(\frac{1}{12}\right) \times b \times D^3 = 512 \quad in4$

$yt := \frac{D}{2} = 4$

$Mcr := fr \times \frac{Ig}{yt} = 5.258 \times 10^4 \quad lbin$

$Mucr := 1.2 \times \frac{Mcr}{12000} = 5.258 \quad kft$

ACI 318-95

$fy = 60 \quad ksi \qquad fcp = 3 \times 10^3 \quad psi \qquad b = 12 \quad in$

$d := wallt \times 12 - 2 - 0.25 = 5.754 \quad in$

$Asmin := \frac{\left(3 \times \sqrt{fcp} \times b \times d\right)}{fy \times 1000} = 0.189 \quad sqin$

Use min #4 bars @ 12" c/c As= 0.20 Sqin

Temp and shrinkage reinforcement AASHTO 8.20

Provide minimum 0.125 sqin per ft in both direction

Use distribution steel #4 @ 18" As=0.13 sqin OK

Wall Design:

Forces

Wind $windfw := pwind \times t1 = 7.5 \times 10^{-3} \quad k$

Water $waterf1w := 0.5 \times 0.0624 \times (t2)^2 = 0.031 \quad k$

$waterf2w := 0.0624 \times t2 \times (t3) = 0.042 \quad k$

Soil $soilfw := 0.5 \times psoil \times (t3)^2 = 0.02 \quad k$

Moment at base

wind $windmw := windfw \times \left(t3 + t2 + \frac{t1}{2}\right) = 0.014$ *kft*

water $waterm1w := waterf1w \times \left(t3 + \frac{t2}{3}\right) = 0.031$ *kft*

$waterm2w := waterf2w \times \left(\frac{t3}{2}\right) = 0.014$ *kft*

$watermw := waterm1w + waterm2w = 0.045$ *kft*

soil $soilmw := soilfw \times \left(\frac{t3}{3}\right) = 4.445 \times 10^{-3}$ *kft*

Moment $momentw := windmw + watermw + soilmw = 0.064$ *kft*

Moment at base is less than Mcr Therefore minimum amount of steel is 1.33 X As required. Provide min #4 @ 12" main reinforcement and #4 @ 18" distribution reinforcement (temp and shrinkage). **Check**

Heel design

Dead load of water, soil and concrete footing

Neglect upward soil bearing pressure

Uniform load $udlh := 0.0624 \times t2 + wwsoil \times t3 + 0.150 \times ftgt = 0.25$ k/ft

Moment at the face of wall

$heelm := udlh \times \left(heel^2\right) \times \left(\frac{1}{2}\right) = 0.282$ *kft*

Moment at the face of wall is less than Mcr Therefore minimum amount steel is 1.33XAs required. Provide min #4 @ 12" main reinforcement and #4 @ 18" distribution reinforcement (temp and shrinkage). Place reinforcement in top of footing. **Check**

Toe design

Moment small.

FLOOD CONTAINMENT WALL

Water depth 18"

Soil Data:

Depth of embedment 1'-6"
Allowable bearing pressure: 1500 PSF — $abp := 1.5 \quad ksf$
Unit weight of dry soil: 110 lbs/cft — $wsoil := 0.110$ k/cft
Unit weight of wet soil: 132 lbs/cft — $wwsoil := 0.132$ k/cft
Backfill pressure (site soil)
At rest - restrained wall (unsaturated): 55 lbs/sft
At rest - restrained wall (saturated): 90 lbs/sft — $psoil := 0.090$ k/sf

Coefficient of base friction:
Soil (dry or saturated): 0.45 — $\mu := 0.45$

Water:
Unit pressure of water: 62.4 lbs/sft — $pwater := 0.0624$ k/sf

Wind:
Wind pressure: Use 15 lbs/sft — $pwind := 0.015$ k/sf

Concrete f'c = 3,000 PSI — $fc := 3 \quad ksi$
Steel fy = 60,000 PSI — $fy := 60 \quad ksi$
Use Ultimate strength

Wall:
Thickness = 8" — $wallt := 0.667 \quad ft$
Height above water surface = 6" — $t1 := 0.5 \quad ft$
Depth of water = 18" — $t2 := 1.5 \quad ft$ **INPUT**
Depth below bottom of canal = 8" — $t3 := 0.6667 \quad ft$

Footing:
Depth = 8" — $ftgt := 0.6667 \quad ft$ **INPUT**
Width = 3 ft — $ftgw := 3 \quad ft$ **INPUT**
Toe width = 4" — $toe := 0.33333 \quad ft$ **INPUT**
Heel width — $heel := ftgw - wallt - toe = 2 \quad ft$

Neglect passive resistance.

STABILITY ANALYSIS:

Stabilizing forces:

Footing	$ftgwt := ftgw \times ftgt \times 0.150 = 0.3$	k
Arm	$ftgarm := \frac{ftgw}{2} = 1.5$	ft
Moment	$ftgm := ftgwt \times ftgarm = 0.45$	kft

Wall

$wallwt := (t1 + t2 + t3) \times wallt \times 0.150 = 0.267$ *k*

$wallarm := toe + \left(\frac{wallt}{2}\right) = 0.667$ *ft*

$wallm := wallwt \times wallarm = 0.178$ *kft*

Water

$waterwt := 0.0624 \times heel \times (t2 + t3) = 0.27$ *k*

$waterarm := ftgw - \frac{heel}{2} = 2$ *ft*

$waterm := waterwt \times waterarm = 0.541$ *kft*

Soil

$soilwt := wwsoil \times heel \times t3 = 0.176$ *k*

$soilarm := ftgw - \frac{heel}{2} = 2$ *ft*

$soilm := soilwt \times soilarm = 0.352$ *kft*

Fill on toe

$fillwt := toe \times t3 \times wwsoil = 0.029$ *k*

$fillarm := \frac{toe}{2} = 0.167$ *ft*

$fillm := fillwt \times fillarm = 4.889 \times 10^{-3}$ *kft*

Total Weight $wt := ftgwt + wallwt + waterwt + soilwt + fillwt = 1.042$ *k*

Stabilizing moment $stams := ftgm + wallm + waterm + soilm + fillm = 1.526$ *kft*

Overturning forces

Wind

$windf := pwind \times t1 = 7.5 \times 10^{-3}$ *k*

Water

$waterf1 := 0.5 \times 0.0624 \times (t2)^2 = 0.07$ *k*

$waterf2 := 0.0624 \times t2 \times (t3 + ftgt) = 0.125$ *k*

$waterf := waterf1 + waterf2 = 0.195$ *k*

Soil

$soilf := 0.5 \times psoil \times (t3 + ftgt)^2 = 0.08$ *k*

Horizontal force

$hf := windf + waterf + soilf = 0.283$ *k*

Overturning moment

Wind $windm := windf \times \left(ftgt + t3 + t2 + \frac{t1}{2}\right) = 0.023$ *kft*

Water

Above soil $waterm1 := waterf1 \times \left[\left(\frac{t2}{3}\right) + t3 + ftgt\right] = 0.129$ *kft*

in soil $waterm2 := waterf2 \times 0.5 \times (t3 + ftgt) = 0.083$ *kft*

total $waterm12 := waterm1 + waterm2 = 0.212$ *kft*

Soil

Saturated $satm := soilf \times \left(\frac{1}{3}\right) \times (t3 + ftgt) = 0.036$ *kft*

Overturning moments $overtm := windm + waterm12 + satm = 0.271$ *kft*

Eccentricity

$$ecc := \left(\frac{ftgw}{2}\right) - \left[\frac{(stams - overtm)}{wt}\right] = 0.296 \quad ft$$

$\left(\frac{1}{6}\right) \times ftgw =$ *ft*

ecc < (1/6)xftgw Hence no uplift footing width is acceptable. **Check**

Factor of safety against overturning

$$Ratio1 := \frac{stams}{overtm} = 5.638$$ > 2 OK **Check**

Factor of safety against sliding

$$Ratio2 := \mu \times \frac{wt}{hf} = 1.661$$ > 1.5 OK **Check**

Bearing pressure

At toe end maximum

$$fmax := \left(\frac{wt}{ftgw}\right) \times \left[1 + \left(6 \times \frac{ecc}{ftgw}\right)\right] = 0.553 \quad ksf$$

At heel end minimum

$$fmin := \left(\frac{wt}{ftgw}\right) \times \left[1 - \left(6 \times \frac{ecc}{ftgw}\right)\right] = 0.142 \quad ksf$$

Allowable soil bearing pressure is 1.5 ksf Hence OK **Check**

Minimum Reinforcement for crack control - AASHTO 8.17.1

Concrete strength $fcp := 3000 \quad psi$

Section width $b := 12 \quad in$

Section depth $D := 8 \quad in$

Mn > 1.2Mcr

Mcr = fr x Ig/yt

$fr := 7.5 \times \sqrt{(fcp)} = 410.792 \quad psi$

$Ig := \left(\frac{1}{12}\right) \times b \times D^3 = 512 \quad in4$

$yt := \frac{D}{2} = 4$

$Mcr := fr \times \frac{Ig}{yt} = 5.258 \times 10^4 \quad lbin$

$Mucr := 1.2 \times \frac{Mcr}{12000} = 5.258 \quad kft$

ACI 318-95

$fy = 60 \quad ksi \qquad fcp = 3 \times 10^3 \quad psi \qquad b = 12 \quad in$

$d := wallt \times 12 - 2 - 0.25 = 5.754 \quad in$

$Asmin := \frac{\left(3 \times \sqrt{fcp} \times b \times \mathrm{d}\right)}{fy \times 1000} = 0.189 \quad sqin$

Use min #4 bars @ 12" c/c As= 0.20 Sqin

Temp and shrinkage reinforcement AASHTO 8.20

Provide minimum 0.125 sqin per ft in both direction

Use distribution steel #4 @ 18" As= 0.0.13 sqin OK

Wall Design:

Forces

Wind $windfw := pwind \times t1 = 7.5 \times 10^{-3} \quad k$

Water $waterf1w := 0.5 \times 0.0624 \times (t2)^2 = 0.07 \quad k$

$waterf2w := 0.0624 \times t2 \times (t3) = 0.062 \quad k$

Soil

$soilfw := 0.5 \times psoil \times (t3)^2 = 0.02 \quad k$

Moment at base

wind $windmw := windfw \times \left(t3 + t2 + \frac{t1}{2}\right) = 0.018$ *kft*

water $waterm1w := waterf1w \times \left(t3 + \frac{t2}{3}\right) = 0.082$ *kft*

$waterm2w := waterf2w \times \left(\frac{t3}{2}\right) = 0.021$ *kft*

$watermw := waterm1w + waterm2w = 0.103$ *kft*

soil $soilmw := soilfw \times \left(\frac{t3}{3}\right) = 4.445 \times 10^{-3}$ *kft*

Moment $momentw := windmw + watermw + soilmw = 0.125$ *kft*

Moment at base is less than Mcr Therefore minimum amount of steel is 1.33XAs required. Provide min #4 @ 12" main reinforcement and #4 @ 18" distribution reinforcement (temp and shrinkage). **Check**

Heel design

Dead load of water, soil and concrete footing

Neglect upward soil bearing pressure

Uniform load $udlh := 0.0624 \times t2 + wwsoil \times t3 + 0.150 \times ftgt = 0.282$ k/ft

Moment at the face of wall

$$heelm := udlh \times \left(heel^2\right) \times \left(\frac{1}{2}\right) = 0.563 \quad kft$$

Moment at the face of wall is less than Mcr Therefore minimum amount of steel is 1.33XAs required. Provide min #4 @ 12" main reinforcement and #4 @ 18" distribution reinforcement (temp and shrinkage). Place reinforcement in top of footing. **Check**

Toe design

Moment small.

FLOOD CONTAINMENT WALL

Water depth 24"

Soil Data:

Depth of embedment 1'-6"
Allowable bearing pressure: 1500 PSF — $abp := 1.5 \quad ksf$
Unit weight of dry soil: 110 lbs/cft — $wsoil := 0.110$ k/cft
Unit weight of wet sol: 132 lbs/cft — $wwsoil := 0.132$ k/cft
Backfill pressure (site soil)
At rest - restrained wall (unsaturated): 55 lbs/sft
At rest - restrained wall (saturated): 90 lbs/sft — $psoil := 0.090$ k/sf

Coefficient of base friction:
Soil (dry or saturated): 0.45 — $\mu := 0.45$

Water:
Unit pressure of water: 62.4 lbs/sft — $pwater := 0.0624$ k/sf

Wind:
Wind pressure: Use 15 lbs/sft — $pwind := 0.015$ k/sf

Concrete f'c = 3,000 PSI — $fc := 3 \quad ksi$
Steel fy = 60,000 PSI — $fy := 60 \quad ksi$
Use Ultimate strength

Wall:
Thickness = 8" — $wallt := 0.667 \quad ft$
Height above water surface = 6" — $t1 := 0.5 \quad ft$
Depth of water = 24" — $t2 := 2 \quad ft$ **INPUT**
Depth below bottom of canal = 8" — $t3 := 0.6667 \quad ft$

Footing:
Depth = 8" — $ftgt := 0.6667 \quad ft$ **INPUT**
Width = 3'-9" — $ftgw := 3.75 \quad ft$ **INPUT**
Toe width = 4" — $toe := 0.33333 \quad ft$ **INPUT**
Heel width — $heel := ftgw - wallt - toe = 2.75 \quad ft$

Neglect passive resistance.

STABILITY ANALYSIS:

Stabilizing forces:

Footing — $ftgwt := ftgw \times ftgt \times 0.150 = 0.375 \quad k$

Arm — $ftgarm := \frac{ftgw}{2} = 1.875 \quad ft$

Moment — $ftgm := ftgwt \times ftgarm = 0.703 \quad kft$

Wall

$wallwt := (t1 + t2 + t3) \times wallt \times 0.150 = 0.317 \quad k$

$wallarm := toe + \left(\frac{wallt}{2}\right) = 0.667 \quad ft$

$wallm := wallwt \times wallarm = 0.211 \quad kft$

Water

$waterwt := 0.0624 \times heel \times (t2 + t3) = 0.458 \quad k$

$waterarm := ftgw - \frac{heel}{2} = 2.375 \quad ft$

$waterm := waterwt \times waterarm = 1.087 \quad kft$

Soil

$soilwt := wwsoil \times heel \times t3 = 0.242 \quad k$

$soilarm := ftgw - \frac{heel}{2} = 2.375 \quad ft$

$soilm := soilwt \times soilarm = 0.575 \quad kft$

Fill on toe

$fillwt := toe \times t3 \times wwsoil = 0.029 \quad k$

$fillarm := \frac{toe}{2} = 0.167 \quad ft$

$fillm := fillwt \times fillarm = 4.889 \times 10^{-3} \quad kft$

Total Weight $wt := ftgwt + wallwt + waterwt + soilwt + fillwt = 1.421 \quad k$

Stabilizing moment $stams := ftgm + wallm + waterm + soilm + fillm = 2.581 \quad kft$

Overturning forces

Wind

$windf := pwind \times t1 = 7.5 \times 10^{-3} \quad k$

Water

$waterf1 := 0.5 \times 0.0624 \times (t2)^2 = 0.125 \quad k$

$waterf2 := 0.0624 \times t2 \times (t3 + ftgt) = 0.166 \quad k$

$waterf := waterf1 + waterf2 = 0.291 \quad k$

Soil

$soilf := 0.5 \times psoil \times (t3 + ftgt)^2 = 0.08 \quad k$

Horizontal force

$hf := windf + waterf + soilf = 0.379 \quad k$

Overturning moment

Wind $windm := windf \times \left(ftgt + t3 + t2 + \frac{t1}{2}\right) = 0.027 \quad kft$

Water

Above soil $waterm1 := waterf1 \times \left[\left(\frac{t2}{3}\right) + t3 + ftgt\right] = 0.25 \quad kft$

in soil $waterm2 := waterf2 \times 0.5 \times (t3 + ftgt) = 0.111 \quad kft$

total $waterm12 := waterm1 + waterm2 = 0.361$ *kft*

Soil

Saturated $satm := soilf \times \left(\frac{1}{3}\right) \times (t3 + ftgt) = 0.036$ *kft*

Overturning moments $overtm := windm + waterm12 + satm = 0.423$ *kft*

Eccentricity

$$ecc := \left(\frac{ftgw}{2}\right) - \left[\frac{(stams - overtm)}{wt}\right] = 0.356 \quad ft$$

$\left(\frac{1}{6}\right) \times ftgw = 0.625$ *ft*

ecc < (1/6)xftgw Hence no uplift footing width is acceptable. **Check**

Factor of safety against overturning

$$Ratio1 := \frac{stams}{overtm} = 6.101$$ > 2 OK **Check**

Factor of safety against sliding

$$Ratio2 := \mu \times \frac{wt}{hf} = 1.688$$ > 1.5 OK **Check**

Bearing pressure

At toe end maximum

$$fmax := \left(\frac{wt}{ftgw}\right) \times \left[1 + \left(6 \times \frac{ecc}{ftgw}\right)\right] = 0.595 \quad ksf$$

At heel end minimum

$$fmin := \left(\frac{wt}{ftgw}\right) \times \left[1 - \left(6 \times \frac{ecc}{ftgw}\right)\right] = 0.163 \quad ksf$$

Allowable soil bearing pressure is 1.5 ksf Hence OK **Check**

Minimum Reinforcement for crack control - AASHTO 8.17.1

Concrete strength $fcp := 3000 \quad psi$

Mn > 1.2Mcr

Section width $b := 12 \quad in$

Section depth $D := 8 \quad in$

Mcr = fr x Ig/yt

$fr := 7.5 \times \sqrt{(fcp)} = 410.792 \quad psi$

$Ig := \left(\frac{1}{12}\right) \times b \times D^3 = 512 \quad in4$

$yt := \frac{D}{2} = 4$

$Mcr := fr \times \frac{Ig}{yt} = 5.258 \times 10^4 \quad lbin$

$Mucr := 1.2 \times \frac{Mcr}{12000} = 5.258 \quad kft$

ACI 318-95

$fy = 60 \quad ksi \qquad fcp = 3 \times 10^3 \quad psi \qquad b = 12 \quad in$

$d := wallt \times 12 - 2 - 0.25 = 5.754 \quad in$

$Asmin := \frac{\left(3 \times \sqrt{fcp} \times b \times \mathrm{d}\right)}{fy \times 1000} = 0.189 \quad sqin$

Use min #4 bars @ 12" c/c As= 0.20 Sqin

Temp and shrinkage reinforcement AASHTO 8.20

Provide minimum 0.125 sqin per ft in both direction

Use distribution steel #4 @ 18" As= 0.0.13 sqin OK

Wall Design:

Forces

Wind $windfw := pwind \times t1 = 7.5 \times 10^{-3} \quad k$

Water $waterf1w := 0.5 \times 0.0624 \times (t2)^2 = 0.125 \quad k$

$waterf2w := 0.0624 \times t2 \times (t3) = 0.083 \quad k$

Soil $soilfw := 0.5 \times psoil \times (t3)^2 = 0.02 \quad k$

Moment at base

wind $windmw := windfw \times \left(t3 + t2 + \frac{t1}{2}\right) = 0.022$ *kft*

water $waterm1w := waterf1w \times \left(t3 + \frac{t2}{3}\right) = 0.166$ *kft*

$waterm2w := waterf2w \times \left(\frac{t3}{2}\right) = 0.028$ *kft*

$watermw := waterm1w + waterm2w = 0.194$ *kft*

soil $soilmw := soilfw \times \left(\frac{t3}{3}\right) = 4.445 \times 10^{-3}$ *kft*

Moment $momentw := windmw + watermw + soilmw = 0.22$ *kft*

Moment at base is less than Mcr Therefore minimum amount of steel is 1.33XAs required. Provide min #4 @ 12" main reinforcement and #4 @ 18" distribution reinforcement (temp and shrinkage). **Check**

Heel design

Dead load of water, soil and concrete footing

Neglect upward soil bearing pressure

Uniform load $udlh := 0.0624 \times t2 + wwsoil \times t3 + 0.150 \times ftgt = 0.313$ k/ft

Moment at the face of wall

$$heelm := udlh \times \left(heel^2\right) \times \left(\frac{1}{2}\right) = 1.183 \quad kft$$

Moment at the face of wall is less than Mcr Therefore minimum amount of steel is 1.33XAs required. Provide min #4 @ 12" main reinforcement and #4 @ 18" distribution reinforcement (temp and shrinkage). Place reinforcement in top of footing. **Check**

Toe design

Moment small.

QUALITY ASSURANCE PLAN

CONSTRUCTION QUALITY ASSURANCE PLAN
ROLLER COMPACTED CONCRETE DAM

CONTRACT FCD 2010C038

Oak Street Detention Basin & Storm Drain Project

PCN 420.04.31

September 28, 2012

Prepared by:

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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
CONTRACT FCD 2010C038
OAK STREET BASIN & STORM DRAIN PROJECT
ROLLER COMPACTED CONCRETE DAM

CONSTRUCTION QUALITY ASSURANCE PLAN

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1.0 INTRODUCTION

This Construction Quality Assurance (CQA) Plan applies to the provision of Construction Quality Assurance (CQA) services for the procurement and construction of components of the Oak Street Roller Compacted Concrete (RCC) Dam. The CQA Plan only applies to the construction of the RCC dam. This CQA Plan has been prepared on behalf of the Flood Control District of Maricopa County (District), the operator of the proposed Oak Street Basin & Storm Drain Project located in east Mesa, Arizona.

1.1 Plan Purpose

The purpose of this CQA Plan is to provide a project-specific technical guide to the Owner, the Owner's representatives, and CQA testing subcontractor to ensure a quality project, defensible documentation, and conformance with the Plans and Special Provisions for the construction of the RCC dam. The CQA Plan details the minimum requirements of the CQA personnel during construction activities.

The Contractor will provide quality control as required by the Special Provisions. The basis for acceptance or rejection of all work will be the Construction Quality Assurance inspection, testing and documentation, and the CQA and Contractor Quality Control (CQC) testing results. The CQA inspection and documentation requirements and testing frequencies listed in this CQA Plan in no way relieve the Contractor from achieving the CQC performance requirements detailed in the Special Provisions.

1.2 Contractor Quality Control (CQC) Requirements

The Contractor's Quality Control system will be responsible to maintain the quality of the work by: (1) controlling the quality of supplies, materials, and services; (2) providing to the District for acceptance only those supplies, materials, services, and construction processes that conform to the construction documents; and (3) maintaining substantiating evidence that supplies, materials, services and construction processes conform to the requirements of the Special Provisions. The Contractor must take corrective action to keep the work in compliance with the construction documents. This quality control system includes continuous field monitoring as well as testing during construction of the RCC dam component of the project.

The major items of work that will require quality control, as listed by the applicable Special Provisions, are:

- Section 208 – Foundation Excavation & Preparation
- Section 222 – Roller Compacted Concrete

The material quantities are provided in the Plans.

1.3 Scope of This Document

The overall goal of the quality assurance program and the Contractor's quality control plan is to complete the construction with the high quality expected on this project and in conformance with the approved Plans and Special Provisions. The achievement of these objectives relies upon the execution of this CQA Plan and the adherence of the Contractor to their Construction Quality Control plan, specifically the field monitoring and documentation of the activities. This CQA plan therefore outlines in detail the CQA procedures that are provided and shall be considered in conjunction with the project contract, Plans and Special Provisions. The construction activities requiring CQA procedures in this document include:

- Meetings
- Measurement and Payment Verification
- Site Visits and Observations
- Earthwork
- Roller Compacted Concrete

Any conflict between the requirements of this document and the approved Plans and Special Provisions shall be reported to the Engineer of Record, for clarification or adjudication, as required. In general, however, the requirements of the Special Provisions shall prevail.

1.4 Definition of Quality Control Versus Quality Assurance

There is often considerable confusion between the definition of quality control and quality assurance. This document refers to the provision of quality control and quality assurance for various components of the project:

- **Quality Control (QC)** refers to those actions taken by all parties involved in the construction, including the Contractor, those parties charged with procurement and installation of manufactured materials, and the placement and compaction of the soil materials, which provide a means to determine and sometimes quantify the characteristics of the product. The results of a quality control program are compared to the Special Provisions or other contractual or regulatory requirements. During each aspect of the handling of these materials, quality control is provided by the manufacturer, fabricator, or installer of materials, or the supplier and earthworks Contractor for the soils, to ensure that the materials and workmanship conform to the Plans and Special Provisions. Quality control responsibility is retained by the Contractor, suppliers, and manufacturers because these entities have the most direct control over qualifications of personnel, specialized experience or expertise, choice in type and quantity of equipment, scheduling, sequencing and workmanship that all factor in to the quality of the finished project.

- **Quality Assurance (QA)** is a planned and systematic pattern of all means and actions intended to provide adequate confidence that the materials and procedures conform to the Plans and Special Provisions, and any applicable regulatory requirements. Quality assurance can be provided by the Owner, or its designated representative, which often is an independent consulting, engineering or construction management firm. Activities performed by quality assurance personnel include: observing construction methods and procedures, reviewing the Contractor's quality control testing, conducting materials testing for evaluation of Contractor's quality control, and other measures to ensure compliance with the contract Special Provisions. Although quality assurance is as important during all phases of the project, construction quality assurance is often in association with those actions taken in relation to the installation of the RCC materials and the placement and compaction of soils materials. Quality assurance is a critical component of a project because field conditions are the most variable and the most difficult to control and documentation is being recognized as invaluable to Owners and other interested parties. The activities and duties for quality assurance personnel are outlined in this Construction Quality Assurance Plan.

It is critical that QA field and laboratory personnel work separately from comparable QC personnel to ensure independent observation, evaluation, and testing results.

2.0 PARTIES TO THE WORK AND RESPONSIBILITIES

The successful completion of the project is dependent on the interaction and cooperation of many parties. The following parties are represented in the project. Specific individuals, agencies and/or firms to be assigned to these roles will be designated at the time of the pre-construction meeting.

- **Construction Manager.** The Construction Manager for this project is responsibility for all activities on-site during construction of the Oak Street Basin & Storm Drain Project, he will function as the **CQA Director** on this project.

- **CQA Engineer of Record.** The CQA Engineer of Record for this project will support the CM/CQA Director and supervise the CQA Project Engineer and required Specialists.

- **CQA Project Engineer** The CQA Project Engineer will be a registered professional engineer (PE) in the State of Arizona and work directly for the CQA Engineer of Record.

- **CQA Consultant.** The CQA Consultant for this project is responsible for monitoring and documentation of the construction activities of the Contractor, and CQA Materials Testing Laboratory and field inspections. The Owner will retain the CQA Consultant.

- **CQA Manager.** The CQA Manager, will be a designated employee of the consultant retained by the Owner, and participates in the monitoring and documentation of the CQA and CQC activities of the Project on an as needed basis.

- **Construction Materials Testing Laboratory.** The CQA Materials Testing Laboratory is a third-party, independent of the CQA Consultant, responsible for the CQA laboratory and field testing. The Owner will retain the CQA Materials Testing Laboratory.

- **Owner.** The Owner, as used and defined for this document, is the Flood Control District of Maricopa County (District).

- **General Contractor.** The General Contractor (Contractor) is the primary construction contract holder responsible for all construction activities including the specified minimum CQC testing requirements. The Contractor is responsible for QC conformance requirements and submittals required by all subcontractors, vendors, material manufactures, etc. used on this project and directly contracted through the Contractor. Testing methods and requirements for CQC earthwork and materials testing are detailed in the Special Provisions.

2.1 CQA Team

The CQA Team shall consist of the following.

- **Construction Manager/CQA Director.** The CM/CQA Director, demonstrating experience in construction and field oversight, will be involved on a full-time basis in construction project management and provide direction to the Engineer of Record, CQA Consultant, the CQA Manager, the CQA Field Representatives and the CQA Testing Lab. The Construction Manager will direct the CQA Team in the field on a daily basis.

- **CQA Engineer of Record.** The CQA Engineer of Record (EoR) shall be a registered professional engineer in the state of Arizona responsible for certifying completion of construction in accordance with the approved project plans and Special Provisions. The CQA EoR will be kept appraised of field progress and decisions and will periodically visit the site to review the operations and progress of the project.

- **CQA Manager**. The CQA Manager, demonstrating experience in construction and field oversight, will be involved on an as needed basis in construction oversight and liaison activities with the Construction Manager. The CQA Manager will provide construction management insight and guidance as needed to the CQA Team in the field and maintain records of the CQC and CQA data. The CQA Manager will assist the Construction Manager in resolving issues regarding conformance or non-conformance of the work and assist with the determination of corrective actions required for compliance with the construction documents.

- **CQA Project Engineer**. The CQA Project Engineer, who is intimately knowledgeable with the design calculations and design intent, will interface in the field with the Construction Manager to provide technical guidance. The CQA Project Engineer will be employed by and work under the direct supervision of the CQA EoR.

- **CQA Field Representative.** The CQA Field Representative is the onsite, full time representative of the CQA Consultant. The CQA Field Representative liaisons directly with the Construction Manager and the CQA Manager and will coordinate with CQA Technicians on site.

- **CQA Field Technicians.** The CQA Field Technicians, employed by the Owner, perform CQA laboratory and field testing, and reporting. CQA Field Technicians may be used on an on-call, as-needed basis.

The specific functions and responsibilities of these personnel are presented in the following sections.

2.1.1 Construction Manager

The Construction Manager has the responsibility for overseeing all construction and related activities for the project. The Construction Manager will be assisted by the EoR and the CQA Manager with the following activities that relate to the CQA Plan:

- Review the CQA Plan, design, Plans and Special Provisions for the project.
- Administer the CQA Program as Director with the CQA Manager, including the supervision of CQA Field Representative and CQA Field Technician.
- Conduct Pre-Bid and Pre-Construction conferences.
- Conduct periodic progress meetings and conduct site visits.
- Scheduling of CQA Team activities.
- Review daily field reports and results of CQA and CQC field and laboratory testing for conformance and acceptability.
- Collect, collate and review the documentation provided by the Contractors and their suppliers of the materials to be used on the project.
- Jointly conduct with the CQA Manager and the CQA Project Engineer the approval of the foundation surface prior to the placement of RCC.
- Observe and verify by review of data made available by the Contractor that construction is performed to the depths, lines and grades as indicated on the Plans.
- Review, approve and process periodic progress payments to the Contractor.
- Review the schedule and progress and provide recommendations for corrective actions, if any.
- Observe construction procedures and, with the CQA Manager and CQA Project Engineer, ensure that the intent of design is being met.
- Review documentation and observe Contractor's conformance with SWPPP requirements.
- Assist in resolving potential issues that may come up, including, but not limited to, schedules, non-conformance to Plans and Special Provisions, methods, equipment, payment and sequencing.
- Conduct "punch list" development and final inspections and documentation.
- Assist with preparing the CQA Final Report.
- Assign the daily responsibilities of all CQA Field Technicians, to ensure that all relevant activities of the Contractor are monitored and documented.

2.1.2 CQA EOR

The CQA EoR will provide technical support to the CQA Manager, CQA Project Engineer, and CQA Field Representatives. In particular, the CQA EoR will conduct the following:

- Review the CQA Plan, design, Plans and Special Provisions for the project.
- Monitor the CQA Program with the Construction Manager, including supervision of the CQA Project Engineer.
- Review progress with the Construction Manager and the CQA Project Engineer.
- Participate in weekly progress meetings.
- Periodically visit the site to review progress of the project.
- Review and approve any proposals for changes to the design, Plans or Special Provisions that may be necessitated by field conditions.
- Review and approve the CQA Final Report.
- Periodically review and comment on the Contractor's submissions of on-site red-line As-Built drawings.

2.1.3 CQA Manager

The CQA Manager is involved in the office and fieldwork and will conduct the following:

- Review the CQA Plan, design, Plans and Special Provisions for the project.
- Co-administer the CQA Program with the Construction Manager, including the supervision of the CQA Field Representatives
- Attend Pre-Bid and Pre-Construction conferences.
- Attend periodic progress meetings and conduct site visits.
- Review daily field reports, and results of field and laboratory testing from the CQA Field Representative and CQA and CQC Technicians for conformance and acceptability.
- Review CQC submittals, from the Contractor, of the daily reports and results of the field and laboratory testing from the Contractor's CQC system.
- Review the documentation provided by the CM of the Contractor's and their suppliers of the materials to be used on the project.
- Coordinate with the CM and the CQA Project Engineer the approval of the foundation surface prior to the placement of fill.

- Verify by review of data made available by the Contractor that construction is performed to the depths, lines and grades as indicated on the Plans.
- Observe construction procedures and advise the CM that the intent of design is being met.
- Assist in resolving potential issues that may come up, including, but not limited to, non-conformance to Plans and Special Provisions, methods and equipment.
- Assist with punch list development and final inspections.
- Prepare the CQA Final Report.

2.1.4 CQA Project Engineer

The CQA Project Engineer will be an integral part of the team both in the office and field by conducting the following:

- Review the CQA Plan, design, Plans and Special Provisions for the project.
- Review the documentation provided by the CM of any Contractor's submittals from their suppliers of the materials to be used on the project.
- Attend progress meetings (as necessary).
- Conduct routine site visits.
- Coordinate with the CM and the CQA Manager in the approval of the foundation and foundation surface preparation prior to placement of RCC materials.
- Provide design clarification as requested.
- Provide technical direction on critical start-up activities, as requested.
- Review, as requested, any Contractor initiated proposals for changes to the design, plans or Special Provisions that may be necessitated by field conditions.
- Assist with final acceptance of CQA Final Report.

2.1.5 CQA Field Representative

The CQA Field Representative is the on-site representative of the CQA Manager. The CQA Field Representative will conduct the following:

- Serve as the representative of the CQA Manager on site.
- Review the CQA Plan, drawings and Special Provisions for the project, and ensure that all CQA Consultant Field Personnel are fully informed of and qualified for the requirements of the work.

- Review the Contractor's CQC field data, to adherence to the Special Provisions.
- Observe, inspect and document the work methods and procedures of the Contractor to ensure that it complies with the Special Provisions.
- Observe, inspect and document the Contractor's dust control efforts.
- Be responsible for the submittal of CQA daily field reports (notes) and test results to the CM and the CQA Manager documenting the activities of the Contractor for each day worked.
- Attend all progress meetings as required plus any activity-specific meetings necessary to review the installation of a critical component and/or CQA activities.
- Report on both a verbal basis, and through periodic submittal of the daily CQA reports, to the CQA Manager to ensure that any problems are identified and communicated to all parties of the project on a timely basis.
- Prepare photo documentation of all project activities and distribute as necessary on a daily basis.
- Assist with final acceptance of CQA Final Report.

2.1.6 CQA Field Technicians

One or more ASTM qualified CQA Materials Testing Technicians will be assigned to the project on an on-call basis to conduct conformance testing. The CQA Testing Technician will be available as needed whenever earthwork or RCC construction is taking place. The activities to be monitored and duties to be carried out within the scope of the overall CQA Program include:

- Schedule, observe, perform and/or report construction materials testing results.
- Select sample locations for CQA testing based on observations of the work performed and of the locations utilized for the quality control testing. Perform the number of tests required by these observations and in accordance with the minimum frequencies and test requirements as specified in Table 1 of the this CQA plan.
- Perform the collection and testing of all samples required for checking for conformance to the Special Provisions. Testing type and minimum frequencies are estimated in Table 1; however, the actual test selection and frequencies of testing may be increased at the direction of the CM and at the discretion of the CQA Manager based on field conditions and / or the construction sequence.
- Prepare daily field reports (notes) documenting their activities for each day worked for transmission to the CM.

2.2 Submittal of Inspection, Documentation and Testing Data

The timely submittal and distribution of CQC and CQA inspection, documentation and testing data is important and required for this project. The CQA inspection, documentation and testing data shall be submitted in a timely manner to the appropriate parties in the following sequence:

The CQA Manager shall receive and review the CQA inspection and testing data, within 48 hours of the work inspected or tested and distribute as necessary.

3.0 MEETINGS

Meetings of all parties are required at various times during the project based on the following objectives:

- Establish work schedules.
- Review Progress.
- Resolve problems.
- Generally maintain good lines of communication.

3.1 Preconstruction Meeting

The Pre-construction Meeting is held in advance of the start of construction, to introduce all parties, and resolve any particular issues prior to the commencement of work, and to establish the requirements for construction quality assurance. The Construction Manager will conduct the pre-construction meeting.

The following is a typical agenda for a pre-construction meeting:

- Use of site by Contractor and Owner.
- Owner's contract or site requirements.
- Construction facilities and temporary controls provided by the Contractor.
- Survey layout.
- Security protocols.
- Housekeeping procedures.
- Public relations and confidentiality protocols.
- Inspections required.
- CQA and CQC of major or critical activities in the project and a methodology.
- Proposed schedules and sequence of activities.
- Identification of the responsibilities of the project team.
- The timing and distribution of project correspondence.
- Establish the lines of authority and communication.
- Health and safety.

The Pre-construction Meeting may also be concluded with a site walk-around to determine the status of activities, and re-discuss items during the meeting (if necessary). This meeting shall be documented by the Contractor and minutes prepared and circulated to all present, plus any other interested parties, for approval and/or revision.

3.2 Progress Meetings

Periodic progress meetings shall be held on a schedule to be determined by the Construction Manager in order to review the status of the schedule, problems, and measures for resolution of problems. These meetings shall be documented, as required, and the decisions reached promulgated to all affected parties.

Areas of concern and potential future problems shall also be outlined, and addressed at the next planned Progress Meeting, unless of sufficient importance or urgency as to warrant an *ad hoc* meeting. The following is a typical agenda for a progress/weekly meeting:

- Review minutes of previous meetings.
- Review work progress.
- Field observations, problems, and decisions.
- Identification of problems that impede planned progress.
- Review submittals schedule and status of submittals.
- Review health and safety concerns and issues.
- Revisions to progress schedule.
- Corrective measures to regain projected schedules.
- Planned progress during succeeding work period.
- Coordination of projected progress.
- Effect of proposed changes on progress schedule and coordination.
- Potential changed conditions or review of change order submittals.

3.3 Project Partnering Meetings

Pursuant to the Supplementary General Conditions, periodic Project Partnering meetings will be scheduled to maintain the desired partnering relationships.

4.0 MEASUREMENT AND PAYMENT VERIFICATION

Based on the final construction documents and bid schedule, the periodic project progress quantities are verified in the field using total count of items or survey of in-place volumes. Measurement calculation shall be initiated in the field by the Construction Manager and supported by a registered land surveyor licensed in the State of Arizona, as required. The calculations are reviewed or checked by a second method to assess reasonableness. Estimated pay measurements will be reported, and any discrepancies will be explained.

4.1 Payment Review and Approval

Upon request of the CQA Director, the CQA Manager will review the Contractor's pay requests to render an opinion of the project progress in light of the amount requested. The review is to include a written analysis discussing the major pay items, make suggested adjustments and note any discrepancies. Finally, again at the request of the CQA Director, the CQA Manager is to provide conclusions and recommendations for approval or rejection of the pay request.

5.0 SITE VISITS AND GENERAL OBSERVATIONS

The CQA Manager and CQA Project Engineer shall conduct independent, periodic site visits, to ensure that all outstanding issues are resolved on a timely basis, and to review personally the progress and methodology of the Contractor. The schedule of these site visits will be determined by project demands. The CQA Manager shall monitor all phases of construction on an as needed basis, and ensure that in his absence another CQA Team member is present during all phases of construction. In addition, the CQA EoR will make site visits when a problem arises which cannot be easily resolved or which impacts the design of the facility. In that regard, the CQA EoR should make periodic site visits in order to review the progress and any aspects of the project that are particularly critical to the performance of the project.

6.0 EARTHWORK

The earthwork associated with the project will consist of the installation of earthen fill materials. The CQA and CQC testing procedures for earthwork materials are presented in the following subsections.

6.1 Soils Testing

6.1.1 Laboratory Soils Testing

Laboratory Soils Testing will include CQC testing by the Contractor's Construction Materials Testing Laboratory and CQA testing by the owner's CQA Testing Laboratory. CQC Laboratory testing of the soils materials to be used at the site shall be carried out for the purpose of materials selection prior to construction and for materials quality control and evaluation during construction operations. The CQA testing lab will perform verification testing of all materials.

All field and laboratory testing shall be correlated with the actual location the materials are placed, including stationing, offset, and elevation to the greatest extent possible.

6.1.2 Laboratory Conformance and Quality Assurance Testing

On site tests are to be carried out to provide and ensure that the source of the materials does not vary significantly or adversely from one area of the source to another and that the properties that are required in the Special Provisions are met. The minimum CQA frequency for a given test is shown in Table 1.

All field and laboratory testing shall be correlated with the actual location the materials are placed including stationing, offset, and elevation to the greatest extent possible.

6.1.3 Laboratory Testing Frequency

The frequency of testing required during the selection process for soil materials is a function of the quantity of each soil type required, in addition to the existing documentation of the source. In general, however, CQC testing shall be conducted in accordance with the requirements of the project Special Provisions, and CQA compliance testing at the minimum frequencies as shown in Table 1 for each test procedure.

It should be noted that, in all cases, at least one test will be completed, regardless of the quantity of materials placed and compacted, where relevant. The CQA Testing Laboratory shall review all CQA laboratory test results and forward a summary of all CQA testing to the CM.

CQA laboratory testing shall be conducted in accordance with the requirements of the project Special Provisions, and, at the minimum frequencies as shown in Table 1 for each test procedure.

6.1.4 Field Soils Testing

The Contractor's CQC testing lab shall be responsible for providing field *in situ* testing of the soils after placement and compaction, to determine their as-compacted properties and confirm conformance with the Special Provisions. The CQA testing lab shall complete field CQA testing as a component of the CQA Plan. The principal *in situ* testing carried out is the field determination of density and moisture content.

The CQA testing lab shall conduct the field soils testing in accordance with the requirements of the project Special Provisions, and, at the minimum frequencies as shown in Table 1 for each test procedure.

All field and laboratory testing shall be correlated with the actual location the materials are placed including stationing, offset, and elevation to the greatest extent possible.

6.2 Contractor's Submittals

In support of the CM, the CQA Manager and the CQA EoR may conduct the following activities regarding the Contractor's materials submittals:

- Review the required Contractor's submittals.
- Review Contractor's Quality Control plan.
- Review submittals for landfill and disposal documentation, if required.
- Review submittals for measurement and pay requests.
- Review submittals for as-built data.
- Review the results of all CQC testing completed by the Contractor's Construction Materials Testing Laboratory.

6.3 Observations

The CQA Field Representative shall conduct the following observations:

- Contractor's daily activities.
- SubContractor's daily activities.
- Surveyor's activities.
- Excavation limits.
- Moisture conditioning.
- Fill and backfill placement.
- Construction materials testing.

6.4 Excavations and Foundation Preparation

The Contractor will be required to set and check all stakes as the work progresses, set grade stakes for all final excavated grades, conduct any and all surveys required for progress payments, and make the required computations for quantities for progress payments.

Intermittent quality assurance inspection of foundation excavations will be required. Materials encountered will be examined to determine their suitability for specified earthfills and foundations for the RCC dam.

Prior to any fill or RCC placement activities the CQA Director, the CQA Manager and the CQA Project Engineer, will perform a visual inspection of the foundation excavation to determine the conditions and any variations of the native materials present. Any required over excavation of unsuitable materials will be performed prior to visual acceptance. Upon approval of the foundation the Contractor will be authorized to proceed per the specifications with foundation preliminary cleanup followed by foundation surface preparation.

The CQA Field Representative shall conduct the following prior to and during excavation activities:

- Verify clearing, grubbing, and stripping has been conducted as necessary.
- Verify the excavation limits are established and agreed upon.
- Observe and verify unsuitable material (i.e. oversize rock and construction debris) does not get commingled with fill materials that may be reused.
- Coordinate foundation excavation and preparation activities with the CQA Director.

The CQA Project Engineer may conduct the following during foundation excavation and preparation activities:

- Review the Contractor's work plans.
- Verify the excavations are conducted to the limits and thickness required as shown on the drawings and that all unacceptable materials are removed from the bottom of the foundation.
- Identify excavation activities and visually inspect the over excavation of unacceptable materials.
- Verify foundation cleaning activities are performed by the Contractor in accordance with the Special Provisions.
- Confirm that all foundation surfaces are prepared in accordance with the Special Provisions.

6.5 Fill Materials

Fill material will be obtained from the required borrow area excavations. The Contractor will provide quality control and testing as required to maintain the quality of the work. The Contractor's quality control activities shall include but are not limited to such items as lift thickness, moisture control, water application, compactive effort, uniformity of processing, and the condition of the subgrade. The Contractor's minimum frequency of testing will be as assigned in the Contract Documents. If noncompliance work is detected, the Contractor's quality control testing frequency may be increased as well as changes made in the Contractor's operation to correct the problem. These changes are to be made based on the quality control and quality assurance testing results, rather than making corrections based only on the quality assurance testing program. The Contractor will be required to provide signed documented evidence that the tests have been conducted and the fill meets the contract requirements. The test results and other control documentation will be provided on a daily basis.

The Contractor will be required to set and check all supplemental stakes as the work progresses and set grade stakes for the final constructed grade. The Contractor will also conduct any and all surveys required for progress payments, and make the required computations for quantities for progress payments.

Continuous quality assurance inspection and documentation of the fill placement by the CQA Field Representative will be required as well as verification of moisture and compaction requirements. Testing of the fill will be coordinated with the Contractor's testing to fully utilize all quality assurance and quality control efforts. The CQA Field Representative shall monitor the lift thickness, water application, compactive effort, suitability of the fill material, and uniformity of the processing.

Testing of the fill materials shall be on a random basis during placement or shall be selected based on observation of fill placement procedures and observation of quality control testing procedures. The minimum CQA testing frequencies for the fill materials are provided in Table 1. The frequency of testing may be increased by the CM after consultation with the CQA Manager or CQA EoR based on the variability of the materials, water content, results of the Contractor's quality control testing, production, and the amount of deficiencies encountered. Testing of the earthen fill shall be in accordance to the referenced testing standards listed on Table 1.

The CQA Field Representative shall conduct the following prior to and during placement of the fill materials:

- Review the Contractor's work plans
- Verify that the foundation is prepared and has been approved by the CQA Project Engineer before fill materials are placed.
- Verify the foundation is prepared and ready to receive RCC and fill materials.
- Observe the placement of the fill materials per project Special Provisions.
- Observe the compaction by method, equipment, and effort that may be necessary to achieve the desired results in the project Special Provisions.
- Observe and identify areas where the fill surface has dried excessively due to temperature and/or wind effects, been exposed to inclement weather (rain or flooding), or deformed due to other activities by the Contractor. Verify that these areas are scarified to a depth of 3 inches and re-conditioned to the appropriate moisture content, prior to placement of the fill materials.
- Verify the fill materials are placed to the limits and thickness required as shown on the Plans.
- Observe and verify the field density and moisture content are as required by the Special Provisions.

7.0 ROLLER COMPACTED CONCRETE

RCC dam construction will consist of furnishing, placing, and compacting RCC materials. The CQA and CQC testing procedures for RCC materials are presented in the following subsections.

7.1 Contractor's Material Submittals

The CQA Manager and/or the CQA Project Engineer shall review the following Contractor submittals:

- Trial Mix Production Submittals
- RCC Pre-production Submittals
  - Job mix
  - Bonding mortar
  - Plant and equipment
  - Personnel
  - Test section plan
- RCC Test Section Submittals
- RCC Production Submittals
- Review Contractor's Quality Control plan.
- Review submittals for measurement and pay requests.
- Review submittals for as-built data.

7.2 RCC Test Section

The CQA Construction Manager and CQA EoR shall conduct the following prior to and during placement of the RCC test section.

- Participate in the pre-test briefing meeting.
- Verify foundation areas have been prepared per project Special Provisions and are ready to receive RCC.
- Approve test section location.
- Observe as necessary stockpiles of aggregate for use in RCC production, and verify the stockpiled material meets project Special Provisions.
- Observe as necessary production of the RCC, including mixing and equipment used.
- Observe the placement per project Special Provisions.

- Observe the compaction method, equipment, and number of passes that may be necessary to achieve the desired results in the project Special Provisions.
- Monitor placement of the RCC.
- Verify the in-place properties (density and compressive strength) meets project Special Provisions. The CQA Construction Manager shall coordinate CQA sampling, field testing, and laboratory testing with the Contractor's Construction Materials Testing Laboratory in accordance with the QA/QC program specified in the Special Provisions.
- Participate in the post-test briefing.
- Review and approve Contractor's RCC placement plan.

7.3 RCC Placement

The CQA Construction Manager shall conduct the following prior to and during placement of RCC.

- Verify foundation areas have been prepared per project Special Provisions and are ready to receive RCC.
- Observe as necessary construction of stockpiles of aggregate for use in RCC production, and verify the stockpiled material meets project Special Provisions.
- Observe as necessary production of the RCC, including mixing and equipment used.
- Observe the placement per project Special Provisions.
- Observe the compaction by method, equipment, and number of passes that may be necessary to achieve the desired results in the project Special Provisions.
- Verify the RCC is placed to the limits and thickness required as shown on the drawings.
- Monitor placement of the RCC.
- Verify the in-place properties (density and compressive strength) meets project Special Provisions. The CQA Construction Manager shall coordinate CQA sampling, field testing, and laboratory testing with the Contractor's Construction Materials Testing Laboratory in accordance with the QA/QC program specified in the Special Provisions

7.4 RCC Lift Joints

The CQA Construction Manager shall conduct the following activities during and after placement of RCC.

- Determine joint maturity as defined by the Special Provisions.
- Determine joint treatment method in accordance with the Special Provisions.

7.5 RCC Testing

The Contractor's CQC testing lab shall be responsible for providing field *in situ* testing of the RCC after placement and compaction, to determine their as-compacted properties and confirm conformance with the Special Provisions. The CQA testing lab shall complete field CQA testing as a component of the CQA Plan. The principal *in situ* testing carried out is the field determination of density.

The CQA testing lab shall conduct the field soils testing in accordance with the requirements of the project Special Provisions, and, at the minimum frequencies as shown in Table 1 for each test procedure.

All field and laboratory testing shall be correlated with the actual location the materials are placed including stationing, offset, and elevation to the greatest extent possible.

8.0 TESTING EQUIPMENT AND FACILITIES

8.1 Testing Equipment

Testing equipment is required for earthwork and RCC testing. All equipment must be calibrated, serviced, and in good serviceable condition. All scales are to be certified by a scale testing service. Field and laboratory testing equipment meeting the applicable ASTM standards required for this project includes, but is not limited to, the following:

- Sand cone with accessories, including calibration sand
- Proctor mold with accessories
- Scales
- Sieves
- Oven with thermometer
- Sieve shaker
- Nuclear density and moisture gauge

9.0 FIELD DOCUMENTATION

9.1 Daily Field Reports

The CQA Field Representative shall document in daily field reports all significant information related to the RCC dam construction including the following:

- Name
- Company name
- Date
- Start time and ending time
- Weather
- Job reference number
- Contractor or subcontractor onsite
- Major equipment onsite and/or used (or list)
- Page number
- Visitors to the site
- Activities performed that consumed the day
- Activities performed started or restarted that day
- Activities performed started or completed critical to the project
- Provide lift elevation, station, and offset from centerline for each test location and/or inspected area
- Include photographs with labels, titles, and descriptions
- Reference attached test results, sketches, etc

9.2 Document Control

The CQA Field Representative shall collect and maintain the following:

- Contractor work plans and material submittals
- Photographs
- Daily field reports
- All laboratory and field testing results

9.3 Photographs and Video Documentation

The CQA Field Representative shall document the following project activities using digital photographs:

- Existing conditions
- Progress of key activities at the various steps or phases of implementation
- Areas or items that are planned to be buried and not expected to be seen
- Completed components of work as project progress
- Completed project from various views (taken in last few days of work)

The CQA Field Representative shall prepare electronic documentation to include photographs and a brief narrative description of each photo including vantage point and location photo was taken.

The CM reserves the right to perform video documentation during critical construction activities or as determined necessary. The CQA Manager shall coordinate all video documentation with the CM.

10.0 CQA FINAL RCC DAM CONSTRUCTION REPORT

Upon completion of the project, the CQA Consultant will prepare the CQA Final Report. This report will be the final record of the CQA and CQC information for the RCC dam construction. In general, the report shall include all submittal items discussed in this CQA Plan. This shall include, at a minimum:

- Field notes from the installation procedure, including such information as weather and unusual circumstances.
- Summary tables of testing results.
- All manufacturers' conformance data.
- Compilation of copies of photo documentation including video or photographic recordings of the final, as-built condition of the project.
- Field notes during construction and installation.
- The results of field and laboratory testing.
- All Requests for Information and their responses
- All executed Change Orders and their documentation.
- Complete set of the Engineer of Record's as-built drawings.

The CQA RCC Dam Construction Final Report shall also provide a narrative description, in general, of the site's construction, noting all unusual occurrences encountered (i.e., failed test results, extreme weather, etc.). The CQA RCC Dam Construction Final Report shall be provided to the CM for review and approval shortly after the completion of work.

TABLES

TABLE 1
MINIMUM CQA TESTING REQUIRED
(FOR EARTHWORK AND RCC MATERIALS)

Test Method		Frequency of Tests[1][2]		
Reference	**Description**	**Dam Backfill**	**RCC**	**Bonding Mortar**
ASTM D 422	Laboratory Test for Particle Size	1 / 500 cy per material or change of source	1 / 1,000 cy or change of source	N/A
ASTM D 698	Laboratory Test for Moisture-Density Relationships for Fill Materials	1 / 500 cy per material or change of source	N/A	N/A
ASTM D 2216 or ASTM C 4643	Laboratory Test for Moisture Content (oven-dry method or microwave method)	1/10 nuclear tests	N/A	N/A
ASTM D 4318	Atterberg Limits	1 / 500 cy per material or change of source	N/A	N/A
ASTM D 1556	Field Test for In-Place Density (sand cone method)	1/10 nuclear tests	N/A	N/A
ASTM D 6938	Field Test for In-Place Density and Moisture (nuclear method)	1 / 250 cy or 2/lift/shift	N/A	N/A
ASTM C 1040	Field Test for In-Place Density of Unhardened and Hardened Concrete, Including Roller Compacted Concrete, by Nuclear Methods	N/A	1 per 100 cy or 3/lift/shift	N/A
ASTM D 3017	Field Test for Water Content of Soil and Rock in Place by Nuclear Methods(Shallow Depth)	N/A	1 per 100 cy or 3/lift/shift	N/A
ASTM C 1176 or ASTM C 1435	Laboratory Test for Molding Roller-Compacted Concrete in Cylinder Molds Using a Vibrating Table/Hammer	N/A	1 set of 9 per 1,500 cy	N/A
ASTM C 39	Compressive Strength Concrete Cylinder	N/A	1 set of 9 per 1,500 cy	N/A
ASTM C 143	Slump of Concrete	N/A	N/A	1 per 9 cy

1) The frequency of testing presented in this table is based on the minimum testing required. Final frequency and quantity of tests per method per material will be at the discretion of the CQA Project Engineer and CQA Manager.

2) Where different frequencies are indicated for the same test and material (e.g., 1/lift/shift or 1/2,000 cy), the number of tests performed shall be that which results in the greatest frequency.

MAINTENANCE PLAN

MEMORANDUM

To:	**Flood Control District of Maricopa County**
From:	**Gavan & Barker Inc.**
Date:	**October, 2012**
Project:	**Oak Street Basin and Storm Drain Design**
Subject:	**Maintenance Plan**

Purpose

The purpose of this memorandum is to provide a long-term maintenance plan for the Oak Street Basin and Storm Drain system. A post-construction maintenance plan that provides a guide for regular and event-based maintenance is vital to the effectiveness and longevity of the storm drain system.

Project Overview

The Oak Street Basin and Storm Drain project consists of the Oak Street Basin, located on a 10-acre site at the northeast corner of Oak Street and Hawes Road, and two large-diameter storm drains; one along Oak Street and the other along Hawes Road that discharge into the basin. The basin drains through a 48-inch basin outlet pipe that discharges to the existing Hawes Road channel. The Hawes Road channel is maintained by the Thunder Mountain Homeowners Association.

Oak Street Basin and Storm Drain Maintenance Components

The following section describes the components of the storm drain system that need to be maintained in order to provide a system that will perform as it was designed. Refer to the attached exhibit for the location of all the storm drain system components.

1) **Oak Street Basin**

   The Oak Street Basin was designed to attenuate the 100-year peak discharge from the Oak Street and Hawes Road storm drains and discharge it to the Hawes Road channel through the basin outlet pipe. The basin was sized to provided one foot of freeboard from the 100-year water surface elevation to the overflow spillway. This freeboard was found to be sufficient to accommodate the anticipated sediment load from the two storm drains and the inflow spillway. The basin can be accessed from either Hawes Road or Oak Street via a 14-foot wide decomposed granite maintenance road.

2) **Drop Inlet Structures**

   There are 6 drop inlet structures along the Oak Street and Hawes Road storm drains that were placed at wash inflow locations where large debris and sediment

loads are anticipated. In order to design for these sediment and debris loads, the size of the drop inlets was calculated based on a 50% clogging factor. The inlets are also equipped with a sloping trash rack that is intended to minimize clogging by allowing floating debris to ride up the rack. The drop inlet structures can be accessed via a gate that is installed as part of the trash rack.

3) **Grated Catch Basins**

There are 3 grated catch basins that drain relatively small drainage areas. The catch basins were sized with a 50% clogging factor, which is in accordance with the FCD Drainage Design Manual. To further reduce the potential of clogging, the grates are not installed flush with the surrounding concrete apron, but instead are raised 4-inches to allow small debris and sediment to pass under the grate and enter the storm drain system. All connector pipes were upsized by one standard size to account for potential clogging of the pipe by debris and/or sediment entering the catch basin.

4) **Curb-Opening Catch Basins**

There are a total of 6 curb opening catch basins along Oak Street. The catch basins were sized with a curb opening length clogging factor of 25%, which is in accordance to the FCD Drainage Design Manual. As with the grated inlets, the connector pipes were upsized by one size to account for potential clogging of the pipe by debris and/or sediment entering the catch basin.

5) **Oak Street Storm Drain Drop Structures**

Due to the relatively steep gradient of Oak Street, the storm drain had to be designed with drop structures to limit the pipe flow velocities to FCD standards. The 8 drop structures limit the slope of the storm drain by providing a 4-foot vertical drop within the structure. The drop structures can be accessed through a 30-inch manhole cover.

6) **Oak Street Storm Drain**

The Oak Street storm drain was designed to convey the 100-year peak discharge as well as the anticipated sediment load from the intercepted washes. The storm drain consists of two major segments (72-inch and 84-inch diameter pipes) that extend from the Oak Street Basin to the wash just east of 87th Street, for a total length of 1,800 feet. The Oak Street storm drain can be accessed via the drop structures.

7) **Hawes Road Storm Drain**

The Hawes Road storm drain was designed to convey the 100-year peak discharge as well as the anticipated sediment load from the three inflow locations. The storm drain consists of two segments (48-inch and 54-inch diameter pipes) that extend from the Oak Street Basin to the existing Granite Mountain channel, for a total length of 1,000 feet. The Hawes Road storm drain can be accessed via several 5-foot diameter manholes.

8) **Basin Outlet Pipe**

The Basin outlet pipe is the primary outlet for the Oak Street Basin. The outlet pipe was designed based on the downstream capacity of the existing Hawes Road channel and the McDowell Road storm drain. Due to a length of only 175 feet, there are no manholes provided. Instead access to the outlet pipe can be attained from the basin outlet structure or the pipe's outlet headwall in the Hawes Road channel.

9) **Basin Inflow Spillway**

The basin inflow spillway consists of a broad crested concrete weir that discharges flow over the maintenance access road into the Oak Street Basin. Access to the spillway can be obtained from the maintenance road, either from Oak Street or Hawes Road.

10) **Storm Drain Outlet Structures**

The Oak Street storm drain outlet structure is located in the southeast corner of the basin and the Hawes Road storm drain outlet structure is located in the northwest corner of the basin. Both outlet structures are equipped with breakaway access barriers that prevent unauthorized access to the storm drain. Access to the storm drain outlet structures can be obtained from the basin.

11) **Basin Outlet Structure**

The basin outlet structure consists of a specially designed primary outlet headwall and secondary trench drain. The intent of the design is to minimize the impact of a potentially clogged primary outlet trash rack by allowing flows to enter the basin outlet pipe through the trench drain. Access to the basin outlet structure can be gained from the basin.

12) **Pipe Outlet at Hawes Road Channel**

The pipe outlet at the Hawes Road channel consists of a headwall that was modified to fit the outlet pipe into channel. The outlet is equipped with a breakaway access barrier that is intended to prevent unauthorized access to the outlet pipe. Access to the pipe outlet can be obtained from Oak Street.

Oak Street Basin and Storm Drain Maintenance Plan

Continued maintenance of the storm drain system is a vital part of its overall effectiveness as a flood mitigation project. All the components of the system, such as the inlet and outlets need to be periodically tended to, as to keep them functional for future storm events. The following procedures outline the steps that need to be performed in order to keep all the various system components functioning at their design capacities.

1) **Oak Street Basin**

The continued performance of the Oak Street Basin is vital to the performance of the overall storm drain system. In order to account for sediment accumulation in the basin, a freeboard volume of 3.7 ac-ft (6000 yd^3) was provided. The total sediment inflow from a 100-year storm is estimated to be 800 yd^3, therefore the basin does not have to be cleaned up after every storm event. The sediment will

accumulate in "delta-like" formations around the storm drain outlet structures. Sediment removal should occur once the total accumulated sediment volume is estimated to be roughly 4000 yd^3, at which time all of the sediment should be removed and the basin bottom restored to the original design plans (or roughly to the storm drain outlet structures invert elevations).

To maintain the aesthetic character that was accepted by the surrounding community, the following guidelines should be observed as part of the maintenance plan:

- Placement of erosion protection (rip-rap), or decomposed granite, shall match the existing rip-rap or decomposed granite in color, shape and size.
- Repair of existing or construction of new concrete structures shall match existing in texture and color. The color matching shall be achieved using natural grey concrete with an applied water-based reactive stain, such as "Natina Concrete" or equal.
- Repair of existing or construction of new steel structures that are visible, such as railings or barriers, shall not be painted, but shall be treated with a water-based reactive stain, such as "Natina Steel" or equal.

2) Drop Inlet Structures

Remove all debris (trash, branches, leafs, etc…) from the trash racks and from inside the drop inlet structure. It is expected that sediment will enter the inlets during almost all storm events, with most of it being conveyed to the basin, but some residual sediment is anticipated. So, some sediment inside of the drop structures can be allowed to accumulate. Start removal of sediment (from drop structure and connector pipe) when the depth of sediment inside of the connector pipe reaches approximately $1/5^{th}$ of the pipe's diameter. For example, this means that for a 24-inch connector pipe sediment should be removed if it has accumulated to a depth of 5-inches or more. Greater accumulated depths are acceptable in larger pipes ($1/5^{th}$ of the pipe's diameter) but in no case should sediment be allowed to accumulate to 12-inches (regardless of connector pipe size). Make sure that all Occupational Safety and Health Administration (OSHA) safety procedures are followed.

3) Grated Catch Basins

Remove all debris from the catch basin grate and from inside the catch basin. Remove all of the sediment that has accumulated at the bottom of the catch basin and in the connector pipes. Make sure that all OSHA safety procedures are followed.

4) Curb-Opening Catch Basins

Remove all debris that has accumulated at the curb opening and inside of the catch basin. Remove all of the sediment that has accumulated inside of the catch basin and in the connector pipes. Make sure that all OSHA safety procedures are followed.

5) **Oak Street Storm Drain Drop Structures**

Similar to the drop inlet structures, any accumulated debris inside of the drop structures is to be removed. Some sediment in the drop structures can be allowed to accumulate, however the sediment should be completely removed if it has accumulated to a depth of 12-inches or more in the downstream storm drain pipe. Make sure to follow all OSHA safety procedures.

6) **Oak Street Storm Drain**

Remove all debris that has accumulated inside of the Oak Street storm drain. It however is not necessary to clean out all of the sediment. As discussed under the maintenance plan for the drop structures, an accumulation of sediment to a depth of 12-inches is allowed. If the sediment within the storm drain exceeds 12-inches, then it should be removed. Make sure to follow all OSHA safety procedures.

7) **Hawes Road Storm Drain**

Similar to the maintenance plan for the Oak Street storm drain; remove all debris that have accumulated inside of the storm drain. Some sediment accumulation is acceptable; only remove sediment in the storm drain pipes if it has accumulated to a depth greater than 12-inches. Make sure to follow all OSHA safety procedures.

8) **Basin Outlet Pipe**

Remove all debris and sediment that has accumulated inside of the basin outlet pipe. Make sure to follow all OSHA safety procedures.

9) **Basin Inflow Spillway**

Clear the spillway of any sediment deposits and replace rip-rap as necessary. The new rip-rap is to match the existing rip-rap in size and color.

10) **Storm Drain Outlet Structures**

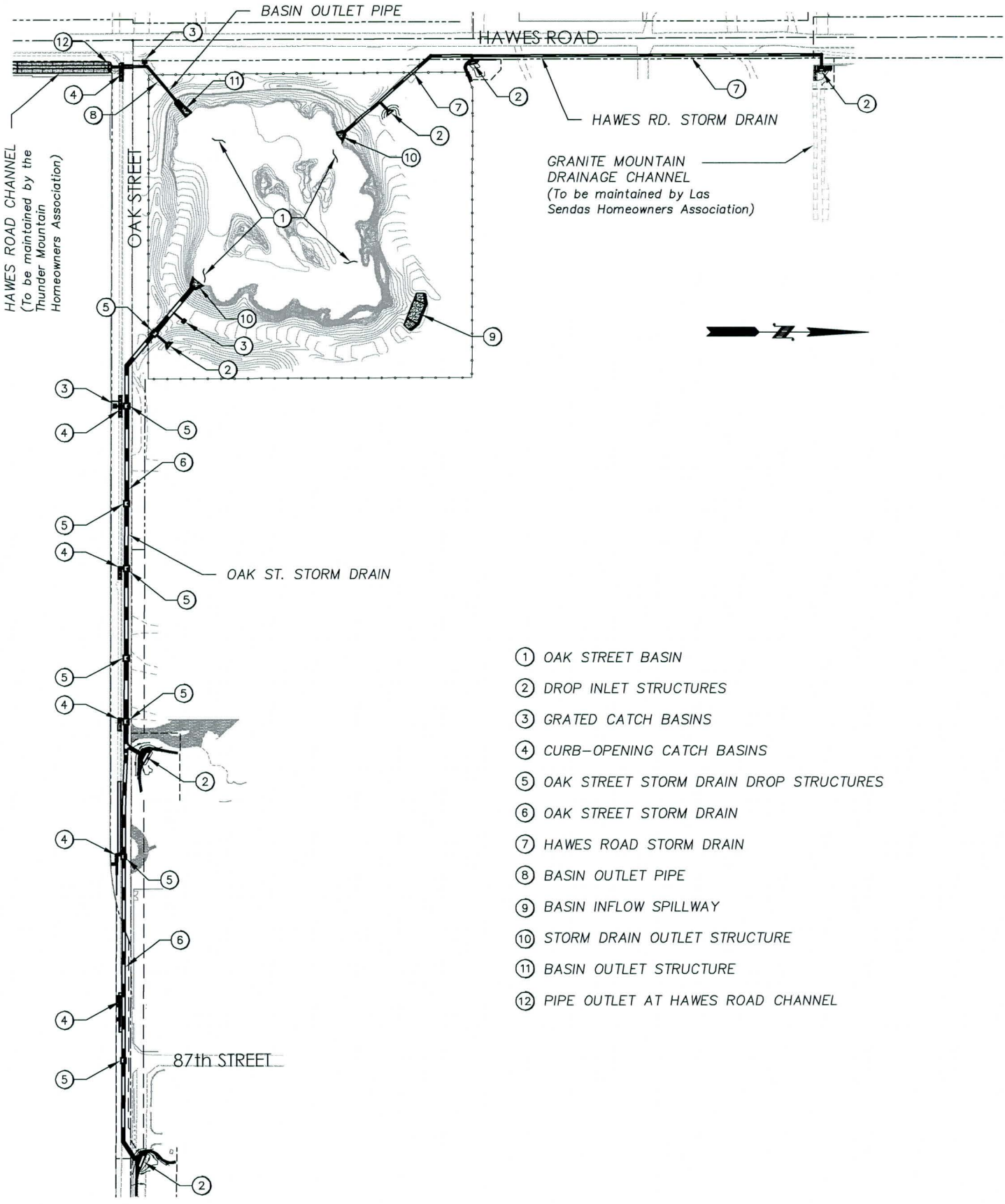
Remove all debris collected at the access barriers. Remove all accumulated sediment from the outlet structures concrete apron if it has accumulated to a depth of 12-inches or more. Note that the "delta-like" sediment formations in the basin just downstream of these outlet structures should be addressed in accordance with the criteria described under "**1) Oak Street Basin.**" Replace the surrounding rip-rap as necessary (match size and color of existing rip-rap).

11) **Basin Outlet Structure**

Remove all debris that has collected at the trash rack and at the trench drain grate. Remove all debris and sediment that have entered through the trash rack and into the basin outlet pipe. Replace the surrounding rip-rap as necessary (match size and color of existing rip-rap).

12) **Pipe Outlet at Hawes Road Channel**

Remove all debris that may have collected on the pipe outlet access barrier. Remove any sediment that might have accumulated at the pipe outlet. Note that the maintenance of the Hawes Road channel is the responsibility of the Thunder Mountain subdivision HOA.



1. OAK STREET BASIN
2. DROP INLET STRUCTURES
3. GRATED CATCH BASINS
4. CURB–OPENING CATCH BASINS
5. OAK STREET STORM DRAIN DROP STRUCTURES
6. OAK STREET STORM DRAIN
7. HAWES ROAD STORM DRAIN
8. BASIN OUTLET PIPE
9. BASIN INFLOW SPILLWAY
10. STORM DRAIN OUTLET STRUCTURE
11. BASIN OUTLET STRUCTURE
12. PIPE OUTLET AT HAWES ROAD CHANNEL

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Civil Engineering
Landscape Architecture

Project Title :

Oak Street Basin and Storm Drain Design

Flood Control District of Maricopa County

Contract FCD - 2009C021

Sheet Title :

MAINTENANCE PLAN

Submittal :

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1 of 1