DESIGN REPORT

Old Cross Cut Canal Drainage Improvements

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

May 24, 1991

Results not correct
★ see TWH memo 7-1-91
with corrected peak flows
available at Watershed Branch
AMM

Greiner

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1.0 INTRODUCTION

1.1 PURPOSE AND GENERAL DESCRIPTION

The purpose of this project is to prepare plans and specifications for construction of improvements to the existing Old Cross Cut Canal (OCCC), from just south of the Arizona Canal (north of Indian School Road) to McDowell Road.

The improvements consist of replacement of the existing open channel with a covered section. The covered section shall be designed to accept existing inflows without causing diversion of drainage, and to convey flows from a 25-year storm using hydrology results from a study prepared by the U.S. Army Corps of Engineers (USCOE). The design shall accommodate 3,000 cfs north of Indian School Road and allow for the connection of a future drainage structure to convey 2,000 cfs under the Arizona Canal from the watershed north of the canal. The design shall accommodate 4,100 cfs at the connection to the existing culverts at McDowell Road. The new section is to be designed to stay within existing Salt River Project (Bureau of Reclamation) right-of-way. The project also includes preparation of plans and specifications for relocations of conflicting water and sewer lines, traffic control provisions and coordination of the relocation of conflicting utilities (electric, teleco, gas, CATV).

Refer to Contract FCD 90-23, Old Cross Cut Canal Design, for a complete description of the Scope of Work and project tasks.

Some project considerations were modified during the preparation of this report, these include:

- Modified hydrology simulation
- Modified design flows

1.2 COORDINATION

Following are the agencies/utilities, with contact name and phone number, involved in this project:

1. City of Phoenix

   Ray Acuna, P.E. at 262-6781
   c/o Improvement District
   125 E. Washington Street
   Phoenix, AZ 85004
   Other Numbers: 262-4052, 262-4057 and 495-5326

   Ray Acuna is contact for: Storm Drains
   Traffic Control
   Major Streets
   Parks Department

Roy Alvarado is Utility Group contact for this project at 262-4970.

Mark Bornhoeft is the Project Manager for the Linear Park Design contract at 262-6653.

2.	City of Phoenix Water and Wastewater	Jerry Arakaki, P.E. at 261-8229 455 N. 5th Street Phoenix, AZ 85004
3.	Electric (SRP)	Chuck Hughes at 236-2090 Senior Power Consultant Distribution Line Design/Engineering P.O. Box 52025 Phoenix, AZ 85072-2025
4.	Electric (APS)	Steve Goodman at 371-6965 P.O. Box 53999, Station 3278 Phoenix, AZ 85036-3999
5.	Southwest Gas	Glenn Festin or Howard Warren at 730-3645 Distribution Engineer 5705 S. Kyrene Road Tempe, AZ 85283
6.	U.S. West (Teleco)	Curt Sayer, Mark Ables or Steve Cockrell at 831-4777 Manager, Public Projects Liaison/Southeast 6350 S. Maple Avenue Tempe, AZ 85283
7.	CATV (Dimension)	John Barnett at 866-0072, Ext. 361 Construction Supervisor Dimension Cable Services 17602 N. Black Canyon Freeway Phoenix, AZ 85023
8.	CATV (U.S. Sprint)	Skip Hughes 401 W. Harrison Street Phoenix, AZ 85004
9.	CATV (Insight Cable)	Roland Faucett Insight Cable Communications 21200 N. Black Canyon Highway Phoenix, AZ 85027

10.	SRP (Operations)	Tim Phillips at 236-2956 Operational Support P.O. Box 52025 Phoenix, AZ 85072-2025	
11.	ADOT/UHS	Steve Jimenez at 255-7545 205 S. 17th Avenue, Room 216E Phoenix, AZ 85007	
12.	HNTB	Tom Klimek at 528-4300 Project Manager Liner Park Design Consultant	
13.	Greiner	Shi-En Shiau, Project Manager Randall Beck, Assistant Project Manager Vince Gibbons, Project Engineer Ron Ferguson, Utilities Rob Pecha, Survey Coordinator Jon King, Hydrology Cheng-Chang Huang, Flooding Impact Jim Barrera, Hydraulics	All at 275-5400

1.3 REFERENCE DATA

1.3.1 Previous Studies

Old Cross Cut Canal, Phoenix, Arizona, Hydrology for Feasibility Studies for Flood Control and Allied Purposes, U.S. Army Corps of Engineers, Los Angeles District, June 1987.

Old Cross Cut Canal Study Post F3 Conference, Full Lafayette Alternative, U.S. Army Corps of Engineers, Los Angeles District, November 1988.

Hohokam Parkway Master Plan Report, Roadway Concept Design Study for the City of Phoenix, Greiner, Inc., February 1989.

Draft Feasibility Report, Old Cross Cut Canal, Phoenix, Arizona, U.S. Army Corps of Engineers, Los Angeles District, April 1989.

McDowell Road/Old Cross Cut Canal, Cannon & Associates, Inc., February 20, 1990.

1.3.2 Maps and As-Built Plans

City of Phoenix Engineering Department, quarter section maps showing right-of-way, property and addresses.

City of Phoenix Water and Wastewater Department, quarter section maps for wastewater.

City of Phoenix Water and Wastewater Department, quarter section maps for water.

City of Phoenix Engineering Department, 400-scale storm drain maps.

City of Phoenix, topographic maps, 1" = 100'.

City of Phoenix, zoning maps.

USGS quadrangle maps.

City of Phoenix, storm and sewer plans, Oak Street, Old Cross Cut Canal To 52nd Street.

City of Phoenix, storm sewer plans, 52nd Street, McDowell Road to Thomas Road.

City of Phoenix, storm drain plans, McDowell Road, 40th Street to 52nd Street.

City of Phoenix, storm drain plans, Virginia Avenue, Line "A."

City of Phoenix, storm drain plans, Thomas Road, 44th Street to 56th Street.

Storm sewer plans, Earll Drive, 48th Street (Old Cross Cut Canal) to 56th Street.

Storm sewer plans, miscellaneous improvements of Old Cross Cut Canal, Van Buren Street to Oak Street.

Storm drain plans, Old Cross Cut Canal, Oak Street to Osborn Road.

Storm drain plans, Old Cross Cut Canal at Holly Street.

Old Cross Cut Canal bridge at Thomas Road.

48th Street bridge at Arizona Canal.

Paving plans for Joe's Place, Old Cross Cut Canal at Coronado Road.

Paving plans, Thomas Road, 44th Street to 56th Street.

Paving plans, Arcadia Vista at 48th Street.

Paving plans, Indian School Road, 32nd Street to 48th Street.

City of Phoenix Wastewater Department, 48th Street, McDowell Road to Holly Street.

City of Phoenix Wastewater Department, 48th Street, Roosevelt Street to Thomas Road.

City of Phoenix Water Department, 48th Street, McDowell Road to Thomas Road.

City of Phoenix Public Works Department, sanitary sewer, 48th Street at Indianola Avenue, 48th Street at Fairmount Avenue, 48th Street at Picadilly Road, and 48th Street at Clarendon Avenue and Weldon Avenue.

City of Phoenix Public Works Department, sanitary sewer, 48th Street, Osborn Road to Indian School Road.

City of Phoenix Public Works Department, sanitary sewer, 48th Street at Osborn Road and Clarendon Avenue.

City of Phoenix Public Works Department, sanitary sewer, 48th Street, Osborn Road north to lateral at 688' north of Osborn Road.

City of Phoenix Engineering Department, sanitary sewer in 48th Street from McDowell Road to Virginia Avenue and east to 50th Street down Virginia Avenue.

City of Phoenix, water improvements south feeder main, plan and profiles from Hubbell Avenue along 48th Street to Virginia Avenue.

Verde water system, plan and profile of 48th Street at Thomas Road.

Orange Valley Estates, water plan as-builts at 48th Street and Osborn Road.

City of Phoenix Division of Water and Sewers, 48th Street from Oak Street to Vernon Avenue.

Motorola, Inc., water line in McDowell Road at 48th Street.

City of Phoenix Public Works Department, water line plan and profile in Indian School Road at 48th Street.

Profile of Oak Street crossing of the Old Cross Cut Canal's 8" water line.

Profile of Windsor Avenue, crossing of the Old Cross Cut Canal's 8" water line.

City of Phoenix, 16" water line from 44th Street and Clarendon Avenue to 52nd Street and Thomas Road.

City of Phoenix Street Transportation Department, McDowell Road/Old Cross Cut Canal.

Arizona Department of Transportation Highway Division, relocation of Old Cross Cut Canal.

Mountain Bell, buried facility and overhead distribution of quarter section maps.

Mountain States Telephone Company, telephone buried cable from Virginia Avenue to Osborn Road.

U.S. West Communications, 48th Street at Indian School Road (buried cable).

Mountain States Telephone Company, 48th Street at Indian School Road (buried cable).

Mountain States Telephone Company, McDowell Road at 48th Street (buried cable).

Mountain States Telephone Company, Virginia Avenue north to Osborn Road (buried cable).

Mountain States Telephone Company, 48th Street at Indian School Road (buried cable).

U.S. West Communications, 48th Street at Indian School Road (buried cable).

Salt River Valley Water Users' Association, water service center area plan for irrigation drainage tiles, field construction report, specifications and construction cost estimates.

Salt River Project Agricultural Improvement and Power District, recorded easement legal descriptions, quit claim deeds for drainage right-of-way, all pertaining to irrigation, drainage, wells to pumping stations.

Salt River Valley Water Users' Association, Arizona Canal (Old Cross Cut Canal) plan and profile as-builts.

Salt River Valley Water Users' Association, waste ditch plan and profiles.

Salt River Power District Underground Division, underground electrical distribution.

Salt River Project Agricultural Improvement Power District, overhead electric, 1/4 section distribution maps.

Southwest Gas, quarter section as-built maps.

Southwest Gas, 600-scale gas distribution maps.

Division Cable Service, cable T.V. quarter section distribution maps.

1.4 SITE CONSIDERATIONS

The OCCC was constructed in the late 1880's to transfer water between the Arizona Canal and the Grand Canal. The canal is located adjacent to 48th Street between the Arizona Canal and McDowell Road. At McDowell Road, it transitions to the west and parallels 46th Street to the Grand Canal outside of the scope of this study.

The Arizona Canal is a water supply canal which carries water between Granite Reef Dam and Skunk Creek. The flow varies between 700 cfs and 1,100 cfs. During periods of high runoff, however, the flow in the canal overtops the southern bank causing flooding south of the canal. In order to control flood waters, the diversion gates at 48th Street were installed to allow excess flow into the OCCC. The main purpose of the OCCC as part of Salt River Project's canal system has been to transfer water from the Arizona Canal to the Salt River. In the process, it also conveys local flow from adjacent watersheds downstream to the Salt River.

The existing channel of the OCCC varies from a rough channel with localized erosion and a 30' top width at the Arizona Canal, to a fairly smooth channel with a 35' top width and a 15' depth at McDowell Road.

The principle land use along the canal is residential with commercial pockets located at Indian School Road, Thomas Road and McDowell Road. There is a small recreational property along side the west bank of the canal south of Indian School Road owned by the Arizona Republic/Phoenix Gazette. St. Francis Cemetery, located along 48th Street, is east of the canal around the Oak Street crossing.

2.0 HYDROLOGY

2.1 INTRODUCTION

This study is a refinement of a previous study done by the U.S. Army Corps of Engineers titled (USCOE) "Old Cross Cut Canal Study Post F3 Conference Full Lafayette Alternative," November 1988, to evaluate flow in the OCCC. It was done in conjunction with an analyses by the Flood Control District of Maricopa County of areas outside the OCCC watershed which impacted conveyance requirements of the proposed Cross Cut Canal improvements. In the 1988 USCOE study, concentrations were calculated at the intersections of the OCCC with Thomas and McDowell Roads for various return periods. The USCOE study included flow contributions from the Arizona Canal at the headwaters of the OCCC, as well as flow from the proposed Lafayette storm drain which would be routed through the OCCC.

The purpose of this section is to determine the concentration point locations and the magnitude of all contributing flows along the OCCC between the Arizona Canal and McDowell Road for sizing of the mainline conduit, as well as the inlet structures. To do this a HEC-1 model was developed for the area immediately adjacent to the canal to determine peak storm runoff and points of concentration. This HEC-1 model was developed in accordance with Flood Control District of Maricopa County standards as shown in the Hydrologic Design Manual for Maricopa County (HDMMC). All computer models were run using HEC-1 on a PC.

The modeling parameters for the contributing areas of the Lafayette storm drain and the Arizona Canal were initially taken from the 1988 USCOE study. The Flood Control District of Maricopa County then refined the USCOE model using hydrologic procedures outlined in the HDMMC. Ultimate modeling of the OCCC in this study is based upon the Flood Control District's 25-year analysis of the proposed Lafayette storm drain.

2.2 METHODOLOGY

The OCCC hydrologic study area consists of some 2.8 square miles of mostly developed urban watershed located east of the project corridor. This tributary area is bounded on the north by the Arizona Canal and on the south by McDowell Road and the Papago Army Air Station. Land use is mostly residential with some commercial and industrial properties.

Drainage areas causing direct runoff into the OCCC which make up the total area of original study for this report were delineated on 100 scale City of Phoenix Topography or, where not available, on United States Geologic Survey Quadrangle Maps. Fifteen subbasin areas were planimetered and flow path lengths taken from these maps. These subbasins can be seen in Figure 2.2-1. The Design Rainfall Depth was found from the Isopluvial maps in the HDMMC for the 25-year, 6-hour storm event.

2.0 HYDROLOGY

The Clark Method was chosen as the unit hydrograph procedure because of its higher degree of accuracy in dealing with the relatively small size of the contributing subbasins. Design storm distribution and time area relations for the Clark Unit Hydrograph were both determined using the Flood Control District's HEC-1 development program, MCUPH1.exe. Individual basin Time of Concentrations (T_c) and Storage Coefficients (R) were developed using MCUPH1.exe and are shown in Table 2.1. The T_c and R values listed in Table 2.1 are for analysis of flow in the OCCC and were used to develop the peak flows listed in Table 2.4. Separate T_c and R values were used to develop the peak flow used to size the proposed stub-outs, as listed in Table 2.6. These T_c and R values reflect the process of centering the design storm over each individual subbasin in order to maximize the peak flows.

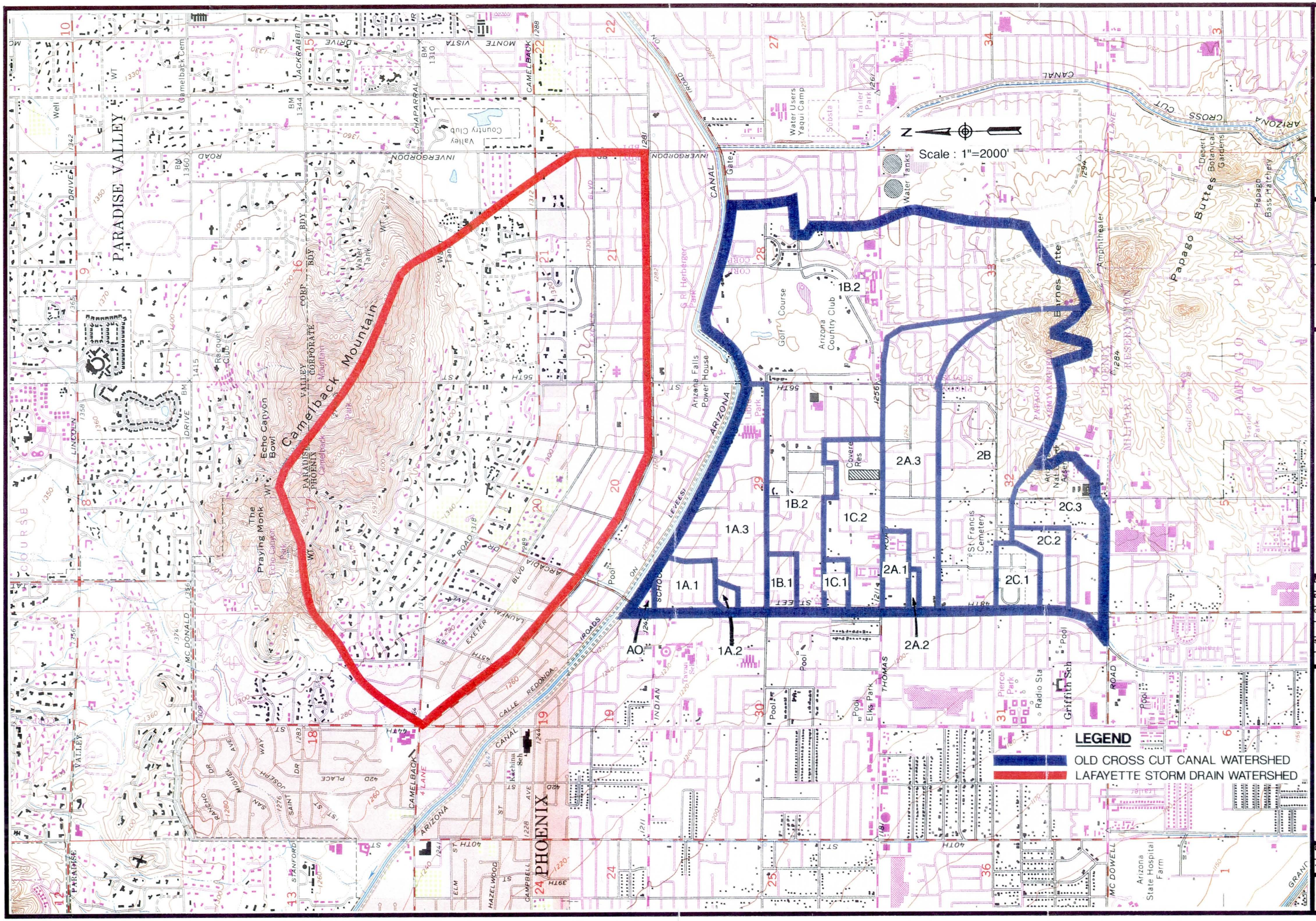
Rainfall loss rates were determined using the Green-Ampt procedure. XKSAT, PSIF and DTHETA values were found by Soil Texture Classification of the various subbasins as shown in the SCS "SOIL SURVEY FOR EASTERN MARICOPA AND NORTHERN PINAL COUNTIES AREA, ARIZONA." Soil textures varied from loams and sandy loams in the flat valley region to gravelly loam at the foot of Barnes Butte to rough broken rock at the summit of Barnes Butte.

Impermeability values were taken from City of Phoenix Zoning Maps (see Figure 2.2-2) and corresponding City of Phoenix Land Use Impermeability Rate Tables (Table 2.2). An effort was made to weigh the total subbasin impermeability by multiplying the individual percent of total area taken up by the various zones within the subbasin by their corresponding impermeabilities and summing the result. The weighted impermeabilities obtained varied from 15 percent in lightly developed areas (such as the St. Francis Cemetery) to 85 percent in mainly commercial subbasins. Impermeabilities used are listed in Table 2.3.

Routing was initially modeled using the kinematic wave method but was changed to the normal depth storage method at the request of the Flood Control District to avoid the stability problems encountered in using the kinematic method. The normal depth storage method utilizes Manning's equation to determine the conveyance capacity of the channel.

Retention of runoff was not included as part of this study. Difficulty in assessing individual compliance with current standards, as well as uncertainty of future standards and land use, have led us to assume no on-site retention.

The majority of flow adjacent to the OCCC will be overland flow. The areas included in this report are subject to flooding along the slight slopes of the valley region and the steep slopes occurring around Barnes Butte. Most of the area included in this study has been developed. When the overland flow of runoff is interrupted by man-made obstructions such as roadways, embankments, housing developments etc., there is the direct possibility of diversion of the flow. Flow paths and drainage boundaries were, therefore, chosen with this in mind. Points of Concentration for the runoff at the OCCC were located at street crossing intersections.



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555 East River Rd., Suite 100/Tucson, Arizona 85704/602 887-1800

Design
Drawn
Check
Scale

OLD CROSS CUT CANAL AND PROPOSED LAFAYETTE STORM DRAIN WATERSHEDS

Date
Job No.

Figure 2.2-1

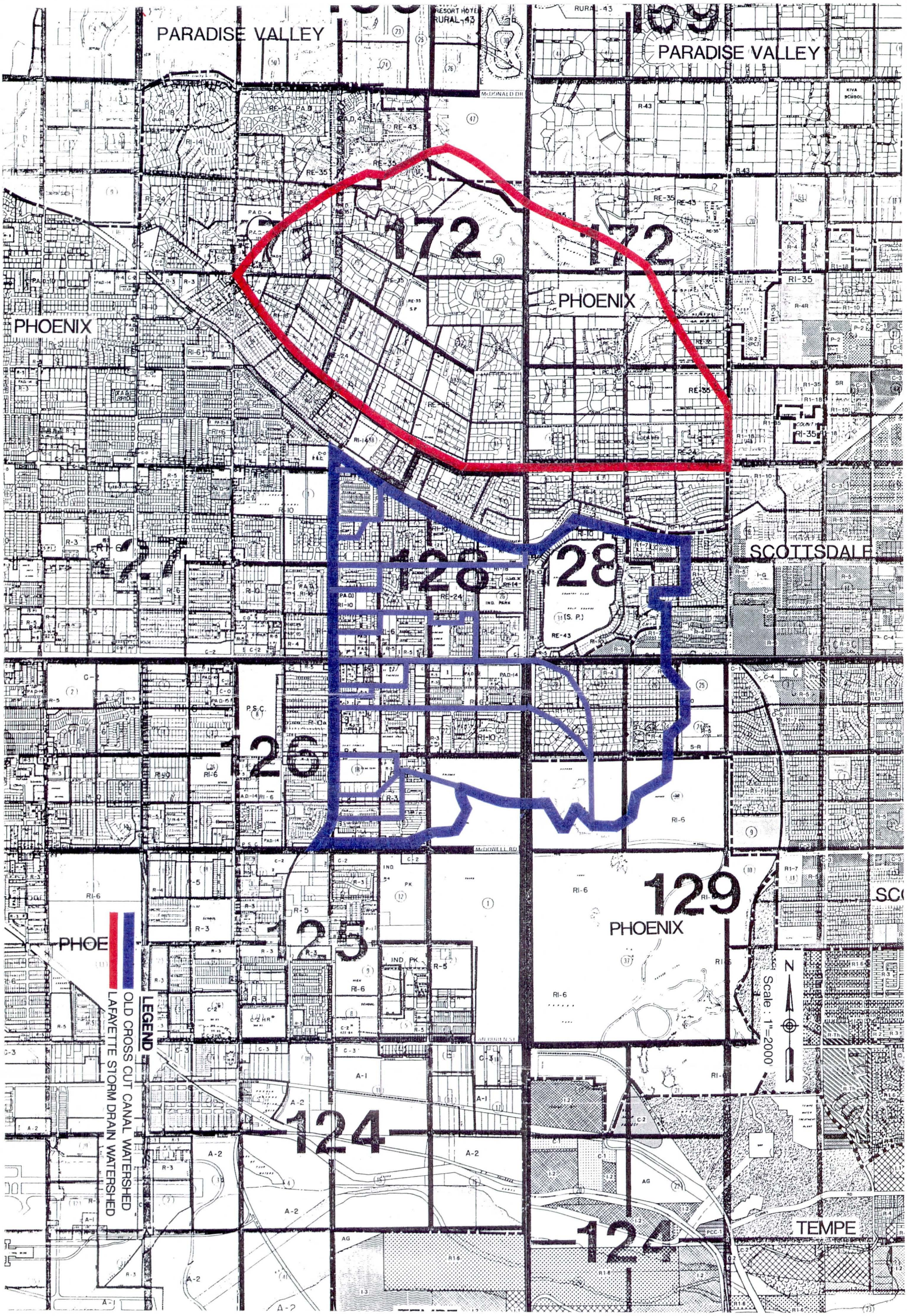

Figure 2.2-2	Job No.	Date	OLD CROSS CUT CANAL WATERSHED LAND USE MAP	Scale	Check	Drawn	Design	Greiner 7310 N. 16th Street, Suite 160/Phoenix, Arizona 85020/602 275-5400 555 East River Rd., Suite 100/Tucson, Arizona 85704/602 887-1800

2.2.1 Development of the HEC-1 Input

The HEC-1 Flood Hydrograph Package has many component options. This section documents the input components used for hydrologic modeling for this study. The skeleton HEC-1 models used in this study were "built" using the Flood Control District's MCUHP1.exe.

Job Initialization. The "IT" card was used to define the time interval of five minutes and the number of hydrograph ordinates to be computed. The "IN" card was used to define the time interval of 15 minutes for reading the "PC" card (cumulative precipitation time series). When the time series data is read from the "PC" card, values are computed internally using linear interpolation to match the tabulation interval on the "IT" card.

Precipitation Data. The synthetic storm used for input was the Maricopa County 6-hour rainfall distribution. The "PC" card was used to input this precipitation mass curve. The "PC" values for precipitation were developed from Distribution Pattern No. 2 of the HDMMC. The "PB" card was used to define the total storm and basin-average precipitation values in inches. The values used for this study were derived from the Maricopa County Isohyetal maps shown in Section 2 of the HDMMC. Precipitation values were areally reduced to realistically model a single storm over the entire OCCC watershed. Precipitation values for the Flood Control District study to obtain the 25-year outflow of the Lafayette storm drain were obtained using areal reduction over the combined OCCC and Lafayette storm drain watersheds.

Loss Rate Data. Loss rate data is based upon the Green and Ampt procedure located in the HDMMC. Rate loss constants were entered using the "LG" card.

Basin Data. The "BA" card was used for subarea runoff computation. The main component for this card is the drainage area in square miles. No flow recession was assumed for our models.

Base Flow. The "BF" card was used to model base flow from the Arizona Canal. A constant flow of 1,000 cfs was used.

Hydrograph Transformation. The "HC" card was used to calculate hydrograph combination.

Routing Data. The "RS, RC, RX and RY" cards were used to perform the normal depth storage routing method as explained earlier.

A copy of the final HEC-1 input file will be provided to the Flood Control District with this report.

Drainage Subbasin AO. Drainage Subbasin AO is bounded on the north and east by the Arizona Canal, on the west by the OCCC and on the south by Indian School Road. The area is zoned commercial and is comprised of approximately 0.013 square mile of mixed residential and commercial uses. The land is gently sloping to the southwest at an approximate 0.8 percent grade. Soil type is predominantly Class B and is made up of sandy loam at the surface, providing for moderately good permeability.

Drainage Subbasin 1A.1. Drainage Subbasin 1A.1 is bounded on the north by Indian School Road, on the west by the OCCC, on the east by 49th Street and on the south by Weldon Avenue. Land use is predominantly residential while total subbasin area is 0.062 square mile. The hydrologic soil group is predominantly Class B and is made up of loams and sandy loams. The land gently slopes to the southwest at approximately 0.7 percent.

Drainage Subbasin 1A.2. Drainage Subbasin 1A.2 is approximately 0.008 square mile and is uniformly homogenous in soil type and land use classification. Soil type is B and is made up of loams, while land use is zoned exclusively for residential land use. Subbasin 1A.2 slopes to the southwest at approximately 0.7 percent.

Drainage Subbasin 1A.3. Drainage Subbasin 1A.3 is bounded on the north by the Arizona Canal and Weldon Avenue, on the east by 56th Street, on the south by Osborn Road and on the west by the OCCC. The drainage area is approximately 0.227 square mile of mixed residential land use including a park and school at the east end of the basin bordering Osborn Road and 56th Street. Several different impermeabilities apply to this subbasin, but on weighted average, the impermeability was taken to be 23 percent. Soil type is Class B and is made up of loams, sandy loams and clayey loams which provide moderately good permeability. This area is completely developed and the natural terrain gently falls to the southwest at 0.6 percent. Drainage paths were taken to concentrate along Osborn Road, taking the flow to the OCCC at 48th Street.

Drainage Subbasin 1B.1. Drainage Subbasin 1B.1 is bounded on the north by Osborn Road on the south by Richardson, on the east by 50th Street and on the west by the OCCC. The area is approximately 0.030 square mile and is made up of soil group B and loamy soil at the surface. The subbasin slope is mild and falls at 0.4 percent to the southwest.

Drainage Subbasin 1B.2. Drainage Subbasin 1B.2 is bounded on the north by Osborn Road, Richardson and the Arizona Canal, on the east by the eastern boundary of the Arizona Country Club Golf Course and extends to Barnes Butte in the south. Subbasin 1B.2 is the most diverse subbasin in this study with various land zoning types, soil groups and landforms. Subbasin 1B.2 is made up of approximately 1.125 square miles of mixed residential, recreational, commercial and industrial land use. Soil types are both B and D, and range from loams near the OCCC on the west to broken rock on the southeast at Barnes Butte. A very large percentage of the drainage area is made up of the Arizona Country Club Golf Course. The subbasin slopes sharply near Barnes Butte to the north and northwest, and levels off to nearly 0.5 percent in the northern and northwestern region of the subbasin were it begins to slope to the west and south. Flows are assumed to accumulate initially as sheet Sp flow and then progress to street flow, eventually concentrating along Earll Drive at the OCCC.

Drainage Subbasin 1C.1. Drainage Subbasin 1C.1 is approximately 0.018 square mile and is bounded on the north by Earll Drive, on the east by 49th Street, on the south by Pinchot Avenue and on the west by the OCCC. Hydrologic soil type is Class B, made up of mainly loams. Land is zoned residential and gently slopes to the southwest at 0.3 percent.

Drainage Subbasin 1C.2. Drainage Subbasin 1C.2 is bounded on the north by Pinchot Avenue and Earll Drive, on the east by 54th Street, on the south by Thomas Road and on the west by the OCCC. Its area is approximately 0.150 square mile and is made up of Soil Class B of loam and clayey loam texture. Land use is zoned as a mix of various residential uses. The average subbasin slope is 0.4 percent.

Drainage Subbasin 2A.1. Drainage Subbasin 2A.1 is approximately 0.047 square mile and is bounded on the north by Thomas Road, on the east by 51st Street, on the south by Windsor Avenue and on the west by the OCCC. Land use is mixed residential and soil type is Class B with a loamy texture. The subbasin slopes to the southwest at approximately 0.3 percent.

Drainage Subbasin 2A.2. Drainage Subbasin 2A.2 is approximately 0.005 square mile and is bounded on the north by Windsor Avenue and on the west by the OCCC. The land is zoned residential and is soil type Class B with a loamy texture. The subbasin slopes to the southwest at approximately 0.4 percent.

Drainage Subbasin 2A.3. Drainage Subbasin 2A.3 is bounded on the west by the OCCC, and extends to the east and south to the bench below Barnes Butte. Area of the subbasin is 0.276 square mile and soil type varies from loam and clayey loam at the gently sloping valley floor to broken rock at the approaches to Barnes Butte. Average subbasin slope is approximately 1.2 percent.

Drainage Subbasin 2B. Drainage Subbasin 2B is approximately 0.505 square mile and is located just to the south of Subbasin 2A.3. Land use is mostly residential. Orange Dale school and St. Francis Cemetery are located near the western border of the subbasin on soils with high permeability, while at the southeastern tip of the subbasin is Papago Army Air Station with a rocky impermeable soil condition. Soil type varies fairly uniformly with the terrain. Starting with broken rock at Barnes Butte, it changes to clayey and gravelly loam as it approaches the western edge of the subbasin at OCCC. Average hydraulic slope used for modeling is approximately 4.0 percent.

Drainage Subbasin 2C.1. Drainage Subbasin 2C.1 is made up almost entirely of St. Francis Cemetery and is, therefore, modeled with a low percentage of impermeability. Subbasin area is approximately 0.053 square mile with loam and clayey loam soil. Slopes are gentle and uniform at about 0.7 percent to the southwest. Subbasin 2C.1 is bounded on the north by Oak, on the south by Holly and on the west by the OCCC.

Drainage Subbasin 2C.2. Drainage Subbasin 2C.2 is approximately 0.057 square mile and is made up of residential land use. Soil type is Class B with loam and clayey loam texture. The subbasin slope is 0.7 percent to the southwest and is nearly uniform. The subbasin boundaries are Holly Street to the north, Granada Road to the south and the OCCC to the west.

Drainage Subbasin 2C.3. Drainage Subbasin 2C.3 is approximately 0.175 square mile of mixed residential and industrial land uses. The eastern extent of the subbasin reaches to the Papago Army Air Station. Hydrologic soil types are B and D, with soils ranging from clayey loams to broken rock at the eastern boundary of the subbasin. Slopes vary along the subbasin from relatively steep slopes at the eastern boundary to fairly gentle slopes at the western boundary at the OCCC. The slope used to generate our hydrologic model was approximately 3.3 percent.

A summary of subbasin drainage characteristics can be found in Table 2.1.

2.3 ALTERNATIVE MODELS

In the course of this study various models of different alternatives were developed. The alternatives developed for this study can be broken down into two categories--Inflow Condition and Return Period of Storm.

Three return periods were initially studied for this report using USCOE hydrograph input for the Lafayette storm drain. These return periods include the 25-, 50- and 100-year storms. Table 2.5 lists the peak flows for these three return periods. Each case was studied using Maricopa County Isopluvial Maps to determine the six-hour storm precipitation for the areas adjacent to the OCCC. Separate Times of Concentration and Storage Constant values were developed for each of the 15 subbasins for each return period to more accurately model the flow conditions of the subbasins. The 25-year flood was the only flood studied using a Flood Control District-developed hydrograph for the Lafayette storm drain.

The total required conveyance of the OCCC will depend heavily upon the inflow conditions at its headworks at the Arizona Canal. There are plans to develop a storm drain system to the north of the Arizona Canal which would be connected through a siphon to the OCCC at the point of the Arizona Canal diversion gates to the OCCC. The flow generated by the proposed Lafayette storm drain combined with the existing requirement that the OCCC convey 1,000 cfs of overflow from the Arizona Canal diversion, will form a large percentage of the needed conveyance in the proposed OCCC.

Several options were considered in modeling the inflow condition. These include:

2.3.1 Alternative 1

Model the Arizona Canal diversion alone as an inflow hydrograph as was done in part of the USCOE study of 1988. We retrieved the generated hydrograph of the Corps study and incorporated it in our model. Without incorporating the Lafayette project inflow, total runoff is significantly reduced, causing a corresponding reduction in required conveyance of the OCCC. Because development of the Lafayette Project is probable, this alternative will not accurately predict required conveyance in the ultimate condition of the OCCC.

2.3.2 Alternative 2

Model the Lafayette and Arizona Canal diversion as combined inflow hydrographs. This option does not accurately model the inflow from the Arizona Canal as a constant flow, as it would occur during a flood event.

2.3.3 Alternative 3

Model the Lafayette storm drain and Arizona Canal as a constant base flow as mentioned in the initial Scope of Services. A constant flow of 3,000 cfs was assumed to flow into the headworks of the OCCC. The constant flow model of the Lafayette alternative is inaccurate in modeling inflow from the proposed storm drain which will not be a constant flow.

2.3.4 Alternative 4

Model the Lafayette storm drain inflow as a hydrograph and the Arizona Canal diversion as a constant base flow. It was determined in speaking with the County that this option provided for the best hydrologic model for the OCCC study. The hydrograph of flow generated by the Lafayette storm drain as determined in the Flood Control District study was combined with a 1,000 cfs base flow and incorporated in our model as an inflow hydrograph.

2.4 MAIN LINES DESIGN FLOW

The initial OCCC design flow values of 3,000 cfs north of Indian School Road and 4,100 cfs at McDowell Road mentioned in the initial Scope of Work were taken from Table 3 of the USCOE 1988 study. These flow values were given as the resulting runoff of a 50-year flood at the Arizona Canal and McDowell Road. This has since been superseded by the Flood Control District study which specified a maximum of 1,600 cfs from the Lafayette storm drain.

In Table 8 of a previous study by the USCOE in June 1987 entitled "OLD CROSS CUT PHOENIX, ARIZONA - HYDROLOGY FOR FEASIBILITY STUDIES FOR FLOOD

CONTROL AND ALLIED PURPOSES," 4,100 cfs was determined to be the output at McDowell Road for the 25-year flood. The apparent discrepancy with the later USCOE report can be clarified by looking at Section 6.03 on Page 30 of this earlier report. It states that for the Lafayette storm drain alternative, "To conform to the level of detail in this study, no routing was performed ..... Flow at convergences were directly summed instead of combined as hydrographs. Thus, these flows have a more conservative estimate of the necessary capacity of the Old Cross Cut Canal than other alternatives for the same frequency."

In a memorandum dated March 1, 1991, the Flood Control District listed design flows of the OCCC for the 25-year storm at major cross streets along the canal. These results were obtained by studying both the Lafayette and OCCC watersheds. Table 2.4 lists these flows alongside the flows generated by this study. The difference between the values obtained by the Flood Control District and the values obtained in this study can be accounted for in area distribution of rainfall. The Flood Control District's study distributed the single storm rainfall over the entire OCCC and Lafayette watersheds, while this study distributed the rainfall over the OCCC watershed alone. Centering the design storm over a smaller area results in less area reduction of rainfall and, consequently, slightly higher runoff.

The ability of the OCCC to carry 3,000 cfs at Indian School Road and 4,100 cfs at McDowell Road as stated in the Scope of Work is adequate to convey the runoff produced by the 25-year storm.

2.5 SIDE INFLOWS

Side inflows were determined at cross streets along the OCCC. Final side inflows were based upon the runoff generated by the 25-year storm. Table 2.6 shows design side inflows with preliminary stub-out sizing. Preliminary stub-outs were sized assuming the entire 25-year flow will reach the corridor and be intercepted by new catch basins connected to the stub-outs. All existing storm drains will be connected to the box culvert, either directly or via the proposed stub-outs. The stub-out size, as shown in Table 2.5, is based on intercepting the entire 25-year peak subbasin Q. The stub-out design flow listed in Table 2.6 in some cases indicates a reduction in the peak Q. This reduction is due to the existing storm drain capacity and is only applied when said capacity is greater than ten percent of the peak Q. This reduced flow would be used to design the stub-outs if the existing storm drain can be directly connected to the main line box culvert and not to the stub-out.

2.5.1 Existing Storm Drains

There are five existing major storm drains which empty into the OCCC within the project boundary. They are located at McDowell Road, Virginia Avenue, Thomas Road, Earll Drive and Indian School Road. There are six other small single and double catch basin drains along the OCCC located just north of Weldon Avenue, south of Weldon Avenue, at

Osborn Road, north of Pinchot Avenue at Oak Street and at Holly Street. These smaller systems do not contribute appreciably to the flow into the OCCC.

The McDowell Road storm drain is a 54" RCP pipe which empties into the OCCC culvert beneath McDowell Road at the canal crossing. Currently, the storm drain extends east along McDowell Road to 52nd Street where it follows 52nd Street north. A future extension of the existing McDowell Road storm drain further north along 52nd Street is planned as part of the 52nd Street project. There is one minor lateral extending north along 48th Street. There are approximately 17 existing catch basins which make up the interception system. Since this storm drain outlets underneath McDowell Road inside the culvert undercrossing of the OCCC, it is technically outside of the scope of this study. However, the flows carried by the storm drain are almost entirely made up of those generated within Subbasin 2C.3 of this study. The extension of the 52nd Street portion of this storm drain will not adversely affect the new box culvert.

The storm drain along Virginia Avenue is a 24" pipe which extends east to 50th Street and includes some 11 catch basin connections. This storm drain has an estimated capacity of 23 cfs. Its watershed takes in portions of Subbasins 2B and 2A.3.

The storm drain along Thomas Road is a 36" pipe which extends east to 56th Street. There is a lateral north on 52nd Street. Approximately 32 catch basins empty into this system. The tributary watershed of this system drains parts of Subbasins 1B.2 and 1C.2.

The Earll Drive storm drain system is a large system which extends to 56th Street. Major laterals exist along 52nd, 54th and 56th Streets. This storm drain outlets into the OCCC through a 54" pipe at Earll Drive. The watershed of this storm drain is contained within Subbasin 1B.2.

There are two storm drains which empty into the OCCC at Indian School Road. The first enters from the west via a 33" pipe which extends to the alley just west of 47th Street. This storm drain has approximately four catch basins. Because its capacity at 20 cfs is negligible in comparison with total flow of the OCCC, its contribution to the OCCC was not considered as part of this report. The other storm drain at Indian School Road approaches from the east and empties into the OCCC through an 18" pipe. This line extends east to 49th Street and totals three catch basins. Its tributary area is located within Subbasin AO.

All City of Phoenix storm drains are sized for the two-year storm. The Manning's equation was used to calculate the capacity of each existing storm drain, utilizing as-built information to determine pipe size, slope and materials. In Table 2.7 the various storm drains and their individual pipe sizes, slopes, materials and estimated capacities are shown versus 25-year peak flows. In developing Table 2.7, it was discovered that not all of the watershed for the various existing and proposed storm drains coincide with the watershed developed for sizing the 25-year stub-out. For this reason, it was necessary to determine the interception capacity of the existing or proposed storm drains, based on the amount of flow intercepted from the watersheds developed for this project. An example of this situation is the proposed Oak Street outfall of the 52nd Street storm drain project. The pipe capacity at the outfall is approximately 105 cfs, but only 76 cfs of this flow is generated from Subbasin 2B, which is the subbasin used to calculate the 25-year stub-out design flow at this location. Therefore, the capacity used for the Oak Street outfall is 76 cfs.

There are two storm drains within the study watershed--52nd Street and Indian School storm drains--that are in the planning or design phase. The 52nd Street project involves several distinct storm drains and is almost complete through the design phase. The Indian School project, however, is still under study and firm plans and locations of proposed improvements are not available at this time.

The portion of the 52nd Street storm drain project which most affects this study consists of a 48" RCP pipe which empties into the OCCC at Oak Street. The design flow is approximately 105 cfs. The watershed of this portion of the 52nd Street storm drain is made up of parts of Subbasins 2B, 2C.1 and 2C.3. Another portion of the 52nd Street storm drain project as discussed earlier will lengthen the existing lateral of the McDowell Road storm drain further north along 52nd Street.

The percentage of expected 25-year runoff carried by existing or proposed storm drains varies from seven to 83 percent. Because the 25-year predicted outflow is approximate, existing storm drain capacities less than ten percent of the expected 25-year peak will be neglected. These will include the storm drain at Oak Street, as well as the six smaller one and two catch basin systems along the alignment. Runoff carried by existing storm drains greater than ten percent of expected 25-year peak flow will be subtracted from peak flow to size the new stub-outs and catch basins.

TABLE 2.1
SUBBASIN - DRAINAGE CHARACTERISTICS

HEC-1 Subbasin	Area (Sq. Mi.)	Hydrologic Soil Group	25-Year T_c (Hr.)	25-Year R (Hr.)	Hydraulic Length (Ft.)	Slope Along Hydraulic Length (Ft./Ft.)
A0	.013	B	.233	.231	1,000	.008
1A.1	.062	B	.400	.353	2,450	.0069
1A.2	.008	B	.267	.368	1,050	.0067
1A.3	.227	B	.600	.500	5,450	.0055
1B.1	.030	B	.383	.438	2,025	.0044
1B.2	1.125	B, D	.733	.549	14,500	.0143
1C.1	.018	B	.383	.460	1,500	.0027
1C.2	.150	B	.600	.602	5,100	.0043
2A.1	.047	B	.450	.463	2,400	.0033
2A.2	.005	B	.250	.359	800	.0038
2A.3	.276	B, D	.500	.499	8,000	.0121
2B	.505	B, D	.300	.190	7,500	.0403
2C.1	.053	B	.300	.253	2,150	.0065
2C.2	.057	B	.333	.308	2,500	.0072
2C.3	.175	B, D	.267	.216	4,875	.0326

TABLE 2.2
CITY OF PHOENIX
ZONING VERSUS PERCENT IMPERVIOUSNESS

Zoning Types	Percent Impervious
RE-43 Single Family Residence 43,560 SF Lots Minimum	15
RE-35 Single Family Residence 35,000 SF Lots Minimum	15
RE-24 Single Family Residence 24,000 SF Lots Minimum	18
RI-18 Single Family Residence 18,000 SF Lots Minimum	18
RI-14 Single Family Residence 14,000 SF Lots Minimum	20
RI-10 Single Family Residence 10,000 SF Lots Minimum	25
RI-8 Single Family Residence 8,000 SF Lots Minimum	25
RI-6 Single Family Residence 6,000 SF Lots Minimum	25
R-3 Multi-Family Residence 1 Unit for Each 3,000 SF	60
R-4 Multi-Family Residence 1 Unit for Each 1,500 SF	65
R-4A Multi-Family Residence 1 Unit for Each 1,000 SF	70
R-5 Multi-Family Residence 1 Unit for Each 1,000 SF	70
C-0 Commercial Office District-Restricted Commercial	75
HR High Rise District	90
PSC Planned Shopping Center	90
C-1 Neighborhood Commercial	85
C-2 Intermediate Commercial	85
C-3 General Commercial	85
P-1 Parking (Open)	85
P-2 Parking (Structures)	85
IND Park Industrial Park	75
A-1 Light Industrial	75

Zoning Types	Percent Impervious
A-2 Heavy Industrial	75
P.A.D. Planned Area Development	85
D.G. Dwelling Group	85
RIGHT-OF-WAY (R.O.W.)	100

TABLE 2.3
SUBBASIN IMPERMEABILITY

	Percent of Impervious Cover		
Subbasin	Minimum	Weighted Average	Maximum
A0	85	85	85
1A.1	25	34	85
1A.2	25	25	25
1A.3	18	23	25
1B.1	25	25	25
1B.2	18	25	85
1C.1	25	25	25
1C.2	25	31	85
2A.1	25	66	85
2A.2	60	60	60
2A.3	25	43	85
2B	15	25	70
2C.1	15	15	15
2C.2	15	35	60
2C.3	25	55	75

TABLE 2.4
PEAK 25-YEAR FLOW ALONG THE OLD CROSS CUT CANAL

	Peak Q in CFS	
Location	**Greiner***	**FCD****
Arizona Canal	2600	2600
Indian School	2606	2615
Weldon	2642	
Whitton	2646	
Osborn	2774	2750
Richardson	2793	
Earll	3328	3200
Pinchot	3340	
Thomas	3432	3300
Windsor	3470	
N. of Virginia	3473	
Virginia	3681	3500
Oak	4031	3800
Holly	4074	
Granada	4118	3900
McDowell	4266	4100

*25-year Lafayette storm drain + 1,000 cfs Arizona Diversion + 25-year side inflows

**Flood Control District of Maricopa County, memorandum dated March 1, 1991

TABLE 2.5
PEAK 25-YEAR, 50-YEAR AND 100-YEAR FLOW ALONG THE OCCC

	Peak Q in cfs		
Cross Street	**25-Year**	**50-Year**	**100-Year**
Indian School	2606	3339	3340 3291
Weldon	2642	3392	3405
Whitton	2646	3398	3409
Osborn	2774	3574	3804
Richardson	2793	3605	3841
Earll	3328	4409	4901
Pinchot	3340	4429	4912
Thomas	3432	4547	5070
Windsor	3470	4598	5127
N. of Virginia	3473	4607	5127
Virginia	3681	4865	5449
Oak	4031	5362	6013
Holly	4074	5427	6086
Granada	4118	5479	6167
McDowell	4266	5663	6381

OK

TABLE 2.6
SIDE INFLOWS AND PRELIMINARY STUB-OUT SIZING

Cross Street	HEC-1 Subbasin	Peak Subbasin Q	Stub-Out		Approximate Station
			Length	Size	
Indian School	A0	24	80	24"	114+99
Weldon	1A.1	71	60	48"	99+56
Whitton	1A.2	9	45	18"	92+25
Osborn	1A.3	196	40	7' x 4' CBC	88+79
Richardson	1B.1	33	80	30"	81+70
Earll	1B.2	802	60	2-10' x 7' CBC	75+40
Pinchot	1C.1	19	95	24"	70+06
Thomas	1C.2	129	90	5' x 5' CBC	62+94
Windsor	2A.1	56	85	42"	55+60
N. of Virginia	2A.2	7	70	18"	54+00
Virginia	2A.3	300	85	10' x 6' CBC	49+09
Oak	2B	917	95	2-10' x 7' CBC	36+00
Holly	2C.1	88	75	48"	25+97
Granada	2C.2	81	60	48"	19+94
McDowell	2C.3	322		54"	9+20

TABLE 2.7
EXISTING STORM DRAIN CAPACITY VERSUS 25-YEAR FLOW

Location	HEC-1 Subbasin	Pipe Size	Pipe Slope	Type Pipe	Estimated Storm Drain Interception Capacity	Q_{25}	Percent of Q_{25}	Stub-Out Design Flow
Indian School	A0	18"	3.730%	RCP	22	24	92	2
Weldon	1A.1					71		71
Whitton	1A.2					9		9
Osborn	1A.3					196		196
Richardson	1B.1					33		33
Earll	1B.2	54"	0.430%	RCP	140	802	17	662
Pinchot	1C.1					19		19
Thomas	1C.2	36"		RCP	59	129	46	70
Windsor	2A.1					56		56
N. of Virginia	2A.2					7		7
Virginia	2A.3	24"	0.868%	RCP	23	300	8	300
Oak	2B	*48"		RCP	76	917	8	917
Holly	2C.1					88		88
Granada	2C.2					81		81
McDowell	2C.3	54"	0.750%	RCP	184	322	57	138

*Proposed storm drain

3.0 HYDRAULICS

3.1 METHODOLOGY

Hydraulic calculations include the application of the continuity equation, energy equation and momentum equation. In addition, the control sections were carefully defined and identified. Three computer models were utilized extensively in analysis and design. These programs include HEC-2, WSPG and STORM. The HEC-2 program was used to simulate the existing flooding condition for various storm frequencies, as well as the overland flooding conditions with the proposed improvements. The WSPG program was used to assess hydraulic performance under various improvement schemes for a segment of the complete reach. The STORM program was used to calculate the hydraulic and energy gradients for the proposed improvement.

3.1.1 HEC-2 Program

The U.S. Army Corps of Engineers' HEC-2 program was developed to determine water surface elevations for specified discharges in natural channels of any cross section for subcritical or supercritical steady-state flow.

The effects of natural obstructions to flow, floodplain encroachment and hydraulic structures may be simulated by the program. Bridges are given special consideration for their impact on the flow hydraulics. Culverts, weirs, channel improvements, embankments and levees may also be considered in the flow profile computation.

3.1.2 WSPG Program

The program was developed by the Design Systems and Standards Group of the Design Division and the Data Processing Section of the Business and Fiscal Division of the Los Angeles County Flood Control District.

The program computes and plots uniform and non-uniform steady flow water surface profiles and pressure gradient sections. The flow in a system may alternate between supercritical, subcritical or pressure flow in any sequence. The program will also analyze natural river channels, although the principle use of the program is intended for determining profiles in improved flood control systems.

3.1.3 STORM Program

STORM is a modular hydraulic analysis program designed to evaluate existing or proposed storm drain systems. The program was developed by the Data Systems Division, Technical Systems Section of the County of Los Angeles Road Department.

The storm drain analysis program will calculate the hydraulic grade line elevations of a proposed or existing storm drain system, given the physical characteristics and the discharge flow (Q).

The program allows for pressure flow or partial flow with cross sections being either circular or rectangular box. A rectangular open channel can be analyzed as a box cross section, providing the results show that it is flowing partially full throughout the entire system, so that the soffit does not affect the computations.

The program starts the computation for the hydraulic grade line by evaluating the friction losses and the minor losses throughout the system. The junction losses are evaluated by equating pressure plus momentum for the incoming and outgoing flows through the junction. This is accomplished by applying the formula developed by the City of Los Angeles, which establishes that the summation of pressures, ignoring friction, is equal to the average cross section flow area, multiplied by the change in the hydraulic gradient through the junction. The basic flow elevations used for the main lines at either end of the junction that apply to the pressure, plus the momentum equation, depend on the type of flow at each end of the junction. These elevations are determined by computing the drawdown curves for each line. The control elevation for the lateral or lateral system is taken as the average of the hydraulic grade line elevations at both ends of the junction. If the water elevation in the lateral is above this control, the momentum contributed by the lateral in the analysis of the junction is decreased in proportion to the ratio of the area in the lateral below the control to the total area of the flow.

The point with greater force will be the control point. The point at the other end of the junction is determined by satisfying the pressure plus momentum equation.

Any of these points may be overridden by the backwater curve originating at the main control at the downstream end of the system. If this is the case, then the pressure plus momentum equation is applied to the point or points determined by the backwater curve during the upstream analysis.

When the flow changes from partial to full, or from full to partial, the program determines and prints the location where this change occurs. If the flow reaches normal depth within a line, the program determines and prints this location. When the flow changes from supercritical to subcritical because of downstream conditions, it happens through a hydraulic jump; the program determines the location of the jump by equating the pressure plus momentum for the two kinds of flow. It prints the jump location, pressure plus momentum at the jump, and the depth of water before and after the jump.

3.2 EXISTING CULVERTS

There are four roadway crossings along the OCCC in the study area. The newly constructed McDowell Road culverts are located at the very downstream end of the project. The new culverts were designed to convey a flow rate of 4,100 cfs. However, some discrepancies were observed during the preliminary study of this project. The capacity of the culvert and its hydraulic performance will set the tone for the remaining improvements and require comprehensive analysis. The culverts at Thomas Road, Osborn Road and Indian School Road were assessed to determine the potential uses for the proposed improvements.

3.2.1 McDowell Road

The existing McDowell Road/OCCC structure is a double barrel, reinforced concrete box with a 10' x 10' cell and a 14' x 10' cell. The 10' x 10' box is composed of two segments. One segment has a slope of approximately one percent and was built in the 1940's. The other segment has a slope of 0.28 percent and was built in 1976. The 14' x 10' box has a slope of one percent and was built in 1990. A 16' wide concrete rectangular channel at a slope of 0.532 percent located at the downstream of the McDowell Road culverts was constructed by the Arizona Department of Transportation (ADOT) as part of the Hohokam Parkway improvements. The McDowell Road culverts and the ADOT channel are connected by a concrete rectangular transition channel at a slope of 1.633 percent. The transition structure is 242.1' long with a 26' width at the upstream and 16' width at the downstream end. The upstream top of the bank elevation is at 1,193.9'.

A capacity/energy gradient rating curve under the condition of no downstream backwater effect was developed. The culvert capacity at bank full is 2,813 cfs. At a flow rate of 4,100 cfs, the energy gradient is at an elevation of 1,196.34' for a complete pressure flow. Under the existing condition, at a flow rate of 4,100 cfs, 3,500 cfs will be flowing through the culvert under pressure flow condition, and the remaining 600 cfs will be over the top of McDowell Road as overland flow at an energy gradient at 1,195.24'. The culvert capacity is reduced to 2,200 cfs at a head elevation of 1,191.0', which provides 2' of freeboard to the existing ground. The elevation at the 2' freeboard is the target optimum design energy elevation for a functional lateral flow collector throughout the entire system.

A backwater analysis determined the existing culvert is controlled by the downstream ADOT channel's energy. This channel begins about 242' south of the downstream face of the McDowell Road CBC. The energy at the upstream face of the McDowell Road CBC is 1,196.61. This is above the existing adjacent top of ground at the north end of the culvert.

When the existing McDowell Road CBC is investigated using an elevation of 1,192.1, which provides a 1' freeboard, the culvert can convey 3,000 cfs. This flow can be conveyed in an upstream channel similar to the ADOT rectangular channel, 16' bottom width and vertical walls.

The following provides backwater calculations which investigate the existing condition:

Location	Y Ft	V fps	$V^2/2g$ Ft	Inv. El.	S^f %	X Ft	S^f %	h^f Ft	h^e Ft	h^c Ft	Eg Ft
End of ADOT Channel	12.69	20.19	6.33	1173.16	.50	--	--	--	--	--	1192.18
D.S. MCC	14.04	11.23	1.96	1177.11	.10	242.1	.30	0.73	--	0.20	1193.11
U.S. MCC	12.23	17.08	4.53	1179.85	.78	274.0	--	2.14	1.36	--	1196.61

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 3-17-91

Description Existing McDowell Rd. Checked By ______ Date ______

Culvert Capacity Rating Curve (Q=4100 cfs) Sheet ___ of ___

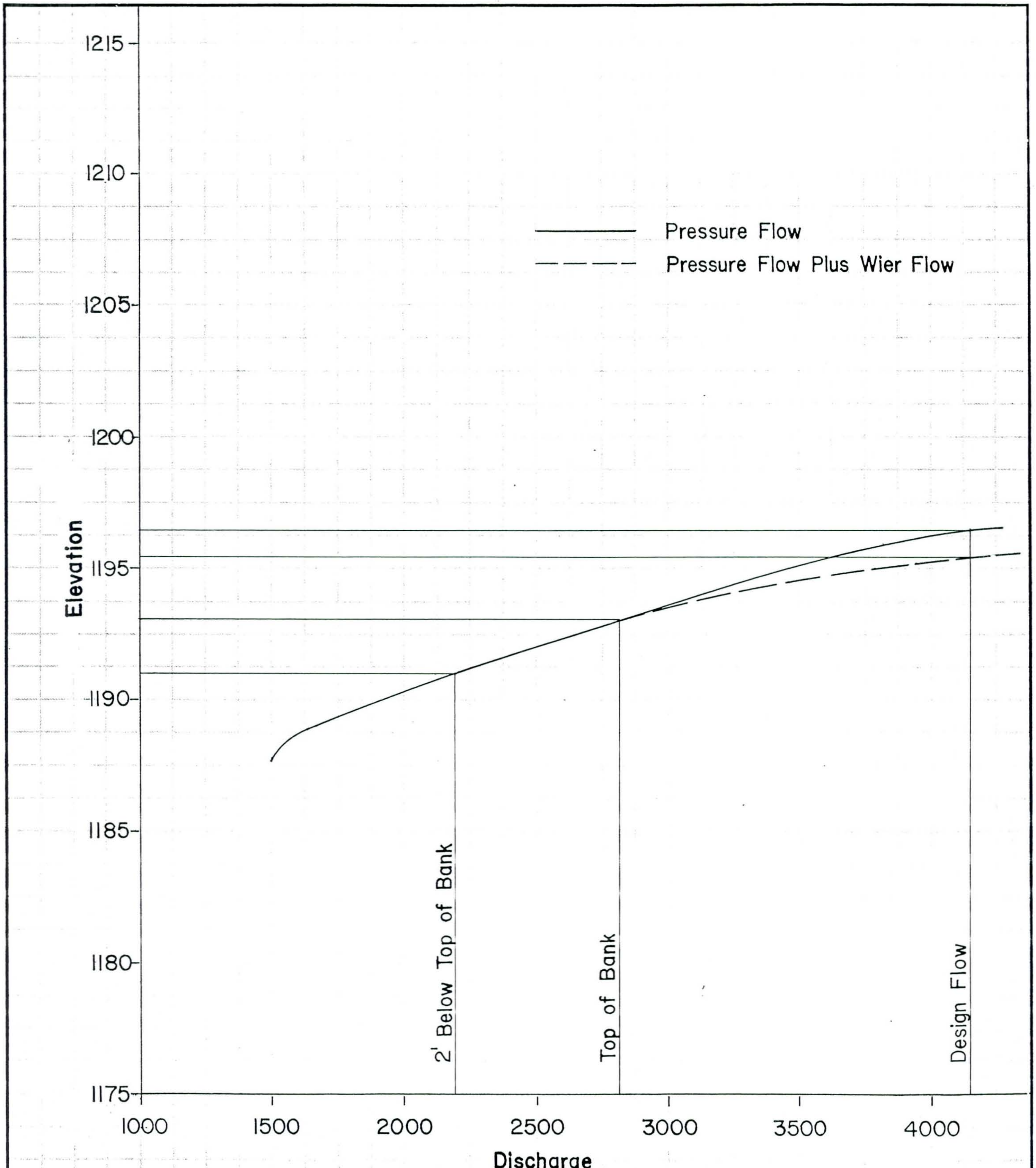

FIGURE 3.2-1

3.2.2 Thomas Road

The existing Thomas Road/OCCC structure is a double barrel reinforced concrete box with a 10' x 10' cell at the west side and a 10' x 9' cell at the east. These boxes are 80' long with different inverts. The west cell has an inlet invert at elevation 1,199.68, an outlet invert at 1,199.21 and a slope of 0.59 percent. The east cell has an inlet invert at elevation 1,200.40, an outlet invert at 1,199.75 and a slope of 0.81 percent. There are inlet and outlet wingwalls at an angle of approximately 75 degrees. Concrete aprons are also constructed at both ends.

A capacity/energy gradient rating curve under the condition of no downstream backwater effect was developed. The culvert capacity at bank full is 1,550 cfs. At a flow rate of 3,600 cfs, the energy gradient will be at an elevation of 1,218' to have a complete pressure flow condition. Under the existing condition, at a flow rate of 3,600 cfs, 2,100 cfs will be flowing through the culvert and the remaining 1,500 cfs will overtop Thomas Road as overland flow at an energy gradient at 1,212.5', which is 2.5' above the Thomas Road pavement. For the proposed enclosure system, the culvert capacity is reduced to 1,180 cfs at an allowable head elevation of 1,008'.

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Job Old Cross Cut Canal E002100 Computed By RHF Date 3-17-91

Description Existing Thomas Road Checked By Date

Culvert Capacity Rating Curve (Q=3600 cfs) Sheet of

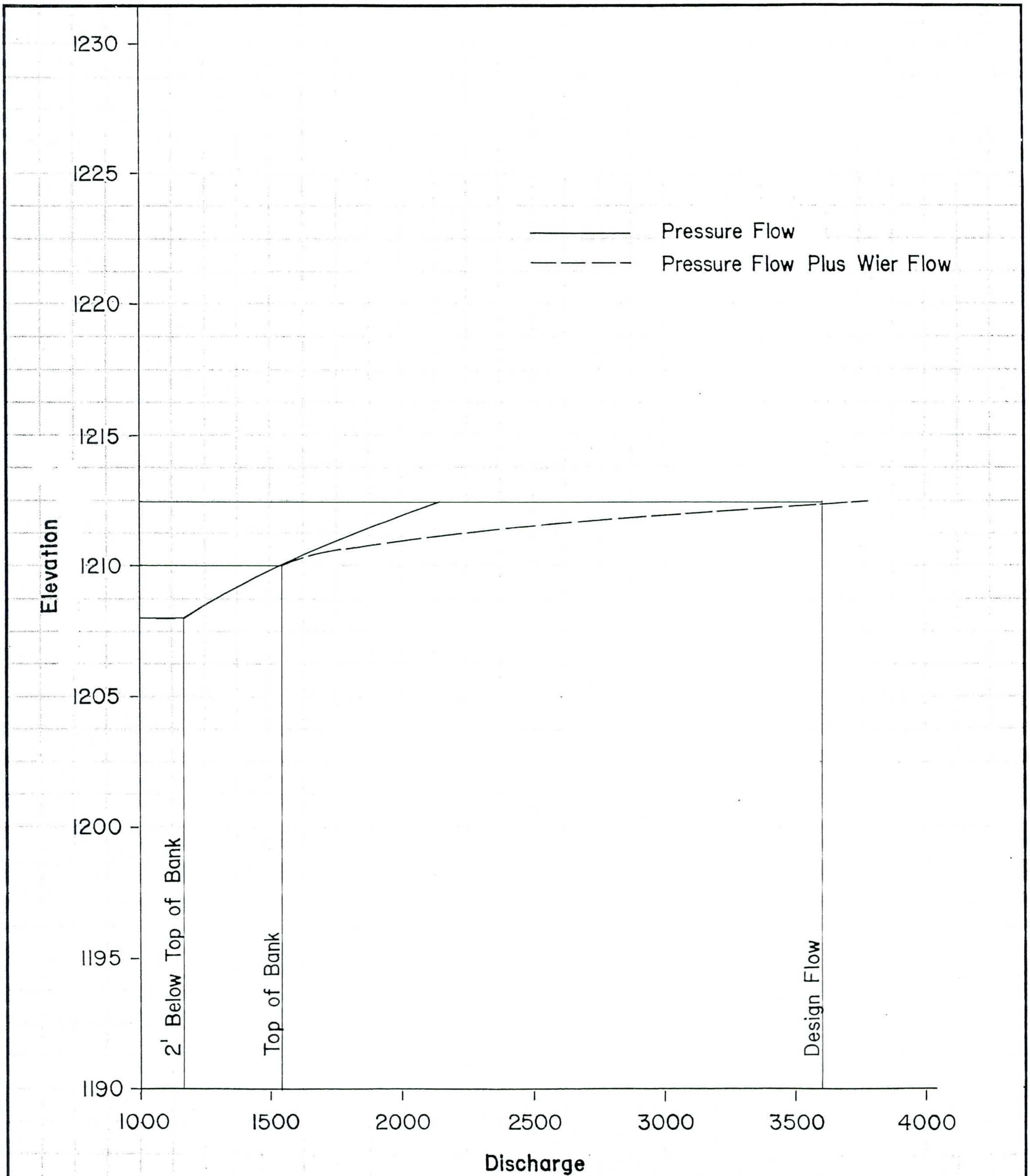

FIGURE 3.2-2

3.2.3 Osborn Road

The existing Osborn Road/OCCC structure has a 7' diameter reinforced concrete pipe at the west side and an 8' x 8' concrete box at the east. These culverts are 61' long with different inverts. The pipe has an inlet invert at elevation 1,206.38, an outlet invert at 1,205.8 and a slope of 0.95 percent. The east box has an inlet invert at elevation 1,206.37, an outlet invert at 1,205.72 and a slope of 1.07 percent. There are inlet and outlet wingwalls at an angle of approximately 90 degrees. Concrete aprons are also constructed at both ends. The top of the road elevation is approximately 1,222.5'.

A capacity/energy gradient rating curve under the condition of no downstream backwater effect was developed. The culvert capacity at bank full is 1,900 cfs. At a flow rate of 3,600 cfs, the energy gradient is at an elevation of 1,300' in order to have a complete pressure flow condition. Under the existing condition, at flow rate of 3,600 cfs, 2,250 cfs will be flowing through the culvert and the remaining 1,350 cfs will overtop Thomas Road as overland flow at an energy gradient of 1,226', which is 3.5' above the Osborn Road pavement. For the proposed enclosure system, the culvert capacity is reduced to 1,700 cfs at an allowable head elevation of 1,120'.

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Job Old Cross Cut Canal E002100 Computed By RHF Date 3-17-91

Description Existing Osborn Rd. Checked By Date

Culvert Capacity Rating Curve (Q=3600 cfs) Sheet of

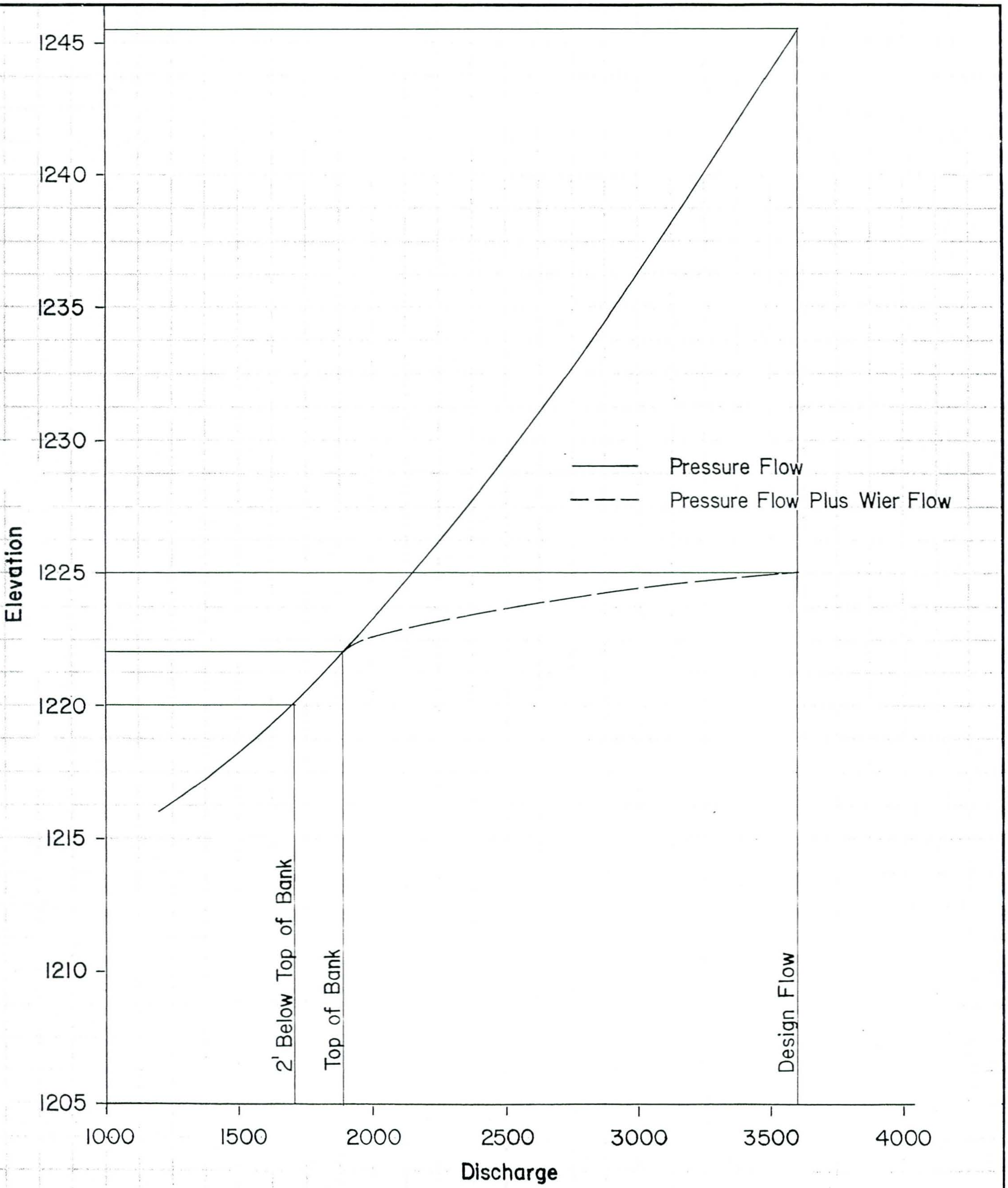

FIGURE 3.2-3

3.2.4 Indian School Road

The existing Indian School Road/OCCC structure has a 7' diameter reinforced concrete pipe at the west side and an 8' x 8' concrete box at the east. These culverts are 133' long with different inverts. The pipe has an inlet invert at elevation 1,229.79, an outlet invert at 1,229.68 and a slope of 0.12 percent. The east box has an inlet invert at elevation 1,229.25, an outlet invert at 1,229.18 and a slope of 0.05 percent. There are inlet and outlet wingwalls at an angle of approximately 45 degrees. Concrete aprons are also constructed at both ends. The top of the road elevation is approximately 1,243.9'.

A capacity/energy gradient rating curve under the condition of no downstream backwater effect was developed. The culvert capacity at bank full is 1,550 cfs. At a flow rate of 3,000 cfs, the energy gradient is at an elevation of 1,262' in order to have a complete pressure flow condition. Under the existing condition, at a flow rate of 3,000 cfs, 1,800 cfs will be flowing through the culvert and the remaining 1,200 cfs will overtop Thomas Road as overland flow at an energy gradient of 1,245.8', which is 19' above the Indian School Road pavement. For the proposed enclosed system, the culvert capacity is reduced to 1,350 cfs at an allowable elevation of 1,241'.

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Job Old Cross Cut Canal E002100 Computed By RHF Date 3-18-91

Description Existing Indian School Rd. Checked By Date

Culvert Capacity Rating Curve (Q=3000 cfs) Sheet ___ of ___

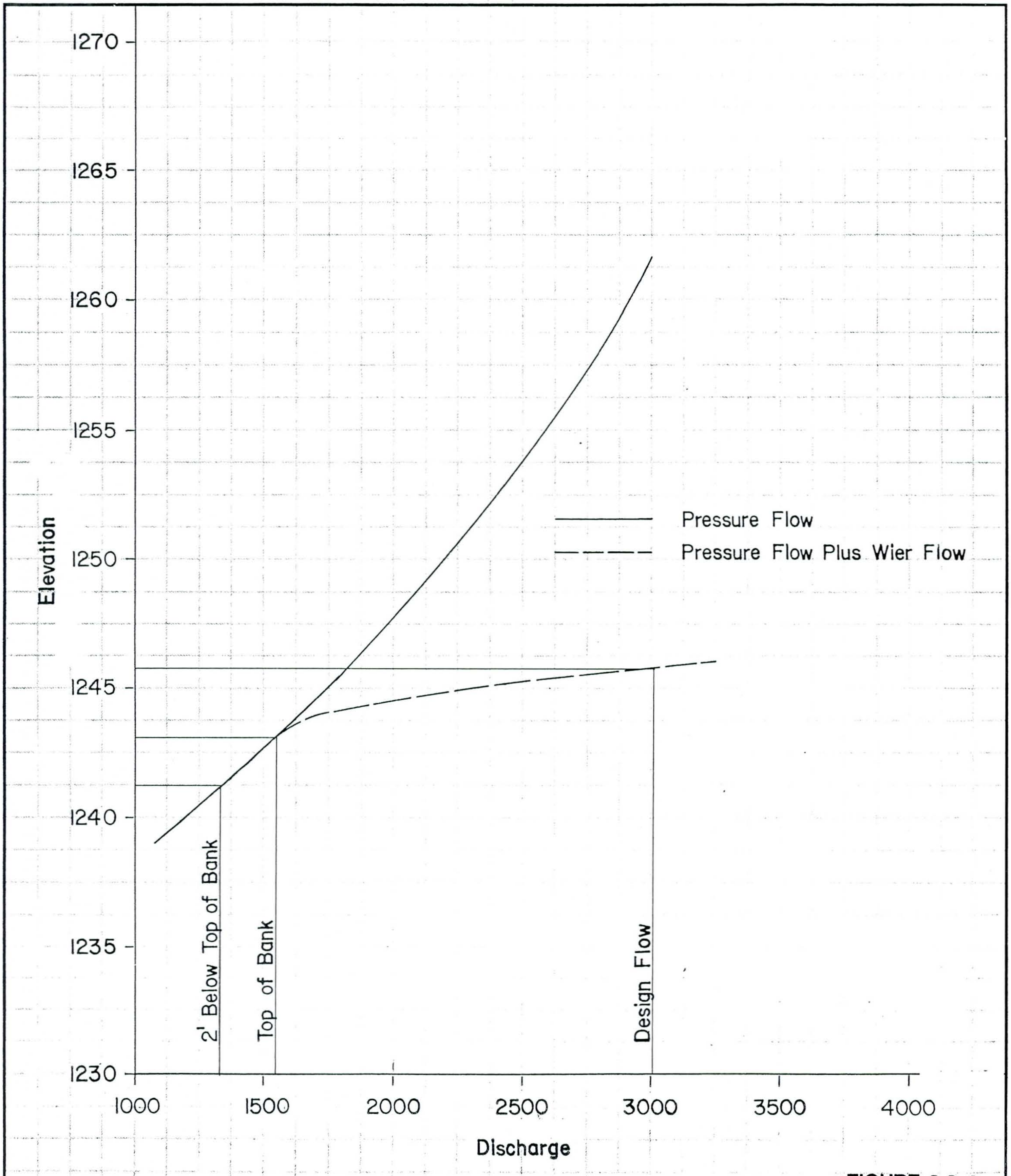


FIGURE 3.2-4

3.3 SIZING THE MAIN LINES

The selected main lines shall have the minimum size, optimum configuration, least cost and will function at the design flow conditions.

For the design, Manning's "n" values of 0.013 and 0.014 were used; however, during the design process, an "n" value of 0.015 was checked. "N" value of 0.013 represents a smooth finished concrete, 0.014 was used for a normal finished concrete surface and 0.015 represents a condition of some wear and tear. Self-cleaning velocity of six fps or higher was also incorporated into the design.

The design flow rates along the main line can be divided into four portions. Portion 1 is 4,100 cfs from McDowell Road to Virginia Avenue; Portion 2 is 3,600 cfs from Virginia Avenue to Osborn Road; Portion 3 is 3,000 cfs from Osborn Road to the Arizona Canal; and Portion 4 is 1,600 cfs from the Arizona Canal to the Lafayette drain at Camelback Road.

Figures 3.3-1 through 3.3-4 are culvert rating curves that were developed for a variety of culvert sizes and configurations for each portion under the design flow rates.

The existing ground along the OCCC alignment may be divided into four reaches based on ground slopes. The first reach, from McDowell Road to Avalon Drive, has a slope of approximately 0.32 percent. The second reach, from Avalon Drive to Whitton Avenue, is at a slope of 0.625 percent. The third reach, from Whitton Avenue to the Arizona Canal, is at a slope of 0.88 percent. The fourth reach, from the Arizona Canal to Camelback Road, is at a slope of 0.9 percent.

Without consideration of the system's minor losses and impacts from downstream structure and segments, optimum system size and configuration may be tentatively established by applying the ground slopes to the capacity rating curve figures for the corresponding portion and reach. This was done by choosing the minimum structure size that will have a friction slope of less or equal to the ground slope. Five structure segments were established with the corresponding portions and reaches.

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description Mainlines McDowell to Checked By ______ Date ______

Virginia Area vs. Friction Slope (Q=4100 CFS) Sheet ___ of ___

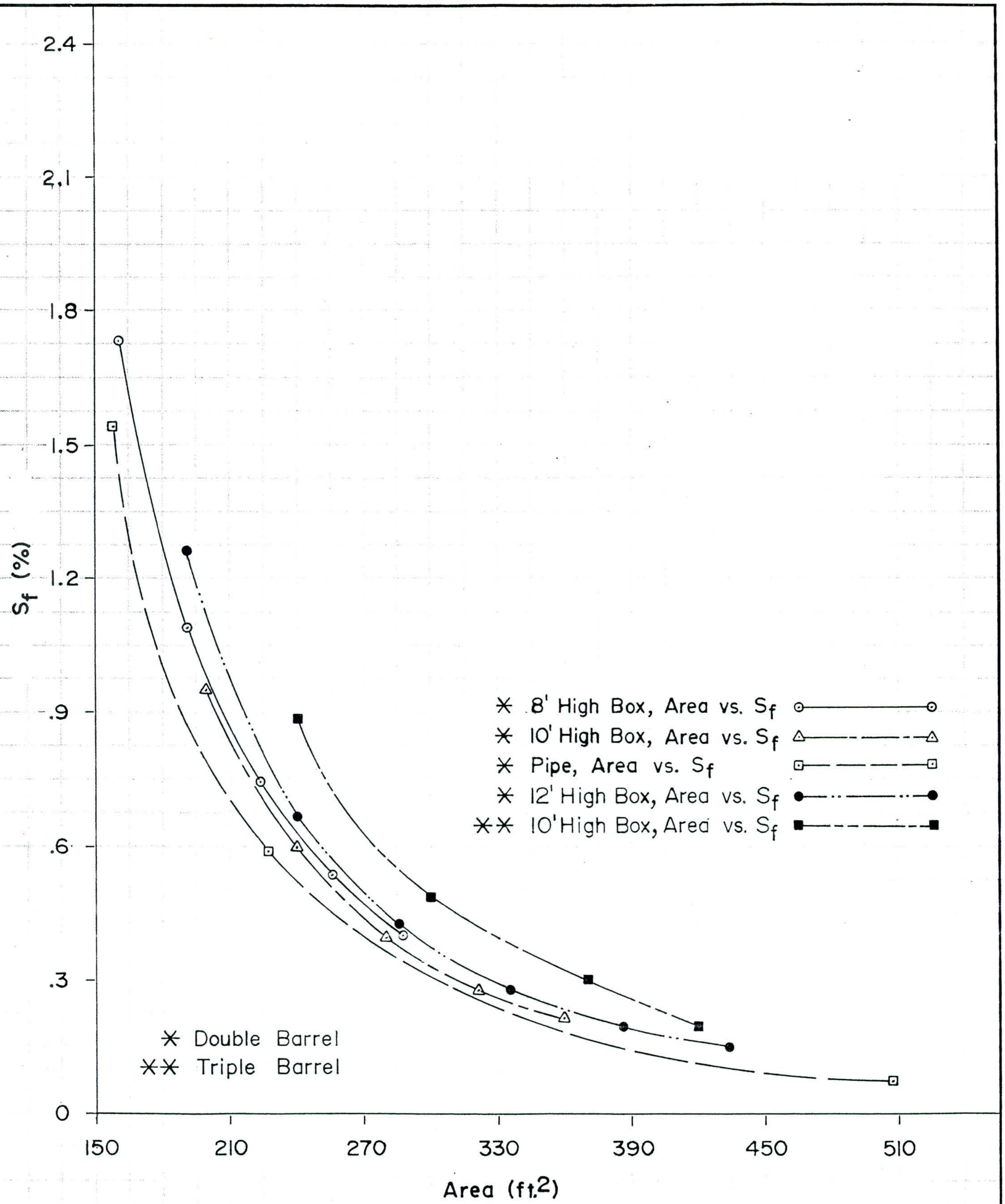


FIGURE 3.3-1

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 3-13-91

Description Virginia to Osborn Main Lines Checked By ________ Date ________

Area vs. Friction Slope (Q=3600 cfs) Sheet ___ of ___

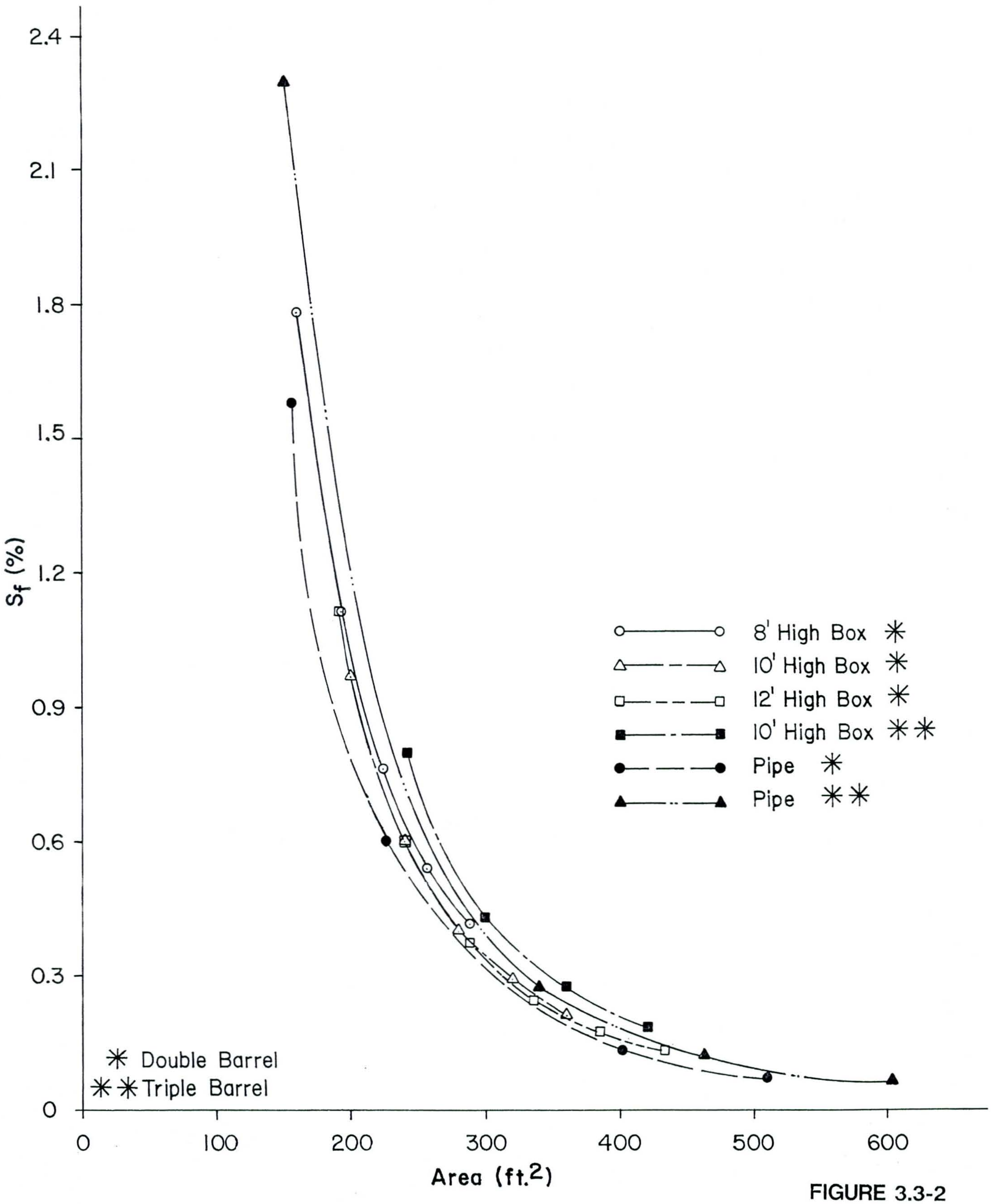


FIGURE 3.3-2

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 3-13-91

Description Osborn to Indian School Rd. Main Lines

Area vs. Friction Slope (Q=3000 cfs) Sheet ____ of ____

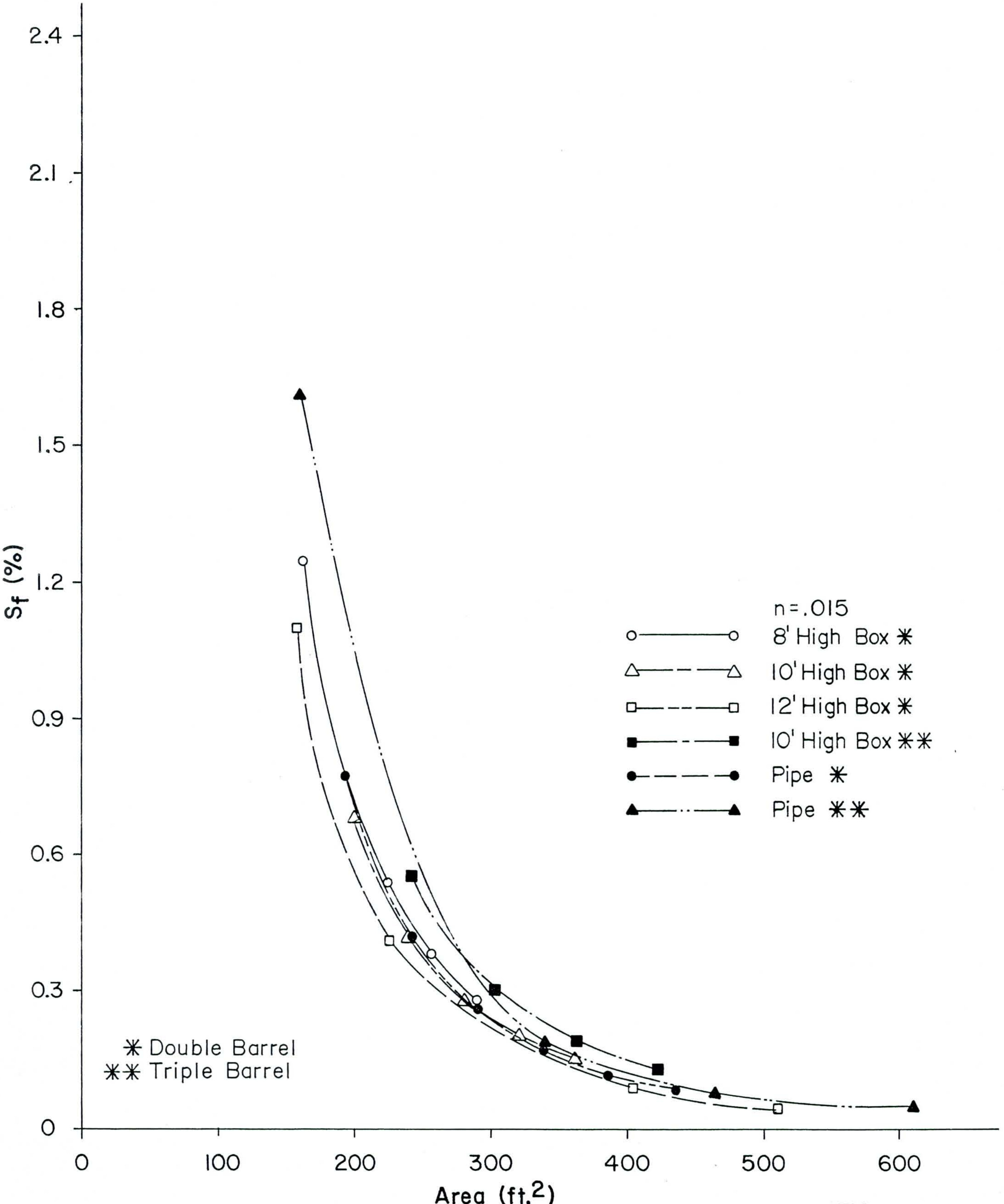


FIGURE 3.3-3

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description Mainline Arizona Canal to Checked By ______ Date ______

Lafayette Drain Area vs. Friction Slope n=.014 Sheet ___ of ___

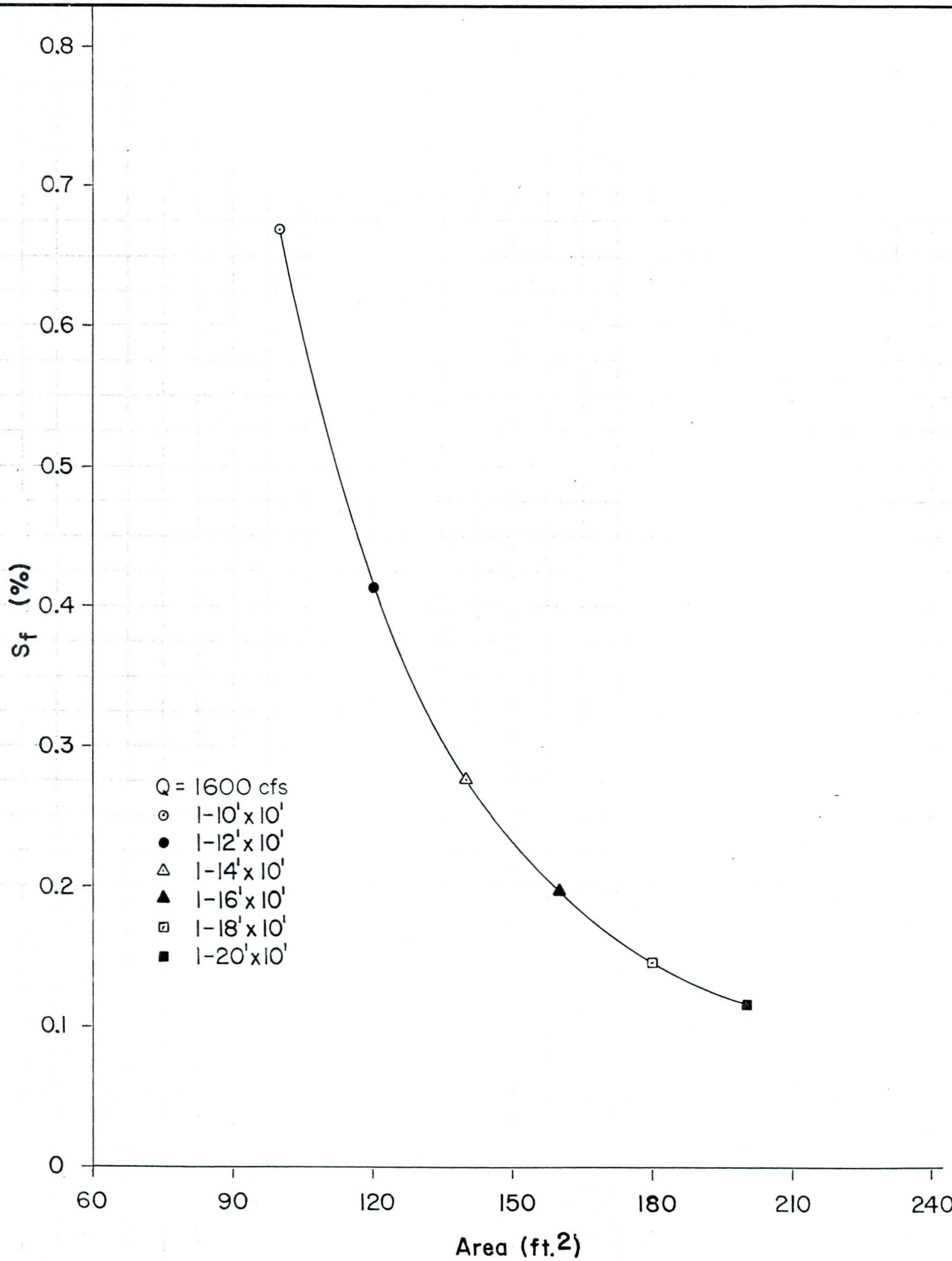


FIGURE 3.3-4

3.3.1 Segment 1 - From McDowell Road to Virginia Avenue

The preliminary culvert alternatives include:

1. Two 16' x 10' box culverts at a friction slope of 0.30 percent
2. Two 14' x 12' box culverts at a friction slope of 0.30 percent
3. Three 12' x 10' box culverts at a friction slope of 0.32 percent
4. Double 18' diameter pipes at a friction slope of 0.08 percent
5. Triple 14' diameter pipes at a friction slope at 0.32 percent
6. A single 16' x 18' box culvert at a friction slope of 0.30 percent

3.3.2 Segment 2 - Virginia Avenue to Avalon Drive

The preliminary culvert alternatives include:

1. Two 16' x 10' box culverts at a friction slope of 0.29 percent
2. Two 14' x 12' box culverts at a friction slope of 0.24 percent
3. Three 12' x 10' box culverts at a friction slope of 0.27 percent
4. Double 18' diameter pipes at a friction slope of 0.075 percent
5. Triple 14' diameter pipes at a friction slope of 0.32 percent
6. A single 16' x 16' box at a friction slope of 0.32 percent

3.3.3 Segment 3 - Avalon Drive to Whitton Avenue

The preliminary culvert alternatives include:

1. Two 16' x 8' box culverts at a friction slope of 0.54 percent
2. Two 10' x 12' box culverts at a friction slope of 0.60 percent
3. Three 10' x 10' box culverts at a friction slope of 0.43 percent
4. Double 12' diameter pipes at a friction slope of 0.60 percent
5. Triple 10' diameter pipes at a friction slope of 0.60 percent
6. A single 16' x 12' box at a friction slope of 0.69 percent

3.3.4 Segment 4 - Whitton Avenue to the Arizona Canal

The preliminary culvert alternatives include:

1. Two 10' x 10' box culverts at a friction slope of 0.68 percent
2. Two 12' x 8' box culverts at friction slope of 0.77 percent
3. Three 10' x 8' box culverts at a friction slope of 0.55 percent
4. Double 12' diameter pipes at a friction slope of 0.41 percent
5. Triple 10' diameter pipes at a friction slope of 0.80 percent
6. A single 16' x 10' box at a friction slope of 0.80 percent

3.3.5 Segment 5 - From the Arizona Canal to Camelback Road

The preliminary culvert size includes a 10' x 10' box culvert at a friction slope of 0.70 percent.

3.4 TRANSITION STRUCTURES

The transition in a channel and/or conduit is a structure designed to change the shape or cross-sectional area of the flow. Under normal design and installation conditions, practically all canals and flumes require some type of transition structure to and from the waterways. The function of such a structure is to avoid excessive energy losses to eliminate cross waves and other turbulence, and to provide safety for the structure and waterway. When the transition is designed to keep streamlines smooth and nearly parallel and to minimize standing waves, the theory of gradually varied flow may be used in the design. The essence of such a design is the application of the energy and momentum principles.

There are three types of transition structures that may be required for the OCCC improvements project, namely:

1. Transition between the canal and flume or tunnel: may include the McDowell Road culvert to the new mainline Segment 1 and transition between mainline segments.

2. Transition between the channel and inverted siphon: may include utility relocation and the McDowell Road culvert.

3. Transition at the junction structures: may include major side inlets and the structure at the Arizona Canal and the Lafayette drain.

4. Transition between culvert barrels as flow equalizes.

3.4.1 Transitions Between the Canal and Flume or Tunnel

On the basis of the performance of existing structures, the following features are important in design.

1. **Proportioning.** For a well-designed transition, the following rules for proportioning should be considered:

   A. The optimum maximum angle between the channel axis and a line connecting the channel sides between entrance and exit sections is 12.5 degrees.

   B. Sharp angles either in the water surface or in the structure that will induce extreme standing waves and turbulence should be avoided.

2. **Losses.** The energy loss in a transition consists of the friction loss and the conversion loss. The friction loss may be estimated by means of any uniform flow formula, such as Manning's formula. The conversion loss is generally expressed in terms of the change in velocity head between the entrance and exit sections of the structure.

3. **Free board at open channel and manholes.**

4. **Elimination of hydraulic jump.** Existence of hydraulic jump in a transition may become objectionable if it hinders the flow and consumes useful energy. When the transition leads from a supercritical flow to a subcritical flow, the hydraulic jump may be avoided by carefully proportioning the transition dimensions.

3.4.2 Transition Between the Channel and Inverted Siphon

1. In the design of an inlet transition, it is generally desirable to have the top of the siphon opening set slightly below the approaching normal water surface. This practice will minimize possible reduction in siphon capacity caused by the introduction of air into the siphon. The depth of submergence of the top of the siphon opening is known as the water seal. The recommended value of the water seal is between a minimum of 1.1 Δh_v and a maximum of 18" or 1.5 Δh_v, whichever is greater. It should be noted that the use of the minimum value in a well-designed transition theoretically allows the flow to barely touch the top of the siphon opening; whereas use of larger values up to the maximum provides a seal of water above the top of the opening. An adequate amount of seal depends upon the slope and size of the siphon barrel. Generally, a large and steep barrel requires a large seal.

   For long siphons, under certain conditions, the inlet may not necessarily be sealed. Consequently, a hydraulic jump may occur in the siphon barrel, and the resulting operating condition will be unfavorable.

2. After the seal is determined for the inlet structure, the velocity at the headwall is computed, and the total drop in water surface, neglecting friction losses, is taken as 1.1 Δh_v. A smooth flow profile is then assumed, tangent to the water surface in the canal at the beginning of the transition and passing through the point at the headwall set by above computation. There are no data available for determining the best form of the flow profile.

3. In the design of the outlet structure, the theoretical rise in water surface from the headwall to the end of the transition, neglecting recovery losses, should be equal to the total change in velocity head Δh_v.

4. In the design of the outlet structure, the bottom slope need not be tangent to the slope of the closed conduit at the headwall as was the case of the inlet, unless the siphon velocity is high and the transition slope is steep.

3.4.3 Transition at Junction Structures

The momentum equation is applied in calculating losses through junction structures.

Figures 3.4-1 through 3.4-5 are junction loss versus junction angles at various locations.

3.4.4 Transition Between Culvert Barrels as Flow Equalizes

Most off-site flows are contributed from the east. Open chambers are required to balance the flow among the culvert barrels.

3.5 SIDE INLETS

Both stub-out and catch basin sizing along the OCCC between the Arizona Canal and McDowell Road were sized using the maximum 25-year, 6-hour storm centered over the contributing subbasin. Design flows generated by this method were larger than those produced by centering the design storm over the total 2.8-square-mile area tributary to the OCCC.

Stub-out sizing methodology was taken from the HEC-2 Water Surface Profile Users Manual, p. IV-21. The head difference was assumed to be uniform 1.0' depth between the OCCC and the head of the flow at the inlet structure. The loss coefficient K was taken to be 1.56. Stub-outs were sized using the total flow developed by the basins.

The design discharge for the grated inlets was based on the assumption the existing storm drain could not be connected to the proposed OCCC culvert and all of the flow for each drainage area would have to be collected at the proposed inlet for each drainage area.

The stub-outs were sized assuming the maximum off-site flow would be collected in the existing off-site storm drain systems and that these systems could be connected to the proposed OCCC. This design assumption provides a conservative approach for cost estimate purposes.

During the design phase of the project, when more detailed information regarding invert elevations and the final horizontal alignment has been determined, it will be resolved if the existing storm drains, which are located beyond the OCCC right-of-way, can be connected to the proposed box system. Once this has been resolved, final grate and stub-out lengths and sizes will be determined and designed accordingly.

The proposed stub-out lengths listed in Table 3.1 are based on a cursory investigation, realizing the final design and alignment of the OCCC are paramount to the actual design of length and location for the stub-outs. The lengths and locations listed in this table were included primarily for cost estimate purposes.

3.6 SURFACE DRAINAGE

Temporary catch basin sizing was done following HEC-12 sizing procedures as outlined in "DRAINAGE OF HIGHWAY PAVEMENTS" HEC-12, FHWA March 1984. Fifty percent clogging was assumed in both weir and orifice flow. The grates were modeled at various sump depths to minimize grate area required, particularly in high flow interception conditions. Due to the hydraulic grade line of the finished OCCC conduit, it may be impossible to accommodate a grate inlet in a sump condition. In this case, other alternatives, including curb-to-curb grate inlets and combination curb and grate opening inlets, will be considered.

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description Existing McDowell Rd. Culvert Checked By ______ Date ______

1-10'x10' & 1-14'x10' Plus Additional Culvert Sheet ___ of ___

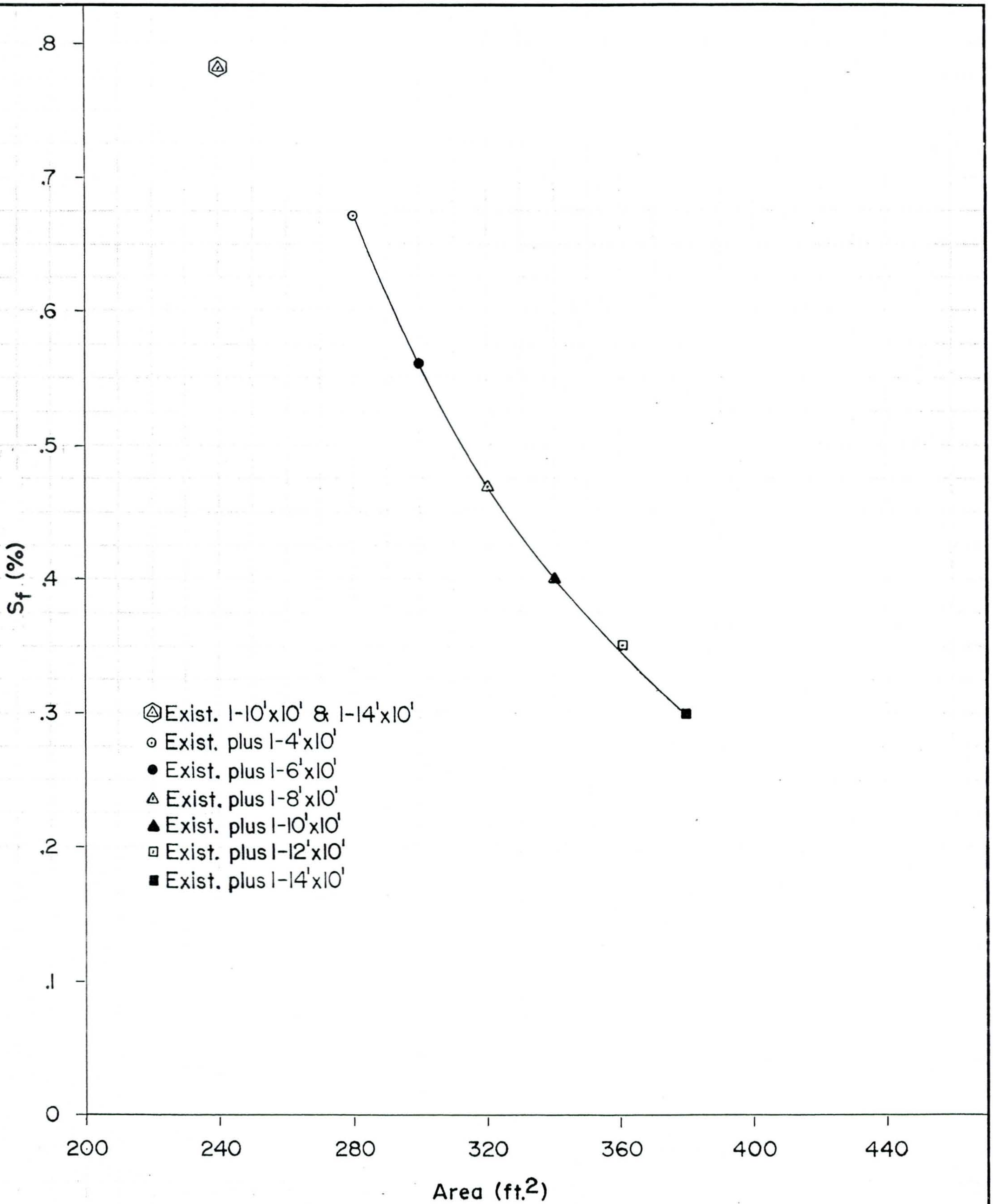


FIGURE 3.6-1

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description McDowell Rd. Culvert Checked By ______ Date ______

14'x10' Box, K_e vs. Headwater Sheet ___ of ___

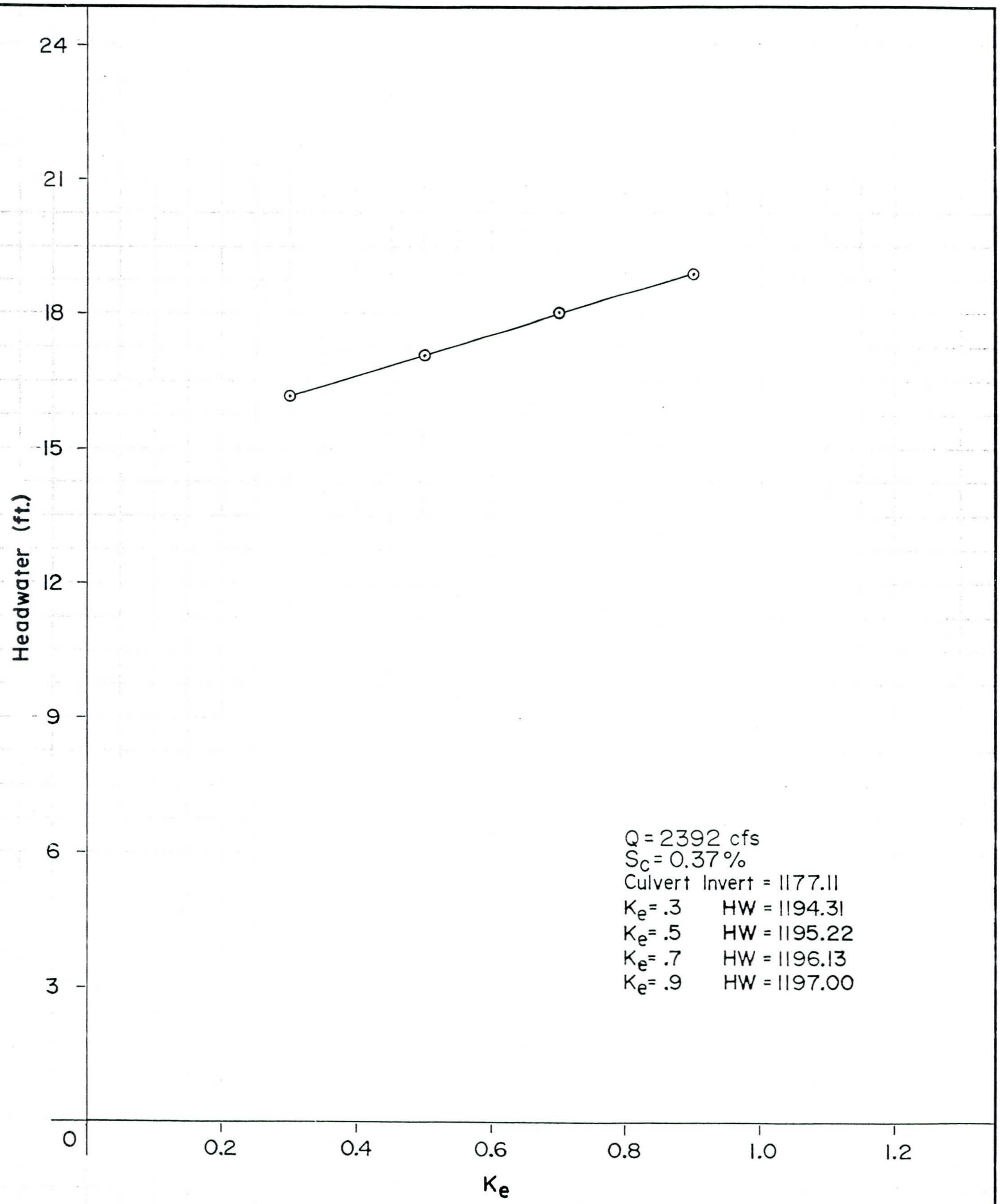


FIGURE 3.6-2

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description McDowell Rd. Culvert Checked By Date

14' x 10' Box; Slope vs. Depth/Energy Sheet of

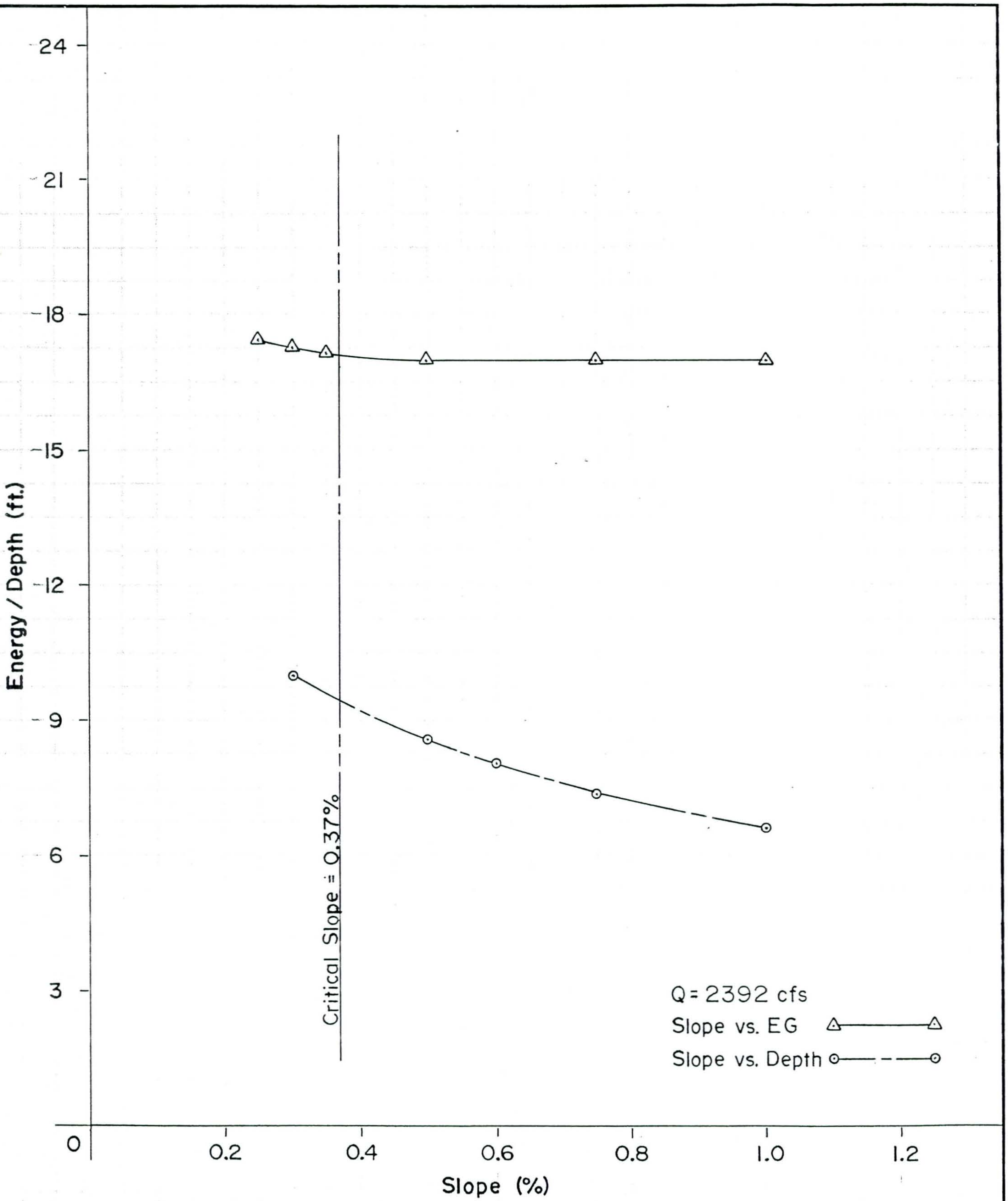


FIGURE 3.6-3

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description 14' Channel Bottom Checked By ______ Date ______

Slope vs. Depth & Slope vs. Energy Sheet ___ of ___

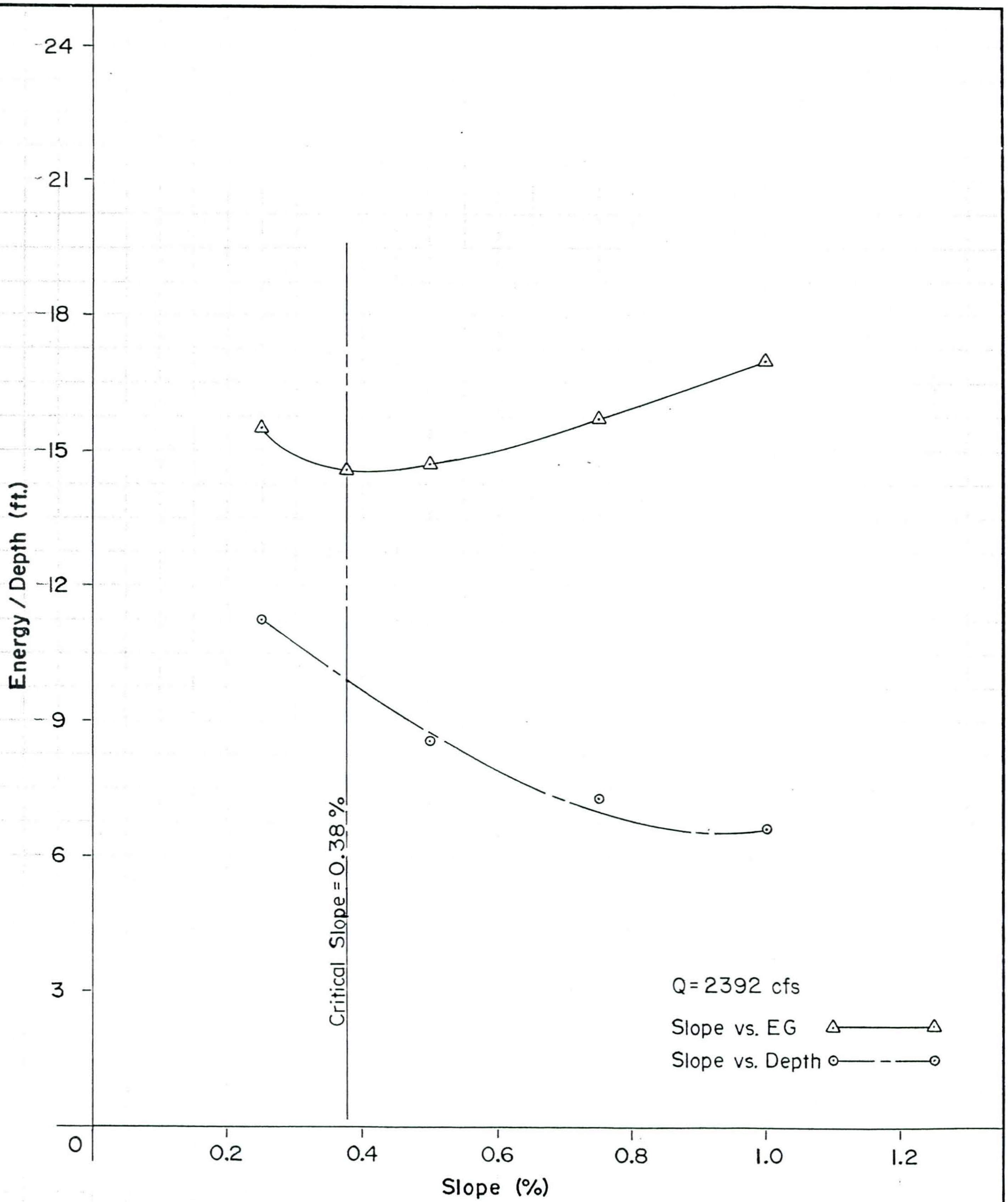


FIGURE 3.6-4

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description McDowell Rd. Culvert Checked By Date

10'x10' Box; Slope vs. Depth/Energy Sheet of

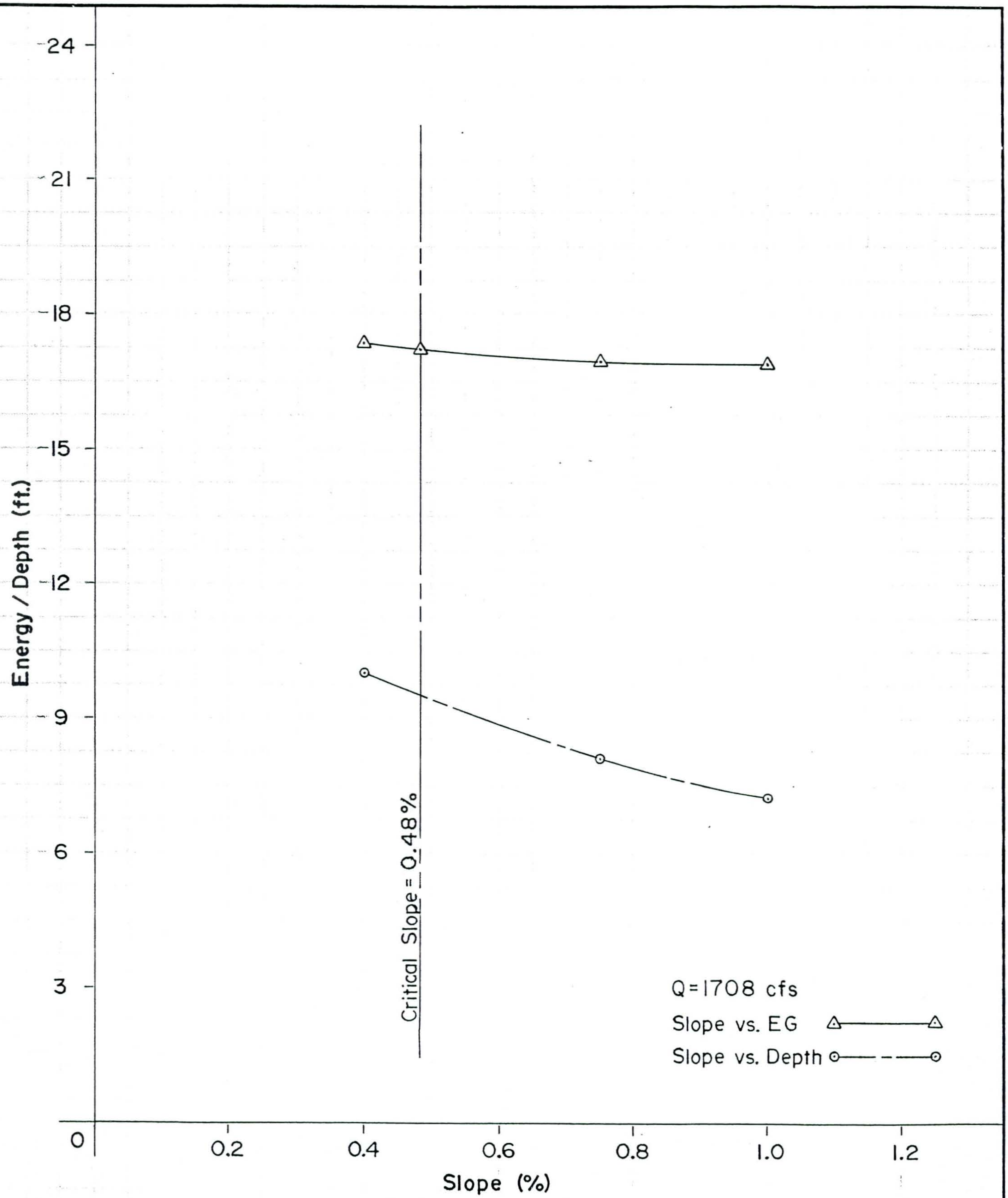


FIGURE 3.6-5

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description 10' Channel Bottom Checked By Date

Slope vs. Depth & Slope vs. Energy Sheet of

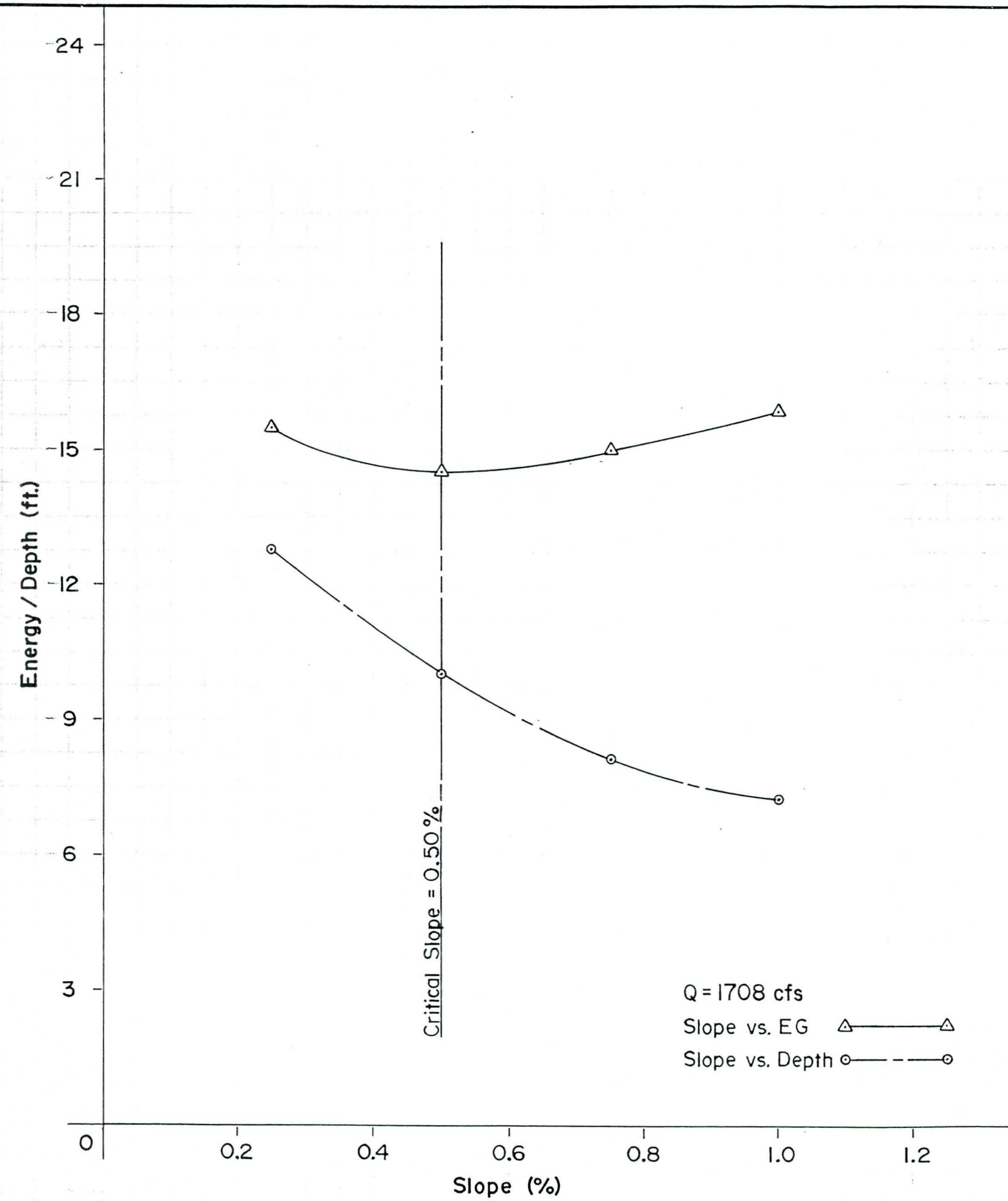


FIGURE 3.6-6

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description 16' Bottom Channel Checked By Date

Q = 2050 cfs, n = .015 Sheet of

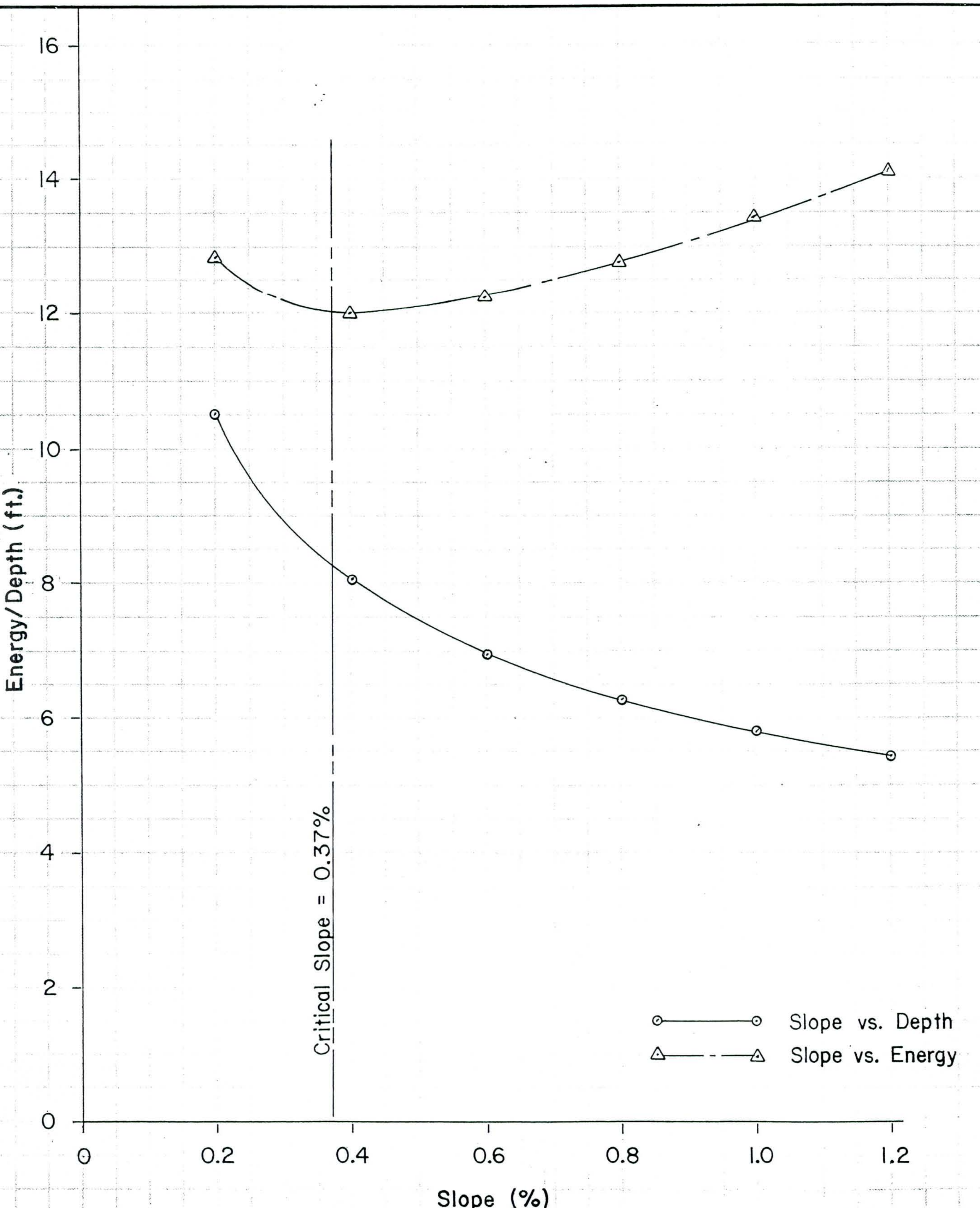


FIGURE 3.6-7

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description 18' Bottom Channel Checked By Date

Q=2050 cfs, n=.015 Sheet of

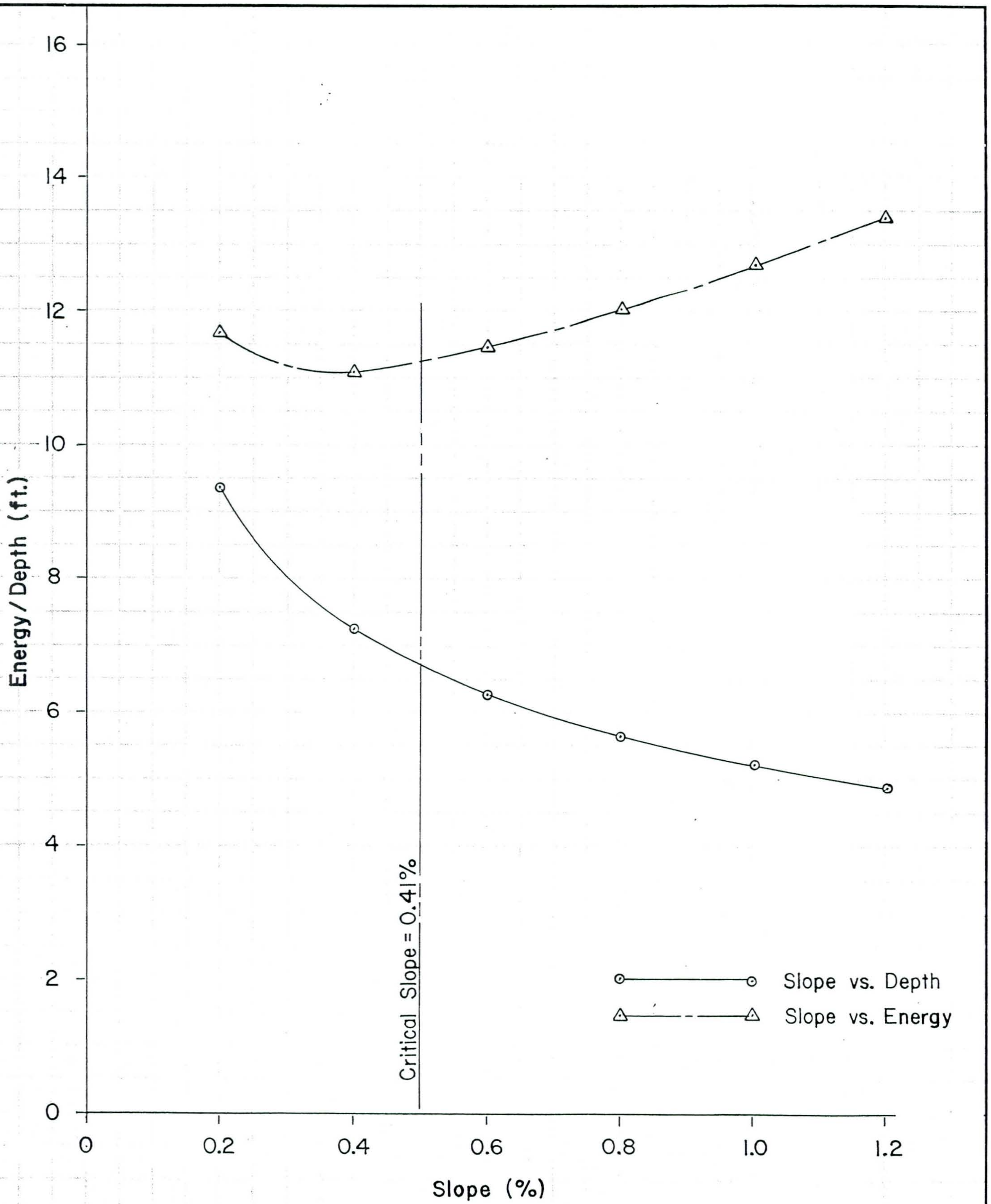


FIGURE 3.6-8

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description 26' Bottom Channel Checked By Date

Q=4100 cfs, n=.015 Sheet of

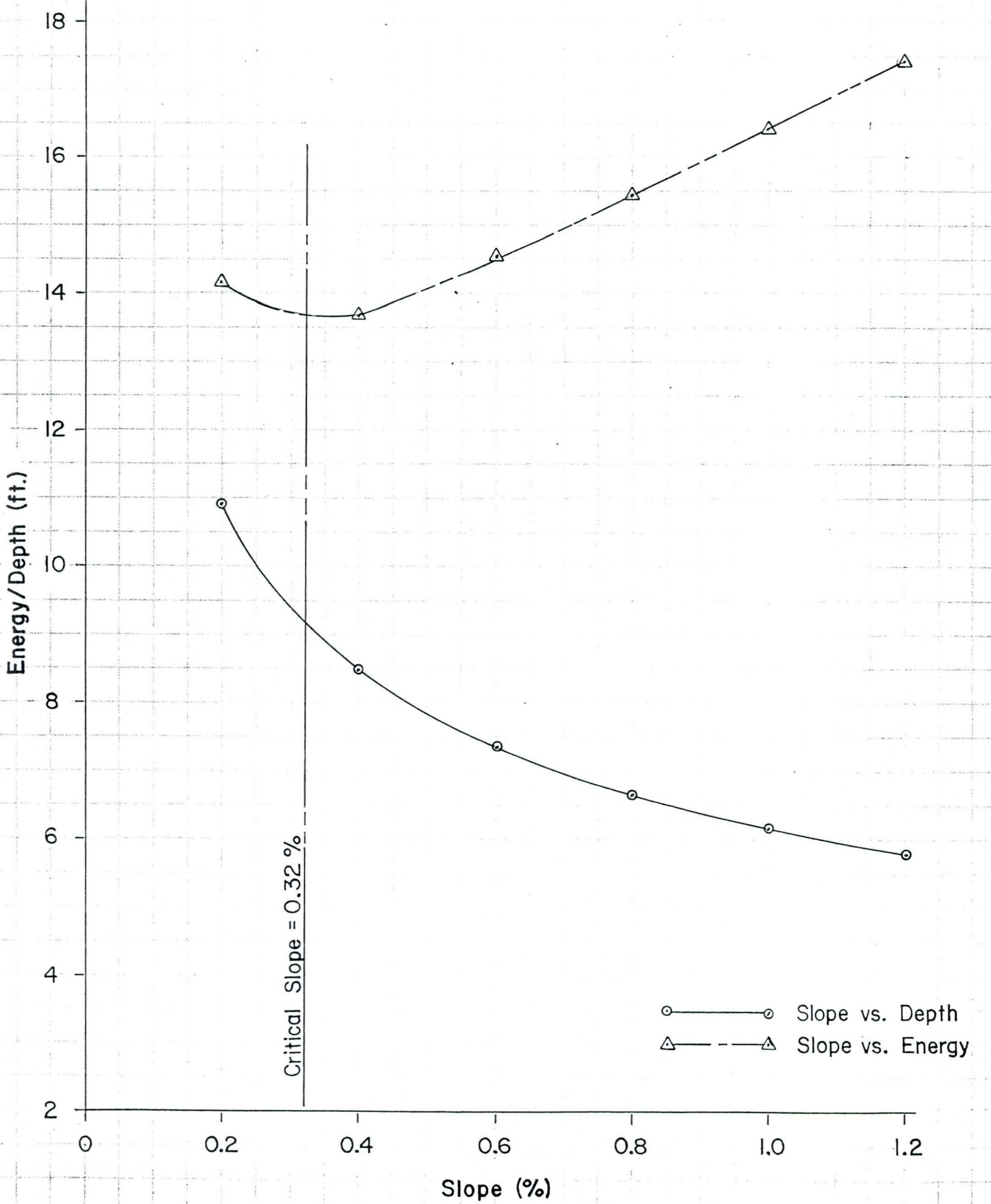


FIGURE 3.6-9

Greiner

Job Old Cross Cut Canal E002100	Computed By RHF	Date 2-01-91
Description McDowell Rd. D.S. Channel	Checked By	Date
BW=16', Q=4100 cfs, n=.013: Energy vs. Slope & Y_n vs. Slope		Sheet ___ of ___

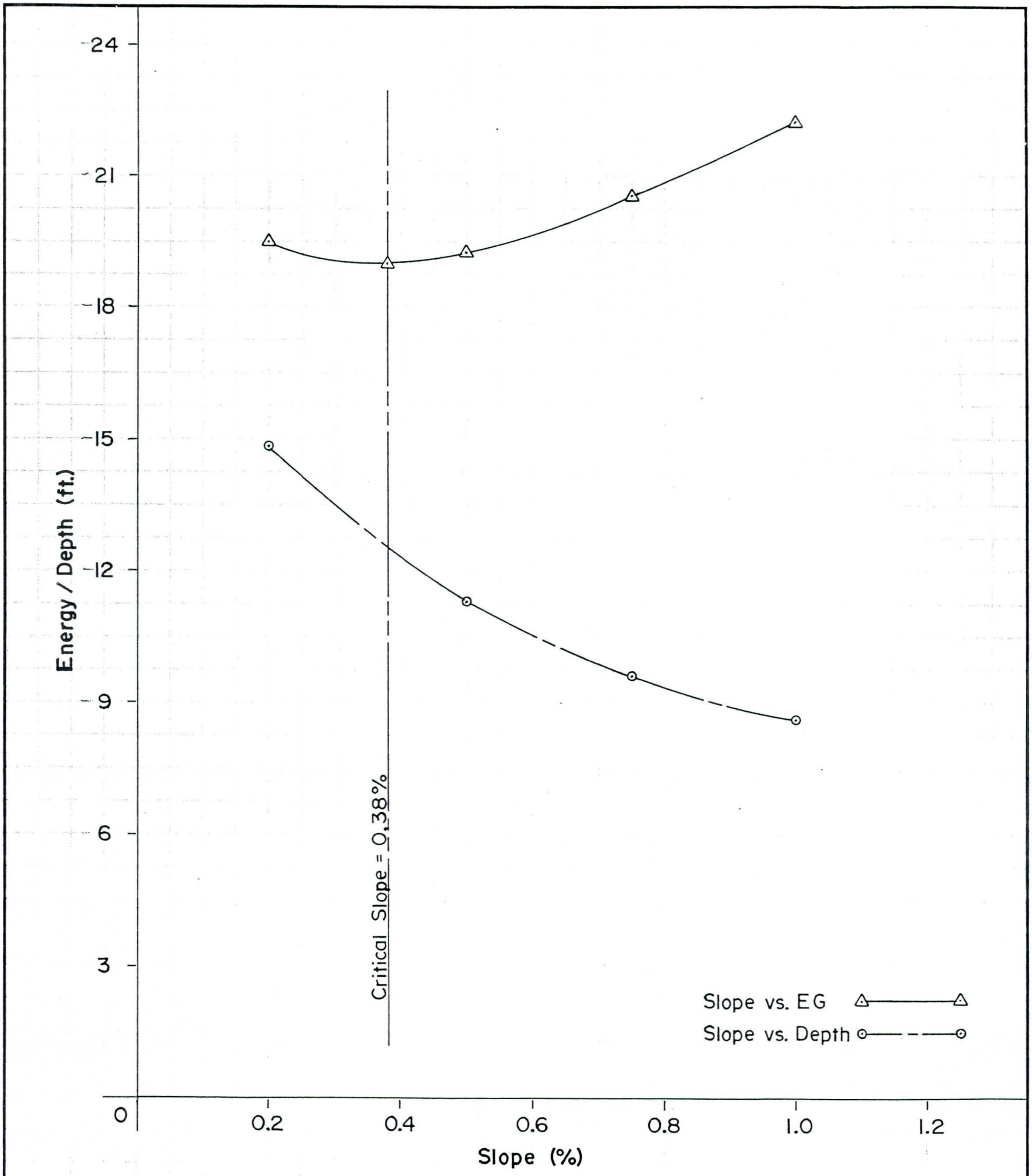


FIGURE 3.6-10

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description McDowell Rd. Culvert Checked By ______ Date ______

Down Stream Channel-BW=23.5', Q=4100, cfs, n=0.013 Sheet ___ of ___

Slope vs. Depth & Slope vs. Energy

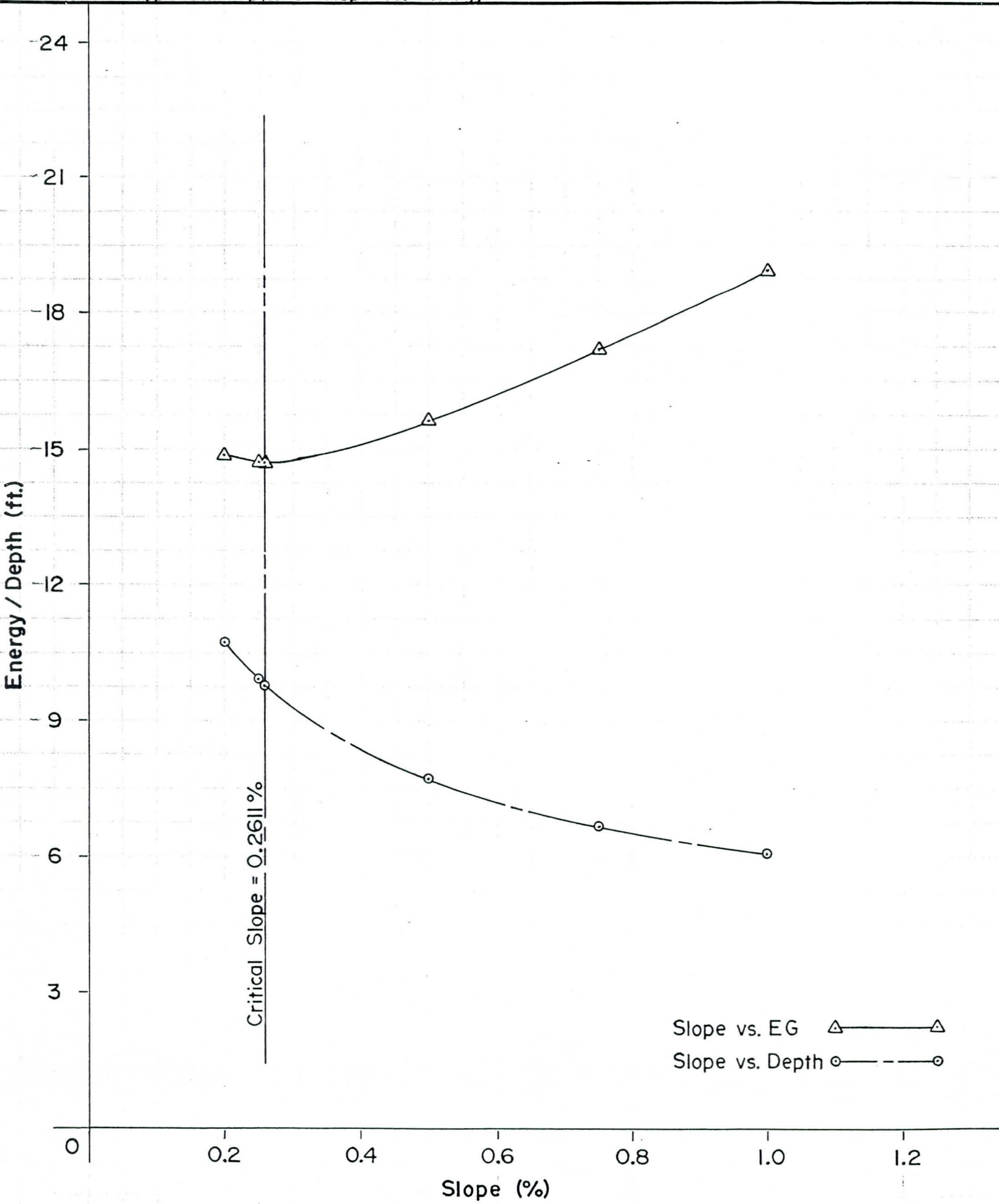


FIGURE 3.6-11

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 3-10-91

Description Transition Channel to Checked By Date

McDowell Rd. Culvert Q=4100 cfs n=.015 Sheet of

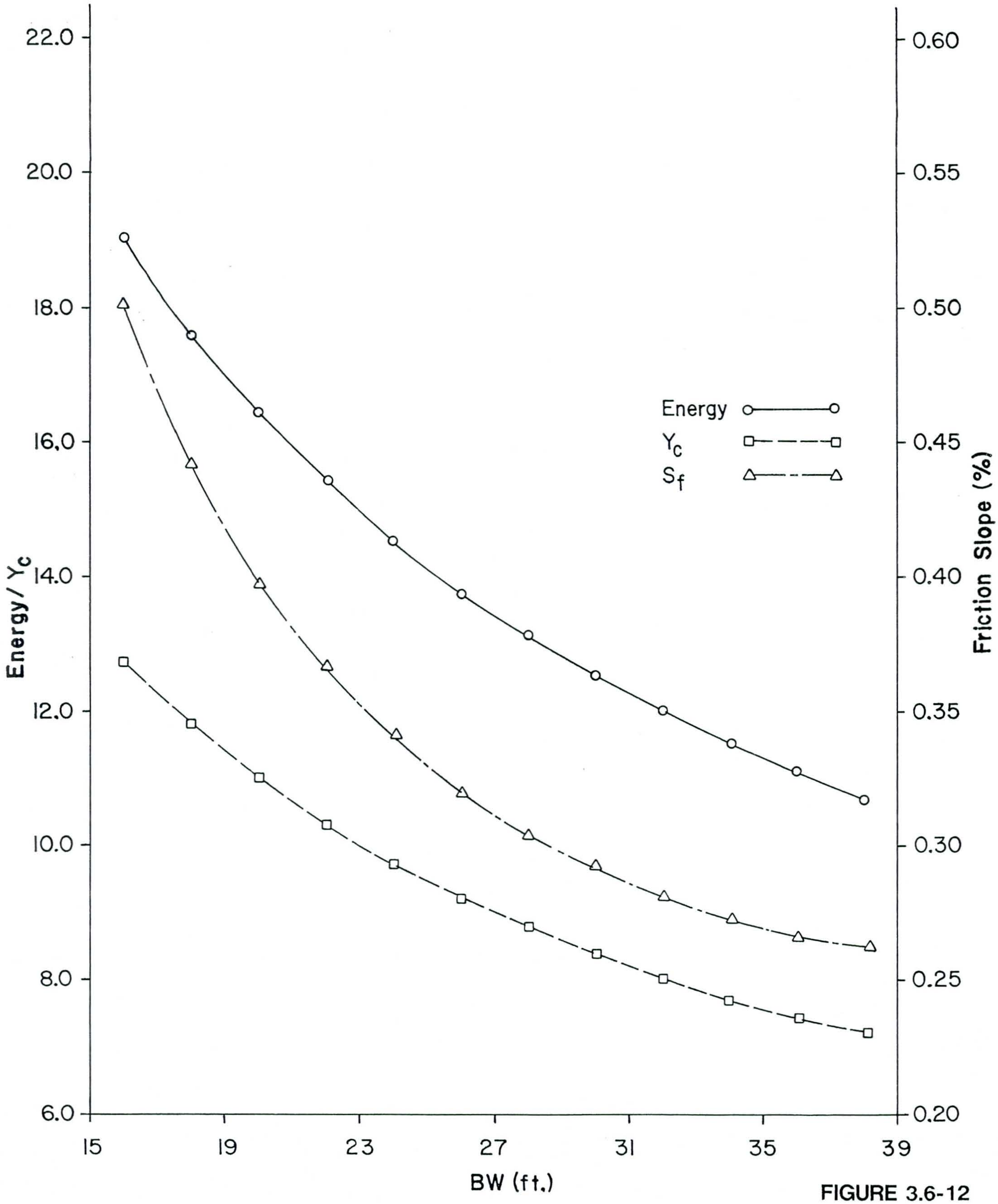


FIGURE 3.6-12

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description McDowell Rd. Culvert Discharge vs. Friction Slope

(Curve 1: 1-14' x 10' & 10' x 10')(Curve 2: 2-16' x 10') Sheet ___ of ___

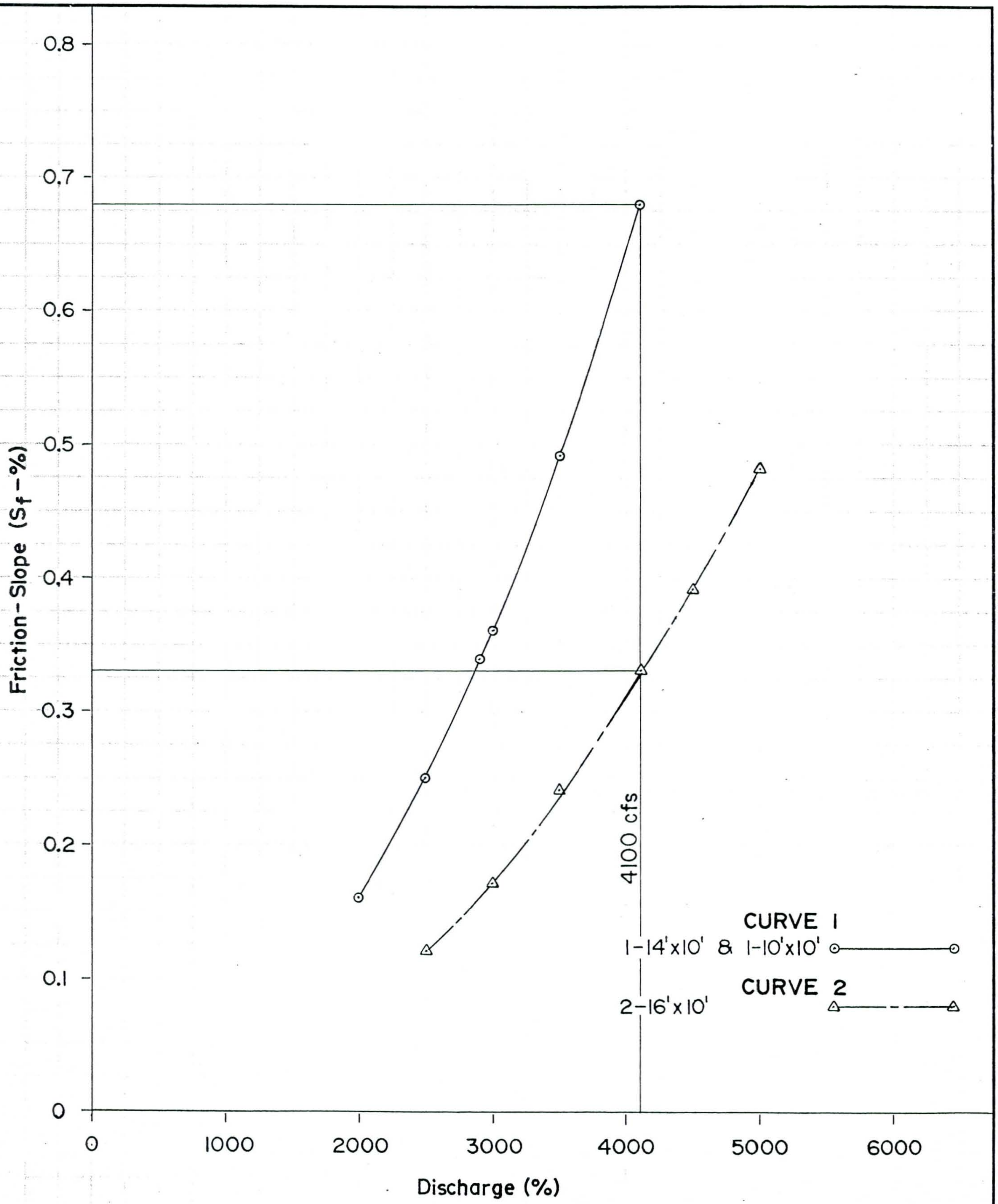


FIGURE 3.6-13

Greiner

Job Old Cross Cut Canal E002100 Computed By RHF Date 2-01-91

Description McDowell Rd. Culvert Checked By ______ Date ______

1-10'x10' & 1-14'x10' Plus Additional Culvert Sheet ___ of ___

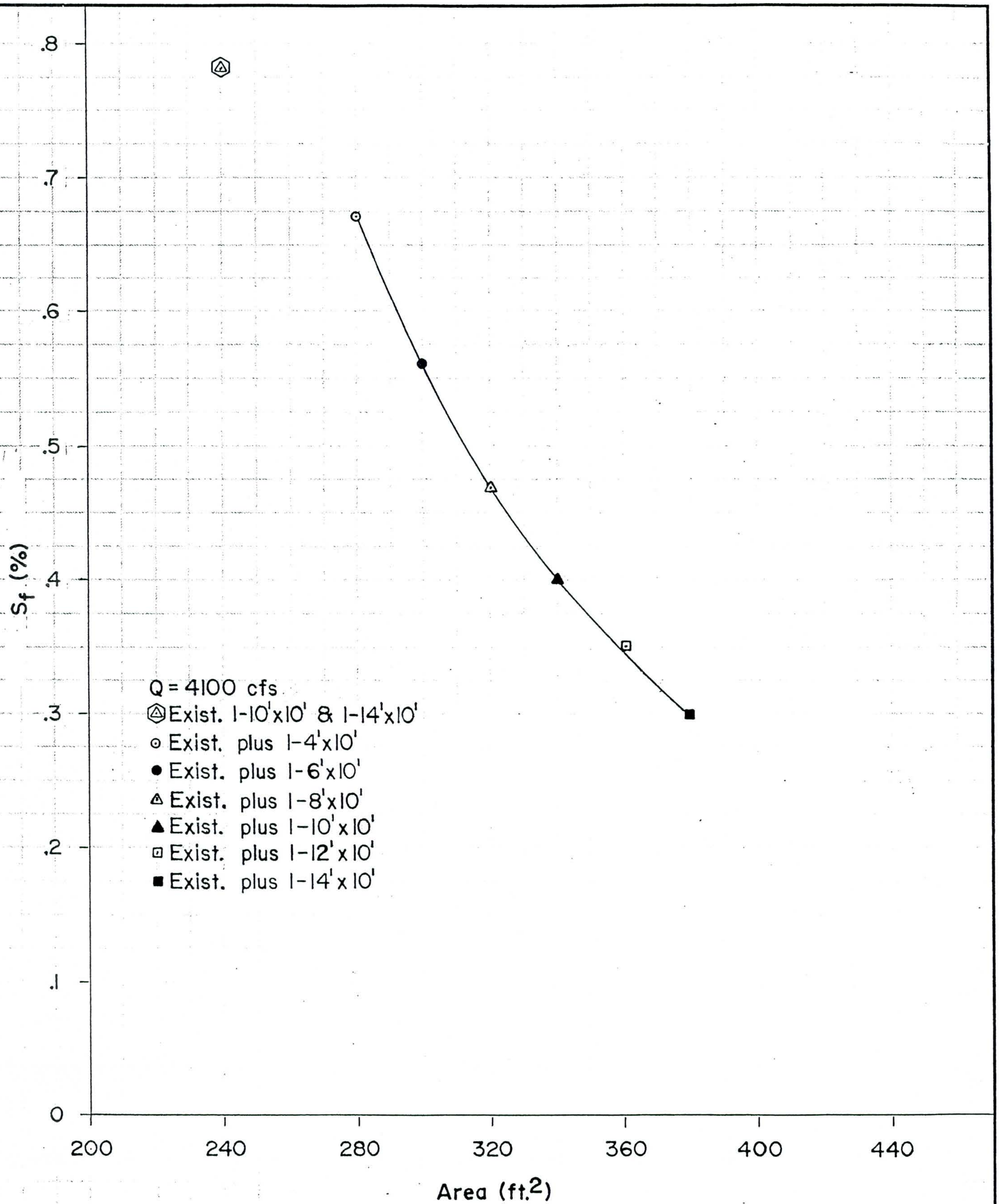


FIGURE 3.6-14

JUNCTION ANGLE VS. HEAD LOSS

OAK STREET
STA. 73+82±
MAINLINE – (2) 16'x10'
LATERAL LINE – (1) 10'x6'

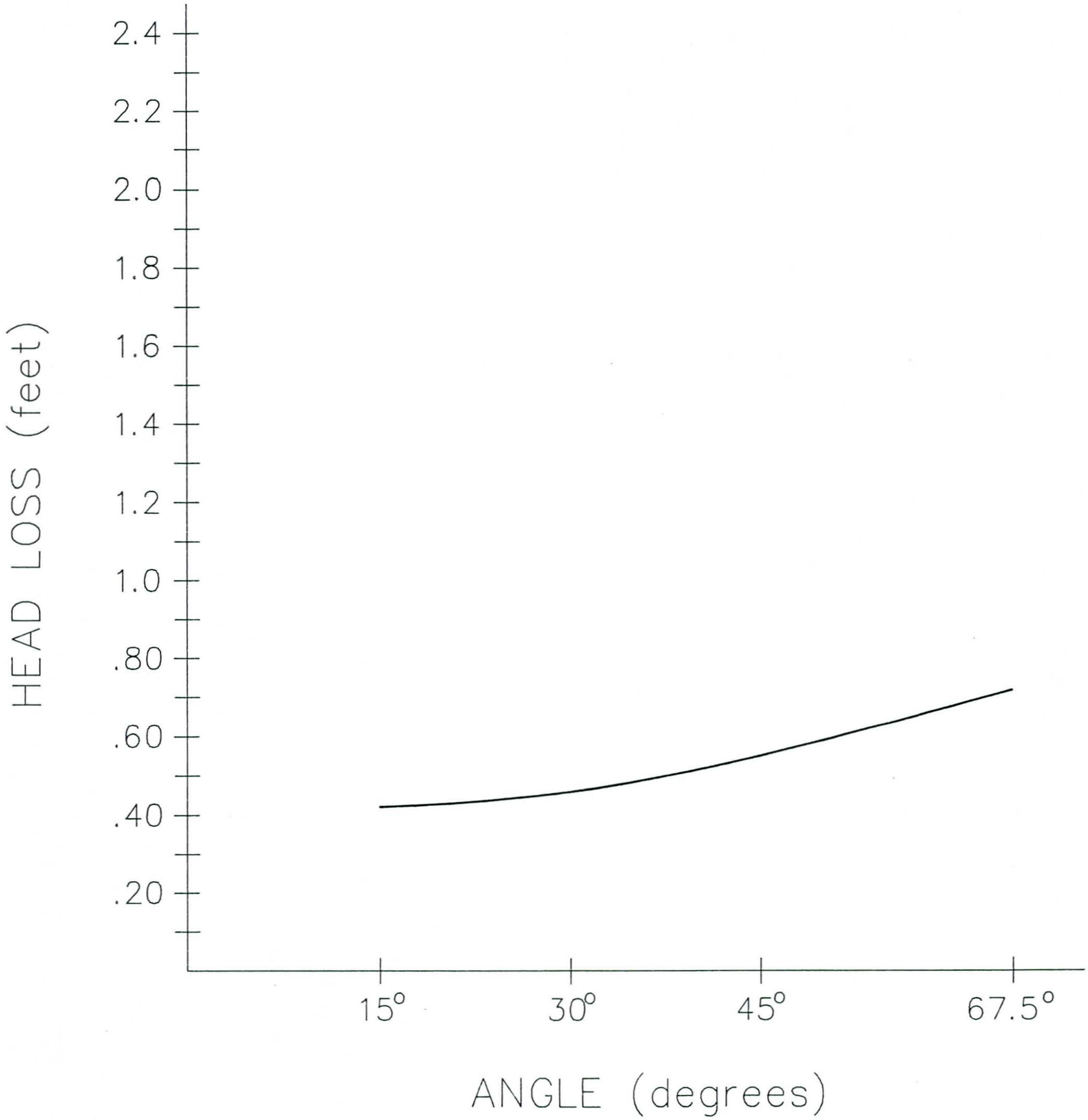


FIGURE 3.6-15

JUNCTION ANGLE VS. HEAD LOSS

OAK STREET
STA. 73+82±
MAINLINE – (2) 18'x10'
LATERAL LINE – (1) 10'x6'

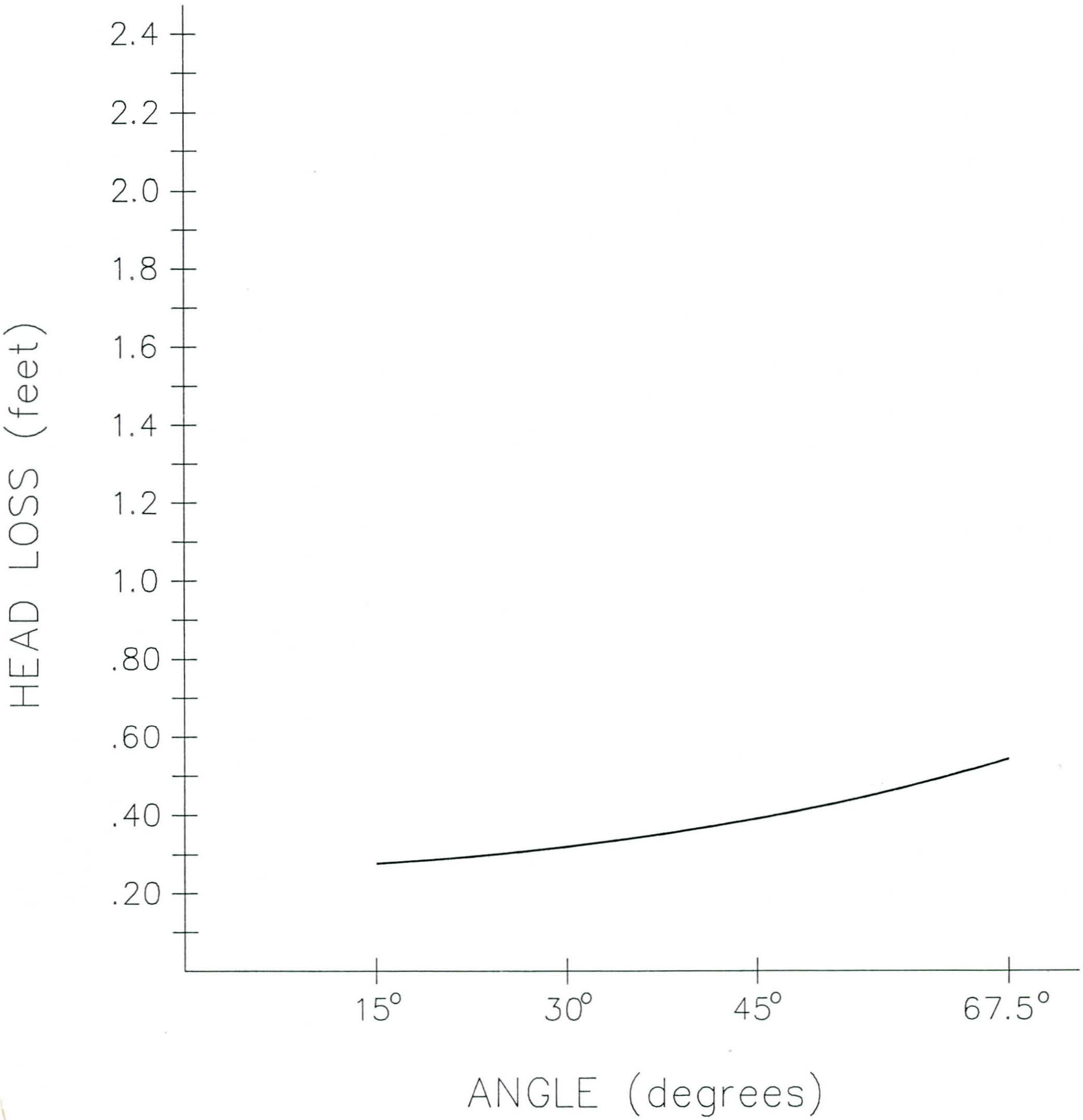


FIGURE 3.6-16

JUNCTION ANGLE VS. HEAD LOSS

VIRGINIA AVENUE
STA. 86+84
MAINLINE – (2) 16'x10'
LATERAL LINE – (1) 10'x5'

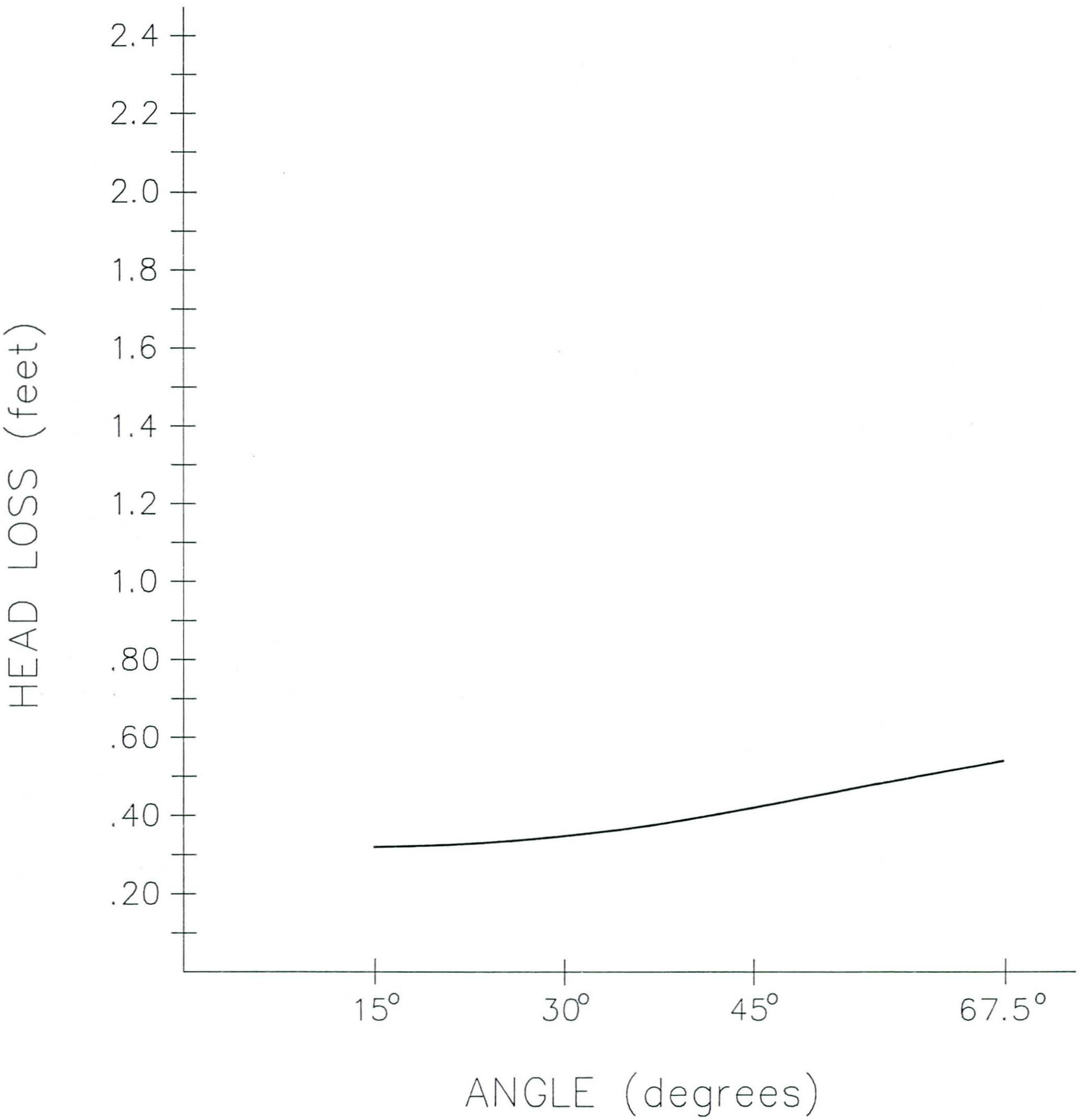


FIGURE 3.6-17

JUNCTION ANGLE VS. HEAD LOSS

VIRGINIA AVENUE
STA. 86+84
MAINLINE – (2) 18'x10'
LATERAL LINE – (1) 10'x5'

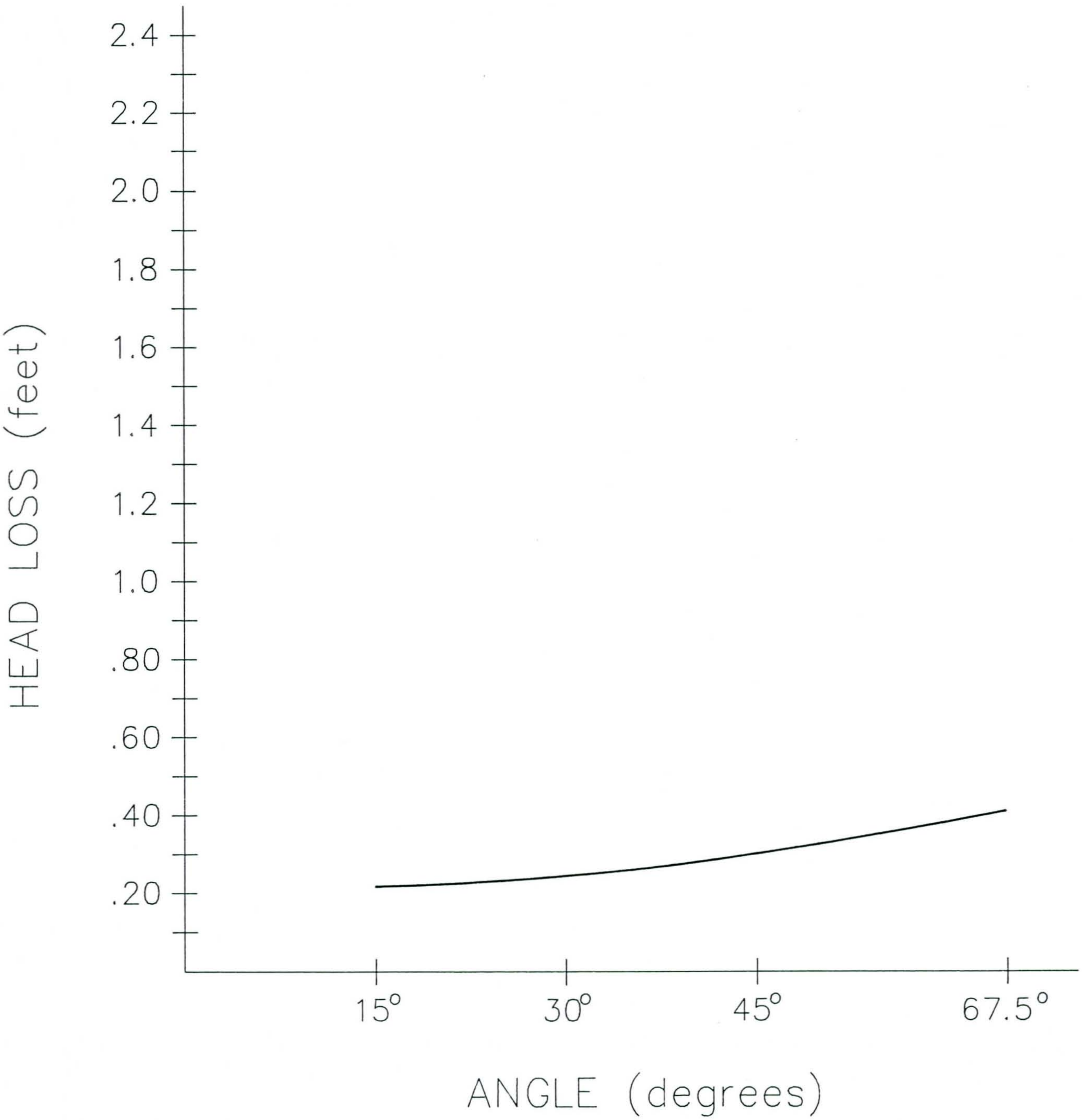


FIGURE 3.6-18

JUNCTION ANGLE VS. HEAD LOSS

EARLL DRIVE
STA. 113+19
MAINLINE – (2) 16'x10'
LATERAL LINE – (2) 10'x8'

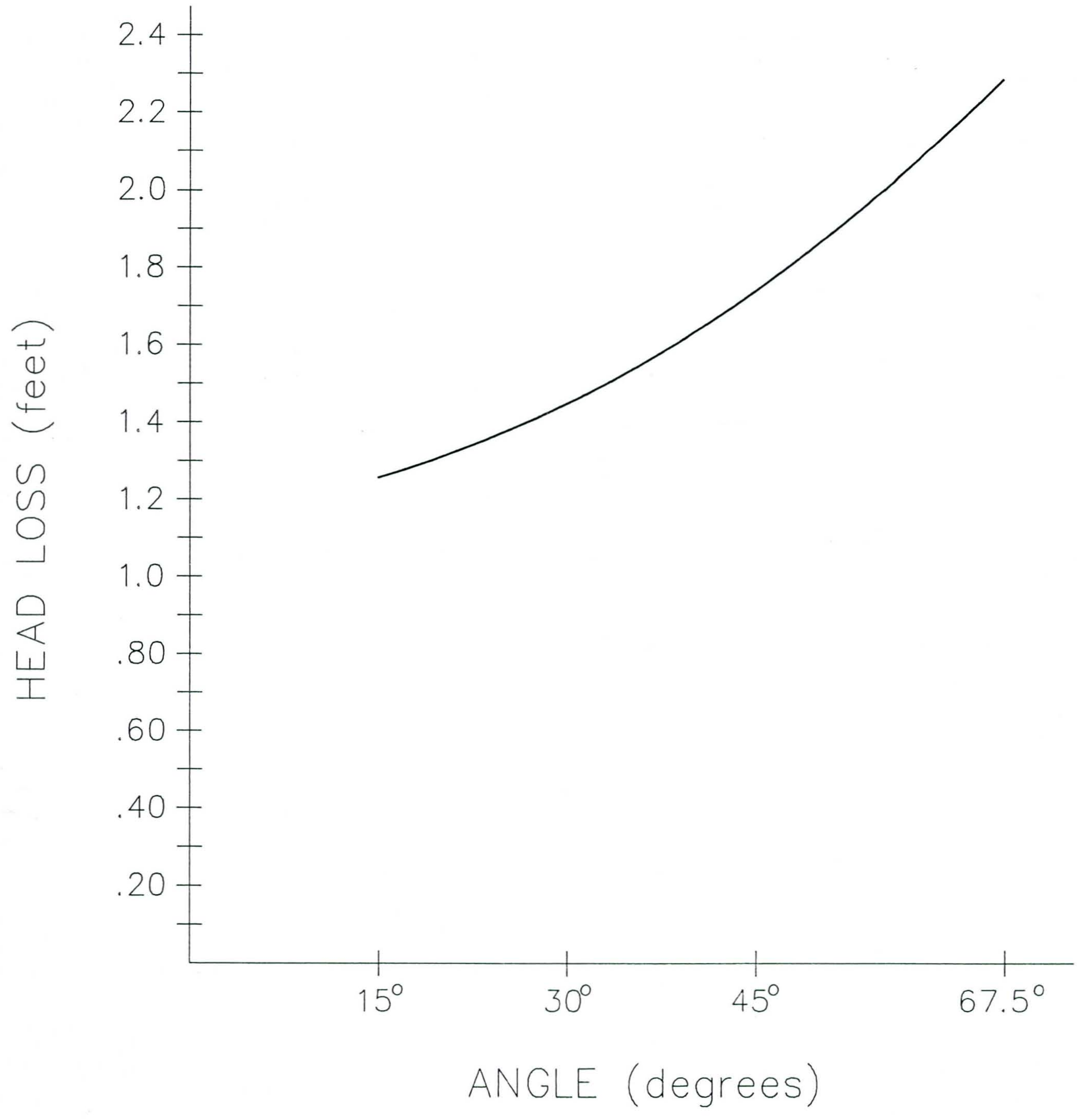


FIGURE 3.6-19

TABLE 3.1
PRELIMINARY SIDE INLET AND GRATE INLET SIZING

Cross Street	Peak Q	Sub-Out Length	Sub-Out Dia.	Grate Inlet Area Required
Indian School	24	80	24"	2-2' x 4'
Weldon	71	60	48"	2-3' x 5'
Whitton	9	45	18"	2' x 4'
Osborn	196	40	7' x 4' CBC	7-2' x 3'
Richardson	33	80	30"	2-2' x 2'
Earll	802	60	2-10' x 7' CBC	14-4' x 5'
Pinchot	19	95	24"	2' x 4'
Thomas	129	90	5' x 5' CBC	4' x 6'
Windsor	56	85	42"	2-2' x 4'
N. of Virginia	7	70	18"	2' x 3'
Virginia	300	85	10' x 6' CBC	10-2' x 4'
Oak	917	95	2-10' x 7' CBC	20-2' x 5'
Holly	88	75	48"	3-2' x 5'
Granada	81	60	48"	4-2' x 5'
McDowell	322	90	54"	10-2' x 5

4.0 SOIL AND GEOTECHNICAL

4.1 INTRODUCTION

This section is a copy of the Geotechnical Report prepared by Thomas-Hartig & Associates, Inc. under contract with Greiner, Inc.

This section presents the results of the geotechnical engineering services authorized on the site for the proposed OCCC from Indian School Road to McDowell Road in Phoenix, Arizona.

The purpose of these services is to determine the soil conditions at the locations indicated which, thereby, provide a basis for the design discussions and recommendations presented herein. Greiner, Inc. and Thomas-Hartig & Associates, Inc. should be notified if conditions other than described herein are encountered during construction.

The services performed provide an evaluation of the soils throughout the zone of significant foundation influence at selected locations. The field services have not included exploration for underlying geologic conditions or evaluation of potential geologic hazards such as seismic activity, faulting, or ground subsidence/cracking potential due to groundwater withdrawal, or the presence of contamination.

The recommendations presented in this report are based upon the project information described in "Scope," Part I. If the design conditions are changed substantially, Thomas-Hartig & Associates, Inc. shall be contacted for review.

4.2 REPORT AND FINDINGS

4.2.1 Scope

The proposed OCCC flood control improvements will eventually consists of a new culvert to carry the canal flow and improved roadways for the Hohokam Parkway and 48th Street. The project extends from McDowell Road north to the Arizona Canal along the OCCC. This phase of the project includes only the installation of the culvert. This report contains a description of our field operations, laboratory results and design recommendations concerning constructibility, excavations and slope stability, bearing capacity and lateral earth pressures, bedding and backfilling materials, and pavement thickness for City of Phoenix cross streets affected by this project.

4.2.2 Site Description

The OCCC in the project area from McDowell Road to the Arizona Canal consists of an open channel with undercrossings at Thomas Road, Osborn Road and Indian School Road. South of Osborn Road, the canal is bounded by an unpaved service road with a pedestrian walkway and bicycle path on the west bank, and by 48th Street on the east bank. North of Osborn Road, the canal is bounded on the west by 48th Street northbound and on the east by residential areas. The canal banks are typically unlined and steeply sloping. As of our field operations, the canal carried only low water flows south of Osborn Road.

4.2.3 Investigation

The field investigation included a site reconnaissance, subsurface exploration and field resistivity testing. The subsurface exploration consisted of drilling 22 test borings at the locations shown on the site plan in Appendix A. The test borings were drilled with a CME 55 drill rig using 7" hollow stem augers. The test borings were drilled to a depth of 25'. Standard Penetration Test (SPT) sampling and driven ring sampling was performed in all borings, alternating at 5' intervals, to obtain an indication of the relative density and/or consistency of the formation being penetrated and to obtain samples for laboratory testing. Where possible, bulk samples were obtained from the cuttings. Groundwater levels were noted during drilling, and in some test borings stabilized groundwater levels were measured in holes left temporarily open.

Piezometers for observing groundwater levels were constructed at Test Borings 3, 8, 13, 16 and 21. These piezometers will be monitored monthly until such time as the design plans are approved and accepted by the Flood Control District. The wells will then become the property and responsibility of the Flood Control District for subsequent monitoring and abandonment. We emphasize that the abandonment must be conducted by the Flood Control District in accordance with the policies and regulations of the Arizona Department of Water Resources (ADWR).

During the field investigation, the soils encountered were visually classified by our field engineer. The results of the test drilling conducted for this project are presented on the boring logs in Appendix A, "Field Results."

The soil resistivity was measured using a four-terminal "Megger Earth Tester" resistivity meter. The resistivity tests were conducted using two different electrode spacings to indicate the variation of soil resistance with depth. The resistivity values ranged from about 1,910 to 9,580 ohm-cm. The results of the field resistivity testing conducted for this project are presented in Appendix A, "Field Results."

4.2.4 Laboratory Investigation

Laboratory testing was conducted on representative soil samples obtained during the test drilling. The testing was conducted to obtain the data necessary to develop design recommendations for this project. The following tests were conducted:

Test	Sample(s)	Purpose
Sieve Analysis and Atterberg	Representative (22)	Classification and correlation engineering properties
Dry Density and Moisture Content	Undisturbed (51) Disturbed (55)*	In-situ density and moisture determination to correlate engineering properties
Direct Shear	Undisturbed (5)	Bearing capacity and slope stability analysis
Compression	Undisturbed (5)	Settlement analyses
Soluble Salts, Sulfates and Chlorides	Representative (5)	Corrosion potential
ASTM D698	Representative Grab Sample (5)	Compaction characteristics
R-Value	Representative Grab Sample (4)	Pavement design
Expansion	Compacted (2) Undisturbed (1)	Expansion potential

*Disturbed samples from SPT sampling tested for moisture content only.

The results of the moisture and density testing are presented on the graphical boring logs in Appendix A. The results of the remainder of the testing are presented in Appendix B.

4.2.5 Soil Conditions

The soil profile at the boring locations is presented on the graphical boring logs in Appendix A. The soil profile along the site consists of a medium dense to dense clayey sand/sandy clay deposit. The deposit is light brown to reddish brown, and contains varying amounts of gravel particles and gravelly lenses. The gravels consist predominantly of subangular to angular granite fragments. The material exhibits moderate to high plasticity. The degree of calcareous cementation varies from light to heavy, and generally increases with depth.

4.0 SOIL AND GEOTECHNICAL

A review of nearby projects in our files indicated that similar materials have been encountered along the alignment and for some distance on either side. Expansion potentials from nearby projects ranged from 0 to 4.6 percent on remolded samples from projects in the area, and from 0.3 to 1.4 percent from this project.

Soil moisture contents at the time of test drilling were generally described as damp to moist above the groundwater level. Groundwater was detected in most of the test borings, as shown on the test boring logs in Appendix A, at depths ranging from 12' to 25' below existing ground surface. These groundwater levels represent only the conditions encountered at the time of our field drilling operations. Groundwater levels may vary with time, seasonal conditions, and/or water flow in the OCCC.

4.2.6 Discussion and Recommendations

1. <u>General</u>: Geotechnical engineering recommendations are presented in the following sections. These recommendations are based upon the results of the field and laboratory testing which are presented in Appendices A and B of this report. Alternative recommendations may be possible and will be considered upon request.

2. <u>Expansion Potential</u>: Existing soils are sandy clays and clayey sands, predominantly of medium plasticity. At existing moisture conditions, the undisturbed soils will demonstrate moderately low potentials for expansion. However, compaction of these soils will further increase expansive potentials, especially if these soils are compacted to relatively high densities at moisture contents below optimum. Expansive potentials of new fills constructed in these soils are estimated on the order of 1/4" to 1/2" per foot of compacted fill. Additionally, significant swelling pressures could develop against culvert walls adjacent to compacted backfills. For this reason, imported granular soils exhibiting low expansive potentials are recommended for any backfills above the base of the excavation for the culvert installation.

3. <u>Culvert Support</u>: The culvert to be installed to convey the canal flow will be placed from 14' to 20' below ground. The soil along the canal is fairly strong, and the culvert will be lighter than the soil it replaces. Therefore, we anticipate low settlements of less than 1/2" with an allowable bearing capacity of 5,000 psf. Two feet of granular fill should be provided below the bottom of the culvert, as described in Site Grading later in this report.

   Because of the shallow groundwater level along the alignment, allowance must be made to prevent buoyant uplift under the condition of high groundwater when the culvert is empty or near empty. A minimum 4' soil cover over the top of the culvert will be sufficient to prevent such uplift.

4. Lateral Design Parameters: The following tabulation presents recommendations for lateral earth pressures expected against buried culvert structures:

Lateral Backfill Pressures:

Above Groundwater Table	60 psf/ft.
Below Groundwater Table	95 psf/ft.

These pressures are equivalent fluid pressures for vertical walls and horizontal backfill surfaces (maximum 12' height). Pressures do not include temporary forces imposed during compaction of the backfill, swelling pressures developed by over-compacted clayey backfill, or surcharge loads. Walls should be suitably braced during backfilling to prevent damage and excessive deflection. We recommend that only manual compaction equipment be used within 5' of culvert walls.

5. Cross Street Pavements: Pavement reconstruction will be required over the culvert installation at Thomas Road, Osborn Road and Indian School Road. Based on discussions with the City of Phoenix Materials staff, we recommend that an 8" thick, full depth asphalt concrete section be used, unless the existing pavements are thicker. Thickness of existing pavement was checked on as-built drawings for the cross streets. The final recommended pavement thicknesses are tabulated below:

Cross Street	Full Depth Asphalt Concrete (Inches)
Thomas Road	8-1/2
Osborn Road	8
Indian School Road	11

Pavement materials should not be placed when the subgrade is wet. The surface should be sealed after weathering is apparent to minimize water infiltration directly through the pavement section and retard oxidation.

6. Excavation Conditions: The test drilling and field sampling at the site were performed for design purposes. It is not possible to accurately correlate auger drilling results with the ease or difficulty of digging for various types and sizes of excavation equipment. We present the following general comments regarding excavatability for the designers' information with the understanding that they are approximations based only on test boring data. More accurate information regarding excavatability should be evaluated by contractors or other interested parties from test excavations using the intended equipment.

The near surface soils are non-cemented to weakly cemented natural soil deposits which can probably be removed with conventional excavating equipment. However, variable carbonate cementation (caliche) was encountered in some locations, typically below about 4', and excavations into these deeper soils could be more difficult. All excavations should be braced or sloped to provide personnel safety and satisfy local safety code regulations. We recommend temporary cut slopes at 1:1 (horizontal:vertical) for the upper 8' and 1/2:1 (horizontal:vertical) for lower portions of the excavation. The excavation will probably encounter groundwater for much of its length.

7. Site Soil Workability: Below the culvert bottom, the moisture content of existing site soils should be maintained between optimum and optimum +3 percent (ASTM D698) during and subsequent to site grading to reduce expansive potentials. At these conditions, some pumping may be experienced under dynamic loading if the compaction is done by very heavy equipment, i.e., loaded scrapers, water-pulls, etc. We would not consider some pumping detrimental in areas below the culvert bottom (i.e., static loading conditions) provided special densities are obtained. Lighter compaction equipment and/or drying of wet soils may be used to reduce pumping if this condition becomes severe.

   In bituminous paved areas, the moisture content of the subgrade and backfill should be maintained at 2 percent below optimum or lower during site grading to reduce the potential for pumping. If in-situ moisture contents are higher than this at the time of construction, pumping may occur, and special precautions should be taken to prevent disturbance, equipment mobility problems and loss of shear strength in the subgrade. These precautions may include spreading and drying of wet soils, removal and replacement of wet soils, construction of temporary gravel roads at channelized traffic areas, and/or use of lighter compaction equipment.

   Because of the shallow groundwater conditions encountered in many of the test borings, the use of a dewatering system will be required during construction. A dewatering scheme we believe to be acceptable would consist of 2' of wash gravel below culvert grade and the employment of pumped drainage sumps to remove accumulating water. The recommendations of material and grading requirements to follow are based upon this system. The geotechnical engineer should be contacted for review of alternative drainage and/or bedding concepts.

4.3 MATERIALS

4.3.1 Fill Materials

All fill materials should be soils free of vegetation, debris, organic contaminants and fragments larger than 6"* in size. The existing site surface soils become moderately expansive when compacted. Therefore, these soils should not be used for backfill against the sides of the culverts, but may be used as backfill above the top of the culverts. All backfills against the side of the culverts should be of imported soils with low expansive potentials.

Backfill materials against culvert sides should be imported granular soils conforming to the following specification requirements:

Maximum Particle Size	6"*
Maximum Percent Expansion	1.5**
Maximum Percent Passing No. 200 Sieve	25***
Maximum Plasticity Index	5***

* Maximum size may be reduced at engineer's direction to satisfy trenching and landscaping requirements, etc.

** Performed on sample remolded to 95 percent of the maximum ASTM D698 density and 2 percent below optimum moisture under a 100 psf surcharge pressure.

*** Required for deep fills or culvert backfills where the fill thickness is greater than 4'.

Two feet of granular material should be provided beneath the bottom of the culverts. This material will provide a working surface for placing precast culverts or forming cast-in-place culverts and a drainage layer for controlling shallow groundwater along the alignment. We recommend a clean wash gravel with 100 percent passing the 1" sieve.

4.3.2 Pavement

Pavement materials should be in accordance with the requirements of the Maricopa Association of Governments Standard Specifications for Asphalt Concrete (Section 710, Type C-3/4).

4.4 EXECUTION

4.4.1 Site Grading

The following recommendations are presented for grading and excavation along the culvert alignment. All phases of earthwork should be performed under observation and testing directed by the geotechnical engineer.

1. Excavation should be performed with as little disturbance to the base of the excavation as possible. We recommend the use of a bucket from above the sides of the excavation instead of a front-end loader from within the excavation. The base of excavation should be at least 2' below the bottom of the culverts.

2. The shallow groundwater conditions along most of the alignment will require the employment of a dewatering system. We recommend the use of a sump system within the excavation.

3. With the water level at or below the base of the excavation, the base should be cleaned of all organic contaminants, debris, utilities or subsurface facility remnants and any loose or disturbed soils encountered. The cleaned surface should be observed for evidences of debris laden soils, disturbance, concealed facility remnants, or loose zones requiring additional removal. The sides should be braced or sloped in accordance with the recommendations under "Excavation Conditions."

4. Place the granular backfill previously described under "Fill Materials" at the base of the excavation to bring the excavation back to the level of the bottom of the culverts.

5. With the water level at or below the top of the granular backfill, construct cast-in-place culverts or place precast culverts.

6. Backfill against the sides of the culverts with imported fill materials as previously described under "Fill Materials." We recommend that only manual compaction equipment be used within 5' of the culvert walls. Backfill should be placed and compacted in horizontal lifts of thicknesses compatible with the compaction equipment used.

7. Natural site soils may be placed above the top of the culverts in non-pavement areas. Where paved surfaces are to be placed over the excavation, imported fill soils should be used. All backfill should be placed and compacted in horizontal lifts of thickness compatible with the compaction equipment used.

8. Compaction of cleaned exposed soil, backfill and granular bedding materials should be accomplished to the following density criteria:

Material	Percent Compaction (ASTM D698)
Exposed Soil Below Base of Culvert	95 Min.
Granular Bedding Below Culvert	100 Min.
Imported Backfill Against Culvert Walls	100 Min.
Backfill Above Culvert Top	
Non-Paved Areas	90 Min.
Paved Areas	95 Min.

Compaction of exposed site soil beneath the base of the excavation should be performed with soils uniformly mixed at a moisture content between optimum and optimum +3 percent. Compaction of imported fill soils with low expansive potentials should be accomplished within the range of optimum moisture content -1 to +3 percent. Compaction of exposed soil and fill material below asphaltic pavement should be accomplished at a moisture content 2 percent below optimum, or lower.

Natural undisturbed soils or compacted soils subsequently disturbed or removed by construction operations should be replaced with materials compacted as specified above.

4.4.2 Paving

Placement requirements for paving should be in accordance with the Maricopa Association of Governments' Specifications for Asphalt Concrete Pavement (Section 321). Observation and testing should be performed as necessary to verify conformance with these recommended specifications, especially compaction requirements for asphaltic concrete surfacing.

4.5 SUPPLEMENT

The purpose of this supplement is to provide responses to comments from the Flood Control District of Maricopa County related in a memorandum from Don Rerick dated February 26, 1991.

4.5.1 Recommendation for Shoring Limits Where Right-of-Way is Tight

Excavation bracing may be required in some areas where utility conflicts or right-of-way limitations exist. Cantilever or tied-back systems may be used at the discretion of the contractor. The contractor should submit a design sealed by a registered professional engineer for required bracing to the geotechnical engineer for review.

4.5.2 Rebar Protection Requirements

Based on pH measurements reported in Appendix B of our report, the soil is an alkaline environment. Field resistivity measurements reported in Appendix A ranged from 1,910 to 9,580 ohm-cm. City of Phoenix Administrative Procedure No. 13 requires corrosion protection for rebar when resistivity measurements are less than 1,500 ohm-cm. Based upon this criterion and our measured results, no corrosion protection for the reinforcing steel is indicated.

4.5.3 Closure of the Piezometer Wells

Abandonment of piezometer wells must be carried out in accordance with the requirements of ADWR regulations. We refer special attention to Article 8, "Well Construction and Licensing of Well Drillers." The following points are extracted for emphasis:

1. According to R12-15-803, Subpart B, the abandonment must be conducted by a well drilling contractor licensed in accordance with R-12-15-804, -805 and -806.

2. According to R12-15-816, Subpart B, a "Notice of Intent to Abandon a Well" must be filed prior to abandoning the piezometer.

3. The ADWR will mail an abandonment authorization to the well drilling contractor after they receive the notice above. The contractor may not proceed without that authorization.

4. After abandonment, the Flood Control District must complete and file a "Well Abandonment Completion Report" within 30 days.

Abandonment procedures are described in R12-15-817, Subparts G, H, I, J and K. Those wells within the excavation must be abandoned for those portions below the bottom of the excavation. To avoid losing the well after excavation, we recommend that the wells be grouted prior to excavation to at least 5' above the bottom of the excavation. The remainder of the well can then be dismantled and removed as construction proceeds. The casings, or at least the top 2' of the casings, should be removed at all piezometers not within the excavation. The well should then be grouted to within 2' of the ground surface and backfilled with soil to the ground surface.

These comments are intended to amplify or point out the specific requirements of the ADWR regulations. These comments do not replace or supersede those regulations in any way.

5.0 IMPROVEMENT ALTERNATIVES FORMULATION

5.1 BASE CONDITION/PRELIMINARY HORIZONTAL AND VERTICAL ALIGNMENT

Base condition is formulated with the following considerations:

- o Compiling and analyzing data developed in Section 3.0, Hydraulics
- o Requirements of the geotechnical findings in Section 4.0, Soil and Geotechnical
- o A layout that may produce minimum earthwork
- o Least disruption during construction - minimize utility relocation needs

5.1.1 Preliminary Horizontal and Vertical Alignment

The preliminary horizontal alignment was established to follow the existing OCCC centerline. The preliminary vertical grades were set to follow the existing top of bank grade with a 4' minimum soil cover.

5.1.2 System Configuration

It is evident that the existing culverts at Thomas Road, Osborn Road and Indian School Road do not have the capacity for the design flow and are all out of the vertical grade requirement. These three structures shall be replaced.

Initial set-up as the base condition for the drainage improvements may include:

1. Existing McDowell Road culverts

2. Transition Structure No. 1, McDowell Road culvert to Segment 1 culvert, width transition from 26' to 37'

3. Segment 1 culvert, from McDowell Road to Avalon Drive, 2-18' x 10' box culverts, 6,650' long at a slope of .23 percent (Transition Structure No. 2 and Segment 2 culvert are the same size as the Segment 1 culvert, and are included as part of Segment 1). The slope of 0.23 percent was used to avoid the relocation of the sewer line north of Thomas Road.

4. Transition Structure No. 3, Segment 1 culvert to Segment 3 culvert width transitions from 2-18' x 10' to 2-12' x 10'

5. Segment 3 culvert, from Avalon Drive to Whitton Avenue, 2-12' x 10' box culverts, 2,200 feet long at a slope of 0.47 percent

6. Transition Structure No. 4, Segment 3 culvert to Segment 4 culvert width transitions from 2-12' x 10' to 2-10' x 10'

7. Segment 4 culvert, from Whitton Avenue to the Arizona Canal, 2-10' x 10' box culverts, 3,000' long at a slope of 0.87 percent

8. Transition Structure No. 5 at the Arizona Canal - this includes connection to the Segment 5 culvert and a drop inlet with energy dissipator features for the Arizona Canal flood relief gates

9. Segment 5 culvert, from the Arizona Canal to Lafayette drain, a single 10' x 10' box culvert. (This segment will be designed as gravity flow, not a siphon. Design of this segment has a made provision for future Lafayette drain connection.)

5.2 UTILITY CONFLICTS

Existing utilities within the project corridor were identified, inventoried and investigated for possible conflicts with the proposed improvements.

A main concern in the development of the horizontal and vertical alignments for the new conduit was avoiding major existing utilities. This information is presented in Section 5.4 of this report.

Representatives of the various utility companies have been contacted and informed of possible conflicts and relocations associated with the development of this project.

5.2.1 Utility Inventory

The following is a summary of existing utilities within the project corridor. See Figures 5.2-1, 5.2-2 and 5.2-3 for approximate utility locations.

1. Southwest Gas

   A. 8" line parallels centerline of 48th Street from beginning of the project, Station 13+40 to Station 15+80 at 14' to 9' east of 48th Street centerline.

   B. 8" line angles to the east beginning at Station 15+80 to Station 24+95, from 9' east of 48th Street centerline to 72' east of 48th Street centerline, or 28$\pm$ west of the east right-of-way line.

   C. 8" line runs 28' west of and parallel with the east right-of-way of 48th Street from Station 24+95 to 62+26, Thomas Road.

   D. 4" line crosses OCCC at Station 62+62, 36' north of the Thomas Road centerline.

E. 8" line runs along centerline of 48th Street right-of-way from Station 62+62, Thomas Road, to Station 87+90 (49' east of the west right-of-way line).

F. 8" line jogs to 34' east of 48th Street west right-of-way line at Station 88+60 to Station 115+50 (15' north of Indian School Road centerline).

G. 4" line crosses 48th Street and OCCC at Station 115+75 (Indian School Road).

H. 8" line angles to the north/northeast from Station 115+75 for 400'.

2. SRP Electrical (Overhead)

A. OHE parallels the 48th Street right-of-way from project beginning crossing 48th Street at Station 19+30 then paralleling east side of Cross Cut Canal to Station 59+30 (south of Thomas Road).

B. OHE crosses 48th Street at Station 10+70 (north of McDowell Road).

C. OHE crosses 48th Street at Station 12+65 (north of McDowell Road).

D. OHE crosses 48th Street at Station 13+25 (north of McDowell Road).

E. OHE parallels the west side of the OCCC west access road from project beginning at Station 23+10.

F. OHE parallels the west side of OCCC from Station 20+20 to Station 23+10 (north of Palm Lane).

G. OHE crosses the OCCC at Station 20+20.

H. OHE parallels east side of 48th Street from Station 16+95 to Station 23+80 (north of Palm Lane).

I. OHE crosses the OCCC at Station 31+30 (north of Monte Vista Road).

J. OHE crosses the OCCC at Station 34+50 (north of Cypress Street).

K. OHE crosses the OCCC at Station 35+00 (north of Cypress Street).

L. OHE crosses 48th Street at Station 36+00 (Oak Street).

M. OHE crosses the OCCC at Station 37+90 (north of Oak Street).

N. OHE crosses the OCCC at Station 41+00 (north of Vernon Avenue).

O. OHE crosses the OCCC at Station 44+30 (north of Lewis Avenue).

P. OHE crosses the OCCC at Station 47+40 (north of Wilshire Drive).

Q. OHE crosses the OCCC at Station 50+80 (north of Virginia Avenue).

R. OHE crosses the OCCC at Station 54+10 (north of Cambridge Avenue).

S. OHE crosses the OCCC at Station 56+90 (north of Windsor Avenue).

T. OHE parallels west side of west 48th Street from Station 59+70 to 74+00 (Avalon Drive).

U. OHE parallels east side of the canal from Station 68+90 (south of Pinchot Avenue) to end of project at the Arizona Canal.

V. OHE parallels both sides of the irrigation canal, west of OCCC, from Station 74+00 (Avalon Drive) to Station 87+40 (south of Osborn Road).

W. OHE crosses the OCCC at Station 82+30 (south of Osborn Road).

X. OHE crosses the OCCC at Station 85+10 (south of Thomas Road).

Y. OHE crosses the OCCC at Station 87+40 (south of Thomas Road).

Z. OHE crosses the OCCC at Station 89+50 (diagonally across Osborn Road).

AA. OHE crosses the OCCC at Station 91+45 (north of Osborn Road).

BB. OHE crosses 48th Street at Station 94+20 (south of Whitton Avenue).

CC. OHE crosses 48th Street at Station 97+00 (north of Whitton Avenue).

DD. OHE parallels the west side of the irrigation ditch from Station 90+80 to Station 98+60 (south of Weldon Avenue).

EE. OHE crosses the OCCC at Station 102+20 (north of Weldon Avenue).

FF. OHE crosses the OCCC at Station 111+60 (south of Indian School Road).

3. SRP Electrical (Underground)

A. Underground electrical parallels and is west of the west top of bank of the canal from approximate Station 25+70 to Station 28+20 (between Hubbell Avenue and Monte Vista Road) 4' outside of the residential property line.

4. SRP Irrigation Pipe

A. Turn-out structure located approximately 30' west of the northeast corner of the southeast quarter of the southeast quarter of Section 31 with two 24" concrete pipes running south and one 30" cast-in-place concrete pipe running north (Oak Street at 48th Street).

B. Turn-out structure at southwest corner of the intersection of McDowell Road and 48th Street with 24" RGRCP running north along west edge of pavement of 48th Street to the turn-out structure 180' north of McDowell Road at centerline.

C. Turn-out structure 250' north of McDowell Road and 38' west of the 48th Street centerline.

D. 8 LF of 24" CP west of the 48th Street centerline 250' north of and 325' north of the McDowell Road centerline.

E. Turn-out structure 610' north of the McDowell Road centerline and 38' west of the 48th Street centerline.

F. 24" RGRCP crosses the OCCC from the turn-out structure in E. at a 74 degree deflection angle left via a manhole at the bottom of the canal to a manhole on the west side of the canal.

G. Manhole 108' south of Thomas Road and 38' west of the centerline of the OCCC.

H. 35 LF of 30" RGRCP runs west from manhole in G. to a turn-out structure.

I. 42" RGRCP runs from turn-out structure in H. north of the Thomas Road centerline 50' where it is tied into a 39" CP that continues north for a distance of 142'.

J. 48" concrete pipe parallels the west side of the OCCC, 111' west of the east quarter corner of Section 30 (Osborn Road), commencing 27' south of the Osborn Road centerline to 19' north of the Osborn Road centerline.

K. Turn-out structure 660' north of Osborn Road, 24' west of 48th Street centerline.

L. Center of 18 LF of 42" concrete pipe, which runs north and south, and is 696' north of Osborn Road and 24' west of the 48th Street centerline.

M. Center of 18 LF of north-south 42" concrete pipe under an east-west driveway is 1,232' north of the Osborn Road centerline and 24' west of the 48th Street centerline.

N. Turn-out structure is 1,305' north of the Osborn Road centerline and 24' west of the 48th Street centerline.

O. 36" concrete pipe parallels the centerline of 48th Street and is 25' west of said centerline commencing 752' south of the Indian School Road centerline and running to the Arizona Canal.

P. Turn-out structure located 405' south of the Indian School Road centerline and 30 feet west of the 48th Street centerline.

Q. Turn-out structure located 37' south of the Indian School Road centerline and 45' west of the 48th Street centerline.

R. 42" concrete pipe commencing at turn-out structure in Q. parallels 48th Street and runs to the Arizona Canal.

S. 18" concrete pipe outfalls from the east into the canal just north of Thomas Road.

T. 12" concrete pipe outfalls from the east into the canal at the alley south of Pinchot Avenue.

U. 10" concrete pipe outfalls from the east into the canal at Earll Drive.

V. 24" concrete pipe outfalls from the east into the canal just north of Earll Drive.

W. 18" concrete pipe outfalls from the east into the canal just south of Osborn Road.

X. 18" concrete pipe outfalls from the east into the canal just north of Osborn Road.

Y. 12" concrete pipe outfalls from the east into the canal at Whitton Avenue.

Z. 12" concrete pipe outfalls from the east into the canal just north of Whitton Avenue.

AA. 12" concrete pipe outfalls from the east into the canal in the box culvert at Indian School Road.

5. Wastewater

A. 8" VCP crosses 48th Street at Station 13+85.

B. 8" VCP parallels 35' east of the 48th Street centerline from beginning of the project to Station 17+35.

C. 8" VCP parallels the east right-of-way line of 48th Street, 6.5' to 7.5' west of east right-of-way line from Station 16+85 to Station 29+60.

D. 10" VCP crosses the OCCC at Station 44+20 (between Lewis Avenue and Wilshire Drive).

E. 10" VCP parallels the east right-of-way line of 48th Street, 8' west of the east right-of-way line from Station 43+50 to Station 55+85.

F. 8" VCP parallels the west right-of-way line of 48th Street, 4' east of the west right-of-way line from Station 59+65 to Station 61+15.

G. 8" VCP crosses the OCCC at Station 64+15 (north of Thomas Road).

H. 8" VCP parallels the east right-of-way line of 48th Street, 12' west of the east right-of-way line from Station 62+62 to Station 75+42.

I. 8" VCP parallels the east right-of-way line of 48th Street, 65' west of the east right-of-way line from Station 82+20 to Station 94+10.

J. 8" VCP parallels the west right-of-way line of 48th Street, 6' east of the west right-of-way line from Station 91+68 to Station 100+00.

K. 8" VCP parallels from the east right-of-way line of 48th Street, 8' west of the east right-of-way line from Station 91+67 to Station 111+85.

6. Water

A. 8" C.I. pipe parallels the 48th Street centerline from the beginning of the project to Station 19+20, and angles 10' west of the 48th Street east right-of-way line at Station 21+65, to 100' east of the west right-of-way at Station 62+80 (Thomas Road), to 17' west of the east right-of-way at Station 75+35 (Earll Drive).

B. 48" concrete pipe parallels 48th Street, 110' east of the west right-of-way line from project beginning to Station 62+13 (Thomas Road). Line turns east.

C. 2" pipe crosses the OCCC at Station 18+35 (south of Granada Road).

D. 6" pipe runs east of the canal at angle point in 8" pipe at Station 20+30 (Granada Road).

E. Pipe crosses the OCCC at Station 20+45 (north of Granada Road).

F. Pipe crosses the OCCC at Station 20+95 (north of Granada Road).

G. 6" pipe crosses the OCCC at Station 21+65 (Palm Lane).

H. 2" pipe crosses the OCCC at Station 22+55 (north of Palm Lane).

I. Pipe crosses the OCCC at Station 23+65 (south of Hubbell Avenue).

J. 6" pipe crosses the OCCC at Station 29+50 (Monte Vista Road).

K. 6" pipe crosses the OCCC at Station 32+80 (Cypress Street).

L. 8" AC pipe crosses the OCCC at Station 36+80 (north of Oak Street).

M. 54" pipe runs parallel to and 45' to 55' west of the OCCC centerline, from Station 24+05 (Hubbell Avenue) to Station 49+55 (Virginia Avenue).

N. 54" concrete pipe crosses the OCCC at Station 49+55 (Virginia Avenue).

O. 6" ACP parallels the west right-of-way of 48th Street 25' east of west right-of-way line, from Station 36+59 (Oak Street) to Station 58+50 (Edgemont Avenue).

P. 2" line runs from Station 58+40 to Station 59+95 (north of Edgemont Avenue).

Q. 8" line crosses the OCCC at Station 55+70 (Windsor Avenue).

R. 45" concrete line crosses the OCCC at Station 62+39 (Thomas Road).

S. 12" transite crosses the OCCC at Station 62+39 (Thomas Road).

T. 6" AC crosses the OCCC at Station 74+55 (Avalon Drive).

U. 8" C.I. parallels the east right-of-way line of 48th Street, 40' west of the east right-of-way line, from Station 81+75 to Station 95+85 (Whitton Avenue).

V. 8" ACP crosses the OCCC at Station 89+15 (Osborn Road).

W. 12" ACP crosses the OCCC at Station 115+43 (Indian School Road).

X. 12" ACP runs in westerly right-of-way of 48th Street from Station 115+43 (Indian School Road) to Station 118+55.

Y. 8" C.I.P. runs in westerly right-of-way of 48th Street from Station 119+05 (end of project) to Station 120+90 (perpendicular to the Arizona Canal).

7. Telephone (Overhead and Underground)

A. Underground line crosses 48th Street at Station 36+23 (Oak Street).

B. Overhead line runs inside the 48th Street east right-of-way line 3' to 9', from beginning of project to Station 23+18 (north of Palm Lane).

C. Overhead line parallels the west right-of-way of 48th Street, 21' east of the west right-of-way line from Station 37+85 (north of Oak Street) to Station 61+85 (Thomas Road).

D. Overhead line parallels the east right-of-way of 48th Street, 26' west of the east right-of-way line from Station 49+00 (Virginia Avenue) to Station 120+90 (end of project).

E. Overhead line parallels the east line of the OCCC from Station 61+85 (Thomas Road) to Station 65+45 (north of Thomas Road).

F. Overhead line parallels the west right-of-way line of 48th Street from Station 64+00 (north of Thomas Road) to Station 65+45.

G. Overhead line parallels the west right-of-way line of the OCCC from Station 89+00 (Osborn Road) to Station 99+80 (Clarendon Avenue).

H. Underground cable parallels the east right-of-way line of 48th Street 40' west of the east right-of-way line, from Station 49+00 (Virginia Avenue) to Station 89+00 (Osborn Road).

I. Underground cable crosses the OCCC at Station 88+76 (24' south of the Osborn Road centerline).

J. Underground cable crosses the OCCC at Station 115+53 (Indian School Road).

8. Television Cable (Underground and Overhead)

A. Cable parallels the east right-of-way line of 48th Street, 15' west of the east right-of-way line, from beginning of project at McDowell Road to Station 61+90 (Thomas Road).

B. Cable crosses the OCCC at Station 62+10 (Thomas Road).

C. Cable parallels the west right-of-way line at 48th Street, 10' east of the west right-of-way line, from Station 59+65 to Station 61+10 (Thomas Road).

D. Overhead lines parallel the 48th Street west right-of-way line from Station 64+95 (north of Thomas Road) to Station 87+35 (south of Osborn Road).

E. Overhead lines cross 48th Street at Station 73+90 (north of Thomas Road).

F. Cable and overhead lines parallel the 48th Street east right-of-way line from Station 68+65 (north of Thomas Road) to Station 73+90.

5.2.2 Utility Conflict Identification

The following is a compilation of utility conflicts classified by owner, type of utility and orientation (crossing or parallel) with a discussion of each conflict, including relocation investigations, possible construction options to avoid relocation and recommended solution.

1. City of Phoenix - Crossing Lines

A. 10" Sewer (VCP) - Station 44+20 (Figure 5.2-11)

Relocate 10" sewer below new conduit. Initially investigated existing upstream and downstream sewer systems in order to lower the 10" sewer and maintain gravity flow. This was not possible due to invert constraints. Also, investigated crossing north or south of the existing location to gain depth and lower the 10" sewer. This was not possible due to invert and slope constraints. Since it is not possible to maintain gravity flow, relocation of the 10" sewer would require the installation of a lift station or a siphon system. Cost estimates for both are listed below:

<u>Lift Station System</u>

10"/ VCP = 190 LF ($7,600)
Drop sewer manhole = 1 EA ($3,000)
Lift station = 1 EA ($30,000)
Bypass pumping = 1 LS ($1,800)
Concrete encasement = 5 CY ($700)
Estimated construction cost = $43,100
Operation and maintenance = $21,750/year

Siphon System

10" VCP = 380 LF ($15,200)
Manhole Structures = 2 EA ($60,000)
Bypass pumping = 1 LS ($1,800)
Concrete encasement = 10 CY ($1,400)
Estimated construction cost = $78,400
Operation and maintenance = $750/year

The siphon system has a higher construction cost, but a much lower operation and maintenance cost. Installation of the siphon system would result in an overall cost savings of $489,700 during an expected service life of 25 years.

It is evident that the siphon system is the most economical and it is also the preferred of the two systems by the City of Phoenix. The City of Phoenix has scheduled flow monitoring tests to be conducted at this location to collect data for the design of the proposed siphon system. Preliminary calculations indicate that a siphon system is feasible for the three sewer crossings discussed in this section. The preliminary cost estimate is based on installation of the siphon system.

On the following pages of this report an 8" and a 12" sewer line conflict are also identified and discussed. Pending authorization from the Flood Control District, another possible solution to the three sewer line conflicts can be investigated during the 30 percent design phase of this project. This solution would entail connecting two or all three of the sewer collection systems together and upsizing the southern downstream pipe to convey the increased flows. This could eliminate one or two of the sewer line crossings and the need for three separate siphon systems. Also, it might be possible to utilize a proposed 15" sewer line by combining the three sewer collection systems as previously mentioned and connecting them to this proposed line. This 15" sewer line is to be installed along Virginia Avenue from 49th Place to 48th Street where it would turn south and continue to McDowell Road. If feasible, connecting the combined systems to this 15" sewer line would eliminate the need for any east-west crossing of the proposed OCCC improvements.

B. 54" Water (PCCP) - Station 49+55 (Figure 5.2-14)

Relocate 54" water below new conduit. This would require the use of four 45 degree bends to achieve a lower elevation, and the replacement of an existing air release valve, which are included in the price of the pipe. Shutdown time for the relocation of the 54" water line will be coordinated with the City of Phoenix Water and Wastewater Engineering Department and should be held to a minimum. Relocation construction should be scheduled to avoid peak water use months as much as possible. Relocation below the new conduit would require the following:

54" PCCP = 100 LF ($46,000)
Concrete encasement = 30 CY ($4,200)
Estimated total cost = $50,200

C. 45" Water (CP) - Station 62+40 (Figure 5.2-17)

Relocate 45" water below new conduit. This would require the use of four 45 degree bends to achieve a lower elevation. Shutdown time for the relocation of the 45" water will be coordinated with the City of Phoenix Water and Wastewater Engineering Department and avoid peak water use months as much as possible. Relocation below the new conduit would require the following:

45" PCCP = 100 LF ($44,000)
Concrete encasement = 25 CY ($3,500)
Estimated total cost = $47,500

D. 8" Sewer (VCP) - Station 64+15 (Figure 5.2-19)

Relocate 8" sewer above new conduit. Initially investigated existing upstream and downstream sewer systems in order to raise the 8" sewer and maintain gravity flow. This was not possible due to invert and slope constraints. Also, investigated crossing north or south of existing location to gain depth to lower the 8" sewer. This was not possible due to invert and slope constraints. The top of the new concrete box culvert is very close to the 8" sewer. Relocation of this utility may be avoided and will be thoroughly investigated in the design phase for this project. Since it is not possible to maintain gravity flow, relocation of the 8" sewer would require the installation of a lift station or a siphon system. Cost estimates for both are listed below:

<u>Lift Station System</u>

8" VCP = 190 LF ($5,700)
Drop sewer manhole = 1 EA ($3,000)
Lift station = 1 EA ($30,000)
Bypass pumping = 1 LS ($1,800)
Concrete encasement = 5 CY ($700)
Estimated construction cost = $41,200
Operation and maintenance = $21,750/year

Siphon System

8" VCP = 380 LF ($11,400)
Manhole Structures = 2 EA ($60,000)
Bypass pumping = 1 LS ($1,800)
Concrete encasement = 10 CY ($1,400)
Estimated construction cost = $72,800
Operation and maintenance = $750/year

As previously mentioned, the siphon system is the most economical and preferred alternative, and the preliminary cost estimate is based on installation of this system.

E. 12" Sewer (VCP) - Station 89+20 (Figure 5.2-25)

Relocate 12" sewer below new conduit. Initially investigated existing upstream and downstream sewer systems in order to lower the 12" sewer and maintain gravity flow. This was not possible due to invert and slope constraints. Also, investigated crossing north or south of the existing location to gain depth to lower the 12" sewer. This was not possible due to invert and slope constraints. Since it is not possible to maintain gravity flow, relocation of the 12" sewer would require the installation of a lift station or siphon system. Cost estimates are listed below:

Lift Station System

12" VCP = 190 LF ($9,500)
Drop sewer manhole = 1 EA ($3,000)
Lift station = 1 EA ($30,000)
Bypass pumping = 1 LS ($1,800)
Concrete encasement = 5 CY ($700)
Estimated construction cost = $45,000
Operation and maintenance = $21,750/year

Siphon System

12" VCP = 380 LF ($19,000)
Manhole Structures = 2 EA ($60,000)
Bypass pumping = 1 LS ($1,800)
Concrete encasement = 10 CY ($1,400)
Estimated construction cost = $80,400
Operation and maintenance = $750/year

As previously mentioned, the siphon system is the most economical and preferred alternative, and the preliminary cost estimate is based on installation of this system.

F. Miscellaneous Crossings

12" Water - Station 62+60 (Figure 5.2-17) - 100 LF
12" Water - Station 115+43 (Figure 5.2-31) - 100 LF
Estimated total cost = $10,000

8" Water - Stations 21+70, 36+80, 55+65 and 89+15
400 LF
Estimated total cost = $16,000

6" Water - Station 29+50 and 32+80
200 LF
Estimated total cost = $6,000

2" Water - Stations 18+40, 20+00, 20+40, 22+50 and 23+50
500 LF
Estimated total cost = $7,500

2. City of Phoenix - Parallel Lines

A. 8" Water - Station 17+40 to Station 63+40 (Figures 5.2-6 and 5.2-17)

Relocate 8" water line to the east, out of the excavation area. The City of Phoenix has indicated that this 8" water line is an old line and they are considering replacing it as an improvement project. Relocation of the 8" water line would require the following:

8" water = 4,600 LF
Estimated total cost = $184,000

B. Other utilities that are parallel to the canal centerline and that are located along cross streets east of the canal (Figures 5.2-1 through 5.2-3) that may require relocation or bracing due to side inlet construction are listed as follows:

Granada Road (Figure 5.2-6) - 48" water (CP), 6" water (ACP) and 8" sewer (VCP).

Holly Street (Figure 5.2-9) - 8" sewer (VCP), 48" water (CP) and 6" water (CIP).

Oak Street (Figure 5.2-10) - 48" water (CP) and 8" water (CIP).

Virginia Avenue (Figure 5.2-13) - 48" water (CP), 6" water (CIP) and 8" sewer (VCP).

South of Windsor Avenue (Figure 5.2-15) - 48" water (CP), 6" water (CIP) and 8" sewer (VCP).

Windsor Avenue (Figure 5.2-16) - 48" water (CP) and 8" sewer (VCP).

Thomas Road (Figure 5.2-17) - 48" water (CP), 12" water (CP) and 8" sewer (VCP).

Pinchot Avenue (Figure 5.2-20) - 8" water (ACP) and 8" sewer (VCP).

Earll Drive (Figure 5.2-21) - 8" water (ACP), 6" water (ACP) and 8" sewer (VCP).

Richardson (Figure 5.2-23) - 8" water (CIP) and 8" sewer (VCP).

Osborn Road (Figure 5.2-25) - 8" water (ACP), 8" sewer (VCP) and 12" sewer (VCP).

Whitton Avenue (Figure 5.2-28) - 6" water (ACP).

Indian School Road (Figure 5.2-31) - 12" water (Transite), and 8" sewer (VCP).

3. City of Phoenix - Traffic Signals and Street Lights

A. Thomas Road Station 62+40 (Figure 5.2-17) - There is a joint use (traffic signal and overhead power) pole on the southeast corner and two combination traffic signal and street light poles, one on the northeast corner and one on the southeast return of the intersection of the existing box culvert and Thomas Road, that require relocation during construction.

Relocate joint use pole = 1 EA
Relocate traffic signal poles = 2 EA
Estimated total cost = $16,000

B. Indian School Road Station 115+40 (Figure 5.2-31) - There are two combination traffic signal and street light poles, one on the south center and the other on the northwest corner of the intersection of the existing box culvert and Indian School Road. There also is a traffic signal pole on the north center of the intersection. These poles will require relocation during construction.

Relocate traffic signal pole = 3 EA
Estimated total cost = $9,000

4. Salt River Project - Parallel Overhead Power Lines

A. 12 kV overhead lines, Station 20+10 to Station 37+80 (east side), Station 37+80 to Station 59+70 (east and west side), Station 61+90 to Station 87+60 (east side) (Figures 5.2-7 through 5.2-24).

Relocate power poles out of the excavation area. As previously mentioned, the pole on the southeast corner at Thomas Road is a joint-use pole with City of Phoenix traffic signals and will require coordination to relocate. Relocation of the power poles would require the following:

Relocate power poles (12 kV) = 42 EA
Estimated total cost = $294,000

B. 69 kV overhead lines, Station 89+20 to Station 118+00. Underground power line, Station 115+85 (Figures 5.2-27 through 5.2-32).

Relocate power poles out of the excavation area. Relocation of the power poles and underground line would require the following:

Relocate power poles (69 kV) = 14 EA
Relocate underground power line = 50 LF
Estimated cost = $215,000

C. Other overhead power poles located along cross streets east of the canal (Figures 5.2-1 through 5.2-3) that may require relocation or bracing due to side inlet construction are as follows:

McDowell Road - 12 kV line, north side.

Oak Street - 69 kV line.

Thomas Road - 12 kV line, south side and a 69 kV line, north side.

5. Salt River Project - Irrigation and Subsurface Drains

A. 24" RGRCP irrigation conflict - Station 15+95 (Figure 5.2-5).

Relocate existing 24" irrigation pipe below new conduit and install new manhole east of new conduit.

Relocate 24" RCRCP = 130 LF ($6,500)
New manhole = 1 EA ($2,000)
Estimated total cost = $7,500

B. As-built information was investigated in order to determine approximate locations for SRP subsurface drains and lateral outfalls that connect into the existing OCCC. Three connections were discovered during investigation of the as-built information provided by SRP. Follow-up site investigations by Greiner and SRP confirmed that the three connections do outfall into the existing OCCC and also produced the identification of six other outfall connections not shown on the as-builts. Further investigation will be conducted and coordinated with SRP to determine the type of pipe material, the invert elevations and the exact location for all the connections. The following is a tabulation of the ten SRP subsurface drains:

Preliminary Size and Type of Pipe	Approximate Station	Figure No.	Estimated Connection Length	Cost Estimate
18" CP	62+50	5.2-17	40'	$1,600
12" CP	68+90	--	40'	1,400
10" CP	75+60	5.2-21	40'	1,200
24" CP	76+20	5.2-22	40	2,000
18" CP	88+70	--	40'	1,600
18" CP	89+40	5.2-26	40'	1,600
12" CP	95+65	5.2-28	40'	1,400
12" CP	96+50	--	40'	1,400
12" CP	115+40	5.2-31	40'	1,400
			Total	**$13,600**

6. Southwest Gas - Crossing Lines

   A. 4" gas - Station 62+60 (Figure 5.2-17)

      Cut and cap existing 4" gas line to accommodate new conduit construction:

      Cut and cap 4" gas = 1 EA
      Estimated total cost = $5,000

   B. 4" Gas - Station 115+60 (Figure 5.2-31)

      Cut and cap existing 4" gas line to accommodate new conduit construction:

      Cut and cap 4" gas = 1 EA
      Estimated total cost = $5,000

7. Southwest Gas - Parallel Lines

   A. 8" High Pressure Gas - Station 89+00 to Station 116+00 (Figures 5.2-25 through 5.2-31)

      The 8" gas line is a high pressure line that is located approximately 36' to 46' east of the existing canal centerline. The 8" line is not expected to require relocation, but is close to the excavation limits, and the contractor shall be alerted to its location.

   B. Other gas lines that are parallel to the canal centerline and that are located along cross streets east of the canal (Figures 5.2-1 through 5.2-3) that may require relocation or support due to side inlet construction are listed as follows:

      Granada Road (Figure 5.2-6) - 8" HP gas

      Holly Street (Figure 5.2-9) - 8" HP gas and a 4" gas

      Oak Street (Figure 5.2-10) - 8" HP gas

      Virginia Avenue (Figure 5.2-13) - 8" HP gas and a 4" gas

      South of Windsor Avenue (Figure 5.2-15) - 8" HP gas and a 4" gas

      Windsor Avenue (Figure 5.2-16) - 8" HP gas and a 4" gas

      Thomas Road (Figure 5.2-17) - 8" HP gas, a 4" gas and a 2-1/2" gas

Pinchot Avenue (Figure 5.2-20) - 8" HP gas and a 2-1/2" gas

Earll Drive (Figure 5.2-21) - 8" HP gas and a 2-1/2" gas

Richardson (Figure 5.2-23) - 8" HP gas and a 2-1/2" gas

Osborn Road (Figure 5.2-25) - 8" HP gas and a 2-1/2" gas that ends on the south side

Whitton Avenue (Figure 5.2-28) - 8" HP gas

Indian School Road (Figure 5.2-31) - 8" HP gas and a 2-1/2" gas

Southwest Gas is in the process of determining the maximum span of excavation and support requirements for the various gas line undercrossings. They are also determining if they have any specifications relating to the construction around high pressure gas lines.

8. U.S. West Telephone - Underground Crossing Lines

A. Telephone Lines - Station 62+00 (Figure 5.2-17)

Relocate existing telephone line to accommodate new conduit construction:

Relocate telephone lines = 100 LF
Estimated cost = $10,000

B. Telephone Duct Bank (10 Ducts) - Station 88+75 (Figure 5.2-25)

Support existing telephone duct to accommodate new conduit construction:

Support telephone duct = 1 EA
Estimated cost = $300,000

C. Telephone Duct Bank (7 Ducts) - Station 115+55 (Figure 5.2-31)

Support existing telephone duct to accommodate new conduit construction:

Support telephone duct = 1 EA
Estimated cost = $300,000

9. U.S. West Telephone - Parallel Lines

A. Telephone Lines - Station 62+80 to Station 65+60 (Figures 5.2-18 and 5.2-19)

Relocate existing telephone lines out of the excavation area:

Relocate telephone lines = 280 LF
Estimated cost = $28,000

B. Telephone Lines - Station 87+40 to Station 88+75

Relocate existing telephone lines out of the excavation area:

Relocate telephone lines = 140 LF
Estimated cost = $13,500

C. Other buried telephone lines that are parallel to the canal centerline and that are located along cross streets east of the canal (Figures 5.2-1 through 5.2-3) that may require relocation or support due to side inlet construction are listed below:

Virginia Avenue (Figure 5.2-13) - two lines

South of Windsor Avenue (Figure 5.2-15) - two lines

Windsor Avenue (Figure 5.2-16) - two lines

Thomas Road (Figure 5.2-17) - one line

Pinchot Avenue (Figure 5.2-20) - three lines

Earll Drive (Figure 5.2-21) - three lines

Richardson (Figure 5.2-23) - three lines

Osborn Road (Figure 5.2-25) - one line

Whitton Avenue (Figure 5.2-28) - one line

Weldon Avenue (Figure 5.2-29) - two lines

Indian School Road (Figure 5.2-31) - one line

10. U.S. West Telephone - Overhead Lines

Information regarding overhead telephone lines was requested from U.S. West representatives at the utility coordination meeting held on February 28, 1991 at the City of Phoenix offices. This information has not been received. U.S. West representatives should refer to the Salt River Project overhead power pole relocations previously identified in this report to identify joint use pole relocations that could affect their overhead facilities.

11. Dimension Cable T.V. - Crossing Lines

A. Underground Cable - Station 62+00 (Figure 5.2-17)

Relocate existing cable line to accommodate new conduit construction

Relocate cable = 100 LF
Estimated cost = $10,000

B. Other buried Cable T.V. lines that are parallel to the canal centerline and that are located along cross streets east of the canal (Figures 5.2-1 through 5.2-3) that may require relocation or bracing due to side inlet construction are listed below:

Virginia Avenue (Figure 5.2-13) - one line

South of Windsor Avenue (Figure 5.2-15) - two lines

Windsor Avenue (Figure 5.2-16) - two lines

Pinchot Avenue (Figure 5.2-20) - two lines

Whitton Avenue (Figure 5.2-28) - one line

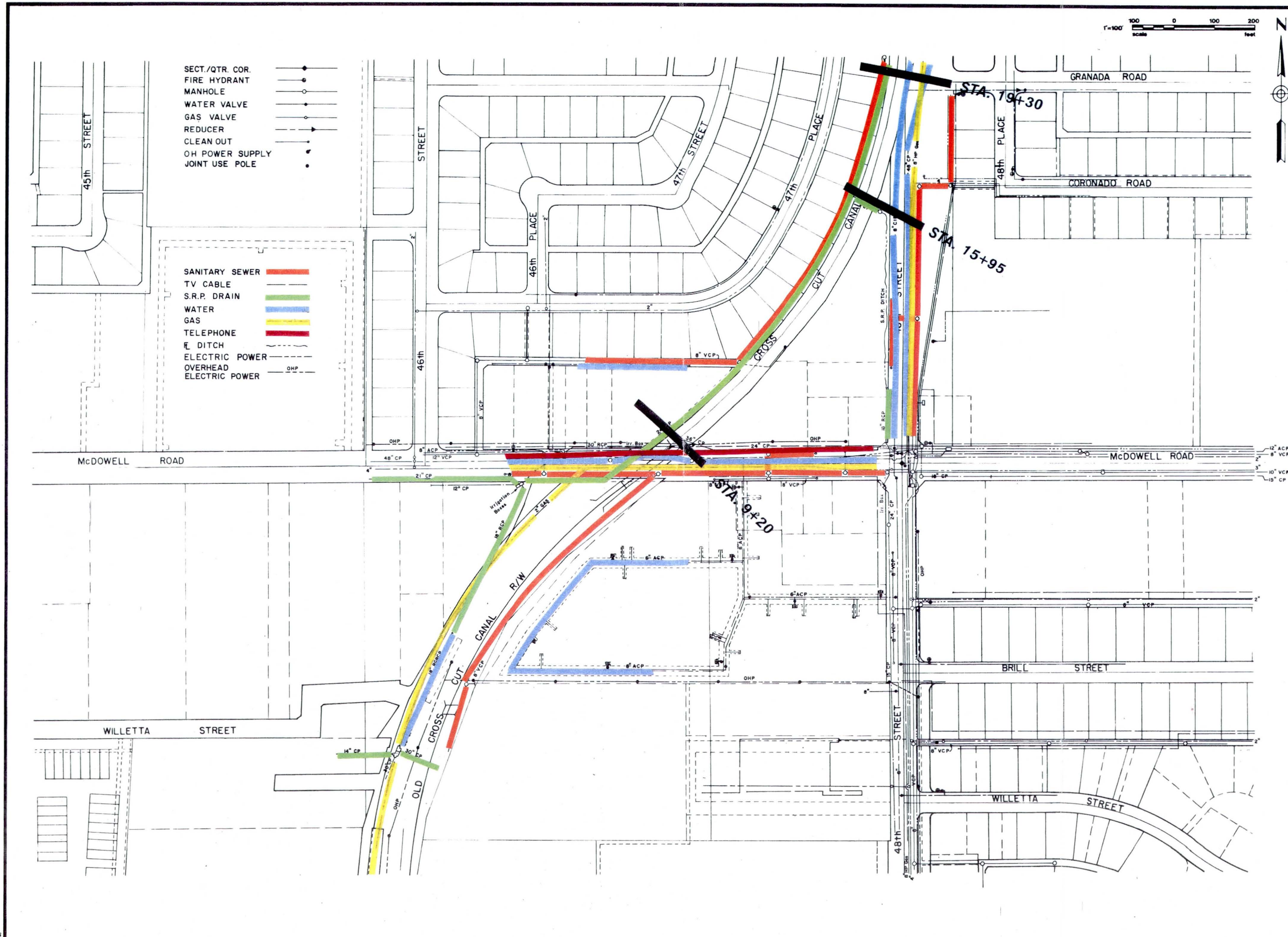
Weldon Avenue (Figure 5.2-29) - two lines

5.2.3 Utility Relocation Preliminary Cost Estimate

Item #	Bid Item	Quantity	Unit	Unit Cost	Total
610.1	Relocate 54" water (PCCP)	100	LF	$460	$46,000
610.2	Relocate 45" water (PCCP)	100	LF	$400	$44,000
610.3	Relocate 12" water	200	LF	$50	$10,000
610.4	Relocate 8" water	5,260	LF	$40	$210,400
610.5	Relocate 6" water	200	LF	$30	$6,000
610.6	Relocate 2" water	500	LF	$15	$7,500
610.7	Concrete encasement	110	CY	$140	$15,400
615.1	Relocate 12" sewer (VCP). Install new siphon system.	1	LS	$80,400	$80,400
615.2	Relocate 10" sewer (VCP). Install new siphon system.	1	LS	$78,400	$78,400
615.3	Relocate 8" sewer (VCP). Install new siphon system.	1	LS	$72,800	$72,800
	Relocate 24" irrigation	170	LF	$50	$8,500
	Relocate 18" irrigation	120	LF	$40	$4,800
	Relocate 12" irrigation	160	LF	$35	$4,800
	Relocate 10" irrigation	40	LF	$30	$1,200
	Irrigation manhole	1	EA	$2,000	$2,000
	Relocate irrigation standpipe	1	EA	$1,500	$1,500
	Cut and cap 4" gas	2	EA	$10,000	$20,000
	Support gas lines for stub-out construction	1	LS	$50,000	$50,000
	Relocate telephone lines	520	LF	$100	$52,000
	Support telephone duct	2	EA	$300,000*	$600,000
	Relocate traffic signal pole	5	EA	$3,000	$15,000
	Relocate joint use pole (12 kV and traffic signal)	1	EA	$10,000	$10,000

Item #	Bid Item	Quantity	Unit	Unit Cost	Total
	Relocate power poles (12 kV)	42	EA	$7,000	$294,000
	Relocate power poles (69 kV)	14	EA	$15,000	$210,000
	Relocate underground power line	50	LF	$100	$5,000
	Relocate underground cable T.V.	100	LF	$100	$10,000
	Preliminary Relocation Total				**$1,860,500**

* Unit price is based on the high end of a cost estimate range provided by U.S. West Communications.



OLD CROSS CUT CANAL FLOOD CONTROL IMPROVEMENTS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

Greiner Engineering

Design
Drawn K.R.S.
Check S.S.
Scale N.T.S.
(Reduced)

UTILITY INVENTORY

Date MARCH 1991
Job No.

FIGURE 5.2-1

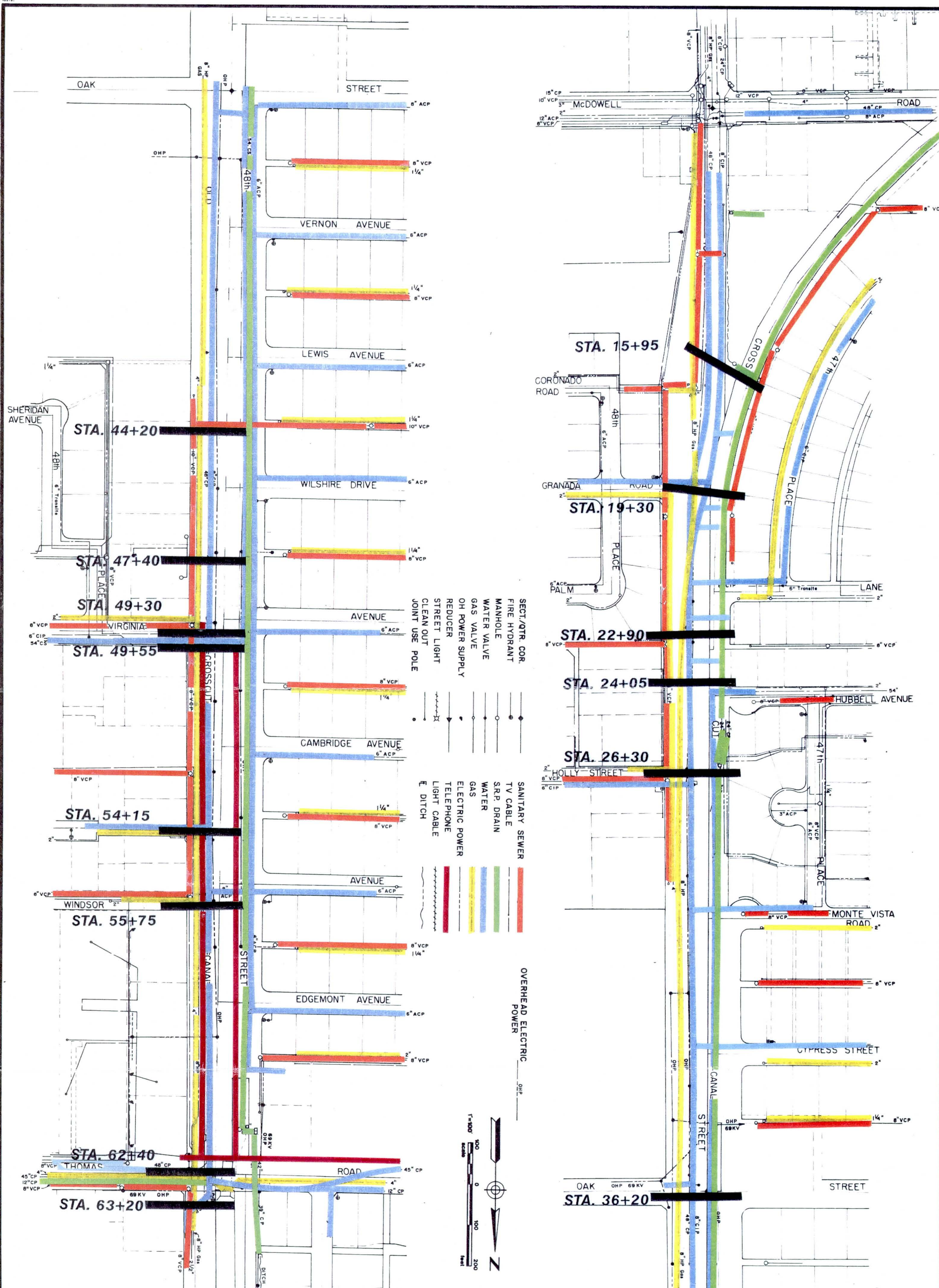
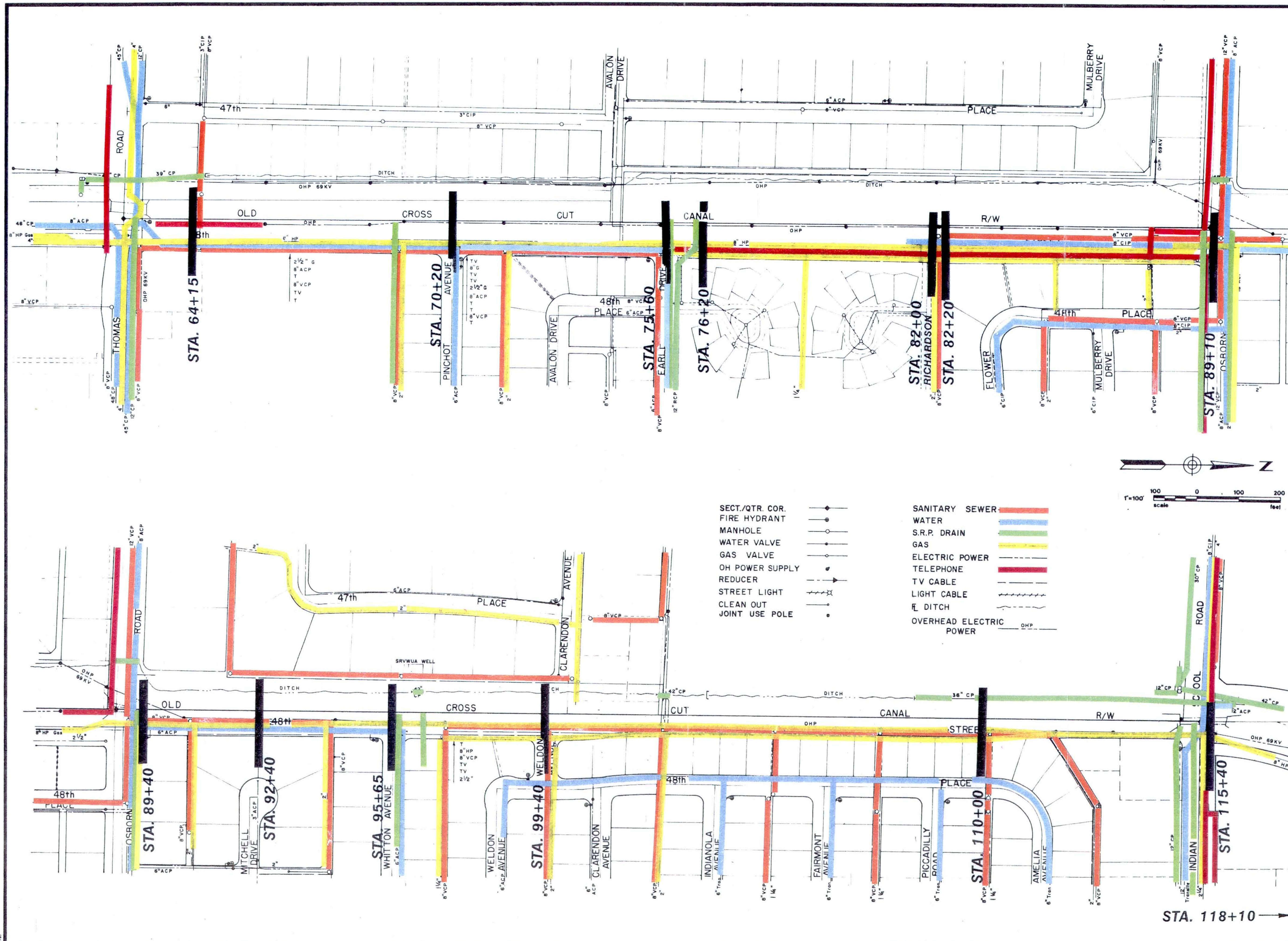


FIGURE 5.2-2	Date MARCH 1991	UTILITY INVENTORY	Scale N.T.S. (Reduced); Check S.S.; Drawn K.R.S.	Greiner Engineering	FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

OLD CROSS CUT CANAL FLOOD CONTROL IMPROVEMENTS



OLD CROSS CUT CANAL FLOOD CONTROL IMPROVEMENTS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

Greiner Engineering

Greiner Engineering Sciences, Inc.

Design

Drawn K.R.S.

Check S.S.

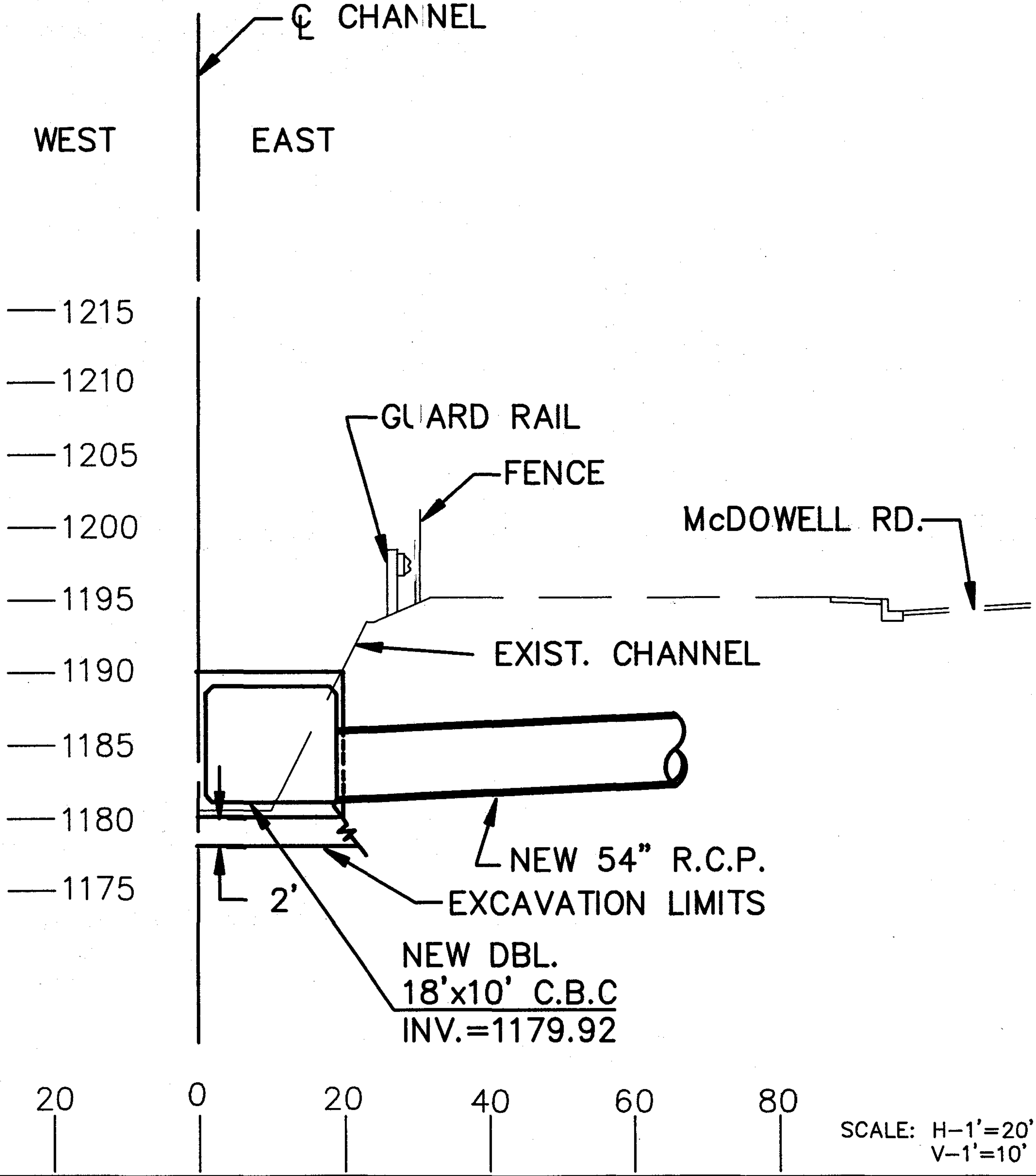
Scale N.T.S. (Reduced)

UTILITY INVENTORY

Date MARCH 1991

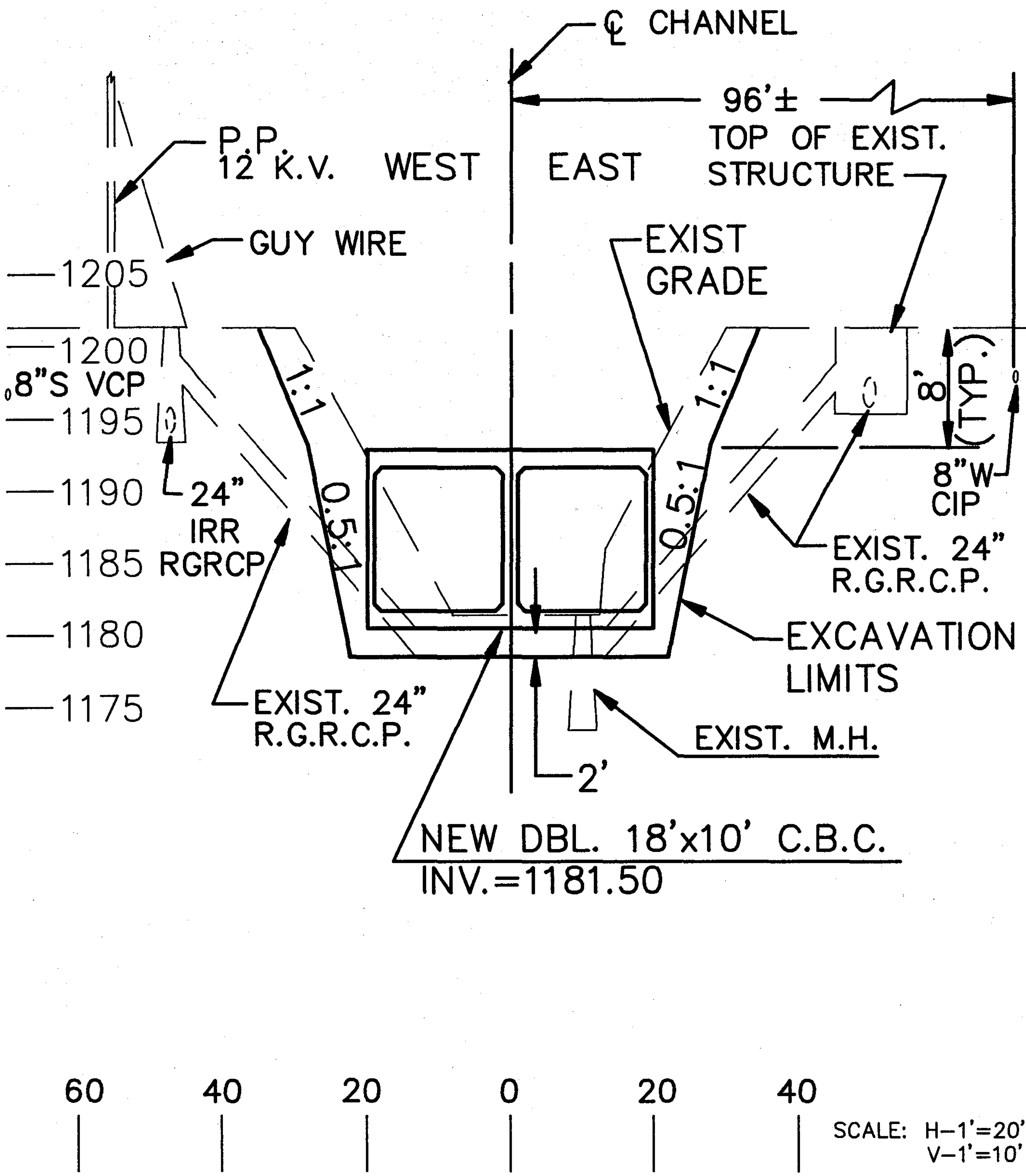
Job No.

FIGURE 5.2-3



OLD CROSS CUT CANAL
McDowell Rd. Sta. 9+20
Stub-out Profile 54" Inlet

FIGURE 5.2-4



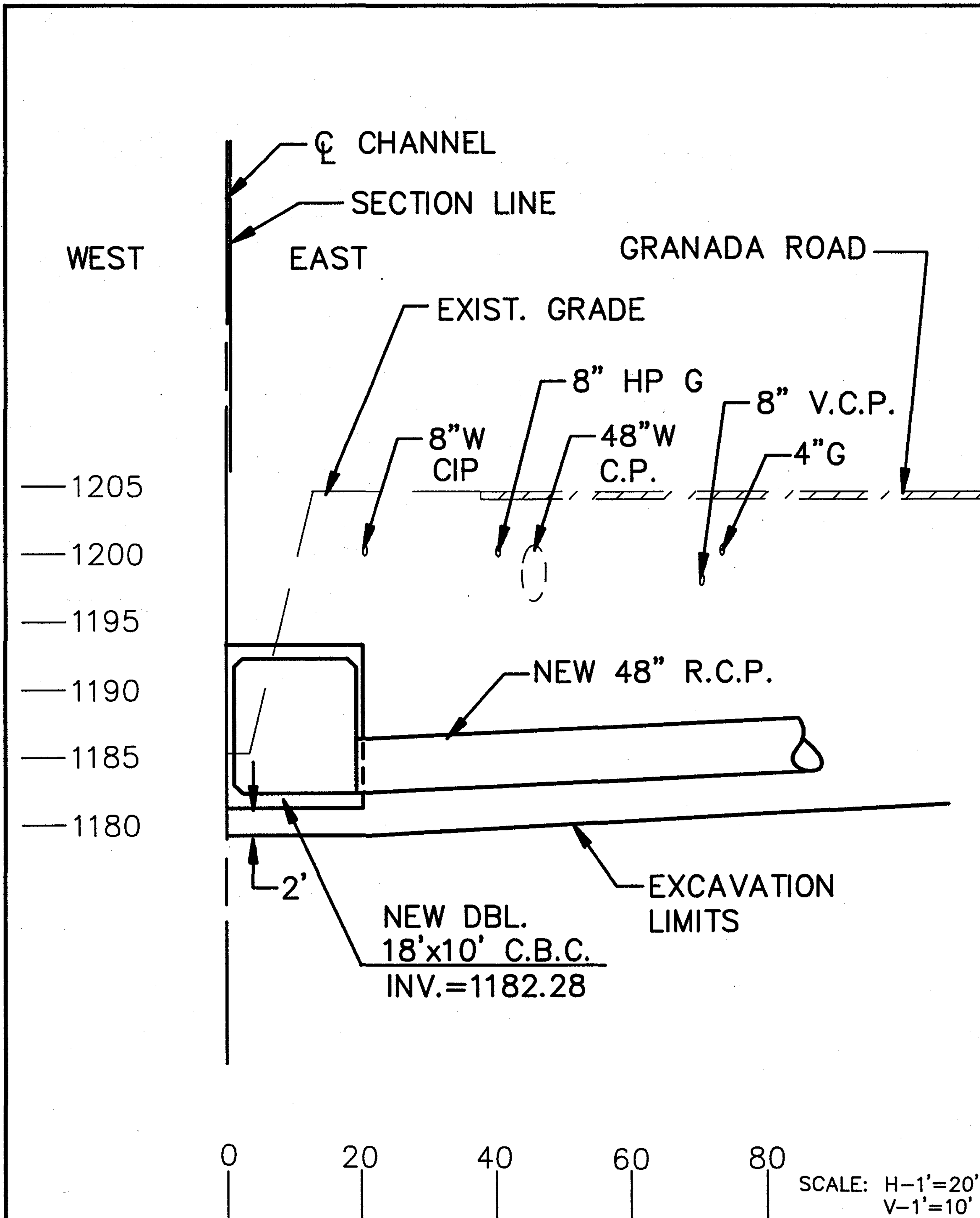
Greiner

OLD CROSS CUT CANAL

Cross Section at Sta. 15+95

New Double 18'x10' C.B.C.

FIGURE 5.2–5



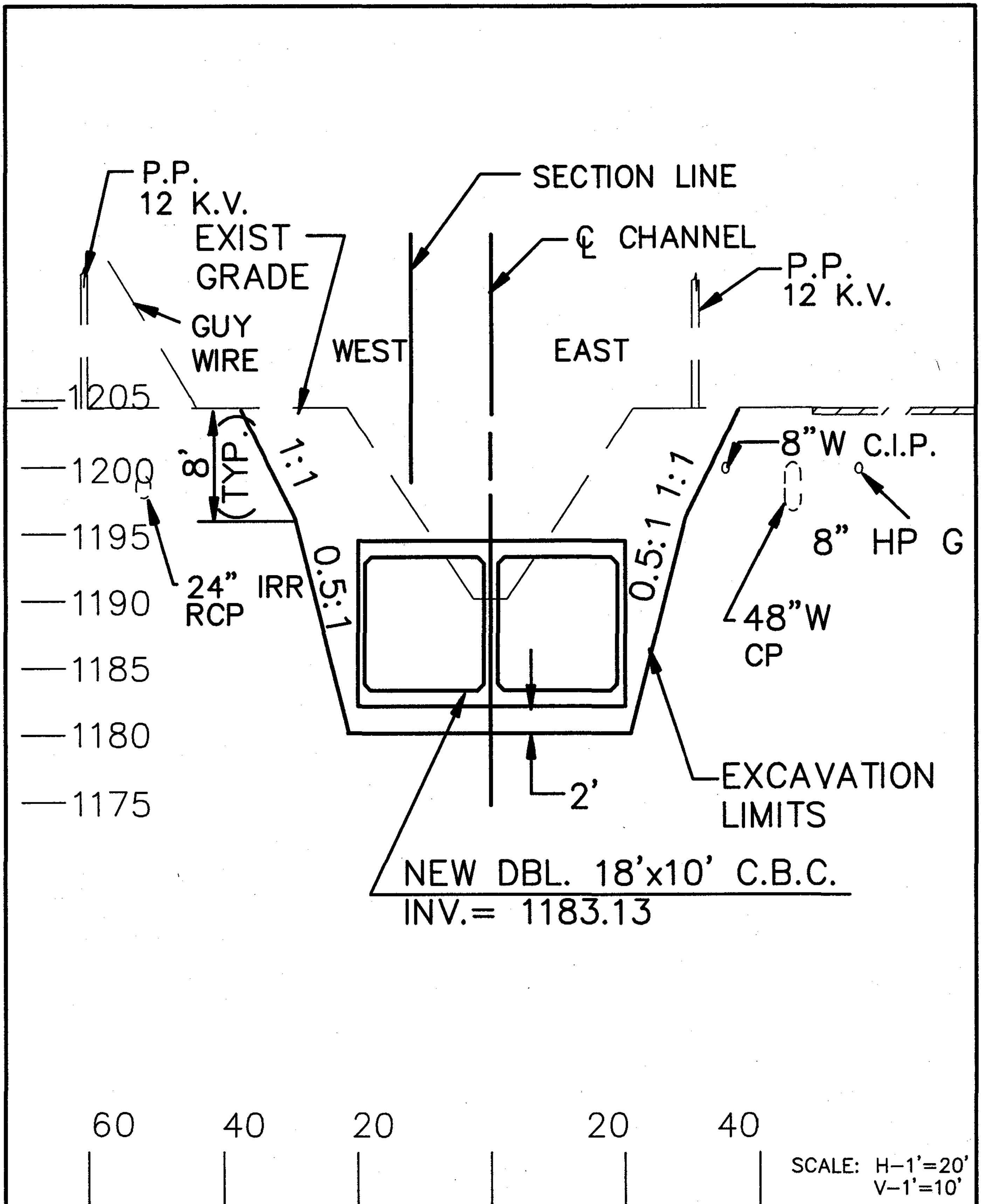
OLD CROSS CUT CANAL

Cross Section at Sta. 19+30

Stub-out Profile 48" Inlet

Greiner

FIGURE 5.2-6



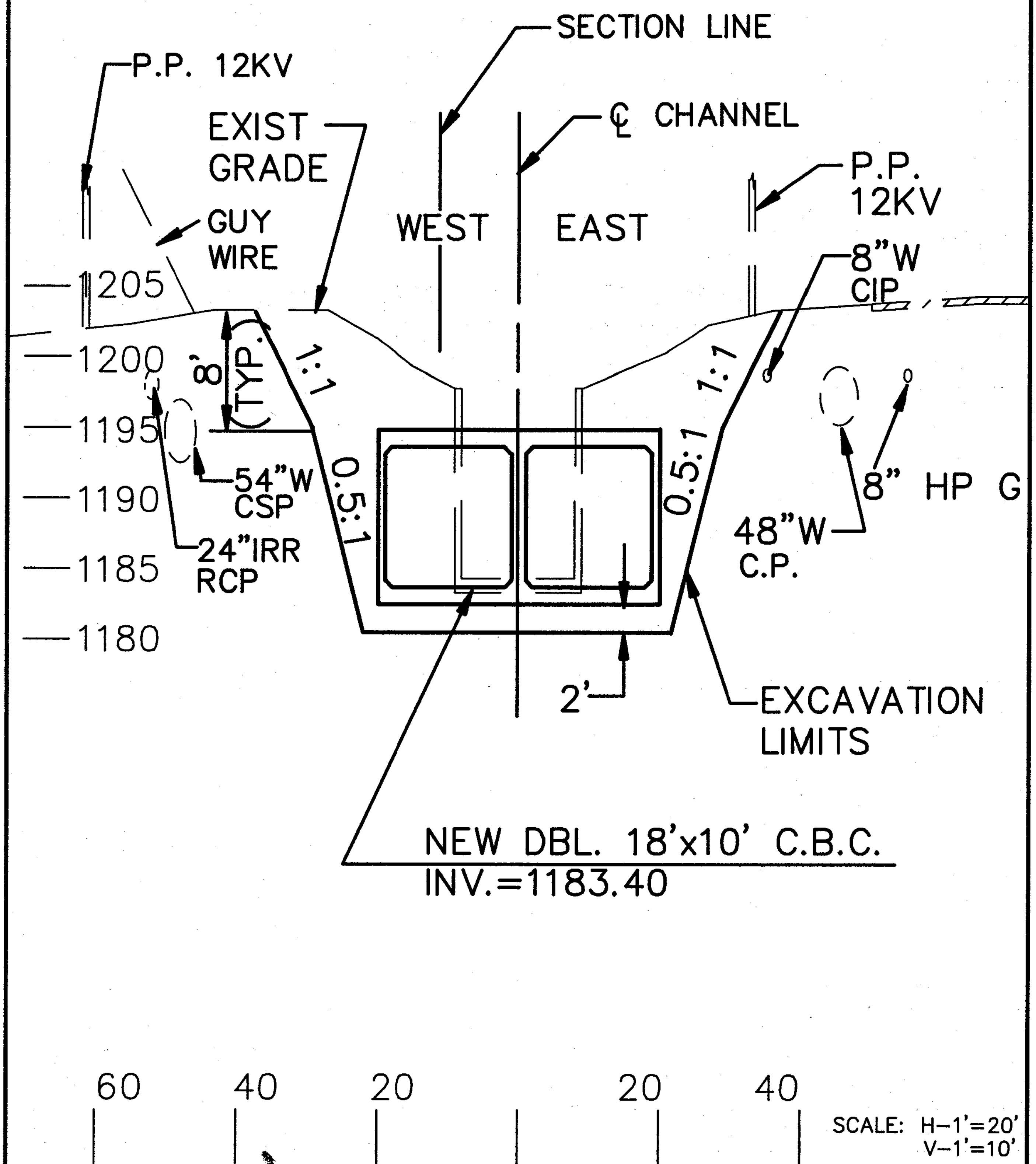
OLD CROSS CUT CANAL

Cross Section at Sta. 22+90

New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2–7



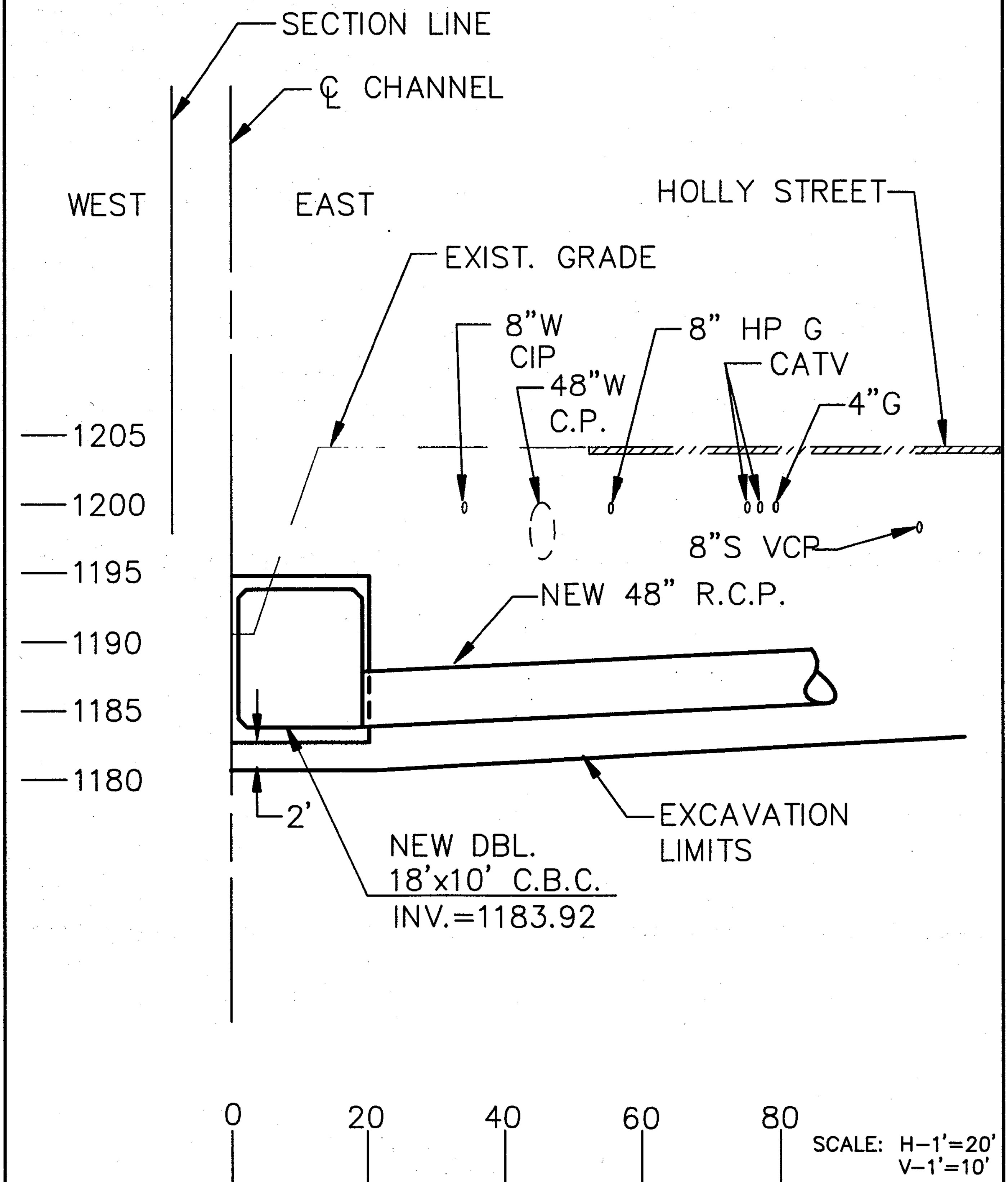
OLD CROSS CUT CANAL

Cross Section at Sta. 24+05

New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2-8



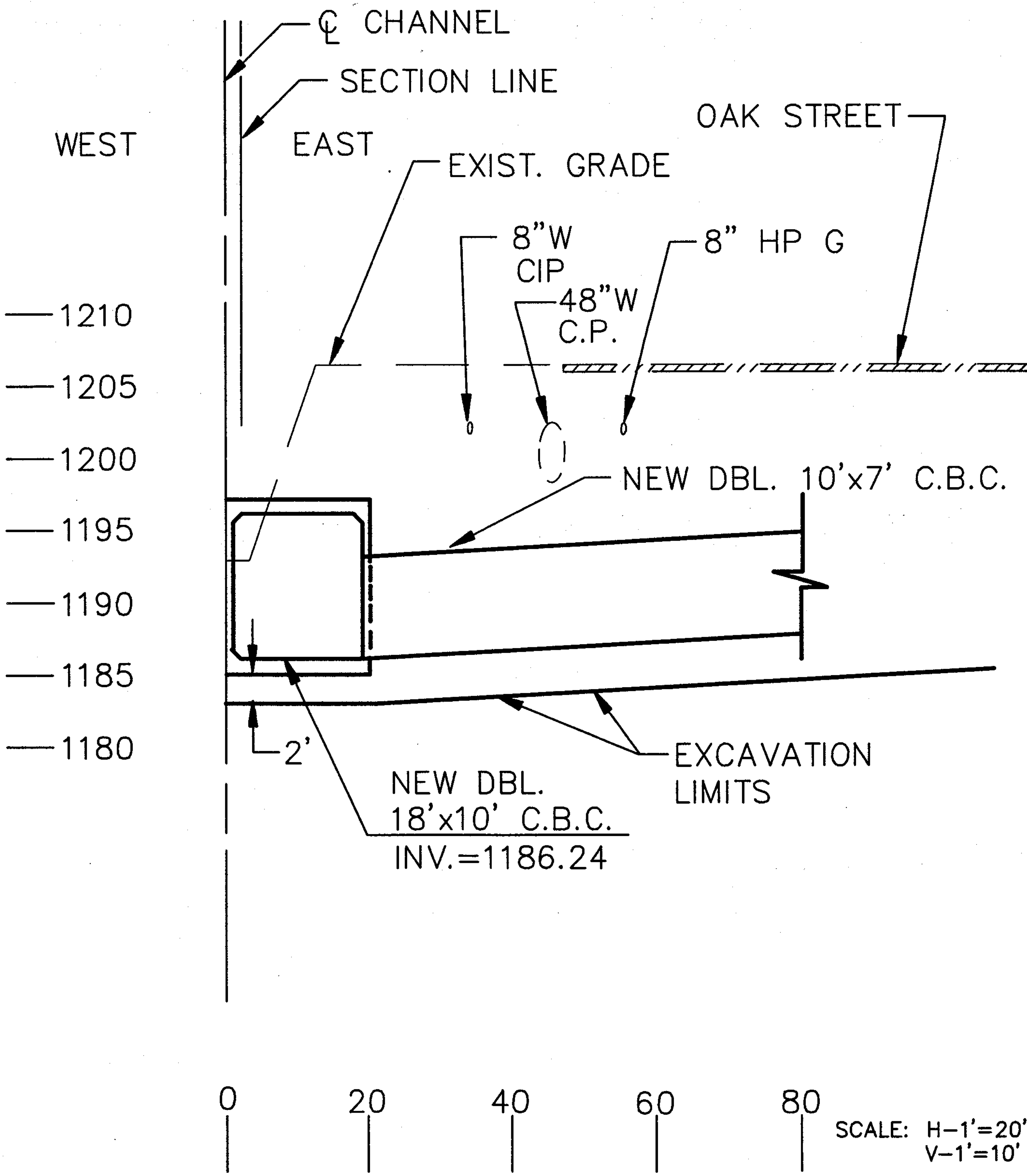
OLD CROSS CUT CANAL

Cross Section at Sta. 26+30

Stub–out Profile 48" Inlet

Greiner

FIGURE 5.2–9



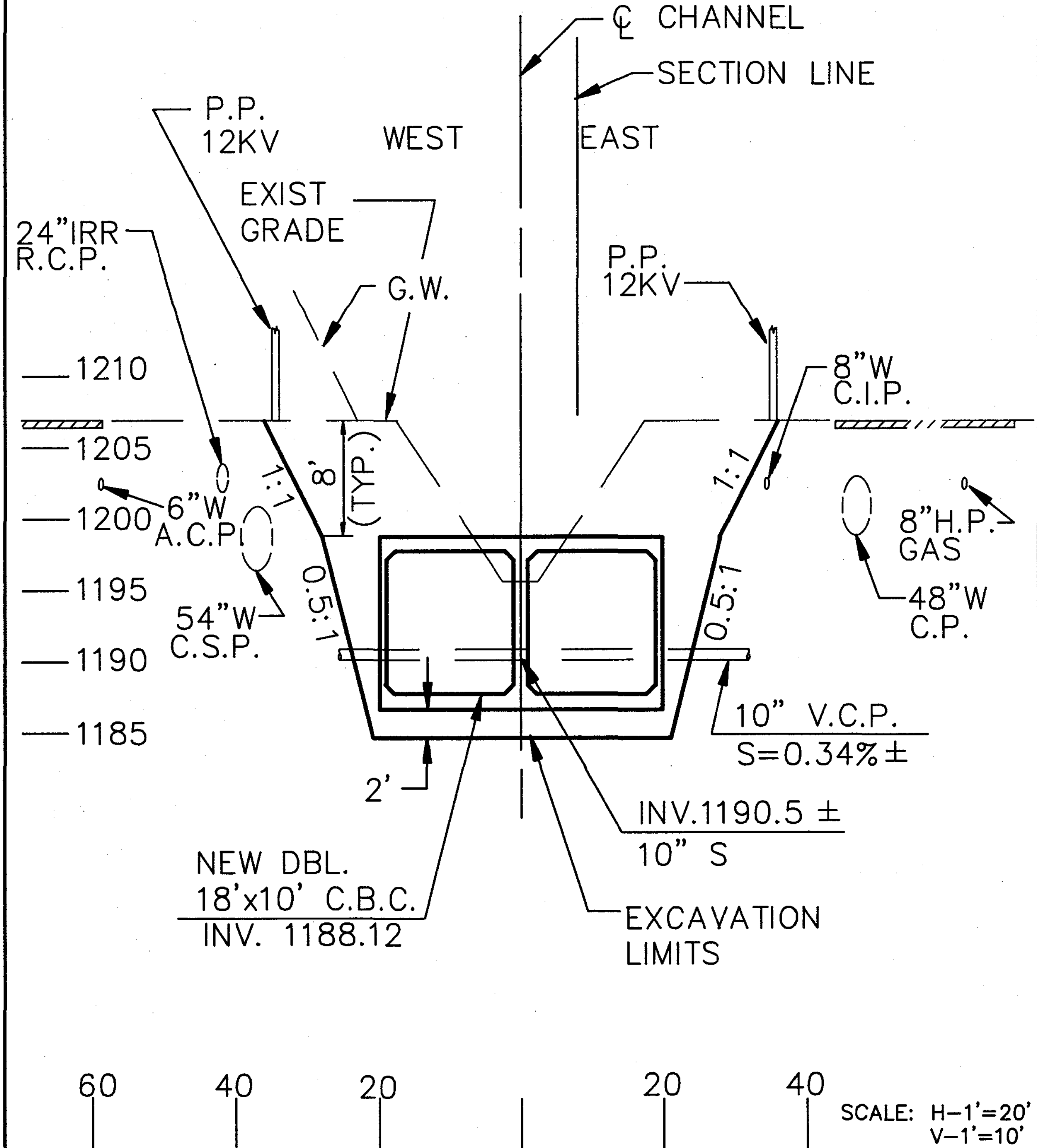
OLD CROSS CUT CANAL

Cross Section at Sta. 36+20

Stub-out Profile 2-10'x7' C.B.C.

Greiner

FIGURE 5.2-10



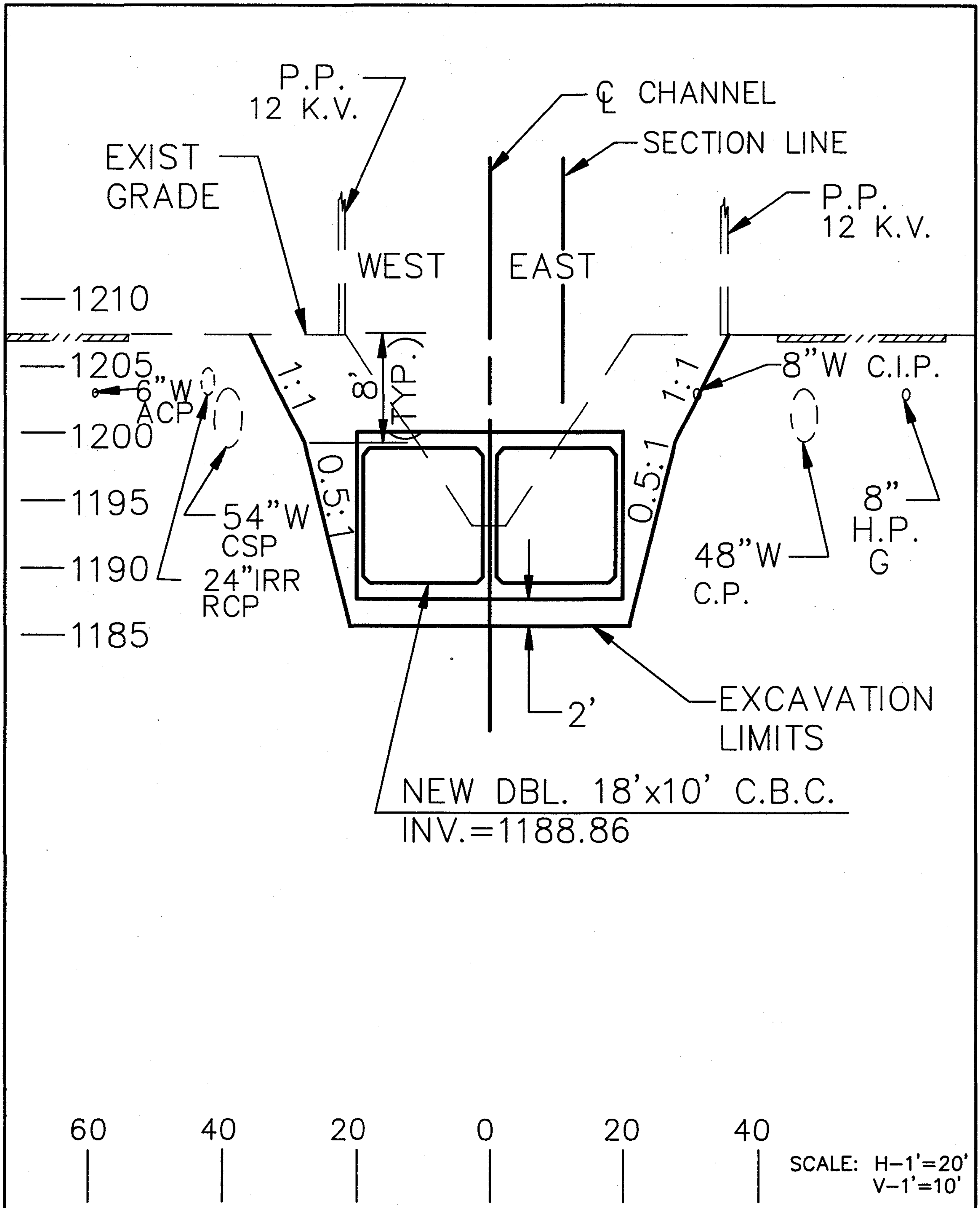
OLD CROSS CUT CANAL

Cross Section at Sta. 44+20

New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2–11



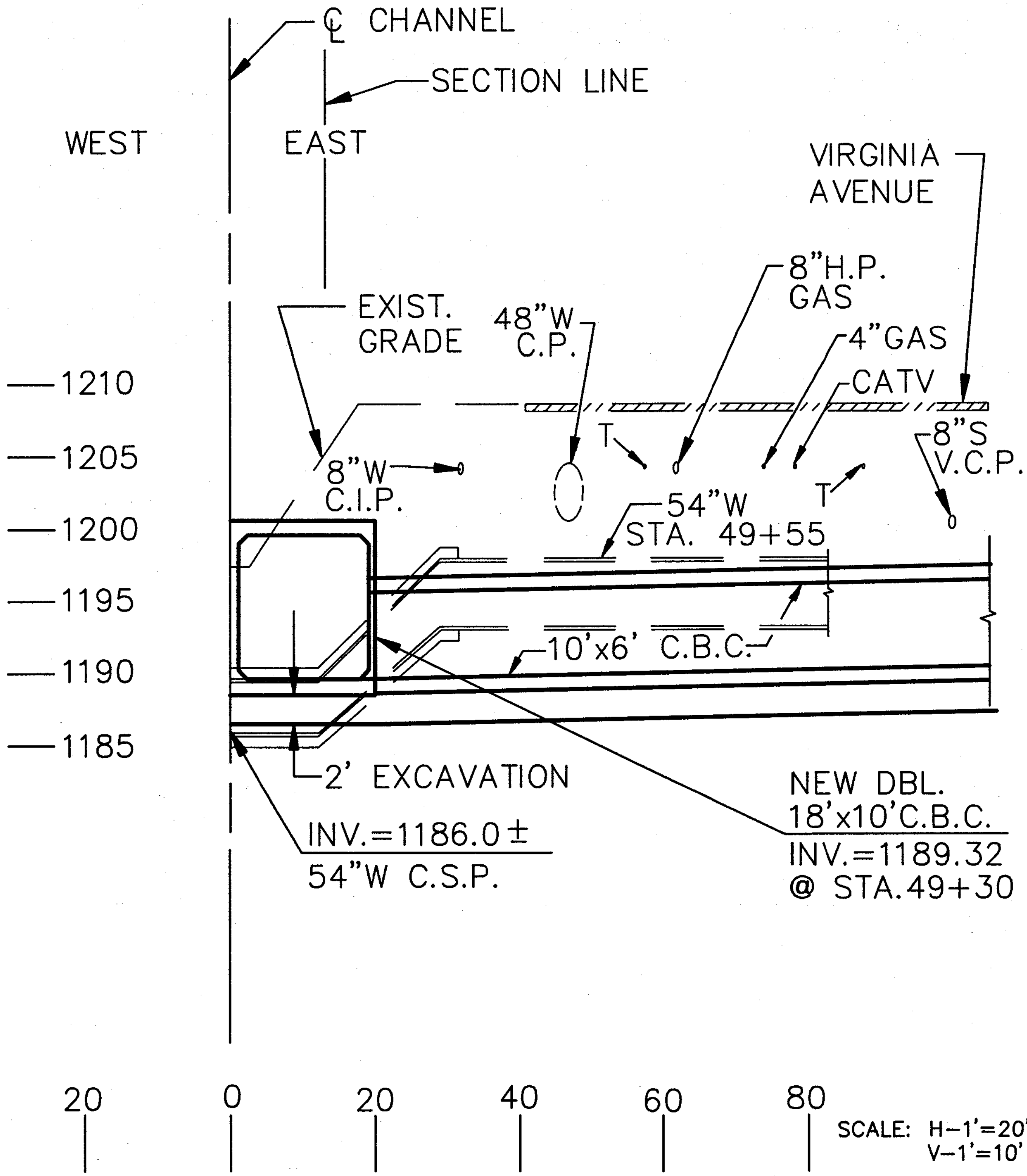
OLD CROSS CUT CANAL

Cross Section at Sta. 47+40

New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2-12



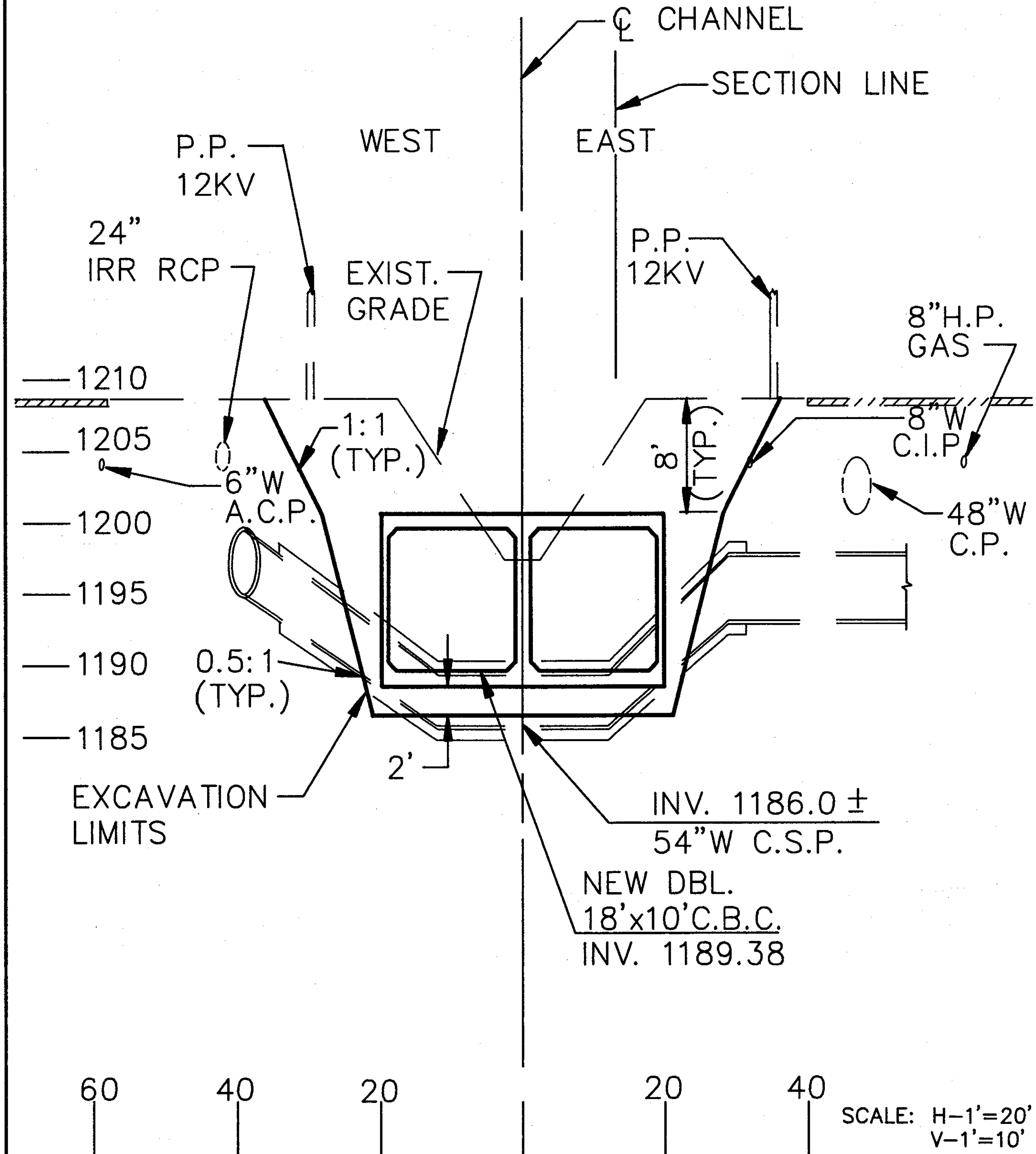
OLD CROSS CUT CANAL

Cross Section at Sta. 49+30

New Double 18'x10' C.B.C.

Greiner

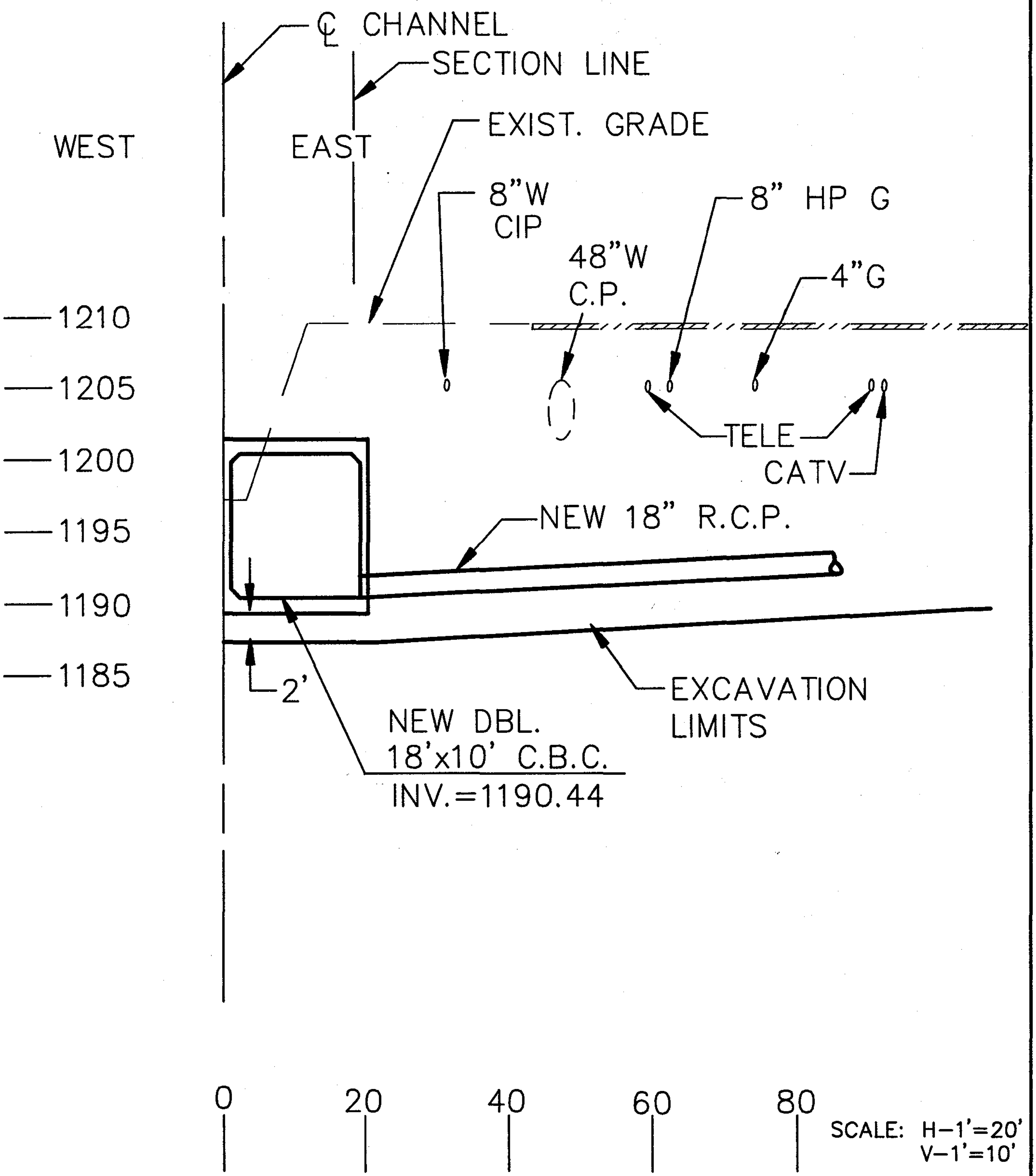
FIGURE 5.2–13



OLD CROSS CUT CANAL
Cross Section at Sta. 49+55
New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2-14



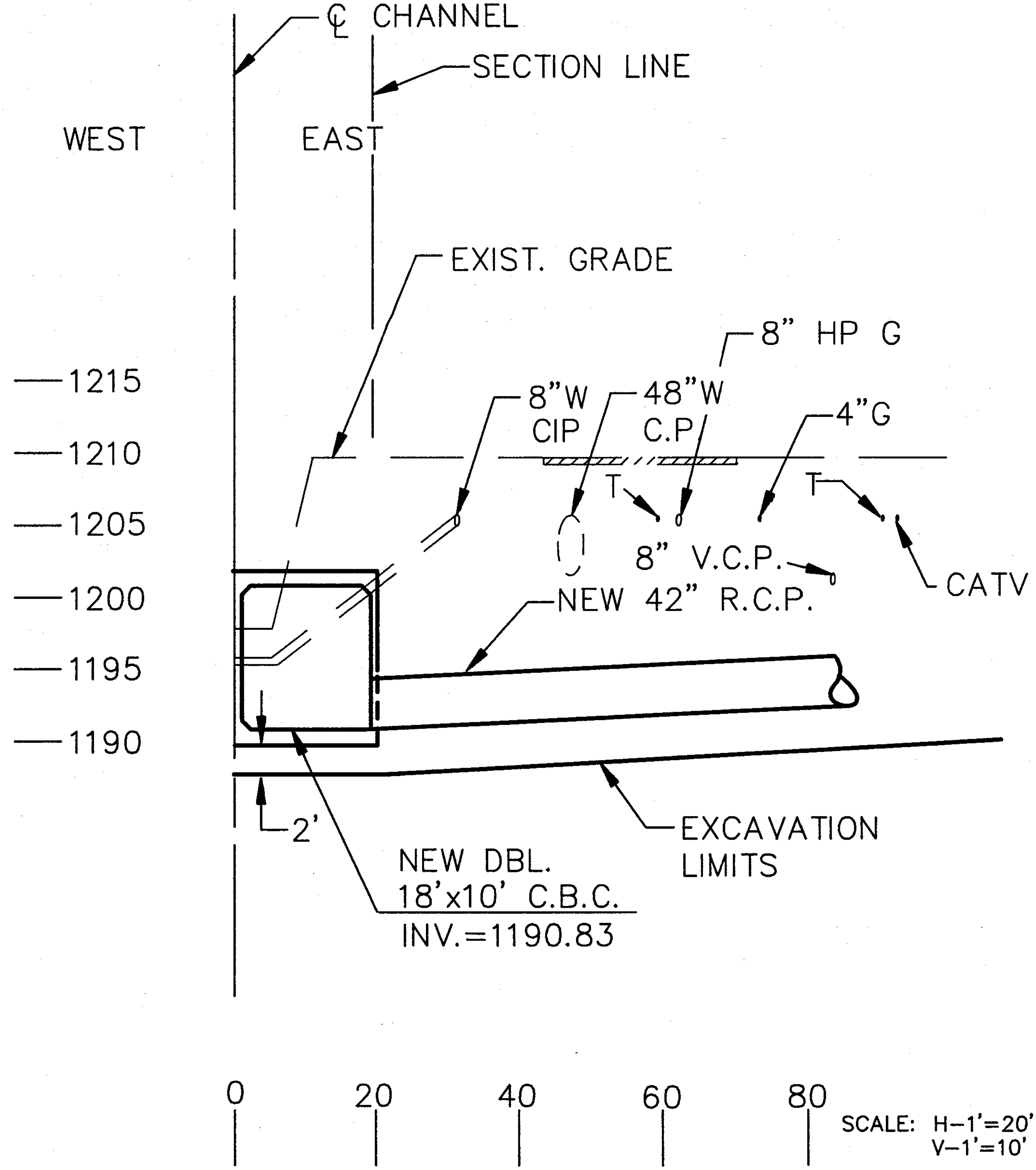
OLD CROSS CUT CANAL

Cross Section at Sta. 54+15

Stub-out Profile 18" Inlet

Greiner

FIGURE 5.2-15



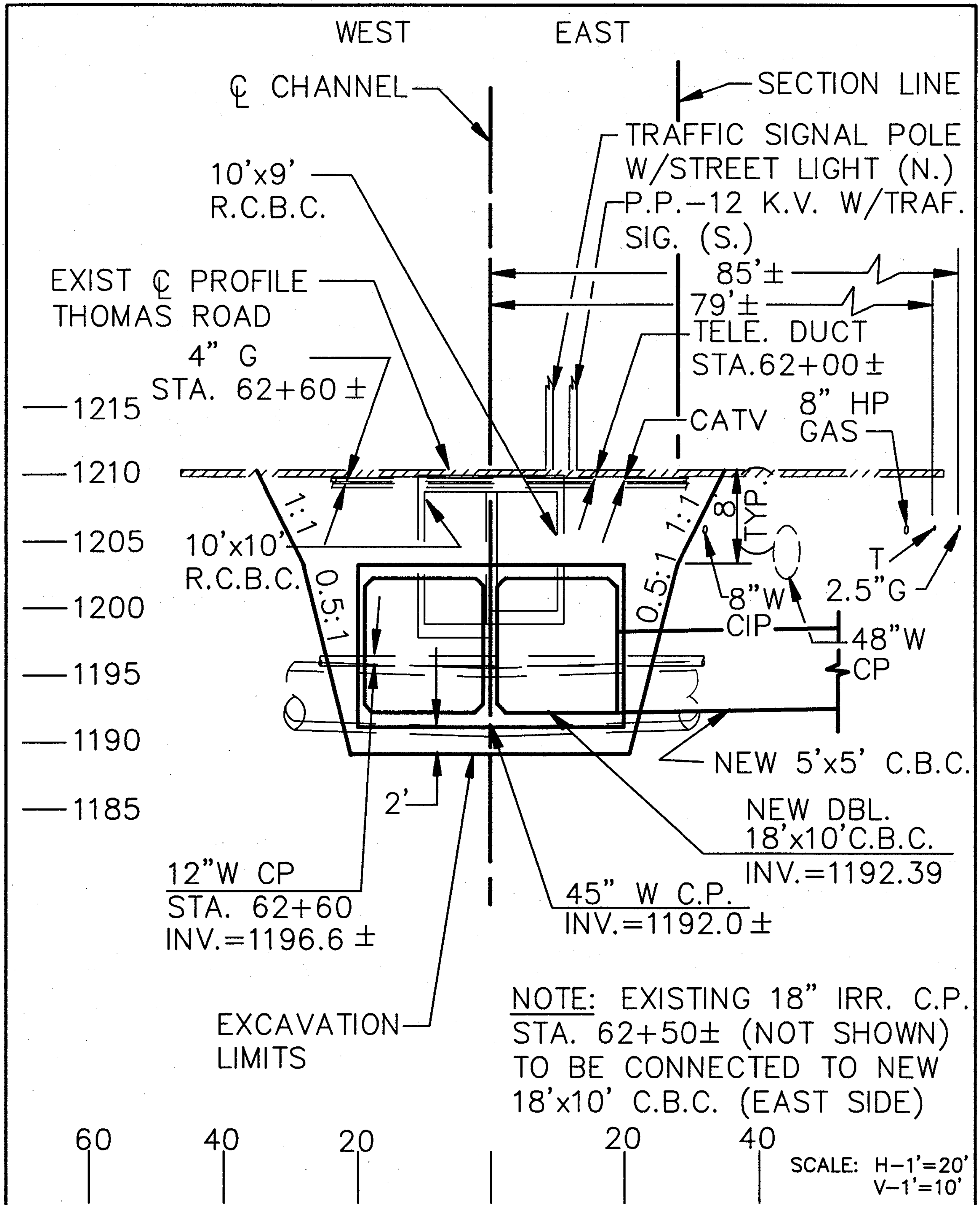
Greiner

OLD CROSS CUT CANAL

Cross Section at Sta. 55+75

Stub-out Profile 42" Inlet

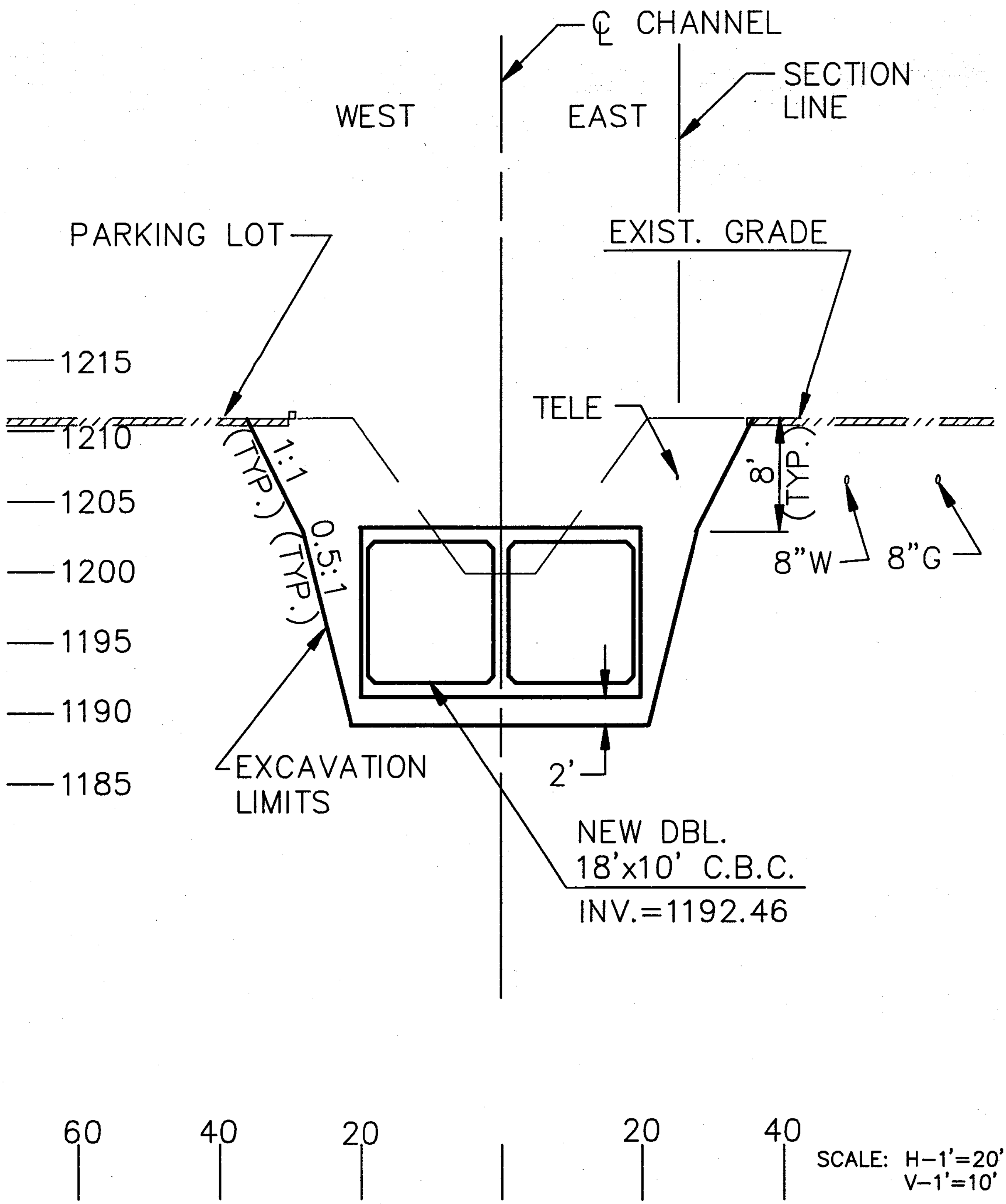
FIGURE 5.2-16



OLD CROSS CUT CANAL
Cross Section at Sta. 62+40
New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2-17



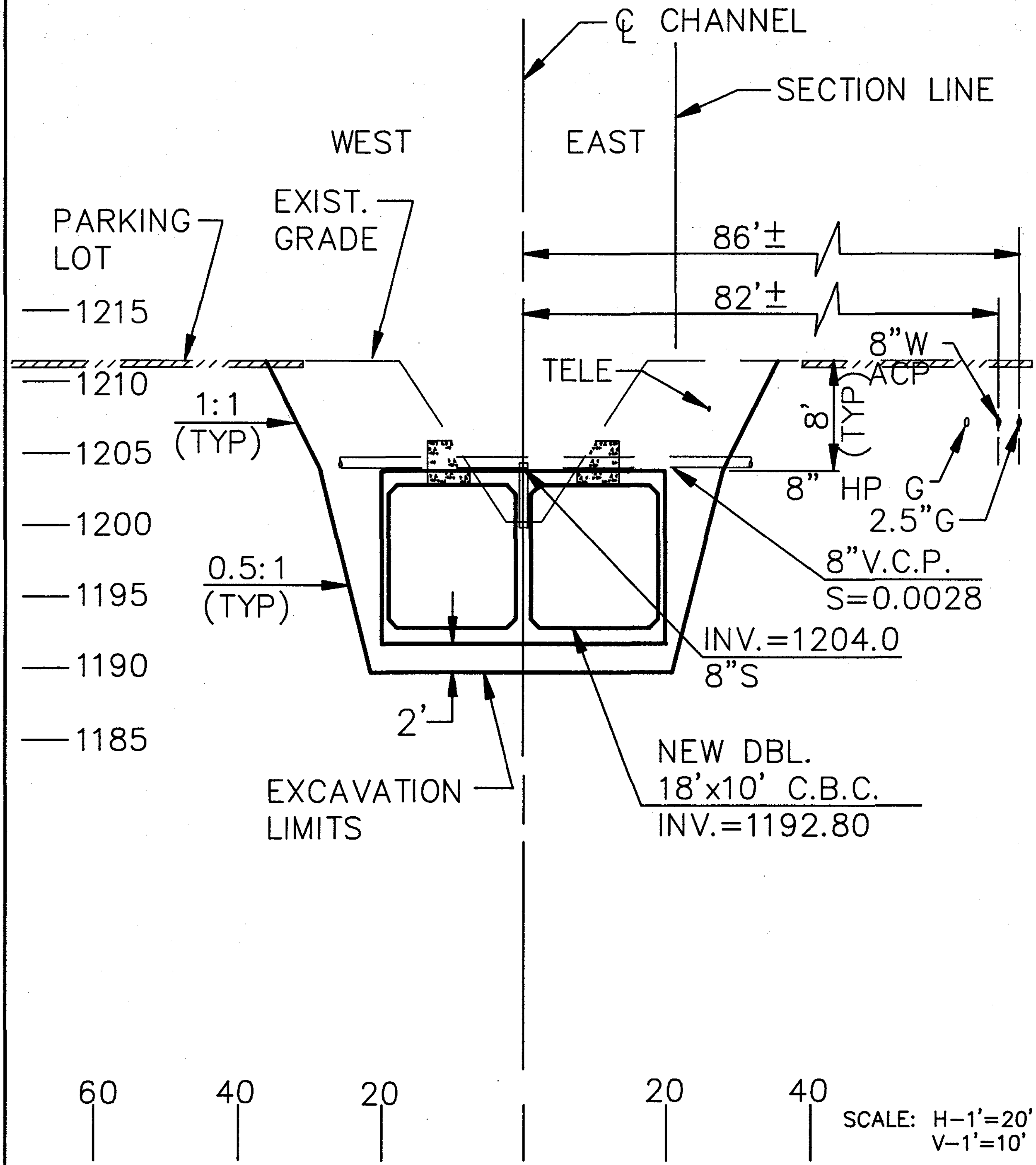
OLD CROSS CUT CANAL

Cross Section at Sta. 63+20

New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2-18



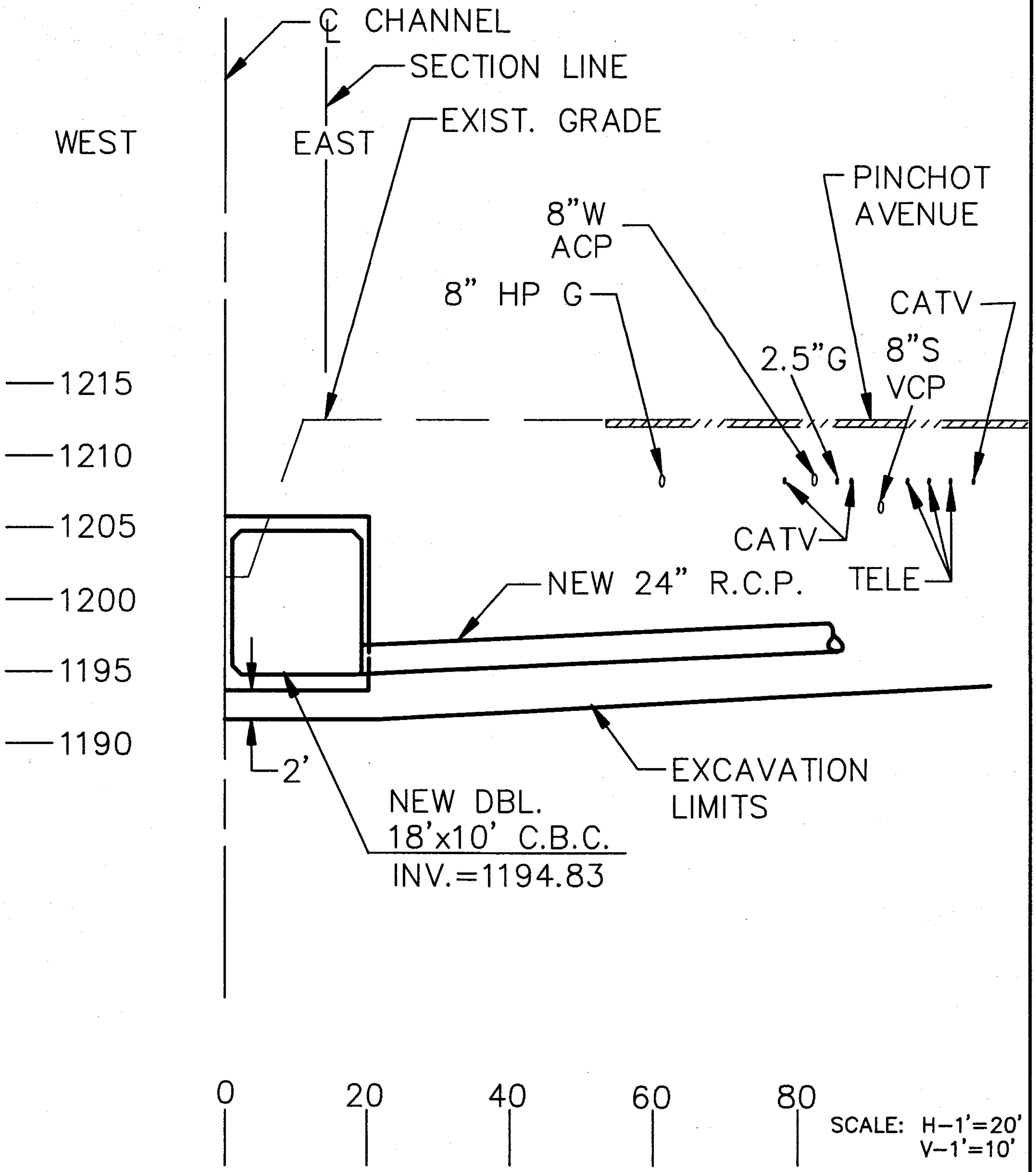
OLD CROSS CUT CANAL

Cross Section at Sta. 64+15

New Double 18'x10' C.B.C.

Greiner

FIGURE 5.2–19

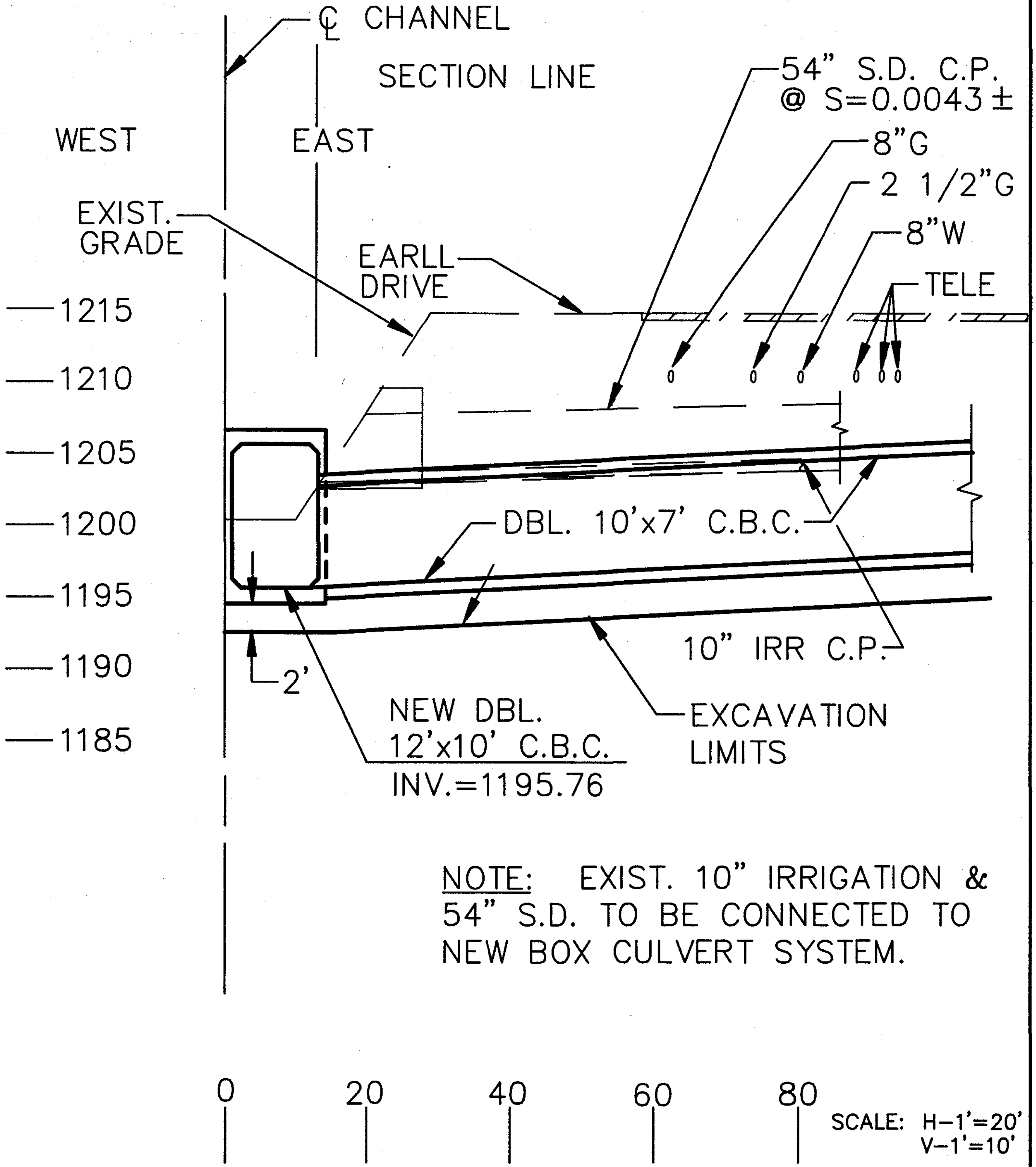


OLD CROSS CUT CANAL

Cross Section at Sta. 70+20
Stub-out Profile 24" Inlet

Greiner

FIGURE 5.2-20



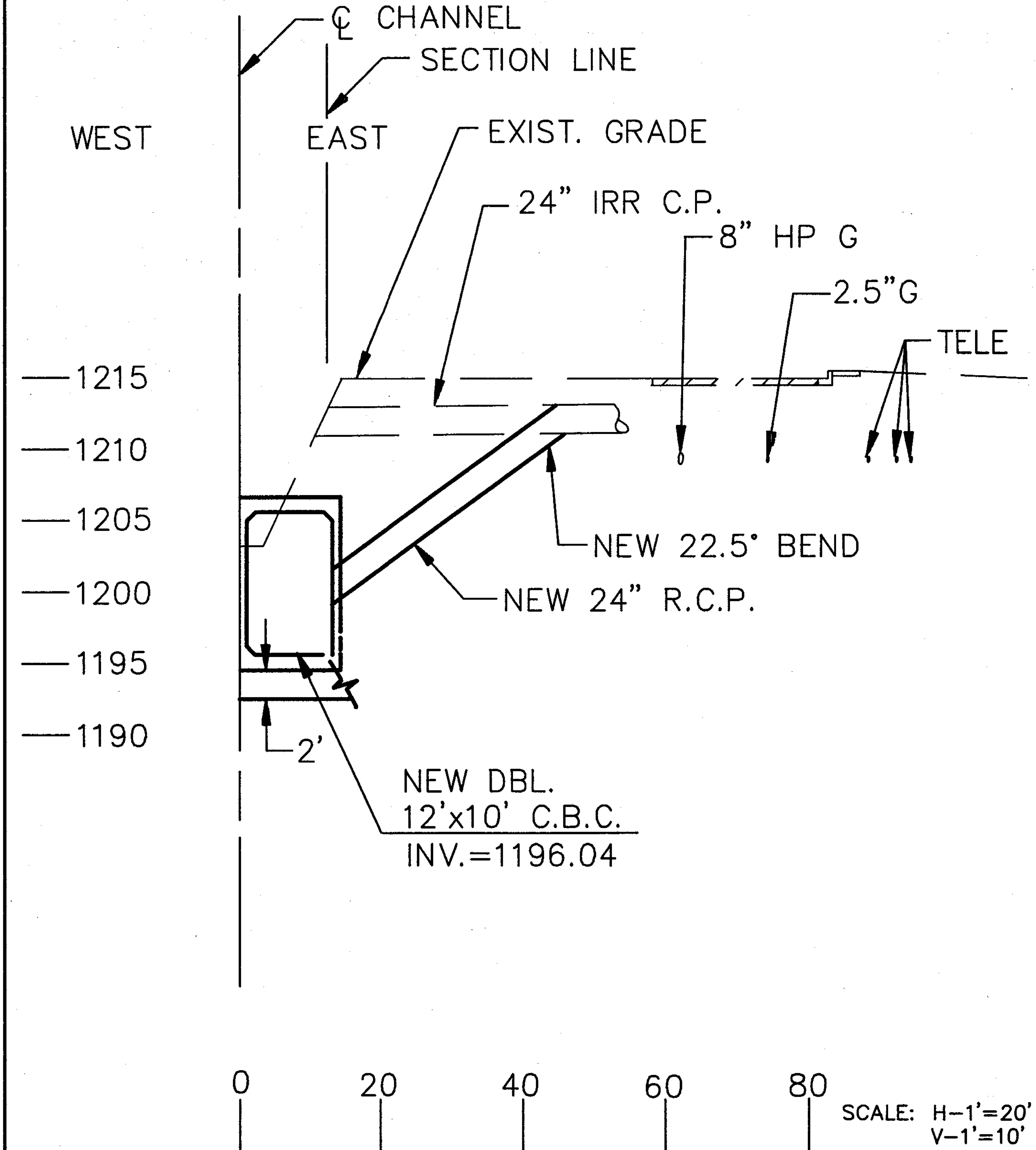
OLD CROSS CUT CANAL

Cross Section at Sta. 75+60

New Double 12'x10' C.B.C.

Greiner

FIGURE 5.2-21



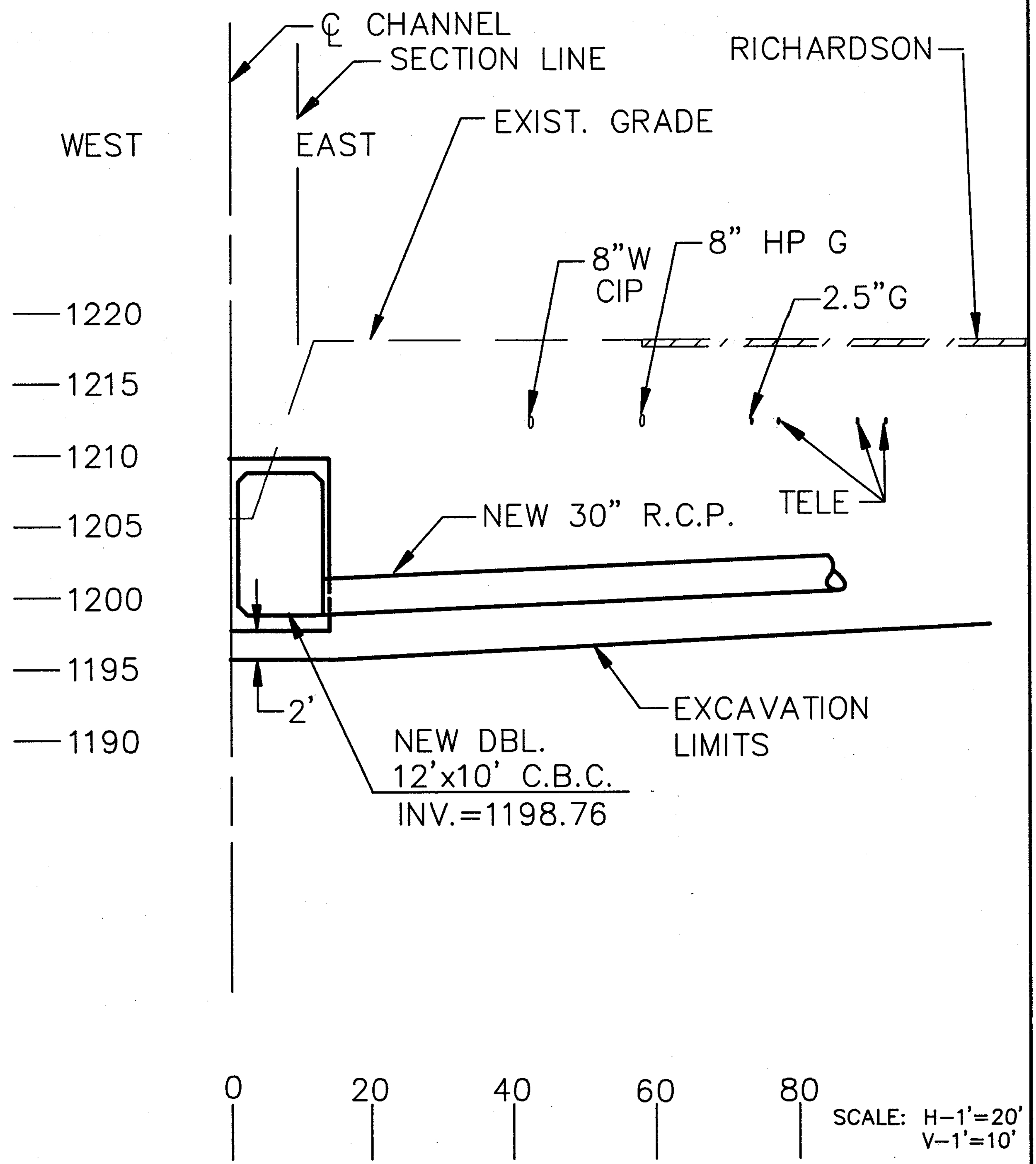
OLD CROSS CUT CANAL

Cross Section at Sta. 76+20

Stub-out Profile 24" Irr. Inlet

Greiner

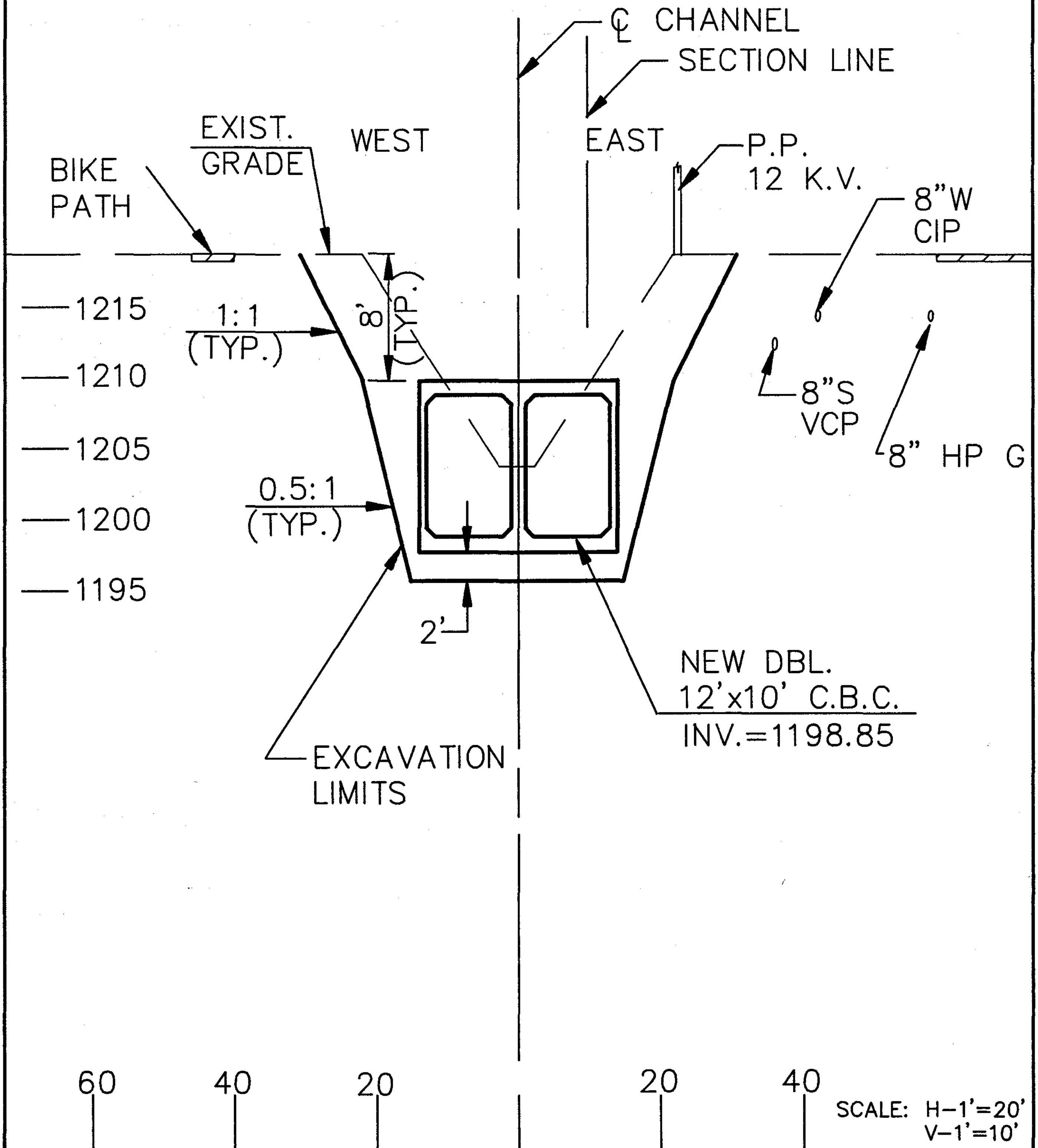
FIGURE 5.2-22



OLD CROSS CUT CANAL
Cross Section at Sta. 82+00
Stub-out Profile 30" Inlet

Greiner

FIGURE 5.2-23

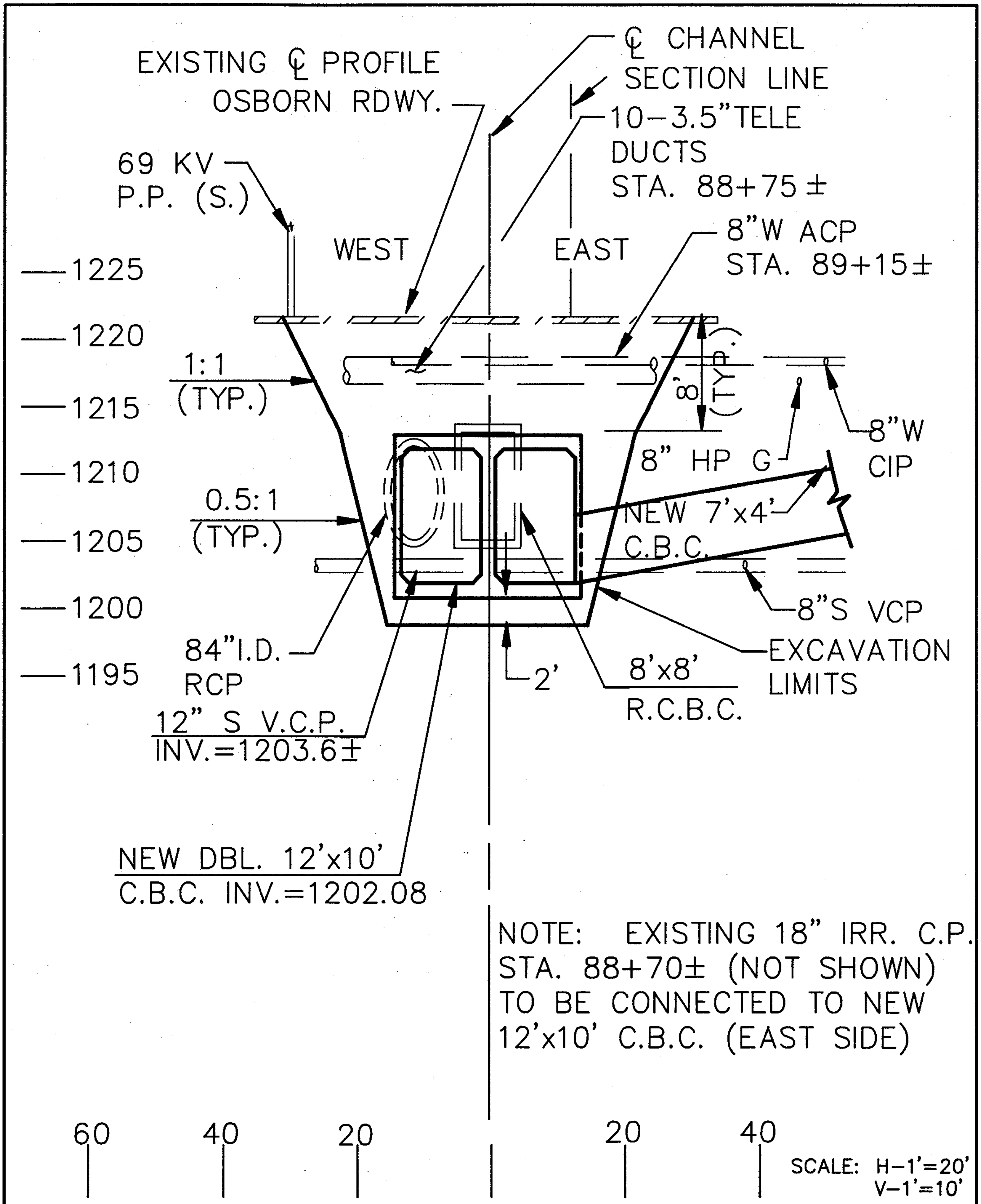


OLD CROSS CUT CANAL

Cross Section at Sta. 82+20
New Double 12'x10' C.B.C.

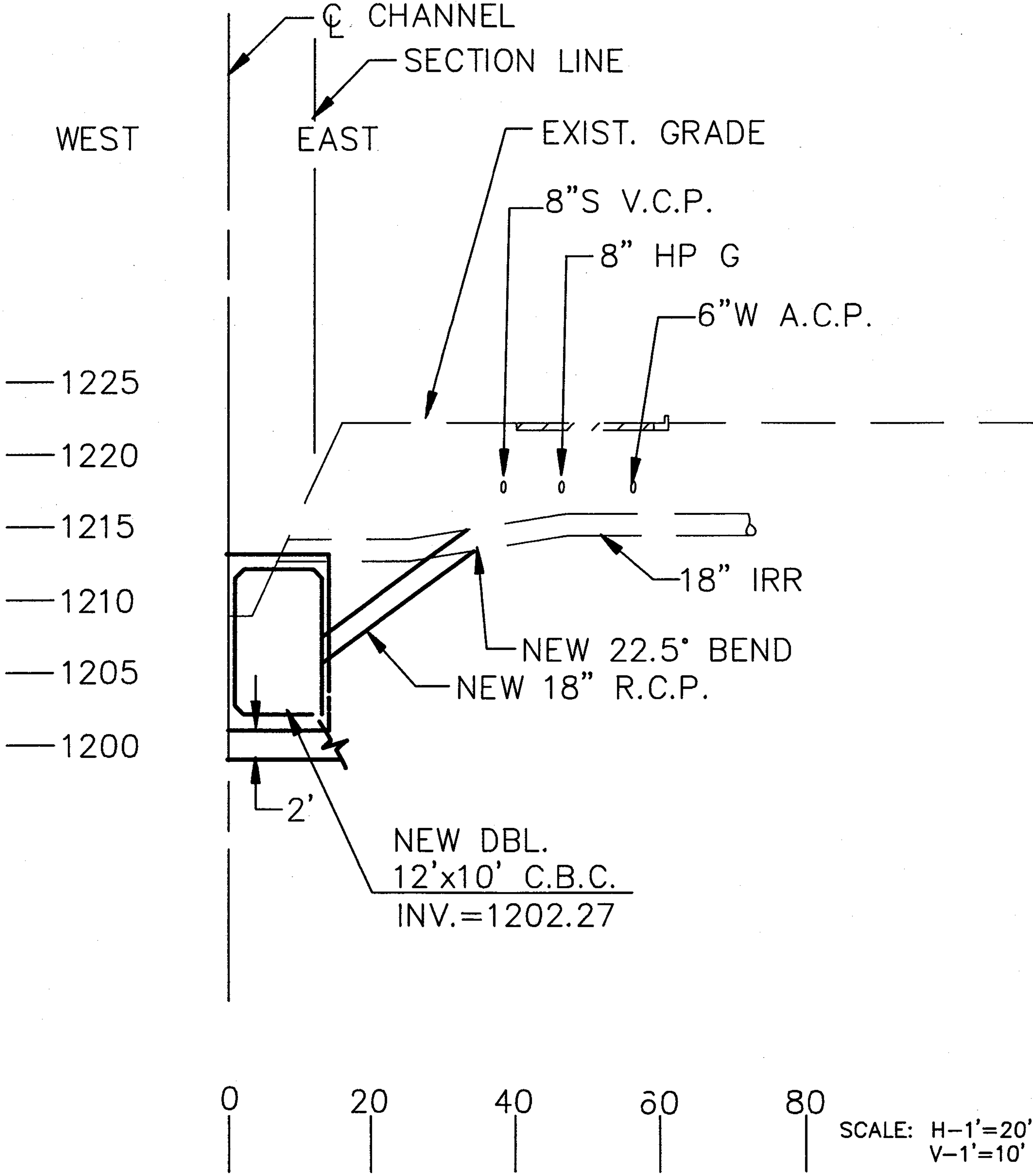
Greiner

FIGURE 5.2-24



OLD CROSS CUT CANAL
Cross Section at Sta. 89+10
New Double 12'x10' C.B.C.

FIGURE 5.2-25



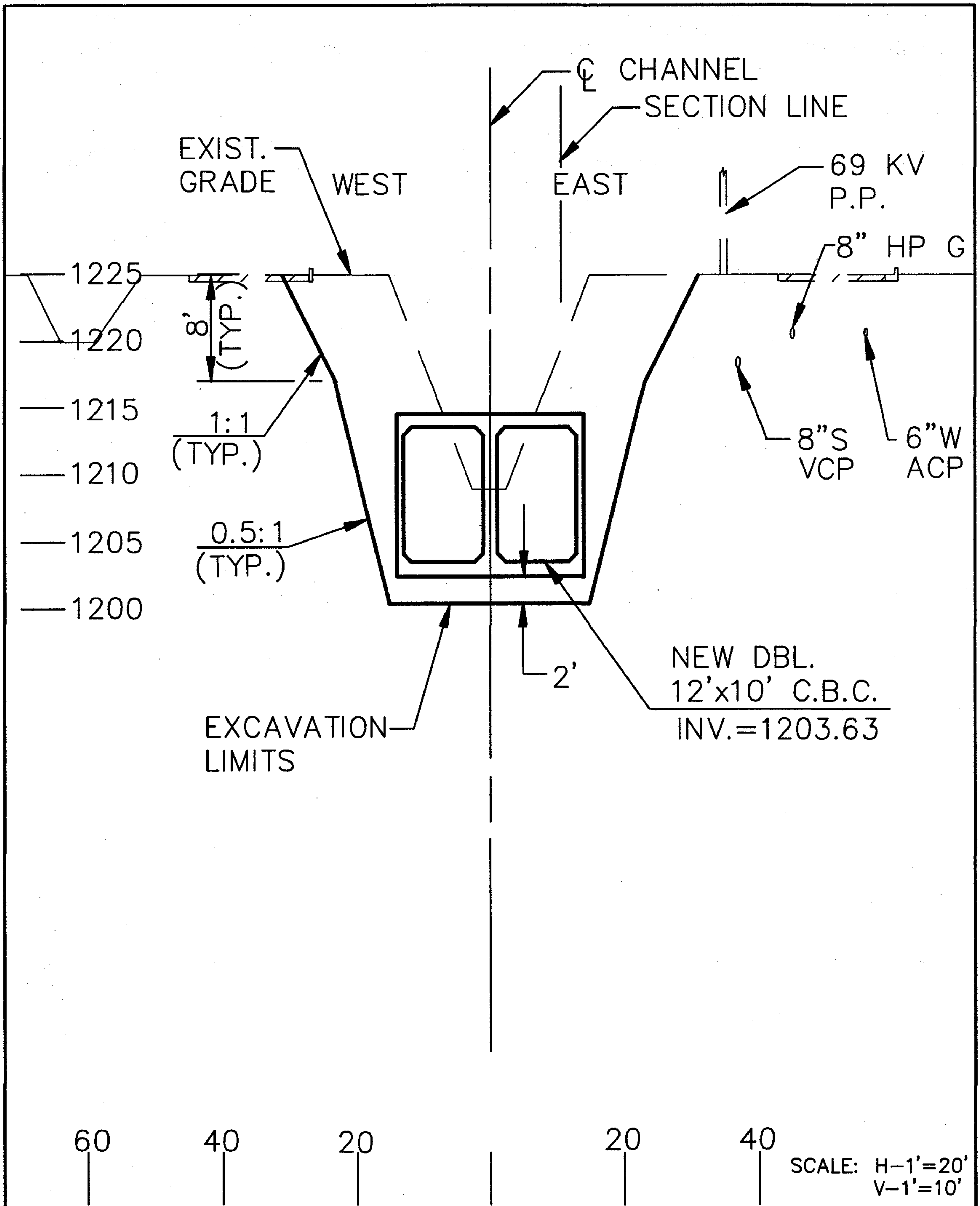
OLD CROSS CUT CANAL

Cross Section at Sta. 89+40

Stub–out Profile Exist. 18" Irr.

Greiner

FIGURE 5.2–26



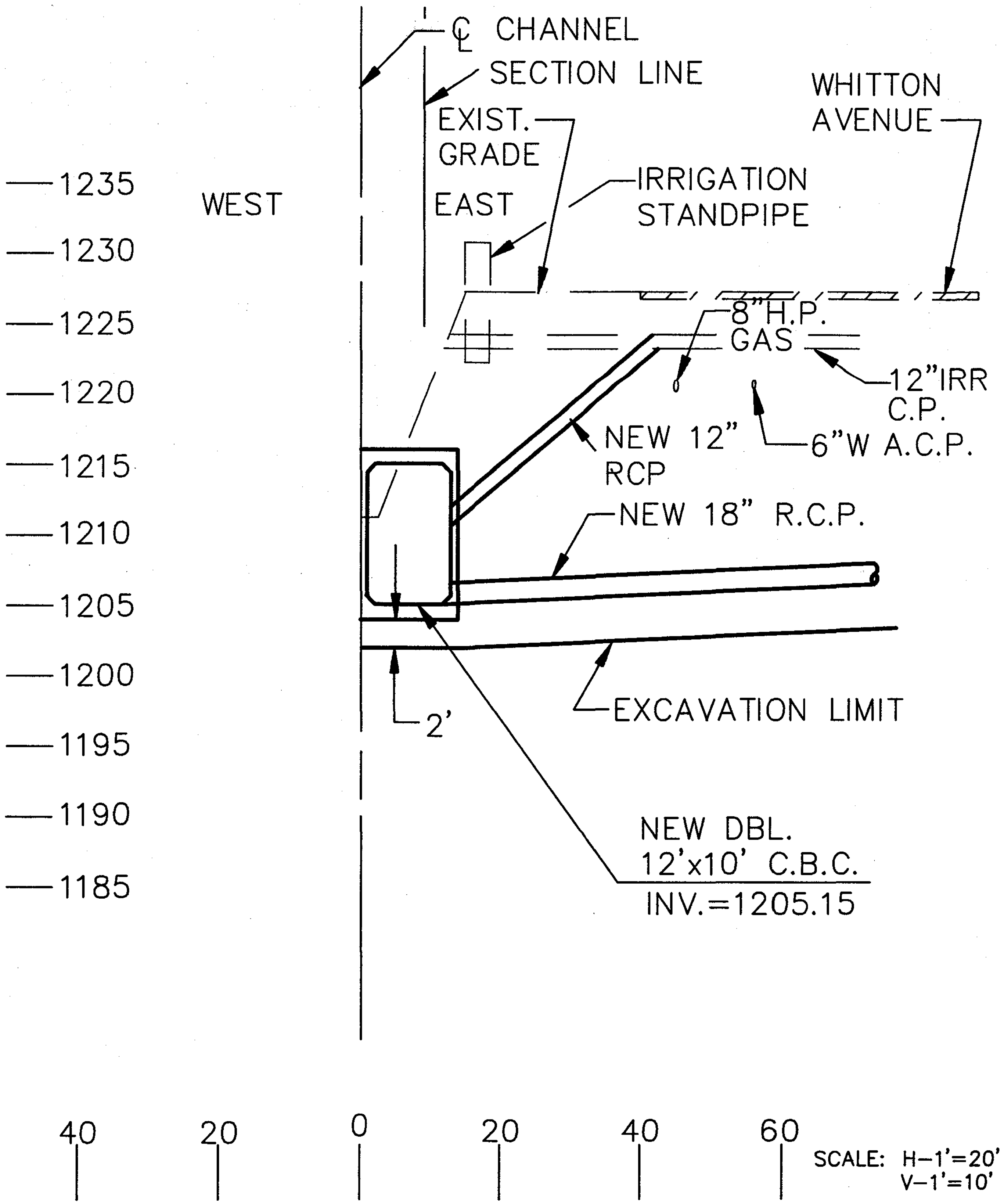
OLD CROSS CUT CANAL

Cross Section at Sta. 92+40

New Double 12'x10' C.B.C.

Greiner

FIGURE 5.2-27



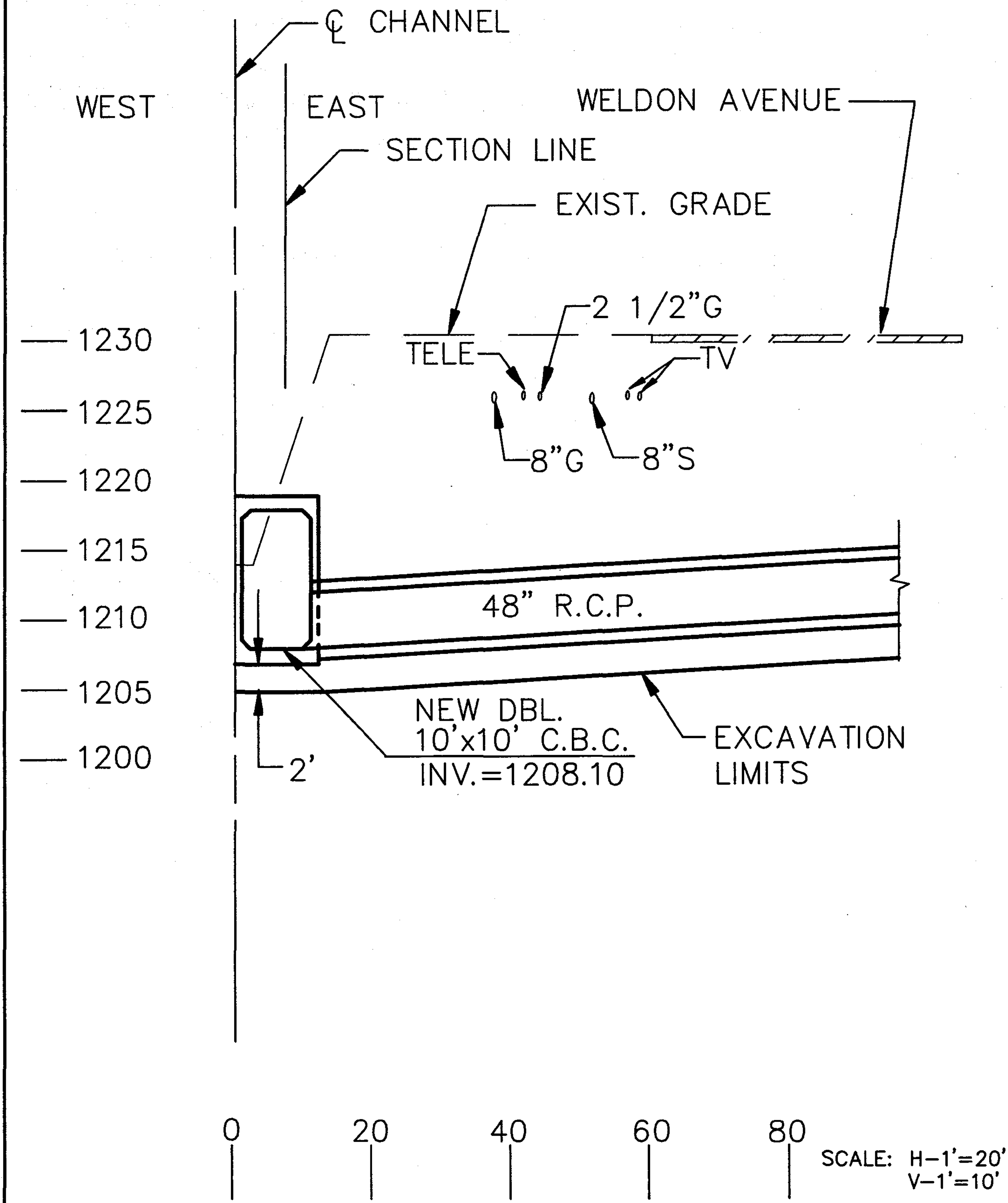
OLD CROSS CUT CANAL

Cross Section at Sta. 95+65

New Double 12'x10' C.B.C.

Greiner

FIGURE 5.2-28



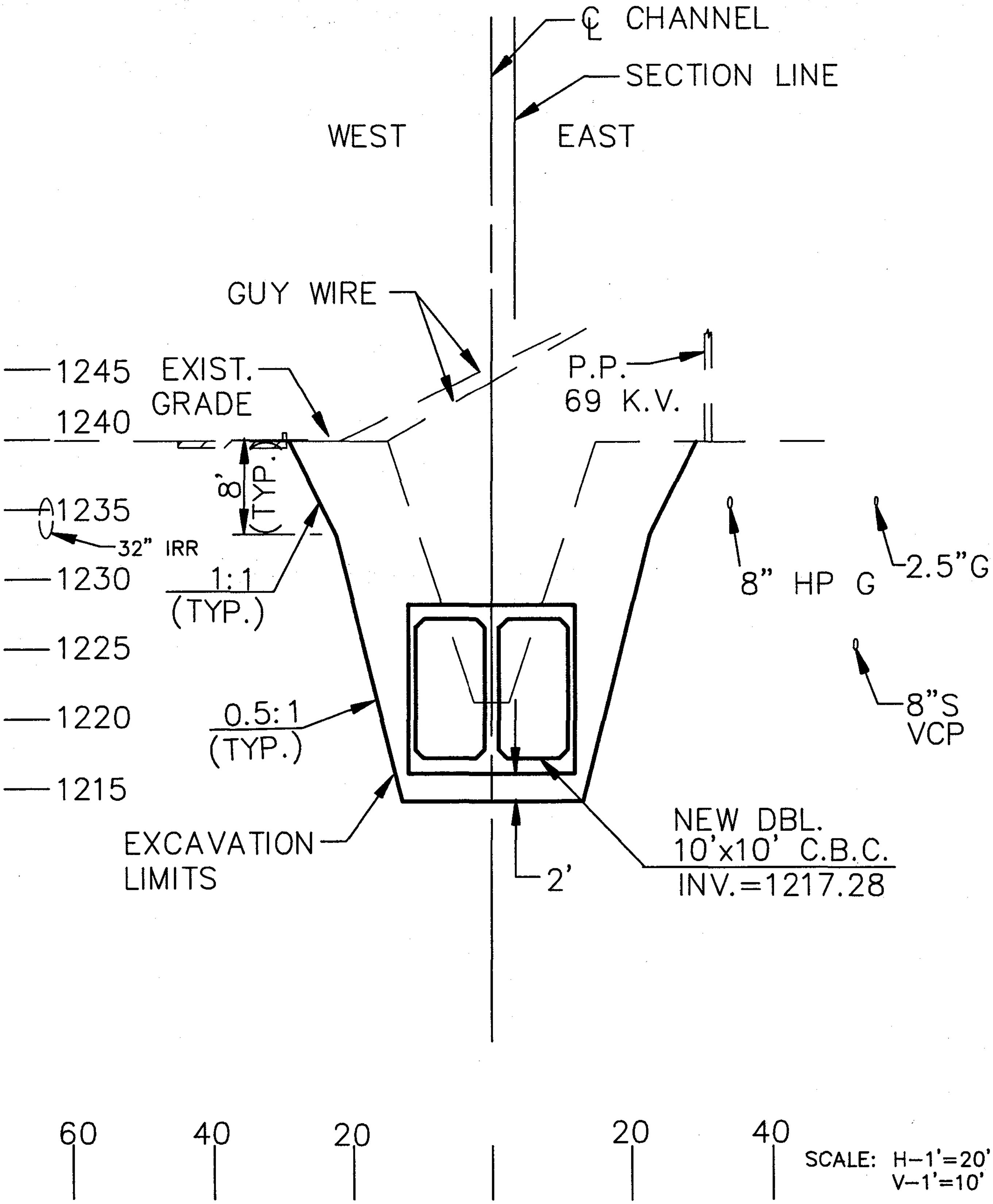
OLD CROSS CUT CANAL

Cross Section at Sta. 99+40

New Double 10'x10' C.B.C.

Greiner

FIGURE 5.2–29

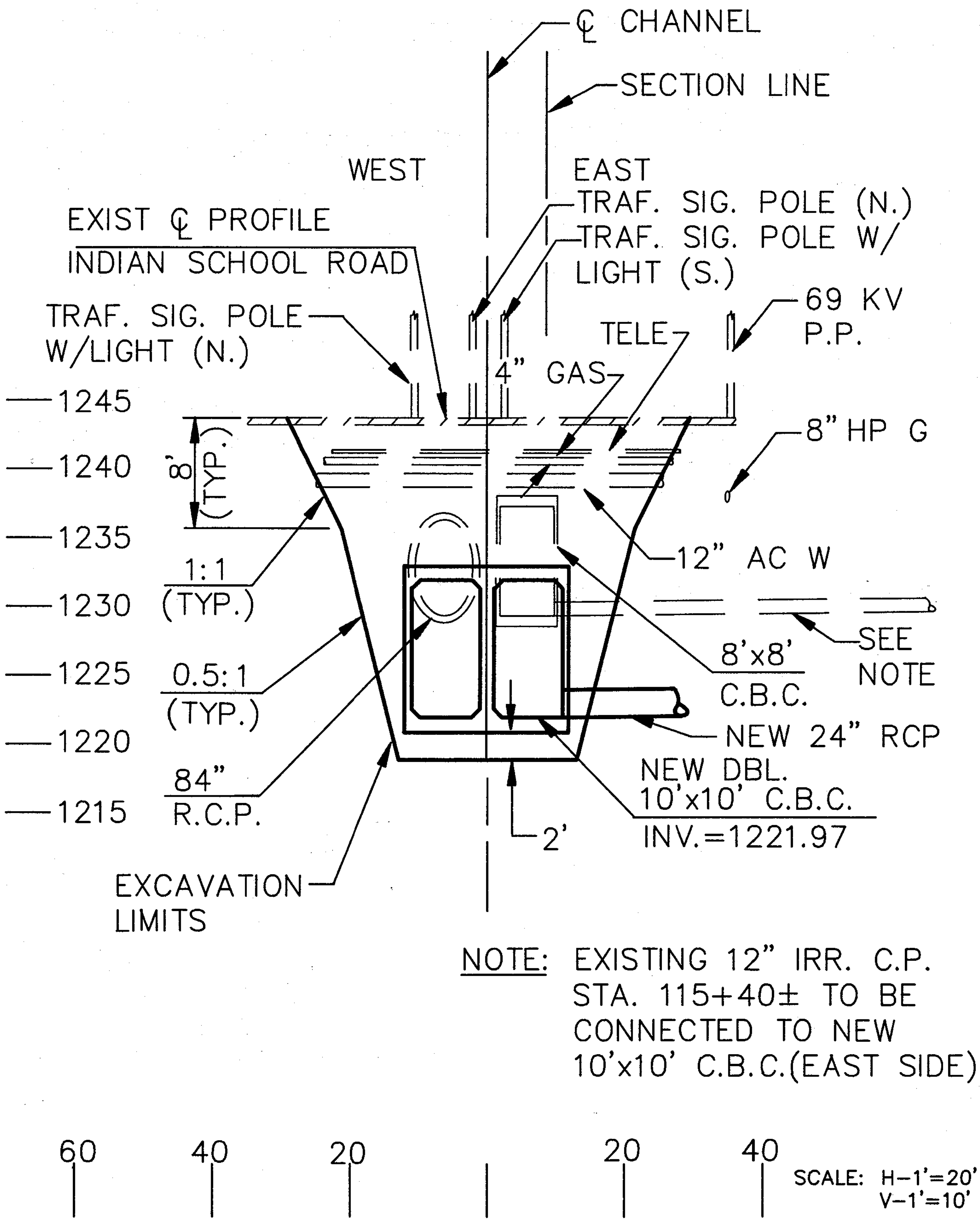


OLD CROSS CUT CANAL

Cross Section at Sta. 110+00
New Double 10'x10' C.B.C.

Greiner

FIGURE 5.2-30

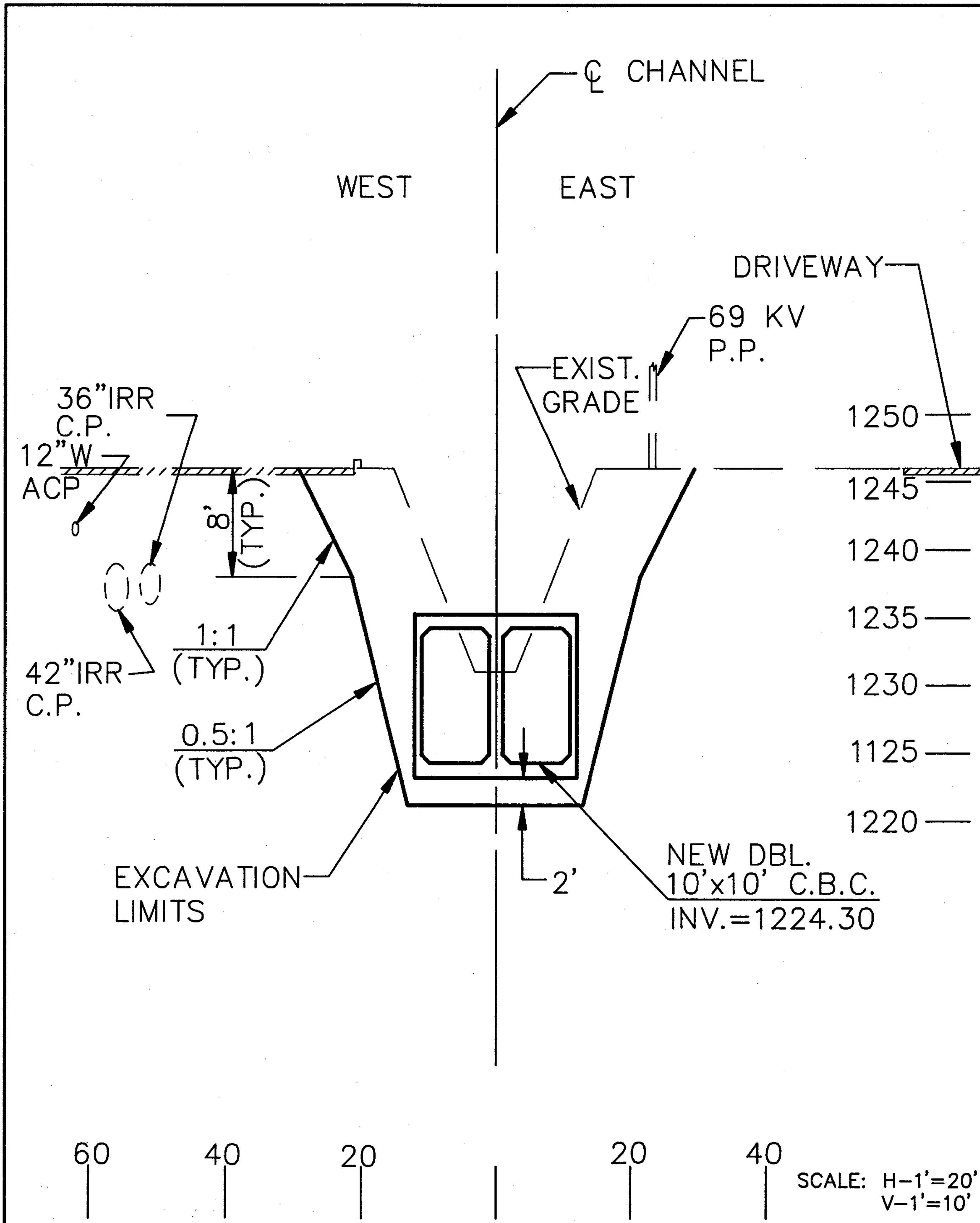


OLD CROSS CUT CANAL

Cross Section at Sta. 115+40
New Double 10'x10' C.B.C.

Greiner

FIGURE 5.2-31



OLD CROSS CUT CANAL

Cross Section at Sta. 118+10
New Double 10'x10' C.B.C.

Greiner

FIGURE 5.2-32

FIGURE
5.2-33

Greiner

5.3 STRUCTURAL ALTERNATIVES

Alternative pipe products were evaluated in terms of cost, performance, constructibility and compatibility with site conditions. The following pipe products were evaluated.

- o Cast-in-Place Concrete Box Culvert (CIPCBC)
- o Precast Concrete Box Culvert (PCBC)
- o Prestressed Concrete Cylinder Pipe (PCCP)
- o Cast-in-Place Pipe (CIPP)

A. Key features of this project that were considered with regard to pipe products are as follows:

- o Length of project = 11,000'±
- o Curved sections north of McDowell Road (Radius = 1500'±) and north of Indian School Road (Radius = 600'±)
- o Parallel and crossing utilities, including overhead electric power lines
- o Pressure and non-pressure flow
- o Limited right-of-way
- o Required 4' minimum cover to counteract uplift from groundwater
- o Major street undercrossings

The following is a discussion of the advantages and limitations of the various products.

5.3.1 Cast-in-Place Concrete Box Culvert

Preliminary hydraulic investigations indicate the need for three different box culvert configurations. The upstream reach, which extends from approximately 200' north of Indian School Road, south to approximately 800' north of Osborn Road (L=2,000'±), will require double 10' x 10' box culverts. The middle reach, which extends from approximately 800' north of Osborn Road, south to approximately 1,200' north of Thomas Road, (L=2,200'±) will require double 12' x 10' box culverts. The downstream reach, which extends from approximately 1,200' north of Thomas Road, south to the existing box culverts at McDowell Road (L=6,600'±), will require double 18' x 10' box culverts.

Advantages of CIPCBC are as follows:

- o Double 18' x 10' size can be constructed using standard hand forming or possibly utilizing rolling forms
- o Locally available
- o Accepted for pressure flow
- o Suitable for local soil conditions
- o Meets cover requirements
- o Meets curvilinear requirements

Disadvantages of CIPCBC are as follows:

- o Quality control/field inspection is intensive
- o Length of time for open trench is greater than the precast options
- o Overall construction time is normally greater than the precast options

5.3.2 Precast Concrete Box Culvert

Manufactured precast box culvert sizes are 10' x 10', 10' x 11', 10' x 12', 11' x 11', and 12' x 12'. As previously mentioned, the upstream reach requires a double 10' x 10' section, the middle reach requires a double 12' x 10' section, and the downstream reach requires a double 18' x 10' section. Due to cover requirements and existing utility undercrossings, it is desirable to use only 10' high box culverts.

Local manufacturers cannot construct precast 18' x 10' concrete box structures. Therefore, it would be necessary to install triple 12' x 10' box culverts at the downstream reach. This would, however, degrade hydraulic efficiency and require complicated and expensive structures to transition from double 12' x 10' box culverts to triple 12' x 10' box culverts and to connect side inlets.

Advantages of PCBC are as follows:

- o Quality control/field inspection is less intensive than cast-in-place
- o Short length of time for open trench
- o Shorter overall construction time
- o Locally available
- o Accepted for pressure flow
- o Suitable for local soil conditions
- o Can accommodate curvilinear deflection requirements

Disadvantages of PCBC are as follows:

- o Manufactured sizes too small for downstream reach
- o Requires complicated and expensive transition and junction structures
- o Requires use of a crane

5.3.3 Prestressed Concrete Cylinder Pipe

Preliminary hydraulic investigations indicate that the upstream reach of the canal would require the installation of double 120" diameter pipes, then transition to double 144" diameter pipes for the middle reach, then transition to double 156" diameter pipes for the downstream reach. As previously mentioned, cover requirements and existing utility constraints dictate using a 10' diameter pipe (120"). Thus, the middle reach would require using three 120" pipes and the downstream reach would require using four 120" pipes to carry the flows.

Advantages of PCCP are as follows:

- o Strength
- o Accepted for pressure flow
- o Locally available
- o Suitable for local soil conditions
- o Relatively short open trench time
- o Can accommodate curvilinear deflection requirements

Disadvantages of PCCP are as follows:

- o Increased trench width due to multiple pipe requirements
- o Requires use of a crane
- o Intensive backfill requirements
- o Multiple conduits would require transition and junction structures

5.3.4 Cast-in-Place Pipe

CIPP was initially considered as an alternative but was eliminated due to the required multiple large diameter pipe installations, which negate most of the advantages for using cast-in-place pipe. Also, CIPP is not recommended for pressure flow applications.

5.3.5 Cost Comparison

The data below provides a cost comparison between the pipe products discussed. Unit costs do not include excavation and backfill or any junction or transition structures.

Product	Size	Unit Cost	Length	Total Cost
CIPCBC	Dbl. 10' x 10'	$678.00/LF	2000 LF	$10,976,600.00
	Dbl. 12' x 10'	$815.00/LF	2200 LF	
	Dbl. 18' x 10'	$1,186.00/LF	6600 LF	
PCBC	Dbl. 10' x 10'	$590.00/LF	2000 LF	$9,452,000.00
	Dbl. 12' x 10'	$700.00/LF	2200 LF	
	Three 12' x 10'	$1,020.00/LF	6600 LF	
PCCP	Dbl. 120"	$620.00/LF	2000 LF	$11,470,000.00
	Three 120"	$930.00/LF	2200 LF	
	Four 120"	$1,240.00/LF	6600 LF	

These cost estimates were based on a preliminary design of the box culverts that used a combination of dead load and HS-20-44 live loading to determine the wall and top and bottom slab concrete thickness and steel reinforcement requirements for the cast-in-place alternative. Elimination of the HS-20-44 live load would result in a material cost reduction for the double 12' x 10' section and the double 18' x 10' section of approximately seven percent and ten percent respectively.

5.3.6 Recommended Pipe Product

The following table ranks the three alternative pipe products with respect to cost, durability, hydraulic performance, constructibility, and utility and site impacts.

Product	Cost	Durability	Hydraulic Performance	Constructibility	Utility & Site Impacts	Total
CIPCBC	0	+	0	0	0	+1
PCBC	+	+	-	+	0	+2
PCCP	-	+	+	-	0	0

From the above comparison and the previous cost comparison, CIPCBC emerges as the best product for this project, except for the fact that the three-barrel downstream reach required for this alternative is not conducive to hydraulic efficiency.

It is our recommendation that the plans and specifications for this project be prepared following investigation of a combination of the CIPCBC and PCBC. A possible combination of the two products is to use precast box culvert at the middle and upstream reaches, and cast-in-place box culvert for the downstream reach.

5.4 ALTERNATIVE SOLUTIONS

Alternatives were formulated in seeking minimizing utility conflict and potential cost savings.

5.4.1 Alternative Configurations

The alternative configurations for the drainage improvements may include:

1. Existing McDowell Road culverts, with or without modification

2. Transition Structure No. 1, McDowell Road culvert to Segment 1 culvert

3. Segment 1 culvert, from McDowell Road to Virginia Avenue, includes the following alternatives:

   A. Two 16' x 10' box culverts at sf of 0.30 percent
   B. Two 14' x 12' box culverts at sf of 0.30 percent

C. Three 12' x 10' box culverts at sf of 0.32 percent
D. Two 18' diameter pipes at sf of 0.08 percent
E. Three 14' diameter pipes at sf of 0.32 percent
F. One 16' x 18' box culvert at sf of 0.3 percent

4. Transition Structure No. 2, Segment 1 culvert to Segment 2 culvert

5. Segment 2 culvert, from Virginia Avenue to Avalon Drive, includes the following alternatives (the downstream transition structure will be eliminated if Segment 1 and Segment 2 are the same size):

   A. Two 16' x 10' box culverts at sf of 0.29 percent
   B. Two 14' x 12' box culverts at sf of 0.24 percent
   C. Three 12' x 10' box culverts at sf of 0.27 percent
   D. Two 18' diameter pipes at sf of 0.075 percent
   E. Three 14' diameter pipes at sf of 0.32 percent
   F. One 16' x 16' box culvert at sf of 0.32 percent

6. Transition Structure No. 3, Segment 2 culvert to Segment 3 culvert

7. Segment 3 culvert, from Avalon Drive to Whitton Avenue, includes the following alternatives:

   A. Two 16' x 8' box culverts at sf of 0.54 percent
   B. Two 10' x 12' box culverts at 0.60 percent
   C. Three 10' x 10' box culverts at sf of 0.43 percent
   D. Two 12' diameter pipes at sf of 0.60 percent
   E. Three 10' diameter pipes at sf of 0.6 percent
   F. One 16' x 14' box at sf of 0.45 percent

8. Transition Structure No. 4, Segment 3 culvert to Segment 4 culvert

9. Segment 4 culvert, from Whitton Avenue to the Arizona Canal, includes the following alternatives.

   A. Two 16' x 10' box culverts at sf of 0.68 percent
   B. Two 12' x 8' box culverts at sf of 0.77 percent
   C. Three 10' x 8' box culverts at sf of 0.55 percent
   D. Two 12' diameter pipes at sf of 0.41 percent
   E. Three 10' diameter pipe at sf of 0.80 percent
   F. One 16' x 14' box at sf of 0.80 percent

10. Transition Structure No. 5, Segment 4 culvert to Segment 5 culvert and the Arizona Canal flood relief gate

11. Segment 5 culvert, the Arizona Canal to Camelback Road, includes the following alternatives:

    The preliminary culvert size includes a 10' x 10' box culvert at sf of 0.70 percent.

5.4.2 Finalize Alternative Systems

Section 3.3, Existing Culvert Analysis, indicates that a higher headwater is required at the existing McDowell Road culvert to handle the proposed design flow. Review of the 1990 McDowell Road culvert reconstruction design documents indicate that the functioning of the reconstructed culvert is based upon an upstream open channel to be constructed. Further review of the preliminary drainage concept, drafted by the U.S. Army Corps of Engineers, indicates that one of the improvement schemes was based on a supercritical flow channel at various widths of 16', 23.5' and 26'. The downstream ADOT channel improvements adopt the open channel concept and a minimum channel width of 16' at a slope of approximately 0.533 percent.

Modifications to the preliminary scheme presentation, Section 5.4.1, are required to mitigate the higher headwater elevation produced at the McDowell Road culvert. These may include:

1. Alternative 1 - Existing McDowell Road Culvert Taper to 16' Upstream Channel and Box

   Attempts were made to develop drainage schemes that will fit the McDowell Road culvert reconstruction design concept. The existing ground grades do not allow a supercritical flow channel construction because the upstream limitation. However, a 16' concrete channel at a slope of 0.23 percent with a transition structure tapering to 26' at the upstream face of the McDowell Road culvert and connected to a 16' x 20' box culvert may produce an acceptable hydraulics performance.

2. Alternative 2 - Existing McDowell Road Culvert Taper to 23.5' Upstream Channel and Box

   A 23.5' concrete channel at a slope of 0.2661 percent with a transition structure tapering to 26' feet at the upstream face of McDowell Road culvert and connected to a 23.5' x 14' box culvert may produce an acceptable hydraulics performance. The flow will be performing in the critical flow regime.

3. Alternative 3 - Existing McDowell Road Culvert Taper to 26' Upstream Channel and Box

A 26' concrete channel at a slope of 0.26 serving as a connector of McDowell Road culvert and a 26' x 12' box culvert may produce an acceptable hydraulics performance. The flow will be performing in the critical flow regime.

4. Alternative 4 - Existing McDowell Road Culvert Taper to Two Upstream 18' x 10' Box Culverts

   This alternative will require up-sizing Segments 1 and 2 mainline box culvert from two 16' x 10' to two 18' x 10' from the first trial size for approximately 6,700 feet.

5. Alternative 5 - Existing McDowell Road Culvert Taper to Two Upstream 16' x 10' Box Culverts by Constructing an Additional Box Culvert

   This alternative will require the addition of a 10' x 10' box culvert to the existing McDowell Road culvert. A one-mile long downstream transition channel may also be required. This transition channel is to alleviate the choking effect from the 16' channel. The transition may be constructed by replacement of the west wall of the ADOT channel.

6. Alternative 6 - Existing McDowell Road Culvert Taper to Two Upstream 16' x 10' Box Culverts by Constructing an Additional Siphon

   This alternative will require the addition of a 12' diameter pipe siphon to the existing McDowell Road culvert. A one-mile long downstream transition channel may also be required. This transition channel is to alleviate the choking effect from the 16' channel. The transition may be constructed by replacement of the west wall of the ADOT channel.

Cost comparisons for Alternative 1 and Alternative 4 are based on cast-in-place main line construction only and do not include excavation or backfill.

Alternative 1:	$12,278,500.00
Alternative 4:	$10,976,600.00

Advantages of Alternative 1 are as follows:

- o Less excavation and backfill
- o Parallel utilities (mainly power poles) would not require relocation
- o Single-barrel configuration

Disadvantages of Alternative 1 are as follows:

- o All crossing utilities will have to be relocated below the new box culvert

- o Top of box at or above existing grade from McDowell Road to approximately Pinchot Avenue. This includes the Thomas Road intersection.

Advantages of Alternative 4 are as follows:

- o Less impact to crossing utilities
- o Provides a minimum of 4' cover over the top of the box culvert

Disadvantages of Alternative 1 are as follows:

- o More parallel utility relocations (power poles)
- o More excavation and backfill is required

5.4.3 Discussion

Alternative 3 is not recommended as it requires the longest top slab. Alternatives 5 and 6 require major downstream ADOT channel modification, as well as major work to the existing McDowell Road culvert, and are not preferred.

Alternative 2 may be considered as an alternative during the design phase if Alternatives 1 and 4 are unacceptable to the FCD. This alternative is not recommended in this report.

Alternative 1, 16' x 20' mainline, and Alternative 4, 2-18' x 10', are preferred.

No further investigation for Alternatives 2, 3, 5 and 6 were conducted. It is obvious that utility conflict, construction complexity and cost are not favorable for these alternatives.

Alternative 1, with minimum trenching requirement, may avoid some parallel utility lines and power line relocation. The system will be running open channel flow, as well as pressure, and would require more air ventilation installation. A major portion of the top slab will be exposed to the ground and may require additional utility relocations at Thomas Road crossing. Normal manhole spacing supplemented with grate inlet catch basins shall provide adequate ventilation. Detail as required will be developed during the final design, if Alternative 1 is chosen.

Alternative 4 will provide adequate top backfill requirements and will not require any groundwater control. This alternative may require extensive parallel utilities relocation. Equivalent three barrel boxes, such as 3-13' x 10', may be used in lieu of the 2-18' x 10' boxes. The concept was to minimize the top and floor slab thickness to result in a cost savings. A three barrel box will require an extensive flow equalizer chamber and may induce more minor head loss. (Most off-site flow is continuous from the east. Flow equalizer chambers will be required to distribute the flow from the east barrel to the west barrel.) The narrow access width and longer net length are not preferred from the operation and maintenance point of view. Besides, the intention of a cost savings in constructing a three barrel system may also be in question.

Other considerations in the final design selection include:

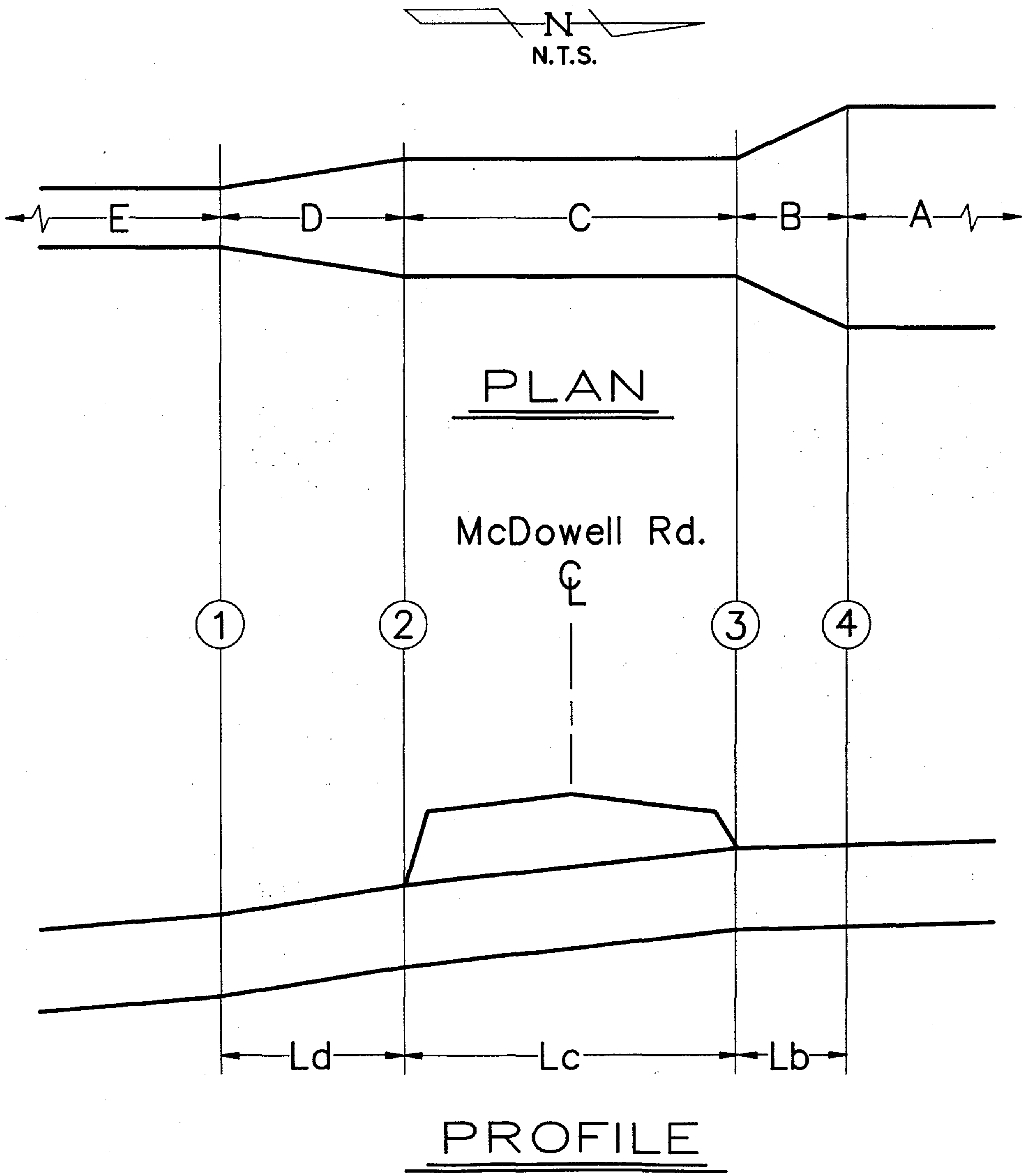
- Although only 4' of cover is required for uplift control, additional cover from the existing top of bank is very desirable to provide a flexible design for the park, roadway and surface drainage construction.

- Developing a combination of the cast-in-place and precast options.

TABLE 5.1
MAINLINE ALTERNATIVES

Structure	Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5	Alternative 6
Downstream Channel Modification	No	No	No	No	Yes	Yes
McDowell Road	Existing	Existing	Existing	Existing	Add 10' x 10' Box	Add 10' x 10' Siphon
Transition No. 1	Yes 26' to 16'	Yes 26' to 23.5'	No	Yes 26' to 37'	Yes 26' to 33'	Yes 26' to 33'
Segment 1	16' x 20' Box	23.5' x 14'	26' x 12'	2-18' x 10'	2-16' x 10'	2-16' x 10'
Transition No. 2	*	*	No	No	No	No
Segment 2	16' x 18'	23.5' x 12'	26' x 12'	2-18' x 10'	2-16' x 10'	2-16' x 10'
Transition No. 3	*	*	*	Yes	Yes	Yes
Segment 3	16' x 14'	23.5' x 10'	26' x 9'	2-12' x 10'	2-12' x 10'	2-12' x 10'
Transition No. 4	*	Yes	Yes	Yes	Yes	Yes
Segment 4	16' x 12'	16' x 12'	16' x 12'	2-10' x 10'	2-10' x 10'	2-10' x 10'
Transition No. 5	Yes	Yes	Yes	Yes	Yes	Yes
Segment 5	10' x 10'	10' x 10'	10' x 10'	10' x 10'	10' x 10'	10' x 10'

*Vertical taper only



OLD CROSS CUT CANAL
McDOWELL ROAD
STRUCTURE ALTERNATIVES

FIGURE 5.4-1

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TABLE 5.2
MCDOWELL ROAD
CONVEYANCE SECTION DESCRIPTION
FOR
EXISTING AND ALTERNATIVES

Description	Conveyance Section					Reach Length		
	A	B	C	D	E	Lb	Lc	Ld
Existing	Channel	Transition	Existing Culvert	Transition	16' Channel	110'	274'	242.1'
Alternative 1	16' x 20'	16' x 20'	16' x 20'	16' x 20'	16' Channel	20'	276'	242.1'
Alternative 4	2-18' x 10'	Transition	Existing Culvert	Transition	16' Channel	20'	274'	242.1'
Alternative 4A	2-16' x 10'	Transition	Existing Culvert	Transition	16' Channel	20'	274'	242.1'
Alternative 5	2-16' x 10'	Transition	Existing + 1-10' x 10' Box	Transition	16' Channel	20'	274'	4,050'
Alternative 6*	2-16' x 10'	Transition	Existing + 1-10' x 10' Siphon	Transition	16' Channel	20'	274'	4,050'

* The backwater for Alternative 6 was not calculated separately, as it will be the same as the backwater for Alternative 5, refer to Table 5.6.

TABLE 5.3
EXISTING SYSTEM BACKWATER

Location	Y Ft	V FPS	$V^2/2g$ Ft	Inv. Ft	Sf %	ΔX Ft	$\overline{Sf}$ %	hf Ft	he Ft	hc Ft	EG Ft
1	12.69	20.19	6.33	1,173.16	0.50	--	--	--	--	--	1,192.18
2	14.04	11.23	1.96	1,177.11	0.101	242.1	0.300	0.73	--	0.20	1,193.11
3	12.23	17.08	4.53	1,179.85	0.780	274.0	--	2.14	1.36*	0	1,196.61
4	12.95	14.74	3.38	1,181.00	0.526	110.0	0.6530	0.72	--	--	1,197.33

*Ke = 0.3 he = Ke $V^2/2g$
Kc = 0.1 hc = Kc $V^2/2g$

Methodology:

1. Critical depth was assumed at Location 1 and its energy was used as the downstream energy for the system.

2. The energy at Location 2 was computed by adding the total head loss through the conveyance system, LD, to the downstream energy at Location 1. A contraction coefficient of 0.1 was used based on a gradual transition geometry. The friction slope for this reach was computed using Manning's equation. The average of the upstream and downstream friction slope for the reach was used to compute the friction loss (average friction slope times reach length).

3. The energy at Location 3 was computed by adding the total head loss through the reach, Lc, to the downstream energy at Location 2. An entrance loss coefficient of 0.3 was used to model the existing headwall geometry, which is asymmetrical and causes poor hydraulic performance. Friction losses for this reach are based on the friction slope of the culvert only.

4. The energy at Location 4 was computed by adding the total head loss through the reach, Lb, to the downstream energy at Location 3. The existing system geometry at Location 4 is a trapezoidal channel with 0.5:1 side slopes, 15-foot bottom width and a Manning's "n" value of 0.025. The average of the friction slopes of the downstream McDowell Road CBC and the upstream end of the reach was multiplied times the reach length, 20 feet, to compute the friction loss. The channel geometry through the reach is sufficiently uniform that no contraction or expansion losses were considered.

TABLE 5.4
ALTERNATIVE 4 - BACKWATER

Location	Y Ft	V FPS	$V^2/2g$ Ft	Inv. Ft	Sf %	ΔX Ft	$\overline{Sf}$ %	hf Ft	he Ft	hc Ft	EG Ft
1	12.69	20.19	6.33	1,173.16	0.50	--	--	--	--	--	1,192.18
2	14.04	11.23	1.96	1,177.11	0.101	242.1	0.300	0.73	0	0.20	1,193.11
3	11.78	17.08	4.53	1,179.85	0.780	274.0	--	2.14	0.91*	0	1,196.16
4	15.64	7.09	0.78	1,179.90	0.0297	20.0	0.40	0.08	0	0.08	1,196.32

*Ke = 0.2 he = Ke $V^2/2g$
Kc = 0.1 hc = Kc $V^2/2g$

Methodology:

1. Critical depth was assumed at Location 1 and its energy was used as the downstream energy for the system.

2. The energy at Location 2 was computed by adding the total head loss through the conveyance system, LD, to the downstream energy at Location 1. A contraction coefficient of 0.1 was used based on a gradual transition geometry. The friction slope for this reach was computed using Manning's equation. The average of the upstream and downstream friction slope for the reach was used to compute the friction loss (average friction slope times reach length).

3. The energy at Location 3 was computed by adding the total head loss through the conveyance system, LC, and adding them to the downstream energy at Location 2. An entrance loss coefficient of 0.2 was used to model the improved entrance geometry proposed at the inlet to the culvert. The friction loss coefficient for the existing culvert was computed using Manning's equation.

4. The energy at Location 4 was computed by adding the total head loss through the conveyance system, Lb, to the downstream energy at Location 3. A contraction loss coefficient of 0.1 was used to model the gradual transition geometry. The upstream friction slope for the conveyance system was computed using Manning's equation, and the average of the upstream and downstream friction slopes was used to compute the friction loss for this reach. Channel geometry for Reach Lb is a rectangular section with a 37-foot bottom, n=.015.

TABLE 5.5
ALTERNATIVE 4A - BACKWATER

Location	Y Ft	V FPS	$V^2/2g$ Ft	Inv. Ft	Sf %	ΔX Ft	$\overline{Sf}$ %	hf Ft	he Ft	hc Ft	EG Ft
1	12.69	20.19	6.33	1,173.16	0.50	--	--	--	--	--	1,192.18
2	14.04	11.23	1.96	1,177.11	0.101	242.1	0.300	0.73	0	0.20	1,193.11
3	11.78	17.08	4.53	1,179.85	0.780	274.0	--	2.14	0.91*	0	1,196.16
4	15.43	8.05	1.01	1,179.90	0.041	20.0	0.411	0.08	--	0.10	1,196.34

*Ke = 0.2 $\quad$ he = Ke $V^2/2g$
Kc = 0.1 $\quad$ hc = Kc $V^2/2g$

Methodology:

1. Critical depth was assumed at Location 1 and its energy was used as the downstream energy for the system.

2. The energy at Location 2 was computed by adding the total head loss through the conveyance system, LD, to the downstream energy at Location 1. A contraction coefficient of 0.1 was used based on a gradual transition geometry. The friction slope for this reach was computed using Manning's equation. The average of the upstream and downstream friction slope for the reach was used to compute the friction loss (average friction slope times reach length).

3. The energy at Location 3 was computed by adding the total head loss through the conveyance system, LC, and adding them to the downstream energy at Location 2. An entrance loss coefficient of 0.2 was used to model the improved entrance geometry proposed at the inlet to the culvert. The friction loss coefficient for the existing culvert was computed using Manning's equation.

4. The energy at Location 4 was computed by adding the total head loss through the reach, Lb, to the downstream energy at Location 3. The proposed system geometry at Location 4 is a rectangular channel with a 33-foot bottom and Manning's "n" of 0.015. The average of the friction slopes of the downstream McDowell Road CBC and the upstream end of the reach was multiplied times the reach length, 20 feet, to compute the friction loss. A contraction loss coefficient of 0.1 was used to model the gradual transition geometry. Total head loss for this reach consists of friction and contraction.

TABLE 5.6
ALTERNATIVE 5 - BACKWATER

Location	Y Ft	V FPS	$V^2/2g$ Ft	Inv. Ft	Sf %	ΔX Ft	$\overline{Sf}$ %	hf Ft	he Ft	hc Ft	EG Ft
1	12.69	20.19	6.33	1,152.90	0.50	--	--	--	--	--	1,171.92
2	7.06	16.13	4.04	1,177.11	0.304	4,050.0	0.402	16.29	0	0	1,188.21
3	10.02	10.77	1.80	1,179.85	0.096	274.0	--	**	**	0	1,191.67**
4	9.31	13.35	2.77	1,179.90	0.168	20.0	0.132	0.03	--	0.28	1,191.97

*Ke = 0.2   he = Ke $V^2/2g$
Kc = 0.1   hc = Kc $V^2/2g$
**Inlet Control

Methodology:

1. In order to design an upstream culvert conveyance system using two 16' x 10' CBC's, it was found that the downstream control section needed to be extended south from its present location 242.1 feet south of the downstream face of the McDowell Road CBC to 4,050 feet south of the downstream face of McDowell Road. This becomes Location 1. Critical depth was assumed at Location 1 and its energy was used as the downstream energy for the system.

2. The energy at Location 2 was computed by adding the total head loss through the conveyance system, Ld, to the downstream energy at Location 1. The transition geometry for the system is sufficiently gradual such that no contraction or expansion coefficients were applied. The friction slope used for the reach is the average of the upstream and downstream friction slopes. The friction loss is computed by multiplying the average friction slope times the reach length.

3. To sufficiently lower the downstream energy to consider using two 16' x 10' CBC's as the upstream conveyance system, an additional 10' x 10' CBC was modeled into the system at the McDowell Road crossing. An entrance loss coefficient of 0.2 was used to model the gradual inlet geometry. The friction slope used was for the culvert only. These two losses, friction and entrance, were the only losses considered for the CBC, and these losses were added to the energy at Location 2 to calculate the energy at Location 3.

4. The energy at Location 4 was calculated by adding the head loss through Reach Lb to the downstream energy at Location 3. A contraction coefficient of 0.1 was used to model the gradual transition geometry. The upstream friction slope was computed using Manning's equation for a rectangular channel with a 33-foot bottom and n=.015. The average friction slope for the upstream section and friction slope for the McDowell Road CBC, described in Number 3 above, was multiplied times the reach length to compute the friction loss. Total head loss for this reach consists of friction and contraction.

TABLE 5.7
MCDOWELL ROAD - CONVEYANCE COMPARISON

Location	2-18' x 10' with Existing McDowell						2-16' x 10' with Existing McDowell					
	$\overline{Sf}$ %	L Ft	hf Ft	he Ft	hc Ft	EG Ft	$\overline{Sf}$ %	L Ft	hf Ft	he Ft	hc Ft	Eg Ft
1	--	--	--	--	--	1,192.18*	--	--	--	--	--	1,1192.18*
2	0.30	242.1	0.73	0	0.20	1,193.11	0.30	242.1	0.73	0	0.20	1,193.11
3	0.78	274.0	2.14	0.91	0	1,196.16	0.78	274.0	2.14	0.91	0	1,196.16
4	0.40	20.0	0.08	0	0.08	1,196.32	0.411	20.0	0.08	0	0.10	1,196.34

Location	2-16' x 10' with Existing McDowell + 1-10' x 10' and 5,293' Transition Extension						2-16' x 10' with Existing McDowell + 1-10' x 10' Siphon and 5,293' Transition Extension					
	$\overline{Sf}$ %	L Ft	hf Ft	he Ft	hc Ft	Eg Ft	$\overline{Sf}$ %	L Ft	hf Ft	he Ft	hc Ft	Eg Ft
1	--	--	--	--	--	1,171.92*	--	--	--	--	--	1,171.92*
2	0.402	4,050.0	16.29	0	0	1,188.21	0.402	4,050.0	16.29	0	0	1,188.21
3	0.096	274.0	**	**	**	1,191.67**	0.096	274.0	**	**	**	1,191.67**
4	0.132	20.0	0.03	--	0.28	1,191.97	0.132	20.0	0.03	--	0.28	1,191.97

*Channel invert elevation + critical specific energy based on a 16' rectangular channel

$\overline{Sf}$ = Sf for culvert flow; (upstream Sf + downstream Sf) 1/2 for open channel flow
**Culvert is in inlet control

5.4.4 Selected Alternative Energy Grade Line

Alternative 4 is the selected alternative. The energy grade line data provided in the following tables reflect two scenarios with varying Manning's "n" values as they relate to this alternative.

The first scenario provides main line data simulating the hydraulic performance of the west barrel.

The second scenario looks at the main line with laterals connected. This design represents hydraulic performance of the east barrel with lateral impact.

Tables 5.8 and 5.9 provide data for Scenario One, while Tables 5.10 and 5.11 summarize Scenario Two.

TABLE 5.8
MAIN LINE ENERGY

Laterals at Streets	Sta. (Ft.)	Yn (Ft.)	Inv. (Ft.)	V (fps)	$V^2/2g$ (Ft.)	HG (Ft.)	EG (Ft.)	Top of Ground (Ft.)	Q (cfs)
---	9+10								
McDowell	9+20	15.63	1179.92	11.4	2.02	1195.55	1197.57	1193.00	2050
Granada	19+94	15.36	1182.43	11.4	2.02	1197.79	1199.81	1203.48	2050
Holly	25+97	15.46	1183.84	10.8	1.81	1199.30	1201.11	1203.97	1950
Oak	36+00	15.00	1186.15	10.8	1.81	1201.15	1202.96	1205.49	1950
Virginia	49+09	14.43	1189.22	10.6	1.74	1203.65	1205.39	1207.06	1900
N. of Virginia	54+00	14.52	1190.39	9.7	1.46	1204.91	1206.37	1207.00	1750
Windsor	55+66	14.38	1190.78	9.7	1.46	1205.16	1206.62	1207.41	1750
Thomas	62+94	13.78	1192.48	9.7	1.46	1206.26	1207.72	1210.74	1750
Pinchot	70+06	13.39	1194.15	9.2	1.31	1207.54	1208.85	1213.00	1650
---	74+04	12.95	1195.17	9.2	1.31	1208.13	1209.44	1213.18	1650
---	74+50	11.38	1195.22	13.8	2.96	1206.60	1209.56	1213.23	1650
Earll	75+40	11.31	1195.57	13.8	2.96	1206.88	1209.84	1213.68	1650
Richardson	81+70	10.84	1198.58	13.3	2.75	1209.42	1212.17	1218.72	1600
Osborn	88+72	10.25	1201.85	13.3	2.75	1212.10	1214.85	1222.05	1600
Whitton	95+25	10.13	1204.95	11.5	2.05	1215.08	1217.13	1227.17	1375
---	96+40	5.00	1205.50	22.9	8.14	1210.50	1218.64	1228.31	1375

Laterals at Streets	Sta. (Ft.)	Yn (Ft.)	Inv. (Ft.)	V (fps)	$V^2/2g$ (Ft.)	HG (Ft.)	EG (Ft.)	Top of Ground (Ft.)	Q (cfs)
---	96+50	6.81	1205.59	20.2	6.34	1212.40	1218.74	1228.41	1375
Weldon	99+56	8.36	1208.21	16.4	4.18	1216.57	1220.75	1230.00	1375
Indian School	114+99	8.09	1221.61	16.2	4.08	1229.70	1233.78	1243.31	1308

Note: Manning's n=0.13

TABLE 5.9
MAIN LINE ENERGY

Laterals at Streets	Sta. (Ft.)	Yn (Ft.)	Inv. (Ft.)	V (fps)	$V^2/2g$ (Ft.)	HG (Ft.)	EG (Ft.)	Top of Ground (Ft.)	Q (cfs)
---	9+10		1179.90			1195.54	1196.32		
McDowell	9+20	15.64	1179.92	11.4	2.02	1195.56	1197.58	1193.00	2050
Granada	19+94	15.72	1182.43	11.4	2.02	1198.15	1200.17	1203.48	2050
Holly	25+97	16.00	1183.84	10.8	1.81	1199.84	1201.65	1203.97	1950
Oak	36+00	15.83	1186.15	10.8	1.81	1201.98	1203.79	1205.49	1950
Virginia	49+09	15.64	1189.22	10.6	1.74	1204.86	1206.60	1207.06	1900
N. of Virginia	54+00	15.84	1190.39	9.7	1.46	1206.23	1207.69	1207.50	1750
Windsor	55+66	15.74	1190.78	9.7	1.46	1206.52	1207.98	1207.41	1750
Thomas	62+94	15.31	1192.48	9.7	1.46	1207.79	1209.25	1210.74	1750
Pinchot	70+06	15.08	1194.15	9.2	1.31	1209.23	1210.54	1213.00	1650
---	74+40	14.74	1195.17	9.2	1.31	1209.91	1211.22	1213.18	1650
---	74+50	13.17	1195.22	13.8	2.96	1208.39	1211.35	1213.23	1650
Earll	75+40	13.15	1195.57	13.8	2.96	1208.72	1211.68	1213.68	1650
Richardson	81+70	13.02	1198.58	13.3	2.75	1211.60	1214.35	1218.72	1600
Osborn	88+72	12.63	1201.85	13.3	2.75	1214.48	1217.23	1222.05	1600
Whitton	95+25	12.99	1204.15	11.5	2.05	1217.93	1219.98	1227.17	1375
---	96+40	12.78	1205.50	11.5	2.05	1218.28	1220.33	1228.31	1375
---	96+50	11.85	1205.59	13.8	2.96	1217.44	1220.40	1228.41	1375

Laterals at Streets	Sta. (Ft.)	Yn (Ft.)	Inv. (Ft.)	V (fps)	$V^2/2g$ (Ft.)	HG (Ft.)	EG (Ft.)	Top of Ground (Ft.)	Q (cfs)
Weldon	99+56	10.73	1208.21	13.8	2.96	1218.94	1221.90	1230.00	1375
Indian School	114+99	8.09	1221.61	16.2	4.08	1229.70	1233.78	1243.31	1308

Note: Manning's n=0.014

TABLE 5.10
MAINLINE ENERGY WITH LATERALS

Street Name	Station (Ft.)	Inv. (Ft.)	Yn (Ft.)	V (fps)	$V^2/2g$ (Ft.)	HG (Ft.)	EG (Ft.)	Top of Ground (Ft.)	Main Q (cfs)	Lateral Q (cfs)	Lateral Pipe Diameter (Ft.)	Control HG (Ft.)	EG at Inlet (Ft.)	Top of Ground (Ft.)
---	9+10	1179.90				1195.54	1196.32	1193.00						
McDowell	9+20	1179.92	15.63	11.4	2.02	1195.55	1197.57	1193.00	2050	322	54"	1195.63	1204.88	1192.35
Granada	19+94	1182.43	15.49	11.4	2.02	1197.92	1199.94	1203.48	2050	81	48"	1198.07	1199.04	1201.68
Holly	25+97	1183.84	15.52	10.8	1.81	1199.36	1201.17	1203.97	1950	88	48"	1199.41	1200.60	1203.00
Oak	36+00	1186.15	15.16	10.8	1.81	1201.31	1203.12	1205.49	1950	917	2-10' x 7'	1201.52	1202.40	1203.70
Virginia	49+09	1189.22	14.80	10.6	1.74	1204.02	1205.76	1207.06	1900	300	1-10' x 8'	1204.23	1204.77	1204.85
N. of Virginia	54+00	1190.39	14.78	9.7	1.46	1205.17	1206.63	1207.00	1750	7	18"	1205.21	1205.81	1207.00
Windsor	55+66	1190.78	14.71	9.7	1.46	1205.49	1206.95	1207.41	1750	56	42"	1205.583	1206.43	1207.06
Thomas	62+94	1192.48	14.19	9.7	1.46	1206.67	1208.13	1210.74	1750	129	1-5' x 5'	1206.80	1207.43	1208.23
Pinchot	70+06	1194.15	13.73	9.2	1.31	1207.88	1209.19	1213.00	1650	19	24"	1207.91	1209.27	1210.72
---	74+40	1195.17	13.36	9.2	1.31	1208.53	1209.84	1213.18	1650	--	--	--	--	--
---	74+50	1195.22	11.79	13.8	2.96	1207.01	1209.97	1213.23	1650	--	--	--	--	--
Earll	75+40	1195.57	12.01	13.8	2.96	1207.59	1210.55	1213.68	1650	802	2-10' x 7'	1208.17	1208.82	1211.90
Richardson	81+70	1198.58	12.32	13.3	2.75	1210.90	1213.65	1218.72	1600	33	30"	1210.98	1212.34	1217.72
Osborn	88+72	1201.85	11.69	13.3	2.75	1213.54	1216.29	1222.05	1600	196	1-7' x 4'	1214.26	1215.28	1221.68
Whitton	95+25	1204.95	11.71	11.5	2.05	1216.66	1218.71	1227.17	1375	9	18"	1216.71	1217.53	1226.11
---	96+40	1205.50	11.57	11.5	2.05	1217.07	1219.12	1228.31	1375	--	--	--	--	--
---	96+50	1205.59	10.63	13.8	2.96	1216.22	1219.18	1228.41	1375	--	--	--	--	--
Weldon	99+56	1208.21	9.61	14.3	3.18	1217.82	1221.00	1230.00	1375	71	48"	1218.20	1218.94	1229.90
Indian School	114+99	1221.61	6.27	20.8	6.72	1227.88	1234.60	1243.31	1308	24	24"	1227.91	1229.90	1243.11
Arizona Canal	121+99	1227.69	8.09	16.2	4.08	1235.78	1239.86	1252.00	1308	--	--	--	--	--

Note: N=0.013

TABLE 5.11
MAINLINE ENERGY WITH LATERALS

Street Name	Station (Ft.)	Inv. (Ft.)	Yn (Ft.)	V (fps)	$V^2/2g$ (Ft.)	HG (Ft.)	EG (Ft.)	Top of Ground (Ft.)	Main Q (cfs)	Lateral Q (cfs)	Lateral Pipe Diameter (Ft.)	Control HG (Ft.)	EG at Inlet (Ft.)	Top of Ground (Ft.)
---	9+10	1179.90				1195.54	1196.32	1193.00						
McDowell	9+20	1179.92	15.64	11.4	2.02	1195.56	1197.58	1193.00	2050	322	54"	1195.63	1205.14	1192.35
Granada	19+94	1182.43	15.85	11.4	2.02	1198.28	1200.30	1203.48	2050	81	48"	1198.43	1199.43	1201.68
Holly	25+97	1183.84	16.06	10.8	1.81	1199.90	1201.71	1203.97	1950	88	48"	1199.95	1201.19	1203.00
Oak	36+00	1186.15	16.06	10.8	1.81	1202.15	1203.96	1205.49	1950	917	2-10' x 7'	1202.36	1203.26	1203.70
Virginia	49+09	1189.22	16.01	10.6	1.74	1205.23	1206.97	1207.06	1900	300	1-10' x 8'	1205.45	1206.00	1204.85
N. of Virginia	54+00	1190.39	16.11	9.7	1.46	1206.50	1207.96	1207.00	1750	7	18"	1206.54	1207.19	1207.00
Windsor	55+66	1190.78	16.08	9.7	1.46	1206.86	1208.32	1207.41	1750	56	42"	1206.90	1207.84	1207.06
Thomas	62+94	1192.48	15.74	9.7	1.46	1208.22	1209.68	1210.74	1750	129	1-5' x 5'	1208.35	1209.00	1208.23
Pinchot	70+06	1194.15	15.43	9.2	1.31	1209.58	1210.89	1213.00	1650	19	24"	1209.62	1211.08	1210.72
---	74+40	1195.17	15.16	9.2	1.31	1210.33	1211.64	1213.18	1650	--	--	--	--	--
---	74+50	1195.22	13.59	13.8	2.96	1208.81	1211.77	1213.23	1650	--	--	--	--	--
Earll	75+40	1195.57	13.86	13.8	2.96	1209.43	1212.39	1213.68	1650	802	2-10' x 7'	1210.03	1210.69	1211.90
Richardson	81+70	1198.58	14.53	13.3	2.75	1213.11	1215.86	1218.72	1600	33	30"	1213.19	1214.64	1217.72
Osborn	88+72	1201.85	14.30	13.3	2.75	1216.15	1218.90	1222.05	1600	196	1-7' x 4'	1216.87	1217.91	1221.68
Whitton	95+25	1204.95	14.59	11.5	2.05	1219.54	1221.59	1227.17	1375	9	18"	1219.60	1220.47	1226.11
---	96+40	1205.50	14.50	11.5	2.05	1220.00	1222.05	1228.31	1375	--	--	--	--	--
---	96+50	1205.59	13.57	13.8	2.96	1219.16	1222.12	1228.41	1375	--	--	--	--	--
Weldon	99+56	1208.21	12.74	13.8	2.96	1220.95	1223.91	1230.00	1375	71	48"	1219.61	1220.38	1229.90
Indian School	114+99	1221.61	12.10	13.1	2.66	1233.71	1236.37	1243.31	1308	24	24"	1230.98	1233.11	1243.11
Arizona Canal	121+99	1227.69	8.09	16.2	4.08	1235.78	1239.86	1252.00	1308	--	--	--	--	--

Note: N=0.014

OLD CROSS CUT CANAL IMPROVEMENTS
25 YEAR ENERGY GRADE LINE

SCALE : HORIZ.– 1"=200'
VERT. – 1"=5'

FIGURE 5.4–2

Greiner

6.0 DRAINAGE IMPACT ASSESSMENT

The impacts of the post-project OCCC were analyzed by comparing the water surface profiles for the pre-project and post-project conditions.

6.1 PRE-PROJECT CONDITION

6.1.1 Water Surface Profile Simulation

The analysis of the pre-project condition included modeling the downstream OCCC of McDowell Road before any City and ADOT improvements. It also included modeling the 9.5'-deep drop structure and the 5'-high end sill immediately downstream of the McDowell Road culvert outlets, and the structures at Indian School Road, Osborn Road, Thomas Road and the Holly Street pedestrian crossing. Water surface profiles for the flow discharge of 25-, 50- and 100-year floods without SRP 1,000 cfs (defined as Case A) and that with SRP 1,000 cfs (defined as Case B) were computed in the pre-project condition.

All water surface profiles were computed using HEC-2. The pre-project channel was analyzed for subcritical flow with computations started at 782' downstream of the McDowell Road culvert outlets. At the outlets of the McDowell Road culvert, flow passes through critical depth which was confirmed to be unsubmerged by the downstream backwater. This was the starting water surface elevation control for the pre-project OCCC model.

1. <u>Cross Section Set-Up</u>

   Cross sections were coded initially at approximately 400' spacing, with additional sections as needed for bridge analyses. It was assumed that the effective flow area of the canal corridor extended only to the fence line on both sides of the channel. Throughout the model, the following coefficient values were used:

2. <u>Loss Coefficient</u>

   Manning's "n"

   n = 0.016 for concrete
   n = 0.025 for the main channel from McDowell Road to Osborn Road
   n = 0.028 for the main channel from Osborn Road to the Arizona Canal
   n = 0.040 for the overbank areas

   Contraction Coefficient

   0.10 at gradual transitions
   0.30 at bridges

Expansion Coefficient

0.35 at gradual transitions
0.50 at bridges

3. McDowell Road Culvert

The culverts at McDowell Road were modeled using the special bridge method. At this location, there exists a 10' x 10' RCB and an 8' diameter RCP. For the model, the pipe section was converted to an equivalent box section by equating the conveyance factor $AR^{2/3}$:

8' x pipe: A = 50.3 square feet, Pw = 25.1 feet, R = 2.00 feet
$AR^{2/3} = 79.8$

Equivalent Box: H = 10 feet (fixed), B = 5.46 feet
A = 54.6 square feet, Pw = 30.9 feet, R = 1.76 Feet
$AR^{2/3} = 79.8$

The coefficients used in the special bridge models were as follows:

Pier Shape Coefficient

Square edged pier: XK = 1.25

Because flow is pressure plus weir, this coefficient for Class A low flow will not be used.

Total Loss Coefficient

XKOR = Ke + Kf + 1
Entrance loss coefficient
From HEC-5, Table 1
Headwall parallel to embankment, square edges
Ke = 0.5
Wingwalls 30 degrees to 75 degrees, square edge barrel
Ke = 0.4
Average Ke = 0.45

Friction loss coefficient

$Kf = 29\ n^2L/R^{4/3}$

$A = 100 + 54.6 = 154.6$

$Pw = 40 + 30.9 = 70.9$

$R = 2.18,\ L = 154$

$Kf = 29(0.0162)^2 154/2.18^{4/3}$

$Kf = 0.40$

$XDOR = 0.4 + 0.4 + 1 = 1.80$

Weir Coefficient

Roadway is broad-crested weir, L = 154'. Obstructions include curbs, guardrail and fences. COFQ = 2.6.

4. <u>Pedestrian Crossing at Holly Street</u>

The next upstream bridge is the pedestrian crossing at Holly Street, which was modeled using the normal bridge method, as the section was not trapezoidal and had no piers.

5. <u>Thomas Road Culvert</u>

For the Thomas Road culvert, the coefficients used in the special bridge model were as follows:

Pier Shape Coefficient

Square edged pier: XK = 1.25

Total Loss Coefficient

$XKOR = Ke + Kf + 1$

Entrance loss coefficient

From HEC-5, Table 1

Wingwalls 30 degrees to 75 degrees, square edge barrel

$Ke = 0.4$

Friction loss coefficient

$Kf = 29n^2L/R^{4/3}$

$A = 190,\ Pw = 78,\ R = 2.44,\ L = 80$

$Kf = 29(0.0162)^2\ 80/2.44^{4/3}$

$Kf = 0.18$

$XKOR = 0.4 + 0.18 + 1 = 1.58$

Weir Coefficient

Roadway is broad-crested weir, L = 80'. Obstructions include curbs, guardrail and fences. COFQ = 2.6

6. Osborn Road and Indian School Road Culverts

The special bridge method was used again for the culverts at Osborn Road and Indian School Road. At both of these locations, there exist an 8' x 8' RCB and a 7' diameter RCP. For the model, the pipe section was converted to an equivalent box section by equating the conveyance factor $AR^{2/3}$:

7' x pipe: $A = 38.5$ square feet, $Pw = 22.0$ feet, $R = 1.75$ feet
$AR^{2/3} = 55.9$

Equivalent box: $H = 8$ feet (fixed), $B = 5.17$ feet
$A = 41.4$ square feet, $Pw = 26.3$ feet, $R = 1.57$ feet
$AR^{2/3} = 55.9$

The coefficients used in the special bridge models were as follows:

Pier Shape Coefficient

$XK = 1.25$

Because flow is pressure plus weir, this coefficient for Class A low flow will not be used.

Total Loss Coefficient

$XKOR = Ke + Kf + 1$
Entrance loss coefficient
From HEC-5, Table 1
Headwall parallel to embankment, square edges
$Ke = 0.5$
Friction loss coefficient
$Kf = 29n^2L/R^{4/3}$
$A = 64 + 38.5 = 102.5$
$Pw = 32 + 22.0 = 54.0$
$R = 1.90, L = 60$
$Kf = 29(0.0162)^2\ 60/1.90^{4/3}$
$Kf = 0.19$
$XKOR = 0.5 + 0.19 + 1 = 1.69$

Weir Coefficient

Roadway is broad-crested weir, L = 60'. Obstructions include curbs, guardrail and fences. COFQ = 2.6

6.1.2 Result

The water surface profiles for the 25-, 50- and 100-year floods without SRP 1,000 cfs (PRE-25-A, PRE-50-A and PRE-100-A) and that with SRP 1,000 cfs (PRE-25-B, PRE-50-B and PRE-100-B) are listed in Table 6.1. The subcritical analysis showed that several sections of the canal are flowing at critical or supercritical depths due to the contractions of the cross sections. Because no extended length of the channel was found to be in supercritical flow, a supercritical HEC-2 run was not performed. The critical depths found at these sections will be a conservative estimate of actual flooding depths.

In the PRE-25-A flood, flooding occurred on the left overbank at Sections 8+10 and 70+05, as shown in Table 6.1. Section 8+10 at the north entrance of the McDowell Road culvert had the deepest flooding depth with DLOB = 0.82'. The pre-project OCCC capacity varies from location to location. In general, the OCCC does not have a consistent capacity to convey a 25-year flow. In the PRE-50-A and PRE-100-A floods, extra flooding occurred from Sections 26+02 to 62+78, and at Sections 74+05, 89+27 and 117+70 for the 50- and 100-year floods. Maximum flooding was located at Section 62+78, which was the north entrance of the Thomas Road culvert. The flooding depth was 4.47' and 5.35', respectively.

Adding the SRP 1,000 cfs to the PRE-25-A flood, significantly enlarged the flooding area of the pre-project OCCC, as shown by the DLOB values of the PRE-25-B flood in Table 6.1. The flooding locations of the PRE-25-B flood are the same as those of the PRE-50-A and PRE-100-A floods. Extra flooding occurred at Section 94+08 for the PRE-50-B and PRE-100-B floods. They all had maximum flooding at Section 62+78 (the north entrance of the Thomas Road culvert) with DLOB = 4.30', 6.07' and 6.82', respectively. In comparing the water surface elevation between Case A and Case B floods of different frequencies, the difference of the water surface elevation (DCWSEL of PRAB-25, PRAB-50 and PRAB-100) can be increased up to 4.84' at Section 62+78, and 1.79' and 1.55' at Section 8+10, respectively.

6.2 POST-PROJECT CONDITIONS

6.2.1 Water Surface Profile Simulation

The analysis of the post-project condition included modeling the downstream OCCC of McDowell Road after any City and ADOT improvements. It also included modeling the upstream OCCC of McDowell Road with modified cross sections and slopes, and adjusted flow discharge for 25-, 50- and 100-year floods.

Table 6.1 Water Surface Elevations of Pre-project Conditions

STREET NAME	SECTION NUMBER	DISTANCE TO McDOWELL (FT)	LOWEST EL LEFT OVERBANK (FT)	PRE-25-A			PRE-50-A			PRE-100-A			PRE-25-B			PRE-50-B			PRE-100-B			PRAB-25	PRAB-50	PRAB-100
				Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	DCWSEL (FT)	DCWSEL (FT)	DCWSEL (FT)
McDOWELL	8+10	0	192.00	3266	192.82	0.82	4663	194.70	2.70	5381	195.50	3.50	4266	194.21	2.21	5663	196.49	4.49	6381	197.05	5.05	1.39	1.79	1.55
GRANADA	18+02	992	200.00	3118	194.30	-5.70	4479	196.29	-3.71	5167	197.14	-2.86	4118	195.77	-4.23	5479	197.80	-2.20	6167	198.46	-1.54	1.47	1.51	1.32
HOLLEY	26+02	1792	203.70	3074	202.23	-1.47	4427	205.89	2.19	5086	206.26	2.56	4074	205.63	1.93	5427	206.50	2.80	6086	206.96	3.26	3.40	0.61	0.70
OAK	34+03	2593	205.10	3031	203.08	-2.02	4362	206.36	1.26	5013	206.83	1.73	4031	206.06	0.96	5362	207.10	2.00	6013	207.63	2.53	2.98	0.74	0.80
VIRGINA	46+03	3793	205.90	2681	205.33	-0.57	3865	207.80	1.90	4449	208.36	2.46	3681	207.49	1.59	4865	208.67	2.77	5449	209.20	3.30	2.16	0.87	0.84
CAMBRIDGE	50+03	4193	206.90	2473	206.13	-0.77	3607	208.19	1.29	4127	208.76	1.86	3473	207.90	1.00	4607	209.08	2.18	5127	209.62	2.72	1.77	0.89	0.86
WINOSOR	54+03	4593	207.70	2470	206.71	-0.99	3598	208.41	0.71	4127	208.95	1.25	3470	208.16	0.46	4598	209.29	1.59	5127	209.82	2.12	1.45	0.88	0.87
THOMAS	62+78	5468	210.10	2432	209.56	-0.54	3547	214.57	4.47	4070	215.45	5.35	3432	214.40	4.30	4547	216.17	6.07	5070	216.92	6.82	4.84	1.60	1.47
PINCHOT	70+05	6195	211.90	2340	212.46	0.56	3429	215.16	3.26	3912	215.99	4.09	3340	214.99	3.09	4429	216.67	4.77	4912	217.40	5.50	2.53	1.51	1.41
EARLL	74+05	6595	214.00	2328	212.91	-1.09	3409	215.36	1.36	3901	216.16	2.16	3328	215.20	1.20	4409	216.84	2.84	4901	217.55	3.55	2.29	1.48	1.39
RICHARDSON	82+07	7397	218.00	1774	214.04	-3.96	2574	215.98	-2.02	2804	216.61	-1.39	2774	215.89	-2.11	3574	217.08	-0.92	3804	217.61	-0.39	1.85	1.10	1.00
OSBORNE	89+27	8117	222.50	1774	222.14	-0.36	2574	224.13	1.63	2804	224.48	1.98	2774	224.38	1.88	3574	225.29	2.79	3804	225.53	3.03	2.24	1.16	1.05
WHITTON	94+08	8598	225.00	1646	222.28	-2.72	2398	224.24	-0.76	2409	224.64	-0.36	2646	224.48	-0.52	3398	225.29	0.29	3409	225.58	0.58	2.20	1.05	0.94
WELDON	98+09	8999	228.80	1642	222.77	-6.03	2392	224.83	-3.97	2405	225.17	-3.63	2642	225.15	-3.65	3392	226.23	-2.57	3405	226.46	-2.34	2.38	1.40	1.29
INDIAN SC.	115+80	10770	243.60	1606	242.95	-0.65	2339	244.95	1.35	2340	244.96	1.36	2606	245.31	1.71	3339	246.15	2.55	3340	246.16	2.56	2.36	1.20	1.20

Legend (example):

PRE-25-A = Case A pre-project condition for the 25-year flood without SRP 1,000 CFS in OCCC
PRE-25-B = Case B pre-project condition for the 25-year flood with SRP 1,000 CFS in OCCC
CWSEL = Computed water surface elevation
DLOB = Flooding water depth above left overbank
DCWSEL (PRAB-25) = CWSEL (PRE-25-B) - CWSEL (PRE-25-A)

In the post-project condition, the pre-project OCCC cross sections were assumed to be completely filled with soil above the box conduits toward the lower top of the channel bank. The channel slope was composed of three segments of linear slope proposed as 0.234 percent, 0.468 percent and 0.204 percent, starting from Sections 8+90 to 74+40, Sections 74+40 to 96+40, and Section 96+40 toward upstream, respectively. At the pre-project bridge crossing, the channel cross sections were modified to accommodate the pre-project road surface.

In the post-project condition, the adjusted flow discharge in the channel was considered to be the balance after excluding the flow discharge in the conduit from the pre-project Case B flow discharge. The conduit discharge was assumed to be approximately Q = 3,000 cfs from McDowell Road to Thomas Road, Q = 2,500 cfs from Thomas Road to Indian School Road, and Q = 2,000 cfs from Indian School Road toward upstream (defined as Case A). In a separate run, the SRP 1,000 cfs was added to the Case A conduit discharge (defined as Case B) to evaluate the impact of the water surface elevation. The final adjusted flow discharge of different frequencies in the OCCC are listed in Table 6.2.

6.2.2 Results

The water surface profiles for the 25-, 50- and 100-year floods without SRP 1,000 cfs in the conduit (POS-25-A, POS-50-A and POS-100-A) and that with SRP 1,000 cfs in the conduit (POS-25-B, POS-50-B and POS-100-B) are listed in Table 6.2. For comparison, the water surface profiles of the Case B pre-project condition are put together with those of Case A and Case B post-project conditions in Table 6.3 and Table 6.4, respectively. The "*" symbol indicates that critical depths are assumed in the HEC-2 modeling at the sections in these tables.

Flooding occurred nearly everywhere in the post-project condition, except at Sections 74+05, 82+07 and 98+09 for the POS-25-B flood, as shown in Table 6.2. Maximum flooding all occurred at Section 18+02 near Granada Street at DLOB = 4.24', 5.32' and 5.84' for the POS-25-A, POS-50-A and POS-100-A floods, and at DLOB = 3.13', 4.56' and 5.10' for the POS-25-B, POS-50-B and POS-100-B floods respectively. The increase of the conduit discharge with SRP 1,000 cfs reduced the flooding potential. The maximum value of DCWSEL is -2.10' for the POAB-25 at Section 26+02, and is -1.25' for the POAB-50 and POAB-100 at Section 115+80.

Table 6.2 Water Surface Elevations of Post-project Conditions

STREET NAME	SECTION NUMBER	DISTANCE TO McDOWELL (FT)	LOWEST EL LEFT OVERBANK (FT)	POS-25-A			POS-50-A			POS-100-A			POS-25-B			POS-50-B			POS-100-B			POAB-25	POAB-50	POAB-100
				Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	DCWSEL (FT)	DCWSEL (FT)	DCWSEL (FT)
McDOWELL	8+10	0	192.00	*1300	195.80	3.80	*2700	197.08	5.08	*3400	197.64	5.64	*300	194.51	2.51	*1700	196.19	4.19	*2400	196.83	4.83	-1.29	-0.89	-0.81
GRANADA	18+02	912	200.00	1100	204.24	4.24	2500	205.32	5.32	3200	205.84	5.84	100	203.13	3.13	1500	204.56	4.56	2200	205.10	5.10	-1.11	-0.76	-0.74
HOLLEY	26+02	1712	203.70	1100	206.16	2.46	2400	207.61	3.91	3050	208.20	4.50	100	204.06	0.36	1400	206.58	2.88	2050	207.30	3.60	-2.10	-1.03	-0.90
OAK	34+03	2513	205.10	1100	207.15	2.05	2400	208.61	3.51	3050	209.22	4.12	100	205.59	0.49	1400	207.54	2.44	2050	208.27	3.17	-1.56	-1.07	-0.95
VIRGINA	46+03	3713	205.90	1100	208.57	2.67	2050	209.85	3.95	2600	210.46	4.56	100	206.55	0.65	1050	208.66	2.76	1600	209.41	3.51	-2.02	-1.19	-1.05
CAMBRIDGE	50+03	4113	206.90	1100	209.09	2.19	2050	210.29	3.39	2600	210.88	3.98	100	207.51	0.61	1050	209.09	2.19	1600	209.82	2.92	-1.58	-1.20	-1.06
WINOSOR	54+03	4513	207.70	1100	209.69	1.99	2050	210.80	3.10	2600	211.37	3.67	100	207.91	0.21	1050	209.65	1.95	1600	210.34	2.64	-1.78	-1.15	-1.03
THOMAS	62+78	5388	210.10	1100	212.56	2.46	2050	213.38	3.28	2600	213.76	3.66	100	210.86	0.76	1050	212.50	2.40	1600	213.02	2.92	-1.70	-0.88	-0.74
PINCHOT	70+05	6115	211.90	1100	214.14	2.24	1900	214.95	3.05	2400	215.38	3.48	*100	212.48	0.58	900	213.91	2.01	1400	214.48	2.58	-1.66	-1.04	-0.90
EARLL	74+05	6515	214.00	1100	215.07	1.07	1900	215.82	1.82	2400	216.23	2.23	100	213.70	-0.30	900	214.85	0.85	1400	215.37	1.37	-1.37	-0.97	-0.86
RICHARDSON	82+07	7317	218.00	1100	219.06	1.06	1350	219.28	1.28	1350	219.15	1.15	100	217.57	-0.43	350	218.16	0.16	350	218.41	0.41	-1.49	-1.12	-0.74
OSBORNE	89+27	8037	222.50	*1100	223.98	1.48	*1350	224.18	1.68	*1350	224.18	1.68	100	222.90	0.40	*350	223.24	0.74	*350	223.24	0.74	-1.08	-0.94	-0.94
WHITTON	94+08	8518	225.00	1100	228.02	3.02	1350	228.24	3.24	1350	228.24	3.24	100	226.54	1.54	350	227.13	2.13	350	227.13	2.13	-1.48	-1.11	-1.11
WELDON	98+09	8919	228.80	*1100	230.01	1.21	1350	230.13	1.33	1350	230.13	1.33	100	228.53	-0.27	350	229.06	0.26	350	229.06	0.26	-1.48	-1.07	-1.07
INDIAN SC.	115+80	10690	243.60	1100	245.79	2.19	1350	246.07	2.47	1350	246.07	2.47	100	244.32	0.72	350	244.82	1.22	350	244.82	1.22	-1.47	-1.25	-1.25

Legend (example):

POS-25-A = Case A post-project condition for the 25-year flood without SRP 1,000 CFS in the box conduit
POS-25-B = Case B post-project condition for the 25-year flood with SRP 1,000 CFS in the box conduit
CWSEL = Computed water surface elevation
DLOB = Flooding water depth above left overbank
DCWSEL (POAB-25) = CWSEL (POS-25-B) - CWSEL (POS-25-A)

Table 6.3 Comparison of Water Surface Elevations between Case B Pre-project and Case A Post-Project Conditions

STREET NAME	SECTION NUMBER	DISTANCE TO McDOWELL (FT)	LOWEST EL LEFT OVERBANK (FT)	PRE-25-B			PRE-50-B			PRE-100-B			POS-25-A			POS-50-A			POS-100-A			PRPO-25	PRPO-50	PRPO-100
				Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	DCWSEL (FT)	DCWSEL (FT)	DCWSEL (FT)
McDOWELL	8+10 8+90	0 0	192.00	4266	194.21	2.21	5663	196.49	4.49	6381	197.05	5.05	*1300	195.80	3.80	*2700	197.08	5.08	*3400	197.64	5.64	1.59	0.59	0.59
GRANADA	18+02 18+02	992 912	200.00	4118	195.77	-4.23	5479	197.80	-2.20	6167	198.46	-1.54	1100	204.24	4.24	2500	205.32	5.32	3200	205.84	5.84	8.47	7.52	7.38
HOLLEY	26+02 26+02	1792 1712	203.70	4074	205.63	1.93	5427	206.50	2.80	6086	206.96	3.26	1100	206.16	2.46	2400	207.61	3.91	3050	208.20	4.50	0.53	1.11	1.24
OAK	34+03 34+03	2593 2513	205.10	4031	206.06	0.96	5362	207.10	2.00	6013	207.63	2.53	1100	207.15	2.05	2400	208.61	3.51	3050	209.22	4.12	1.09	1.51	1.59
VIRGINA	46+03 46+03	3793 3713	205.90	3681	207.49	1.59	4865	208.67	2.77	5449	209.20	3.30	1100	208.57	2.67	2050	209.85	3.95	2600	210.46	4.56	1.08	1.18	1.26
CAMBRIDGE	50+03 50+03	4193 4113	206.90	3473	207.90	1.00	4607	209.08	2.18	5127	209.62	2.72	1100	209.09	2.19	2050	210.29	3.39	2600	210.88	3.98	1.19	1.21	1.26
WINOSOR	54+03 54+03	4593 4513	207.70	3470	208.16	0.46	4598	209.29	1.59	5127	209.82	2.12	1100	209.69	1.99	2050	210.80	3.10	2600	211.37	3.67	1.53	1.51	1.55
THOMAS	62+78 62+78	5468 5388	210.10	3432	214.40	4.30	4547	216.17	6.07	5070	216.92	6.82	1100	212.56	2.46	2050	213.38	3.28	2600	213.76	3.66	-1.84	-2.79	-3.16
PINCHOT	70+05 70+05	6195 6115	211.90	3340	214.99	3.09	4429	216.67	4.77	4912	217.40	5.50	1100	214.14	2.24	1900	214.95	3.05	2400	215.38	3.48	-0.85	-1.72	-2.02
EARLL	74+05 74+05	6595 6515	214.00	3328	215.20	1.20	4409	216.84	2.84	4901	217.55	3.55	1100	215.07	1.07	1900	215.82	1.82	2400	216.23	2.23	-0.13	-1.02	-1.32
RICHARDSON	82+07 82+07	7397 7317	218.00	2774	215.89	-2.11	3574	217.08	-0.92	3804	217.61	-0.39	1100	219.06	1.06	1350	219.28	1.28	1350	219.15	1.15	3.17	2.20	1.54
OSBORNE	89+27 89+27	8117 8037	222.50	2774	224.38	1.88	3574	225.29	2.79	3804	225.53	3.03	*1100	223.98	1.48	*1350	224.18	1.68	*1350	224.18	1.68	-0.40	-1.11	-1.35
WHITTON	94+08 94+08	8598 8518	225.00	2646	224.48	-0.52	3398	225.29	0.29	3409	225.58	0.58	1100	228.02	3.02	1350	228.24	3.24	1350	228.24	3.24	3.54	2.95	2.66
WELDON	98+09 98+09	8999 8919	228.80	2642	225.15	-3.65	3392	226.23	-2.57	3405	226.46	-2.34	*1100	230.01	1.21	1350	230.13	1.33	1350	230.13	1.33	4.86	3.90	3.67
INDIAN SC.	115+80 115+80	10770 10690	243.60	2606	245.31	1.71	3339	246.15	2.55	3340	246.16	2.56	1100	245.79	2.19	1350	246.07	2.47	1350	246.07	2.47	0.48	-0.08	-0.09

Legend (example):

PRE-25-B = Case B pre-project condition for the 25-year flood with SRP 1,000 CFS in OCCC
POS-25-A = Case A post-project condition for the 25-year flood without SRP 1,000 CFS in the box conduit
CWSEL = Computed water surface elevation
DLOB = Flooding water depth above left overbank
DCWSEL (PRPO-25) = CWSEL (POS-25-A) - CWSEL (PRE-25-B)

Table 6.4 Comparison of Water Surface Elevations between Case B Pre-project and Case B Post-Project Conditions

STREET NAME	SECTION NUMBER		DISTANCE TO McDOWELL (FT)		LOWEST EL LEFT OVERBANK (FT)	PRE-25-B			PRE-50-B			PRE-100-B			POS-25-B			POS-50-B			POS-100-B			PRPO-25	PRPO-50	PRPO-100
						Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	Q (CFS)	CWSEL (FT)	DLOB (FT)	DCWSEL (FT)	DCWSEL (FT)	DCWSEL (FT)
McDOWELL	8+10	8+90	0	0	192.00	4266	194.21	2.21	5663	196.49	4.49	6381	197.05	5.05	*300	194.51	2.51	*1700	196.19	4.19	*2400	196.83	4.83	0.30	-0.30	-0.22
GRANADA	18+02	18+02	992	912	200.00	4118	195.77	-4.23	5479	197.80	-2.20	6167	198.46	-1.54	100	203.13	3.13	1500	204.56	4.56	2200	205.10	5.10	7.36	6.76	6.64
HOLLEY	26+02	26+02	1792	1712	203.70	4074	205.63	1.93	5427	206.50	2.80	6086	206.96	3.26	100	204.06	0.36	1400	206.58	2.88	2050	207.30	3.60	-1.57	0.08	0.34
OAK	34+03	34+03	2593	2513	205.10	4031	206.06	0.96	5362	207.10	2.00	6013	207.63	2.53	100	205.59	0.49	1400	207.54	2.44	2050	208.27	3.17	-0.47	0.44	0.64
VIRGINA	46+03	46+03	3793	3713	205.90	3681	207.49	1.59	4865	208.67	2.77	5449	209.20	3.30	100	206.55	0.65	1050	208.66	2.76	1600	209.41	3.51	-0.94	-0.01	0.21
CAMBRIDGE	50+03	50+03	4193	4113	206.90	3473	207.90	1.00	4607	209.08	2.18	5127	209.62	2.72	100	207.51	0.61	1050	209.09	2.19	1600	209.82	2.92	-0.39	0.01	0.20
WINOSOR	54+03	54+03	4593	4513	207.70	3470	208.16	0.46	4598	209.29	1.59	5127	209.82	2.12	100	207.91	0.21	1050	209.65	1.95	1600	210.34	2.64	-0.25	0.36	0.52
THOMAS	62+78	62+78	5468	5388	210.10	3432	214.40	4.30	4547	216.17	6.07	5070	216.92	6.82	100	210.86	0.76	1050	212.50	2.40	1600	213.02	2.92	-3.54	-3.67	-3.90
PINCHOT	70+05	70+05	6195	6115	211.90	3340	214.99	3.09	4429	216.67	4.77	4912	217.40	5.50	*100	212.48	0.58	900	213.91	2.01	1400	214.48	2.58	-2.51	-2.76	-2.92
EARLL	74+05	74+05	6595	6515	214.00	3328	215.20	1.20	4409	216.84	2.84	4901	217.55	3.55	100	213.70	-0.30	900	214.85	0.85	1400	215.37	1.37	-1.50	-1.99	-2.18
RICHARDSON	82+07	82+07	7397	7317	218.00	2774	215.89	-2.11	3574	217.08	-0.92	3804	217.61	-0.39	100	217.57	-0.43	350	218.16	0.16	350	218.41	0.41	1.68	1.08	0.80
OSBORNE	89+27	89+27	8117	8037	222.50	2774	224.38	1.88	3574	225.29	2.79	3804	225.53	3.03	100	222.90	0.40	*350	223.24	0.74	*350	223.24	0.74	-1.48	-2.05	-2.29
WHITTON	94+08	94+08	8598	8518	225.00	2646	224.48	-0.52	3398	225.29	0.29	3409	225.58	0.58	100	226.54	1.54	350	227.13	2.13	350	227.13	2.13	2.06	1.84	1.55
WELDON	98+09	98+09	8999	8919	228.80	2642	225.15	-3.65	3392	226.23	-2.57	3405	226.46	-2.34	100	228.53	-0.27	350	229.06	0.26	350	229.06	0.26	3.38	2.83	2.60
INDIAN SC.	115+80	115+80	10770	10690	243.60	2606	245.31	1.71	3339	246.15	2.55	3340	246.16	2.56	100	244.32	0.72	350	244.82	1.22	350	244.82	1.22	-0.99	-1.33	-1.34

Legend (example):

PRE-25-B = Case B pre-project condition for the 25-year flood with SRP 1,000 CFS in OCCC
POS-25-B = Case B post-project condition for the 25-year flood with SRP 1,000 CFS in the box conduit
CWSEL = Computed water surface elevation
DLOB = Flooding water depth above left overbank
DCWSEL (PRPO-25) = CWSEL (POS-25-B) - CWSEL (PRE-25-B)

In general, the Case A post-project condition increased the flooding potential, as compared to the Case B pre-project condition except at a few sections such as Section 62+78 to 74+05, and Section 89+27. This was indicated from the increased water surface elevation (DCWSEL of PRPO-25, PRPO-50 and PRPO-100) as indicated in Table 6.3. The Case B post-project condition reduced this flooding potential due to the increase of the conduit discharge for SRP 1,000 cfs, especially for the 25-year flood. This is indicated in Table 6.4. The maximum increase of the water surface elevation (DCWSEL) all occurred at Section 18+02 near Granada Street. The DCWSEL values are 8.47', 7.52' and 7.37' for the PRPO-25, PRPO-50 and PRPO-100 in the Case A post-project condition, and are 7.36', 6.76' and 6.64' for the PRPO-25, PRPO-50 and PRPO-100 in the Case B post-project condition.

6.3 PROVISION TO OTHER IMPROVEMENTS

The result of the water surface simulation for the post-project condition indicated that the flooding potential along the OCCC was due to the excess flow discharge above the conduit in the completely filled OCCC. These excess flow discharges were 1,300 cfs, 2,700 cfs and 3,400 cfs in the Case A post-project condition, and were 300 cfs, 1,700 cfs and 2,400 cfs in the Case B post-project condition for the 25-, 50- and 100-year floods, respectively, conveyed to McDowell Road. These excess flow discharges must be considered in other improvements to prevent possible flooding along the OCCC. The side flow along the OCCC needs to be collected into catch basins and conveyed to the conduit to absorb the 25-year flood of the pre-project condition.

7.0 OTHER CONSIDERATIONS

7.1 ROADWAY INTERSECTIONS

7.1.1 Indian School Road

Existing Conditions

The Indian School Road crossing consists of approximately 62' of pavement curb to curb. Within the 62' of pavement are three eastbound lanes, one center through lane and two westbound lanes. The ADT for this segment of Indian School Road is 42,000 vehicles per day.

Utilities

An existing 12" ACP water line is located in Indian School Road and crosses the OCCC alignment. The water line should not require relocation; however, it will require replacement with DIP and support during construction. An 8" HP gas line parallels the project on the east side and a 36" irrigation line parallels on the west side. The gas line and irrigation line are not expected to be impacted by construction of the OCCC.

Traffic

A possible detour around construction at Indian School Road does not appear feasible due to the lack of available room north or south to construct a detour. The construction of the OCCC improvements should be accomplished under traffic, maintaining three open lanes at a time. The westbound direction should consist of two lanes and the eastbound should consist of one lane during the morning peak traffic. The eastbound should consist of two lanes and the westbound one lane during the evening peak traffic. A uniformed off-duty police officer should be present during construction. Left turns should be prohibited during construction for safety and flow of traffic considerations.

Construction Materials and Methods

Due to the heavy traffic volume on Indian School Road, rapid construction within the roadway prism is essential. Conventional cast-in-place techniques are not desirable due to the time required to accomplish this construction. Precast conduit is recommended to minimize the length of time required to cross the intersection. Half of the structure can be built, traffic shifted to the other side of Indian School Road, then the other half of the crossing constructed. The City of Phoenix's policies on barricading should be followed.

7.1.2 Osborn Road

Existing Conditions

Osborn Road at the OCCC consists of one lane in each direction, with a total pavement width of approximately 24'.

7.0 OTHER CONSIDERATIONS

The 48th Street alignment changes at Osborn Road from the east side to the west side of the canal going north. On the east side of the canal and south of Osborn Road, 48th Street accommodates two-way traffic. On the west side of the canal and north of Osborn Road, 48th Street accommodates one-way traffic northbound to Indian School Road.

Utilities

An 8" ACP water line is located in Osborn Road north of the monument line. This water line will require replacement with ductile iron within the limits of the OCCC improvements, but should not require relocation.

A 12" VCP sewer line is also located in Osborn Road just north of the monument line and will require relocation.

An existing 8" gas line and 8" VCP sewer line run parallel to the canal on the east side.

Traffic

Two options exist for maintaining traffic in the vicinity of Osborn Road and the OCCC. Option 1 consists of constructing a detour around the canal crossing south of Osborn Road, and Option 2 consists of closing Osborn Road to through traffic and allowing left turn only (north) for eastbound traffic and left turn only (south) for westbound traffic.

Option 1. This option involves the construction of a temporary detour to the south around the canal crossing. A design speed of 25 mph was used to establish a minimum radius of 205'. A radius of 220' was selected. The superelevation required is about 2.5 percent. No additional right-of-way will be required for this detour option.

A jersey barrier will be required along the detour on each side to separate traffic from the construction activities. A 5' sidewalk would also be required on one side of the detour to accommodate pedestrian traffic.

A sufficient length of box must be built to allow two-way traffic back on Osborn Road before the detour is removed. Temporary drainage structures will be required while the detour is in place.

Under Option 1, 48th Street on the east side of the canal in both the north/south direction would be closed at Osborn Road. Properties requiring access along 48th Street between Thomas Road and Osborn Road can be serviced through Earll Drive.

Option 2. This option consists of closing Osborn Road to through vehicular traffic, and closing north/south traffic along 48th Street at Osborn Road on both the west and east side of the canal. Pedestrian access will need to be provided through Osborn Road for students of Tavan Elementary School. Section 7.3, Traffic Control, addresses street closures and access requirements along the project corridor. Option 2 is less costly than Option 1. No detour or temporary drainage structures would be required.

7.1.3 Thomas Road

Existing Conditions

The Thomas Road crossing consists of approximately 64' of pavement curb-to-curb with three eastbound lanes, two westbound lanes and one center turn lane. The ADT for this segment of Thomas Road is 51,000 vehicles per day.

Utilities

Existing 12" and 45" water lines cross the canal at Thomas Road. Both of these water lines are in conflict with the new box and will require relocation. Existing gas, telephone and cable T.V. lines are attached to the box culvert and will require support during construction.

Traffic

A detour at Thomas Road is not feasible due to inadequate space north and south of the roadway in which to construct a detour. The construction of the canal improvements should be performed under traffic, maintaining three open lanes at a time. The westbound direction should consist of two lanes, and the eastbound should consist of one lane during the morning peak traffic. The eastbound should consist of two lanes, and the westbound one lane during the evening peak traffic.

A uniformed, off-duty police officer should be present during construction. Left turns should be prohibited during construction for safety and flow of traffic considerations.

Construction Materials and Methods

Due to the heavy traffic volume on Thomas Road, rapid construction within the roadway prism is essential. Conventional cast-in-place techniques are not desirable due to the time required to accomplish this construction. Precast conduit is recommended to minimize the length of time required to cross the intersection, but due to manufacturing limitations as previously mentioned in Section 5.3.2, this will not be possible at this location. Half of the structure can be built and then traffic shifted to the other side of Thomas Road; then the other half of the crossing can be constructed. The City of Phoenix's policies on barricading should be followed.

7.2 MAINTENANCE ACCESS

Maintenance access will be provided along the mainline conduit via manholes spaced approximately every 500'.

Every effort will be made during the design phase to locate these access manholes outside of the future roadway pavement. This will require coordination with the City of Phoenix and HNTB as they develop the design plans for the roadway and park.

Access manholes will be located next to one of the walls (outside or center) of the box culvert to provide a continuous stairway from the surface to the invert. An opening will be provided in the center wall of the box culvert at the manhole locations for access to both chambers.

Due to the fact that these manholes will be located close to both roadway and parkway maintenance traffic, the manhole shafts and covers will be designed to handle HS-20-44 loading.

A preliminary cost estimate based on 500' spacing and a configuration similar to the manholes used on the ADOT OCCC project south of McDowell Road is shown below.

Preliminary Cost Estimate

Access manholes - 22 each at $5,000.00 = $110,000.00

7.3 TRAFFIC CONTROL

Construction access to the proposed double box culvert and stub-outs along the existing OCCC alignment will necessitate some road closures and traffic restrictions. These will affect 48th Street from McDowell Road on the south to just below the Arizona Canal on the north, as well as the Indian School Road, Thomas Road and Osborn Road overcrossings.

Indian School Road and Thomas Road are major arterials for east/west traffic. It is, therefore, desirable to keep these crossings open to facilitate this traffic movement and to stagger the construction of each intersection to provide minimum degradation of the level of service for the area. Because of its small size, we recommend completely closing the Osborn Road crossing to vehicular traffic, thereby decreasing total construction time and minimizing traffic control costs associated with constructing a detour at this intersection.

In contacting various schools located in the Osborn Road area, it was determined that pedestrian and bicycle access will need to be provided at Osborn Road for students of the Tavan Elementary School. Principal Bruce Burns indicated that Osborn Road is a main route for many of his students coming to and leaving Tavan Elementary School. This pedestrian access would not be required if the intersection was constructed between July and August.

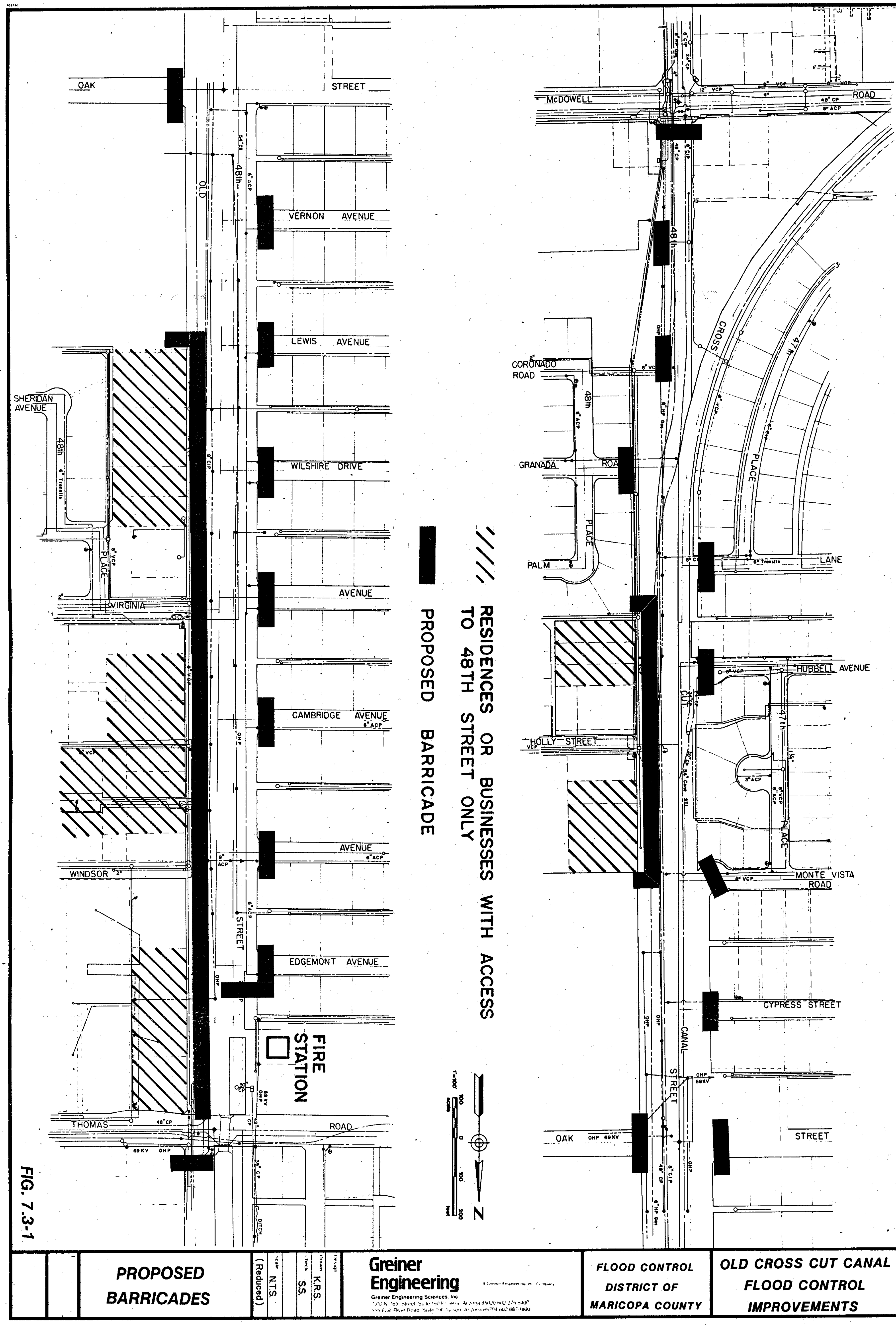


FIG. 7.3-1

PROPOSED BARRICADES	Scale N.T.S. (Reduced) Check S.S. Drawn K.R.S. Design	Greiner Engineering A Greiner Engineering Inc. Company Greiner Engineering Sciences, Inc.	FLOOD CONTROL DISTRICT OF MARICOPA COUNTY	OLD CROSS CUT CANAL FLOOD CONTROL IMPROVEMENTS

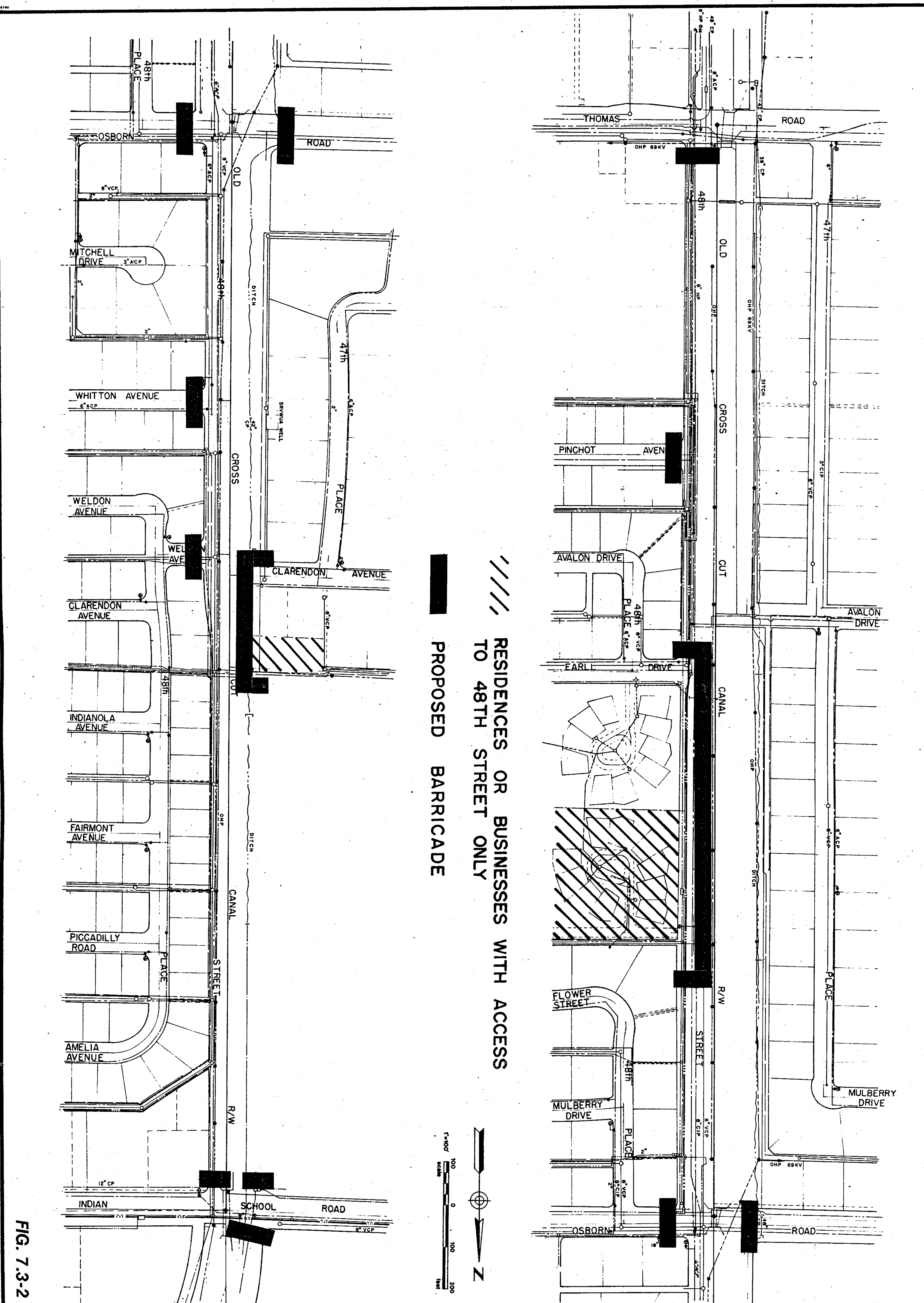


FIG. 7.3-2

PROPOSED BARRICADES	Design: Drawn: K.R.S. Check: S.S. Scale: N.T.S. (Reduced)	**Greiner Engineering** — A Greiner Engineering Inc. Company — Greiner Engineering Sciences, Inc. 7310 N. 16th Street Suite 160 Phoenix Arizona 85020 602 275-5400; 555 East River Road Suite 100 Tucson Arizona 85704 602 887-1800	**FLOOD CONTROL DISTRICT OF MARICOPA COUNTY** — **OLD CROSS CUT CANAL FLOOD CONTROL IMPROVEMENTS**

7.0 OTHER CONSIDERATIONS

While total closure of 48th Street is desirable, it will not be completely possible. In evaluating the access needs of each residence and business near the proposed improvements, it was found that, in several instances, the only street access available to a private home or business was onto 48th Street (see Figures 7.3-1 and 7.3-2). In these cases, vehicle access will need to be maintained for daily traffic, as well as emergency vehicle use and trash collection. One lane each for entry and exit will be provided in such cases.

The following areas will need to be provided with access:

Beginning at approximately 300' north of Granada Road on the east side of 48th Street and ending 300' north of Holly Street. This will best be accomplished by routing traffic through Holly Street and barricading 48th Street above and below this area.

St. Francis Cemetery has access to both of its properties from Oak Street as well as 48th Street. In a telephone conversation on March 25th with Frank Aragon, Assistant Director of the Cemetery, he mentioned that with several weeks notice, he could operate completely out of the Oak Street access to the cemetery, thus allowing the closure of 48th Street along this property.

Starting at the northern boundary of St. Francis Cemetery and proceeding north along the east side of 48th Street to Thomas Road, access must be provided for private residences and businesses. Two lanes along 48th Street will be required for access in this area with connections to Virginia Avenue and Thomas Road. Windsor Avenue is not a suitable eastern exit, as it dead ends into a 12' alley east of 48th Street.

There is a fire station just south of Thomas Road on the west side of 48th Street. Its access is through the 48th Street/Thomas Road intersection. Fire Chief Tom Stanley of the Phoenix Fire Department, said that the two-lane north/south access along west 48th Street would be needed, as well as east/west access along Thomas Road. The fire engines used at this station are 37' long and do not handle well in mud or pavement depressions. It will, therefore, be necessary to take special precautions at the construction site in this area to accommodate these special needs. Chief Stanley also asked us to continue to coordinate with him and keep him up-to-date on any problems or questions we may have in planning for the fire engine access at this point.

Approximately 600' north of Earll Drive on the east side of 48th Street is a private roadway leading to a small housing development. Access will need to be provided for these residents along 48th Street from Earll Drive to the south.

At the south end of the Republic/Gazette property on the west side of the canal below Indian School Road is a single resident who accesses solely through west 48th Street. Options for providing access during construction include keeping west 48th Street open from Osborn Road to Indian School Road, or providing access through Clarendon Avenue just to the south of the property. Clarendon Avenue currently dead-ends just west of 48th Street with an irrigation canal and a barricade separating the streets. A temporary irrigation ditch crossing would need to be provided, as well as the removal of the barricade to access this residence through Clarendon Avenue. Construction access will be limited predominantly to west 48th Street from Osborn Road to Indian School Road. For this reason, we recommend minimizing special access requirements. The Clarendon Avenue opening access would provide the least amount of interference to construction access.

There is an area of possible access conflict just south of the Arizona Canal along the west side of 48th Street. There is a small insurance business that does not access into the adjacent shopping center parking lot on the south side. Two driveways along 48th Street provide access to this business just south of the Arizona Canal. It may be possible to close 48th Street from Indian School Road to the northernmost driveway and still provide access to this property from the north.

Another item of coordination will be trash collection. There are two sanitation service areas within the project boundaries. These include the Sky Harbor service area with Brenda Marshall as Director; and the Salt River service area with Joe Robledo as Director. The Sky Harbor area encompasses everything north of Thomas Road, while the Salt River area consists of Thomas Road and areas south. In a conversation with Joe Robledo, he stated that with our current plan to provide access to the areas tied to 48th Street, trash collection could continue. In a meeting with the Sky Harbor Service District, we discussed the District's needs in terms of access to 48th Street. The District said that there were three principal areas needing trash collection access along east 48th Street. These include: the alleyways bounding Pinchot Avenue, an alleyway approximately 400' north of Earll Drive and the alleyways bounding the Mitchell Drive cul-de-sac.

In the case of Pinchot Avenue and Mitchell Drive, street side trash collection can be substituted for alleyway trash collection with a one-week notice to the Sanitation District. This would free-up 48th Street for construction use in this area. The District also asked that we provide room for their vehicles to turn around if Pinchot Avenue must be barricaded at 48th Street.

The objective of traffic closure for the OCCC project is to maintain needed access to residents and businesses for private, emergency and trash collection purposes while, at the same time, restricting any unnecessary through traffic to expedite construction and to keep traffic control costs to a minimum.

7.4 RIGHT-OF-WAY

Right-of-way requirements for the proposed improvements of the OCCC are based upon the preliminary horizontal and vertical alignment of the CBC. This preliminary alignment follows the centerline of the existing canal and varies in depth. Vertical placement of the proposed CBC culverts can be seen in the cross sections included in Section 5.2 of this report.

7.0 OTHER CONSIDERATIONS

In the initial walk-through of the project, four possible right-of-way conflicts were seen. These included: (1) north of Indian School Road, (2) the northwest corner of Thomas Road at Ted's Kitchen Restaurant, (3) the southwest corner of Oak Street, and (4) northeast of McDowell Road.

Permanent right-of-way will need to be acquired from the parcel on the west side of the canal from Indian School Road to approximately 284' north of Clarendon Avenue (Figure 7.4-4). There is some question regarding the location of the eastern property line for this parcel. The City of Phoenix quarter section maps and the assessors maps show this property to be bounded to the east by the section line. If this information is correct, we will need to acquire new right-of-way 25' off of the section line to the west. A temporary construction easement 15' further to the west from this new right-of-way line will also be needed for excavation width.

Initially, it was thought that the parking lot of Ted's Kitchen Restaurant at Thomas Road marked the property line and that an easement would be required. The City of Phoenix right-of-way map, together with the assessors map, show this parking lot to be an encroachment on City of Phoenix right-of-way. Based on this information, there will not be a need for any new right-of-way or temporary easement here.

A 10' wide temporary construction easement will be needed along the southwest lot at Oak Street (Figure 7.4-2).

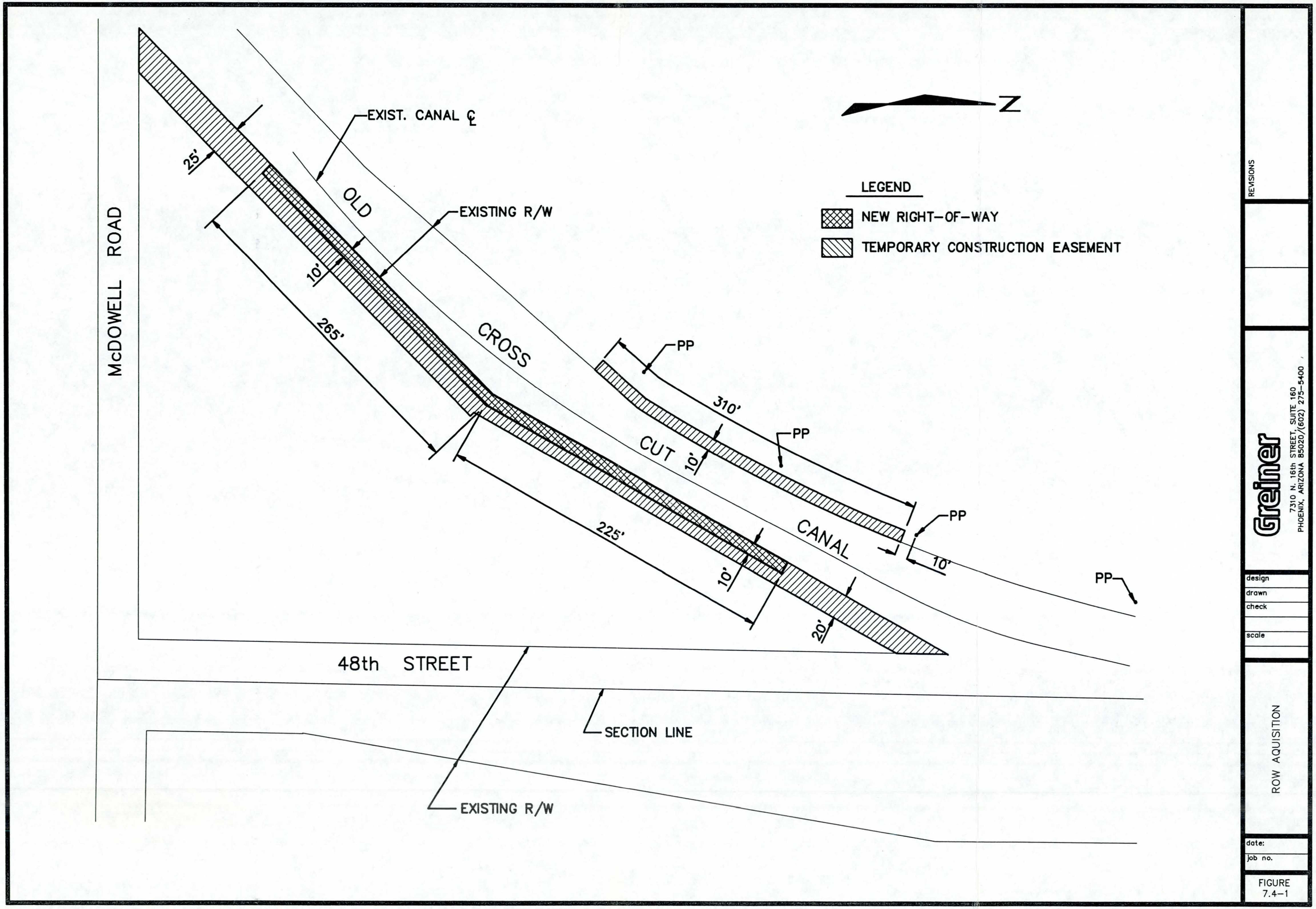
Between McDowell Road and Granada Road on the east side, new 10' wide right-of-way will be needed, as will temporary construction easements of 20' and 25' offsets (Figure 7.4-1). It appears that the edge of the required right-of-way abuts against an existing shed and home. In order to avoid this, it would be necessary to move the CBC alignment 10' or more to the west in this area.

North of McDowell Road on the west side, a 10' wide temporary construction easement will be needed that extends some 310' (Figure 7.4-1).

There are a number of possible temporary construction easements that may be required for side inlet and catch basin construction, depending on their final location and size. Preliminary dimensions and locations are summarized in Table 7.4-1, and are shown on Figures 7.4-2 through 7.4-4.

TABLE 7.4-1
SIDE DRAINAGE TEMPORARY CONSTRUCTION EASEMENTS

Granada	Southeast corner 20' x 30'
Oak	Southeast and northeast corner 25' x 100'
Windsor	Southeast corner 30' x 30'; northeast corner 20' x 20'
Virginia	Southeast corner 20' x 15'; northeast corner 20' x 20'
Thomas	Southeast corner 45' x 60'; northeast corner 45' x 60'
Pinchot	Southeast corner 20' x 50'
Earll	Southeast corner 20' x 20'; triangle northeast corner 20' x 50' and 20' x 50'
Osborn	Southeast corner 15' x 50'; northeast corner 20' x 50' and 10' x 50'
Whitton	Southeast corner 25' x 50'; northeast corner 25' x 50'
Weldon	No easement needed
Indian School	Southeast corner 30' x 60'; northeast corner 30' x 50'


EXIST. CANAL ℄
EXISTING R/W
OLD CROSS CUT CANAL
McDOWELL ROAD
48th STREET
SECTION LINE
EXISTING R/W
25'
10'
265'
225'
10'
20'
310'
10'
10'
PP
PP
PP
PP
LEGEND
NEW RIGHT-OF-WAY
TEMPORARY CONSTRUCTION EASEMENT
REVISIONS
Greiner
7310 N. 16th STREET, SUITE 160
PHOENIX, ARIZONA 85020/(602) 275-5400
design
drawn
check
scale
ROW AQUISITION
date:
job no.
FIGURE
7.4-1

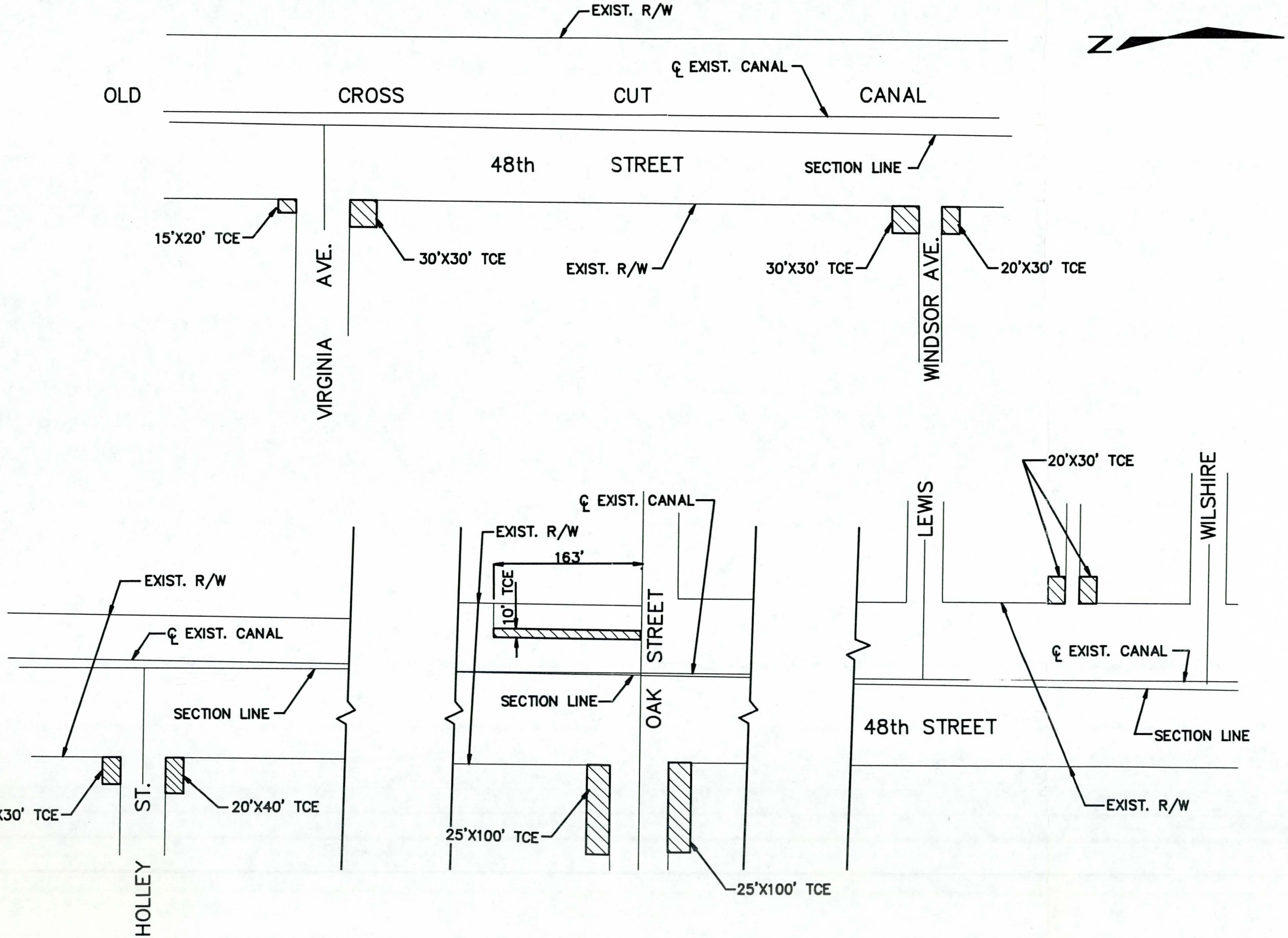


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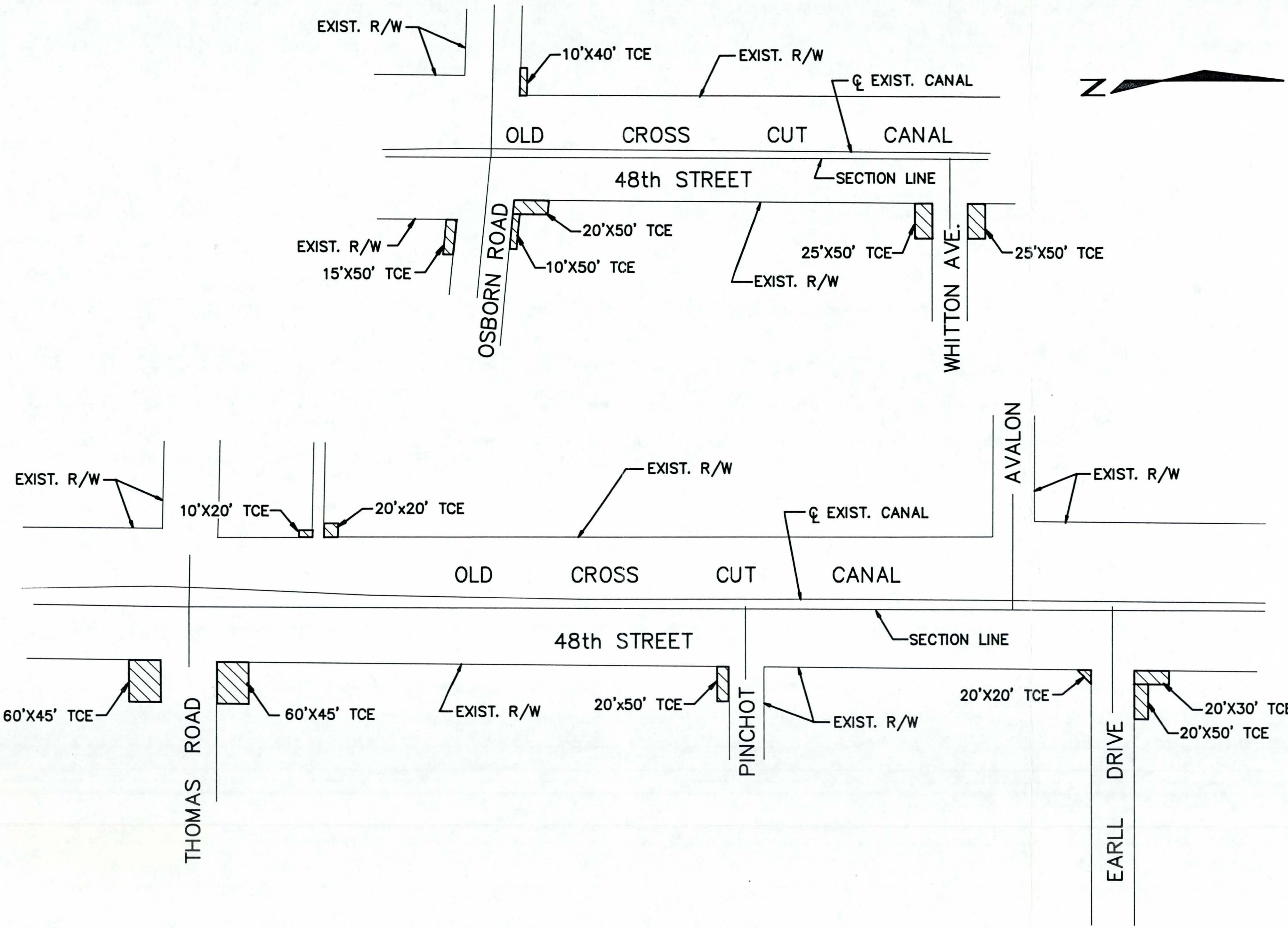
FIGURE 7.4-2



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date:
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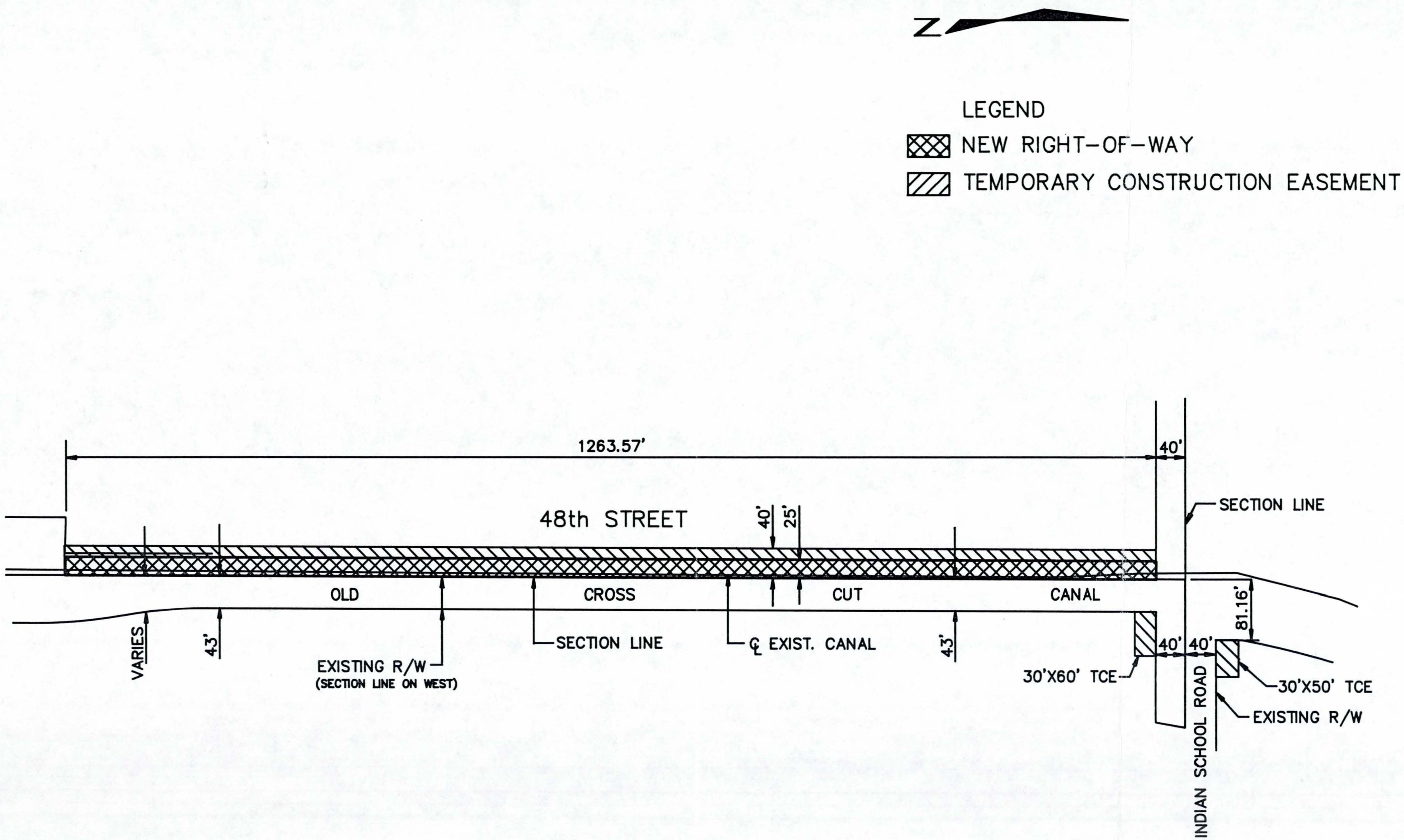
FIGURE 7.4-3



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design
drawn
check
scale

ROW AQUISITION

date:
job no.

FIGURE 7.4-4

7.5 IMPACTS TO THE NEIGHBORHOOD

The OCCC storm drain will significantly impact the surrounding area. Long-term benefits of the proposed facilities will outweigh temporary inconvenience caused by construction. Possible negative impacts of the construction will include traffic interruptions, access limitations, and noise and dust generation in the area.

Positive impacts of the project include the elimination of the open channel section which is currently not fenced. This will increase the safety of the area as well as the aesthetics. The possibility of the relocation of the 12 and 69 kV power lines from overhead to underground would be another significant improvement to the aesthetics of the area. The proposed improvements will also provide greater east/west access along 48th Street which may eventually be used for the Hohokam Parkway and a City of Phoenix park. This would provide recreational facilities and improve traffic flow through the corridor.

7.6 ARCHAEOLOGICAL AND ENVIRONMENTAL IMPACTS

7.6.1 Archaeological Concerns

The Flood Control District recognizes the importance of historic and prehistoric cultural resources and the educational, recreational and scientific sources of information they provide for the community as a whole.

Research and review of available literature have produced some evidence of buried archaeological resources along the proposed channel improvements between McDowell and Indian School Roads. This section is located within an area of approximately 30,000 square miles in southern Arizona that was once inhabited by the Hohokam Indians. The Hohokam community existed from approximately 800 A.D. to 1450 A.D.

The Hohokam Indians are credited with the construction of as many as 200 miles of canals in the Salt River Valley. Their settlements began as simple groups of pit dwellings and culminated in large above-ground apartment houses made of adobe, stone and wood. The remains of one such settlement can be found at Pueblo Grande. It has been classified as a National Landmark and is owned and operated as a museum by the City of Phoenix.

The Pueblo Grande settlement may have included sites as far north as McDowell Road. It is possible that special-use sites (cemeteries, farmland, ball courts, etc.) existed even further north than the settlement. The exact location of the sites, if any, may be difficult to identify due to the urban development in this area. The land has already been disturbed to a considerable extent by cultivation and urban development.

Based on a review of available literature and discussions with Pueblo Grande Museum staff, there is no record or knowledge of any buried archaeological sites of any significance within this segment of the proposed channel improvement corridor. However, prior storm drain excavation along Van Buren Street at the OCCC exposed numerous burials. Excavation work for the new conduit should be carefully monitored and the appropriate city and state agencies contacted, should any Indian cultural remains be found.

Based on discussions with the Historic Preservation Branch of the Arizona State Parks Department staff, two buildings classified on the historic register are located within the channel improvement corridor. Rancho Joaquina at 4630 E. Cheery Lynn Road is an authentic adobe structure built in the early 1920's. Listed on the National Register, Rancho Joaquina is a two-story, single-family dwelling consisting of approximately 6,700 square feet. The other historical building within the corridor is the Michael John Murphy building located at 4900 E. Thomas Road. It was built some time during the 1920's-1930's and is also constructed of adobe. Owned by Creighton United Methodist Church, it is currently used for an office and classrooms for the church.

Rancho Joaquina and the Michael John Murphy building are outside the limits of the improvements, so they will not be affected.

In order to preserve any important archeological or historical findings, a section covering this subject will be developed and included in the final contract specifications which shall be similar to the following:

a. General - Federal legislation provides for the protection, preservation and collection of scientific, prehistorical, historical and archeological data, including relics and specimens, which might otherwise be lost due to alteration of the terrain as a result of any federal construction project.

   Should the Contractor or any of the Contractor's employees, or parties operating or associated with the Contractor in performance of this contract discover evidence of possible scientific, prehistoric, historical or archeological data, the Contractor shall immediately cease work at that location and notify the Contracting Officer, giving the location and nature of the findings. The Contractor shall forward written confirmation to the Contracting Officer within two days. The Contractor shall exercise care so as not to disturb or damage artifacts or fossils uncovered during excavation operations, and shall provide such cooperation and assistance as may be necessary to preserve the findings for removal or other disposition by reclamation.

   Any person who, without permission, injures, destroys, excavates, appropriates or removes any historical or prehistorical artifact, object of antiquity, or archeological resource on the public lands of the United States is subject to arrest and penalty under law.

   Where appropriate by reason of discovery, the Contracting Officer may order delays in the time of performance or changes in the work, or both. If such delays or changes are ordered, an equitable adjustment will be made in the contract in accordance with the applicable clauses of the contract.

b. Cost - Except as provided above, the cost of complying with this paragraph shall be included in the prices bid in the schedule for other items of work.

7.6.2 Environmental Impacts

Short-term environmental impacts will occur through the duration of the construction activity within the project corridor that will affect adjacent properties.

Existing land uses along the project corridor beginning at the southern end (at McDowell Road) consist of a mix of commercial and multi-family residential uses with commercial establishments clustered at the intersection of McDowell Road and 48th Street, and along the north side of McDowell Road to 46th Street.

North of McDowell Road to Oak Street, the corridor is predominantly single-family residential dwellings. Isolated multi-family units and a local market located at Holly Street constitute the remaining uses.

The majority of the area between Oak Street and Thomas Road contains medium density single-family residential dwellings, transitioning to commercial properties at Thomas Road. St. Francis Cemetery is located in the northeast and southeast corners of Oak Street and the proposed roadway. Fire Station 13 is located near the southwest corner of the Thomas Road intersection.

North of the commercial property on Thomas Road to the intersection at Indian School Road, the corridor is primarily medium density single-family residential with one townhome community located to the north of Earll Drive. Arcadia High School is located near the southwest corner of Indian School Road adjacent to the R&G Ranch. Commercial development covers the remaining corners of this intersection.

The construction and development of the proposed project will result in temporary noise increases within the improvement area. The noise would be generated primarily from heavy equipment used in hauling materials and building the roadway. Sensitive areas located close to the construction alignment may temporarily experience increased noise levels.

Possible provisions to reduce the effects of construction noise may include the limitation of certain construction activities during the evenings, weekends or holidays; and the location of storage and staging facilities away from noise sensitive areas.

Construction activities can have a short-term impact on local air quality primarily during periods of site preparation, with particulate matter (dust) having the greatest impact. This impact would occur in association with excavation and earth moving, cement, asphalt and aggregate handling, heavy equipment operation, use of haul roads and wind erosion of exposed areas and materials storage piles.

The effects of dust would be temporary and would vary in scale depending on local weather conditions, level of construction activity and the nature of the operation. Where excess dust is likely to become a problem, effective dust control measures can be implemented including:

- o minimization of exposed erodible earth area to the extent possible;
- o stabilization of exposed earth with grass, mulch, pavement or other core as early as possible;
- o periodic sweeping or application of water or stabilizing agents to the working and haulage areas;
- o covering, shielding or stabilizing of stockpiled material as necessary; and
- o the use of covered haul trucks.

7.7 CONSTRUCTIBILITY

Construction of the new box culverts could be staged from south to north in approximate quarter reaches as follows:

1. McDowell Road to Oak Street

2. Oak Street to Thomas Road

   - o Thomas Road will be constructed in phases with one half remaining open at all times

3. Thomas Road to Osborn Road

   - o Osborn Road will be completely closed during construction

4. Osborn Road to Indian School Road

   - o Indian School Road will be constructed in phases with one half remaining open at all times

More detailed descriptions of how these intersections will function during construction can be found in Section 7.1 of this report.

During construction, excavation material can generally be stored adjacent to the ditch. In some areas due to access constraints, as described in Section 7.3, the Contractor may need to haul excavated material to another location along the canal corridor. Possible storage locations are:

1. The northwest corner of the McDowell Road/OCCC intersection.

2. Along 48th Street with a barricaded area approximately 600' north of McDowell Road where the OCCC channel transitions parallel to 48th Street.

3. Along west 48th Street parallel to the new box culverts between Monte Vista Road and Edgemont Avenue.

4. Along west 48th Street parallel to the new box culverts between Thomas Road and Osborn Road.

Hauling should only be necessary either within the reach under construction or to the reach directly south of the area under construction.

Other than the access constraints noted in Section 7.3, complete access to the job site will be available for the Contractor along east and west 48th Street.

The Contractor will also need to handle possible storm runoff during construction. Local flows must be routed around the site without damage to the new construction or to the surrounding areas. To do this, small retention ponds upstream of the work in progress could be excavated and, with the use of sump pumps, runoff could be pumped back into the completed conduit to the south.

SRP has dewatering pumps and wells in the area and they have indicated that these could be available for the Contractor's use should high groundwater levels become a problem during construction.

Bracing of excavation may be needed in areas where utilities or right-of-way conflicts exist. Cantilever or tied-back systems may be used at the discretion of the Contractor. However, as stated in the soils report, a bracing design, sealed by a registered professional engineer, will be required for review and approval by the geotechnical engineer. Results of the soils report can be found in Section 4.

Underdrain construction may be required under the new conduit, depending on the final invert location.

The Contractor must also be made aware of the 12 kV and 69 kV overhead power lines along the canal corridor. At least one overhead line runs parallel to the canal, almost the entire length of the job. In some cases, two parallel lines exist--one on each side of the canal. Also, there are several crossing lines at various locations. This will restrict the height and type of equipment that can be used.

A temporary increase in the noise levels will be felt in the neighborhoods surrounding the construction. The majority of the area is single-family homes or townhomes, transitioning to light commercial properties near arterial roadway intersections. To mitigate the effects of construction noise, certain activities may be restricted during evenings, weekends and holidays. Storage areas and staging facilities should be located away from noise-sensitive areas.

Construction of the new improvements will temporarily generate dust which will have an impact on the local air quality. Wherever excess dust becomes a problem, the Contractor must implement effective dust control measures.

Reducing the effects of construction noise and dust control methods are discussed in greater detail in Section 7.6.2 of this report.

8.0 IMPROVEMENTS SPECIFICATIONS

8.1 FACILITIES

The following is a list of facilities that are recommended as part of the improvements to the OCCC. The sizes and lengths listed may be subject to change as the project progresses.

Extend headwall at McDowell Road

Construct 54" stub-out and catch basins at McDowell Road

Construct 1,074' of double 18' x 10' CBC

Construct 48" stub-out and catch basins at Granada Road

Construct 603' of double 18' x 10' CBC

Construct 48" stub-out and catch basins at Holly Road

Construct 1,003' of double 18' x 10' CBC

Construct double 10' x 7' stub-out and catch basins at Oak Street

Construct 1,309' of double 18' x 10' CBC

Construct 10' x 6' stub-out and catch basins at Virginia Avenue

Construct 491' of double 18' x 10' CBC

Construct 18" RCP stub-out and catch basins at N. of Virginia Avenue

Construct 160' of double 18' x 10' CBC

Construct 42" RCP stub-out and catch basins at Windsor Avenue

Construct 734' of double 18' x 10' CBC

Construct 5' x 5' stub-out and catch basins at Thomas Road

Extend conduit underneath Thomas Road

Construct 712' of double 18' x 10' CBC

Construct 24" stub-out and catch basins at Pinchot Avenue

Construct 534' of double 18' x 10' CBC

Construct double 10' x 7' CBC stub-out and catch basins at Earll Drive

Construct 100' transition structure double 18' x 10' to double 12' x 10'

Construct 630' of double 12' x 10' CBC

Construct 30" stub-out and catch basins at Richardson

Construct 709' of double 12' x 10' CBC

Construct 7' x 4' CBC stub-out and catch basins at Osborn Road

8.0 IMPROVEMENTS SPECIFICATIONS

Construct 346' of double 12' x 10' CBC

Extend conduit underneath Osborn Road

Construct 18" stub-out and catch basins at Whitton Road

Construct 731' of double 12' x 10' CBC

Construct 48" stub-out and catch basins at Weldon Road

Construct 100' transition structure double 12' x 10' to double 10' x 10'

Construct 1,543' of double 10' x 10' CBC

Extend conduit underneath Indian School Road

Construct 24" stub-out and catch basins at Indian School Road

Construct transition to natural channel

Construct access manholes at 500'$\pm$ intervals

9.0 CONSTRUCTION SCHEDULE AND COST ESTIMATE

9.1 CONSTRUCTION SCHEDULE

Implementing the proposed improvements to the OCCC will involve coordinating construction of the main line box culvert, construction of various side inlets from the east, traffic flow restrictions, and public and service access. Due to the size and complexity of the project, it is important to develop a construction schedule geared to minimize traffic disruption, construction time and impacts to residential and commercial properties along the project corridor.

There is a possibility of separating the construction of this project into a utility relocation contract and a main line construction contract. The utility relocation portion would deal with relocating the various utilities that would ultimately be affected by the proposed storm drain. The second contract would deal exclusively with construction of the OCCC improvements.

Because of access requirements, the intersections at Holly Street, Virginia Avenue, Windsor Avenue and Earll Drive will need to remain open during construction. These intersections will also be impacted by construction of side inlets and catch basins. The results of maintaining traffic through the intersections while constructing the main line CBC, as well as the side inlets and catch basins could be a complicated traffic control problem. It may be desirable to construct the side inlets and the main line conduit separately at these locations. Coordination with the City of Phoenix Traffic Department will be essential in determining their requirements at these critical locations.

As mentioned in Section 7.7, a preliminary plan for the construction of the Old Cross Cut storm drain could proceed from south to north in quarter reaches of the complete job. These reaches include:

1. From McDowell Road to Oak Street

2. From Oak Street to Thomas Road

   o Thomas Road intersection - construct half at a time

3. From Thomas Road to Osborn Road

   o Osborn Road intersection - complete closure of the intersection

4. From Osborn Road to Indian School Road

   o Indian School Road intersection - construct half at a time

Splitting the construction into these four parts minimizes the restricted time needed at any one area and minimizes the resources needed at any one time for traffic control.

Right-of-way restrictions and constraints in several areas along the construction corridor may cause delays. These areas include the property east of the canal just south of Indian School Road held by the security company, the property south of the Republic/Gazette property on the west side of the canal, the property west of the canal south of Oak Street belonging to Mr. Royce, and the property along both sides of the canal route north of McDowell Road.

A major factor in deciding to proceed from south to north is the required relocation of existing power poles along the canal and the long lead time requested by SRP in relocating the existing power poles. The majority of the power pole relocations occur in the north half of the job. By scheduling this construction last, more advanced time could be given to relocate these poles. Of course, if these relocations could be completed prior to starting the main line construction, this would not be a critical issue in construction scheduling.

Another consideration in south to north construction is nuisance water. The Contractor will need to provide a way to pass incidental flows along the canal and possible storm runoff through the conduit under construction without damaging the unfinished sections. This could require excavating ahead of the construction, and incorporating local, small retention ponds upstream utilizing sump pumps to pump water to the south, thus bypassing any uncured conduit sections. In treating nuisance flows, it appears that the south to north or outworks to headworks construction will be the best way of handling this problem.

Another factor affecting scheduling is the construction of the side inlet at Thomas Road. This inlet will be a large CBC and will need excavation and construction space inside of the intersection. Placement of the side inlet will, therefore, need to be considered in light of keeping half the intersection open at all times for east/west traffic. Various horizontal locations will be investigated to best accommodate traffic flow during construction. Also, separate construction for the main line conduit and the side inlet may be required to meet traffic demands. As mentioned, coordination with the City of Phoenix Traffic Department will be essential in establishing criteria to best serve their needs during construction of this project.

Individual utility relocation windows will be a major factor in construction scheduling. SRP irrigation lines are used on a cyclical basis--two days on and 12 days off between April and October, and two days on and 28 days off between November and March. Small changes in scheduling are permitted by SRP with prior notice. Water and electric utilities favor winter shutdowns. Southwest Gas prefers that its lines crossing the canal be cut and capped between April through September.

There are several projects that will involve construction within this project's boundary in the future. These include a 12" and 16" water line through Indian School Road (COP Project No. W-888982). Consideration of the construction of these lines should be taken into account in the overall project scheduling to avoid multiple openings of the intersection. Another upcoming project with possible implications to utility relocations in this job is the proposed 15" sewer line (COP Project No. S-875427) planned to be built along east 48th

Street and tying into an existing sewer in McDowell Road. If the existing 8", 10" and 12" sewer crossings could be tied to this proposed line, a significant savings in money and time to relocate could result.

A detailed construction schedule, including all of the individual utility shutdown requirements will not be addressed in this report. Further meetings and coordinations will take place as the project design progresses to develop the required detailed construction schedule.

9.2 COST ESTIMATE

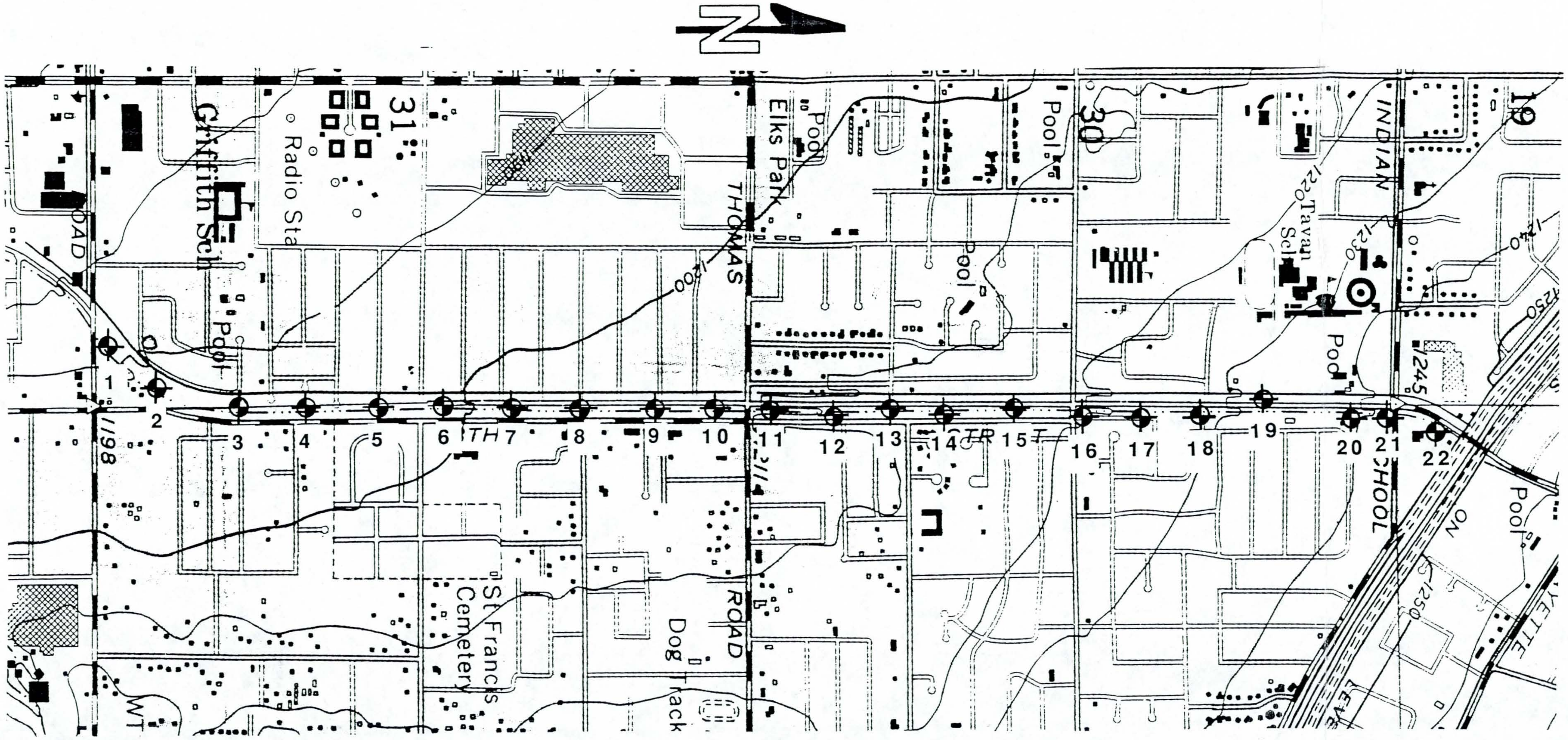
The following is a preliminary cost estimate for construction of the OCCC storm drain. It includes removal and replacement of the overcrossings at Indian School Road, Osborn Road and Thomas Road, and is based on the recommended box culvert configuration with excavation and backfill included in the unit cost for the various box culvert sizes, using cast-in-place construction. Transition structures are included in the cost of the various box culvert configurations. The preliminary utility relocation cost estimate is also included as a total sum. For the detailed utility relocation cost estimate, see Section 5.2 of this report. Costs for the acquisition for permanent right-of-way and temporary construction easements are not included in this estimate.

Item No.	Bid Item	Quantity	Unit	Unit Cost	Total
301	Subgrade preparation	1,150	SY	$4.00	$4,600.00
313	Aggregate base course	250	TON	15.00	3,750.00
321	Asphalt concrete pavement	100	TON	35.00	3,500.00
340.1	Concrete curb and gutter	360	LF	7.00	2,520.00
340.2	Concrete sidewalk	2,060	SF	2.00	4,120.00
350	Remove existing improvements	1	LS	90,000.00	90,000.00
401	Traffic control	1	LS	60,000.00	60,000.00
618.1	Double 18' x 10' concrete box culvert	6,600	LF	1,605.00	10,593,000.00
618.2	Double 12' x 10' concrete box culvert	2,200	LF	1,240.00	2,728,000.00
618.3	Double 10' x 10' concrete box culvert	2,000	LF	1,095.00	2,190,000.00
618.4	Double 10' x 7' concrete box culvert	60	LF	590.00	35,400.00
618.5	10' x 6' concrete box culvert	80	LF	320.00	25,600.00

9.0 CONSTRUCTION SCHEDULE AND COST ESTIMATE

Item No.	Bid Item	Quantity	Unit	Unit Cost	Total
618.6	7' x 4' concrete box culvert	40	LF	220.00	8,800.00
618.7	5' x 5' concrete box culvert	90	LF	175.00	15,750.00
618.8	54" reinforced concrete pipe	130	LF	110.00	14,300.00
618.9	48" reinforced concrete pipe	220	LF	90.00	19,800.00
618.10	42" reinforced concrete pipe	85	LF	80.00	6,800.00
618.11	33" reinforced concrete pipe	40	LF	70.00	2,800.00
618.12	30" reinforced concrete pipe	100	LF	60.00	6,000.00
618.13	24" reinforced concrete pipe	370	LF	50.00	18,500.00
618.14	18" reinforced concrete pipe	260	LF	40.00	10,400.00
618.13	12" reinforced concrete pipe	200	LF	35.00	7,000.00
618.14	10" reinforced concrete pipe	40	LF	30.00	1,200.00
618.15	Catch basin construction	1	LS	90,000.00	90,000.00
625	Access manholes	22	EA	5,000.00	110,000.00
	Utility relocation total	1	LS	1,860,000.00	1,860,000.00
				Construction total	**$17,912,340.00**
				Contingency 15 percent	**2,686,851.00**
				GRAND TOTAL	**$20,599,191.00**
	Land acquisition Right-of-way Temporary construction easement	 55,760 71,330	 SF SF		

APPENDIX A
FIELD RESULTS



LEGEND:

Test Boring Location

Elevations approximated from Plan and Profile provided by Greiner Engineering

NOTE: Test borings 3, 8, 13, 16, & 21 completed as piezometer wells

Site Plan

Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

LEGEND

SOIL CLASSIFICATION

COARSE-GRAINED SOIL

More than 50% larger than 200 sieve size

SYMBOL	LETTER	DESCRIPTION	MAJOR DIVISIONS
	GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES. LESS THAN 5% - #200 FINES	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
	GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES. LESS THAN 5% - #200 FINES	
	GM	SILTY GRAVELS. GRAVEL-SAND-SILT MIXTURES. MORE THAN 12% - #200 FINES	
	GC	CLAYEY GRAVELS. GRAVEL-SAND-CLAY MIXTURES. MORE THAN 12% - #200 FINES	
	SW	WELL-GRADED SANDS OR GRAVELLY SANDS. LESS THAN 5% - #200 FINES	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
	SP	POORLY-GRADED SANDS OR GRAVELLY SANDS. LESS THAN 5% - #200 FINES	
	SM	SILTY SANDS. SAND-SILT MIXTURES MORE THAN 12% - #200 FINES	
	SC	CLAYEY SANDS. SAND-CLAY MIXTURES MORE THAN 12% - #200 FINES	

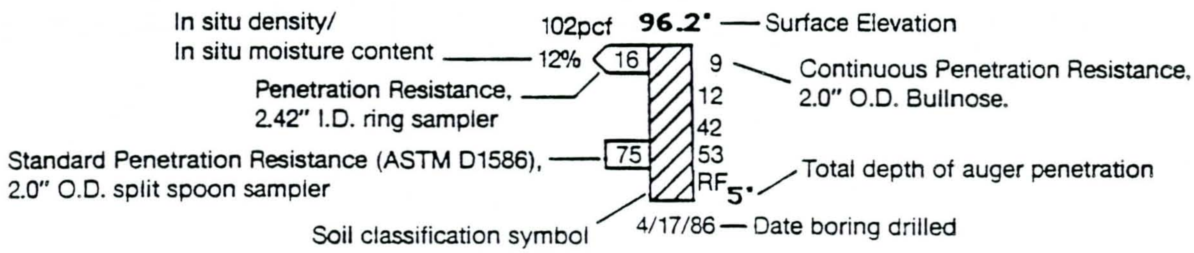
FINE-GRAINED SOIL

More than 50% smaller than 200 sieve size

SYMBOL	LETTER	DESCRIPTION	MAJOR DIVISIONS
	ML	INORGANIC SILTS. ROCK FLOUR. AND FINE SANDY OR CLAYEY SILTS OF LOW TO MEDIUM PLASTICITY	SILTS AND CLAYS Liquid limit less than 50
	CL	INORGANIC CLAYS. GRAVELLY CLAYS. SANDY CLAYS. SILTY CLAYS. AND LEAN CLAYS OF LOW TO MEDIUM PLASTICITY	
	OL	ORGANIC SILTS AND ORGANIC SILT-CLAY MIXTURES OF LOW TO MEDIUM PLASTICITY	
	MH	INORGANIC SILTS. MICACEOUS OR DIATOMACEOUS. AND FINE SANDY OR CLAYEY SILTS OF HIGH PLASTICITY	SILTS AND CLAYS Liquid limit greater than 50
	CH	INORGANIC CLAYS. FAT CLAYS. AND SILTY CLAYS OF HIGH PLASTICITY	
	OH	ORGANIC CLAYS AND ORGANIC SILTS OF MEDIUM TO HIGH PLASTICITY	
	PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	

LEGEND FOR GRAPHICAL BORING LOGS:

Log denotes visual approximation unless accompanied by mechanical analysis and Atterberg limits.



PENETRATION RESISTANCE: Blows per foot using 140 lb. hammer with 30" free-fall unless otherwise noted.

GRAIN SIZES							
	U.S. STANDARD SERIES SIEVE: 200	40	10	4	CLEAR SQUARE SIEVE OPENINGS: 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 (Plastic Limit)	MOIST	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).

Project No. 90-0863

LEGEND OF SOIL TYPES

<u>ASPHALT CONCRETE, AGGREGATE BASE</u>

<u>CLAYEY SAND/SANDY CLAY</u> **(SC, SC/CL, CH, CL);** light brown to reddish brown; sand fraction primarily fine to medium; occasional to frequent zones with traces to some small gravel, predominantly consisting of subangular to angular granite fragments; scattered gravelly lenses or layers; medium dense to dense; moderate to high plasticity; variable light to heavy calcareous cementation; damp to wet.

▽ 1-11-91 Approximate groundwater level encountered during drilling.

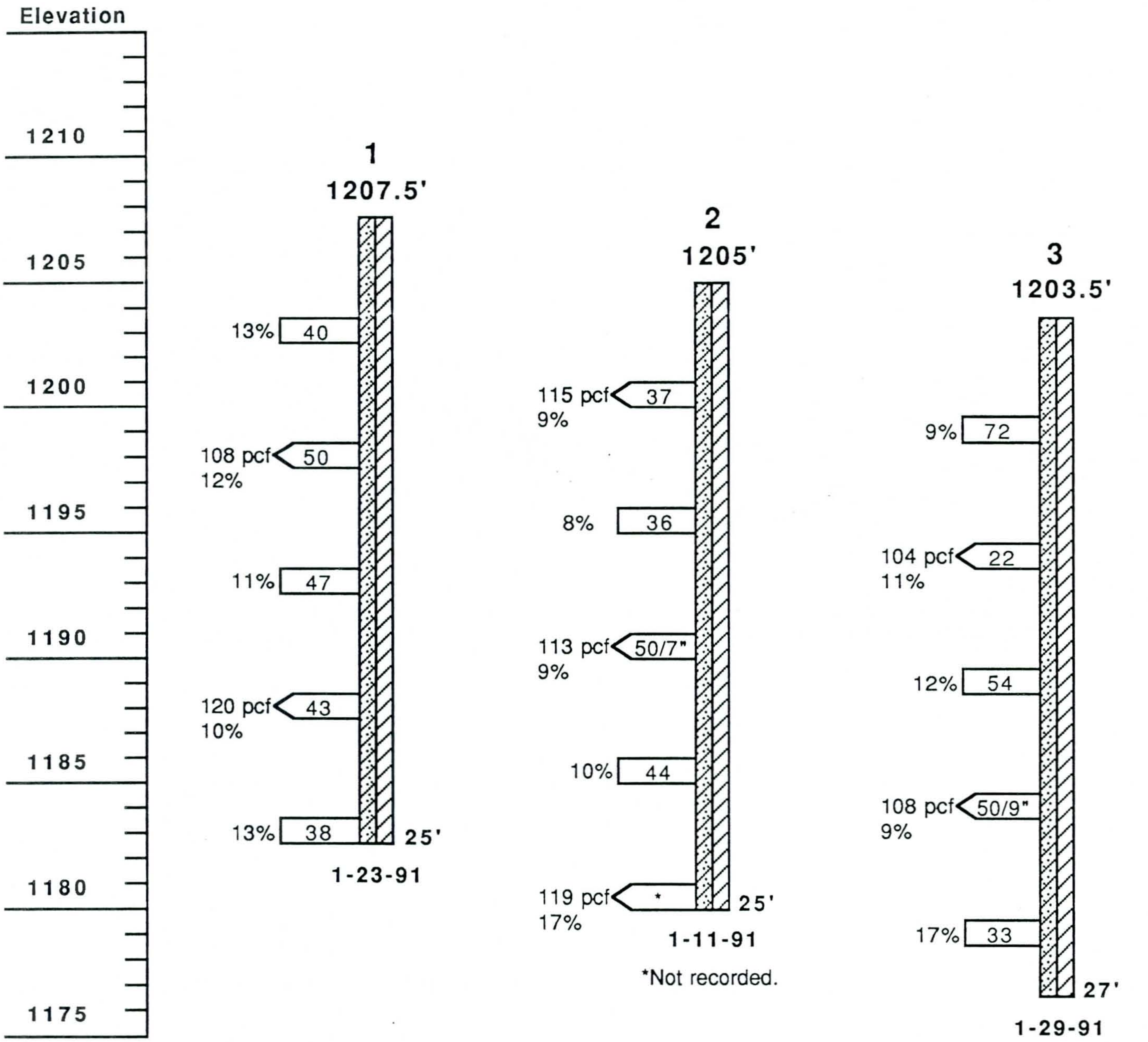
▼ 1-11-91 Stabilized groundwater level observed in temporarily open hole or piezometer.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



NOTE: Completed as a piezometer. See Schematic Well Construction Plan.

No free groundwater was encountered in the test borings during drilling unless otherwise noted.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

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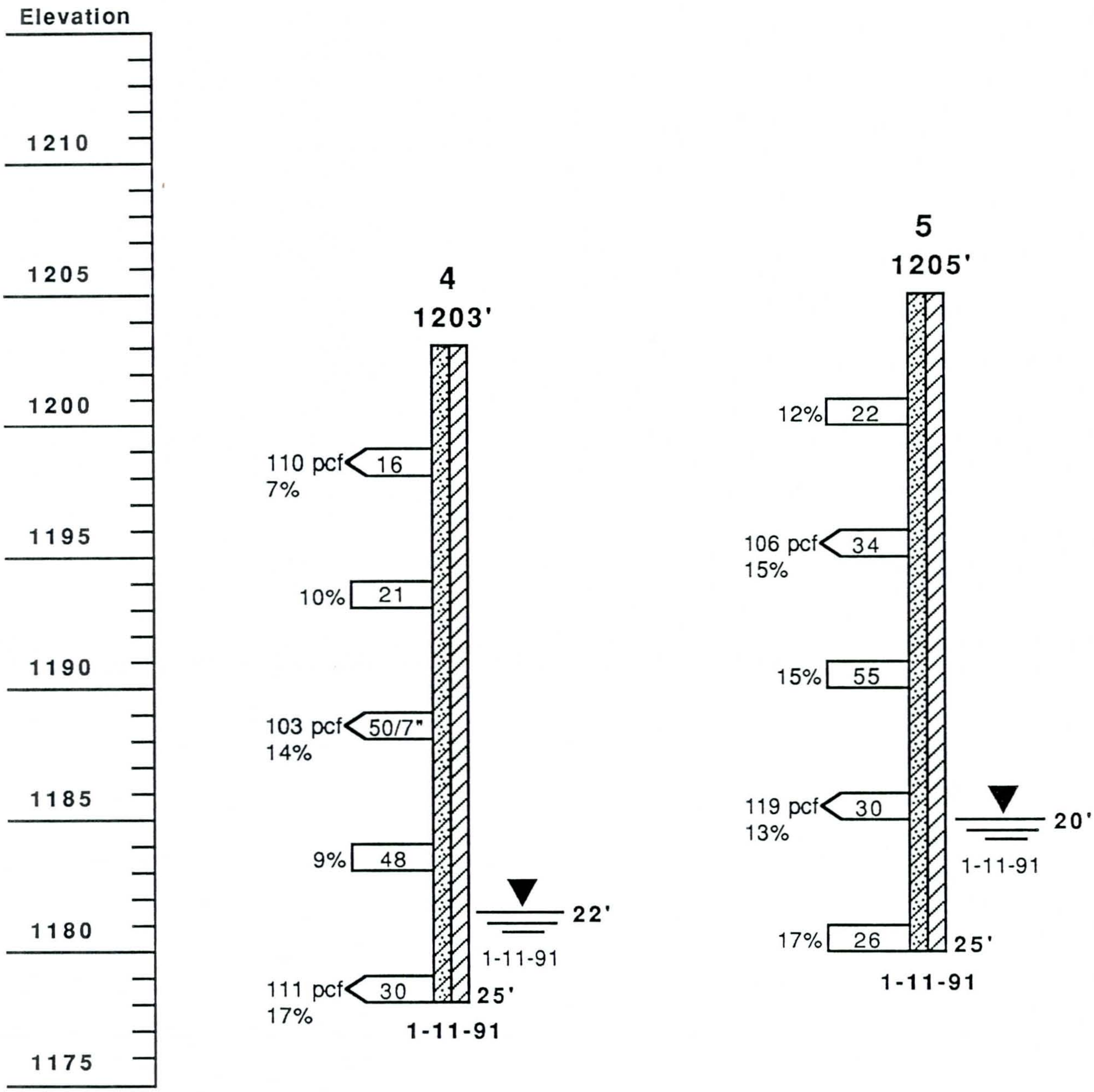

Free groundwater was encountered in the test borings during drilling as noted above.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

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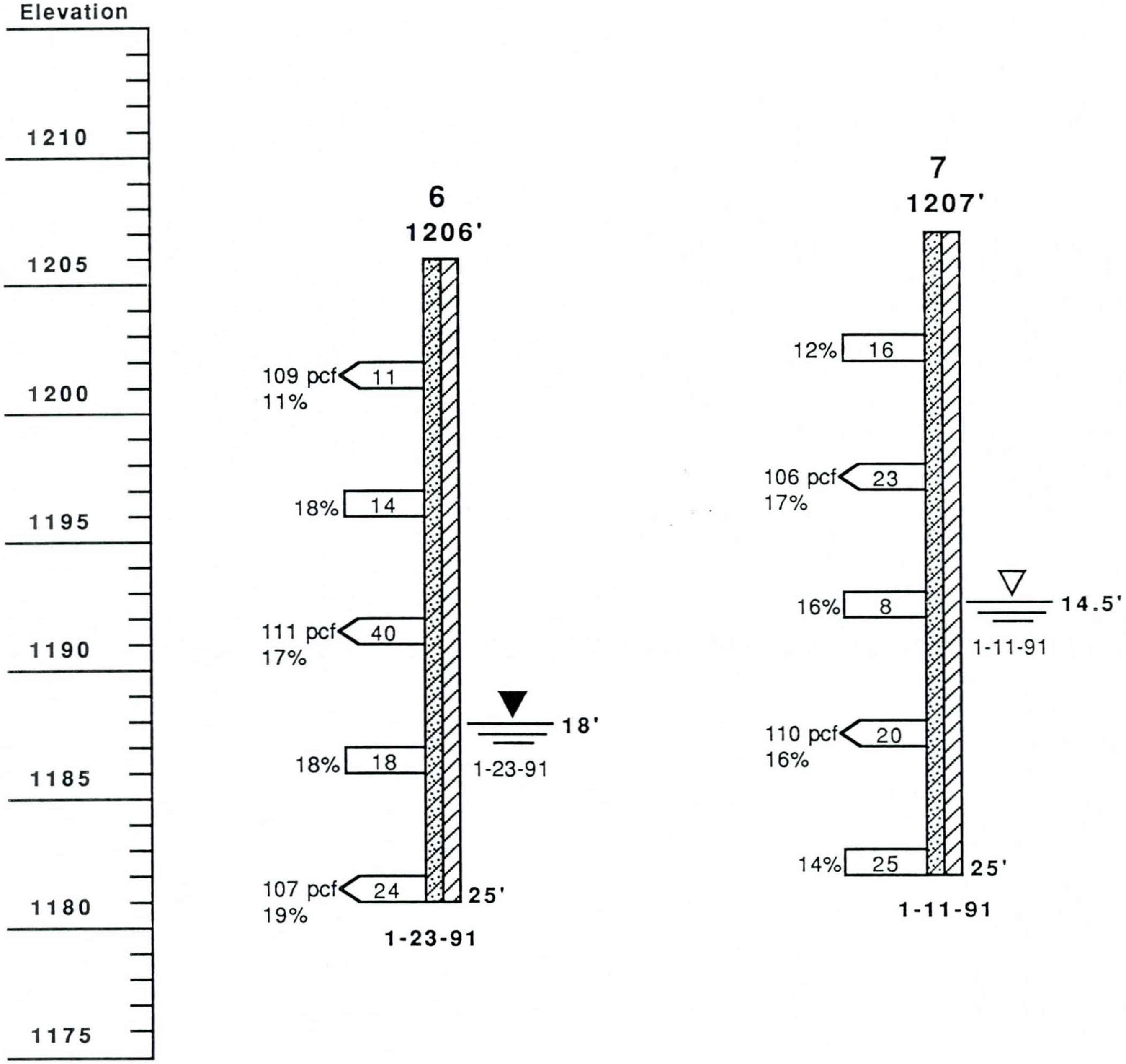

Free groundwater was encountered in the test borings during drilling as noted above.

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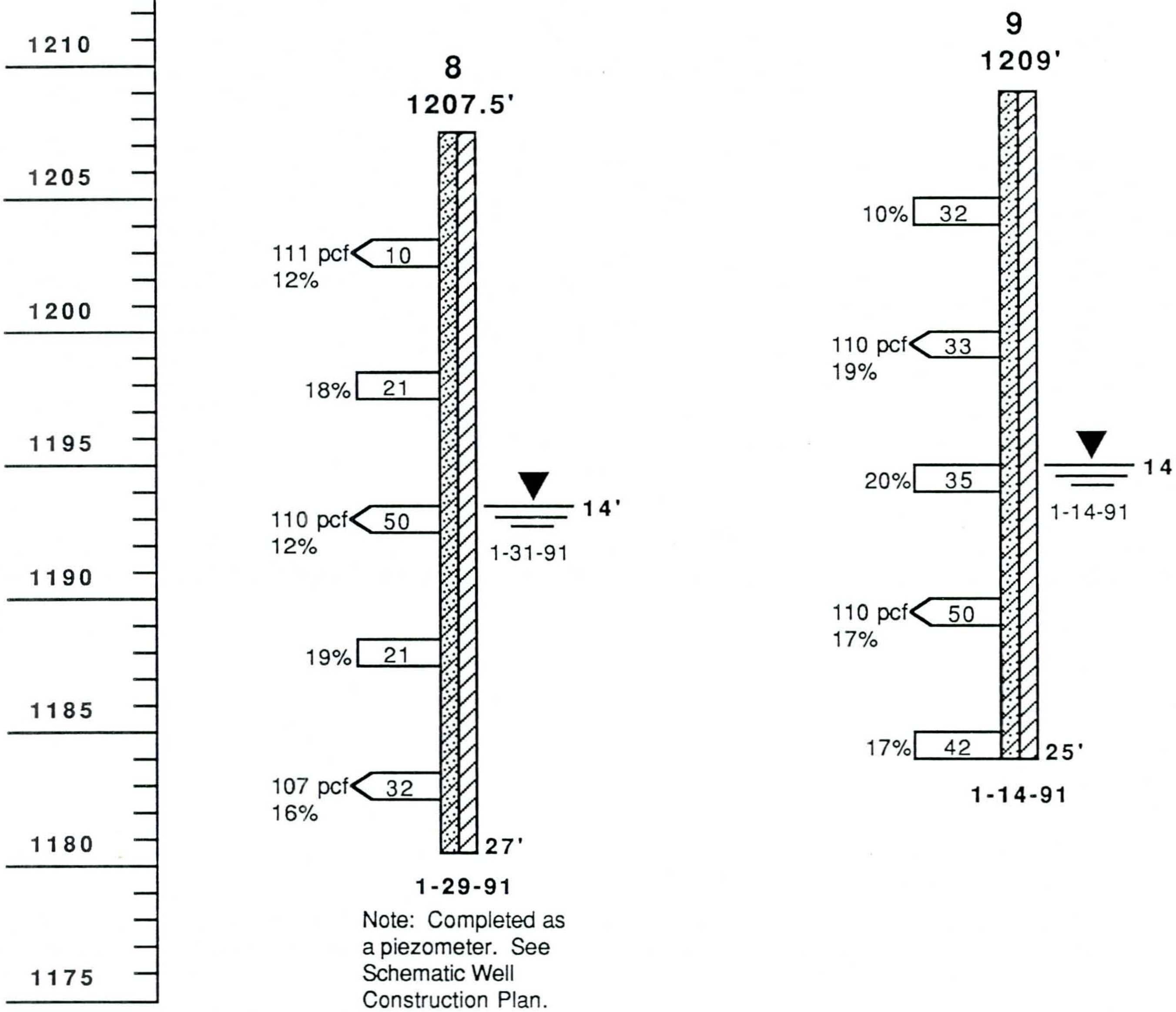

Note: Completed as a piezometer. See Schematic Well Construction Plan.

Free groundwater was encountered in the test borings during drilling as noted above.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

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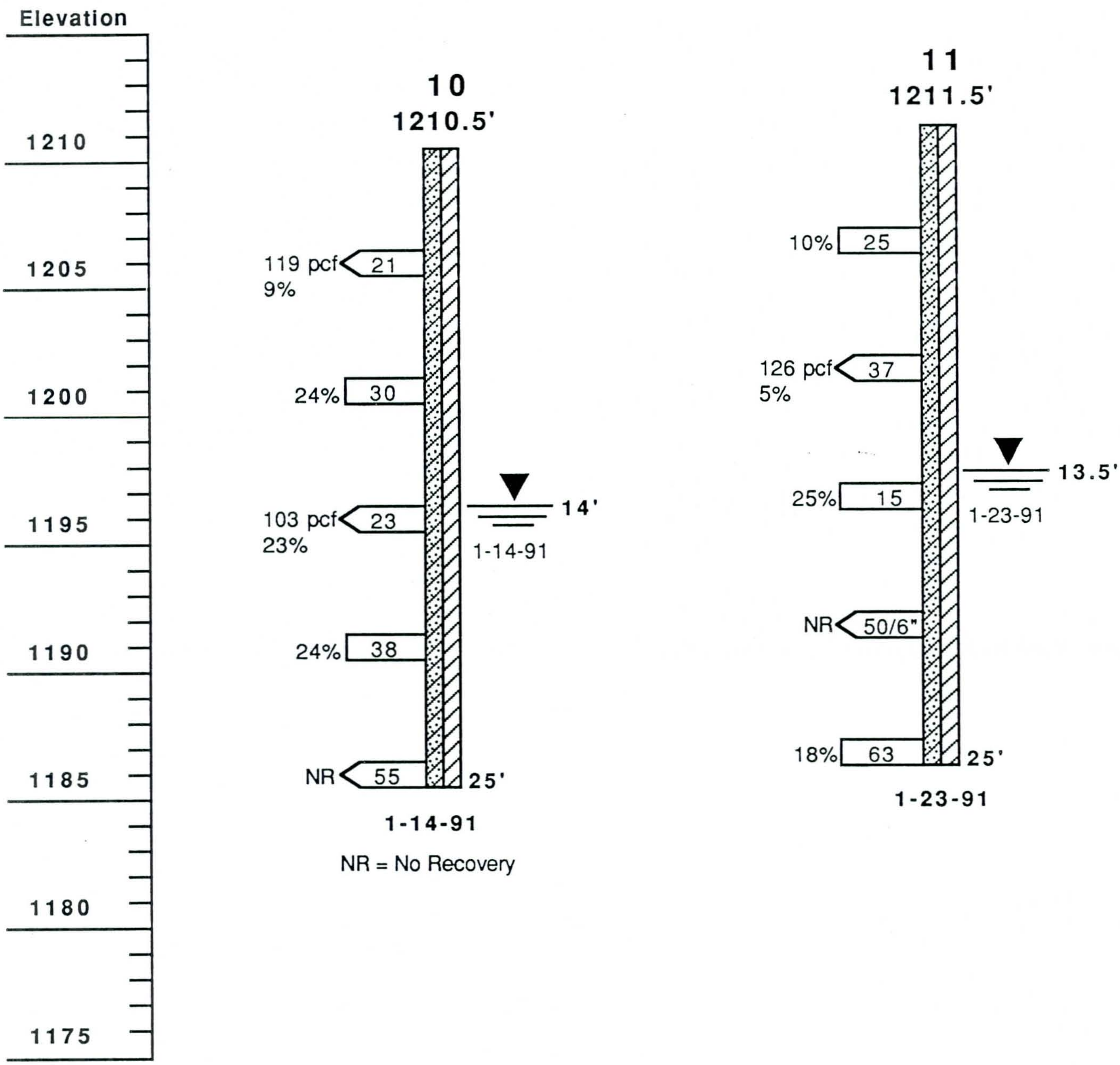


NR = No Recovery

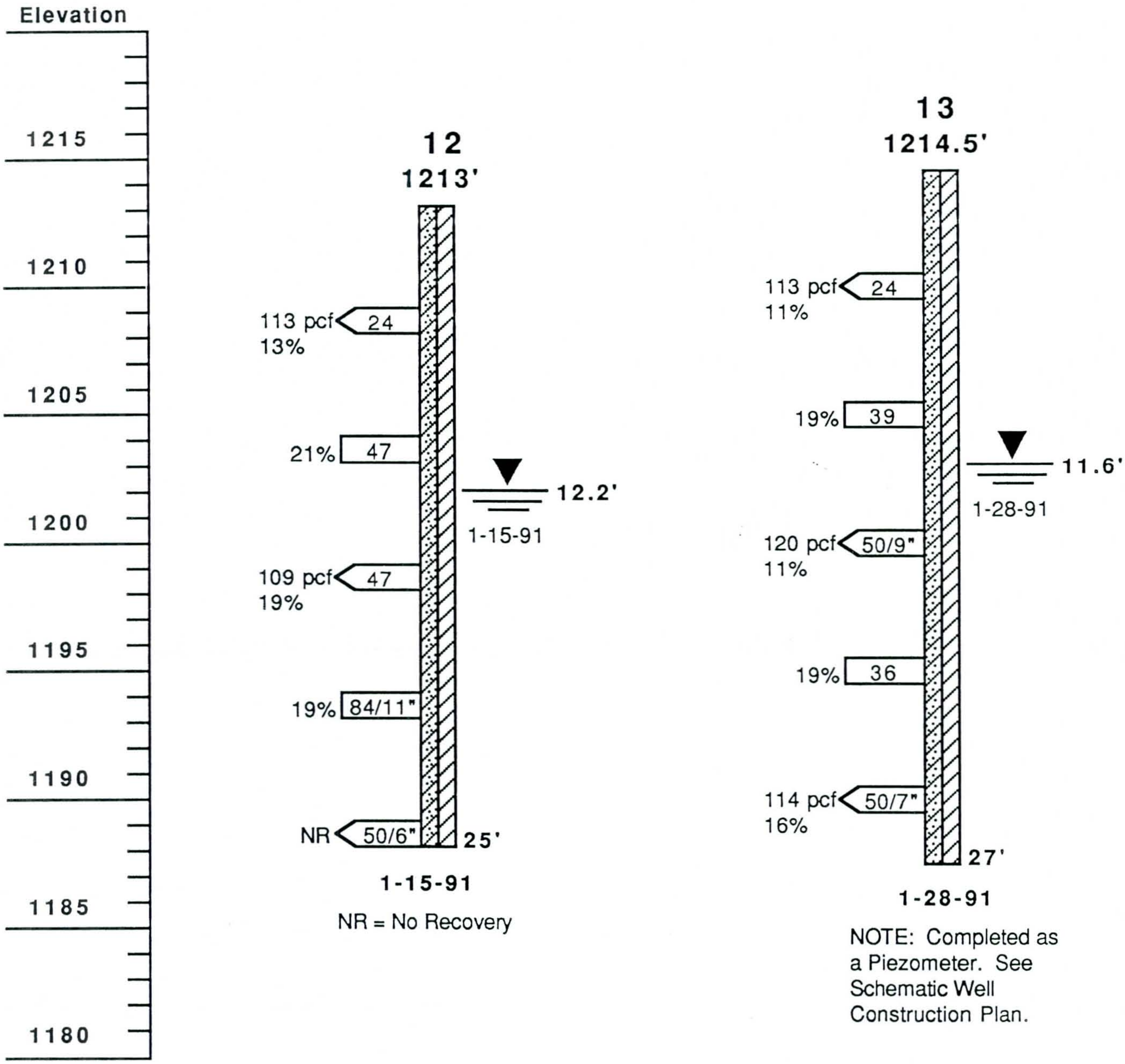
Free groundwater was encountered in the test borings during drilling as noted above.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



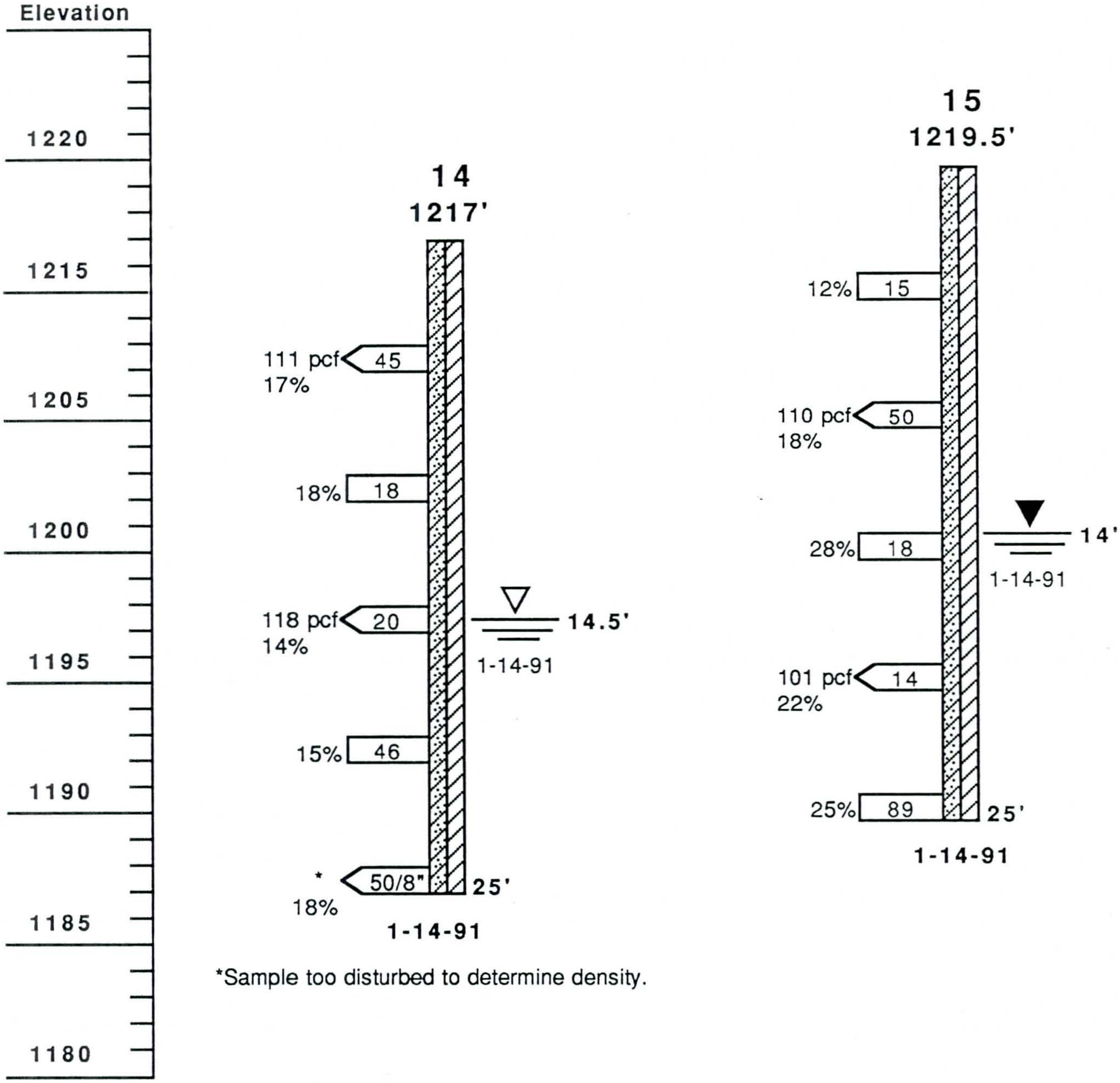
Free groundwater was encountered in the test borings during drilling as noted above.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



*Sample too disturbed to determine density.

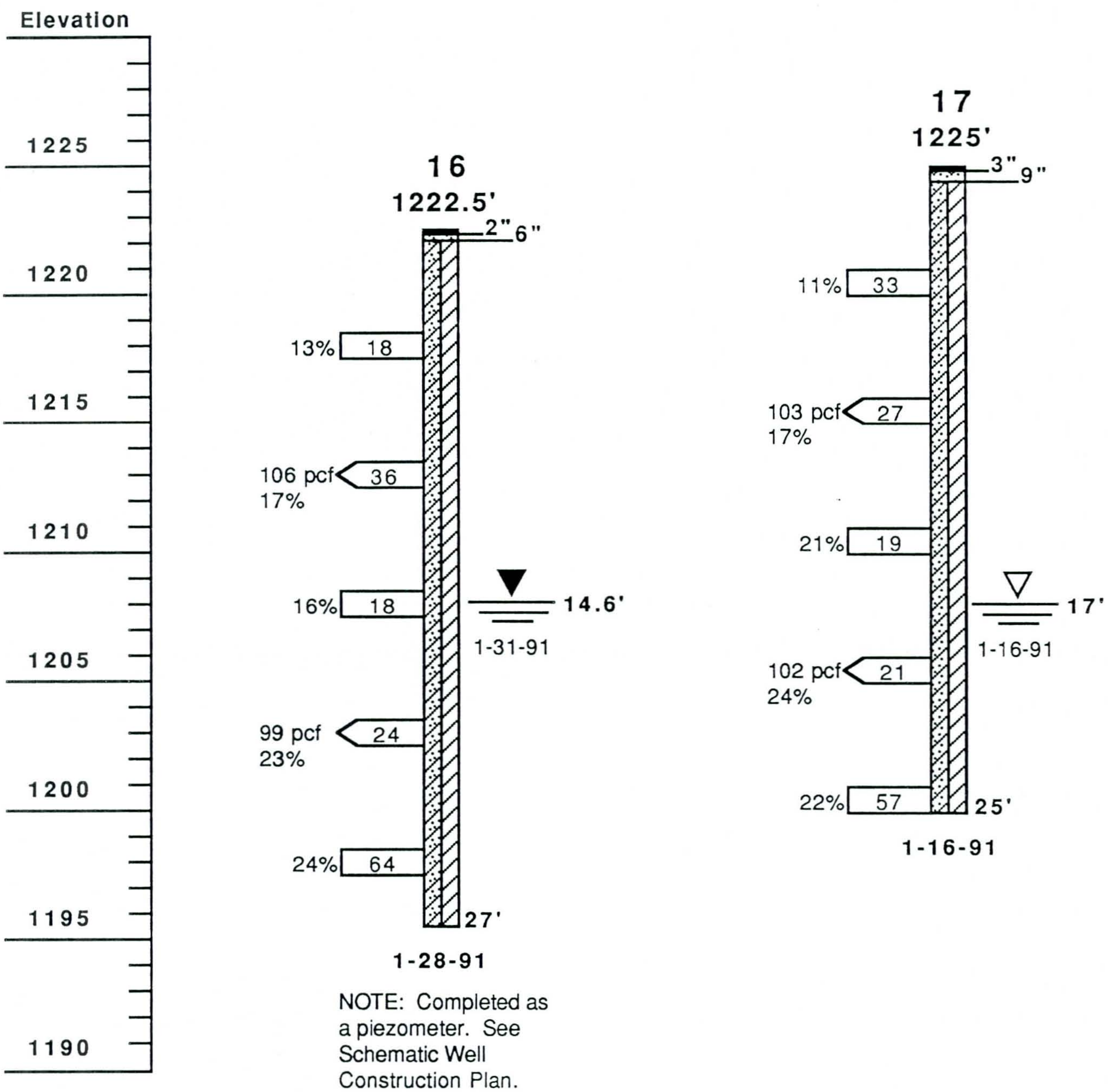
Free groundwater was encountered in the test borings during drilling as noted above.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



Free groundwater was encountered in the test borings during drilling as noted above.

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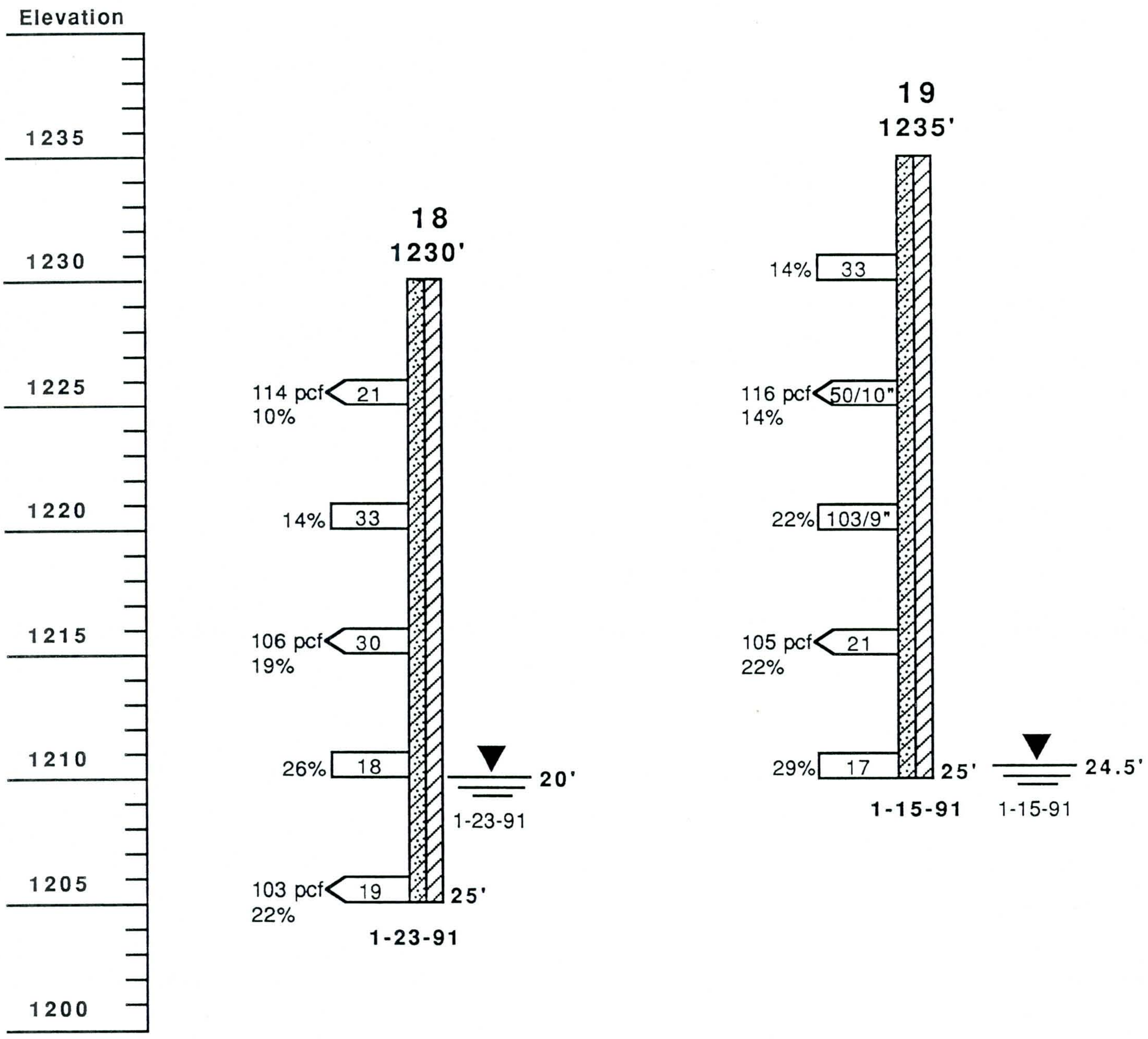
GRAPHICAL BORING LOGS



Free groundwater was encountered in the test borings during drilling as noted above.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

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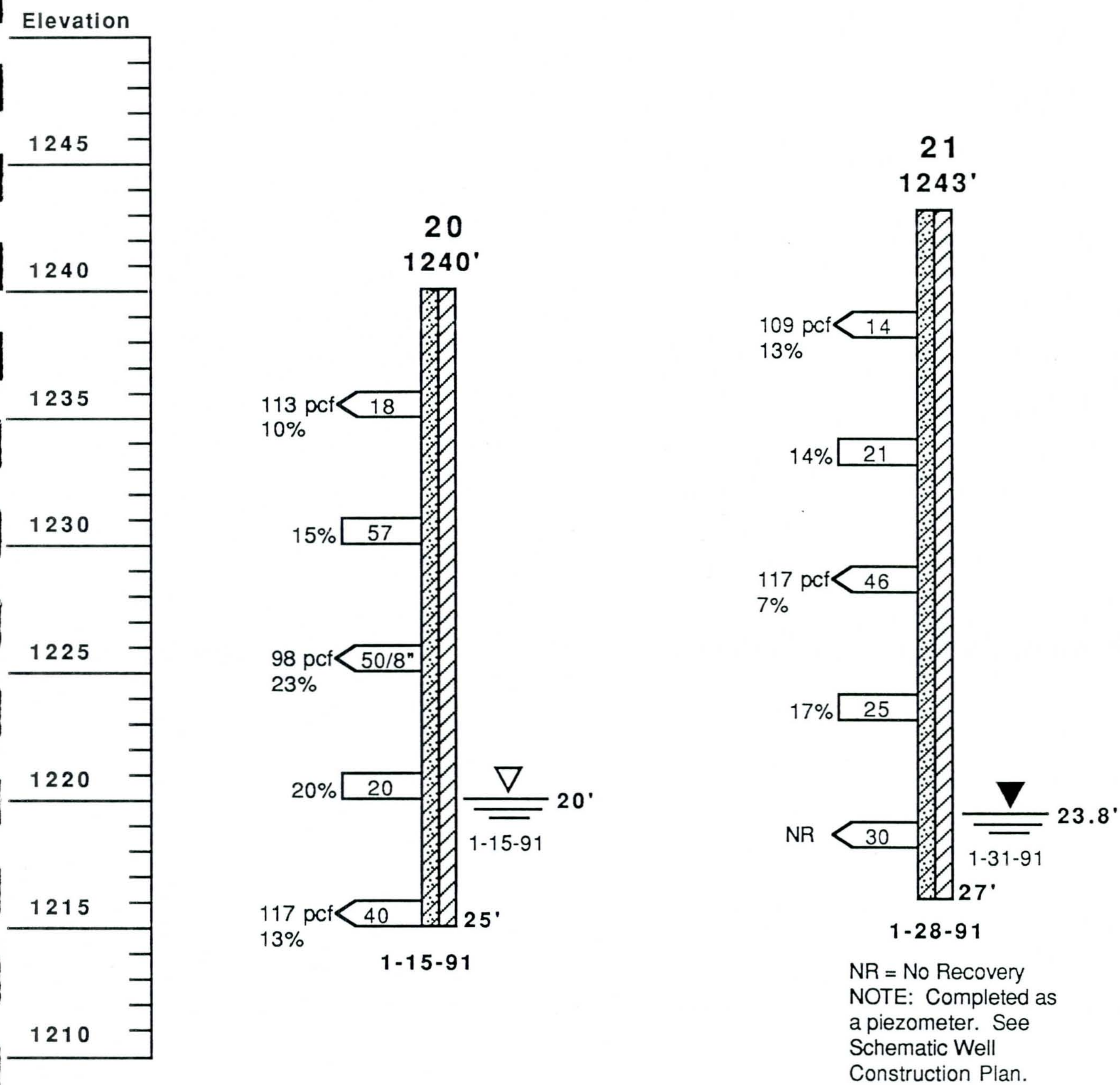
GRAPHICAL BORING LOGS



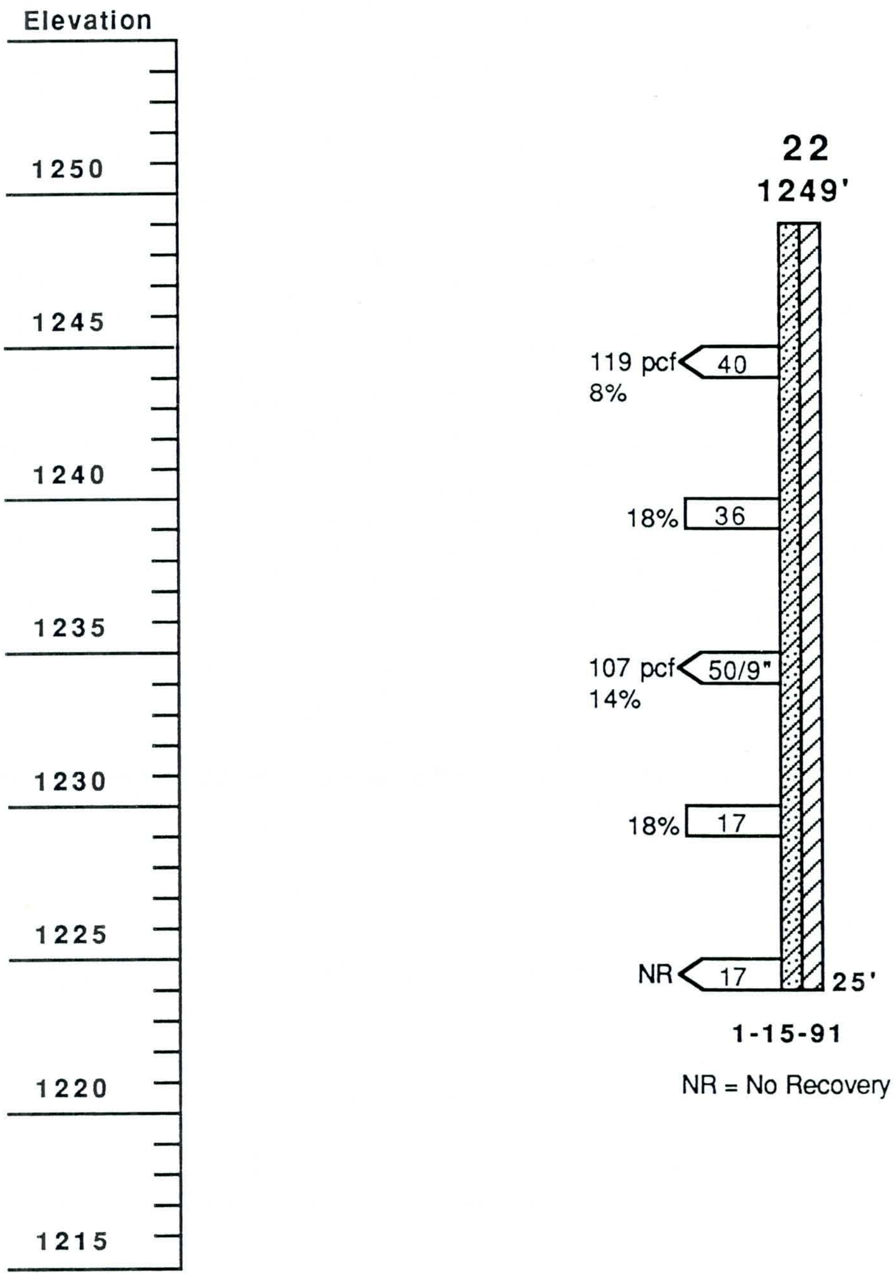
Free groundwater was encountered in the test borings during drilling as noted above.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS

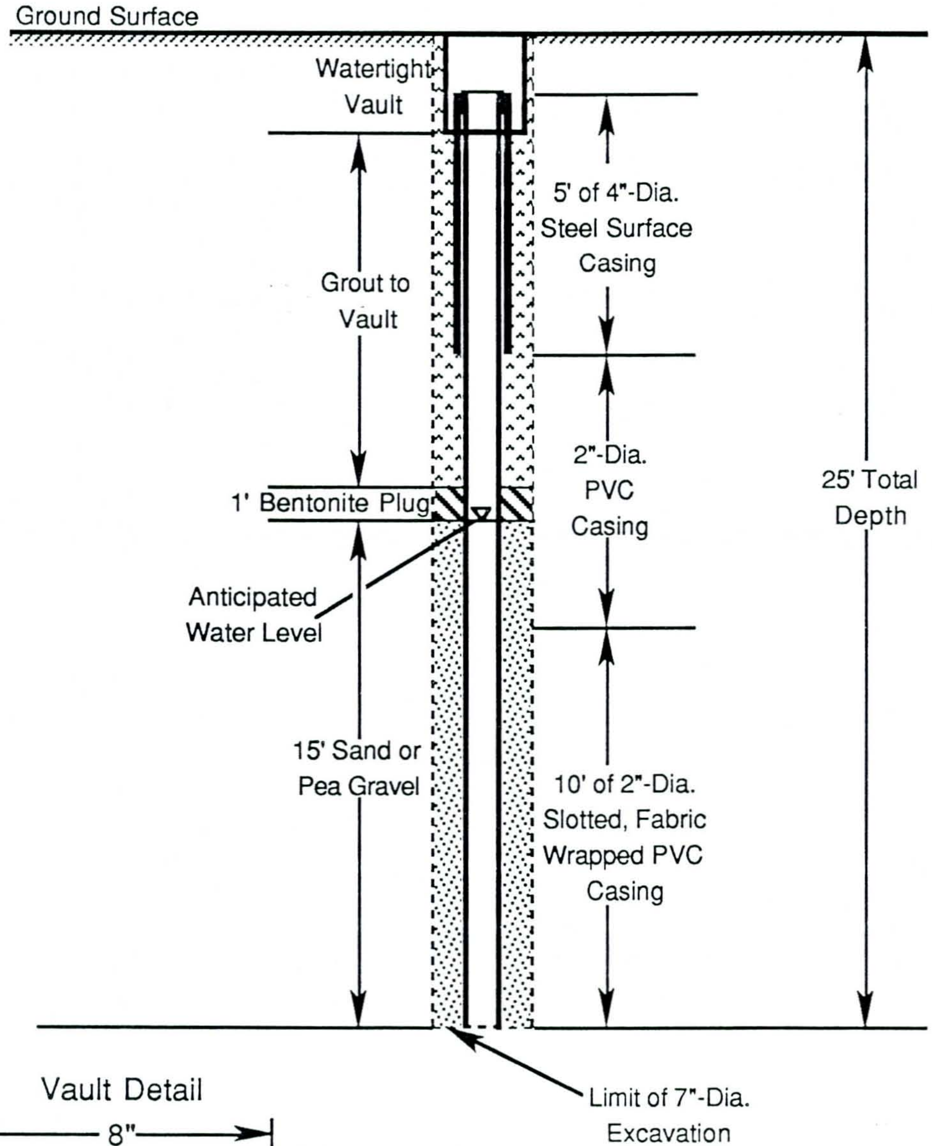


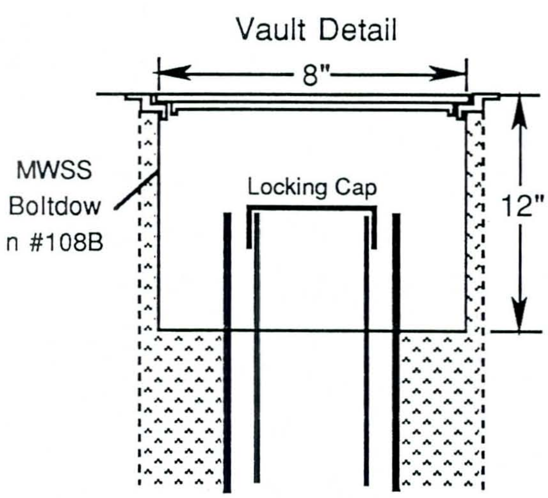
No free groundwater was encountered in the test borings during drilling unless otherwise noted.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

Project No. 90-0863
Thomas-Hartig & Associates





Not to Scale

Schematic Well Construction Plan

PROJECT NO. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON FIELD RESISTIVITY TESTS

DESCRIPTION: Date: 12/30/90 and 1/2/91

Location: Noted Below

Material: Subsurface Soil

Performed By: TH/R. Thompson

TESTED: Field electrical resistivity using the 4-probe method.

RESULTS:

Test Boring	Resistivity (ohm-cm) 0 - 12 ft.	Resistivity (ohm-cm) 0 - 25 ft.
1	2940	2390
2	2320	3350
3	3060	3020
4	3290	2300
5	3080	2300
6	3700	2680
7	5790	3500
8	4760	3690
9	5520	5170
10	7010	4400
11	5330	3540
12	2550	2820
13	2870	2730
14	2670	2300
15	1910	2630
16	2990	2780
17	2830	9580
18	2480	2110
19	3260	2630
20	3680	3160
21	3150	3590
22	3150	3250

APPENDIX B
LABORATORY RESULTS

REPORT ON LABORATORY TESTS

SAMPLE: Date: 1/28/91

Source: Noted Below

Type: Bulk Samples

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: Gradation and Plasticity Index

RESULTS

			Sieve Size - Accumulative % Passing											*
Sample	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.
1; 0' - 8'	38	14	36	44	49	53	59	68	79	95	95	100		SC
2; 4' - 9'	46	21	49	57	62	66	71	80	90	100				SC/CL
3; 14' - 15'	56	21	42	52	58	64	71	81	90	100				SM
4; 10' - 17'	59	31	38	45	49	53	58	68	80	100				SC
5; 17' - 24'	58	32	33	40	45	49	61	64	78	100				SC
6; 16' - 24'	54	29	47	53	57	61	66	75	87	100				SC/CH
7; 8' - 16'	61	33	48	55	60	64	70	78	90	100				SC/CH
8; 17' - 24'	56	29	48	56	62	68	75	86	95	100				SC/CH
9; 0' - 8'	39	19	33	41	46	51	59	73	87	100				SC
10; 7' - 14'	61	35	38	44	48	53	59	69	84	100				SC
11; 7' - 17'	51	28	28	33	37	42	49	63	82	100				SC
12; 8' - 14'	67	40	49	56	61	65	71	81	92	100				SC/CH
13; 9' - 16'	54	28	52	60	66	71	78	87	96	100				CH/SC
14; 0' - 9'	39	20	34	41	46	51	59	71	87	100				SC
15; 18' - 24'	56	31	58	67	72	76	80	87	95	100				CH
16; 0' - 8'	33	12	38	47	51	55	61	71	84	97	100			SC
17; 7' - 14'	58	28	34	39	44	49	59	75	92	100				SC

*Unified Soil Classification

REPORT ON LABORATORY TESTS

SAMPLE: Date: 1/28/91

Source: Noted Below

Type: Bulk Samples

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: Gradation and Plasticity Index

RESULTS

			Sieve Size -					Accumulative % Passing						*
Sample	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.
18; 8' - 16'	51	25	57	65	69	72	78	86	96	100				CH
19; 16' - 24'	57	28	52	59	64	68	75	84	92	100				CH/SC
20; 0' - 8'	42	19	41	49	53	57	65	77	92	100				SC
21; 8' - 16'	62	33	46	52	56	60	67	77	89	100				CH
22; 0' - 8'	52	25	31	36	39	42	49	64	83	100				SC

*Unified Soil Classification

Project No. 90-0863

Thomas-Hartig & Associates, Inc.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/23/91

Source ·Test Boring 5; 19 - 20'

Type Driven ring sample; 119 pcf dry density; 13% field moisture

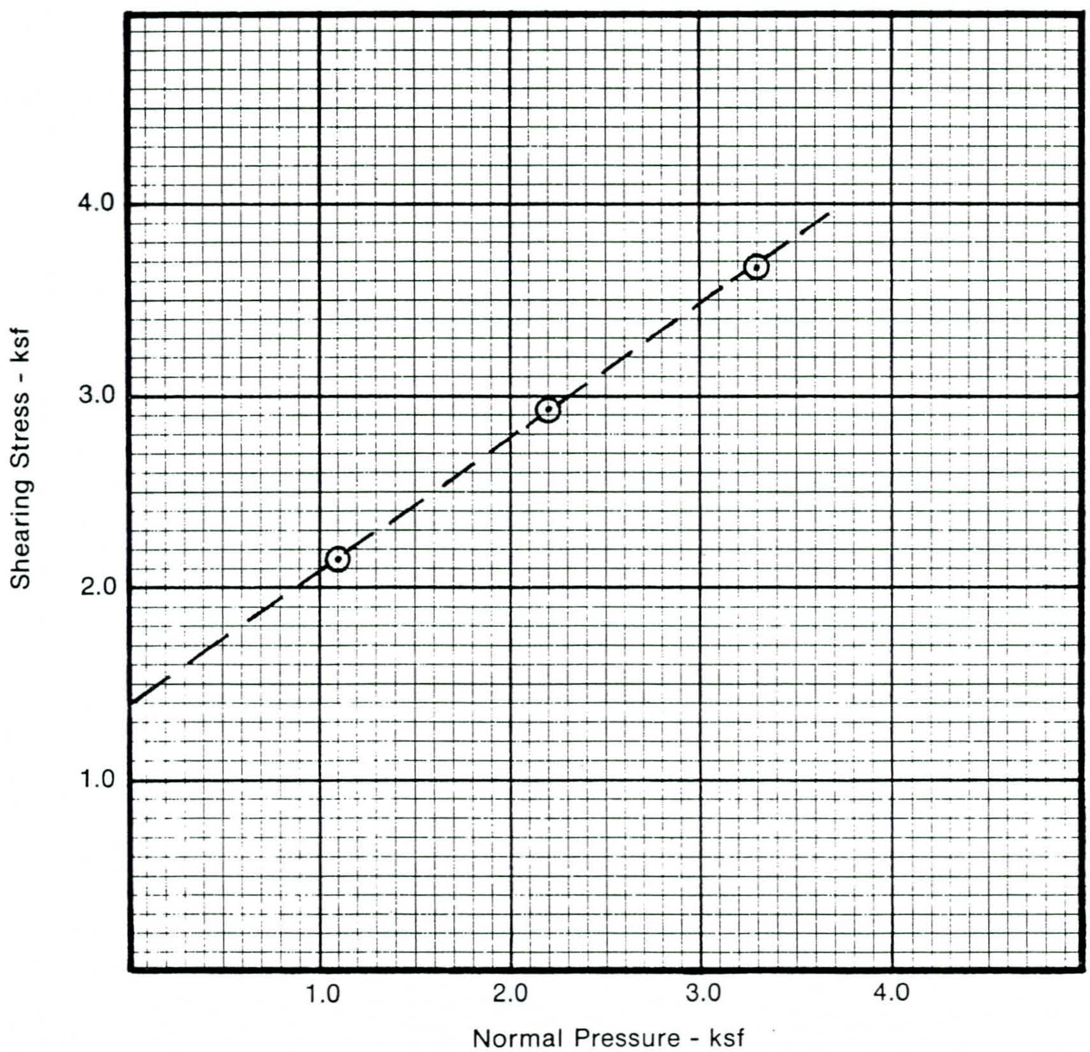
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = 35° Cohesion (c) = 1.4 ksf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/23/91

Source Test Boring 7; 9 - 10'

Type Driven ring sample; 106 pcf dry density; 17% field moisture

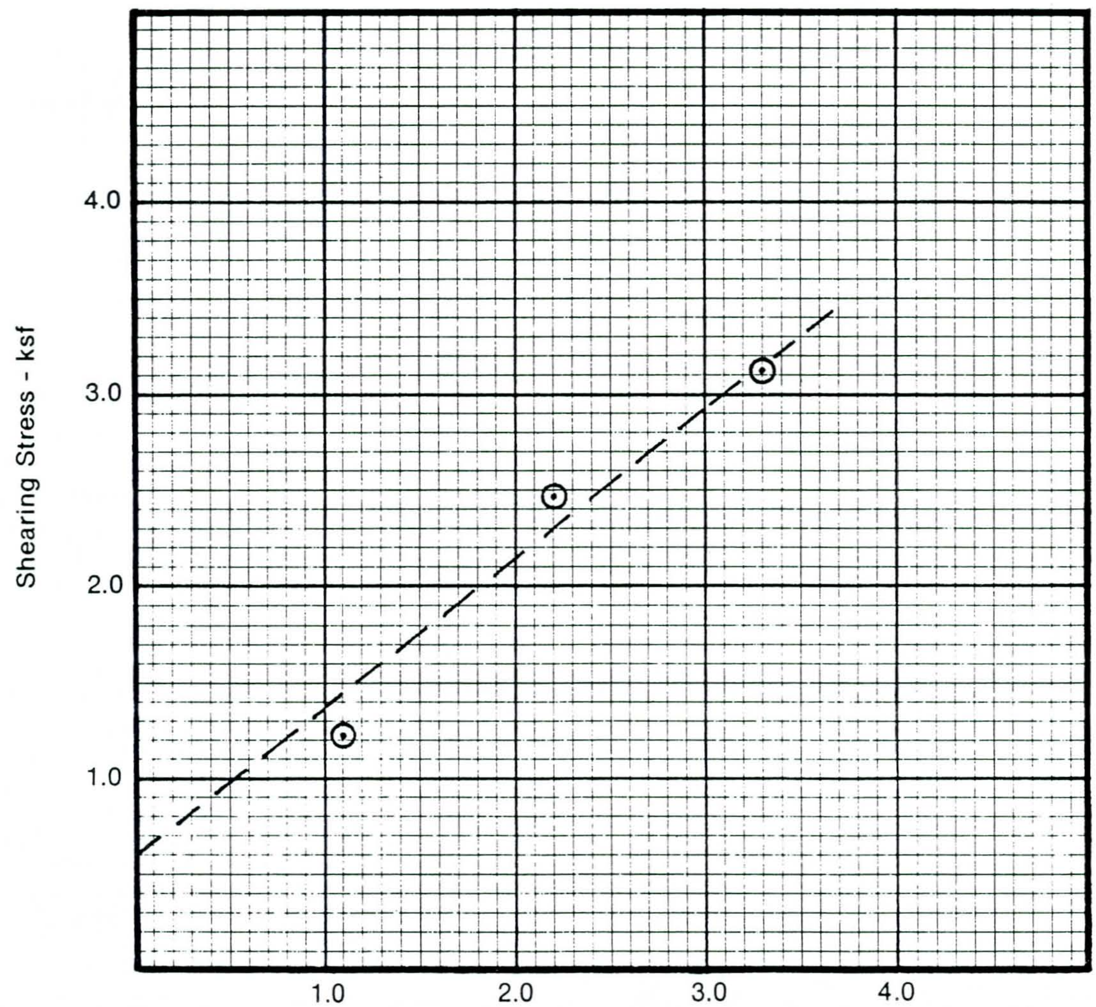
Material Clayey Sand/ Sandy Clay (SC/CH)

Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = 38° Cohesion (c) = 0.6 ksf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 12; 4 - 5'

Type Driven ring sample; 113 pcf dry density; 13% field moisture

Material Clayey Sand (SC)

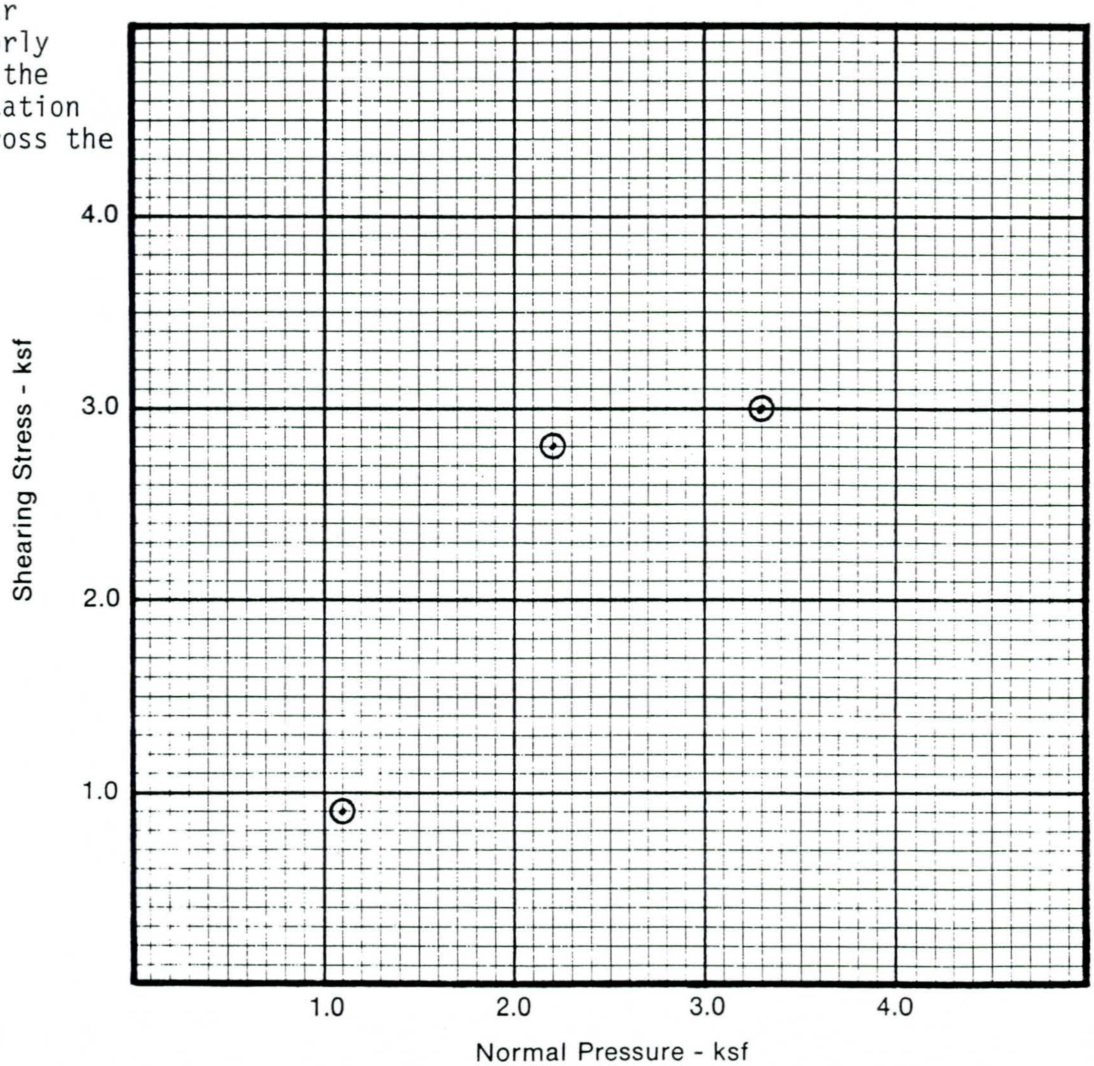
Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = Cohesion (c) =

NOTE: The shear envelope is poorly defined due to the variable cementation and nodules across the failure plane.



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 15; 9 - 10'

Type Driven ring sample; 110 pcf dry density; 18% field moisture

Material Clayey Sand/ Sandy Clay (SC/CH)

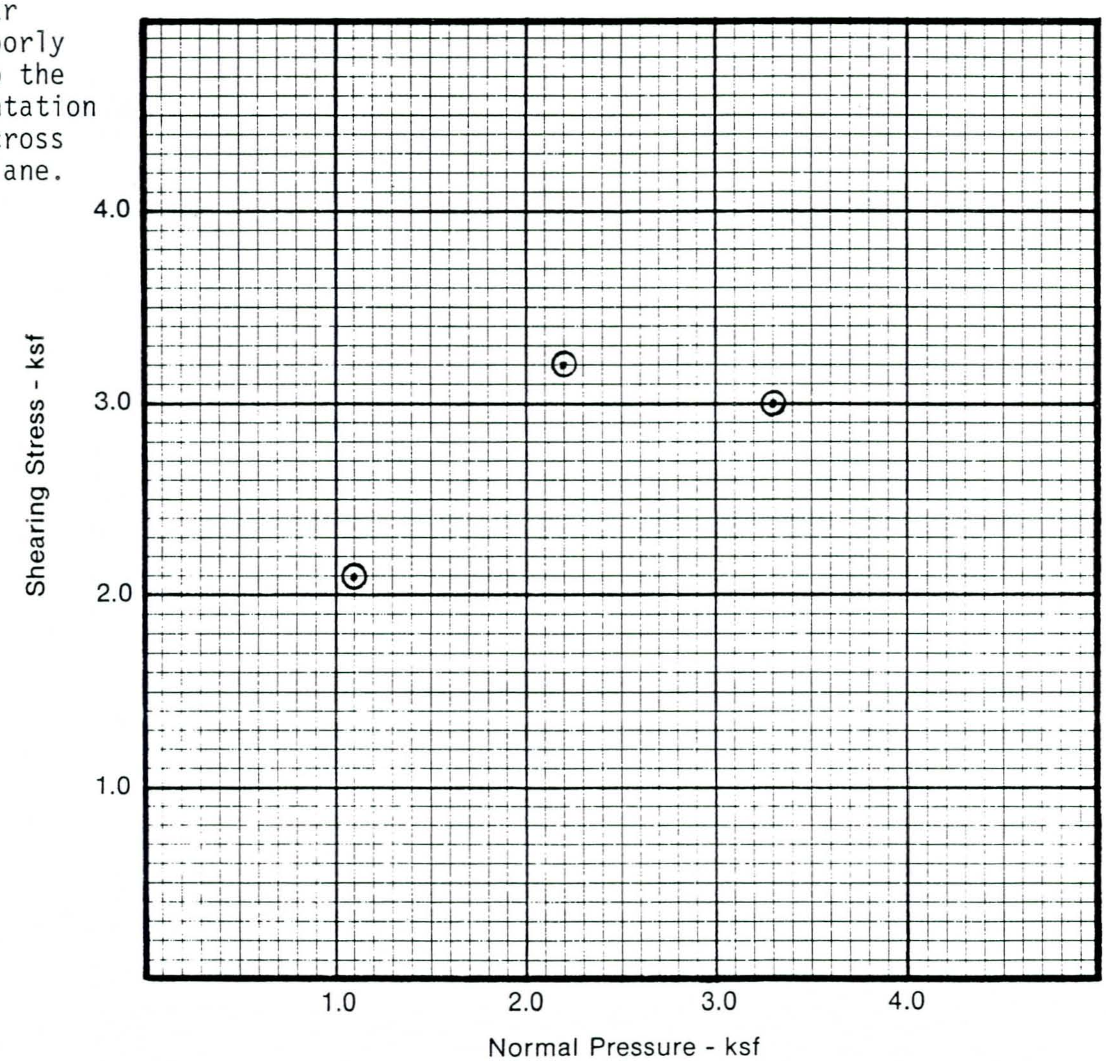
Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = Cohesion (c) =

NOTE: The shear envelope is poorly defined due to the variable cementation and nodules across the failure plane.



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/91

Source Test Boring 20; 24 - 25'

Type Driven ring sample; 117 pcf dry density; 13% field moisture

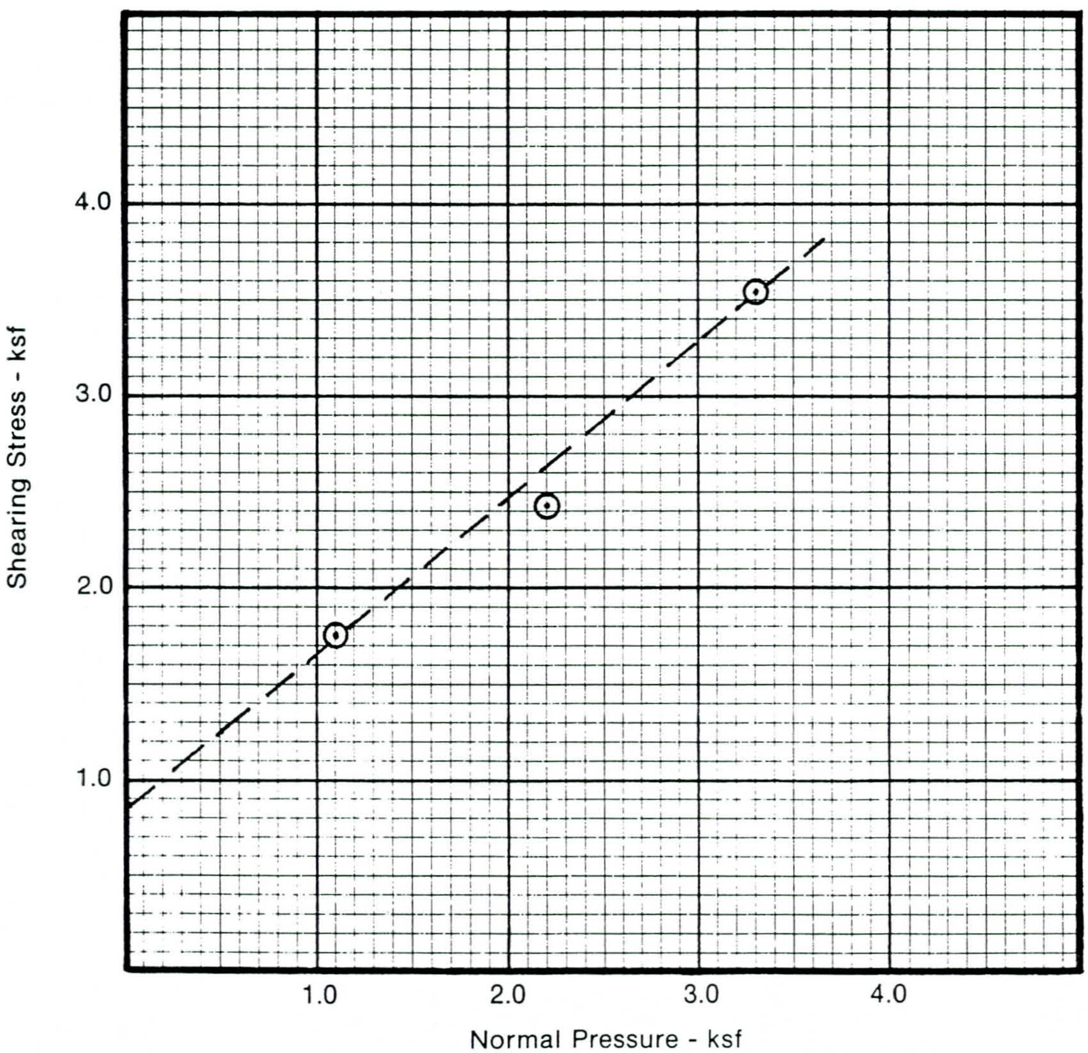
Material Sandy Clay (SC)

Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = 39° Cohesion (c) = 0.9 ksf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/91

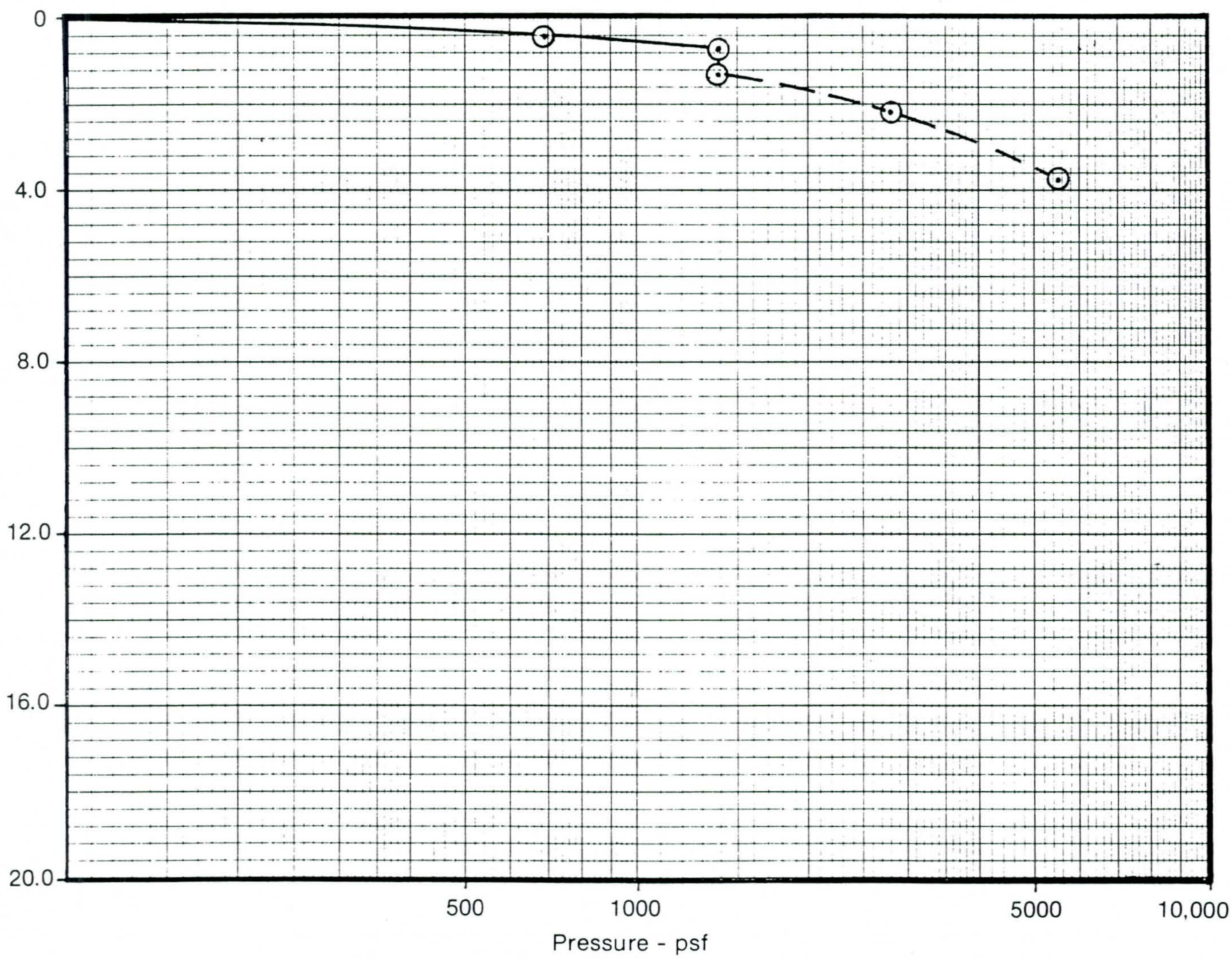
Source Test Boring 2; 4 - 5'

Type Driven ring sample; 115 pcf dry density; 9% field moisture

Material Clayey Sand/ Sandy Clay (SC/CL)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 1385 psf



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/91

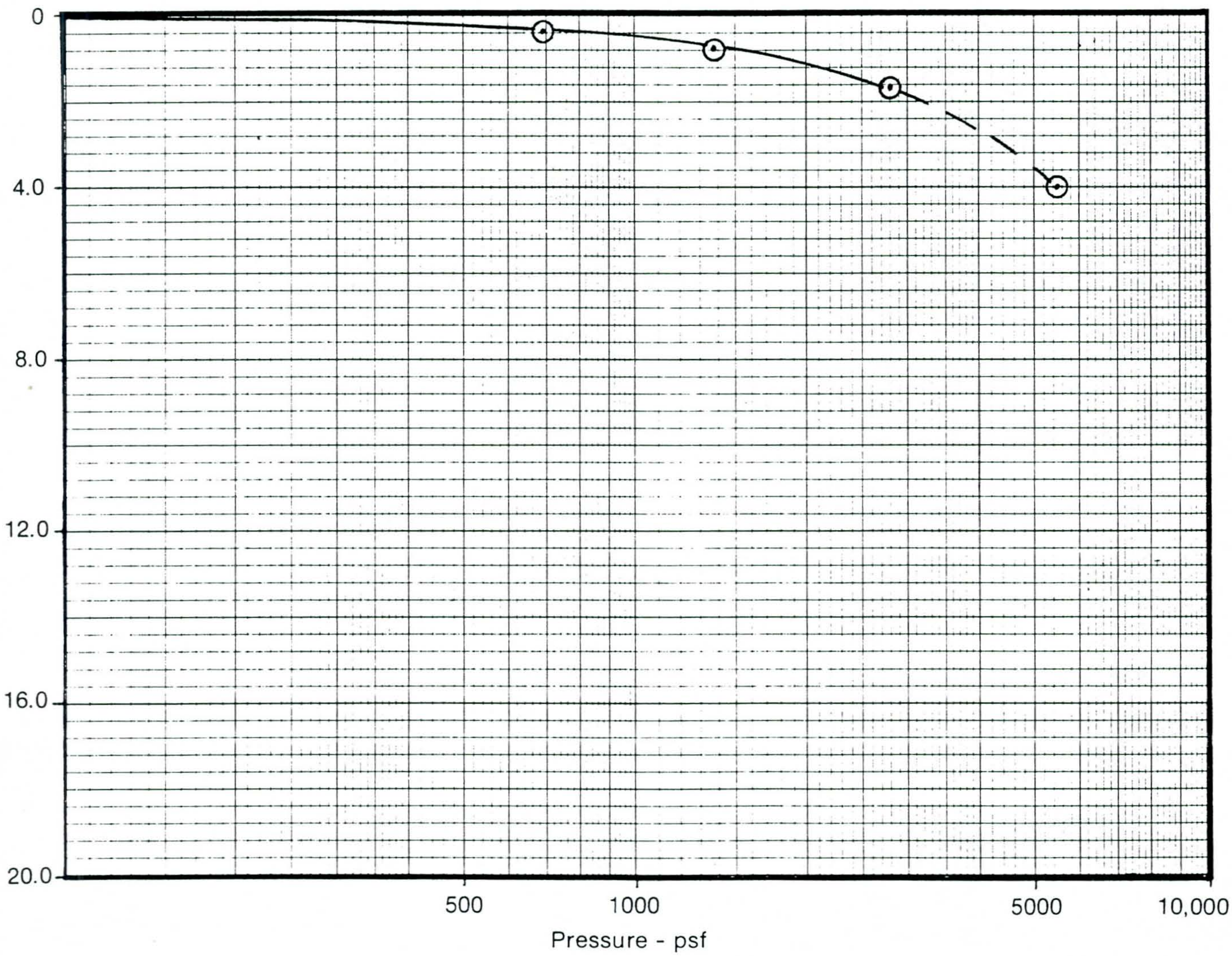
Source Test Boring 7; 9 - 10'

Type Driven ring sample; 106 pcf dry density; 17% field moisture

Material Clayey Sand/Sandy Clay (SC/CH)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/28/91

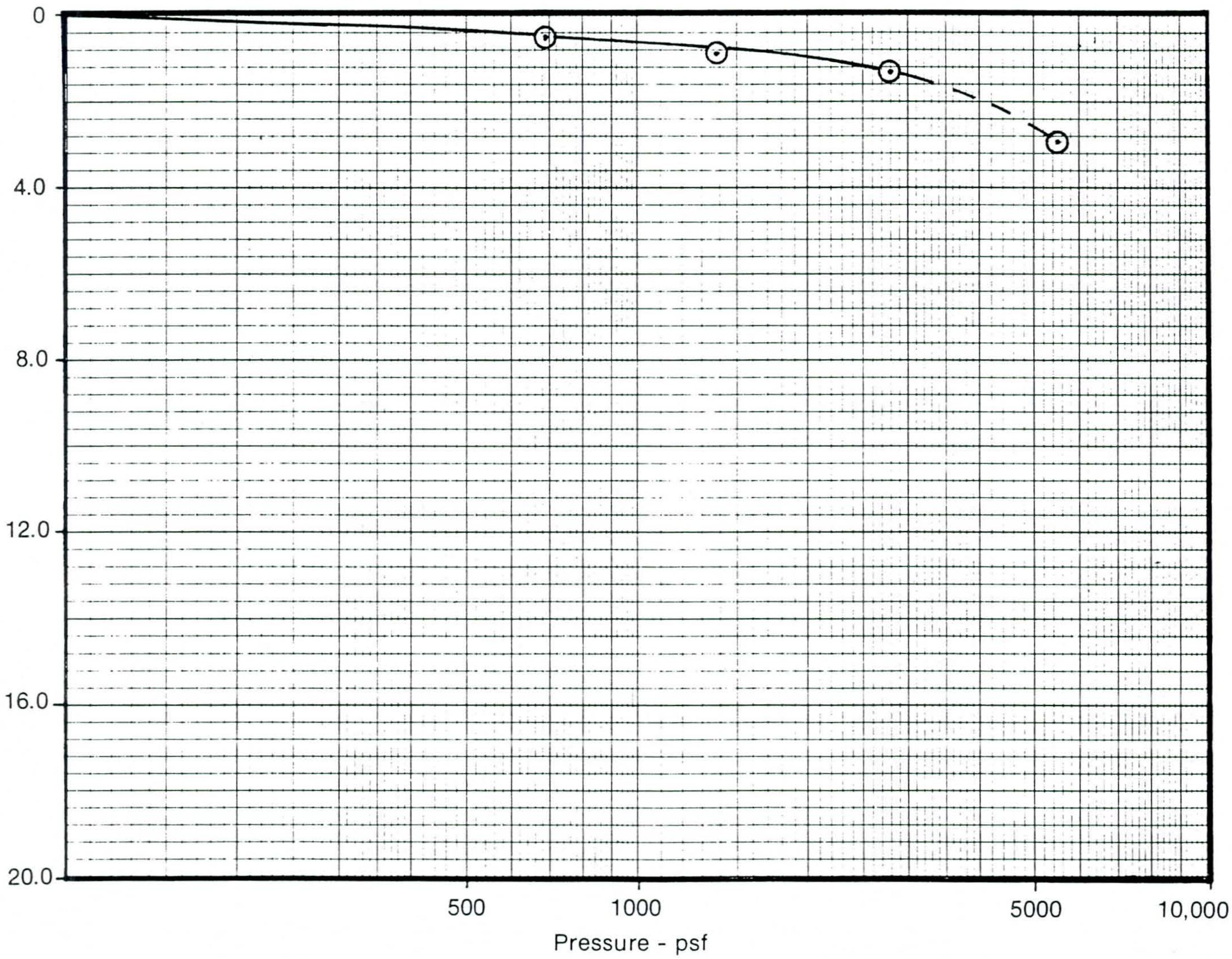
Source Test Boring 10; 14 - 15'

Type Driven ring sample; 103 pcf dry density; 23% field moisture

Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/28/91

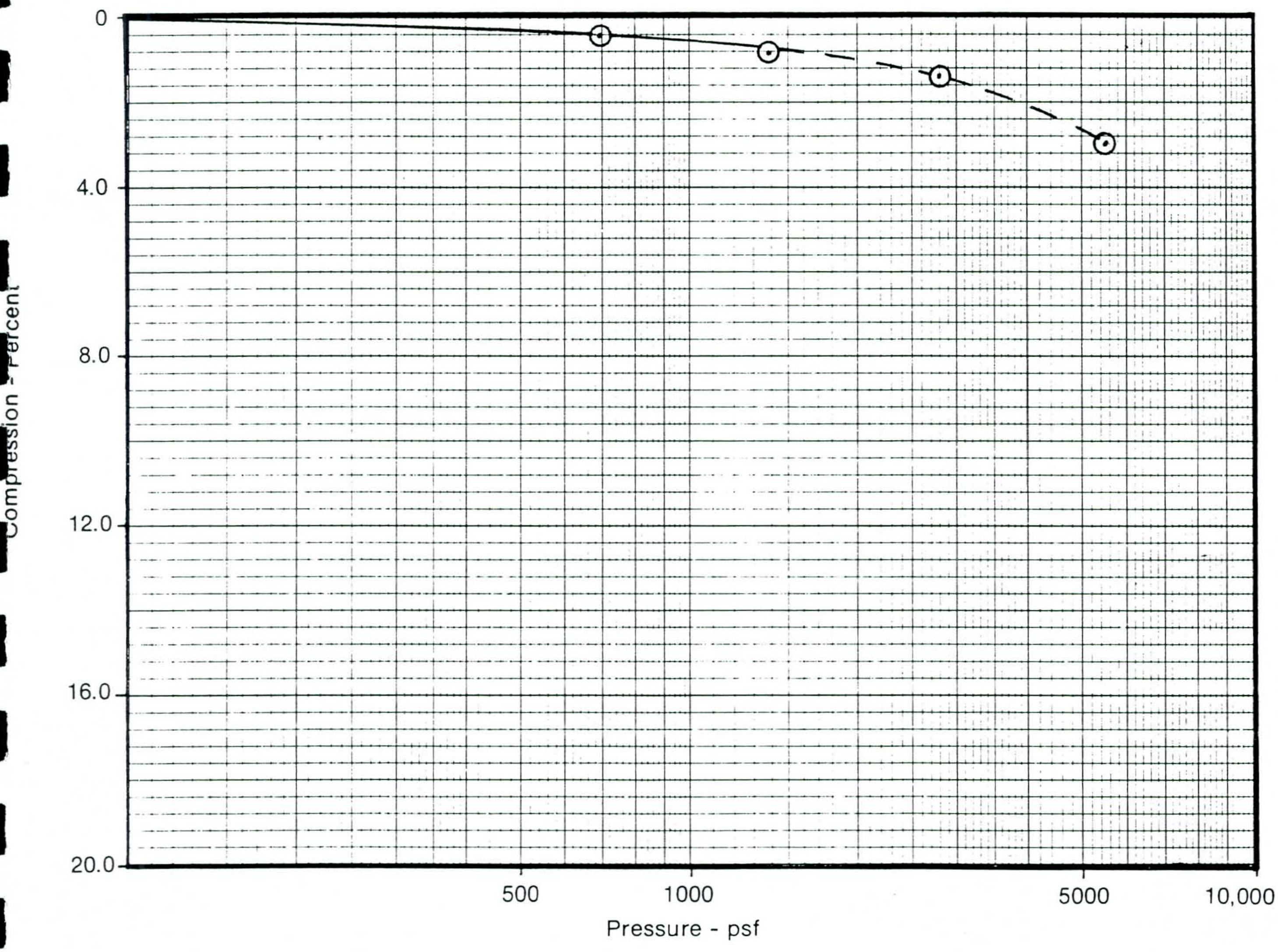
Source Test Boring 17; 9 - 10'

Type Driven ring sample; 103 pcf dry density; 17% field moisture

Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 1385 psf



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/28/91

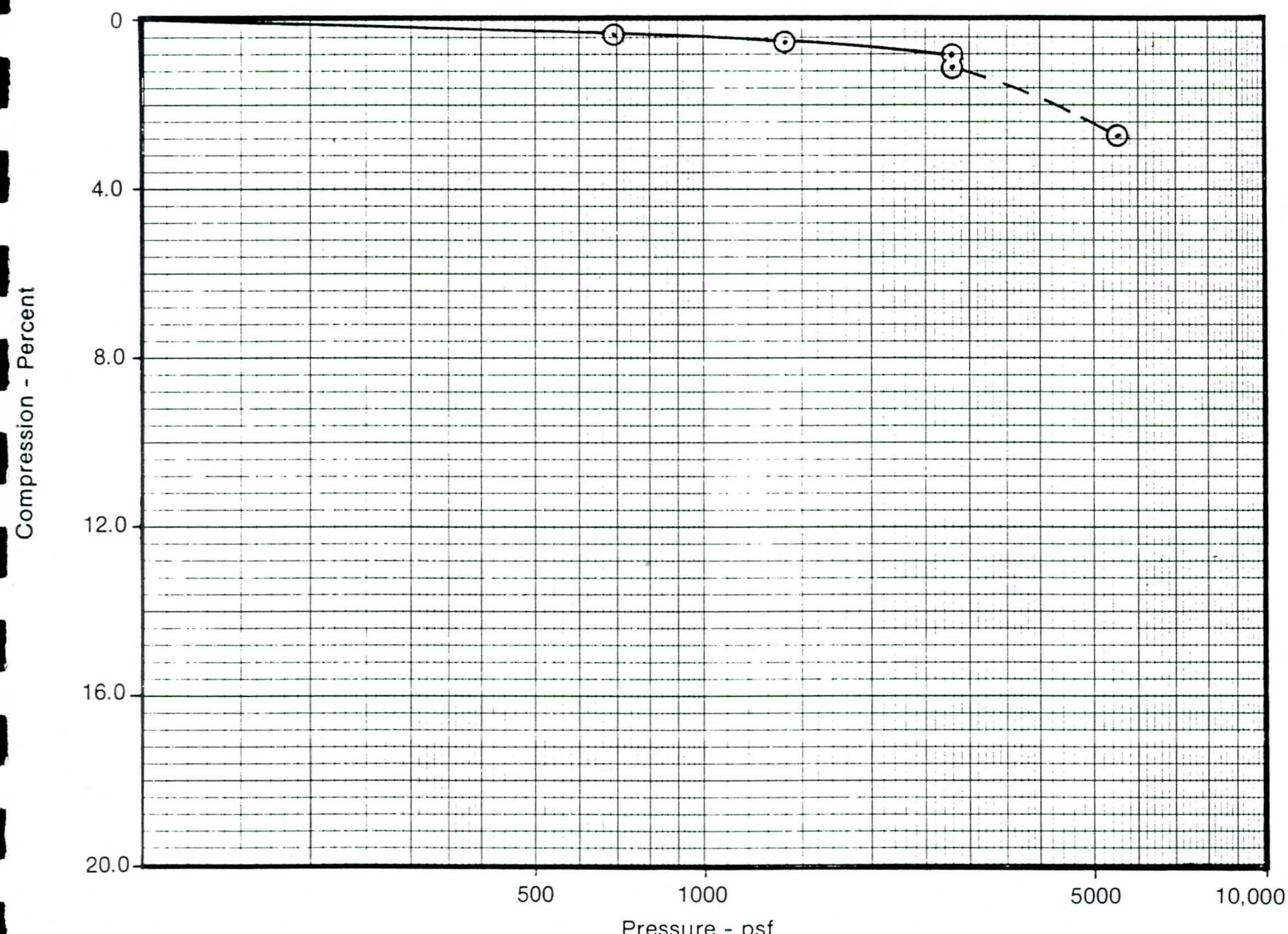
Source Test Boring 22; 4 - 5'

Type Driven ring sample; 119 pcf dry density; 8% field moisture

Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 90-0863

REPORT ON pH, SOLUBLE SALTS, SULFATES, & CHLORIDES

SAMPLE: Date: 1/21/91

Source: Noted Below

Type: Bulk Sample

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: pH, Soluble Salts, Sulfates, & Chlorides

TEST RESULTS

Sample	pH	Soluble Salts (%)	Sulfates Percent	Chlorides Percent
4; 0 - 3'	8.2	0.510	0.190	0.060
9; 8 - 16'	8.3	0.090	0.006	0.011
15; 8 -16'	8.5	0.180	0.027	0.030
19; 0 - 8'	8.0	0.084	0.012	0.009
22; 16 -24'	8.3	0.085	0.003	0.012

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 2; 0 - 4' & 4 - 9'

Type Bulk Sample

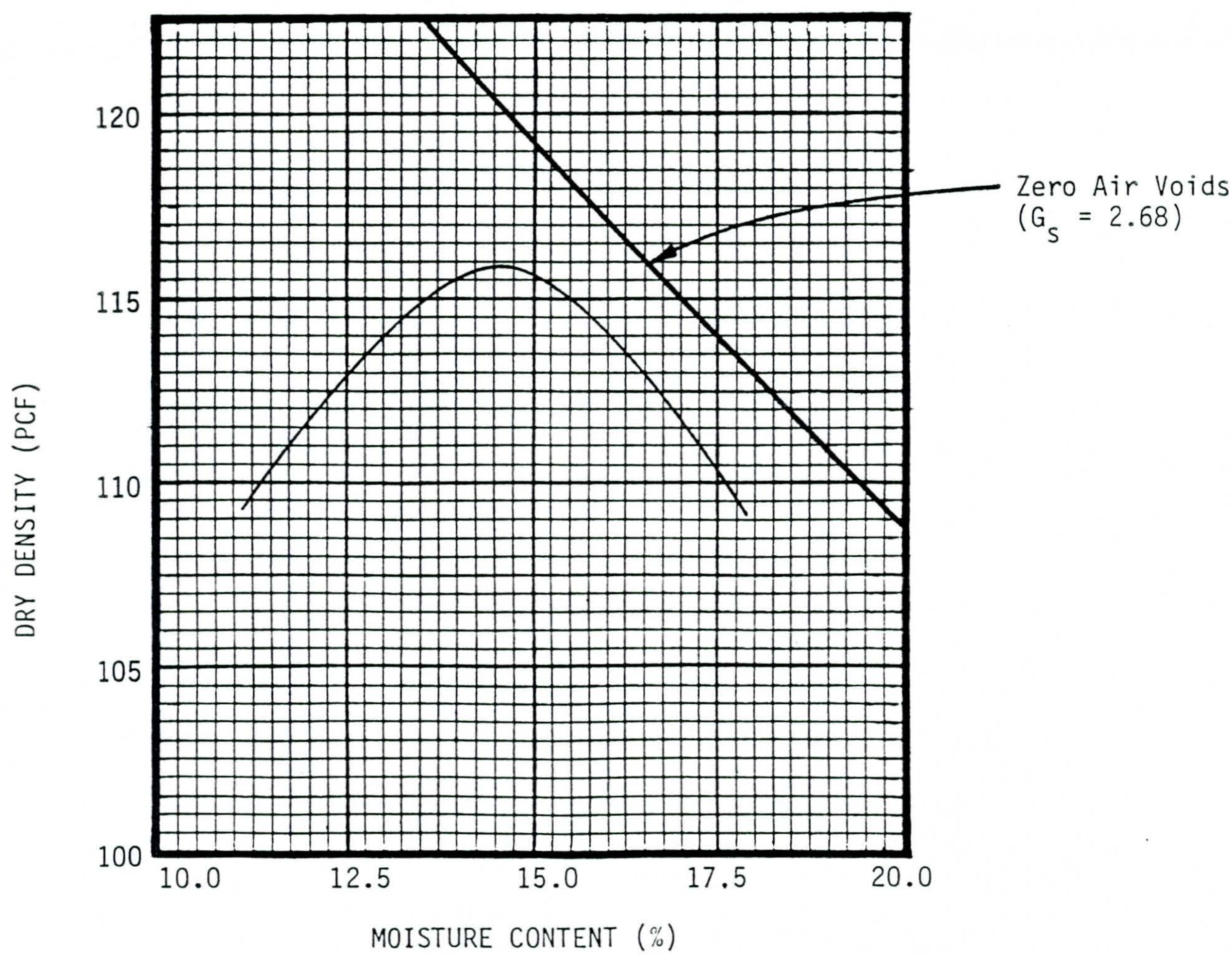
Material Clayey Sand/Sandy Clay (SC/CL)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 116 Optimum Moisture Content (%) 14.5



REPORT ON LABORATORY TESTS

SAMPLE: Date 1/23/91

Source Test Boring 15; 0 - 8'

Type Bulk Sample

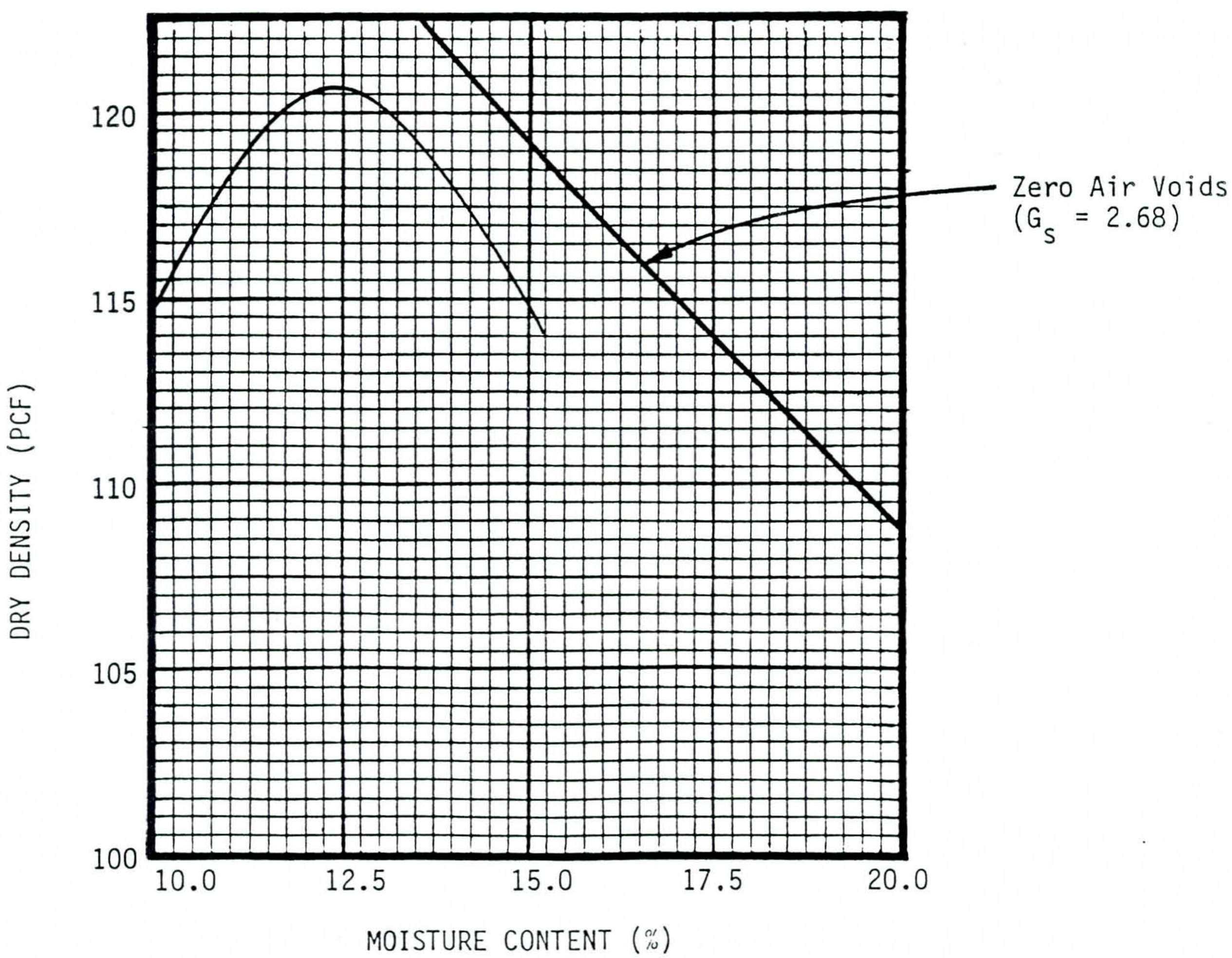
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 121 Optimum Moisture Content (%) 12.5



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/22/91

Source Test Boring 7; 0 - 8'

Type Bulk Sample

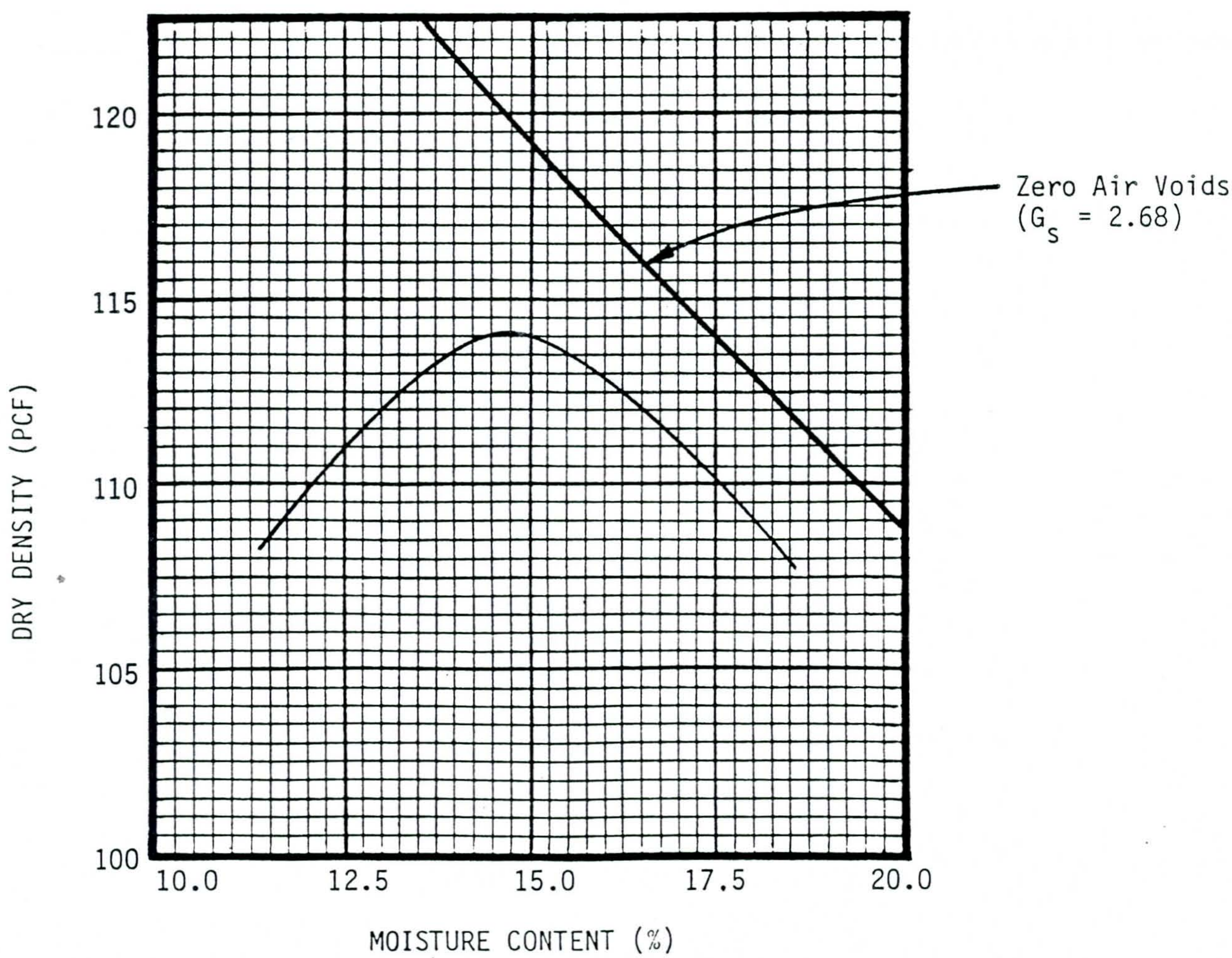
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 114 Optimum Moisture Content (%) 14.5



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 14; 0 - 9' & 9 - 14'

Type Bulk Sample

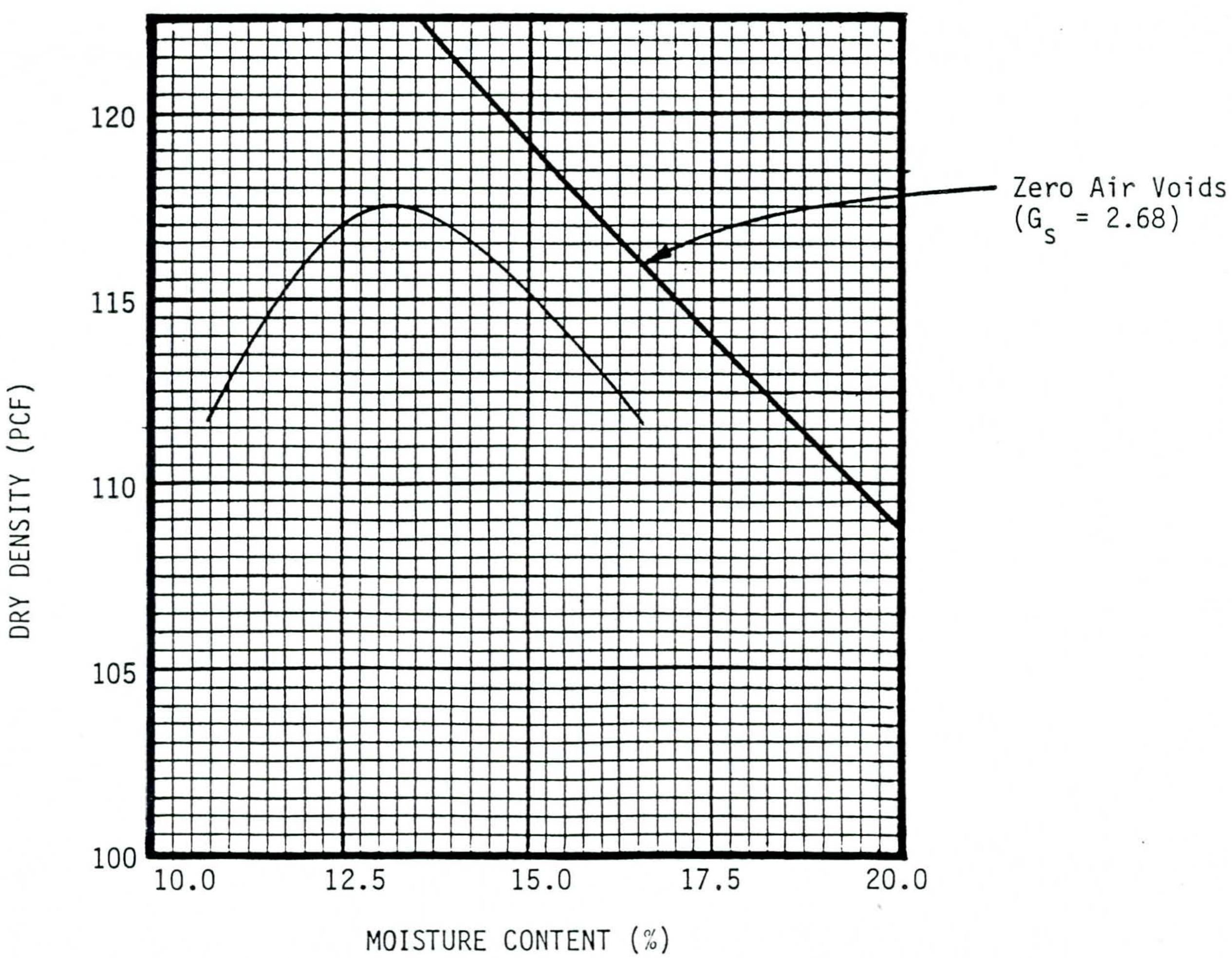
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 117.5 Optimum Moisture Content (%) 13



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 19; 0 - 8' & 8 - 16'

Type Bulk Sample

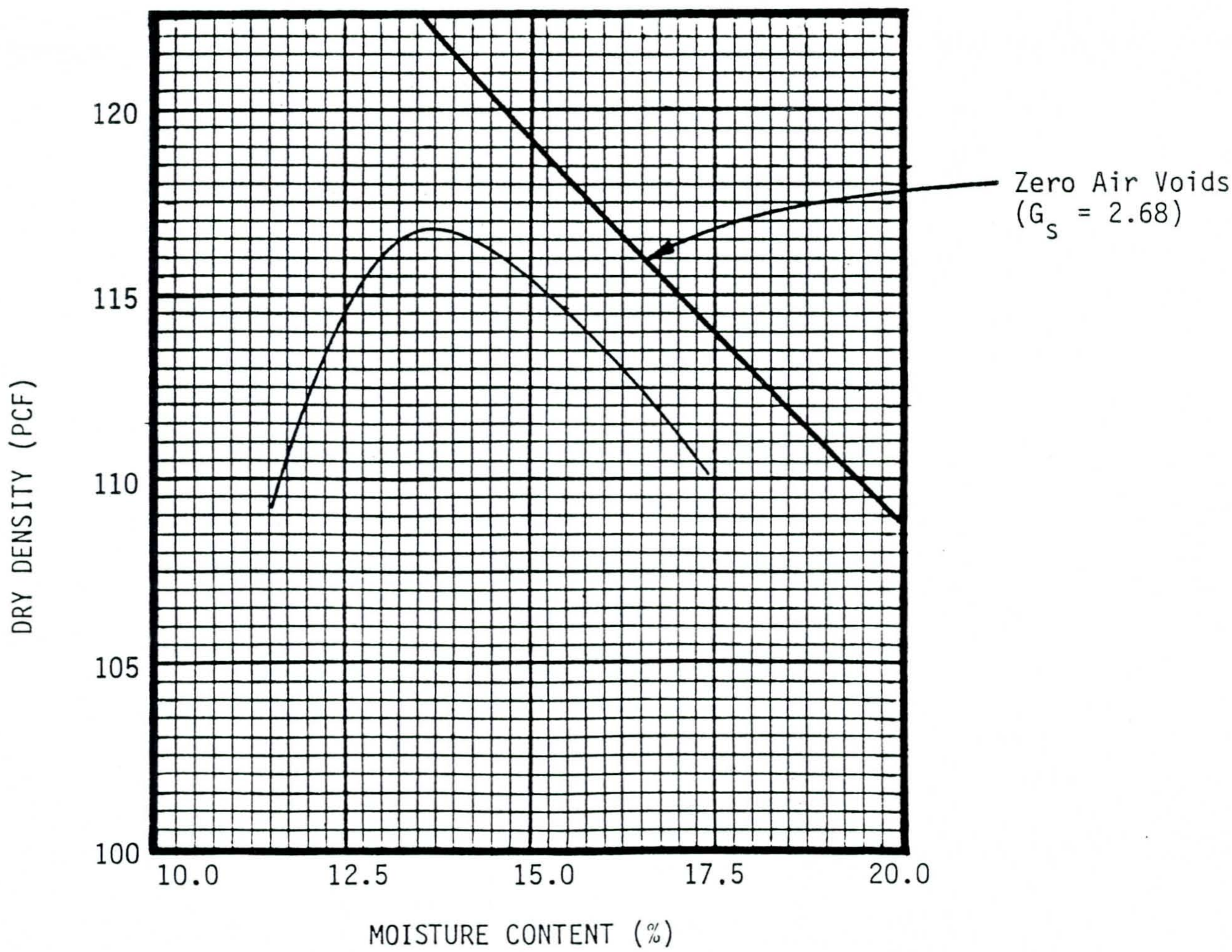
Material Sandy Clay/ Clayey Sand (CH/SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 117 Optimum Moisture Content (%) 14



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date: 1-22-91

Source: Noted Below

Type: Grab Samples

Material: Surface Soil

Sampled By: TH/Thompson

TESTED: R-Value

TEST RESULTS

Location	**R-Value
10; 0 - 7' & 7 - 14' *	34
12; 0 - 5' & 5 - 8' *	23
20; 0 - 8' & 8 - 16' *	35
22; 0 - 8' & 8 - 16' *	39

*Composite Samples
**Corrected to 300psi exudation pressure

Project No. 90-0863

Thomas-Hartig & Associates, Inc.

REPORT ON REMOLDED EXPANSION TEST

SAMPLE: Date: 2/4/91

Source: Noted Below

Type: Bag Samples (*) and Driven Ring Samples

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: Percent expansion upon soaking of remolded sample compacted to approximately 95% of the maximum ASTM D698 dry density at approximately 2% less than optimum moisture content..

TEST RESULTS

Sample	Dry Density (pcf)	Initial Moisture (Percent)	Surcharge Pressure (psf)	Expansion Upon Soaking (Percent)
8; 4' - 5'	108	12.5	100	0.30
13; 0' - 8'*	112	11.0	100	1.40

Project No. 90-0863

REPORT ON UNDISTURBED EXPANSION TEST

SAMPLE:

Date: 2/4/91

Source: Noted Below
Type: Driven Ring Samples
Material: Subsurface Soil
Sampled By: TH/Thompson

TESTED: Percent expansion upon soaking of undisturbed sample.

TEST RESULTS

Sample	Dry Density (pcf)	Initial Moisture (Percent)	Surcharge Pressure (psf)	Expansion Upon Soaking (Percent)
16; 9' - 10'	106	17	100	0.62