PRELIMINARY DESIGN REPORT

For The

HAYDEN ROAD BRIDGE OVER THE SALT RIVER

Prepared For The

MARICOPA COUNTY HIGHWAY DEPARTMENT

MARICOPA COUNTY
1871
ARIZONA

PROJECT No. 68109

MAY 1981

Boyle Engineering Corporation
consulting engineers / architects

Registered Professional Engineer (Civil)
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ARIZONA, U.S.A.

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May 11, 1981

Maricopa County Highway Department
3325 West Durango Street
Phoenix, Arizona 85009

Preliminary Design Report
Hayden Road Bridge
Over the Salt River

Transmitted herewith is a copy of our report entitled "Preliminary Design Report, Hayden Road Bridge over the Salt River", dated May, 1981. This report has been prepared as a part of the County's engagement of Boyle Engineering Corporation, on December 18, 1980, to provide professional engineering services in connection with the Hayden Road Bridge Design.

This report has been prepared by the undersigned who are responsible for the conclusions and opinions expressed therein. It is suggested that the County review this report at their earliest convenience so that we may schedule a meeting to review its contents and establish final design criteria.

BOYLE ENGINEERING CORPORATION

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PHOENIX, ARIZONA

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I. PROJECT DESCRIPTION

This Maricopa County project encompasses the design of a new multispan vehicular bridge with approach roadways and pertinent related channelization work of the Salt River. The essence of the total project is to upgrade the allowable flow capacity of the Salt River through the county-owned bridge structure at Hayden Road to 200,000 cubic feet per second (cfs) and at the same time maintain vehicular traffic.

The Hayden Road bridge crossing of the Salt River is located in the eastern boundaries of metropolitan Phoenix and is one of the major thoroughfares connecting Tempe and Scottsdale, Arizona.

The bridge replacement and channelization upgrading program has resulted from an increase in predicted flow of the Salt River from approximately 20,000 cfs set as previous design criteria to 200,000 cfs as now predicted by engineering studies and the necessity for a structure capable of providing the transport of commerce over the Salt River during flood conditions similar to those witnessed during 1978, 1979, and 1980.

Tasks undertaken during this portion of the total design project have included an indepth channel flow study assisted by computer modeling, an economic study of alternate bridge types, and a geotechnical investigation at the project site.

II. HYDRAULIC AND SCOUR ANALYSIS

As evidenced by the damage sustained by the Hayden Road Bridge during the flows of February, 1980, the hydraulic characteristics of the Salt River can and will continue to detrimentally influence structure stability. Boyle Engineering Corporation, in conjunction with the geotechnical consultant, Dames & Moore, has undertaken the task of attempting to predict the severity of these influencing characteristics over the design life of the proposed replacement structure by using a computer modeling program HEC-2.

In evaluating the approach to this task, it was determined that a section of the Salt River from Alma School Road to Mill Avenue be analyzed as that portion of the river which would directly affect the integrity of the proposed structure at Hayden Road. Alma School Road is 3 miles east and upstream from Hayden Road; Mill Avenue is located 2 miles west and downstream from Hayden Road.

Ideally, a much larger reach of the Salt River encompassing all of the constrictions, changes in direction, slopes, tributaries, etc., should be considered in such an analysis. However, due to economically feasible limits of consideration, the 5-mile reach outlined was determined to be the most practical alternative.

Cross sections of the existing topography covering the studied area were developed by aerial photography at approximately 500-foot intervals. Stereo plotting techniques were then employed to convert these cross sections to a computerized format compatible with the U.S. Army Corps of Engineers HEC-2 computer program. This information was then used to model the flow in the Salt River throughout the study reach.

Criteria used in the hydraulic and scour analysis are as follows:

1. Design flow = 200,000 cfs

   This design flow was provided by Maricopa County and based in part on actual measured flows in the Salt River during February, 1980, of approximately 186,000 cfs.

2. Desired maximum velocity = 10 fps

3. Channelization recommendations and approach design should preclude over bank flooding thus providing continuous use of Hayden Road over the Salt River in the event of a 200,000-cfs flow.

The results of the hydraulic and scour analysis are contained in Appendix I of this report.

III. GEOTECHNICAL INVESTIGATION

The consulting firm of Dames & Moore was also engaged by Boyle Engineering Corporation to perform a detailed soils and geotechnical investigation of the project site.

Preliminary channelization and structural design information provided by Boyle Engineering Corporation to be considered in the report were:

1. Preliminary channelization data.
2. Preliminary structural loadings.
3. Anticipated piles section alternates.
4. Caisson-type pier sizes and locations.

The final geotechnical report establishes basic substructure design criteria for the bridge, channelization requirements, and pertinent points of concern derived from the findings relating to probable mining activities in the main Salt River channel immediately downstream of the proposed bridge.

Appendix II of this report is the comprehensive report of the geotechnical investigation prepared by the firm of Dames & Moore which outlines the subsurface characteristics which will influence the proposed replacement structure. Topics addressed in this report are as follows:

1. Site surface and subsurface conditions
2. Seismicity
3. Scour - general and local

4. Possible impacts of sand and gravel mining
5. Foundation types
6. Foundation settlement
7. Liquefaction
8. Cement
9. Earthwork
10. Lateral earth pressures

The investigation methods used and the criteria upon which design recommendations are based are discussed in this geotechnical report.

IV. TYPE SELECTION

The Hayden Road bridge type selection study analyzed the relative economics of five alternative bridge superstructures, each with five different span length layouts and each with two types of substructure support systems.

The construction budget of each bridge alternate studied was established from preliminary structural quantities, pertinent construction procedures, and current material costs.

Structural quantities for each bridge were estimated from established quantity survey charts and preliminary member sizes. Quantities for foundation piles and/or caissons were estimated after preliminary bearing capacities were established and relative bridge highway loadings applied thereto. Substructure quantity items include excavation, backfill, concrete, reinforcing steel, and piles or caissons. Superstructure quantities include concrete, structural steel, precast members, posttensioning steel, and barrier railings.

Unit prices for construction operations and materials reflect quotations from local material suppliers, experienced bridge contractors familiar with the project site and relative construction costs of five recent bridge projects, some of which are located on the Salt River.

One preliminary bridge layout was assumed for all bridge alternates studied. This assumed bridge was assigned a width of 84 feet and a length of 1,330 feet. The bridge soffit was set at

1,173.00, channel bottom at 1,150.00, and scour at 1,116.00. The channel width at high water was assumed to be 1,170.00 +/- feet.

A. <u>Bridge Superstructure</u>

Superstructure type alternatives considered in the study include:

1. Composite welded steel girder
2. Precast I-girder
3. Cast-in-place T-beam
4. Cast-in-place box girder
5. Cast-in-place prestressed box girder

B. <u>Bridge Span Layout</u>

The maximum span lengths with the corresponding number of piers considered for each alternative were established as follows (each layout included two abutments):

1. 98' - 13 piers
2. 115' - 11 piers
3. 127' - 10 piers
4. 156' - 8 piers
5. 177' - 7 piers

C. <u>Bridge Substructure</u>

Two types of bridge support systems were evaluated. The first system included conventional solid piers with driven piles. The second system employed drilled caissons which extend to the bridge superstructure, thereby eliminating the need for support piers. The five substructure alternates include:

1. HP - 10 x 42 piles
2. HP - 14 x 73 piles
3. 4' diameter caissons
4. 6' diameter caissons
5. 7' diameter caissons

Estimated capacities of piles and caissons used in this study are as follows:

HP - 10 x 42 pile	@ - 40'	=	140^k
HP - 14 x 73 pile	@ - 40'	=	200^k
4' diameter caissons	@ - 75'	=	235^k
	@ - 95'	=	585^k
6' diameter caissons	@ - 75'	=	255^k
	@ - 95'	=	815^k
7' diameter caissons	@ - 75'	=	265^k
	@ - 95'	=	875^k

NOTE: Allowable caisson capacities include correction for in-place member weights.

D. Type Selection Costs

A summary of the detailed type selection economic study has been prepared in a tabular format which permits a condensed review of alternate bridge type costs. This summary, presented on the following page, indicates the relative cost per square foot of deck for each type of superstructure versus costs for each type of substructure support system considered.

Relative costs for type combinations which are not cost effective due to either limiting span lengths or excessive substructure costs are not included in this cost summary.

Bridge type Alternates 3, 4, and 5 require inplace long-time shoring in the Salt River channel which would necessitate additional construction time for falsework erection and would at the same time be susceptible to damage from river flooding. Both of these factors increase relative square foot bridge costs.

Dewatering costs for pile foundations were estimated to run as high as $300,000, yet would remain below caisson-support systems studied.

Approach roadway costs were considered to be relatively equal for all bridge alternates and were not included in this portion of the study.

BRIDGE TYPE SELECTION COST DATA SUMMARY: L = 1330', W = 84', A = 111,270
(RELATIVE ESTIMATED COSTS)

4-27-81

BRIDGE TYPE	SPAN LAYOUT	STR. DEPTH	SUPER STRUCT $	SUBSTRUCTURE TYPE & DOLLARS (Pile or Caisson) HP-10x42	HP-14x73	4'0"	6'0"	7'0"	MINIMUM TOTAL BRIDGE $	COST SF SUPER	COST SF SUBST	TOTAL SF COST
<u>Composite Steel</u>												
1-1	14	5.3'	3,537	2,682	2,578	2,085	3,441	3,021	5,623	31.67	18.66	50.33
2-1	12	5.8	3,886	2,338	2,252	2,307	3,129	3,705	6,138	34.78	20.16	54.94
3-1	11	6.0	3,739	2,173	2,083	2,129	2,895	3,607	5,822	33.47	18.64	52.11
4-1	9	6.3	4,764	1,849	1,752	2,218	3,051	3,803	6,516	42.64	15.68	58.32
5-1	8	7.0	5,411	1,677	1,587	2,307	3,285	3,412	6,998	48.43	14.21	62.64
<u>Precast I-Beam</u>												
1-2	14	5.0	3,256	2,761	2,663	3,597	4,532	5,227	5,919	29.14	23.84	52.98
2-2	12	6.3	2,904	2,459	2,334	2,884	3,987	4,977	5,238	25.99	20.89	46.88
3-2	11	7.0	2,420	2,252	2,137	2,662	3,675	4,585	4,557	21.66	19.13	40.79
4-2	9	--	--	--	--	--	--	--	--	--	--	--
5-2	8	--	--	--	--	--	--	--	--	--	--	--
<u>Cont. T-Beam</u>												
1-3	14	5.2	2,620	2,845	2,696	3,328	4,611	5,758	5,316	23.45	24.13	47.58
2-3	12	6.0	2,827	2,514	2,374	3,373	4,845	4,977	5,201	25.30	21.25	46.55
3-3	11	7.0	2,932	2,362	2,229	3,550	4,455	5,564	5,160	26.24	19.95	46.19
4-3	9	--	--	--	--	--	--	--	--	--	--	--
5-3	8	--	--	--	--	--	--	--	--	--	--	--
<u>Cont. CIP Box</u>												
1-4	14	5.0	2,714	2,803	2,670	3,328	3,597	4,488	5,384	24.29	23.90	48.19
2-4	12	5.0	2,998	2,478	2,334	3,373	3,987	4,977	5,332	26.83	20.89	47.72
3-4	11	6.0	3,099	2,332	2,191	3,106	4,455	4,585	5,290	27.74	19.61	47.35
4-4	9	7.0	3,199	2,000	1,870	2,603	2,895	3,412	5,069	28.63	16.74	45.37
5-4	8	--	--	--	--	--	--	--	--	--	--	--
Post Ten-Box												
1-5	14	4.0	2,740	2,784	2,648	3,328	3,597	4,488	5,388	24.53	23.70	48.23
2-5	12	4.5	2,912	2,459	2,334	2,884	3,987	4,977	5,246	26.07	20.89	46.96
3-5	11	5.0	3,009	2,288	2,174	3,106	3,675	4,585	5,183	26.93	19.46	46.39
4-5	9	6.2	3,195	1,976	1,857	2,189	2,895	3,412	5,052	28.60	16.62	45.22
5-5	8	7.0	3,858	1,829	1,703	2,344	3,259	3,266	5,560	34.53	15.24	49.77

NOTE: $ IN 1,000

E. Type Selection Recommendations

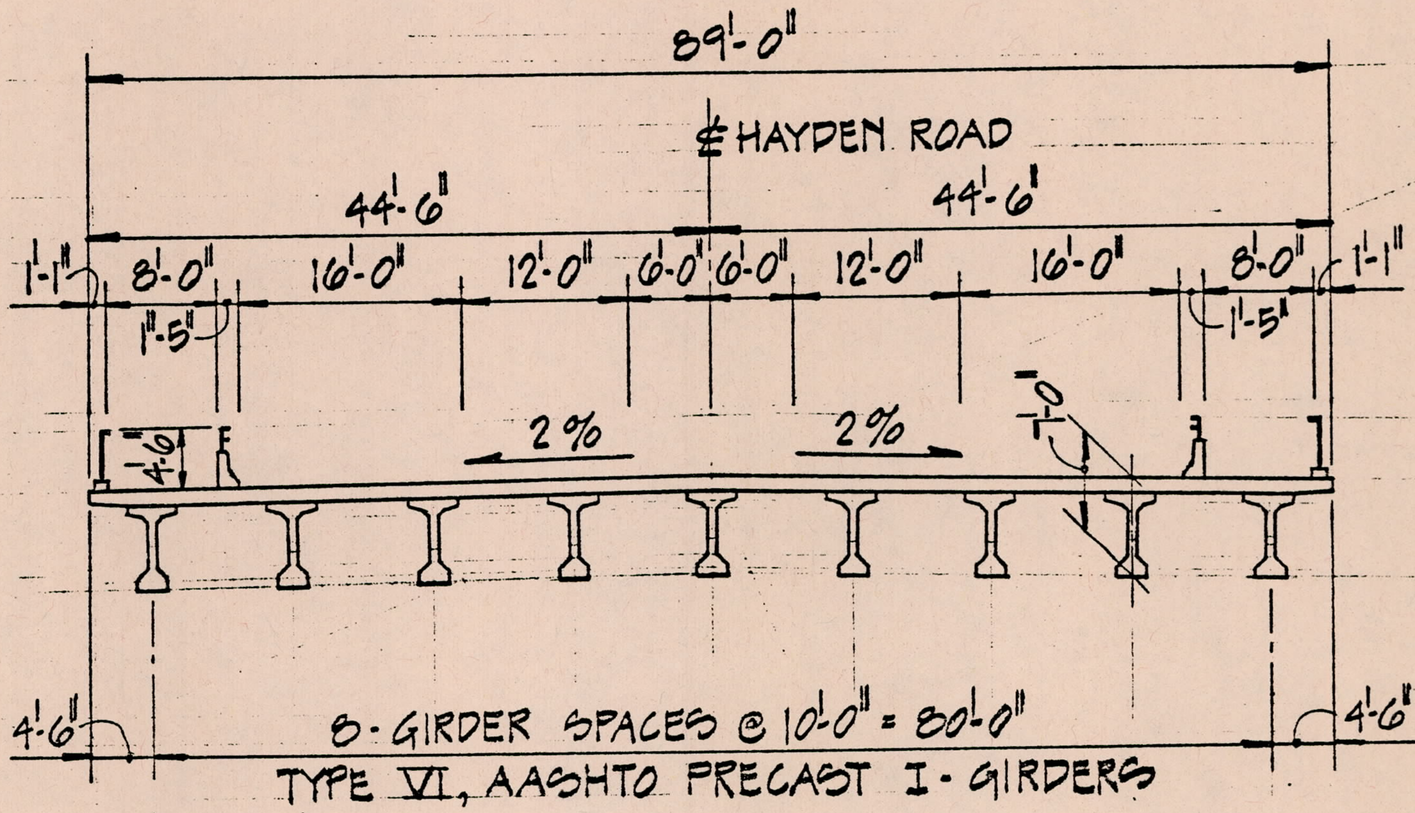
The bridge type economic study establishes the most cost-effective bridge type for the Hayden Road crossing of the Salt River as a precast I-girder superstructure with composite deck supported by conventional piers and piles. Piers shall be skewed 75 degrees from centerline of Hayden Road to parallel the Salt River channel.

Typical sections of the superstructure and substructure for the recommended bridge are shown on Figures 1 and 2, respectively.

F. Substructure Commentary

During the accumulation of data for the bridge type selection, pertinent information relating to the type of foundation system to be employed was obtained. This information, influential to the substructure design and final construction costs, does not appear in the type selection study. A summary of this information follows:

> Primary advantages of the pile-pier substructure system are: (1) piles may be test driven to verify expected capacities; (2) piles equipped with proper driving points are not anticipated to encounter excessive difficulty in driving through large cobbles or boulders located in the river bottom; (3) pile cap footings located below local scour are anticipated to behave well during the varying river bottom movements caused by erosion during high rates of channel flow;



BRIDGE
TYPICAL SECTION
SUPERSTRUCTURE

BRIDGE LAYOUT

CENTER MEDIAN = 12'-0"

TRAFFIC LANES = TWO EACH WAY WITH 4'-0" SHOULDER @ EXTERIOR LANES

BARRIER RAILING = CONCRETE AND RAIL

WALKWAY AND BIKE PATH = 8'-0" CLEAR

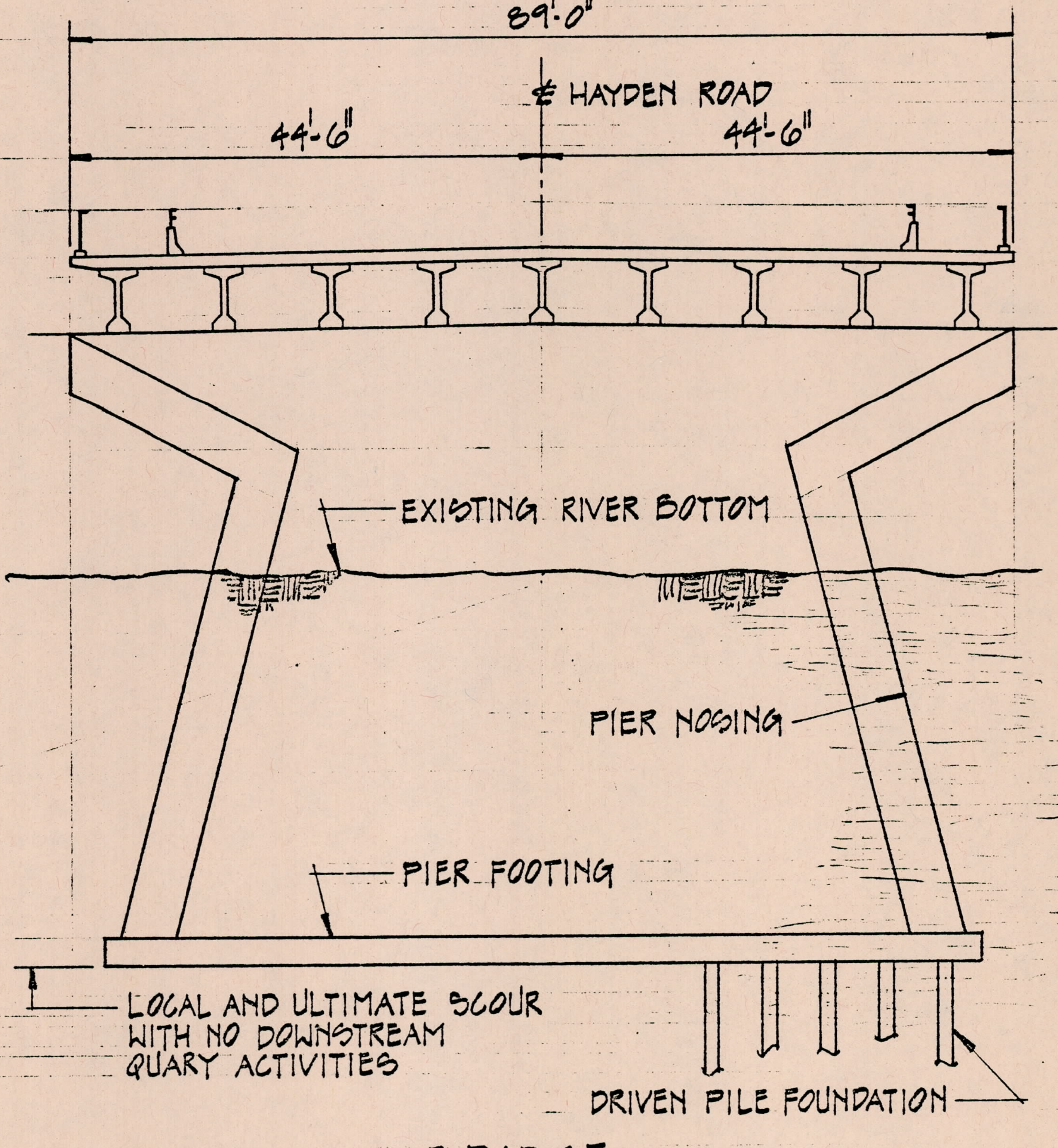
HANDRAILING = PICKET TYPE

TOTAL BRIDGE LENGTH = 1112'-0" ×

TYPICAL SPAN LENGTH = 123'-1" X

SUPERSTRUCTURE DEPTH = 7'-0"

FIGURE 1



BRIDGE
TYPICAL PIER

PIER AND RIVER SKEW TO HAYDEN ROAD = 15°

FIGURE 2

and (4) solid piers, skewed to match channel centerline, would assist in directing channel flow through the bridge structure. Although upstream nosings on the piers are not anticipated, vertical and tapered upstream pier edges are recommended for the deflection of debris and river channelization. Downsloping upstream pier edges are not recommended due to the inherent tendency to entrap debris which would create a downward nozzle effect, increasing local scour around the pier footing. The disadvantage of the pile-supported system is that of dewatering costs during construction when groundwater or river water is encountered. However, this disadvantage may be reduced to a minimum with proper construction timing.

The drilled caisson substructure support system is clean in appearance, requires no massive foundation excavation other than drilling, dewatering may not be a major consideration, and a cost savings from reduced structural weight may be realized. However, the disadvantages of drilled piers include: (1) difficulties in drilling through rocks and cobbles as observed with similar projects in river bottoms, (2) the inability to properly clean the bottom of the caisson shaft after drilling and thereby eliminating end bearing capabilities of the caisson, (3) the lack of control over the setting of reinforcing steel cages in the drilled holes allowing an undetermined amount of gouged material to collect in the bottom of the caisson, (4) the lack of control over side clearance between the reinforcing steel cage and foundation materials, and (5) it is not cost effective to load test large diameter drilled caissons for verification of either capacity or settlement.

A supportive conclusion for the recommended use of a pile-supported foundation system may be drawn from this commentary. Also, this recommendation parallels the cost effectiveness of utilizing piles as shown in the type selection economic study and as discussed in the geotechnical report for this project.

V. ROADWAY APPROACHES

The approach roadway will be constructed at a grade of 3 percent requiring a length of approximately 500 feet to the south and 430 feet to the north matching the existing roadway section. Materials used to construct approaches will be available locally from the channelization excavation adjacent to the proposed bridge.

A. Alignment

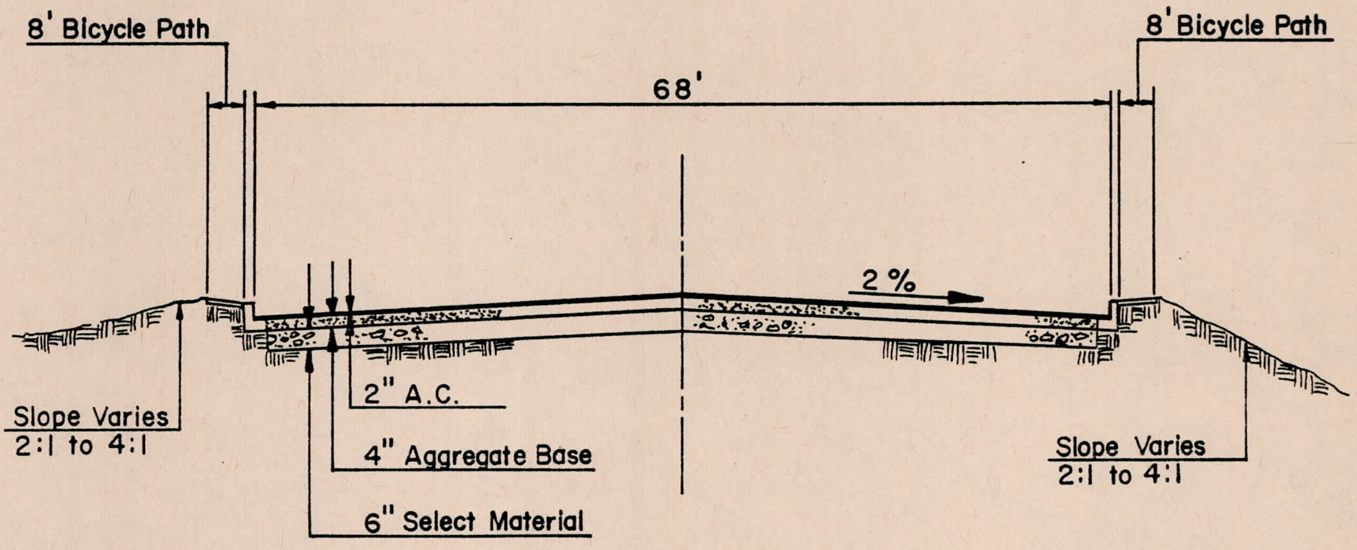
The horizontal alignment will be consistent with the existing centerline of Hayden Road. The vertical alignment will be dictated by the high water elevation developed in the hydraulic analysis prepared by Dames & Moore together with sight distances and freeboard criteria.

B. Roadway Section

Hayden Road is a major street requiring a width of 68 feet (face to face of curb). Curb, gutter, sidewalk, and landscaped side slopes will be included. The sidewalks will be 8 feet wide to make them compatible with the City of Tempe's bike path system along arterial streets. See Roadway Approach - Typical Cross Section, Figure 3.

C. Access

Secondary street access to Hayden Road now exists and will be accommodated in the proposed design provided sight distance and right-of-way alignment requirements are satisfied. This access is via Pima Street which enters Hayden Road adjacent



ROADWAY APPROACH

TYPICAL CROSS-SECTION

FIGURE 3

to the south abutment of the proposed replacement structure. The City of Tempe has indicated that although Pima Street is under the jurisdiction of Maricopa County, access should be maintained in order to accommodate businesses located in this area. Maintaining access to Hayden Road in this area may require the acquisition of additional right-of-way. Upon the completion of final design, definite right-of-way needs will be established.

VI. UTILITIES

A. Maricopa County - 24-Inch Storm Drain

Located south of the Salt River channel parallel to and 39 feet west of the centerline of Hayden Road is located a 24-inch storm drain outfall. This drain line discharges directly into the river through the south bank of the existing channel. The alignment of this storm drain will be revised to discharge further downstream of the proposed structure.

B. City of Tempe - 36-Inch Waterline

Alignment varies from 40 feet to 200 feet west of the centerline of the proposed structure. Actual depth and location will require verification in the field to determine any conflict.

C. City of Phoenix - Proposed 48-Inch Waterline

The proposed alignment of this waterline across the Salt River channel is 126 feet east of the centerline of Hayden Road. This alignment deflects to the west at approximately 45 degrees in the area of the bridge approaches and will not conflict with the bridge structure.

D. Arizona Public Service - 10-Inch High Pressure Gasline

Record drawings indicating the exact depth and location of this line are not available. The existing information (1973 Hayden Road Bridge Plans) indicates that this line is 66 feet east and parallel with the centerline of Hayden Road. At the intersection of Pima Street and Hayden Road, the line deflects approximately 45 degrees from its alignment until it reaches a distance of 8 feet east of the centerline and then continues south in a direction paralleling Hayden Road. The exact location and depth of this line will be verified in the field prior to construction and relocated if necessary.

E. Overhead Powerlines

APS presently has two sets of overhead powerlines near this project. These powerlines are parallel to the centerline of Hayden Road and are loated 149 feet and 214 feet, respectively, to the west. Near the northern end of this project, the lines turn to the east and cross Hayden Road. These lines should not conflict with the bridge construction. However, notes on the drawings will be required to draw the attention of the contractor to safety precautions.

VII. ESTIMATED PROJECT CONSTRUCTION COST

The estimated initial budget for the total bridge replacement, river channelization, and approach roadways is estimated to be $6,751,340. This budget estimate is based upon current prices for materials, labor, and includes contractor markup percentages. The bridge costs are relative as to the procedures and quantities estimated in the type selection study. Major subdivision items of construction will ultimately vary in quantity as final design develops as well as the correction for inflation factors used based upon the final construction scheduling. Project subdivision estimates costs are:

Bridge construction:		
a) Width @ 84', Length @ 1151.67'	=	$3,946,100
*b) Additional width @ 5'	=	235,000
BRIDGE TOTAL		$4,181,100
Approach roadways:		
a) Basic roadway @ 84' x 430/500	=	$ 163,500
*b) Additional width @ 5'	=	11,500
ROADWAY TOTAL		$ 175,000
Channelization = 2,000'		950,000
Slope protection =		320,020
Engineer's Estimate		$5,626,120
Contingencies and O&P @ 20% =		$1,125,220
ESTIMATED TOTAL		$6,751,340

*Increase in bridge width to allow for additional sidewalk clearance.

VIII. FINAL DESIGN CRITERIA

The final design of the new Hayden Road bridge and Salt River channelization upgrading will be based upon data developed during the assembly of this preliminary design report and the governing criteria as outlined below. Refer to Attachment 1 for the General Plan of the proposed bridge structure. Review comments regarding recommendations contained in this report and advisement as to the disposition of the concern over probable downstream river mining operations as presented herein will be instituted into the project final design.

A. Civil

1. Design Specifications - A policy on design of urban highways and arterial streets - AASHTO - 1975.

   Uniform Standard Specifications and Details for Public Works Construction - Maricopa Association of Governments, 1979.

   Arizona Department of Transportation Standard Specifications for Road and Bridge Construction - 1969.

2. Construction Specifications - Uniform Standard Specifications and Details for Public Works Construction - Maricopa Association of Governments - 1979.

   Arizona Department of Transportation Standard Specifications for Road and Bridge Construction - 1969.

3. Design Parameters

a. Design speed = 55 mph.

b. Vertical alignment grade = 3 percent.

c. Crown = 2 percent.

d. Roadway cross section (see Roadway Approach Typical Cross Section).

(1) 68 feet wide, face of curb to face of curb.

(2) Bicycle path 8 feet wide clear.

(3) New Jersey barrier.

(4) Side slopes maximum 2.5:1, minimum 4:1.

(5) Pavement section 2-inch A.C. over 4-inch ABC over 6-inch select material.

e. Channel

(1) Side slopes 2.5:1.

(2) Width 1,000 feet.

(3) 3 feet freeboard.

(4) Channel bottom elevation 1,150 at bridge.

(5) High water elevation 1,169.56 at bridge.

(6) Mannings "n" channel 0.035
overbank 0.040

(7) Design velocity 10 fps maximum.

(8) Channel slope 0.001 ft/ft.

B. Structural

1. Design Specifications - AASHTO Standard Specifications for Highway Bridges, 12th Edition 1977, and Interim Specifications dated 1978 and 1979. Load Factor design method will be used.

2. Construction Specifications - Uniform Standard Specifications and Details for Public Works Construction - Maricopa County Association of Governments, 1979.

3. Superstructure - The bridge superstructure will consist of precast I-girders with a cast-in-place deck slab. Structural continuity will be afforded through 3 span continuous segments for negative live loads. The precast I-girders will conform to AASHTO Standards for Type VI girders.

4. Substructure - The supporting substructure will have solid piers supported by driven piles. The piers will be skewered parallel to the flow in the main river

channel. In conjunction with the recommended channelization and embankment slope protection, the abutments will not be designed for complete approach roadway washout.

Estimated load capacity of driven piles:*

Pile Type	Bearing Capacity Below Scour		
	-30'	-40'	-50'
HP-10 x 42	75k	140k	--
HP-14 x 73	110k	200k	300k

*Refer to soils investigation performed by Dames & Moore.

5. Design Loadings

a. Dead Loads - concrete deck, railings, girders, and future utilities. An additional 2-inch future asphalt wearing surface at 25 psf of roadway surface will be included in dead load calculations.

b. Earth Pressures - Soil loading shall be assumed at 120 pounds per cubic foot for vertical loading and 88 pounds per cubic foot equivalent fluid lateral pressure for structure retaining earth.

c. Live Loads - The basic live loading shall be the standard AASHTO HS 20-44 with overload provisions. The standard lane loading will be a distributed 5/11.0 lanes per girder for each girder including the exterior sidewalk support girder. Sidewalk

~~the exterior sidewalk support girder.~~ Sidewalk design live load will be 85 psf for transverse deck slab design.

d. Longitudinal Forces - The design longitudinal forces of 5 percent traffic live load plus friction at expansion bearing shall be accommodated.

e. Wind Loads - Forces generated from wind loadings shall be applied to the superstructure and substructure in grouping combinations as specified in the AASHTO design specification.

f. Thermal Forces - Provision shall be made for stresses and movements resulting from temperature variations. The range of temperature shall be according to the following:

Mean temperature $70^{\circ}F$

Temperature Rise $40^{\circ}F$

Temperature Fall $40^{\circ}F$

g. Buoyancy - The buoyancy shall be considered as it affects the complete structure including piling.

h. Force of Stream Current - All portions of the structure subjected to streamflow forces will be designed to accommodate such forces. The maximum streamflow velocity used for this structure will be 12 fps. Maximum high water shall be set at elevation 1,170.0 which will be a minimum of 3 feet below the bridge soffit.

i. Creep and Shrinkage - Stresses resulting from creep and shrinkage which occur at various stages of construction for composite design shall be accommodated. Primary stresses resulting from creep and shrinkage occur during release of prestress for precast girders, time of composite deck placement, and under alternate loadings during the service life of the structure.

j. Earthquake Stresses - The Equivalent Static Force Method for determination of forces on the bridge structure shall be employed. The location of the structure will be in Zone 2; distribution of forces shall be as set forth in Section 1.2 of the AASHTO Design Specifications.

k. Loading Combinations - Group loading combinations to which the structure will be subjected shall conform to the requirements of Section 1.2.22 of the AASHTO Standard Specifications.

6. <u>Design Materials and Allowable Stress</u>

a. Reinforced Concrete -

(1) Deck slab:

fc' = 4,000 psi at 28 days, Class AA. (2) Piers, abutments, and footings:

fc' = 3,000 psi at 28 days, Class A.

(3) All other concrete:

fc' = 3,000 psi at 28 days, Class A.

b. Prestressed Concrete - Precast prestressed I-girders:

fc' = 5,000 psi at 28 days.

c. Prestressing Steel - Seven wire, uncoated, stress relieved:

ASTM A 416, 1/2" diameter at Fu = 270 ksi.

d. Reinforcing Steel - All sizes: ASTM A 615, Grade 60.

7. <u>Design Grades and Elevations</u>

a. Design channel bottom EL = 1,150.0

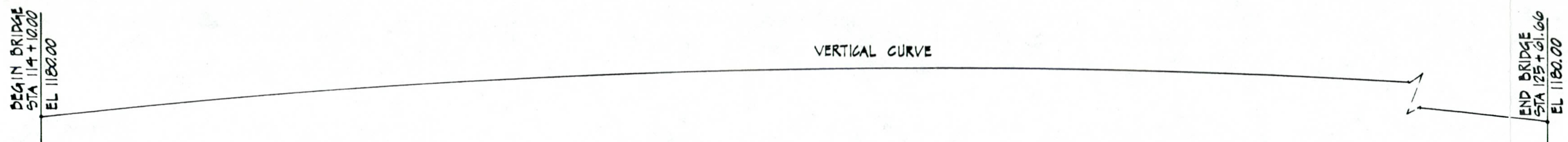
b. Design high water EL = 1,170.0

c. Minimum soffit EL = 1,173.0

d. Minimum ultimate scour without downstream river mining EL = 1,116.0

e. Maximum embankment slope, 2-1/2:1.

Fed. Road Div. No.	State	Project No.	Sheet No.	Total Sheets	As Built
7	ARIZONA				



PROFILE GRADE

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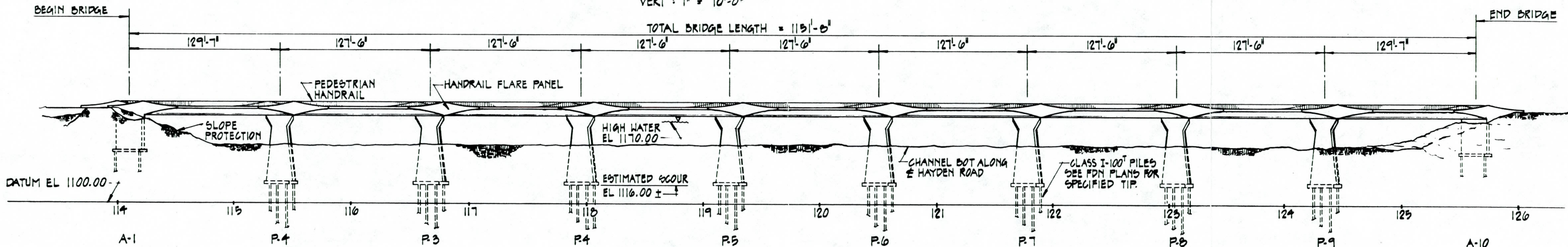
VERT : 1" = 10'-0"



ELEVATION

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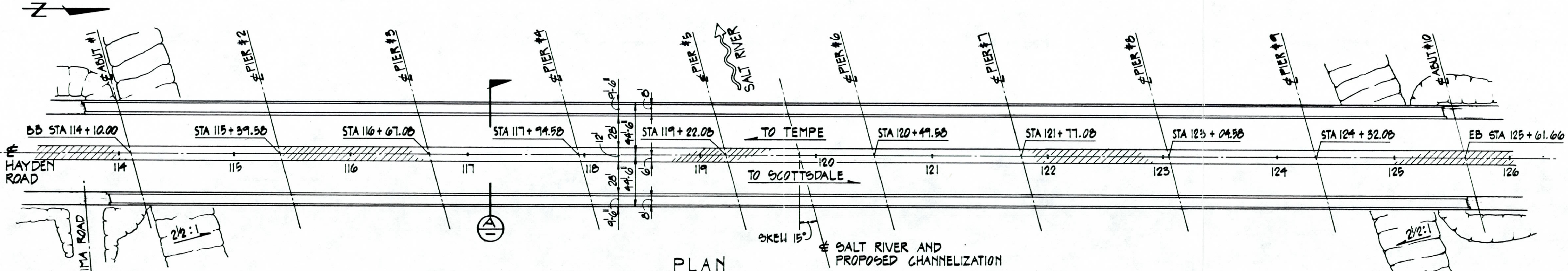
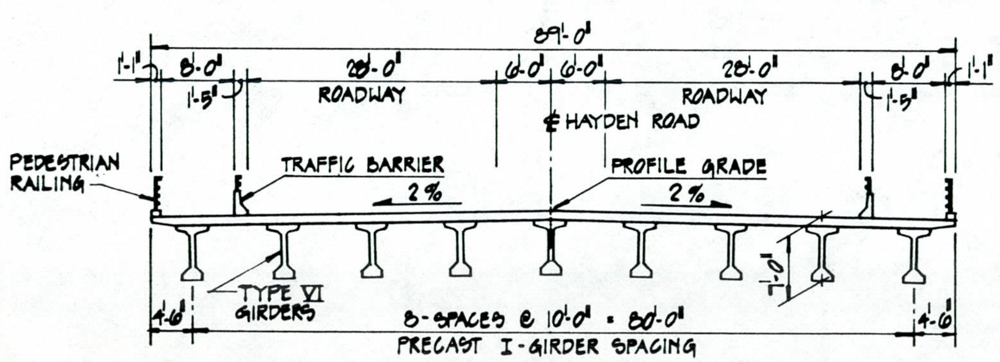


PLAN

SCALE 1" = 40'-0"

SKEW = 15°
SIN = .2588
COS = .9659
TAN = .2679

LIVE LOADING = HS20-44
STREAM FLOW = 200,000 CFS



TYPICAL SECTION (A)

SCALE : 1" = 10'-0"

MARICOPA COUNTY HIGHWAY DEPT.

M.C.H.D. PROJECT NO. | M.C.H.D. DRAWING NO.

HAYDEN ROAD BRIDGE
GENERAL PLAN

REV. | DATE | DESCRIPTION | APP

DESIGN / DRAWN / CHECKED / DATE / F.B.

Boyle Engineering Corporation
consulting engineers / architects

PH-M04-101-01

SHEET NO. OF SHEETS

APPENDIX I

REPORT

HYDRAULIC AND SCOUR ANALYSES
PROPOSED HAYDEN ROAD BRIDGE
MARICOPA COUNTY, ARIZONA

FOR

MARICOPA COUNTY DEPARTMENT
OF TRANSPORTATION

Dames & Moore

Dames & Moore

234 North Central Avenue, Suite 111-A
Phoenix, Arizona 85004
(602) 257-9440
TWX: 910-951-0637 Cable address: DAMEMORE

April 21, 1981

Boyle Engineering Corporation
3625 North 16th Street
Suite 107
Phoenix, Arizona 85016

Gentlemen:

We are pleased to present five copies of our "Report, Hydraulic and Scour Analyses, Proposed Hayden Road Bridge, Maricopa County, Arizona." This report was prepared under the terms of your Standard Form of Agreement with Consultant for Professional Services dated December 23, 1980 and our proposal to you dated December 19, 1980.

The purpose of this report was to assist Boyle Engineering Corporation in determining various design criteria for a proposed new bridge for Hayden Road at the Salt River.

We have enjoyed working on this interesting and challenging project. If you have any questions regarding the contents of this report or if we can be of additional service, please contact us.

Very truly yours,

DAMES & MOORE

William D. Webb

William D. Webb
Partner

WDW:jc

Attachment

REPORT

HYDRAULIC AND SCOUR ANALYSES
PROPOSED HAYDEN ROAD BRIDGE
MARICOPA COUNTY, ARIZONA

FOR

MARICOPA COUNTY DEPARTMENT
OF TRANSPORTATION

DAMES & MOORE
APRIL 1981

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10465
WEBB
Date Signed 4.22.81
ARIZONA, U.S.A.

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of our investigation of the hydraulic and scour design criteria for the proposed Hayden Road Bridge. Maricopa County has decided to install this structure to replace the existing bridge which was damaged by floodwaters in February 1980. Boyle Engineering Corporation is under contract with Maricopa County to develop design criteria for the proposed bridge. Dames & Moore is under subcontract to Boyle Engineering Corporation to conduct a feasibility-level hydraulic and scour analyses to aid in design.

1.2 BACKGROUND

The Hayden Road Bridge site at the Salt River was only a dip crossing for many years. In 1973, Maricopa County completed the bridge which is today at the site. This four-lane bridge has a spread-footing foundation, and was designed to pass 25,000 cubic feet per second (cfs). The structure was designed as a "perched" bridge, with the north approach lower than the top of the bridge; when the bridge design capacity was exceeded, the flow would cross the north approach and, if necessary, the approach material could be sacrificed to save the bridge. During periods of high flow in 1978 and 1979, the approach material was removed by erosion and had to be subsequently replaced. However, during the spring of 1980, a flow of approximately 180,000 cfs was experienced in the river, and local scour caused one pier to settle. Since that time the bridge has not been reopened, and traffic presently bypasses the bridge on a dip crossing.

The Salt River above the Hayden Road Bridge site drains over 13,000 square miles, having elevations ranging from 1,150 feet at the bridge to over 12,000 feet in the White Mountains. Flows within the watershed are partially controlled by six water conservation dams operated by the Salt River Project (SRP). Four of these dams are located on the Salt River, with the other two on the major tributary, the Verde River. Releases from the lowermost dam on each river are normally diverted into irrigation canals at Granite Reef Diversion Dam which is located below the confluence of the two rivers.

The SRP water conservation dams have only small outlet works. When heavy inflows occur it is not possible to achieve large discharges until the water level reaches the spillway crest, after which large releases are often necessary to protect the dams. A series of wet years have kept the system nearly full in recent years, with repeated large releases required.

The Salt River near the Hayden Road crossing has a wide alluvial bed. There is historical evidence of braiding, and a large meander loop is visible on the north side of the channel and upstream of Hayden Road. The river is relatively steep through the Phoenix area, with an approximate slope of 9 feet per mile. The high flows of the recent years have relocated large amounts of material in the river, and the channel is presently near the south side of the river with a large scour hole downstream of and adjoining the south end of the old bridge. The old Hayden Road Bridge is still in place.

Sand and gravel mining in the Salt River occurs both upstream and downstream of the Hayden Road Bridge site. The open pits may range up to

100 feet deep, and often in the past were used as sanitary landfill locations.

Approximately 1 mile downstream from the Hayden Road Bridge site, construction of a new bridge for Scottsdale Road is underway. This will be a four-lane bridge supported on drilled piers and designed to pass 200,000 cfs.

1.3 SCOPE OF WORK

The scope of work for the feasibility-level hydraulic and scour analysis included the following four items:

1. Determine the length of bridge required to span the river under design flow conditions without channelization.

2. Determine a reasonable channel configuration which would allow a shorter bridge to be installed without raising the water surface elevation upstream of the bridge.

3. Determine the maximum flow which could pass under the bridge as defined in Item 2 above, if the channelization was not included, without raising the water surface elevation upstream of the bridge.

4. Estimate the projected depth of scour at the bridge site for the bridge and channel as defined in Item 2 above.

2.0 HYDRAULIC EVALUATION

Surface water profiles through the study reach were modeled using the U.S. Army Corps of Engineers HEC-2 computer program. This program can accept a variety of input to model the flow, and this study used the following data:

1. Digitized cross sections along the study reach of the river.
2. The design flow.
3. A known water surface elevation at a downstream control point.
4. The Manning's "n" values for friction losses in the channel and overbank areas.
5. Certain dimensions of the bridges required to model the effects of each bridge on the flow.

The digitized cross sections were provided by Boyle Engineering Corporation. The design flow is 200,000 cfs. The Mill Avenue Bridge was selected as the downstream control point because of a known water surface elevation (1,153.2 feet) at that site during the February 1980 flow of about 180,000 cfs. An estimated water surface elevation of 1,162 feet at the Scottsdale Road Bridge location during the same flow was used to calibrate the computer model. The Manning's "n" values used for the channel and overbank areas were 0.035 and 0.040, respectively. The bridge design for the new Hayden Road Bridge is described in Section 2.2 of this report. A set of the final design drawings for the new Scottsdale Road Bridge was provided by Boyle Engineering Corporation.

The output from the HEC-2 computer program provides a wide variety of data for each cross section, including the water surface elevation, top width of flow, and the velocity of the water in the channel. This data is summarized in Table 1 for the required design conditions.

TABLE 1

FLOW CONDITIONS AT HAYDEN ROAD BRIDGE SITE

Flow Condition	Flowrate (cfs)	Water Velocity Under Bridge (fps)	Water Surface Elevation Under Bridge (ft)	Water Surface Elevation 2,000 ft Upstream (ft)	Top Width of Flow (ft)
BASELINE CONDITIONS	200,000	10.01	1,170.24	1,174.54	2,500
NEW BRIDGE WITH CHANNELIZATION	200,000	9.59	1,169.56	1,174.06	1,100
NEW BRIDGE WITHOUT CHANNELIZATION	200,000	13.21	1,169.54	1,175.30	1,100
	185,000	12.78	1,168.92	1,174.50	1,100

2.1 BASELINE CONDITIONS

The baseline conditions of this study were used to provide water surface elevations for the design flow without the new Hayden Road Bridge in place. This allows the subsequent comparison of the water surface elevations associated with the new bridge in place to those under "without bridge" or baseline conditions. The baseline conditions for this study included the new bridge at the Scottsdale Road crossing but without any channelization at that site. For the Hayden Road Bridge site, the baseline conditions used the existing channel contours but with the old bridge removed.

The results of the computer model indicate that the top width of flow would equal approximately 2,500 feet under design flow without channelization. Therefore, a bridge having a length of 2,500 feet would be required to span the river under these conditions.

2.2 NEW BRIDGE WITH CHANNELIZATION

Boyle Engineering Corporation provided Dames & Moore with the following criteria for the new Hayden Road Bridge:

1. Maximum water velocity of 10 fps is desirable under the bridge.
2. For the design flow, the low chord of the bridge would be 3 feet above the water surface.
3. No overbank flow would occur across the abutments or approaches.
4. Bridge piers will be 30 inches wide and aligned with the major direction of flow.
5. Bridge piers will be on 100-foot centers.
6. Vertical, semi-circular pier noses will be used.
7. A channel invert elevation of 1,150 feet will be used.

The assumed bridge section is shown on Plate 1.

A bridge with an approximate length of 1,100 feet would fulfill the above requirements if combined with a suitably channelized section of the river. The proposed channel would have a bottom width of 1,000 feet and would extend about 1,000 feet upstream of the bridge with sides parallel to each other and the bridge piers, but at an angle of about 75 degrees with the bridge alignment. This 1,000-foot length is necessary to align the flow with the bridge piers. The downstream channelization would also extend 1,000 feet with the south bank on the same alignment as the upstream south bank. The north bank of the downstream channel would provide an expansion angle of about 10 degrees for the flow. For this feasibility-level report, a channel slope of 0.001 was used to model the flow. Although this is a somewhat smaller slope than the average river gradient, it does represent a slope which fits well into the existing topography and performs well hydraulically. A plan and profile of the proposed channelization is shown on Plate 2. The proposed channel profile is shown at the slope used for the computer model. The thalwag shown on Plate 2 passes through the local scour hole caused by the old Hayden Road Bridge and it appears that the channel is above the riverbed. Actually, the channel will require extensive excavation in all areas except those adjacent to the thalwag. The downstream channelization would lower the water surface under the bridge to compensate for the rise in the water surface caused by the bridge piers and approaches constricting and obstructing the flow. This channelization with the bridge as described above would pass the design flow with a water surface elevation of 1,169.56 feet and velocity of about 9.6 feet per second.

It is important to note that the proposed channelization and the shorter bridge act together as a system. The channelization lowers both the water surface and the approach velocity of the water. It also aligns the flow with the bridge piers thus reducing local turbulence adjacent to the bridge piers.

2.3 NEW BRIDGE WITHOUT CHANNELIZATION

If the new bridge were constructed without the required channelization, the flow parameters would be significantly different from those described in Section 2.2. Table 1 shows projected flow parameters for the new bridge if no channelization was included. The computer modeling for this analysis assumed that the existing channel contours along the bridge alignment would not be significantly altered by the new bridge construction. The bridge design would be similar to that described in Section 2.2 and have a length of 1,100 feet as previously determined. The 200,000 cfs design flow would pass under the bridge with a water surface elevation of 1,169.54 feet which is comparable to the elevation of the channelized flow. However, the water velocity would be increased to about 13.2 feet per second which exceeds the desired maximum of 10 feet per second. Also, the water surface elevation at a point 2,000 feet upstream from the bridge would be raised 1.24 feet above the channelized flow elevation and 0.76 feet above the baseline flow elevation at the same point. For the non-channelized bridge, the flow would have to be reduced to about 185,000 cfs in order for the water surface elevation to be approximately equal to the baseline water surface elevation at the point 2,000 feet upstream of the new bridge as shown on Table 1.

The impacts of not installing the required channelization are further discussed in Section 3.4 of this report.

3.0 SCOUR ANALYSIS

The scour analysis for this study considered general riverbed degradation, local scour and scour due to contracted section.

3.1 GENERAL RIVERBED DEGRADATION

Degradation or general scour is the lowering of the channel bed over a large reach and a long period of time. A comparison of topographic data for the Salt River available for different periods during the past 29 years indicates that about 12 feet of general degradation has taken place at the Hayden Road Bridge site.

At least two explanations are available for the apparent degradation. First, upstream supplies of sediment have been reduced by construction of the SRP dams on the Salt River and Verde River. The clear water released from the lower dams on each river has a greatly increased capacity to transport sediment. The river will attempt to modify its slope as it picks up sediment, thus restoring equilibrium between its sediment load and sediment carrying capacity. Second, degradation may also be caused by the removal of large quantities of riverbed material by sand and gravel mining. During high flows, the river will attempt to modify its bed to a uniform gradient. This involved erosion of the high areas and deposition of material into the lower mined-out areas.

There is a ridge of rock extending across the Salt River at the present location of the Mill Avenue Bridge which is about 2 miles downstream from the Hayden Road crossing. The elevation of this rock outcrop varies across the river, but averages near the 1,120-foot level. The rock is presently exposed adjacent to several of the Mill Avenue Bridge piers.

The average elevation of the top of the alluvial material under the Mill Avenue Bridge was at about 1,150 feet in 1911 and at about 1,130 feet in 1929. The present conditions also show this elevation to be at about 1,130 feet. There were several significant flows recorded between 1911 and 1929, and there were also significant flows in 1941, 1965-66, 1973, 1978, 1979, and 1980. During the period 1952 to 1979, the general riverbed degradation at the Interstate 10 Bridge which is located about 5 miles downstream from the Mill Avenue Bridge has been estimated to be 25 feet. The general riverbed degradation at the Hayden Road crossing has been only about 12 feet during this same period. It appears that the rock ridge across the river acts, to some extent, as a check dam for sediment movement in the river. It is expected that this ridge of rock will continue to act in this manner in the future. Therefore, under worst case natural conditions, the general riverbed degradation could conceivably reach elevation 1,120 feet at the Hayden Road crossing. From a more realistic standpoint, the river would be expected to retain some slope above the top of the exposed rock. Although the overall river slope is approximately 9 feet per mile (0.0017), the 2-mile reach immediately upstream of the Mill Avenue Bridge presently has a smaller slope of 6.7 feet per mile (0.0013) as might be expected behind a sediment check dam. With a conservative slope of 0.001, the general riverbed degradation could be expected to reach elevation 1,130 feet in the vicinity of the Hayden Bridge crossing under natural conditions. This would probably occur at a rate of 1 to 3 feet per 100-year design flow.

It is always possible that the excavation of a large pit in the river channel could intensify general riverbed degradation in the vicinity of the pit. If such a pit were placed downstream of the proposed facility, the headcutting which would accompany normal flows in the river could be more significant on a local basis than the influence of the downstream rock check dam. A study recently completed by Anderson-Nichols/West for the Arizona Department of Transportation utilized a model study to determine the impacts of gravel mining in the Salt River.[1] The study used a flow of 210,000 cfs and concluded that a 60-foot-deep pit centered in the channel could influence the river channel as shown on Table 2.

TABLE 2

INFLUENCE ON RIVER CHANNEL OF 60-FEET-DEEP PIT

	Migration Distance (ft)	Migration Depth (ft)
Headcut	2,700	23
Lateral	300	7
Downstream	900	12

The report also concludes that "the creation of pits as a result of gravel extraction will result in serious damages to the channel and associated structures during flood events unless extraction is carefully controlled. Erosion processes, specifically downstream migration and long-term channel degradation, have the potential to substantially modify the channel bottom and undercut dikes, bridge piers, and other structures."

[1]Anderson-Nichols/West, Impact of Gravel Mining on the Salt River Channel at the I-10 Bridge, Phoenix, Arizona, 14 January 1981.

A sand and gravel operation located on the privately-owned land 1,600 feet downstream of the proposed new bridge could easily be excavated to a depth of 60 feet. This could seriously endanger the new facilities and possibly initiate structural failure. It is therefore highly recommended that sand and gravel mining between the proposed facilities and the Mill Avenue Bridge be prohibited. If this is not possible, then a monitoring program should be established to annually evaluate potential problems and initiate protection measures in advance of a flow which could cause failure. It may also be possible to reach an agreement with downstream land owners such that controlled excavations are permitted.

3.2 LOCAL SCOUR

Local scour is caused by disturbances in the water flow generated by the bridge piers. The vortices and eddies generated by the piers and any debris held against the nose or sides of the piers by the flowing water have an increased capacity to transport sediment. This increased sediment transport capacity can cause a scour hole to develop to the size at which the strength of the vortex is reduced and equilibrium is reached, i.e., the sediment supplied by the incoming flow is equal to that removed by the outgoing flows. In addition, the effects of local scour may be increased if the water strikes the piers at an angle instead of parallel to the piers as designed. Assuming parallel flows, the local scour was estimated by the following four methods: (1) Shen's Formula, utilizing the pier Reynold's number[2] (2) Neil's Formula[2], and (3) a modification of Neil's Formula[3]. The computed results and adopted value are given in Table 3.

TABLE 3

DEPTH OF LOCAL SCOUR (FEET) AROUND A PIER

Method 1	Method 2	Method 3	Adopted
6.09	6.96	7.48	7.0

The local scour analysis was made utilizing the computed main channel velocity and maximum flow depth for the peak discharge of 200,000 cfs. The computed main channel velocity and maximum depth from the HEC-2 program were 9.59 feet/second and 19.56 feet, respectively. The adopted value of 7.0 feet local scour depth is conservative.

3.3 SCOUR DUE TO CONTRACTED SECTION

Scour is sometimes caused by the increased velocity and turbulence in the section between the piers (contracted section). The increased velocities will remove material from the riverbed until the waterway cross sectional area is increased sufficiently to lower the velocities. This type of scour often occurs when the effective width of the piers is increased by accumulations of debris on the nose and sides of the piers. At some point,

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[2]National Cooperative Highway Research Program Synthesis of Highway Practice Scour at Bridge Waterways, 1970.
[3]Simons, Daryl B. and Fuat Senturk, Sediment Transport Technology, Fort Collins, Colorado, 1977.

the incoming sediment load will equal the outflowing sediment load and the system will be in equilibrium.

Scour due to a constricted section is not believed to be significant at this level of study with the 100-foot pier spacing of the proposed bridge design.

3.4 TOTAL DEPTH OF SCOUR

In general, bridge foundations are designed for the additive effects of local scour and general riverbed degradations. General riverbed degradation at the Hayden Road Bridge site under natural conditions will be limited to about elevation 1,130 feet by the downstream control created by the exposed rock ridge under the Mill Avenue Bridge. Local scour is caused by the contracted section of the bridge section, pier (and debris) obstruction to the flow, and possible skew between the water and pier alignments. Local scour with the new bridge and channel in place has been estimated to be approximately 7.0 feet for the design flow, no obstructions, and no skew. A factor of safety of 2.0 should be used to allow for debris build-up on the pier nose and/or up to 10 degrees of skew between the approaching water and the pier alignments. Therefore, an allowance of 14.0 feet for local scour is recommended for design purposes.

The lowest elevation at which scour could be expected to reach without the influence of a sand and gravel mining is about 1,116 feet (1,130-14). However, a large pit excavated below elevation 1,116 feet and downstream of the new bridge could intensify the projected scour problems at the bridge site and cause the bridge to fail. Sand and gravel mining in the river below the new bridge and upstream of Mill Avenue should be monitored to determine if a potential problem exists. Corrective action could then be taken to provide additional support for the bridge.

Without the channelization proposed with the new bridge, the higher water velocities could cause significant problems. If the flow approaches the piers on an angle greater than the 10 degrees accounted for by the factor of safety used in the local scour estimate, then the local scour could exceed the 14.0 feet used in this study for the channelized conditions. In addition, without the channelization, the major flow could continue to concentrate along the south bank of the river resulting in a more rapid degradation under the south end of the bridge with the north end of the bridge becoming less effective at passing the water under the bridge. With the major flow concentrated at the south end of the bridge, the bridge piers in this area would collect most of the floating debris which could also increase the effects of local scour at the south end of the bridge.

4.0 CHANNEL SIDE SLOPE PROTECTION

The calculated velocity of flow in the proposed channel in the vicinity of the bridge is 9.6 fps for the design flow rate of 200,000 cfs. This velocity is an average value for the channel; it does not consider possible higher velocities caused by local concentrations of flows. These local velocities can be expected to reach 14 to 15 fps. Therefore, the design of the side slope protection should assume a water velocity of 15 fps.

Armoring of the side slopes of the channel will be required to provide protection against erosion. This armoring should extend over the full length of the proposed channelization. The armoring should also be extended to protect the toes and tops of the slopes to prevent undercutting and topcutting, respectively. Alternative construction materials were evaluated for channel side slope protection. These were:

A. Gabion baskets
B. Riprap
C. Fabriform mats (grout-filled nylon forms)
D. Grouted riprap
E. Soil cement

A comparison of the estimated unit costs for these five alternatives based on an assumed design indicates that soil cement, fabriform mats, and riprap are the most economically attractive.

There is some doubt regarding the durability of the fabriform mat slope protection under the heavy abrasive action encountered during flows of the rate and velocity that may be encountered in the Salt River. Failure of only one mat under conditions of high flow could lead to rapid and complete slope failure of the channel bank. Because of this concern, the fabriform mat alternative was dropped from further consideration. Riprap is preferable to soil cement, in our opinion, because of its excellent durability. Riprap also has the ability to settle and redistribute its weight without detrimental effects on its performance as slope protection. However, the availability and cost of the riprap in the Phoenix area is questionable.

Based on discussions with the Portland Cement Association, soil cement has been used successfully for side slope protection for similar purposes and its performance has been satisfactory. Therefore, a properly designed and constructed soil cement slope lining should provide adequate protection against erosion by occasional flood flows in the river.

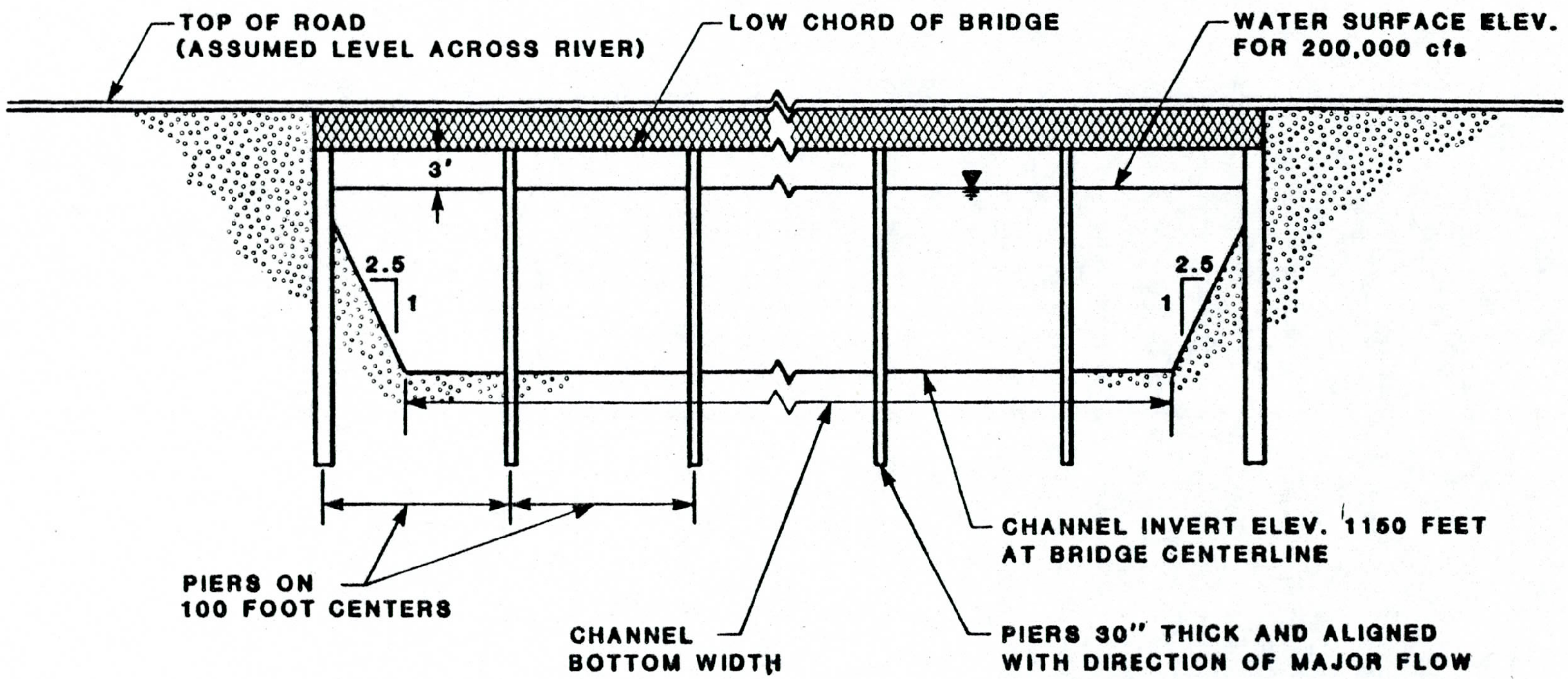
Based on the above considerations, the riprap and soil cement alternatives both have important advantages. Both are considered suitable from the technical standpoint, but the riprap is preferable.

If riprap is used, a 3:1 side slope is recommended. The riprap should have a median diameter of 15 inches. The riprap lining should be designed to remain stable under a flow velocity of 14 to 15 fps, with a 1.5 safety factor. Horizontal riprap protection for the toe of the slopes should be included to protect the banks up to a depth of 10 feet of local scour.

If soil cement slope lining is used, the channel side slopes should be 2:1. The slope protection should be designed to resist uplift pressures assuming the worst case condition that the soils behind the lining are fully saturated and the river channel is empty. Riprap toe protection is recommended for the soil cement lining. The toe protection should extend 3 feet up the side slopes to protect against abrasion from cobbles and rocks carried by the flow.

5.0 SUMMARY

A 2,500-foot-long bridge would be required to span the design flow under baseline conditions. However, an 1,100-foot-long bridge with the channelization as described in this report will also span the design flow and meet all of the design criteria for the bridge by Boyle Engineering Corporation. If the 1,100-foot-long bridge were to be constructed without the channelization, the design flow could pass under the bridge. However, the net effect of not including the recommended channelization would be that the depth, angle of approach, and velocity of the water passing under the bridge will vary greatly from one end of the bridge to the other, and could also vary during the flow period. The river has, in the past, exhibited the ability to concentrate flows at one end of a bridge during a flow and move to the other end during a subsequent flow. These concentrated flows and the accompanying increased scour would have to be anticipated and the bridge foundation overdesigned to reduce the potential for a bridge failure during the design flow or perhaps during a smaller flow. Sand and gravel mining in the vicinity of the new bridge should be controlled to the highest degree possible. Regardless of the level of control, this mining should be monitored so that potential problems can be identified and corrective measures taken prior to a flow which could otherwise close the bridge. General river-bed degradation should not lower the channel below the 1,130 foot level at the bridge unless sand and gravel mining aggravates the degradation. The natural degradation (no sand and gravel mining) should occur slowly as it is a function of the size and frequency of flows in the river. Local scour in the channelized section with the design flow should not exceed 14.0 feet.

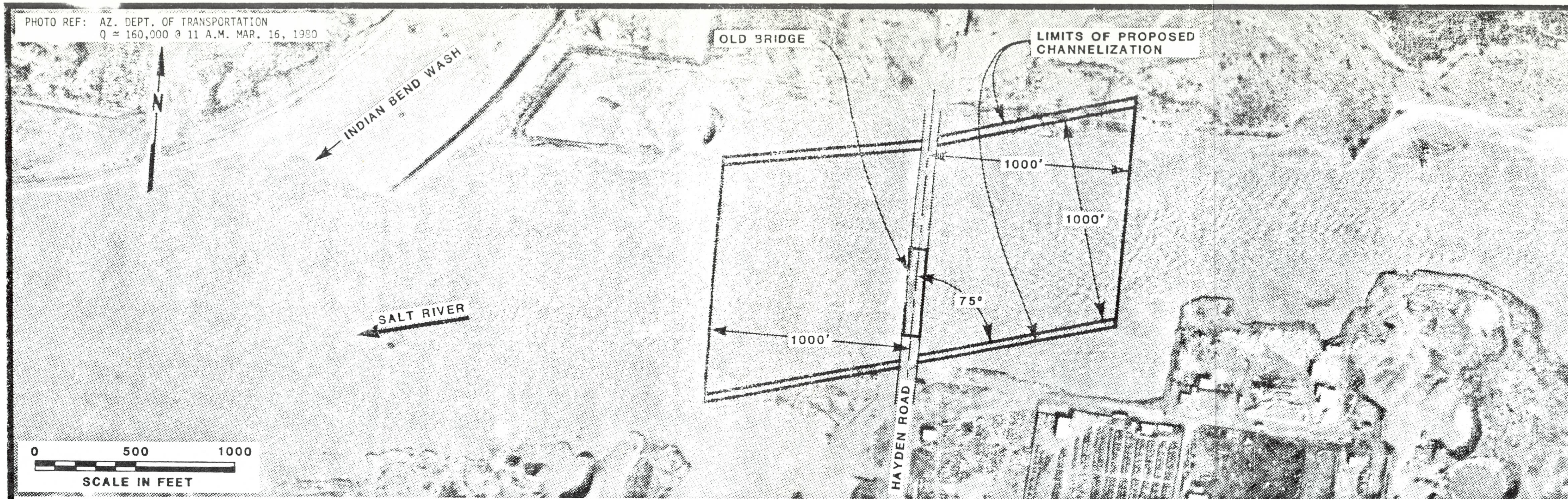


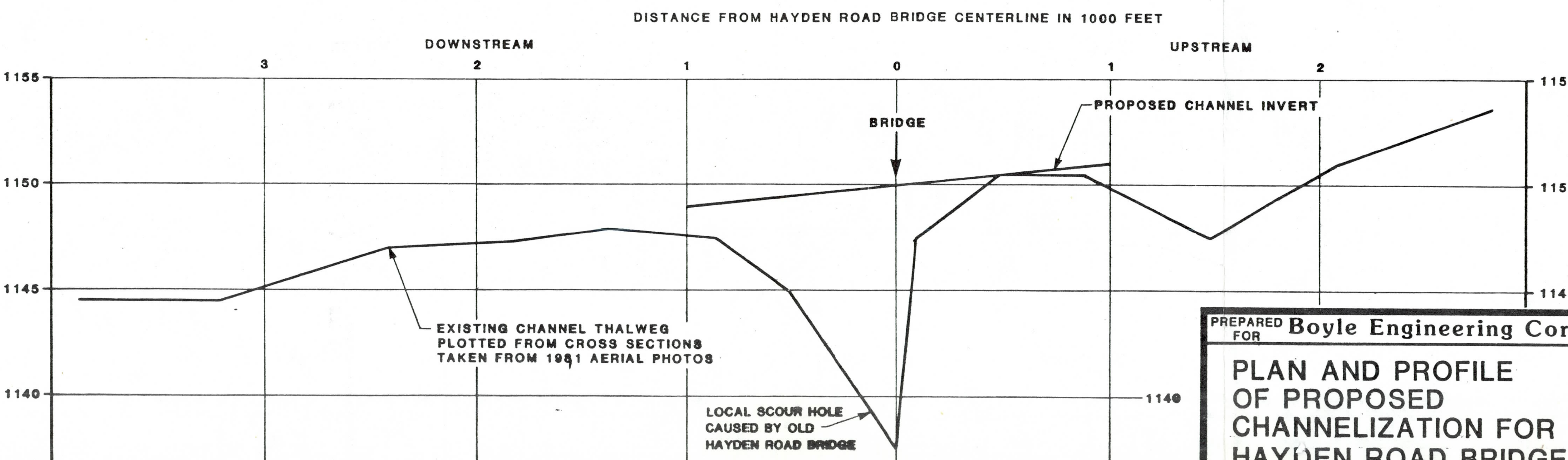
NO SCALE

PREPARED FOR Boyle Engineering Corp.

HAYDEN ROAD BRIDGE DESIGN ASSUMPTIONS FOR COMPUTER MODELLING

BY **Dames & Moore** Plate 1





PREPARED FOR **Boyle Engineering Corp.**

PLAN AND PROFILE OF PROPOSED CHANNELIZATION FOR HAYDEN ROAD BRIDGE

BY **Dames & Moore** Plate 2

APPENDIX II

REPORT

GEOTECHNICAL INVESTIGATION
PROPOSED HAYDEN ROAD BRIDGE
MARICOPA COUNTY, ARIZONA

FOR

MARICOPA COUNTY DEPARTMENT OF TRANSPORTATION

Dames & Moore

Dames & Moore

234 North Central Avenue, Suite 111-A
Phoenix, Arizona 85004
(602) 257-9440
TWX: 910-951-0637 Cable address: DAMEMORE

April 21, 1981

Boyle Engineering Corporation
3625 North 16th Street
Suite 107
Phoenix, Arizona 85016

Attention: Mr. Tom Larson

Gentlemen:

With this letter we are transmitting five copies of our "Report, Geotechnical Investigation, Proposed Hayden Road Bridge, Maricopa County, Arizona, for Maricopa County Department of Transportation." The purpose and scope of our investigation are outlined in our proposal dated December 9, 1980.

We appreciate the opportunity of performing this investigation. Please contact us if you have any questions.

Yours very truly,

DAMES & MOORE

William D. Webb

William D. Webb
Partner

WDW:jc

Attachments

REPORT

GEOTECHNICAL INVESTIGATION
PROPOSED HAYDEN ROAD BRIDGE
MARICOPA COUNTY, ARIZONA

FOR

MARICOPA COUNTY DEPARTMENT OF TRANSPORTATION

DAMES & MOORE
April 1981
Job No. 12269-001-16

Registered Professional Engineer
CERTIFICATE NO.
10465
WEBB
Date Signed 4-23-81
ARIZONA, U.S.A.

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TABLE OF CONTENTS (Concluded)

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation performed for the proposed Hayden Road Bridge located in Maricopa County, Arizona. The proposed bridge will replace the existing Hayden Road Bridge at the Salt River. The new bridge structure will be about 1,100 feet long and 84 feet wide; it will be designed to provide access across the Salt River during a flow of 200,000 cubic feet per second (cfs).

2.0 PURPOSE AND SCOPE

The geotechnical investigation was planned in discussions with Messrs. Tom Larson and Dave Scherchel of Boyle Engineering Corporation. The purpose of the investigation was to provide soil information and recommendations for the foundation design of the proposed Hayden Road Bridge.

The scope of the geotechnical investigation was divided into three tasks: (1) field investigation, (2) laboratory testing, and (3) engineering analysis and report preparation. The field investigation included drilling five borings and obtaining bulk soil samples for use in laboratory testing and identification. The locations at which the borings were drilled are shown on Plate 1, Plot Plan. The laboratory tests were performed in order to provide engineering data for use in our analyses and development of recommendations; descriptions and results of the laboratory tests are presented in the appendix of this report. Site conditions, project considerations, and our conclusions and recommendations are presented in the subsequent sections of this report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The Salt River near the Hayden Road crossing has a wide alluvial bed. There is historical evidence of braiding, and a large meander loop is visible on the north side of the channel and upstream of Hayden Road.

The ground surface at the proposed bridge site is alluvial material consisting of silts, sands, gravels, and cobbles. Existing surface elevations along the proposed bridge alignment vary from approximately 1,150 feet to 1,166 feet as shown on Plate 1. The existing Hayden Road Bridge, which was irreparably damaged by scour during the floods that occurred in 1980, is still present at the site. It is planned that this structure will be removed prior to construction of the new Hayden Road Bridge.

3.2 SUBSURFACE CONDITIONS

3.2.1 Subsoils

The subsurface conditions were investigated by drilling five borings ranging in depth from 69 to 78 feet. Detailed descriptions of the subsoils encountered are shown on Plates A-1A through A-1E, Log of Borings, in the appendix of this report. The locations of the borings are shown on Plate 1.

The subsurface soils encountered in the borings consisted typically of a mixture of silt, sand and gravel with frequent cobbles. Based on penetration resistances measured during advancement of casing with the Becker Hammer Drill, the subsoils range from medium dense to very dense. Bedrock was not encountered in any of the borings; however, the U.S. Geological Survey (Cooley, 1973) reports the bedrock surface to be underlying the alluvial materials at a depth of about 1,200 feet in this area.

3.2.2 Ground Water

The regional ground water table was not encountered within the maximum depth explored, 78 feet, by the borings drilled during this investigation. A few moist zones were encountered in the borings, but moisture in these zones is believed to consist of residual moisture resulting from the infiltration and percolation of water from flows in the river during the spring of 1980.

The U.S. Geological Survey (Osterkamp, 1973) reported that the ground water table in the vicinity of Hayden Road and the Salt River was at a depth ranging from 100 feet to 200 feet in 1972. However, depth to the ground water table in the Salt River channel is subject to large fluctuations depending on the frequency and duration of flows in the river. Based on discussions with sand and gravel companies with mining operations in the Salt River, the ground water table has risen more than 70 feet in elevation along certain portions of the Salt River channel since 1977. Infiltration of water during river flows in 1978, 1979 and 1980 has recharged the local ground water system, resulting in a much higher ground water elevation than would be expected under normal conditions.

3.3 SEISMICITY

The site is located in a part of the Basin and Range Physiographic Province which is characterized by very low historic seismicity. A map showing epicenter locations and magnitudes of earthquakes recorded in the region during the past 126 years (Sumner, 1976) is presented on Plate 2, Earthquake Epicenter Map.

Intensive investigations of the faulting and historical seismicity of the Basin and Range Province within Arizona were made during site selection studies for the Palo Verde Nuclear Generating Station (Fugro, 1974), which is now under construction about 50 miles west of Phoenix. No damaging earthquakes are known to have occurred in the region of the Basin and Range Province near Phoenix during historic time, and no evidence of active surface faulting has been found within 50 miles of the Palo Verde project.

The Basin and Range Province extends westward to the San Andreas Fault Zone, the zone of intense seismic activity nearest to Phoenix. The point on the San Andreas Fault system closest to the site is on the south branch of the San Andreas Fault (Crowell, 1975; Greensfelder, 1974), about 150 miles away.

Based upon recent work sponsored by the National Bureau of standards and the National Science Foundation for developing an expectancy map as shown on Plate 3 for effective peak accelerations within the United States, a design seismic acceleration of 0.05g may be used for the Hayden Road Bridge with a 90 percent probability that such an acceleration will not be exceeded in a 50-year period.

4.0 RIVERBED SCOUR

4.1 GENERAL

Foundation problems, including failures, have been experienced by several of the bridges crossing the Salt River within the Phoenix metropolitan area during flood discharges of recent years. Practically all of these foundation problems have been attributed to general and local scour. Furthermore, it is recognized that sand and gravel mining activities in the riverbed have aggravated and intensified the effects of general and local scour on many of the bridge foundations. It is our opinion that foundation design for the new Hayden Road Bridge must consider the possible additive effects of these scour mechanisms.

4.2 GENERAL SCOUR

General scour or degradation is the lowering of the riverbed channel over a long reach and a long period of time. A comparison of topographic data available for the Hayden Road crossing for the years 1952 and 1981 indicates that about 12 feet of degradation has taken place during this 29-year period.

At least two explanations are available for the apparent degradation. First, upstream supplies of sediment have been reduced by construction of the dams on the Salt and the Verde Rivers. The clear water released from the lower dams on each river has a greatly increased capacity to transport sediment. The river will attempt to modify its slope as it picks up sediment, thus restoring equilibrium between its sediment load and sediment carrying capacity. Second, degradation may also be caused by the downstream removal of large quantities of riverbed material by sand and gravel mining. During high flows, the river will attempt to modify its bed to a uniform gradient. This involves erosion of the high areas and deposition of material into the lower mined-out areas.

The magnitude of degradation that can be expected during each flood event is dependent on many factors and is difficult to estimate without the aid of a mathematical or physical model. Based on a mathematical model study performed for the I-10 Bridge (Dames & Moore, 1980) approximately 3 feet of degradation is predicted at that site during passage of a 100-year flood event. It is expected that a similar or slightly lower magnitude of degradation would be experienced at the Hayden Road Bridge crossing during a 100-year discharge.

As successive floods take place, the depth of degradation is expected to progressively increase until the riverbed reaches a stable, uniform gradient or until it is limited by some downstream control. It is our opinion that the near-surface bedrock extending across the Salt River at the Mill Avenue Bridge two miles downstream will provide such a control.

The Mill Avenue Bridge is supported by spread footing foundations based on bedrock. The rock is exposed adjacent to several of the bridge piers. The elevation of the bedrock varies, but averages about 1,120 feet. The average elevation of the riverbed at Mill Avenue Bridge in 1911 was at about 1,150 feet with about 30 feet of alluvial materials overlying the bedrock. The riverbed elevation was at about 1,130 feet in 1929. The channel bed elevation today is still about 1,130 feet. Based on these data, it appears that the elevation of the riverbed at the Mill Avenue crossing is relatively stable and will serve as a control against degradation of the channel for the reach of the river immediately upstream. Consequently, we believe it is reasonable to assume that degradation at the Hayden Road Bridge should not progress below approximately 1,130 feet.

4.3 LOCAL SCOUR

Local scour is caused by disturbances in the water flow generated by the individual bridge piers. The vortices and eddies generated by each pier have an increased capacity to transport sediment. This increased sediment transport capacity can cause a scour hole to develop at each pier to the size at which the strength of the vortex is reduced and equilibrium is reached, i.e., the sediment supplied by the incoming flow is equal to that removed by the outgoing flows. The magnitude of local scour that can be expected for the proposed Hayden Road Bridge was evaluated and is discussed in a separate report (Dames & Moore, 1981). For design purposes, it was recommended that 14 feet of local scour be assumed for a flood discharge of 200,000 cfs and the bridge layout, pier, and channelization configurations assumed in the analyses.

The effects of local scour should be additive to those of general scour. In other words, for design purposes, it should be assumed that the riverbed materials adjacent to the bridge piers could eventually be removed to approximately elevation 1,116 feet (1,130 feet-14 feet) due to the combined effects of general and local scour. Elevation 1,116 feet corresponds to a depth of 34 feet below the proposed channel elevation beneath the new bridge. It should be noted that the magnitude of combined general and local scour mentioned above does not include the impacts of possible sand and gravel mining in the immediate vicinity of the bridge.

4.4 POSSIBLE IMPACTS OF SAND AND GRAVEL MINING

A land ownership map provided to us by Boyle Engineering Corporation shows several properties downstream from the proposed Hayden Road Bridge to be owned by sand and gravel mining companies (The Tanner Companies and Union Rock and Materials, Inc.). The property owned by Union Rock and Materials, Inc. is situated immediately adjacent to the downstream right-of-way of the Hayden Road Bridge. The Tanner Companies' property extends from about 1,500 to 5,000 feet downstream from the bridge.

Communication by Boyle Engineering Corporation with these companies revealed that, in addition to the properties they own, the companies hold leases that allow them to mine sand and gravel from other properties in the immediate vicinity of the bridge. We understand that Union Rock and Materials, Inc. is uncertain regarding the depth to which they will mine materials from its property, but the maximum possible depth would be to the water table which they indicate is about 100 feet in depth. Union Rock and Materials, Inc. has mined to a depth of about 40 feet in this area in the past. The Tanner Companies indicated that new equipment that it is purchasing will have the capability to mine to a depth of 200 feet regardless of the depth to the water table.

Because of the uncertainty regarding the mining plans of both companies, it is very difficult, if not impossible, to accurately assess the impacts that their sand and gravel mining operations will have on the proposed bridge. However, based on a physical model study conducted to assess the impact of sand and gravel pits on the I-10 Bridge (Anderson-Nichols/West, 1981), it was found that erosion processes associated with the pits are not sensitive to the areal extent of the pits, but the erosion processes increase with increasing pit depth. Therefore, the depth of sand and gravel mining on the properties downstream from the proposed Hayden Road Bridge is believed to be the most important factor in assessment of impacts of the mining on foundation design of the bridge.

The headcutting which extends during high flows from the sand and gravel pits downstream from the bridge is the erosion process of greatest concern. The distance upstream to which the headcutting will extend will be dependent on pit depth. Based on results of the physical model study for the I-10 Bridge referenced in the previous paragraph, headcutting for a 60-foot-deep pit is predicted to extend for a distance of 2,700 feet upstream from the pit after a 210,000 cfs flood hydrograph. Our own extrapolitation of data presented in the report of the referenced study indicates that a 100-foot-deep pit would possibly result in a headcutting distance on the order of 4,000 feet and a 200-feet-deep pit would possibly result in a headcutting distance of over 7,000 feet upstream. Therefore, headcutting from sand and gravel mining activities located over a mile downstream from the bridge could conceivably have a detrimental impact on the bridge.

Assuming a pit is excavated immediately downstream from the bridge to a depth of 100 to 200 feet, it is estimated that headcutting during the passage of a 200,000 cfs flow would remove riverbed materials from under the bridge to depths in the range of 40 to 70 feet. This 40 to 70 feet range of maximum headcutting depths is considered only a gross estimate and should not be used for design purposes. A physical model study would be required to more accurately predict the maximum depths of headcutting at the bridge resulting from sand and gravel mining activities downstream. However, it is our opinion that the effects of headcutting should be of serious concern in design of foundations for the bridge and that the maximum depths of headcutting could be much greater than the predicted depths of general and local scour.

5.0 FOUNDATION SUPPORT

5.1 GENERAL

In view of the total depth of potential scour that could be experienced at the Hayden Road crossing, foundation support of the proposed bridge should be provided by deep foundations. The foundations should be designed to develop sufficient capacity to support the bridge from soils below the maximum depth of anticipated scour.

Ignoring the potential erosion effects at the bridge by headcutting due to the possible future development of sand and gravel pits downstream, foundations should be designed to derive their support from soils below the influence of general and local scour. As discussed previously in this report, the combined influence of general and local scour is predicted to not extend below elevation 1,116 feet.

However, it is not recommended that the potential erosion effects by downstream sand and gravel mining be ignored during design of the bridge. It is roughly estimated that, in the most extreme case, about 70± feet of the riverbed soils at the bridge crossing could be eroded away by headcutting from development of sand and gravel pits immediately downstream. Thus, development of foundation support below the limits of general and local scour under natural conditions would entail a certain degree of risk and may not be adequate if sand and gravel mining is permitted. On the other hand, design of foundations to develop support below the most extreme forseeable depth of headcutting will greatly increase the cost of the bridge and may not be practical. In other words, more cost-effective solutions to control the influences of downstream sand and gravel mining on the bridge may be available and should be investigated. These might include:

- Control of sand and gravel mining activities in the vicinity of the bridge by purchase of certain of the properties or by development of cooperative agreements that establish limitations on the location and depth of mining.
- Design and construction of improvements downstream of the bridge that will control the impacts of localized sand and gravel mining on the bridge.

In any event, we believe that a physical model study should be conducted for the project. The objective of the physical model study would be to better assess and predict the impact of erosional processes on the bridge resulting from localized sand and gravel mining and to better establish design criteria for foundations of the proposed Hayden Road Bridge.

5.2 ALTERNATIVE FOUNDATION TYPES

5.2.1 General

Two alternative deep foundation types were evaluated for support of the bridge. These were: (1) driven steel H piles and (2) drilled, cast-in-place concrete caissons. In our opinion both foundation types can be installed to develop sufficient bearing capacity below the limits of general and local scour to support the bridge. However, only drilled caissons can be extended to the depth required to develop sufficient bearing capacity below the maximum depth to which headcutting might extend. Due to the penetration resistance offered by dense strata and cobbles in the riverbed soils, it is doubtful that steel piles can be driven below a depth of about 50 feet.

Conventional design normally requires that the pile cap for driven steel piles be established below the maximum limits of anticipated scour. On the other hand, large diameter concrete caissons are often designed to provide direct support of the bridge superstructure without the requirement of a pile cap.

Both driven steel H piles and drilled, cast-in-place concrete caissons have been designed and have been or are being installed for several new bridges that are being constructed over the Salt River.

5.2.2 Driven Steel H Piles

5.2.2.1 General

In conjunction with Boyle Engineering Corporation, two different steel pile sections, HP 10x42 and HP 14x73, were selected for evaluation of their respective vertical bearing capacity and the lateral load-deflection capabilities.

Both the HP 10x42 and HP 14x73 pile sections have been successfully driven in the Salt River. However, we recommend the use of the heavier pile section because of its greater resistance to damage during driving. In addition, tip reinforcement should be required, and a Pruyn HP 77750 Point or approved equivalent is recommended for use.

5.2.2.2 Vertical Bearing Capacity

The allowable bearing capacities for the two H-pile sections were analyzed, and the results are presented graphically on Plate 4, Allowable Vertical Loads, H Piles. It was assumed in our analyses that the allowable vertical capacity would be derived by side friction only. End bearing was not considered in the bearing capacity analysis because of the uncertainty regarding the types of material on which the tips of individual piles might bear upon. For example, significant end bearing capacity may be realized if the tip of an individual pile was driven to bear on a large cobble. However, we believe that negligible end bearing capacity would be realized if the pile were driven to bear in fine sand. The allowable load capacities presented on Plate 4 include a factor of safety of 2.

5.2.2.3 Lateral Load Capacity

Analyses were conducted to evaluate the lateral load-deflection (P-Y) response of the soil to the H-pile sections. These evaluations were based on the procedures developed by Reese, Cox and Koop (1974) to create a family of P-Y curves for a pile section at various depths of imbedment. The P-Y curves relate soil resistance to pile deflection and depend on several parameters, including the soil shear strength, effective pile diameter, and depth of imbedment. The results of these analyses are presented as Plates 5 and 6, Lateral Load-Deflection. The P-Y curves shown were evaluated for an application of lateral load only, with the load applied at the ground surface.

5.2.2.4 Pile Group Efficiency

The effect of pile groupings is a function of the geometric arrangement of the group. For pile group capacity determination, 100 percent of the allowable vertical capacity may be assumed for each pile if the piles are placed in a linear arrangement parallel to the direction of lateral load and spaced not less than 5 pile diameters apart. For rectangular pile groupings and piles placed perpendicular to the direction of lateral load, pile spacings of not less than 3 diameters are recommended.

5.2.2.5 Pile Driving and Load Testing Program

As mentioned previously, we anticipate that high penetration resistance will be encountered during driving of steel H piles due to the erratic presence of dense substrata and cobbles. Should H piles be chosen for the proposed Hayden Road Bridge foundation system, we recommend that a number of test piles be driven at the site to evaluate the significance of this potential problem and to aid in developing final driving criteria for the piles. In addition, we recommend that at least one of the test piles be selected for load testings to confirm the calculated vertical bearing capacity presented in this report and to evaluate the load-settlement characteristics of the piles.

5.2.3 Drilled Cast-In-Place Concrete Caissons

5.2.3.1 General

The drilled caissons that are currently under construction at the Scottsdale Road and Country Club Road Bridges are both being installed by drilling with a bentonite mud. The concrete is then tremied into place, displacing the bentonite mud. We have assumed that this same displacement method of caisson construction would be used at the proposed Hayden Road Bridge should the drilled caisson foundation system be used. In conjunction with Boyle Engineering Corporation, three different caisson diameters were selected for evaluation of their respective allowable vertical and lateral load characteristics. The results of these analyses are discussed below.

5.2.3.2 Vertical Bearing Capacity

The results of analyses of the allowable vertical load capacities for 4, 6 and 7-foot diameter drilled caissons are presented graphically on Plate 7. A factor of safety of 2 is included in the allowable vertical loads. Only side friction was assumed in computation of the allowable vertical capacities. No end bearing was assumed because of the inherent problem involved with construction of drilled caissons by the tremie method. It is our opinion that unless careful hand cleaning of the bottom of the caisson excavation is performed prior to concreting, the end-bearing capacity of a drilled caisson should be assumed to be zero. We believe this opinion is substantiated by a research report conducted for the Arizona Department of

Transportation (Beckwith and Bedenkop, 1973), in which approximately 3 inches of sluff material was intentionally left at the base of a drilled pile. The research report indicates that very low strains were required to mobilize side shear, with considerably higher strains required to mobilize end bearing. By displacing bentonite mud with concrete, the type and thickness of material at the caisson base is a major question and point of uncertainty. With a possible compressible layer at the caisson's base, the strain required to mobilize end bearing is an unknown, and could be on the order of inches.

5.2.3.3 Lateral Load Capacity

Lateral load-deflection (P-Y) characteristics were evaluated for 4, 6, and 7-foot diameter drilled caissons. The analyses for the drilled caissons were similar to those performed for the driven H piles, with the subsequent development of a family of P-Y curves for the different imbedment depths. As with the H piles, these P-Y curves relate the soil's resistance to lateral displacements to caisson deflection and depend on the soil's shear strength, caisson diameter, and the depth of imbedment. The results of the analyses are presented as Plates 8, 9, and 10, Lateral Load-Deflection. The lateral load analysis assumed that the lateral load is applied at the ground surface.

5.2.3.4 Caisson Group Efficiency

It is anticipated that any drilled caissons would be placed in a linear arrangement parallel to the flow of the Salt River. Using this linear geometry, we recommend that the caissons be placed a minimum of 5 diameters apart. At a linear group spacing of 5 diameters, the reduction in bearing capacity from soil interaction between the caissons is minimal. The efficiency of each individual caisson should be reduced if closer spacing is required.

5.2.3.5 Test Caisson Installation

It is recommended that a test caisson be satisfactorily installed at the site prior to installation of any caissons to support the bridge piers. Difficulties are being encountered during the installation of caissons for several bridges currently under construction. In our opinion, many of the difficulties are due to lack of familiarity of the contractors with the soil conditions in this area. We believe that a test caisson is necessary to allow the selected contractor to identify and solve any installation and equipment problems he might have prior to attempting to install caissons that will support the bridge.

5.3 FOUNDATION SETTLEMENT

Assuming the allowable capacities and related recommendations presented in the previous sections of this report are adopted for design, total foundation settlement of each pier should not exceed 1 inch for either of the foundation systems considered. Because of the granular nature of the supporting subsoils, the settlement is expected to take place very quickly after the load is applied.

5.4 LIQUEFACTION

Problems resulting from liquefaction are usually associated with loose, saturated silts and sands. Soils encountered during the field investigation for the proposed Hayden Road Bridge indicate only occasional traces of fines, with most of the soils in a medium to dense condition. In our opinion, these factors in conjunction with the low seismic risk of the site indicate that the liquefaction potential of the structure is negligible.

5.5 CEMENT

The Maricopa Association of Governments' Specification 725.2 indicates that Type V cement is required if the level of soluble sulfates in the soil or water exceeds 1,500 ppm. Laboratory testing of the alluvial soils of the riverbed contains approximately 359 ppm soluble sulfates as presented in Plate A-8, Chemical Test Results, in the attached appendix. A recent chemical study of the Salt River (U.S. Geological Survey, 1980) indicates levels of soluble sulfates in the water on the order of 380 ppm. Based on the described chemical testing, we believe that Type V cement is not required and that Type II cement should resist chemical reactions associated with the soluble sulfates.

6.0 CUTS AND FILLS

6.1 CUT AND FILL SLOPES

The cut and fills for the proposed Hayden Road Bridge should be constructed at a maximum slope of 2:1 (horizontal:vertical) for temporary purposes. The temporary cut and fill criteria applies only to that construction which is free of water and not intended for permanent use. Cuts and fills that are intended to stand as permanent features should be constructed with maximum slopes of 2.5:1 (horizontal:vertical).

6.2 SITE PREPARATION AND STRUCTURAL BACKFILLS

The preparation of the proposed bridge site will be as required and approved by the site engineer for construction of the deep foundations, the bridge piers, and the bridge abutments. It is our opinion that with minimal processing the sands and gravels excavated for the bridge piers and abutments can be used as structural backfill. Structural backfills should be free-draining sands and gravels with no particles larger than about 3 inches in maximum dimension. In addition, the backfill material should contain less than 10 percent by weight passing the U.S. standard No. 200 sieve.

The structural backfills should be placed in layers not exceeding 8 inches in loose thickness before compaction and be placed at a moisture content of ± 2 percent of optimum moisture as defined by ASTM D 1557-78. Requirements for preparation of the structural backfills should include compaction of each lift to at least 90 percent of the maximum dry density of the soil as determined by ASTM D 1557-78.

7.0 LATERAL EARTH PRESSURES

Lateral earth pressures were evaluated for design of abutments and underground structures associated with the proposed bridge. These pressures are expressed as equivalent fluid weights for both active and passive conditions. Equivalent fluid pressures of 88 and 300 pounds per cubic foot are recommended for active and passive earth pressures, respectively. The equivalent fluid pressures presented above assume saturated soil conditions.

*   *   *

The following plates, references, and appendix are attached and complete this report:

Plate 1 - Plot Plan

Plate 2 - Earthquake Epricenter Map

Plate 3 - Contour Map of Effective Peak Acceleration (EPA)

Plate 4 - Allowable Vertical Load, H-Piles

Plate 5 - Lateral Load-Deflection, HP 10x42

Plate 6 - Lateral Load-Deflection, HP 14x73

Plate 7 - Allowable Vertical Load, Drilled Caissons

Plate 8 - Lateral Load-Deflection, 4-Foot-Diameter Drilled Caissons

Plate 9 - Lateral Load-Deflection, 6-Foot-Diameter Drilled Caissons

Plate 10 - Lateral Load-Deflection, 7-Foot-Diameter Drilled Caissons

References

Appendix A - Field Exploration and Laboratory Testing

Respectfully submitted,

DAMES & MOORE

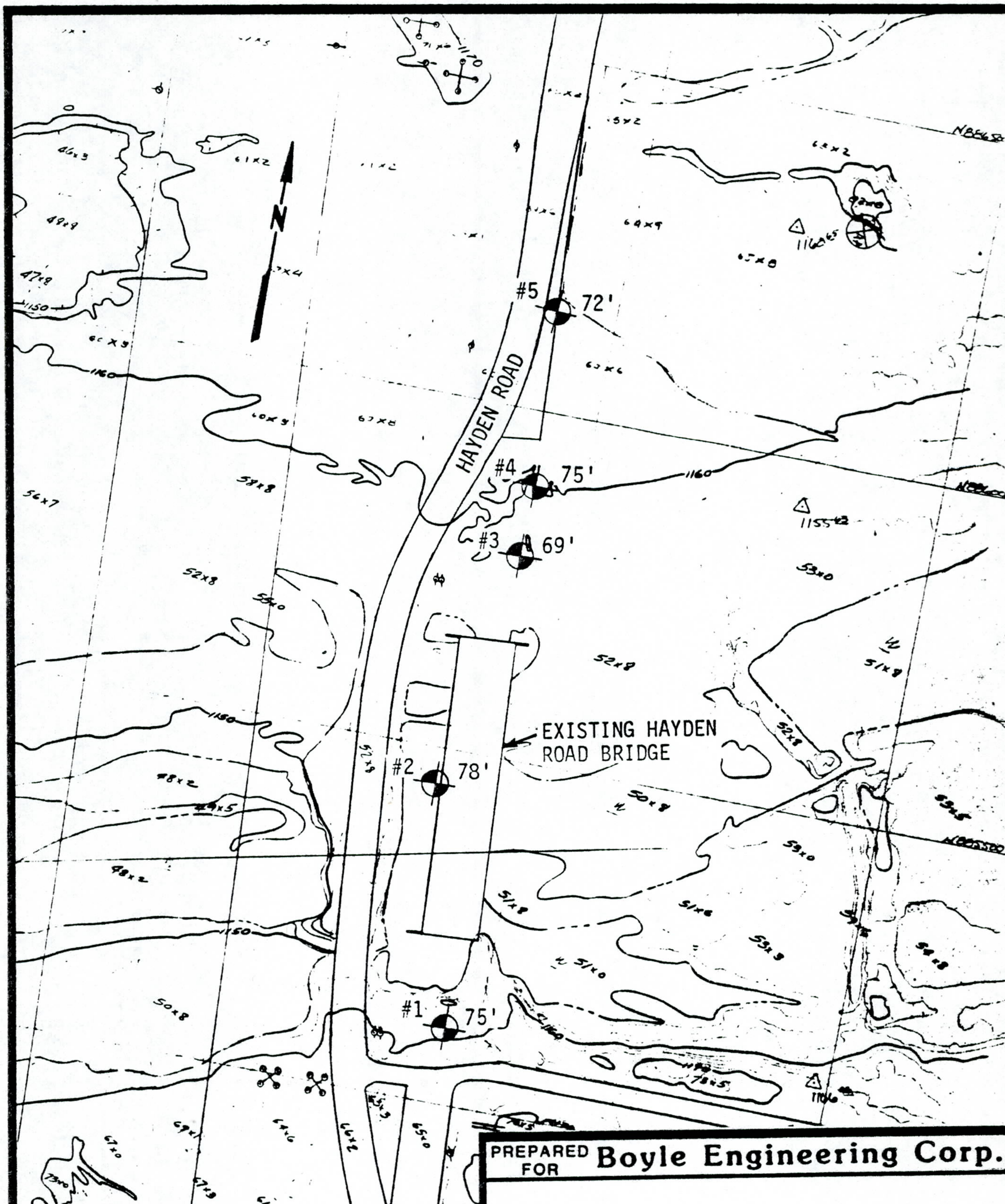
William D. Webb

William D. Webb
Partner

Robert D Brathovde

Robert D. Brathovde
Staff Engineer

WDW:RDB:jc



KEY:

SCALE: 1"= 200'

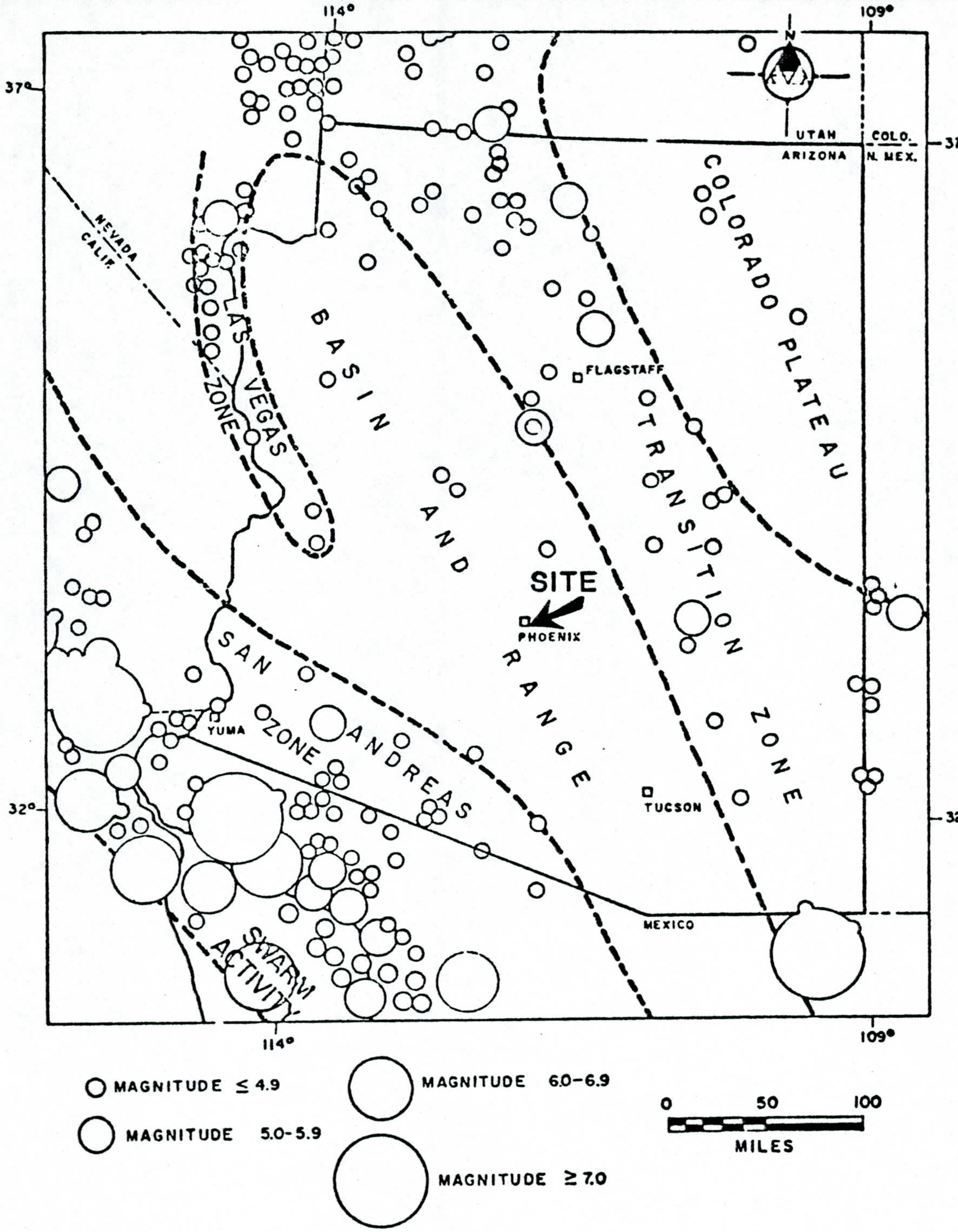
#1 75' DAMES & MOORE BORING NUMBER AND FINAL DEPTH

REFERENCE: BOYLE ENGINEERING DRAWING 790225-H, MILL & HAYDEN FLT 2/3A-4A

PREPARED FOR **Boyle Engineering Corp.**

PLOT PLAN

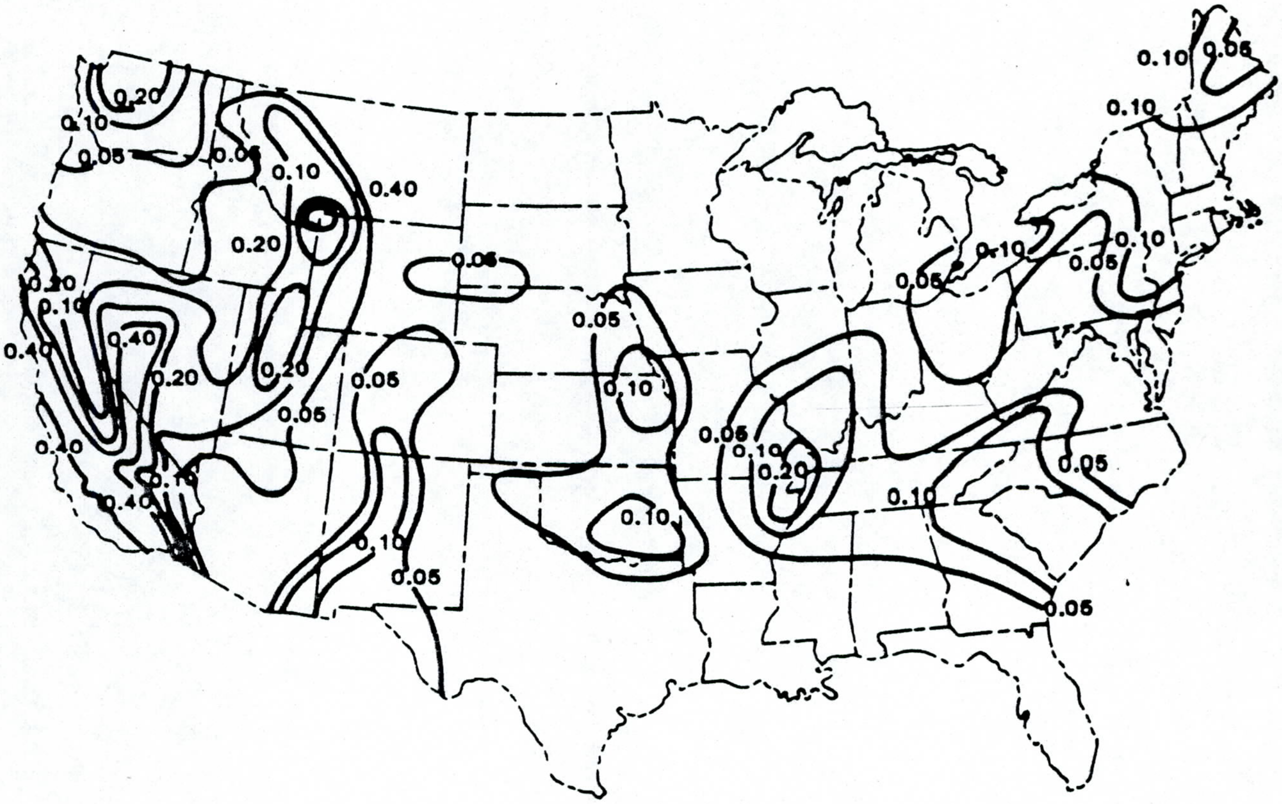
BY **Dames & Moore** Plate 1



EARTHQUAKE EPICENTER MAP

Reference: "Earthquakes in Arizona" by Sumner, FIELDNOTES, Vol.6, No.1, March, 1976, Arizona Bureau of Mines

BY **Dames & Moore** Plate 2



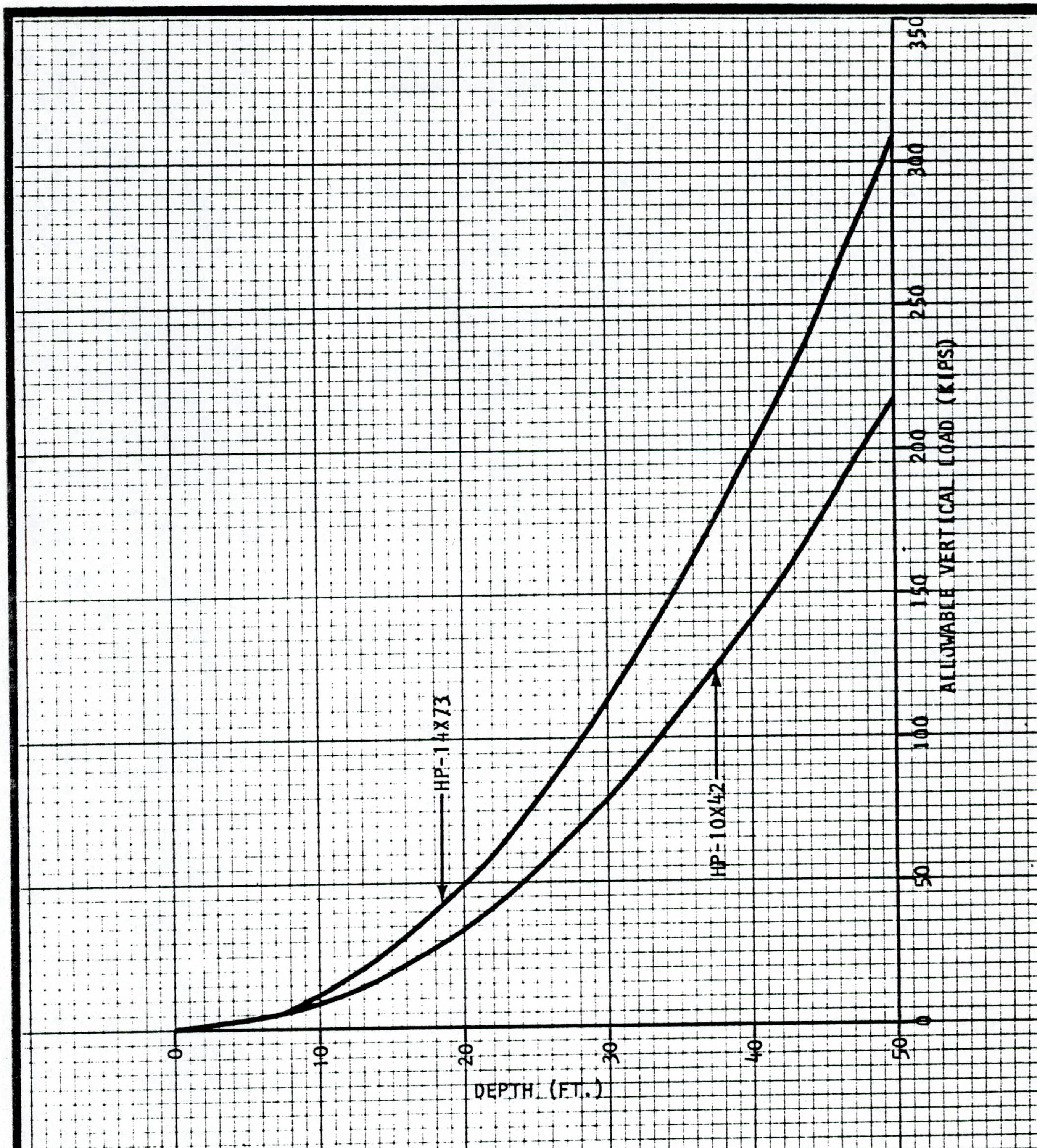
CONTOURS REPRESENT EPA LEVELS WITH A NON-EXCEEDANCE PROBABILITY OF BETWEEN 80 AND 90 PERCENT DURING A 50 YEAR PERIOD

CONTOUR MAP OF

EFFECTIVE PEAK ACCELERATION (EPA)

REFERENCE: DONOVAN, NEVILLE C., BOLT, BRUCE A., AND WHITMAN, ROBERT V., "DEVELOPMENT OF EXPECTANCY MAPS AND RISK ANALYSIS".

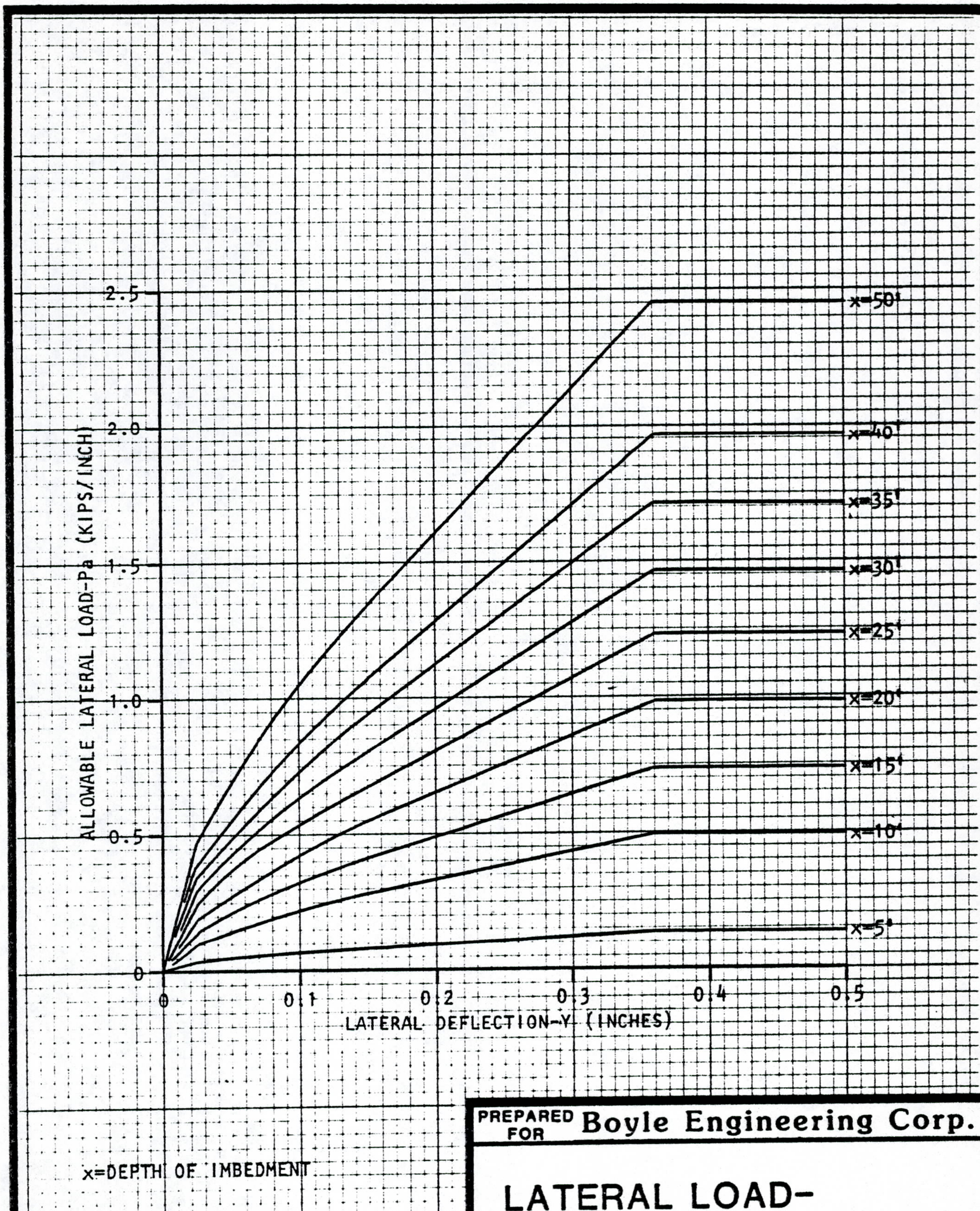
BY **Dames & Moore** Plate 3



PREPARED FOR **Boyle Engineering Corp.**

ALLOWABLE VERTICAL LOAD H-PILES

BY **Dames & Moore** Plate 4

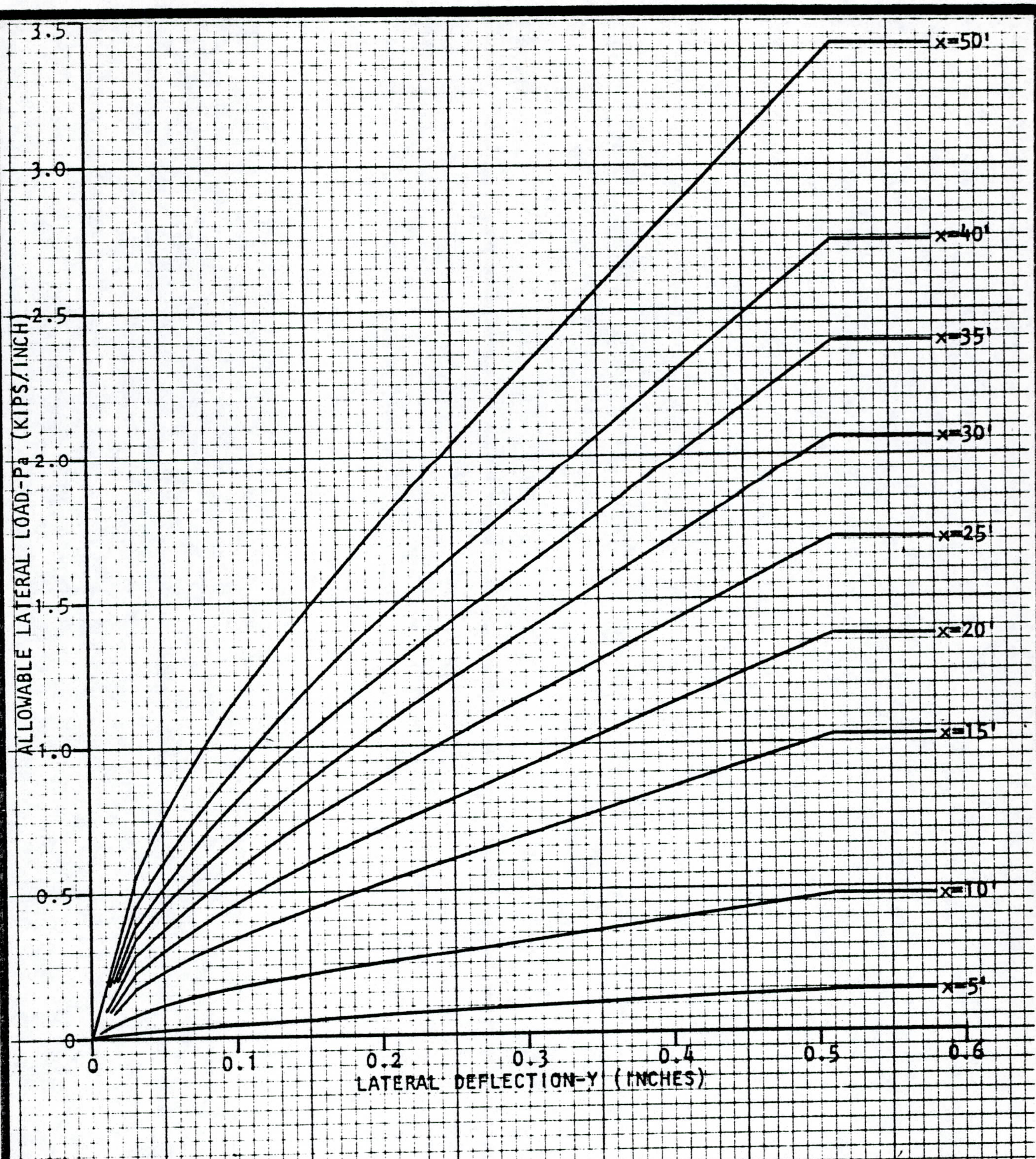


PREPARED FOR **Boyle Engineering Corp.**

LATERAL LOAD-DEFLECTION HP 10X42

BY **Dames & Moore**

Plate 5

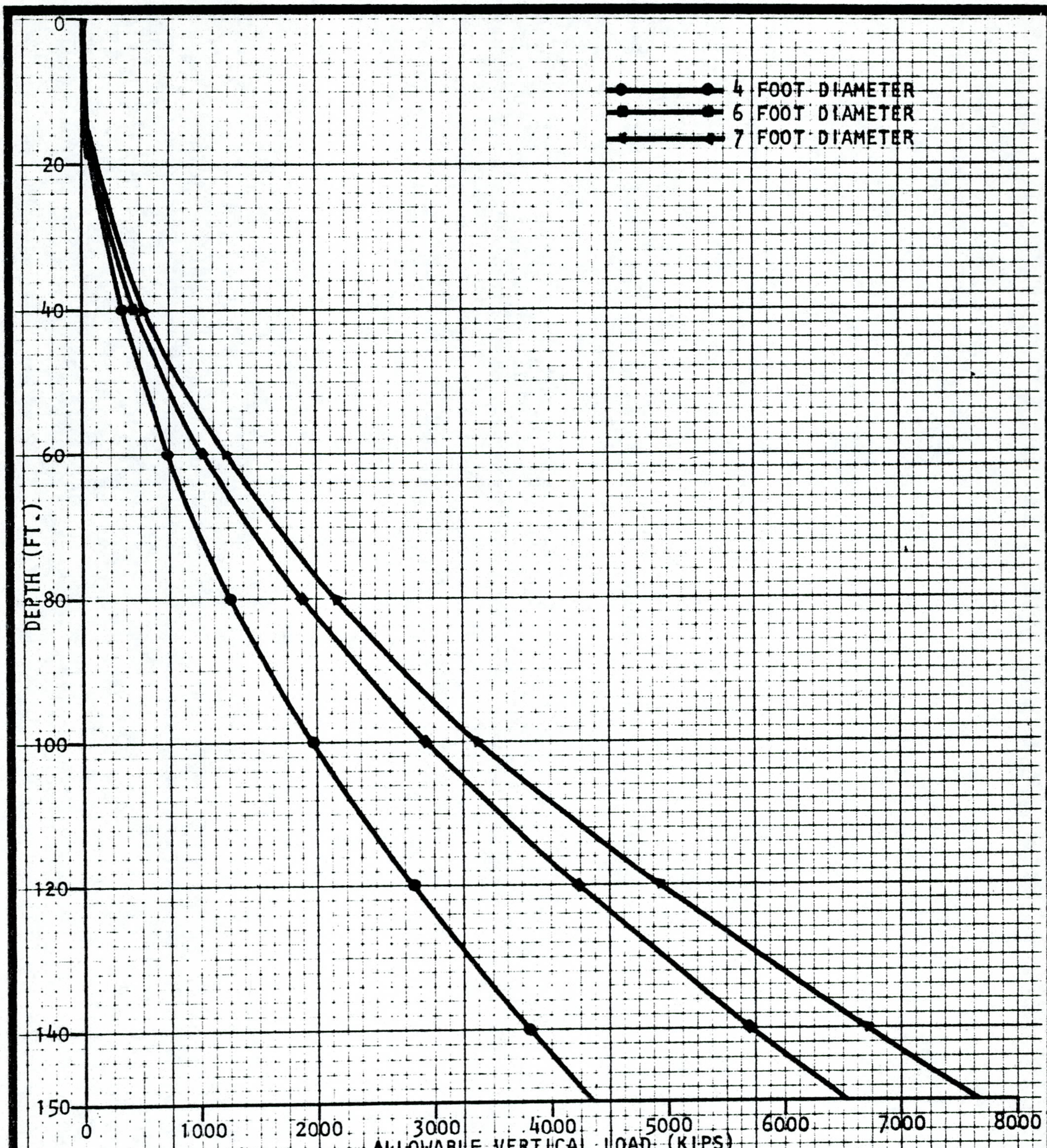


PREPARED FOR **Boyle Engineering Corp.**

LATERAL LOAD-DEFLECTION HP 14X73

BY **Dames & Moore**

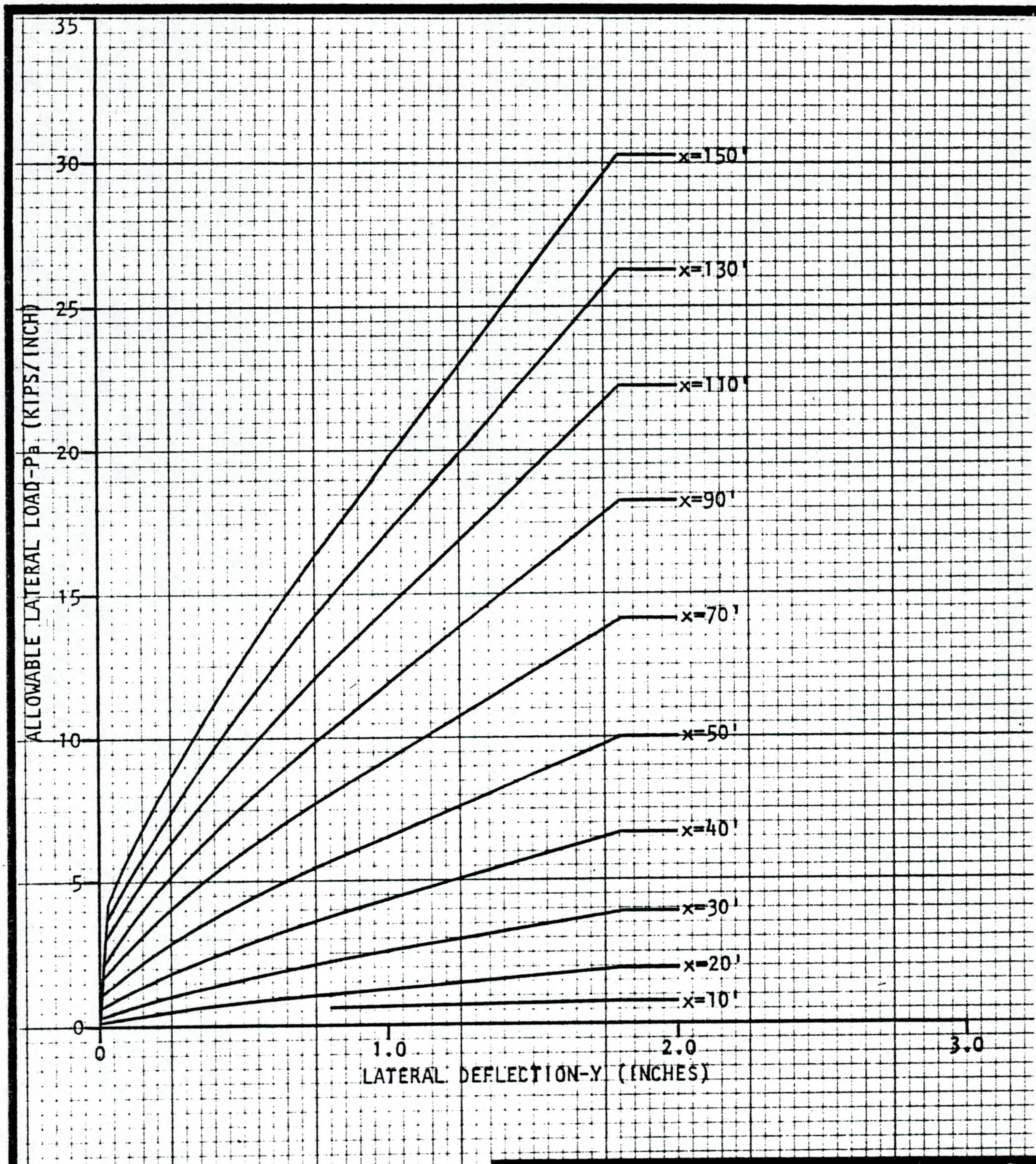
Plate 6



PREPARED FOR **Boyle Engineering Corp.**

ALLOWABLE VERTICAL LOAD DRILLED CAISSONS

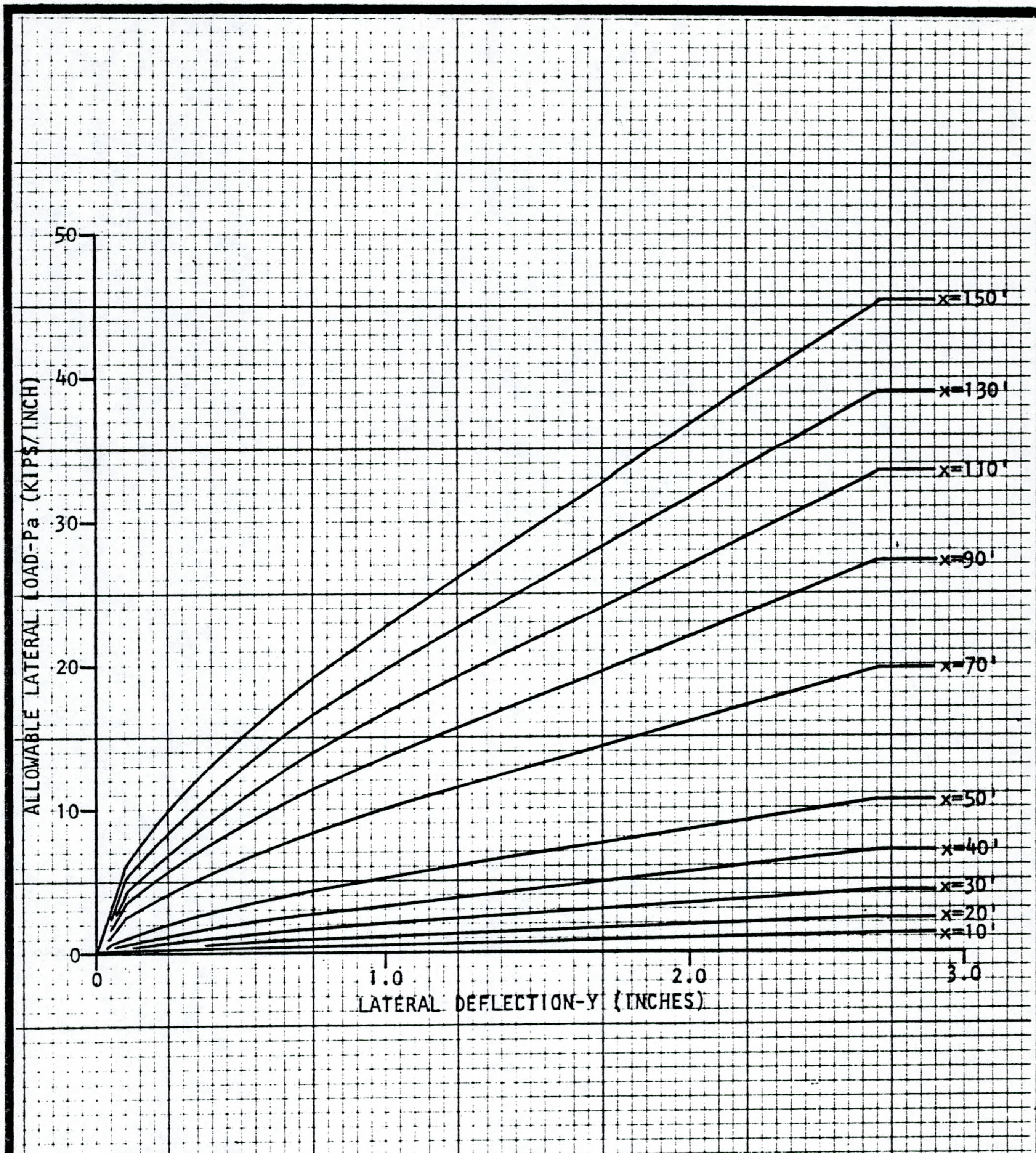
BY **Dames & Moore** Plate 7



PREPARED FOR **Boyle Engineering Corp.**

LATERAL LOAD-DEFLECTION 4 FOOT DIAMETER DRILLED CAISSON

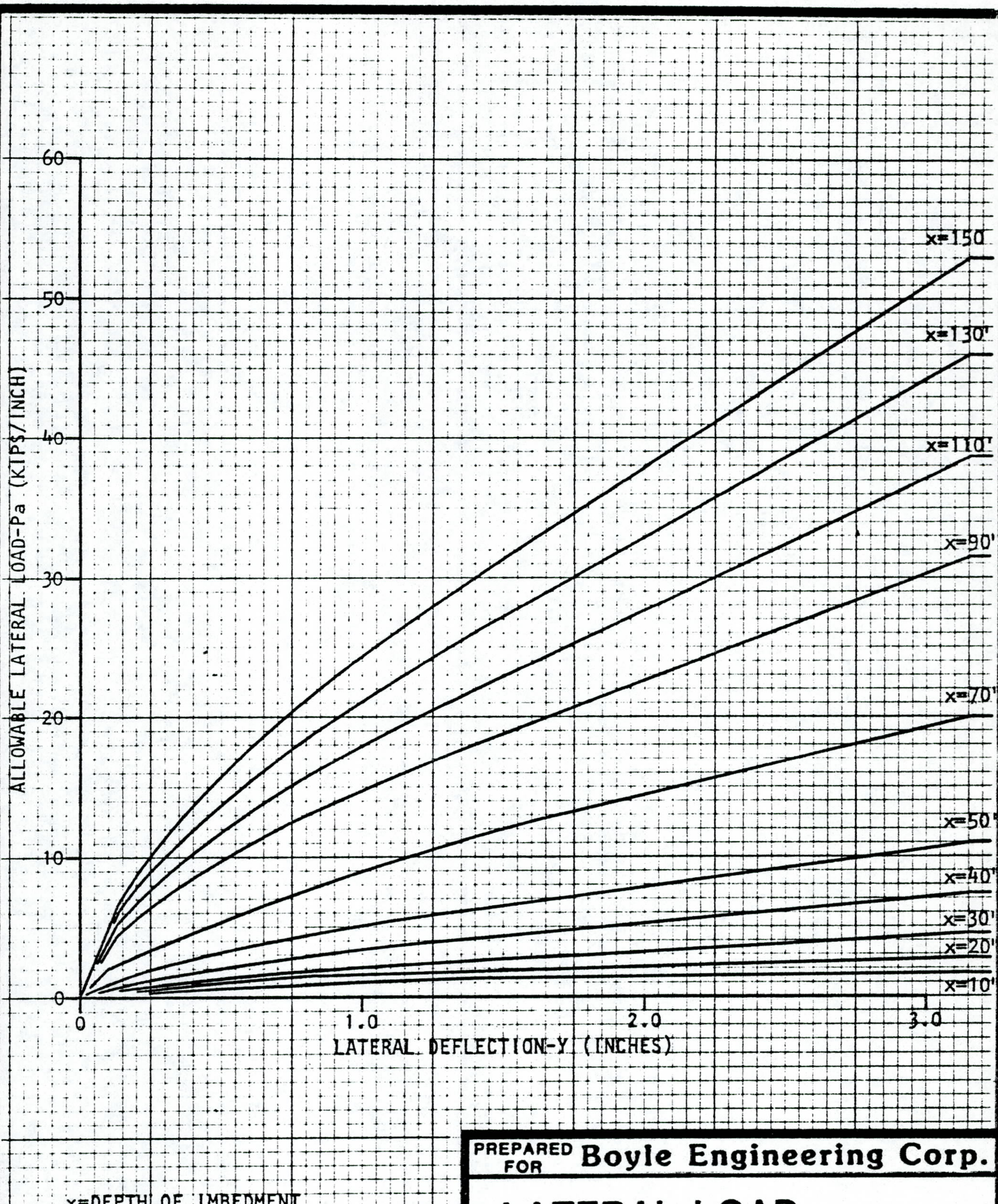
BY **Dames & Moore** Plate 8



PREPARED FOR **Boyle Engineering Corp.**

LATERAL LOAD-DEFLECTION 6 FOOT DIAMETER DRILLED CAISSON

BY **Dames & Moore** Plate 9



PREPARED FOR **Boyle Engineering Corp.**

LATERAL LOAD-DEFLECTION 7 FOOT DIAMETER DRILLED CAISSON

BY **Dames & Moore** Plate 10

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U.S. Geological Survey, "National Stream Quality Accounting Network," February, 1980.

APPENDIX

APPENDIX

FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION

The subsurface conditions were explored by drilling five borings to depths ranging from 69 to 78 feet. The borings were drilled with a Becker Hammer Drill, a specialized drill rig which is particularly efficient for drilling through materials containing cobbles and boulders. Soil samples recovered during the drilling were classified by our field geologist at the time of drilling the borings from inspection of the samples obtained. In addition, the number of blows per foot of advancement of the driven casing were noted and recorded. The soil samples were classified in accordance with the Unified Soil Classification System. The samples were packed and sealed in sample containers and were labeled for identification.

The Log of Borings is presented on Plates A-1A through A-1E. The Key to Log of Borings is presented on Plate A-2, and the Unified Soil Classification System is presented in summary form on Plate A-3.

LABORATORY TESTING

The laboratory testing program included direct shear tests, a compaction test, grain size distribution analyses, and chemical tests to evaluate the pH and soluble sulfates in two samples.

Direct Shear Tests

The method of performing the direct shear tests is described on Plate A-4. The results of the shear tests are presented on the Log of Borings and summarized below in Table A-1, Direct Shear Results.

TABLE A-1

DIRECT SHEAR RESULTS

Sample	Dry Density (pcf)	Moisture Content (%)	Percent Compaction (%)	Normal Pressure (psf)	Peak Shear Strength (psf)
1[a]	112	7.7	81	1,000	840
1[a]	116	7.7	84	1,000	950
1[a]	111	7.6	80	3,000	1,830
1[a]	116	7.6	84	3,000	2,140
1[a]	110	7.4	80	5,000	3,010
1[a]	116	7.8	83	5,000	3,290

[a]Bulk surface sample near Boring #2.

Compaction Test

The method of performing the compaction test is described on Plate A-5. The results of the test on bulk Sample No. 1 is presented on Plate A-6, Compaction Test Data.

Grain Size Analysis

The grain size analysis was performed in accordance with ASTM Standard Test Procedure D-421 and D-422. Mechanical analyses were performed because of the trace amounts of the sample which are finer than the 200 mesh sieve. The grain size curve is presented on Plate A-7.

Chemical Tests

Soluble sulfate and pH tests were performed in accordance with standard test procedures by Arizona Testing Laboratories in Phoenix, Arizona. The results are presented on Plate A-8. These test results were used in the recommendations for the appropriate cement type in those elements of the proposed bridge that will be in contact with the soils and the water.

* * *

The following plates are attached and complete this Appendix:

Plates A-1A through A-1E	-	Log of Borings
Plate A-2	-	Key to Log of Borings
Plate A-3	-	Unified Soil Classification System
Plate A-4	-	Method of Performing Direct Shear Tests
Plate A-5	-	Method of Performing Compaction Tests
Plate A-6	-	Compaction Test Data
Plate A-7	-	Grain Size Distribution
Plate A-8	-	Chemical Test Results

BORING 1

SURFACE ELEVATION: 1157 FEET

	LABORATORY TEST DATA												
		ATTERBERG LIMITS		STRENGTH TEST DATA									
DEPTH IN FEET	TESTS REPORTED ELSEWHERE	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	BLOWS/FT.	SAMPLES	SYMBOLS	DESCRIPTION
0										53 59/3'8"		GP/SP	TAN TO LIGHT BROWN VERY FINE TO COARSE SAND; WITH SOME GRAVEL AND COBBLES, AND TRACE OF SILT
5										78 90 76			
10											■	SP	TAN TO LIGHT BROWN, FINE TO MEDIUM SAND; SOME GRAVEL: SAND GENERALLY ANGULAR/SUBANGULAR
15										22 78 80 82		GP/SP	LIGHT BROWN TO TAN, VERY FINE TO COARSE SANDY GRAVEL TO GRAVELLY SAND; GRAVEL ROUNDED TO WELL ROUNDED
20										70	■		
25										85 94 74 63 46 48			
30										34 53 61 40	■	GW	LIGHT TO MEDIUM BROWN, FINE TO COARSE SANDY GRAVEL WITH SOME COBBLES; GRAVEL ROUNDED TO WELL ROUNDED (MOIST)
35										27 40 59 34/9" 51		SP	LIGHT TO MEDIUM BROWN FINE TO MEDIUM SAND WITH TRACE OF GRAVEL
40										65 88 98 64 76			
45										130 104 86		GW	LIGHT TO MEDIUM BROWN FINE TO COARSE GRAVEL WITH SOME COBBLES AND SAND, GRAVEL ROUNDED TO WELL ROUNDED
50										96	■		
55										78 100 90 120			GRADING SLIGHTLY SANDIER SAND AT 55 FEET
60											■		
65										140 160 92			
70										110 82 174			TAN, SANDY GRAVEL WITH SOME COBBLES
75													BORING COMPLETED AT 75 FEET ON 12/29/80. GROUNDWATER TABLE NOT ENCOUNTERED.
80													

LOG OF BORINGS

DAMES & MOORE

BORING 2

SURFACE ELEVATION: 1150 FEET

LABORATORY TEST DATA

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
0									
5									
10									
15									
20									
25									
30									
35									
40									
45									
50									
55									
60									
65									
70									
75									
80									

BLOWS/FT.: 60, 41, 36, 28, 31, 35, 42, 47, 44, 59, 52, 50, 54, 51, 59, 78, 60, 126, 82, 55, 70, 75, 94, 82, 84, 78, 80, 75, 80, 93, 68, 90, 106, 77, 93, 97, 79, 107, 97, 88, 76, 110, 88, 86, 83, 81, 101, 80, 87, 92, 74, 100, 100, 111, 138, 82, 67, 108, 99, 70, 82, 89, 100, 122, 127, 110

SAMPLES

SYMBOLS	DESCRIPTION
SP	LIGHT BROWN FINE TO COARSE SAND WITH TRACE OF GRAVEL
GP/SP	LIGHT BROWN GRAVELLY FINE TO COARSE SAND TO SANDY GRAVEL WITH TRACE OF COBBLES GRAVEL ROUNDED TO WELL ROUNDED
	TAN VERY FINE TO COARSE SAND, TRACE SILT
	INCREASING GRAVEL AT 40 FEET (MOIST)
	SLIGHTLY SILTY AT 50 FEET
SP	LIGHT BROWN TO BROWN FINE TO COARSE SAND WITH SOME GRAVEL

BORING COMPLETED AT 78 FEET ON 12/31/80.
GROUNDWATER TABLE NOT ENCOUNTERED.

LOG OF BORINGS

DAMES & MOORE

BORING 3

SURFACE ELEVATION: 1159 FEET

LABORATORY TEST DATA

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
0									
5									
10									
15									
20									
25									
30									
35									
40									
45									
50									
55									
60									
65									
70									
75									
80									

BLOWS/FT.: 19, 23, 32, 78, 65, 75, 125, 105, 68, 58, 42, 43, 52, 73, 83, 334, 101, 93, 98, 98, 85, 85, 99, 88, 127, 138, 101, 88, 85, 74, 45, 29, 41, 59, 118, 87, 67, 77, 116, 115, 123, 137, 121, 107, 103, 113, 67, 43, 39, 89, 115, 103, 112

SAMPLES ■ (at approximately 10, 20, 30, 41, 46, 50 and 62 feet)

SYMBOLS / **DESCRIPTION**

GP/SP — TAN TO LIGHT BROWN, FINE TO COARSE SANDY GRAVEL TO GRAVELLY SAND WITH SOME COBBLES; GRAVEL ROUNDED TO WELL ROUNDED

INCREASING GRAVEL BELOW 10 FEET

MOIST AT 28 FEET

INCREASING SAND AT 30 FEET

INCREASING GRAVEL AT 42 FEET

DECREASING GRAVEL BELOW 44 FEET

SLIGHTLY CEMENTED BETWEEN 46 FEET AND 49 FEET WITH TRACE OF SILT AND CLAY

FREE WATER AT 54 FEET

MOIST AT 56 FEET

DRY AT 57.5 FEET

BORING COMPLETED AT 69 FEET ON 12/30/80.
GROUNDWATER TABLE NOT ENCOUNTERED.

LOG OF BORINGS

DAMES & MOORE

BORING 4

SURFACE ELEVATION: 1164 FEET

LABORATORY TEST DATA

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
0									
5									
10									
15									
20									
25									
30									
35									
40									
45									
50									
55									
60									
65									
70									
75									
80									

BLOWS/FT.

10, 24, 26, 34, 26, 45, 47, 50, 33, 35, 30, 19, 25, 27, 35, 38, 48, 39, 43, 81, 69, 47, 50, 132, 133, 91, 72, 57, 122, 126, 116, 97, 108, 112, 112, 159, 159, 130, 127, 110, 93, 108, 72, 88, 115, 104, 91, 154, 128, 135, 105, 146, 105, 121, 173, 135, 193, 144, 174, 140, 194, 241, 169, 224, 165, 125, 88, 114, 96, 71

SAMPLES · **SYMBOLS** · **DESCRIPTION**

GP/SP — TAN TO LIGHT BROWN, VERY FINE TO COARSE SANDY GRAVEL TO GRAVELLY SAND WITH TRACE OF COBBLES

GP — LIGHT BROWN TO MEDIUM BROWN, VERY FINE TO COARSE SANDY GRAVEL

SLIGHTLY MOIST AT 26 FEET

BORING COMPLETED AT 75 FEET ON 12/30/80.
GROUNDWATER TABLE NOT ENCOUNTERED.

LOG OF BORINGS

DAMES & MOORE

BORING 5

SURFACE ELEVATION: 1166 FEET

LABORATORY TEST DATA

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
0									
5									
10									
15									
20									
25									
30									
35									
40									
45									
50									
55									
60									
65									
70									
75									
80									

BLOWS/FT.: 17, 16, 19, 25, 29, 38, 35, 23, 45, 62, 60, 33+, 121, 63, 87, 114, 120, 89, 72, 45, 163, 175, 164, 114, 36, 25, 38, 49+, 124, 101, 71, 60, 64, 68, 83, 76, 78, 108, 106, 144, 88, 95, 134, 112, 110, 123, 144, 176, 204, 174, 80, 74, 185, 239, 235, 110, 134, 180, 128, 121, 106, 100, 92, 134, 94, 85, 110, 117, 115

SAMPLES: at approximately 10, 20, 30, 40 and 50 feet

DEPTH (FEET)	SYMBOLS	DESCRIPTION
0–10	SW	TAN TO LIGHT BROWN, VERY FINE TO MEDIUM SAND WITH SOME GRAVEL AND TRACE OF SILT AND COBBLES; GRAVEL ROUNDED TO WELL ROUNDED INCREASING GRAVEL BELOW 5 FEET
10–20	GP	BROWN, FINE TO COARSE GRAVEL WITH SOME SAND AND TRACE OF SILT; GRAVEL ROUNDED TO WELL ROUNDED SLIGHTLY MOIST AT 15 FEET
20–30	SP	LIGHT TO MEDIUM BROWN, VERY FINE TO MEDIUM SAND WITH SOME GRAVEL AND A TRACE OF SILT (MOIST)
30–72	SP/GP	LIGHT BROWN TO BROWN GRAVELLY FINE TO MEDIUM SAND TO SANDY GRAVEL WITH A TRACE OF SILT AND COBBLES; GRAVEL ROUNDED TO WELL ROUNDED INCREASING FINE TO MEDIUM SAND AT 40 FEET INCREASING FINE GRAVEL AT 50 FEET

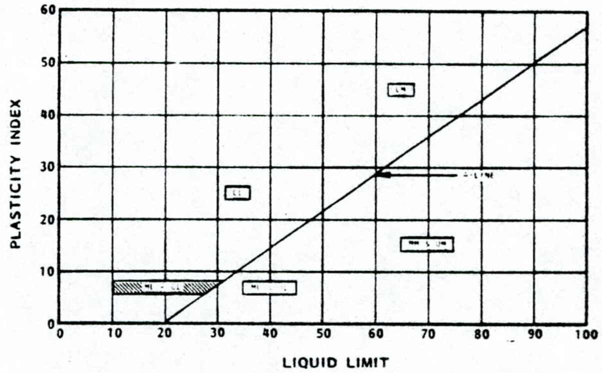
BORING COMPLETED AT 72 FEET ON 12/31/80.
GROUNDWATER TABLE NOT ENCOUNTERED.

LOG OF BORINGS

DAMES & MOORE

SYMBOL	TYPE OF TEST
M	MOISTURE
QD	QUICK MD TEST BASED ON ASSUMED SPECIFIC GRAVITY
MD	MOISTURE-DENSITY
CD	CHUNK DENSITY ON BULK SAMPLE
RD	RELATIVE DENSITY
COMP	COMPACTION CURVE
CI	CALIFORNIA IMPACT
CC	COMPACTED CORE
G	SPECIFIC GRAVITY
pH	HYDROGEN ION CONCENTRATION
MA (10%)	MECHANICAL ANALYSIS (INDICATES PERCENT PASSING #200 SIEVE)
SA (10%)	SIEVE ANALYSIS (+200 ONLY) (INDICATES PERCENT PASSING #200 SIEVE)
HA	HYDROMETER ANALYSIS (-200 ONLY)
AL	ATTERBERG LIMITS (LL & PL)
SL	SHRINKAGE LIMIT
FS	FREE SWELL
SS	SHRINK-SWELL
EXP	EXPANSION
C	CONSOLIDATION
VC	VIBRATING CONSOLIDATION
P	PERMEABILITY
FP	FIELD PERMEABILITY
UC	UNCONFINED COMPRESSION
	FRICTION TEST
F/UU	1. UNCONSOLIDATED-UNDRAINED
F/CU	2. CONSOLIDATED-UNDRAINED
F/CU/M*	3. CU/MULTIPHASE**
F/CU/PP	4. CU/WITH PORE PRESSURE MEASUREMENTS
F/CD	5. CONSOLIDATED-DRAINED
	DIRECT SHEAR TEST
DS/UU	1. UNCONSOLIDATED-UNDRAINED
DS/CU	2. CONSOLIDATED-UNDRAINED
DS/CD	3. CONSOLIDATED-DRAINED
DS/CD/M*	4. CD/MULTIPHASE**
LV	TORVANE SHEAR (LAB VANE SHEAR)

* INCLUDES COMPLETE ANALYSIS, SIEVING AND HYDROMETER
** SERIES OF TESTS RUN ON SAMPLE



PLASTICITY CHART

A - ACKER SOIL SAMPLER

D - DAMES & MOORE, TYPE D SAMPLER

P - DAMES & MOORE PISTON SAMPLER

U - DAMES & MOORE TYPE U SAMPLER

PT - PITCHER TUBE SAMPLER

NX - NX CORE SAMPLER

TW - DAMES & MOORE TYPE U SAMPLER WITH THIN WALL ATTACHMENT

SPT - STANDARD PENETRATION TEST SAMPLER

ST - SHELBY TUBE SAMPLER

KEY TO SAMPLERS

■ INDICATES DEPTH OF UNDISTURBED SAMPLE

⊠ INDICATES DEPTH OF DISTURBED SAMPLE

□ INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY

◪ INDICATES DEPTH OF STANDARD PENETRATION TEST

□ INDICATES DEPTH OF STANDARD PENETRATION TEST WITH NO RECOVERY

70% - 5% I INDICATES DEPTH AND LENGTH OF CORE RUN

RQD (ROCK QUALITY DETERMINATION) PERCENT OF THE TOTAL CORE RUN HAVING AN UNFRACTURED LENGTH OF 4" OR MORE

PERCENT OF CORE RUN RECOVERED

■ INDICATES DEPTH OF FIELD VANE SHEAR TEST

NOTE:
UNLESS OTHERWISE NOTED SAMPLING RESISTANCE IS MEASURED IN BLOWS PER FOOT REQUIRED TO DRIVE SAMPLER 12-INCHES AFTER SAMPLER HAS BEEN SEATED 6-INCHES. A 140-POUND HAMMER, FREE FALLING A DISTANCE OF 30 INCHES IS USED TO DRIVE THE SAMPLER.

KEY TO SAMPLES

KEY TO LOG OF BORINGS

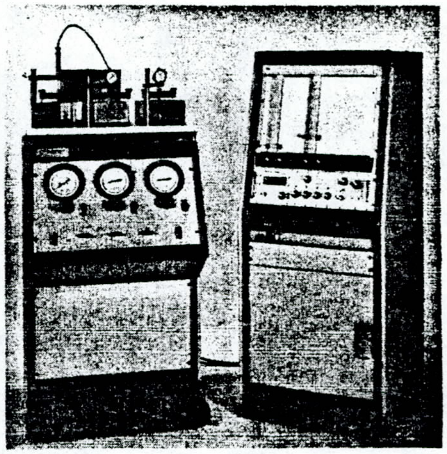
BY **Dames & Moore** Plate A-2

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

Method Of Performing Direct Shear and Friction Tests



DIRECT SHEAR APPARATUS WITH ELECTRONIC RECORDER

Direct shear tests are performed to determine the shearing strengths of soils. Friction tests are performed to determine the frictional resistances between soils and various other materials such as wood, steel, or concrete. The tests are performed in the laboratory to simulate anticipated field conditions.

Each sample is tested in a split sample holder, two and one-half inches in diameter and one inch high. Undisturbed samples of in-place soils are extruded from rings taken from the sampling device in which the samples were obtained. Loose samples of soils to be used in constructing earth fills are compacted in rings to predetermined conditions and tested.

Direct Shear Tests

A one-inch length of the sample is tested in direct single shear. A constant pressure, appropriate to the conditions of the problem for which the test is being performed, is applied normal to the ends of the sample through porous stones. A shearing failure of the sample is caused by moving the upper sample holder in a direction perpendicular to the axis of the sample. Transverse movement of the lower sample holder is prevented.

The shearing failure is accomplished by applying to the upper sample holder a constant rate of deflection. The shearing load and the deflections in both the axial and transverse directions are recorded and plotted. The shearing strength of the soils is determined from the resulting load-deflection curves.

Friction Tests

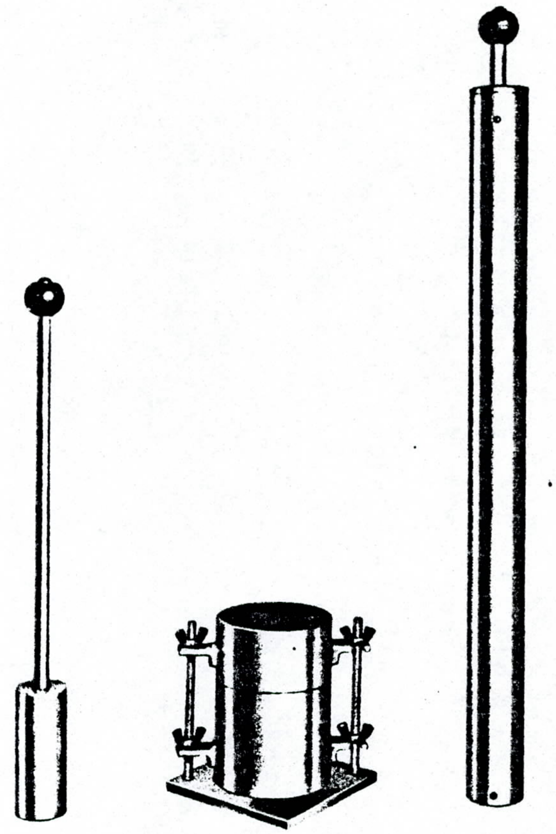
In order to determine the frictional resistance between soil and the surfaces of various materials, the lower sample holder in the direct shear test is replaced by a disk of the material to be tested. The test is then performed in the same manner as the direct shear test by forcing the soil over the friction material surface.

METHOD OF PERFORMING COMPACTION TESTS

(STANDARD AND MODIFIED A.A.S.H.O. METHODS)

IT HAS BEEN ESTABLISHED THAT WHEN COMPACTING EFFORT IS HELD CONSTANT, THE DENSITY OF A ROLLED EARTH FILL INCREASES WITH ADDED MOISTURE UNTIL A MAXIMUM DRY DENSITY IS OBTAINED AT A MOISTURE CONTENT TERMED THE "OPTIMUM MOISTURE CONTENT," AFTER WHICH THE DRY DENSITY DECREASES. THE COMPACTION CURVE SHOWING THE RELATIONSHIP BETWEEN DENSITY AND MOISTURE CONTENT FOR A SPECIFIC COMPACTING EFFORT IS DETERMINED BY EXPERIMENTAL METHODS. TWO COMMONLY USED METHODS ARE DESCRIBED IN THE FOLLOWING PARAGRAPHS.

FOR THE "STANDARD A.A.S.H.O." (A.S.T.M. D698-66T & A.A.S.H.O. T99-61) METHOD OF COMPACTION A PORTION OF THE SOIL SAMPLE PASSING THE NO. 4 SIEVE IS COMPACTED AT A SPECIFIC MOISTURE CONTENT IN THREE EQUAL LAYERS IN A STANDARD COMPACTION CYLINDER HAVING A VOLUME OF 1/30 CUBIC FOOT, USING TWENTY-FIVE 12-INCH BLOWS OF A STANDARD 5-1/2 POUND RAMMER TO COMPACT EACH LAYER.



SOME APPARATUS FOR PERFORMING COMPACTION TESTS
Shows, from left to right, 5-1/2 pound rammer (sleeve controlling 12" height of drop removed), 1/30 cubic-foot cylinder with removable collar and base plate, and 10 pound rammer within sleeve.

IN THE "MODIFIED A.A.S.H.O." (A.S.T.M. D-1557-66T & A.A.S.H.O. T 180-61) METHOD OF COMPACTION A PORTION OF THE SOIL SAMPLE PASSING THE NO. 4 SIEVE IS COMPACTED AT A SPECIFIC MOISTURE CONTENT IN FIVE EQUAL LAYERS IN A STANDARD COMPACTION CYLINDER HAVING A VOLUME OF 1/30 CUBIC FOOT, USING TWENTY-FIVE 18-INCH BLOWS OF A 10-POUND RAMMER TO COMPACT EACH LAYER. SEVERAL VARIATIONS OF THESE COMPACTION TESTING METHODS ARE OFTEN USED AND THESE ARE DESCRIBED IN A.A.S.H.O. & A.S.T.M. SPECIFICATIONS.

FOR BOTH METHODS, THE WET DENSITY OF THE COMPACTED SAMPLE IS DETERMINED BY WEIGHING THE KNOWN VOLUME OF SOIL; THE MOISTURE CONTENT, BY MEASURING THE LOSS OF WEIGHT OF A PORTION OF THE SAMPLE WHEN OVEN DRIED; AND THE DRY DENSITY, BY COMPUTING IT FROM THE WET DENSITY AND MOISTURE CONTENT. A SERIES OF SUCH COMPACTIONS IS PERFORMED AT INCREASING MOISTURE CONTENTS UNTIL A SUFFICIENT NUMBER OF POINTS DEFINING THE MOISTURE-DENSITY RELATIONSHIP HAVE BEEN OBTAINED TO PERMIT THE PLOTTING OF THE COMPACTION CURVE. THE MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT FOR THE PARTICULAR COMPACTING EFFORT ARE DETERMINED FROM THE COMPACTION CURVE.

SAMPLE BULK SAMPLE 1 DEPTH SURFACE

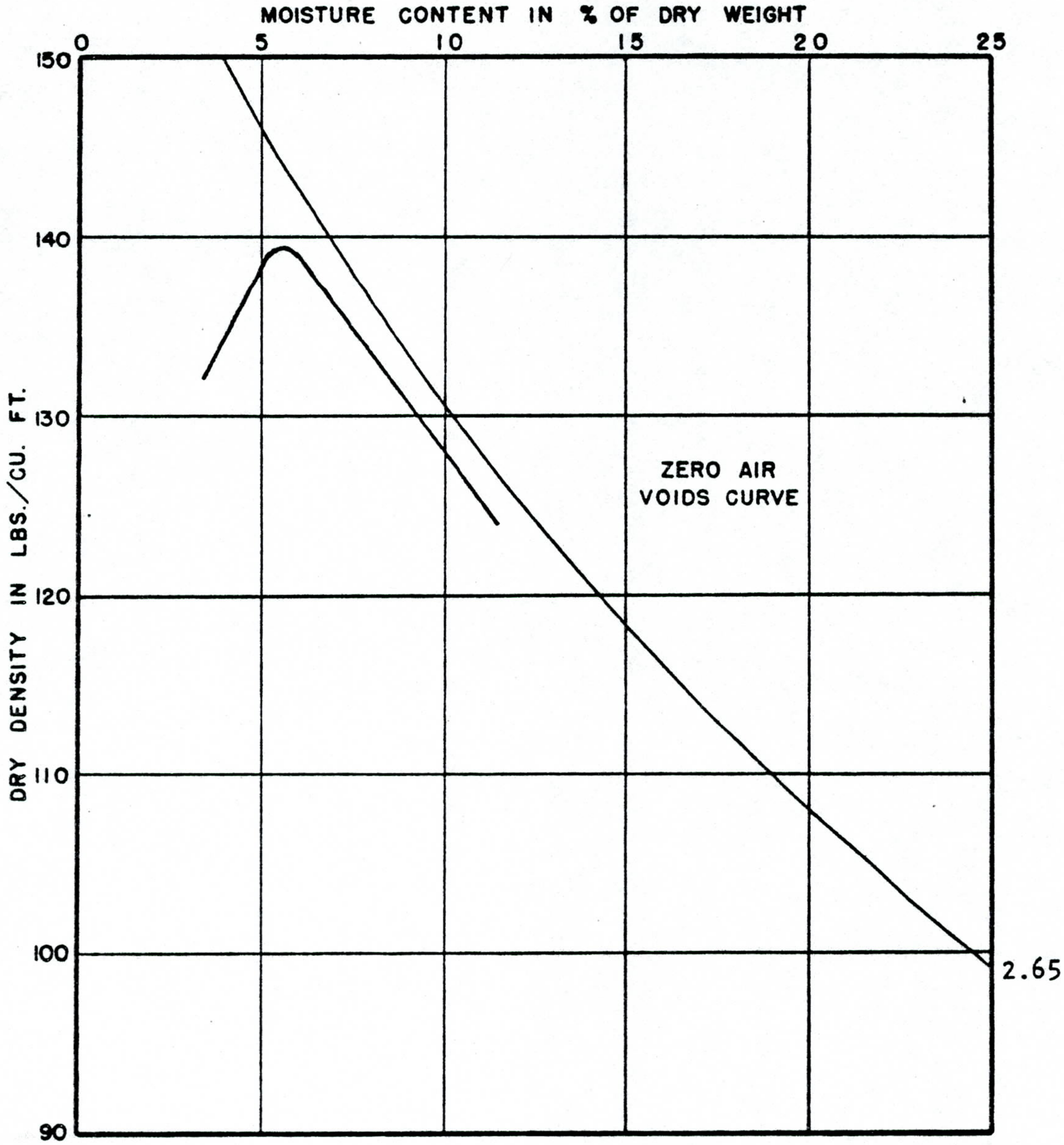
SOIL BROWN GRAVELLY FINE TO COARSE SAND-SP

LOCATION HAYDEN BRIDGE, AZ.

OPTIMUM MOISTURE CONTENT 5.5%

MAXIMUM DRY DENSITY 139 PCF

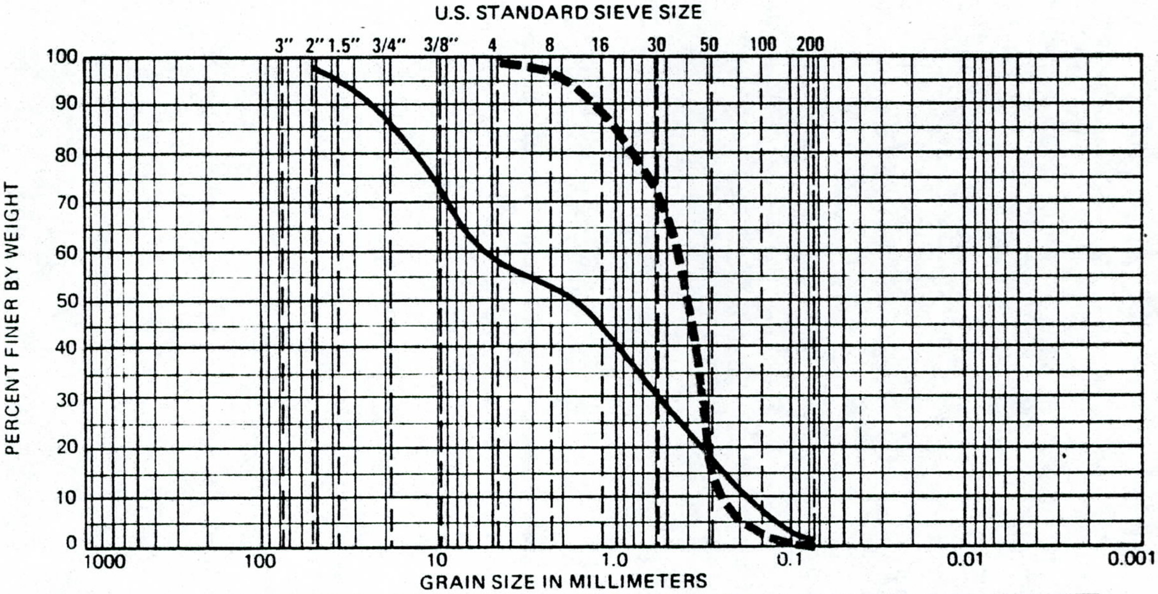
METHOD OF COMPACTION ASTM(D1557-78)(METHOD D)



PREPARED FOR Boyle Engineering Corp.

COMPACTION TEST DATA

BY Dames & Moore Plate A-6



SAMPLE	DEPTH (FT.)	SOIL CLASSIFICATION	SYMBOL	KEY
BULK SAMPLE 1	SURFACE	BROWN GRAVELLY FINE TO COARSE SAND	SP	———
BULK SAMPLE 2	SURFACE	LIGHT TO DARK GRAY FINE TO COARSE SAND	SP	- - - -

PREPARED FOR **Boyle Engineering Corp.**

GRAIN-SIZE DISTRIBUTION

BY **Dames & Moore** Plate A-7

Arizona Testing Laboratories

817 West Madison · Phoenix, Arizona 85007 · Telephone 254-6181

For: Dames & Moore
234 North Central, Suite 111A
Phoenix, Arizona 85004
Attn: Mr. Thomas Lee

Date: February 18, 1981

Lab. No.: 0133

Sample: Soil - Bulk sample

Marked: City of Tempe
12269-001

Received: 2-17-81

Submitted by: Same

REPORT OF LABORATORY TESTS

Sample No.	Soluble Sulfate	pH
#1 SW	380 ppm	8.1
#2 SP	25 ppm	8.0

Respectfully submitted,

ARIZONA TESTING LABORATORIES

Claude E. McLean Jr.

Claude E. McLean, Jr.

PREPARED FOR **Boyle Engineering Corp.**

CHEMICAL TEST RESULTS

BY **Dames & Moore** Plate A-8