Glendale Avenue and
16th Street Bridges
Over Proposed ACDC Canal
Maricopa County, Arizona
FCD Contract 88-36
for
Entranco-Mann-Johnson
Job No. 2128J186
Revision No. 1

WESTERN
TECHNOLOGIES
INC.
The Quality People

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16th Street Bridges
Over Proposed ACDC Canal
Maricopa County, Arizona
FCD Contract 88-36
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RECEIVED
MAR 7 1989
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Entranco-Mann-Johnson
8805 North 23rd Avenue, Suite 9
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March 6, 1989

Attn: Mr. Jack Gonsalves

Re: Glendale Avenue and 16th Street Bridges
Over Proposed ACDC Canal
Maricopa County, Arizona
FCD Contract 88-36

Job No. 2128J186
Revision No. 1

Gentlemen:

Western Technologies Inc. has completed the subsurface exploration for the proposed bridges to be located in Maricopa County, Arizona. This study was requested and authorized by Bill Kantor in general accordance with our proposal number 2128A164 dated October 3, 1988. This revision should be used to replace our original report dated January 17, 1989.

The results of our study, including the boring location diagram, laboratory test results, test boring records, and the geotechnical recommendations needed to aid in the design and construction of foundations for this project are attached.

We have appreciated being of service to you in the subsurface exploration phase of this project and are prepared to assist you during the construction phases as well. If you have any questions concerning this report or any of our testing, inspection, design and consulting services, please do not hesitate to contact us.

Sincerely,
WESTERN TECHNOLOGIES INC.
Geotechnical Services

Frank Costello, P.E.

Enamul Hoque JPA

Reviewed by: Enamul Hoque, P.E.

sfp/7990G

Copies to: Addressee (5)

Entranco-Mann-Johnson
Job No. 2128J186

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Entranco-Mann-Johnson
Job No. 2128J186

Geotechnical Exploration
Glendale Avenue and 16th Street Bridges
Over Proposed ACDC Canal
Maricopa County, Arizona
FCD Contract 88-36
Revision No. 1
March 6, 1989

INTRODUCTION

This report contains the results of our geotechnical exploration for the proposed bridges to be located on Glendale Avenue and 16th Street where the ACDC Canal will be constructed. The purpose of these services is to provide information and recommendations relative to foundation design, lateral earth pressures, and site preparation.

PROPOSED CONSTRUCTION

Based on information from Entranco-Mann-Johnson, we understand that the bridges will be constructed prior to canal excavation. Drilled pier foundations will be used for support of the precast concrete bridges. The canal invert will be approximately 30 feet below the bridge decks, and the concrete sides of the canal will be constructed along exposed piers. The bridge deck will be designed to provide lateral support at the pier head without transferring moment to the piers. Some piers may be used for wingwall support.

Piers for bridge support will have either 2.5-foot or 3.0-foot diameters and be constructed on 6-foot or 8-foot center-to-center spacings. Three-foot diameter piers with 8-foot center-to-center spacing may be used for wingwall support. Total axial loads on the 2.5-foot and 3.0-foot diameter bridge piers are understood to be 144 and 181 kips, respectively.

All pier types will have a modulus of elasticity of 4.03×10^6 psi. The 2.5-foot and 3.0-foot diameter bridge piers will use 4 percent steel and have moments of inertia of 5.16×10^4 and 1.02×10^5 in.^{4}, respectively. The 3-foot diameter wingwall piers will use 8 percent steel and have a moment of inertia of 1.30×10^5 in.^{4}.

SITE CONDITIONS

At the time of our exploration, the sites were arterial roadway right-of-ways. The ground surface was relatively flat and paved or graded. Site drainage was controlled by street and right-of-way grades. Underground utilities exist in the right-of-way.

SCOPE OF SERVICES

Four borings were drilled with an auger drill rig to refusal at depths ranging from 3 to 14 feet. Subsequently, one boring at each bridge site was advanced with a Becker-type drill rig. At Glendale Avenue, the Becker-type boring was advanced to a depth of 65 feet, and at 16th Street, the Becker-type boring was advanced to refusal at a depth of 36-1/2 feet. The boring locations are shown on the site plans. During exploration, subsoils were examined visually and sampled at selected intervals. The samples obtained during exploration were remolded for laboratory direct shear tests.

INTERPRETATION OF SUBSURFACE CONDITIONS

Exploration: As presented on Logs of Borings, the predominant surface and subsoils to the full depth of exploration at each site were found to be dense to very dense sandy gravels and gravelly sand of low plasticity. Light to moderate calcite cementation with

intermittent zones of heavy cementation was encountered to the full depth of exploration. A groundwater table was not encountered in any boring at the time of exploration. Probable bedrock was encountered at a depth of approximately 36 feet at the 16th Street site.

Testing: Four direct shear tests were performed on remolded grab samples obtained from the Becker-type borings. Peak friction angles for samples obtained from depths of 10 to 15 feet, 19 to 29 feet, 20 to 25 feet, and 31 to 35 feet were 35, 38, 38, and 42 degrees, respectively. Apparent cohesion values for the samples were 300, 300, 300, and 0 psf, respectively.

CONCLUSIONS AND RECOMMENDATIONS

General: The recommendations presented in this report are based on the assumption that the soil conditions do not deviate appreciably from those disclosed by the borings. If variations are encountered during construction, or if changes are made in site plans, structural loadings, foundation type, or bridge levels, we should be notified for supplemental recommendations.

Deep Foundations: The following criteria should be used in design for machine-cleaned, straight-shaft drilled piers:

- end bearing only for proposed 2.5-foot and 3.0-foot diameter piers
- bearing capacity of 30,000 psf with a factor of safety of 2.5, or a bearing capacity of 25,000 psf with a factor of safety of 3.0
- bearing on very dense sandy gravels, gravelly sands, or bedrock at least ten feet below the proposed canal invert approximately 38 to 40 feet below existing grade

- wingwall piers should be extended to a depth of at least 20 feet below the canal invert to provide lateral support for their unrestrained condition
- bearing capacity may be increased one-third when considering total loads including wind and seismic

Estimated settlements including creep for drilled piers are 1/2 inch or less for maximum loads of 181 and 144 kips for pier diameters of 3.0 and 2.5 feet, respectively. Maximum lateral deflections of either 2.5-foot or 3.0-foot diameter bridge piers are expected to be minimal. Lateral deflection of the unrestrained wingwall piers may approach 3-1/2 inches at the top. Deflection, moment, shear, and soil response data for each pier option are presented in tabular and graphical forms in the "Computer Analysis" section of the Appendix. Adequate soil arching to prevent excessive slope failure between the piers during canal excavation is anticipated with either 6-foot or 8-foot center-to-center spacing.

The bearing surface must be cleaned prior to concrete placement. A representative of the geotechnical engineer should inspect the bearing surface and pier configuration.

Lateral Earth Pressures: For soils above any free water surface, recommended equivalent fluid pressures and coefficients of base friction for unrestrained elements are:

- Active:
  Undisturbed subsoil ---------- 30 psf/ft
  Compacted granular backfill ---------- 30 psf/ft
- Passive:
  Drilled Piers ---------- 750 psf/ft
- Coefficient of base friction ---------- 0.40*

*The coefficient of base friction should be reduced to 0.30 when used in conjunction with passive pressure.

Where the design includes restrained elements, the following equivalent fluid pressures are recommended:

- At-rest:
  Undisturbed subsoil ---------- 50 psf/ft
  Compacted granular backfill ---------- 55 psf/ft

Seismic: The proposed bridge sites in Phoenix, Arizona have a Zone 2 seismic risk classification. This classification places the sites in an area of moderate damage. For structural analysis, a maximum acceleration of 0.08g and a period of 1.0 to 1.2 seconds are recommended.

Earthwork:

- General: The conclusions contained in this report for the proposed construction are contingent upon compliance with recommendations presented in this section.

- Site Clearing: Strip and remove existing pavement, debris, and other deleterious materials from the bridge areas. All exposed surfaces should be free of mounds and depressions which could prevent uniform compaction.

- Excavation:

  1. We anticipate that shallow excavations (0 to 5 feet) for the proposed construction can be accomplished with conventional equipment.

  2. Deeper excavations are expected to require at least heavy-duty equipment and may require specialized equipment and/or procedures.

- Materials:

1. Clean on-site soils placed at optimum moisture conditions or imported materials may be used as miscellaneous backfill material

2. Imported soils should conform to the following:

- Gradation (ASTM C136): percent finer by weight

6"	100
4"	70-100
No. 4 Sieve	50-100
No. 200 Sieve	50 (max)

- Maximum expansive potential(%)* 1.5

- Maximum soluble sulfates(%) 0.10

*Measured on a sample compacted to approximately 95 percent of the ASTM D698 maximum dry density at about 3 percent below optimum water content. The sample is confined under a 100 psf surcharge and submerged.

3. Aggregate base should conform to Maricopa Association of Government (MAG) specifications.

- Placement and Compaction:

1. Place and compact fill in horizontal lifts, using equipment and procedures that will produce recommended water contents and densities throughout the lift.

2. Uncompacted fill lifts should not exceed 10 inches.

3. Materials should be compacted to the following:

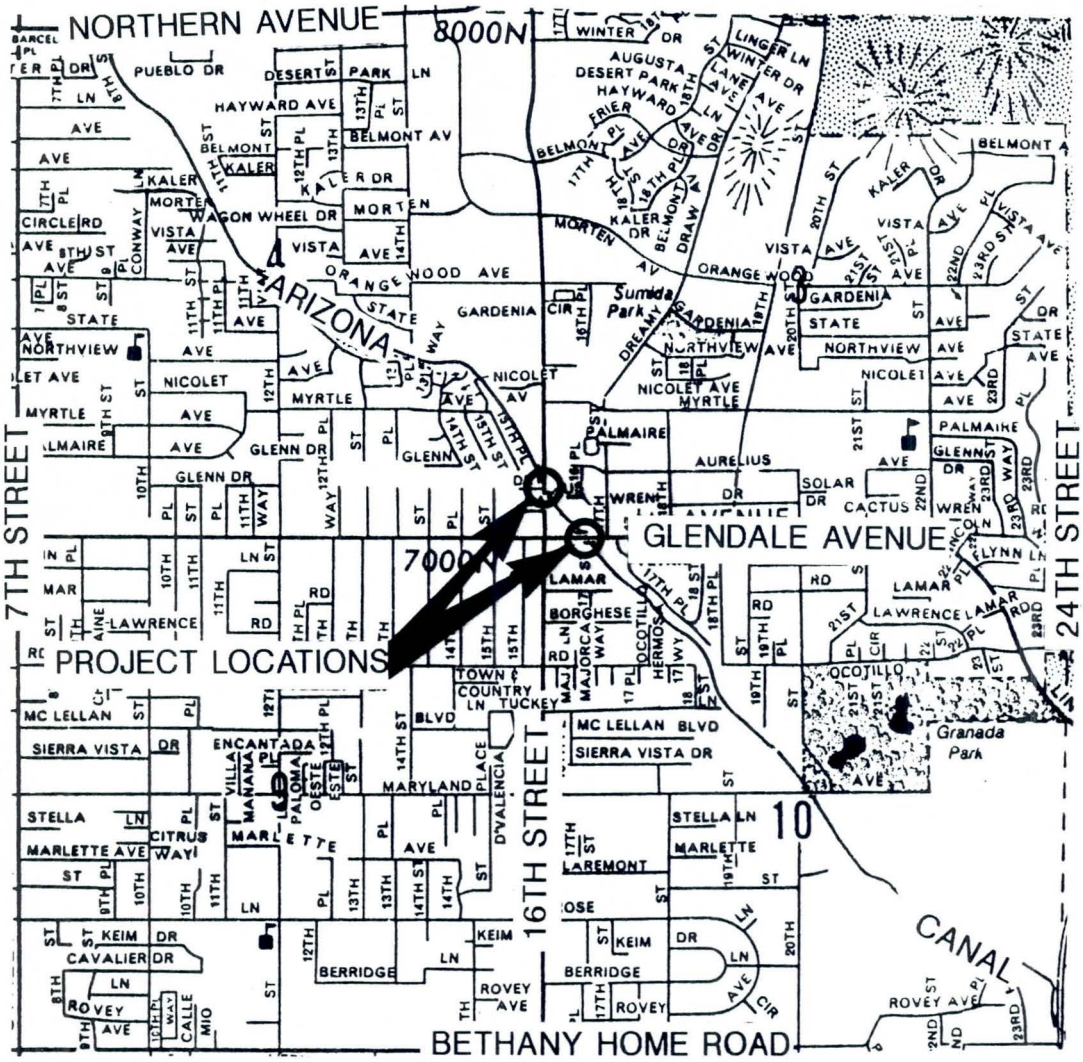
Material	Minimum Percent Compaction (ASTM D698)
On-site soils, reworked and fill	95
Imported fill	95
Aggregate base	95

4. On-site and imported soils should be compacted within a moisture range of 3 percent below to 3 percent above optimum.

CLOSURE

Our conclusions and recommendations are based on observation and testing of the earthwork and foundation preparations directed by the geotechnical engineer. If plans, specifications, or field applications deviate from our recommendations, we shall be relieved of responsibility unless our written concurrence has been obtained.

16th Street & Glendale Avenue Bridges

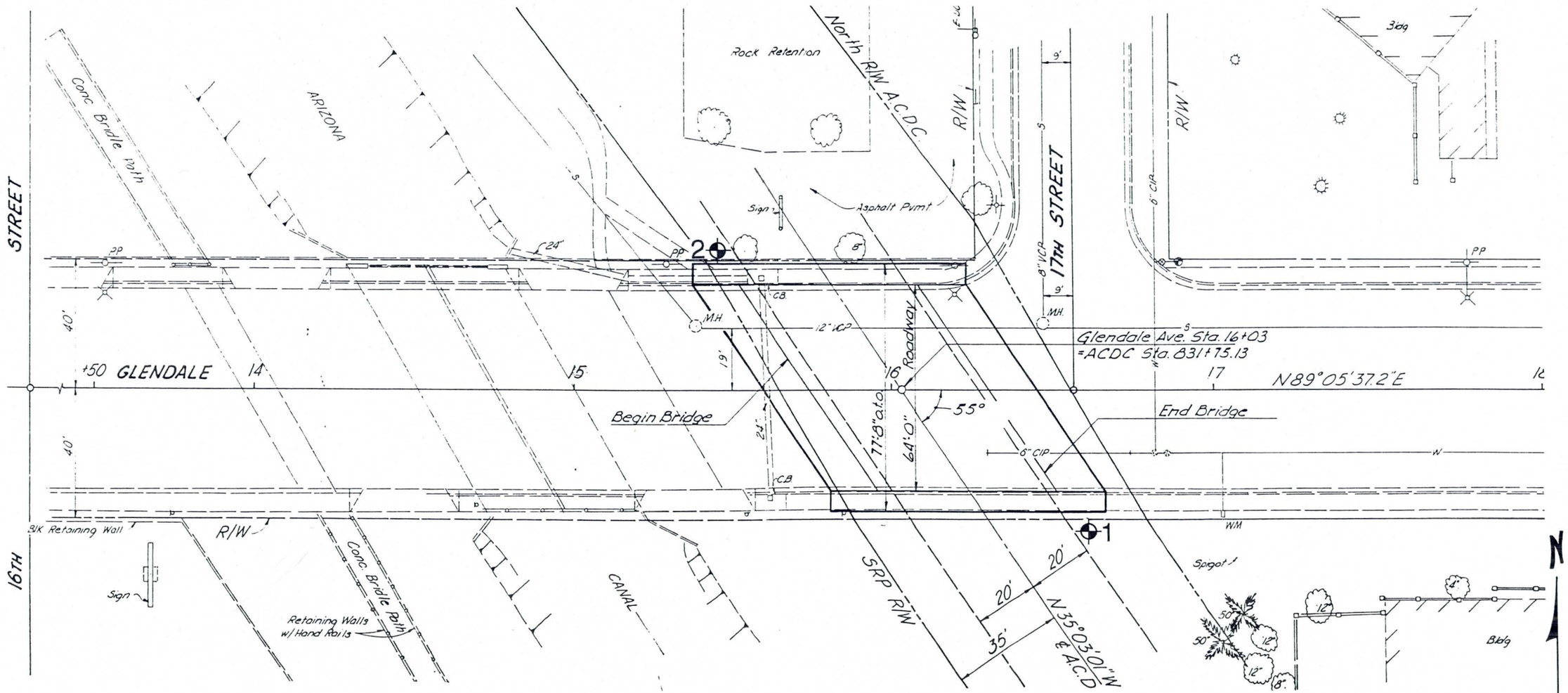


NOT TO SCALE

JOB NO. 2128J186

REVIEWED R. BERGQUIST, P.E.	**WESTERN TECHNOLOGIES INC.**
PREPARED MAXWELL	The Quality People

Glendale Avenue Bridge



BORING LOCATION

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REVISION #1

REVIEWED	R. BERGQUIST, P.E.
PREPARED	MAXWELL

WESTERN TECHNOLOGIES INC.
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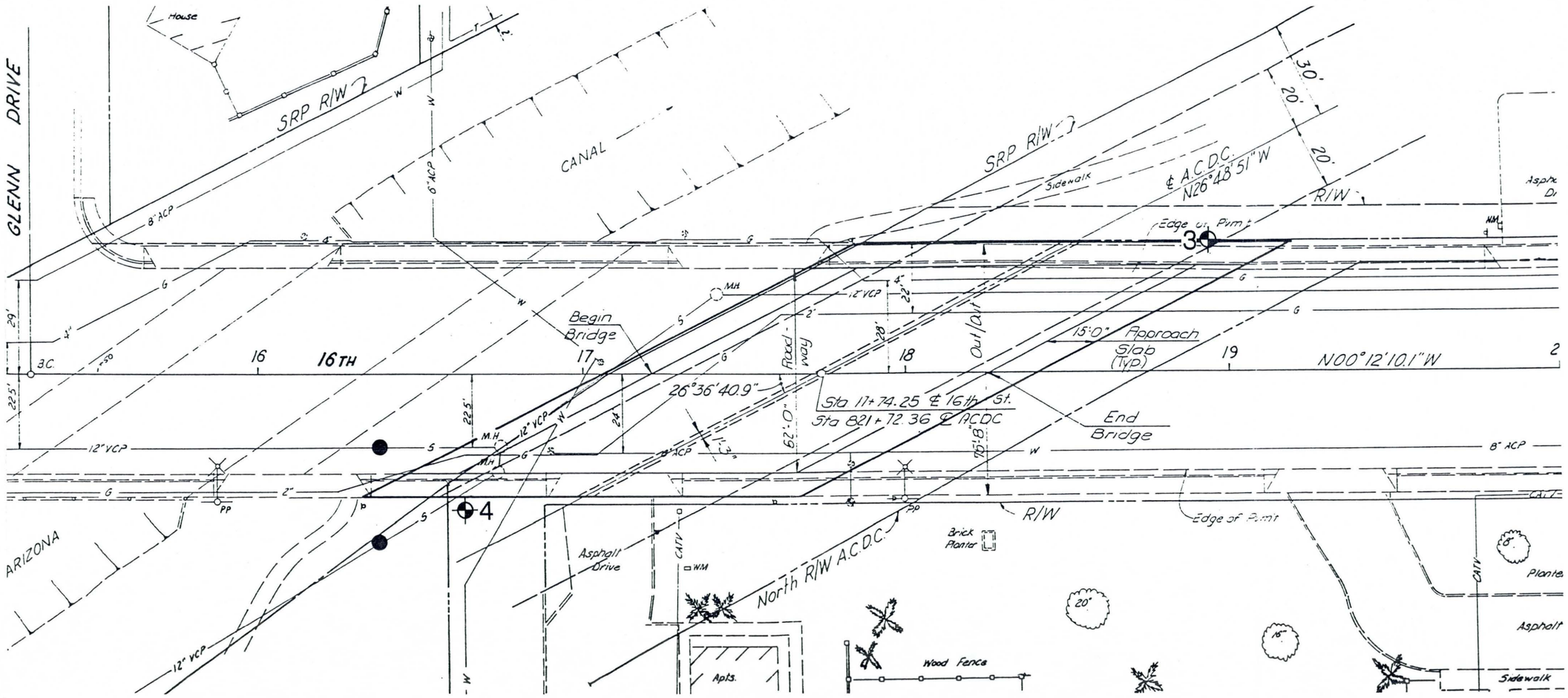
16th Street Bridge



BORING LOCATION

NOT TO SCALE

JOB NO. 2128J186
REVISION #1

REVIEWED	R. BERGQUIST, P.E.
PREPARED	MAXWELL

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DEFINITION OF TERMINOLOGY

Allowable Soil Bearing Capacity **Allowable Foundation Pressure**	The recommended maximum contact stress developed at the interface of the foundation element and the supporting material.
Backfill	A specified material placed and compacted in a confined area.
Base Course	A layer of specified material placed on a subgrade or subbase.
Base Course Grade	Top of base course.
Bench	A horizontal surface in a sloped deposit.
Caisson	A concrete foundation element cast in a circular excavation which may have an enlarged base. Sometimes referred to as a cast-in-place pier.
Concrete Slabs-on-Grade	A concrete surface layer cast directly upon a base, subbase or subgrade.
Crushed Rock Base Course	A base course composed of crushed rock of a specified gradation.
Differential Settlement	Unequal settlement between or within foundation elements of a structure.
Engineered Fill	Specified material placed and compacted to specified density and/or moisture conditions under observation of a representative of a soil engineer.
Existing Fill	Materials deposited through the action of man prior to exploration of the site.
Existing Grade	The ground surface at the time of field exploration.
Expansive Potential	The potential of a soil to expand (increase in volume) due to the absorption of moisture.
Fill	Materials deposited by the action of man.
Finished Grade	The final grade created as a part of the project.
Gravel Base Course	A base course composed of naturally occurring gravel with a specified gradation.
Heave	Upward movement.
Native Grade	The naturally occurring ground surface.
Native Soil	Naturally occurring on-site soil.
Rock	A natural aggregate of mineral grains connected by strong and permanent cohesive forces. Usually requires drilling, wedging, blasting or other methods of extraordinary force for excavation.
Sand and Gravel Base	A base course of sand and gravel of a specified gradation.
Sand Base Course	A base course composed primarily of sand of a specified gradation.
Scarify	To mechanically loosen soil or break down existing soil structure.
Settlement	Downward movement.
Soil	Any unconsolidated material composed of discrete solid particles, derived from the physical and/or chemical disintegration of vegetable or mineral matter, which can be separated by gentle mechanical means such as agitation in water.
Strip	To remove from present location.
Subbase	A layer of specified material placed to form a layer between the subgrade and base course.
Subbase Grade	Top of subbase.
Subgrade	Prepared native soil surface.

METHOD OF SOIL CLASSIFICATION
(ASTM D 2487)

COARSE-GRAINED SOILS

LESS THAN 50% FINES*

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% FINES	
GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, MORE THAN 12% FINES	
GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, MORE THAN 12% FINES	
SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% FINES	
SM	SILTY SANDS, SAND-SILT MIXTURES, MORE THAN 12% FINES	
SC	CLAYEY SANDS, SAND-CLAY MIXTURES, MORE THAN 12% FINES	

NOTE:
Coarse-grained soils receive dual symbols if they contain 5 to 12% fines (e.g. SW-SM, GP-GC, etc.)

FINE-GRAINED SOILS

MORE THAN 50% FINES*

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS	SILTS AND CLAYS Liquid limit less than 50
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
OL	ORGANIC SILTS OR ORGANIC SILTY-CLAYS OF LOW PLASTICITY	
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS	SILTS AND CLAYS Liquid limit more than 50
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY	
PT	PEAT, MUCK, AND OTHER HIGHLY ORGANIC SOILS	HIGHLY ORGANIC SOILS

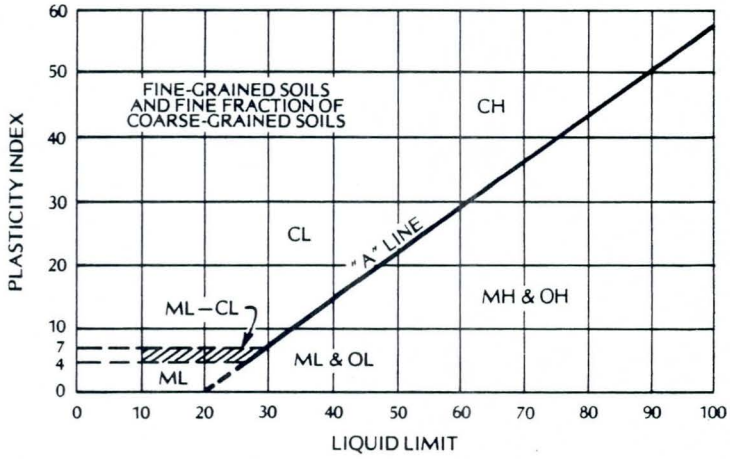
NOTE:
Fine-grained soils receive dual symbols if their limits plot in the hatched zone on the Plasticity Chart (ML-CL)

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	ABOVE 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	¾ in. to 3 in.
Fine	No. 4 to ¾ in.
SAND	No. 200 to No 4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
*FINES (Silt or Clay)	BELOW No. 200

NOTE:
Only sizes smaller than three inches are used to classify soils.

PLASTICITY CHART



BORING LOG NOTES

The number shown in "LOG OF BORING NO." refers to the approximate location of the same number indicated on the "Site Plan" as positioned in the field by measurements from property lines and/or existing features.

"STA" refers to the approximate stationing of the boring along 16th Street or Glendale Avenue according to the plan sheets prepared by Entranco-Mann-Johnson.

"R" or "L" refers to the approximate lateral offset, right or left respectively, from the centerline of 16th Street or Glendale Avenue.

"ELEVATION" refers to ground surface elevation at the boring location relative to the indicated "DATUM" established by the client.

"TYPE/SIZE BORING" refers to the exploratory equipment used in the boring wherein HSA = hollow-stem auger and Casing = reverse circulation percussion (Becker-type).

"C" in "Blows/Foot" refers to the number of blows by a diesel hammer required to advance the double-walled casing one foot. The casing has an outer diameter of 9.0". The diesel hammer is operated in two power modes; normal (8100 ft-lbs) and supercharged (9300 ft-lbs to 9700 ft-lbs).

"N" in "Blows/Foot" refers to the number of blows of a 140-pound weight, dropped 30 inches, required to advance a two-inch-outside-diameter split-barrel sampler a distance of 1 foot, Standard Penetration Test (ASTM D1586). Refusal to penetration is defined as more than 50 blows per 6 inches.

"Sample Type" refers to the form of sample recovery, in which N = Split-barrel sample, G = Grab sample.

"Unified Classification" refers to the soil type as defined by "Method of Soil Classification". The soils were classified visually in the field and, where appropriate, classifications were modified by visual examination of samples in the laboratory and/or by appropriate tests.

These notes and boring logs are intended for use in conjunction with the purposes of our services defined in the text. Boring log data should not be construed as part of the construction plans nor as defining construction conditions.

Boring logs depict our interpretations of subsurface conditions at the locations and on the date(s) noted. Variations in subsurface conditions and soil characteristics may occur between borings. Groundwater levels may fluctuate due to seasonal variations and other factors.

In general, terms and symbols on the boring logs conform with "Standard Definitions of Terms and Symbols Relating to Soil and Rock Mechanics" (ASTM D653).

LOG OF BORING NO. 1

Project Glendale Avenue Bridge Job No. 2128J186

Elevation Approximately 1244 LOCATION Sta 16+60, 45'R

Type/Size Boring HSA/7"; Casing 9" Rig Type CME 75; AP-1000

Groundwater Conditions No Groundwater Encountered Date 12-19-88; 12-23-8

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	19					SC-GC	SAND AND GRAVEL; with clay, brown
	30		G				
	39						light brown, moderate cementation
	60						
5	77	75/10"	N			GP-GC	SANDY GRAVEL; some clay, light brown, light cementation
	92						
	107						
	127						
	77						
10	45					SP-SC	GRAVELLY SAND; some clay, brown, light cementation
	47	40/2"	N				
	47						
	51						
	110					GP-GC	SANDY GRAVEL; some clay, brown, light cementation
15	55						
	172						light brown, moderate cementation
	124		G				with cobbles
	148						
	297						boulder
20	153					SC	GRAVELLY SAND; with clay, brown, heavy cementation
	124						
	130		G				
	131						moderate cementation
	154					GP-GC	SANDY GRAVEL; some clay, brown, moderate cementation
25	169						
	206						heavy cementation
	307						
	118					SC	GRAVELLY SAND; with clay, brown, moderate cementation
	135						
30	74					GP	GRAVELS AND COBBLES; some sand, brown, moderate cementation

Continued on next page...

LOG OF BORING NO. 1 CONTINUED

Project Glendale Avenue Bridge Job No. 2128J186

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Moisture Content, %	Unified Classification	Description
	72					GP	GRAVELS AND COBBLES; cont'd
	72						
	115						
	116						
35	88						
	103						
	53						
	112						
	97						
40	32		G			SP-SC	GRAVELLY SAND; some clay, brown, light cementation
	28	75/5"	N				
	53						
	144					GP	SANDY GRAVEL; some cobbles, brown, light cementation
	110						
45	162						
	200						
	139						
	80						
	84						
50	28					SP-SC	GRAVELLY SAND; some clay, brown, light cementation
	46						
	43						
	38						
	50						
55	28						
	34						
	70					GP	SANDY GRAVEL; some cobbles, brown, light cementation
	47						
	93						
60	42					SP	GRAVELLY SAND; some cobbles, brown, light cementation

LOG OF BORING NO. 1 CONTINUED

Project Glendale Avenue Bridge Job No. 2128J186

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Moisture Content, %	Unified Classification	Description
	54 65					SP	GRAVELLY SAND; cont'd
	124 138					GP	GRAVEL AND COBBLES; some sand, brown, light cementation
65	344						near bedrock (rock cuttings)
							Becker-type rig stopped at 65 feet
							Auger rig refusal encountered at depth of 14 feet
70							
75							
80							
85							
90							

LOG OF BORING NO. 2

Project: Glendale Avenue Bridge — Job No. 2128J186

Elevation: Approximately 1244 — LOCATION: Sta 16+60, 45'R

Type/Size Boring: HSA/7" — Rig Type: CME 75

Groundwater Conditions: No Groundwater Encountered — Date: 12-19-88

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
						GP-GC	SANDY GRAVEL; with cobbles, some clay, brown
						SP-SC	GRAVELLY SAND; some clay, brown
5						GC	SANDY GRAVEL; with clay, light brown, light cementation
		50/4"	N				Auger refusal at 5-1/2 feet
10							
15							
20							
25							
30							

LOG OF BORING NO. 3

Project 16th Street Bridge Job No. 2128J186

Elevation Approximately 1244 LOCATION Sta 16+60, 45'R

Type/Size Boring HSA/7"; Casing 9" Rig Type CME 75; AP-1000

Groundwater Conditions No Groundwater Encountered Date 12-19-88; 12-23-8

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
	20					GC	SANDY GRAVEL; with clay, brown
	16		G				
	22						
	25						light brown, light cementation
5	26						
	28	55	N				
	54						heavy cementation
	94						
10	132					GP-GC	SANDY GRAVEL; some clay, brown, moderate to heavy cementation
	70						
	53						
	65		G				
	72						
15	50						
	92						
	95						some cobbles
	134						
	106						
20	32					SP-SC	GRAVELLY SAND; some cobbles, some clay, brown, light cementation
	37						
	37		G				
	46						
	96					GP	SANDY GRAVEL; some cobbles, light brown, light cementation (minimal recovery from cyclone)
25	55						
	71						
	238						
	183					SP-SM	GRAVELLY SAND; some silt, yellowish to reddish brown, light cementation
	176		G				
30	106						

LOG OF BORING NO. 3 CONTINUED

Project 16th Street Bridge Job No. 2128J186

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Moisture Content, %	Unified Classification	Description
	70					SP-SM	GRAVELLY SAND; cont'd
	88						
	87		G			SP	GRAVELLY SAND; yellowish to reddish brown, probably crushed conglomerate
	135						
35	196						
	308		G				probably bedrock (rock cuttings)
	404/6½"						
							Becker-type rig refusal encountered at depth of 36½ feet
40							
							Auger rig refusal encountered at depth of 9½ feet
45							
50							
55							
60							

188

LOG OF BORING NO. 4

Project 16th Street Bridge Job No. 2128J186

Elevation Approximately 1244 LOCATION Sta 16+60, 45'R

Type/Size Boring HSA/7" Rig Type CME 75

Groundwater Conditions No Groundwater Encountered Date 12-19-88

Depth, feet	Blows/Foot C	Blows/Foot N/R	Sample Type	Dry Density pcf	Water Content, %	Unified Classification	Description
						SC	GRAVELLY SAND; with clay, brown
						GC	SANDY GRAVEL; some clay, some cobbles, brown
5							Auger rig refusal at depth of 3 feet (two borings attempted)
10							
15							
20							
25							
30							

SOIL PROPERTIES

Job No. 2128J186

Boring No.	Depth, ft	Soil Class.	Soil Property		Compression/Consolidation			Expansion			Shear Strength			Water Soluble Matter, PPM		Remarks
			Initial Dry Density pcf	Initial Water Content, %	Surcharge KSF	Total Comp. %	Void Ratio	Surcharge KSF	Expansion %	Max. Swell Pressure KSF	Test Method	C KSF	ø Deg.	Salts	Sulfates	
1	19-23	SC	118	9.8							DS	0.3	38			
3	10-15	GP-GC	118	9.8							DS	0.3	35			
3	20-25	GP-GC	118	9.8							DS	0.3	38			
3	31-35	SP-SM	118	9.8							DS	0.0	42			

Boring No.	Depth, ft	Comments
1	19-23	Remolded sample of material passing No. 4 sieve.
3	10-15	Remolded sample of material passing No. 4 sieve.
3	20-25	Remolded sample of material passing No. 4 sieve.
3	31-35	Remolded sample of material passing No. 4 sieve.

Note: Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.

LEGEND

Shear Strength Test Method
- DS Direct Shear
- DS Direct Shear (saturated)
- UC Unconfined Compression
- UU Unconsolidated Undrained
- CU Consolidated Undrained w/pore press.
- CU Consolidated Undrained
- CD Consolidated Drained

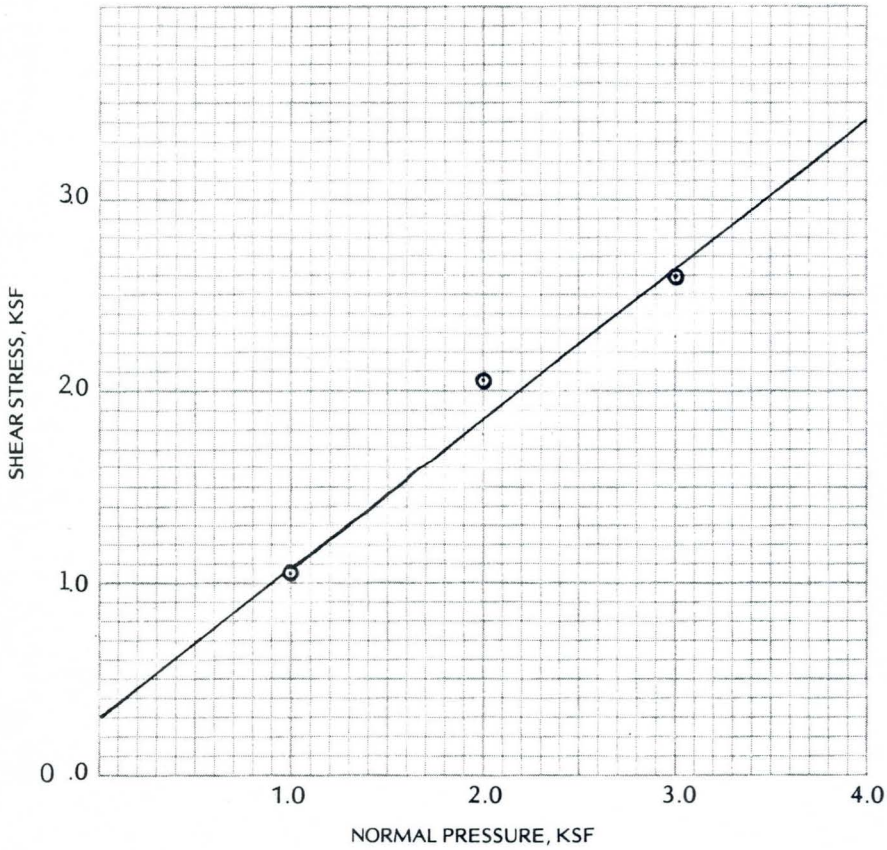
REMARKS

1. Compacted density (approx. 95% of ASTM D698 max. density at moisture content slightly below optimum).
2. Compacted density (approx. 95% of ASTM D1557 max. density at moisture content slightly below optimum).
3. Submerged to approximate saturation.
4.

DIRECT SHEAR TEST

Job No. 2128J186

Lab/Invoice No.



Type of Material SC

Source of Material 1 (19-23)

Sampled By E. Hoque Date 12-23-88

Submitted By E. Hoque Date 1-27-89

Reviewed By F. Costello Date 2-20-89

Test Procedure ASTM D3080- Single Shear

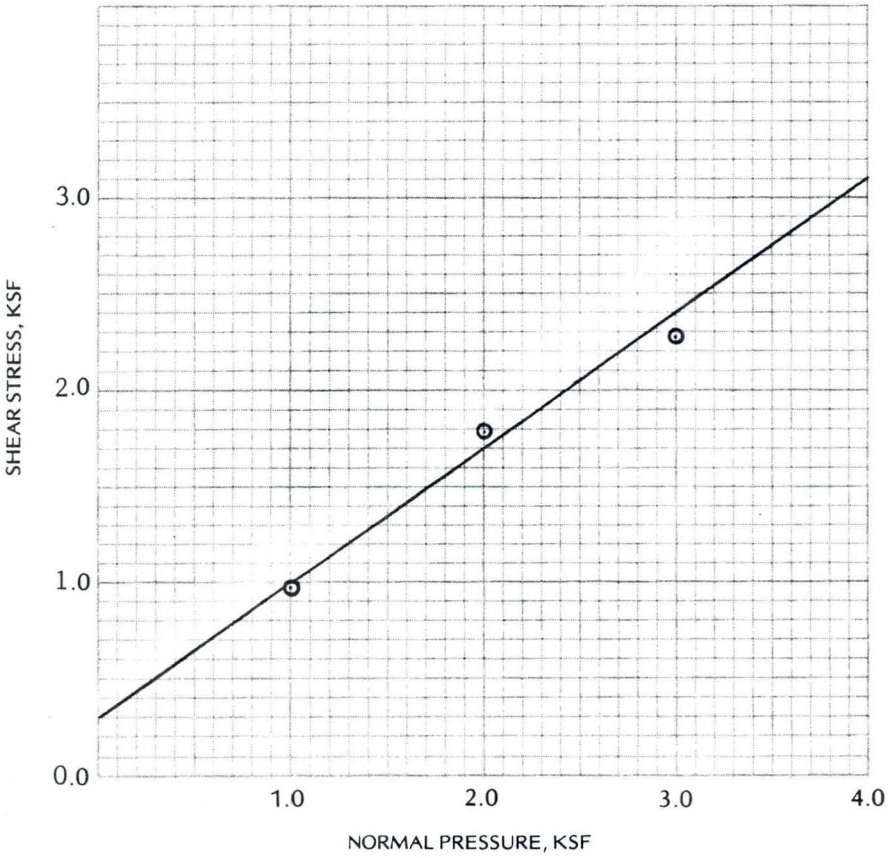
Test Condition: —— InSitu ---- Saturated

Sample Condition: ☐ Undisturbed ☒ Remolded

Initial Dry Density, pcf 118

Initial Moisture Content, % 9.8

ϕ = 38 ° C = 0.3 Kips/Sq. Ft.



Type of Material GP-GC

Source of Material 3 (10-15)

Sampled By E. Hoque Date 12-23-88

Submitted By E. Hoque Date 1-27-89

Reviewed By F. Costello Date 2-20-89

Test Procedure ASTM D3080- Single Shear

Test Condition: —— InSitu ---- Saturated

Sample Condition: ☐ Undisturbed ☒ Remolded

Initial Dry Density, pcf 118

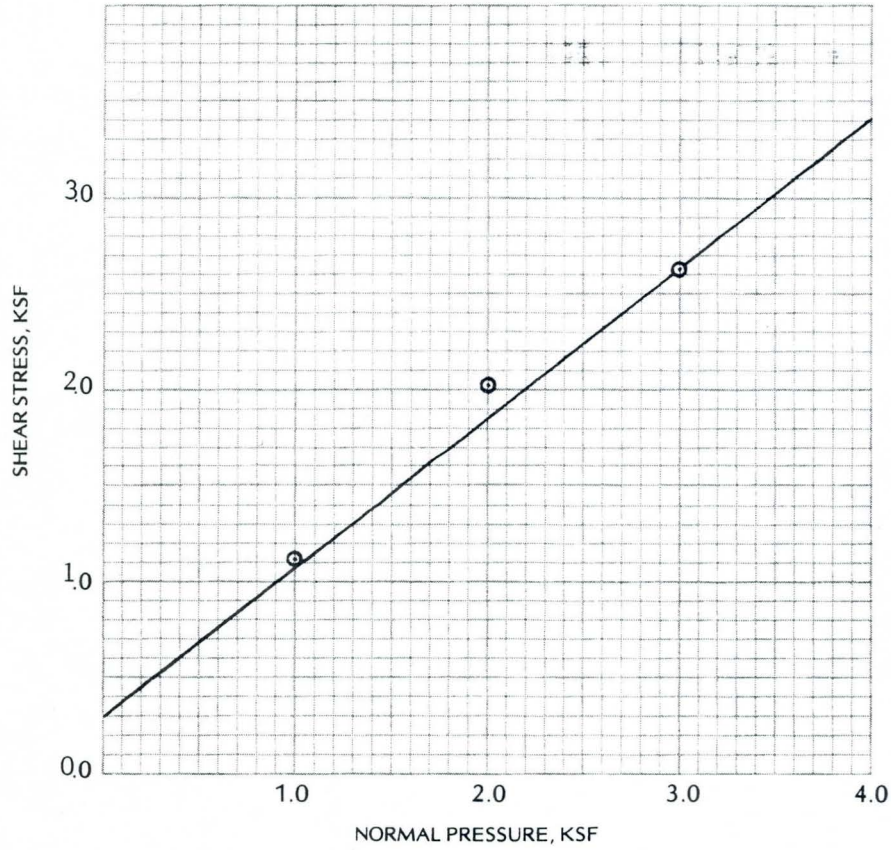
Initial Moisture Content, % 9.8

ϕ = 35 ° C = 0.3 Kips/Sq. Ft.

DIRECT SHEAR TEST

Job No. 2128J186

Lab/Invoice No.



Type of Material GP-GC

Source of Material 3 (20-25)

Sampled By E. Hoque Date 12-23-88

Submitted By E. Hoque Date 1-26-89

Reviewed By F. Costello Date 2-20-89

Test Procedure ASTM D3080- Single Shear

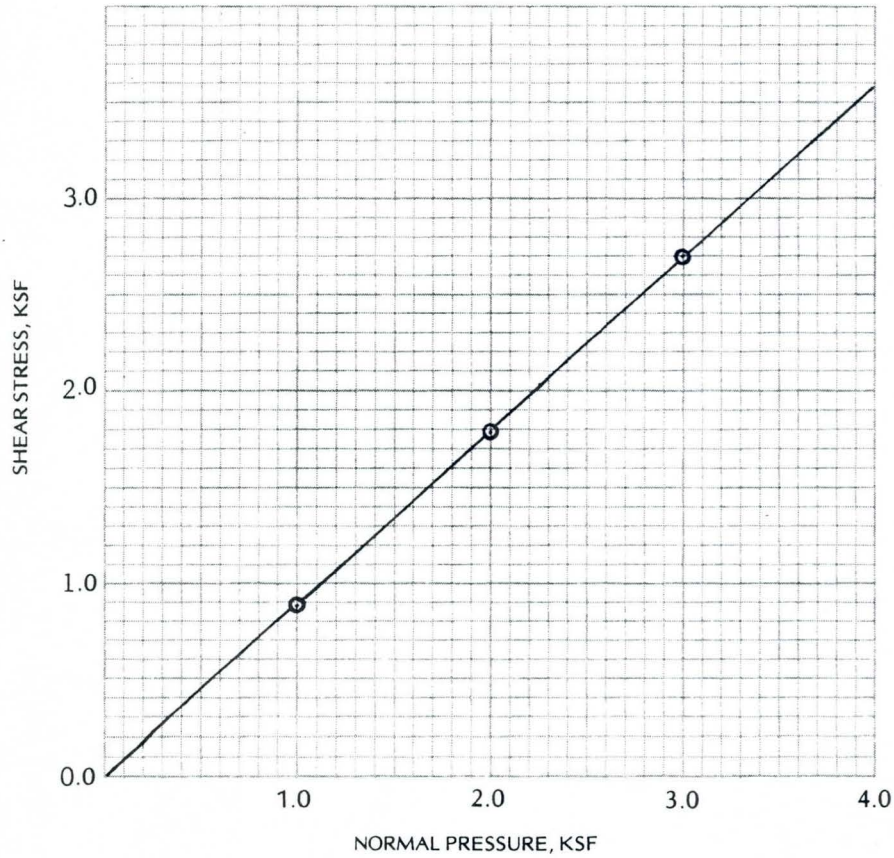
Test Condition: —— InSitu ---- Saturated

Sample Condition: ☐ Undisturbed ☒ Remolded

Initial Dry Density, pcf 118

Initial Moisture Content, % 9.8

ϕ = 38 ° C = 0.3 Kips/Sq. Ft.



Type of Material SP-SM

Source of Material 3 (31-35)

Sampled By E. Hoque Date 12-23-88

Submitted By E. Hoque Date 1-26-89

Reviewed By F. Costello Date 2-20-89

Test Procedure ASTM D3080- Single Shear

Test Condition: —— InSitu ---- Saturated

Sample Condition: ☐ Undisturbed ☒ Remolded

Initial Dry Density, pcf 118

Initial Moisture Content, % 9.8

ϕ = 42 ° C = 0 Kips/Sq. Ft.

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Glendale Ave & 16th St Bridges; Strutted w/36" Dia Piers & 4% Steel

UNITS--ENGLISH UNITS

I N P U T I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 480.00 IN
2 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
0.00	36.000	0.102D+06	0.102D+04	0.403D+07
480.00	36.000	0.102D+06	0.102D+04	0.403D+07

SOILS INFORMATION

```
X AT THE GROUND SURFACE                  =      360.00 IN

1 LAYER(S) OF SOIL

LAYER  1
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER                =      360.00 IN
X AT THE BOTTOM OF THE LAYER             =     1000.00 IN
MODULUS OF SUBGRADE REACTION             =  0.225D+03 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                    2 POINTS
          X,IN      WEIGHT,LBS/IN**3
        360.00        0.80D-01
       1000.00        0.80D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                    2 POINTS
          X,IN        C,LBS/IN**2      PHI,DEGREES      E50
        360.00      0.000D+00        0.450D+02        -----
       1000.00      0.000D+00        0.450D+02        -----
```

BOUNDARY AND LOADING CONDITIONS

```
DISTRIBUTED LOAD CURVE              2 POINTS
                          X,IN     LOAD,LBS/IN
                          0.00       0.000D+00
                        360.00       0.100D+04


LOADING NUMBER   1

BOUNDARY-CONDITION CODE                  =          1
LATERAL LOAD AT THE PILE HEAD            = -0.746D+05 LBS
MOMENT AT THE PILE HEAD                  =  0.000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD              =  0.181D+06 LBS


FINITE-DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                                  =         40
    DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE           =  0.100D-05 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS     =        100
    MAXIMUM ALLOWABLE DEFLECTION                               =   0.36D+03 IN


OUTPUT CODES
    KOUTPT =  1
    KPYOP  =  0
    INC    =  1
```

OUTPUT INFORMATION

****

DISTRIBUTED LOAD CURVE 2 POINTS

X, IN	LOAD, LBS/IN
0.00	0.000D+00
360.00	0.100D+04

36" ϕ Shafts

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = -0.746D+05 LBS
MOMENT AT THE PILE HEAD = 0.000D+00 IN LBS
AXIAL LOAD AT THE PILE HEAD = 0.181D+06 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	-0.156D-01	0.218D-06	-0.746D+05	0.000D+00	0.177D+03	0.411D+12
12.00	0.325D-01	-0.904D+06	-0.744D+05	0.000D+00	0.337D+03	0.411D+12
24.00	0.804D-01	-0.180D+07	-0.738D+05	0.000D+00	0.496D+03	0.411D+12
36.00	0.128D+00	-0.269D+07	-0.728D+05	0.000D+00	0.653D+03	0.411D+12
48.00	0.174D+00	-0.357D+07	-0.714D+05	0.000D+00	0.807D+03	0.411D+12
60.00	0.219D+00	-0.442D+07	-0.696D+05	0.000D+00	0.958D+03	0.411D+12
72.00	0.262D+00	-0.525D+07	-0.674D+05	0.000D+00	0.110D+04	0.411D+12
84.00	0.304D+00	-0.606D+07	-0.648D+05	0.000D+00	0.125D+04	0.411D+12
96.00	0.343D+00	-0.682D+07	-0.618D+05	0.000D+00	0.138D+04	0.411D+12
108.00	0.380D+00	-0.755D+07	-0.584D+05	0.000D+00	0.151D+04	0.411D+12
120.00	0.415D+00	-0.824D+07	-0.546D+05	0.000D+00	0.163D+04	0.411D+12
132.00	0.446D+00	-0.887D+07	-0.504D+05	0.000D+00	0.174D+04	0.411D+12
144.00	0.475D+00	-0.946D+07	-0.458D+05	0.000D+00	0.185D+04	0.411D+12
156.00	0.500D+00	-0.998D+07	-0.408D+05	0.000D+00	0.194D+04	0.411D+12
168.00	0.522D+00	-0.104D+08	-0.354D+05	0.000D+00	0.202D+04	0.411D+12
180.00	0.540D+00	-0.108D+08	-0.296D+05	0.000D+00	0.209D+04	0.411D+12
192.00	0.554D+00	-0.112D+08	-0.234D+05	0.000D+00	0.215D+04	0.411D+12
204.00	0.564D+00	-0.114D+08	-0.168D+05	0.000D+00	0.219D+04	0.411D+12
216.00	0.570D+00	-0.116D+08	-0.980D+04	0.000D+00	0.222D+04	0.411D+12
228.00	0.573D+00	-0.116D+08	-0.240D+04	0.000D+00	0.223D+04	0.411D+12
240.00	0.571D+00	-0.116D+08	0.540D+04	0.000D+00	0.223D+04	0.411D+12
252.00	0.565D+00	-0.115D+08	0.136D+05	0.000D+00	0.221D+04	0.411D+12
264.00	0.555D+00	-0.113D+08	0.222D+05	0.000D+00	0.217D+04	0.411D+12
276.00	0.541D+00	-0.110D+08	0.312D+05	0.000D+00	0.211D+04	0.411D+12
288.00	0.523D+00	-0.105D+08	0.406D+05	0.000D+00	0.204D+04	0.411D+12
300.00	0.502D+00	-0.999D+07	0.504D+05	0.000D+00	0.194D+04	0.411D+12
312.00	0.477D+00	-0.932D+07	0.606D+05	0.000D+00	0.182D+04	0.411D+12
324.00	0.448D+00	-0.853D+07	0.712D+05	0.000D+00	0.168D+04	0.411D+12
336.00	0.417D+00	-0.760D+07	0.822D+05	0.000D+00	0.152D+04	0.411D+12
348.00	0.383D+00	-0.655D+07	0.936D+05	0.000D+00	0.133D+04	0.411D+12
360.00	0.347D+00	-0.535D+07	0.105D+06	0.000D+00	0.112D+04	0.411D+12
372.00	0.309D+00	-0.400D+07	0.109D+06	-0.450D+03	0.884D+03	0.411D+12
384.00	0.270D+00	-0.272D+07	0.100D+06	-0.100D+04	0.658D+03	0.411D+12
396.00	0.229D+00	-0.159D+07	0.847D+05	-0.155D+04	0.458D+03	0.411D+12
408.00	0.188D+00	-0.676D+06	0.631D+05	-0.203D+04	0.297D+03	0.411D+12
35' 420.00	0.147D+00	-0.575D+05	0.390D+05	-0.199D+04	0.188D+03	0.411D+12
36' 432.00	0.106D+00	0.275D+06	0.168D+05	-0.172D+04	0.226D+03	0.411D+12
444.00	0.648D-01	0.361D+06	-0.834D+03	-0.123D+04	0.241D+03	0.411D+12
456.00	0.238D-01	0.270D+06	-0.113D+05	-0.515D+03	0.225D+03	0.411D+12
468.00	-0.171D-01	0.105D+06	-0.119D+05	0.415D+03	0.196D+03	0.411D+12
480.00	-0.579D-01	0.000D+00	0.000D+00	0.156D+04	0.177D+03	0.411D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -0.337D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.250D-07 LBS

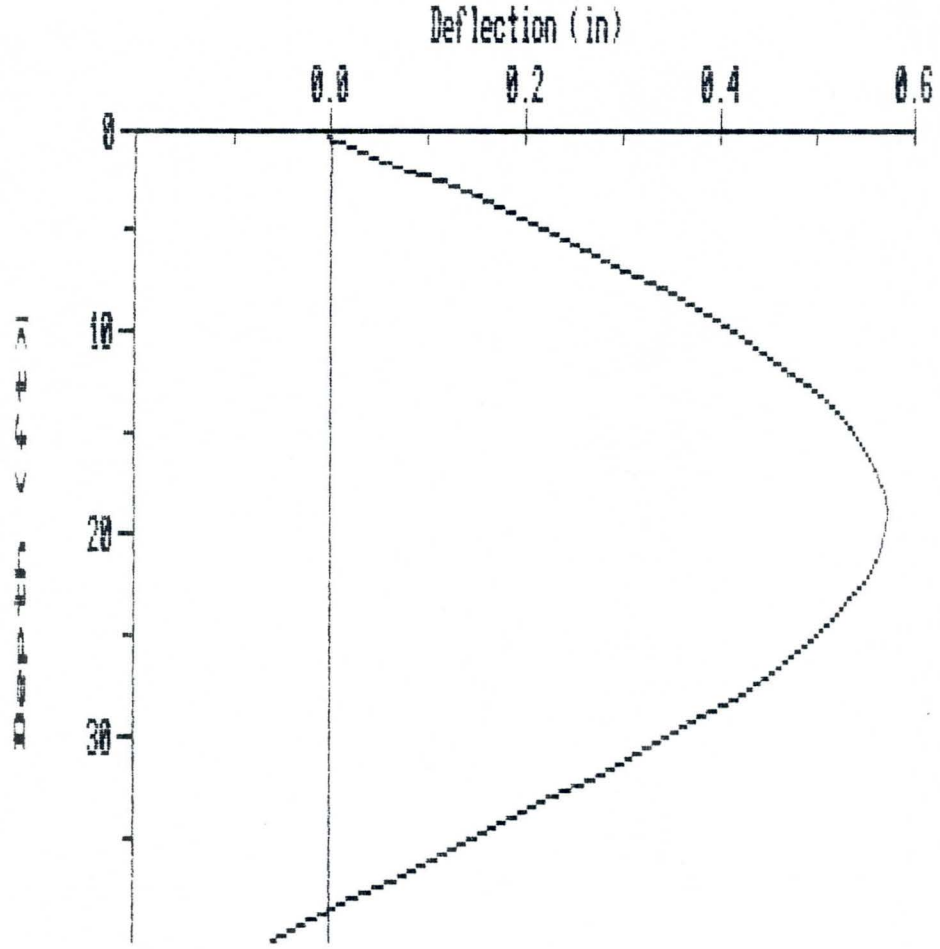
OUTPUT SUMMARY

PILE-HEAD DEFLECTION	=	-0.156D-01 IN
MAXIMUM BENDING MOMENT	=	-0.116D+08 LBS-IN
MAXIMUM SHEAR FORCE	=	0.109D+06 LBS
NO. OF ITERATIONS	=	25
NO. OF ZERO DEFLECTION POINTS	=	2

S U M M A R Y T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
-0.7460D+05	0.0000D+00	0.1810D+06	-0.1560D-01	-0.1164D+08	0.1087D+06

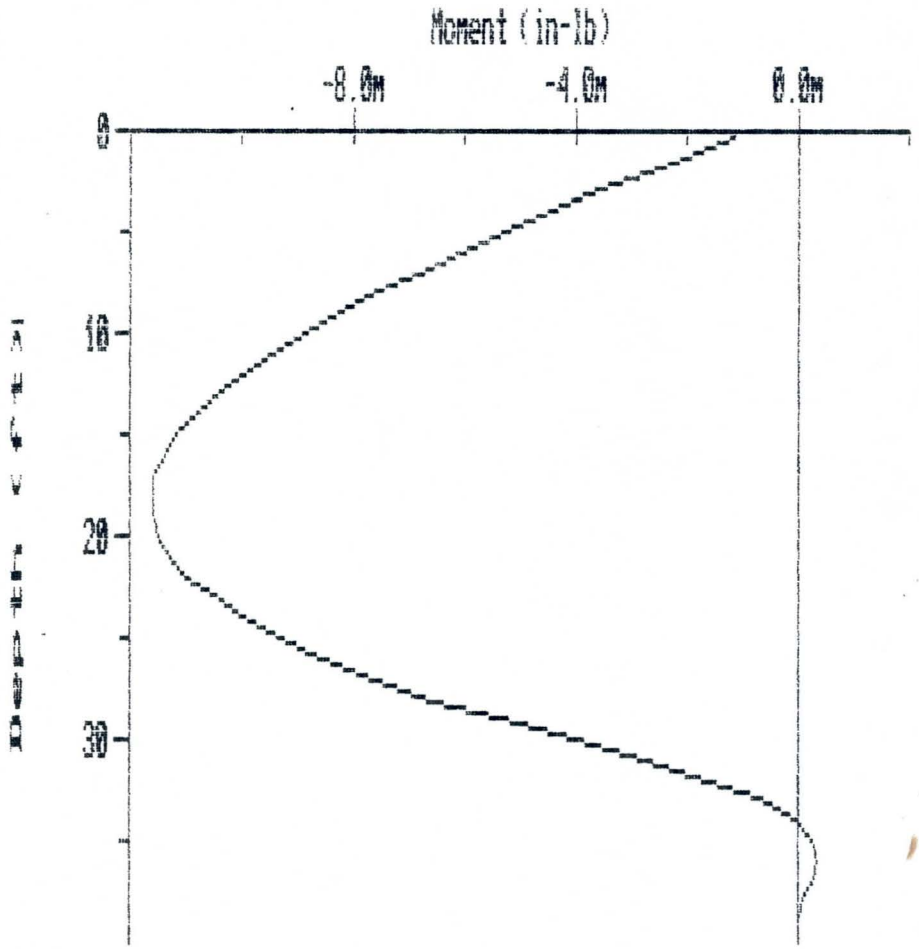
Glendale Ave. & 16th St. Bridges w/Struts, 36" Dia. Piers & 4% Steel



Deflection Curve

JOB NO. 2128J186 REVISION #1	REVIEWED R. BERGQUIST, P.E. PREPARED MAXWELL	**WESTERN TECHNOLOGIES INC.** The Quality People

Glendale Ave. & 16th St. Bridges w/Struts, 36" Dia. Piers & 4% Steel



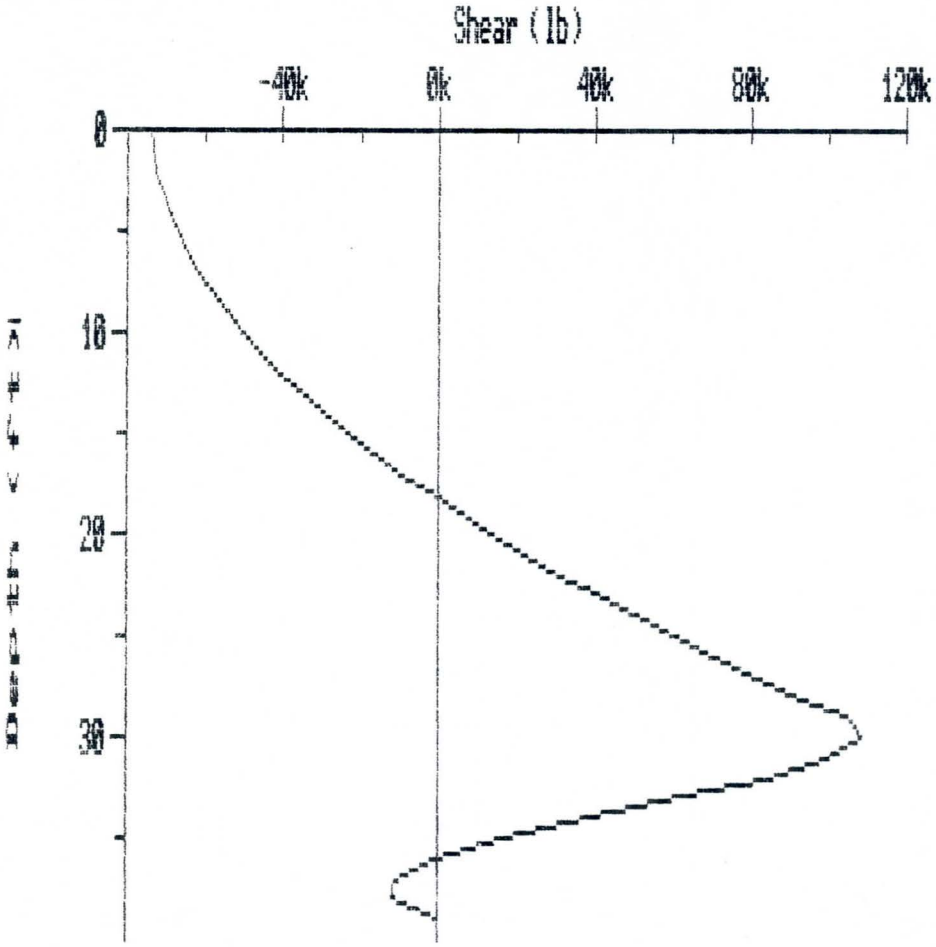
Moment Curve

JOB NO. 2128J186
REVISION #1

REVIEWED R. BERGQUIST, P.E.
PREPARED MAXWELL

WESTERN TECHNOLOGIES INC.
The Quality People

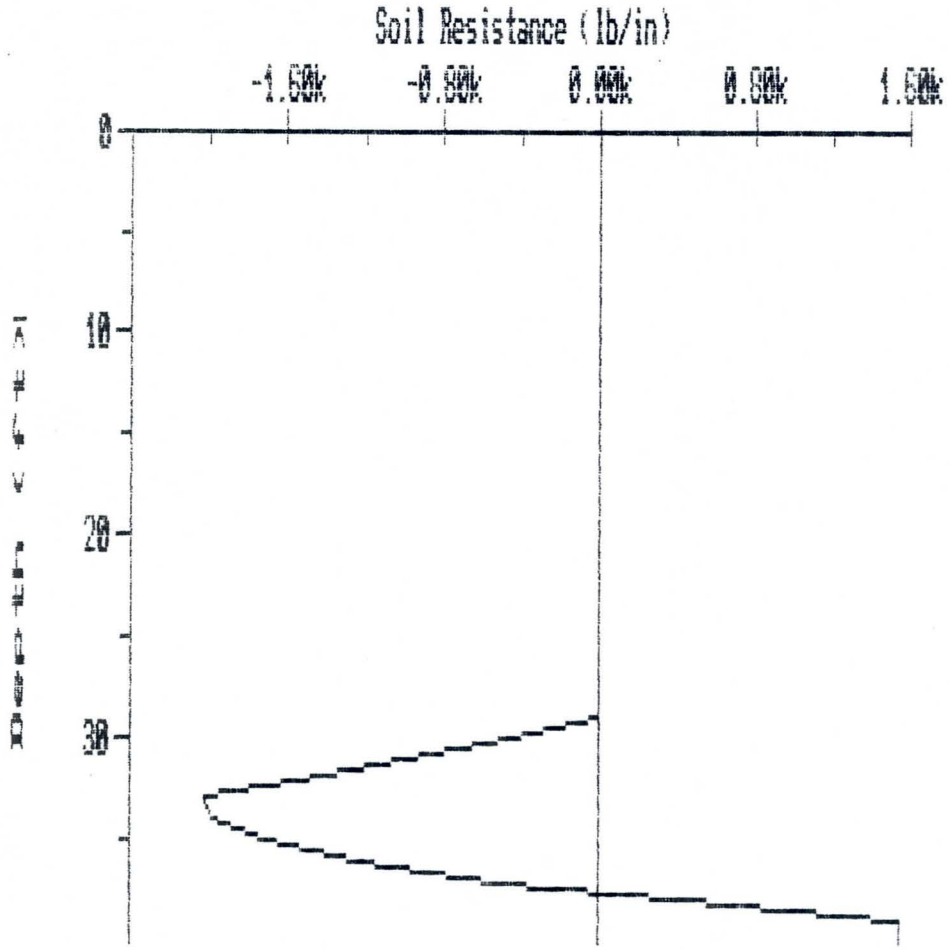
Glendale Ave. & 16th St. Bridges w/Struts, 36" Dia. Piers & 4% Steel



Shear Curve

JOB NO. 2128J186 REVISION #1	REVIEWED R. BERGQUIST, P.E. PREPARED MAXWELL	**WESTERN TECHNOLOGIES INC.** The Quality People

Glendale Ave. & 16th St. Bridges w/Struts, 36" Dia. Piers & 4% Steel



Soil Response Curve

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Glendale Ave & 16th St Bridges; Strutted w/30" Dia Piers & 4% Steel

UNITS--ENGLISH UNITS

I N P U T I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 480.00 IN
2 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
0.00	30.000	0.516D+05	0.710D+03	0.403D+07
480.00	30.000	0.516D+05	0.710D+03	0.403D+07

SOILS INFORMATION

```
X AT THE GROUND SURFACE                =      360.00 IN

1 LAYER(S) OF SOIL

LAYER  1
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER              =      360.00 IN
X AT THE BOTTOM OF THE LAYER           =     1000.00 IN
MODULUS OF SUBGRADE REACTION           =  0.225D+03 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                    2 POINTS
          X,IN      WEIGHT,LBS/IN**3
        360.00        0.80D-01
       1000.00        0.80D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                    2 POINTS
          X,IN       C,LBS/IN**2      PHI,DEGREES      E50
        360.00      0.000D+00        0.450D+02       -----
       1000.00      0.000D+00        0.450D+02       -----
```

BOUNDARY AND LOADING CONDITIONS

```
DISTRIBUTED LOAD CURVE            2 POINTS
                        X,IN     LOAD,LBS/IN
                        0.00       0.000D+00
                      360.00       0.750D+03


LOADING NUMBER   1

BOUNDARY-CONDITION CODE                =          1
LATERAL LOAD AT THE PILE HEAD          = -0.530D+05 LBS
MOMENT AT THE PILE HEAD                =  0.000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD            =  0.144D+06 LBS


FINITE-DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS                                  =         40
DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE           =  0.100D-05 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS     =        100
MAXIMUM ALLOWABLE DEFLECTION                               =   0.30D+03 IN


OUTPUT CODES
KOUTPT =  1
KPYOP  =  0
INC    =  1
```

OUTPUT INFORMATION

****

DISTRIBUTED LOAD CURVE 2 POINTS

X, IN	LOAD, LBS/IN
0.00	0.000D+00
360.00	0.750D+03

30" ϕ Shafts

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = -0.530D+05 LBS
MOMENT AT THE PILE HEAD = 0.000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD = 0.144D+06 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	-0.392D-01	-0.200D-07	-0.530D+05	0.000D+00	0.203D+03	0.208D+12
12.00	0.262D-01	-0.645D+06	-0.528D+05	0.000D+00	0.390D+03	0.208D+12
24.00	0.910D-01	-0.129D+07	-0.524D+05	0.000D+00	0.577D+03	0.208D+12
36.00	0.155D+00	-0.192D+07	-0.516D+05	0.000D+00	0.761D+03	0.208D+12
48.00	0.218D+00	-0.254D+07	-0.506D+05	0.000D+00	0.943D+03	0.208D+12
60.00	0.279D+00	-0.315D+07	-0.493D+05	0.000D+00	0.112D+04	0.208D+12
72.00	0.337D+00	-0.374D+07	-0.476D+05	0.000D+00	0.129D+04	0.208D+12
84.00	0.393D+00	-0.431D+07	-0.457D+05	0.000D+00	0.146D+04	0.208D+12
96.00	0.446D+00	-0.486D+07	-0.434D+05	0.000D+00	0.161D+04	0.208D+12
108.00	0.496D+00	-0.537D+07	-0.409D+05	0.000D+00	0.176D+04	0.208D+12
120.00	0.542D+00	-0.585D+07	-0.380D+05	0.000D+00	0.190D+04	0.208D+12
132.00	0.584D+00	-0.629D+07	-0.349D+05	0.000D+00	0.203D+04	0.208D+12
144.00	0.622D+00	-0.670D+07	-0.314D+05	0.000D+00	0.215D+04	0.208D+12
156.00	0.655D+00	-0.706D+07	-0.276D+05	0.000D+00	0.225D+04	0.208D+12
168.00	0.683D+00	-0.737D+07	-0.236D+05	0.000D+00	0.235D+04	0.208D+12
180.00	0.706D+00	-0.763D+07	-0.192D+05	0.000D+00	0.242D+04	0.208D+12
192.00	0.724D+00	-0.784D+07	-0.146D+05	0.000D+00	0.248D+04	0.208D+12
204.00	0.736D+00	-0.799D+07	-0.965D+04	0.000D+00	0.252D+04	0.208D+12
216.00	0.743D+00	-0.807D+07	-0.440D+04	0.000D+00	0.255D+04	0.208D+12
228.00	0.744D+00	-0.809D+07	0.115D+04	0.000D+00	0.256D+04	0.208D+12
240.00	0.740D+00	-0.804D+07	0.700D+04	0.000D+00	0.254D+04	0.208D+12
252.00	0.730D+00	-0.792D+07	0.132D+05	0.000D+00	0.251D+04	0.208D+12
264.00	0.714D+00	-0.772D+07	0.196D+05	0.000D+00	0.245D+04	0.208D+12
276.00	0.693D+00	-0.745D+07	0.264D+05	0.000D+00	0.237D+04	0.208D+12
288.00	0.667D+00	-0.709D+07	0.334D+05	0.000D+00	0.226D+04	0.208D+12
300.00	0.636D+00	-0.664D+07	0.407D+05	0.000D+00	0.213D+04	0.208D+12
312.00	0.601D+00	-0.610D+07	0.484D+05	0.000D+00	0.198D+04	0.208D+12
324.00	0.561D+00	-0.546D+07	0.563D+05	0.000D+00	0.179D+04	0.208D+12
336.00	0.518D+00	-0.473D+07	0.646D+05	0.000D+00	0.158D+04	0.208D+12
348.00	0.471D+00	-0.390D+07	0.731D+05	0.000D+00	0.134D+04	0.208D+12
360.00	0.421D+00	-0.296D+07	0.820D+05	0.000D+00	0.106D+04	0.208D+12
372.00	0.370D+00	-0.192D+07	0.839D+05	-0.429D+03	0.761D+03	0.208D+12
384.00	0.317D+00	-0.935D+06	0.756D+05	-0.960D+03	0.475D+03	0.208D+12
396.00	0.264D+00	-0.895D+05	0.610D+05	-0.147D+04	0.229D+03	0.208D+12
408.00	0.210D+00	0.545D+06	0.410D+05	-0.187D+04	0.361D+03	0.208D+12
420.00	0.157D+00	0.910D+06	0.173D+05	-0.209D+04	0.467D+03	0.208D+12
432.00	0.104D+00	0.974D+06	-0.542D+04	-0.169D+04	0.486D+03	0.208D+12
444.00	0.523D-01	0.795D+06	-0.215D+05	-0.988D+03	0.434D+03	0.208D+12
456.00	0.943D-03	0.474D+06	-0.275D+05	-0.204D+02	0.340D+03	0.208D+12
468.00	-0.501D-01	0.149D+06	-0.203D+05	0.122D+04	0.246D+03	0.208D+12
480.00	-0.101D+00	0.000D+00	0.000D+00	0.217D+04	0.203D+03	0.208D+12

33'
34'

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -0.261D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.134D-07 LBS

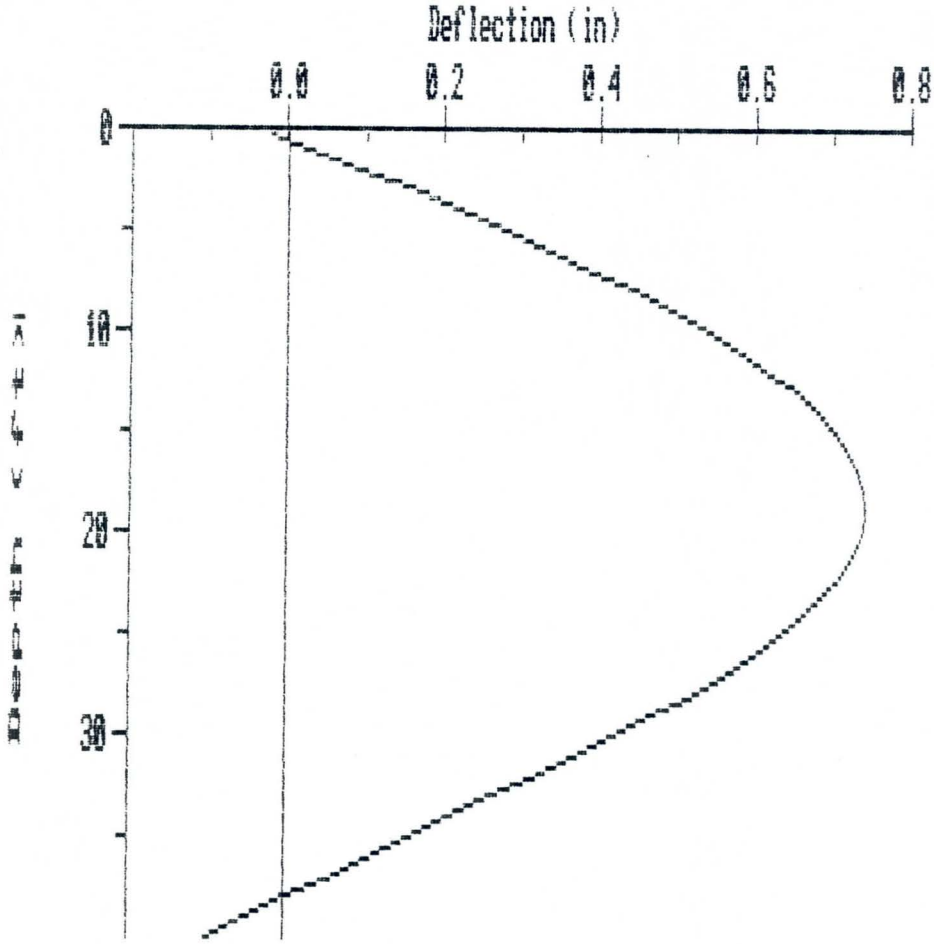
OUTPUT SUMMARY

PILE-HEAD DEFLECTION	=	-0.392D-01 IN
MAXIMUM BENDING MOMENT	=	-0.809D+07 LBS-IN
MAXIMUM SHEAR FORCE	=	0.839D+05 LBS
NO. OF ITERATIONS	=	35
NO. OF ZERO DEFLECTION POINTS	=	2

S U M M A R Y T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
-0.5300D+05	0.0000D+00	0.1440D+06	-0.3917D-01	-0.8093D+07	0.8392D+05

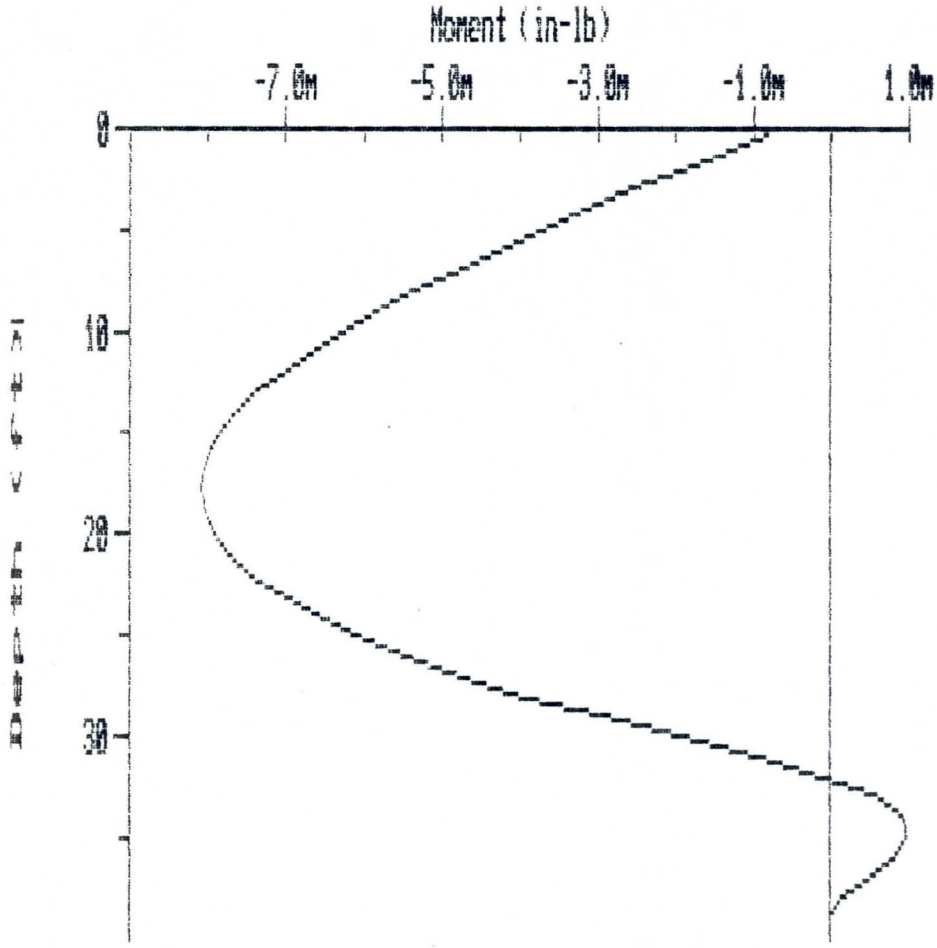
Glendale Ave. & 16th St. Bridges w/Struts, 30" Dia. Piers & 4% Steel



Deflection Curve

JOB NO. 2128J186 REVISION #1	REVIEWED R. BERGQUIST, P.E. PREPARED MAXWELL	**WESTERN TECHNOLOGIES INC.** The Quality People

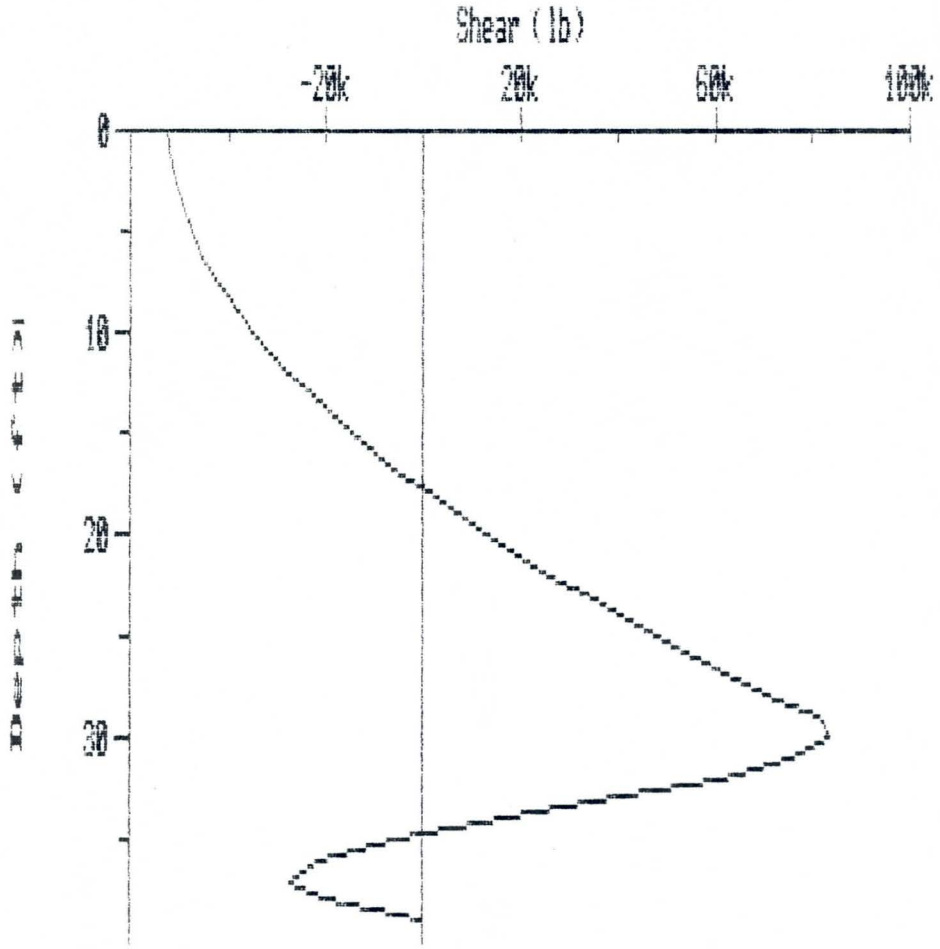
Glendale Ave. & 16th St. Bridges w/Struts, 30" Dia. Piers & 4% Steel



Moment Curve

JOB NO. 2128J186 REVISION #1	REVIEWED R. BERGQUIST, P.E. PREPARED MAXWELL	**WESTERN TECHNOLOGIES INC.** The Quality People

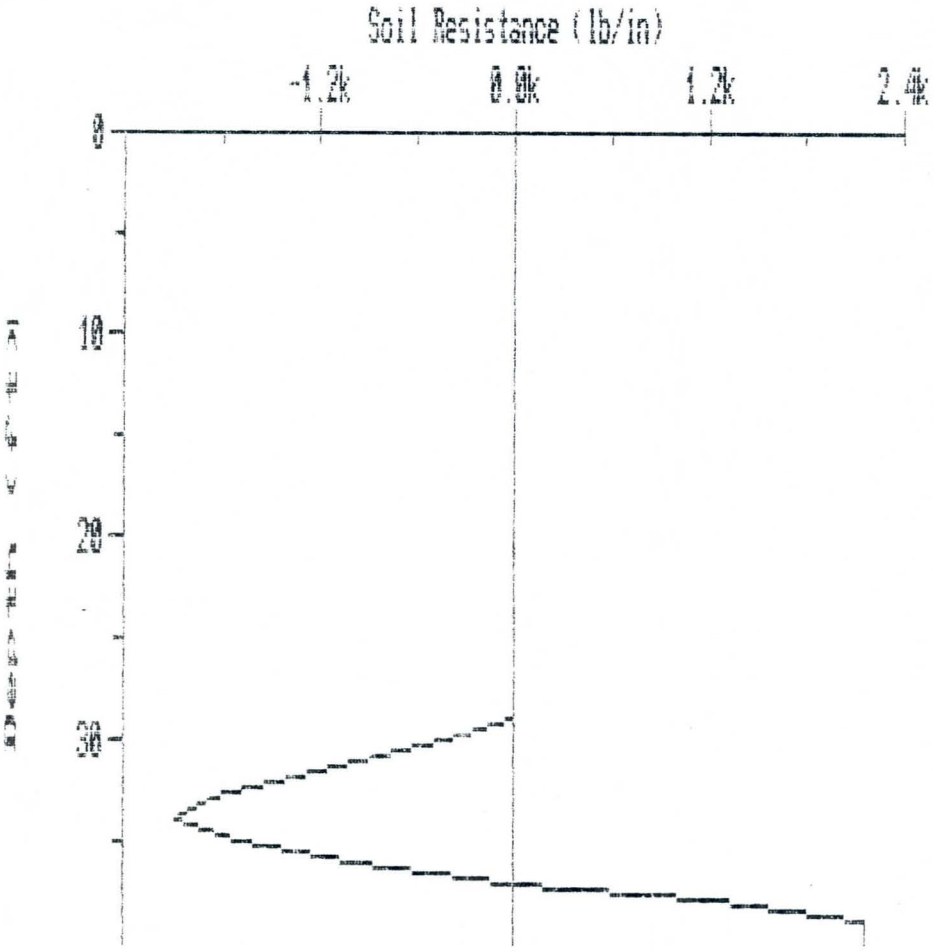
Glendale Ave. & 16th St. Bridges w/Struts, 30" Dia. Piers & 4% Steel



Shear Curve

JOB NO. 2128J186 REVISION #1	REVIEWED R. BERGQUIST, P.E. PREPARED MAXWELL	**WESTERN TECHNOLOGIES INC.** The Quality People

Glendale Ave. & 16th St. Bridges w/Struts, 30" Dia. Piers & 4% Steel



Soil Response Curve

JOB NO. 2128J186
REVISION #1

REVIEWED R. BERGQUIST, P.E.
PREPARED MAXWELL

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*              PHOENIX, ARIZONA 85040               *
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Glendale Ave & 16th St Bridges; Free-End w/36" Dia Piers & 8% Steel

UNITS--ENGLISH UNITS

I N P U T I N F O R M A T I O N

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 600.00 IN
2 POINTS

X	DIAMETER	MOMENT OF INERTIA	AREA	MODULUS OF ELASTICITY
IN	IN	IN**4	IN**2	LBS/IN**2
0.00	36.000	0.130D+06	0.102D+04	0.403D+07
600.00	36.000	0.130D+06	0.102D+04	0.403D+07

SOILS INFORMATION

```
X AT THE GROUND SURFACE                =      360.00 IN

1 LAYER(S) OF SOIL

LAYER  1
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER              =      360.00 IN
X AT THE BOTTOM OF THE LAYER           =     1000.00 IN
MODULUS OF SUBGRADE REACTION           =  0.225D+03 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                        2 POINTS
            X,IN      WEIGHT,LBS/IN**3
          360.00        0.80D-01
         1000.00        0.80D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                        2 POINTS
            X,IN        C,LBS/IN**2      PHI,DEGREES      E50
          360.00       0.000D+00        0.450D+02       -----
         1000.00       0.000D+00        0.450D+02       -----
```

BOUNDARY AND LOADING CONDITIONS

```
DISTRIBUTED LOAD CURVE              2 POINTS
                        X,IN      LOAD,LBS/IN
                        0.00        0.000D+00
                      360.00        0.620D+03


LOADING NUMBER   1

BOUNDARY-CONDITION CODE                  =          1
LATERAL LOAD AT THE PILE HEAD            =  0.000D+00 LBS
MOMENT AT THE PILE HEAD                  =  0.000D+00 IN-LBS
AXIAL LOAD AT THE PILE HEAD              =  0.000D+00 LBS


FINITE-DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                                  =        100
    DEFLECTION TOLERANCE ON DETERMINATION OF CLOSURE           =  0.100D-05 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS     =       1000
    MAXIMUM ALLOWABLE DEFLECTION                               =   0.36D+03 IN


OUTPUT CODES
    KOUTPT =  1
    KPYOP  =  0
    INC    =  2
```

O U T P U T I N F O R M A T I O N

****

DISTRIBUTED LOAD CURVE 2 POINTS

X, IN	LOAD, LBS/IN
0.00	0.000D+00
360.00	0.620D+03

LOADING NUMBER 1

BOUNDARY CONDITION CODE = 1
LATERAL LOAD AT THE PILE HEAD = 0.000D+00 LBS
MOMENT AT THE PILE HEAD = 0.000D+00 IN LBS
AXIAL LOAD AT THE PILE HEAD = 0.000D+00 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.341D+01	0.116D-03	0.000D+00	0.000D+00	0.161D-07	0.524D+12
12.00	0.331D+01	0.372D+03	0.124D+03	0.000D+00	0.515D-01	0.524D+12
24.00	0.321D+01	0.372D+04	0.496D+03	0.000D+00	0.515D+00	0.524D+12
36.00	0.311D+01	0.130D+05	0.112D+04	0.000D+00	0.180D+01	0.524D+12
48.00	0.301D+01	0.312D+05	0.198D+04	0.000D+00	0.433D+01	0.524D+12
60.00	0.291D+01	0.614D+05	0.310D+04	0.000D+00	0.850D+01	0.524D+12
72.00	0.281D+01	0.106D+06	0.446D+04	0.000D+00	0.147D+02	0.524D+12
84.00	0.271D+01	0.169D+06	0.608D+04	0.000D+00	0.234D+02	0.524D+12
96.00	0.261D+01	0.253D+06	0.794D+04	0.000D+00	0.350D+02	0.524D+12
108.00	0.251D+01	0.360D+06	0.100D+05	0.000D+00	0.499D+02	0.524D+12
120.00	0.241D+01	0.495D+06	0.124D+05	0.000D+00	0.685D+02	0.524D+12
132.00	0.231D+01	0.659D+06	0.150D+05	0.000D+00	0.912D+02	0.524D+12
144.00	0.221D+01	0.856D+06	0.179D+05	0.000D+00	0.118D+03	0.524D+12
156.00	0.211D+01	0.109D+07	0.210D+05	0.000D+00	0.151D+03	0.524D+12
168.00	0.201D+01	0.136D+07	0.243D+05	0.000D+00	0.188D+03	0.524D+12
180.00	0.191D+01	0.167D+07	0.279D+05	0.000D+00	0.232D+03	0.524D+12
192.00	0.181D+01	0.203D+07	0.317D+05	0.000D+00	0.281D+03	0.524D+12
204.00	0.171D+01	0.243D+07	0.358D+05	0.000D+00	0.337D+03	0.524D+12
216.00	0.161D+01	0.289D+07	0.402D+05	0.000D+00	0.400D+03	0.524D+12
228.00	0.152D+01	0.340D+07	0.448D+05	0.000D+00	0.471D+03	0.524D+12
240.00	0.142D+01	0.397D+07	0.496D+05	0.000D+00	0.549D+03	0.524D+12
252.00	0.133D+01	0.459D+07	0.547D+05	0.000D+00	0.636D+03	0.524D+12
264.00	0.124D+01	0.528D+07	0.600D+05	0.000D+00	0.731D+03	0.524D+12
276.00	0.114D+01	0.603D+07	0.656D+05	0.000D+00	0.835D+03	0.524D+12
288.00	0.105D+01	0.685D+07	0.714D+05	0.000D+00	0.949D+03	0.524D+12
300.00	0.965D+00	0.775D+07	0.775D+05	0.000D+00	0.107D+04	0.524D+12
312.00	0.879D+00	0.871D+07	0.838D+05	0.000D+00	0.121D+04	0.524D+12
324.00	0.796D+00	0.976D+07	0.904D+05	0.000D+00	0.135D+04	0.524D+12
336.00	0.715D+00	0.109D+08	0.972D+05	0.000D+00	0.151D+04	0.524D+12
348.00	0.637D+00	0.121D+08	0.104D+06	0.000D+00	0.167D+04	0.524D+12
360.00	0.563D+00	0.134D+08	0.112D+06	0.000D+00	0.185D+04	0.524D+12
372.00	0.492D+00	0.147D+08	0.111D+06	-0.512D+03	0.204D+04	0.524D+12
384.00	0.425D+00	0.160D+08	0.101D+06	-0.114D+04	0.222D+04	0.524D+12
396.00	0.362D+00	0.171D+08	0.831D+05	-0.178D+04	0.237D+04	0.524D+12
408.00	0.305D+00	0.180D+08	0.582D+05	-0.235D+04	0.249D+04	0.524D+12
420.00	0.252D+00	0.185D+08	0.273D+05	-0.280D+04	0.256D+04	0.524D+12
432.00	0.204D+00	0.186D+08	-0.828D+04	-0.309D+04	0.258D+04	0.524D+12
444.00	0.162D+00	0.183D+08	-0.459D+05	-0.305D+04	0.253D+04	0.524D+12
456.00	0.124D+00	0.175D+08	-0.804D+05	-0.268D+04	0.243D+04	0.524D+12
468.00	0.913D-01	0.164D+08	-0.110D+06	-0.222D+04	0.227D+04	0.524D+12
480.00	0.631D-01	0.149D+08	-0.133D+06	-0.170D+04	0.207D+04	0.524D+12
492.00	0.389D-01	0.132D+08	-0.151D+06	-0.116D+04	0.183D+04	0.524D+12
504.00	0.184D-01	0.113D+08	-0.161D+06	-0.596D+03	0.157D+04	0.524D+12
516.00	0.999D-03	0.936D+07	-0.165D+06	-0.351D+02	0.130D+04	0.524D+12
528.00	-0.138D-01	0.739D+07	-0.162D+06	0.523D+03	0.102D+04	0.524D+12
540.00	-0.266D-01	0.550D+07	-0.152D+06	0.108D+04	0.761D+03	0.524D+12
552.00	-0.379D-01	0.376D+07	-0.136D+06	0.164D+04	0.520D+03	0.524D+12
564.00	-0.482D-01	0.225D+07	-0.113D+06	0.221D+04	0.312D+03	0.524D+12
576.00	-0.578D-01	0.107D+07	-0.829D+05	0.281D+04	0.148D+03	0.524D+12
588.00	-0.671D-01	0.284D+06	-0.454D+05	0.344D+04	0.394D+02	0.524D+12
600.00	-0.764D-01	0.000D+00	0.000D+00	0.412D+04	0.000D+00	0.524D+12

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -0.116D-03 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -0.102D-04 LBS

**** WARNING ****
MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -0.116D-03 IN-LBS

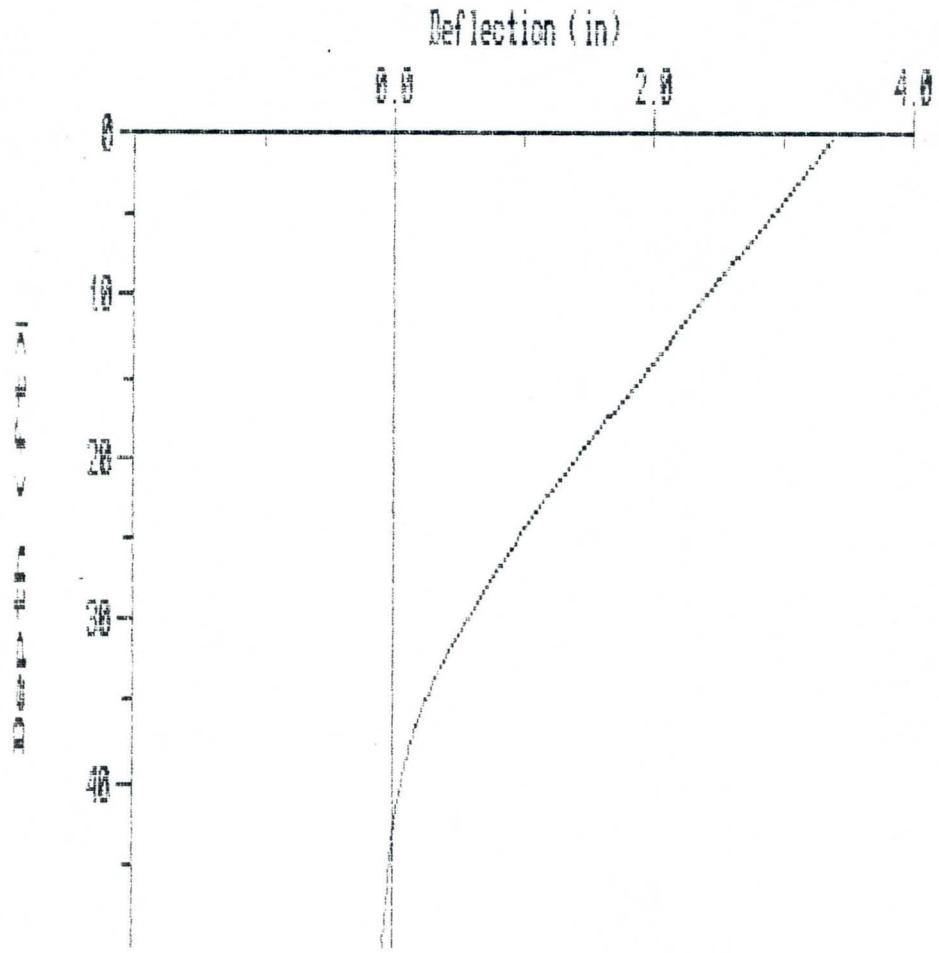
OUTPUT SUMMARY

PILE-HEAD DEFLECTION	=	0.341D+01 IN
MAXIMUM BENDING MOMENT	=	0.186D+08 LBS-IN
MAXIMUM SHEAR FORCE	=	-0.165D+06 LBS
NO. OF ITERATIONS	=	17
NO. OF ZERO DEFLECTION POINTS	=	1

S U M M A R Y T A B L E
****

BOUNDARY CONDITION BC1	BOUNDARY CONDITION BC2	AXIAL LOAD LBS	PILE HEAD DEFLECTION IN	MAX. MOMENT IN-LBS	MAX. SHEAR LBS
0.0000D+00	0.0000D+00	0.0000D+00	0.3408D+01	0.1863D+08	-0.1649D+06

Glendale Ave. & 16th St. Bridges Free-End w/36" Dia. Piers & 8% Steel



Deflection Curve

JOB NO. 2128J186
REVISION #1

REVIEWED R. BERGQUIST, P.E.
PREPARED MAXWELL

WESTERN TECHNOLOGIES INC.
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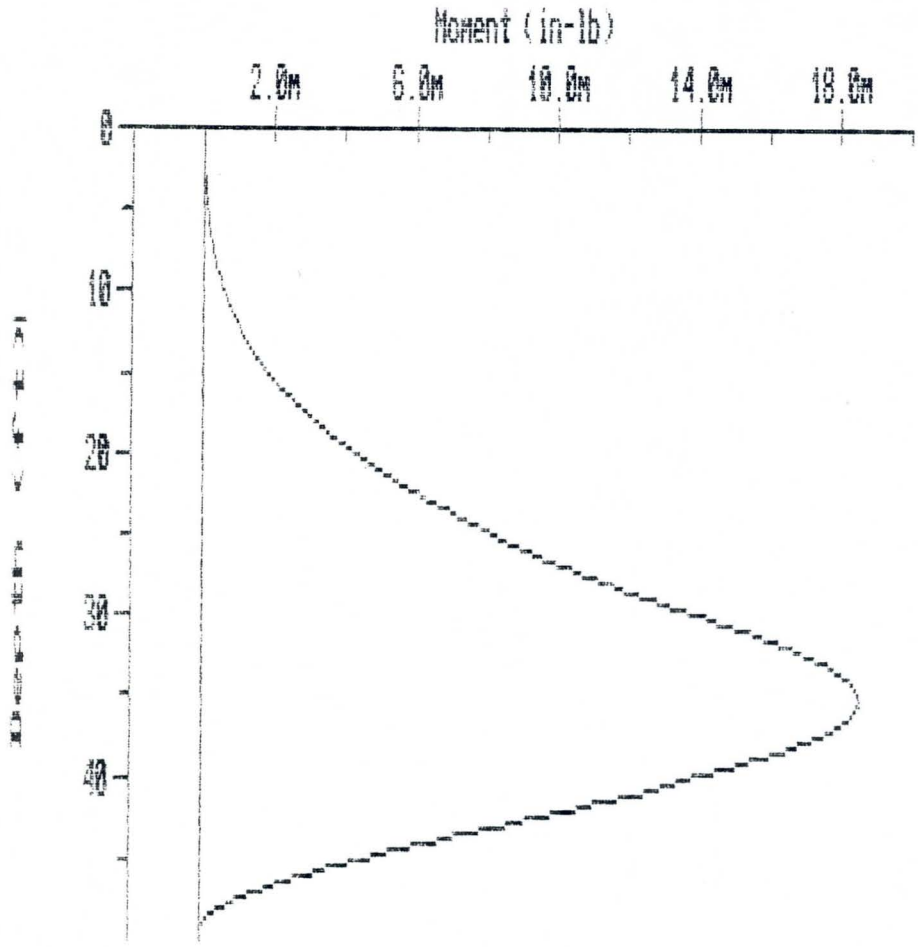
Glendale Ave. & 16th St. Bridges Free-End w/36" Dia. Piers & 8% Steel



Moment Curve

JOB NO. 2128J186
REVISION #1

REVIEWED R. BERGQUIST, P.E.
PREPARED MAXWELL

WESTERN TECHNOLOGIES INC.
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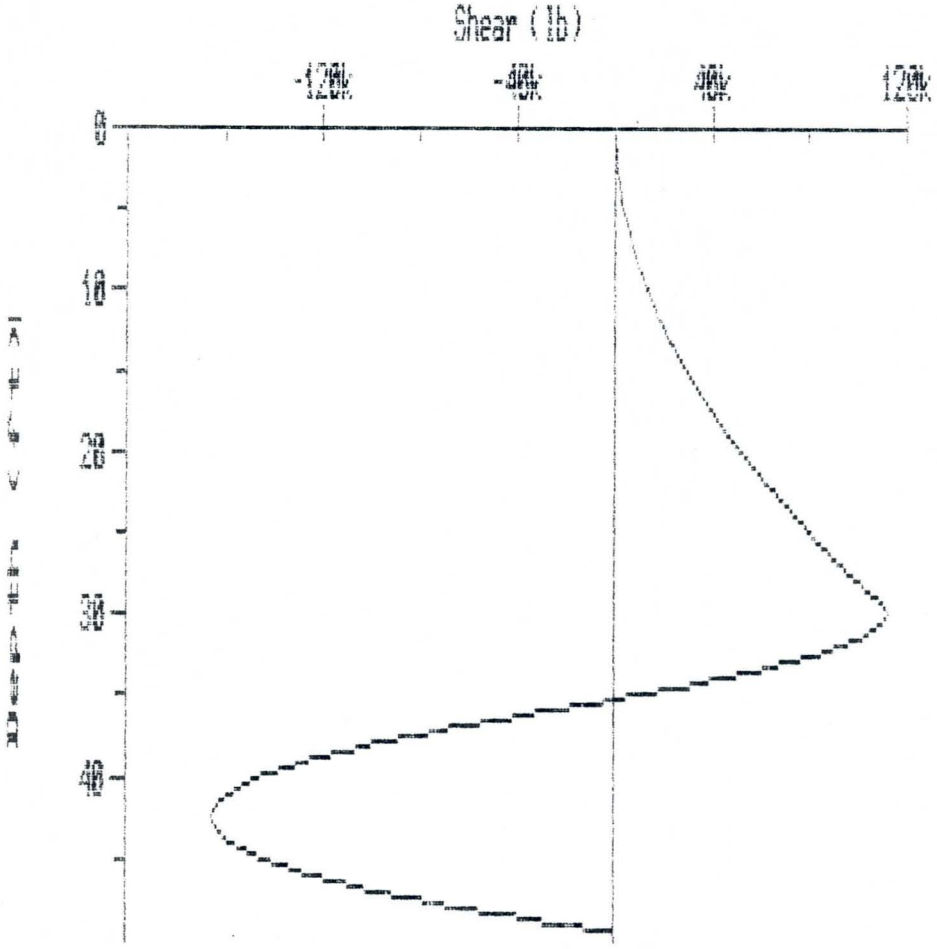
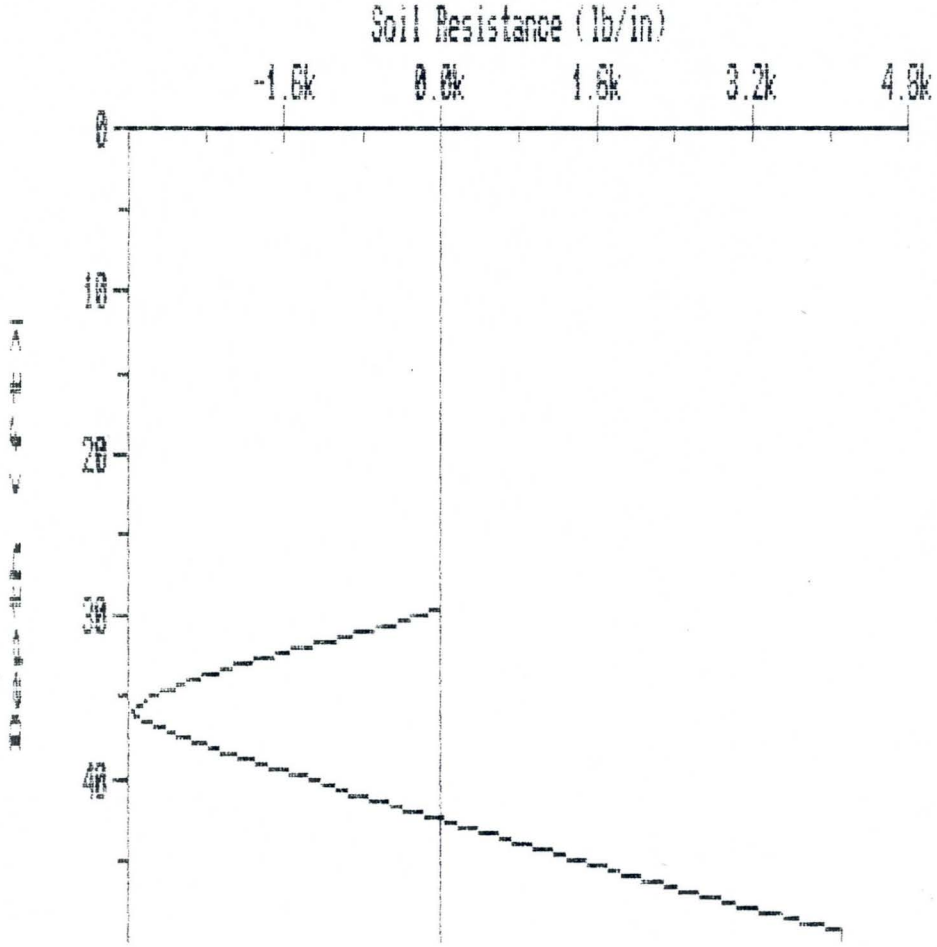
Glendale Ave. & 16th St. Bridges Free-End w/36" Dia. Piers & 8% Steel



Shear Curve

JOB NO. 2128J186 REVISION #1	REVIEWED R. BERGQUIST, P.E. PREPARED MAXWELL	**WESTERN TECHNOLOGIES INC.** The Quality People

Glendale Ave. & 16th St. Bridges Free-End w/36" Dia. Piers & 8% Steel



Soil Response Curve

JOB NO. 2128J186 REVISION #1	REVIEWED R. BERGQUIST, P.E. PREPARED MAXWELL	**WESTERN TECHNOLOGIES INC.** The Quality People

References Used:

1. "Foundations and Earth Structures" – NAVFAC DM-7.2, May 1982
2. "Pile Foundation Analysis and Design" – Poulos and Davis.
3. "Bearing Capacity and Settlement of Pile Foundations" – Meyerhof, ASCE, GT3, 1976

A. 16th Street Bridge:

1. Choose the soil strength Parameter:

Based on direct shear tests on the assemblage material of Fanglomerate at the proposed foundation level,

$\phi = 37^\circ$
$c = 0$
$\gamma = 125$ pcf
Modulus of deformation of soil, E_s
$E_s = 20000$ psi (Chart table 2-3, Page 35 Bowles, "Foundation Analysis and Design".)

2. Determine the allowable bearing capacity

According to Nicholas P. Karan, P.E. chief

WESTERN TECHNOLOGIES INC.

Engineering Division, FCD of Maricopa County, the pier should be designed as end bearing only. It was decided on Jan. 23, 1989 meeting of the analyst with Mr. Karan that when calculating bearing capacity, the effect of foundation size and cohesion on bearing capacity will not be taken into consideration.

$$\therefore \quad q_{ult} = \bar{q} \, N_q$$

q_{ult} = ultimate bearing capacity

$\bar{q}$ = surcharge = $D * \gamma$, D = depth of foundation below finished grade

γ = Unit weight of surcharge

N_q = Bearing capacity factor.

In the following lines q_{ult} is calculated using different chart for ϕ vs. N_q.

a. NAVFAC: (Factor of Safety used = 3.0)

$$q_{ult} = 10 * 125 * 38 \text{ psf}$$
$$= 47,500 \text{ psf}$$

$$q_{all} = 15.8 \text{ ksf} = 16 \text{ ksf} \checkmark$$

JAE 2/16/89

b. Use Poulos and Davis book (Berezantzev's Chart)

$q_{ult} = 10 * 125 * N_q$

N_q for $(\phi - 3) = 34° = 60$

(Note: The Berezantzev's chart suggest to use $(\phi - 3)$ for drilled foundation)

$\therefore q_{ult} = 10 * 125 * 60 = 75000$ psf

with FS = 3.0

$q_{all} = 25000$ psf = $\boxed{25 \text{ ksf}}$ ✓

c. Using Meyerhof's Chart

$$\frac{D_b}{B} = \frac{\text{Depth of pier tip}}{\text{Width of foundation}} = \frac{10}{3} = 3.3$$

Use $\phi = \phi_{actual} - 3$ for drilled shaft

$= 34°$

$\therefore N_q = 60$ (same as Berezantzev)

$\therefore q_{all} = \boxed{25 \text{ ksf}}$

Use $q_{all} = 25$ ksf

JHE 2/14/89

WESTERN TECHNOLOGIES INC.

3. Calculate the axial Capacity

$Q_a = q_{all} * Area$

$= 25 * \pi/4 \, D^2 = 25 * .785 * 9$

$= \boxed{177 \text{ kips}}$ ✓

The analyst / designer anticipate that the proposed 3' diameter drilled pier will support the 181 kip load and under the existing subsurface condition a bearing capacity failure is unlikely. ✓

4. Calculate Settlement

Use NAVFAC criteria and calculate settlement for a shallow foundation of same size and bearing pressure. The deep foundation is expected to settle not more than half the settlement of equivalent shallow foundation. Also incorporate creep for 10 years.

Two different methods of calculations for settlement is performed in following pages. The creep for 10 years is then applied to the largest of these settlement

JAR 2/16/89

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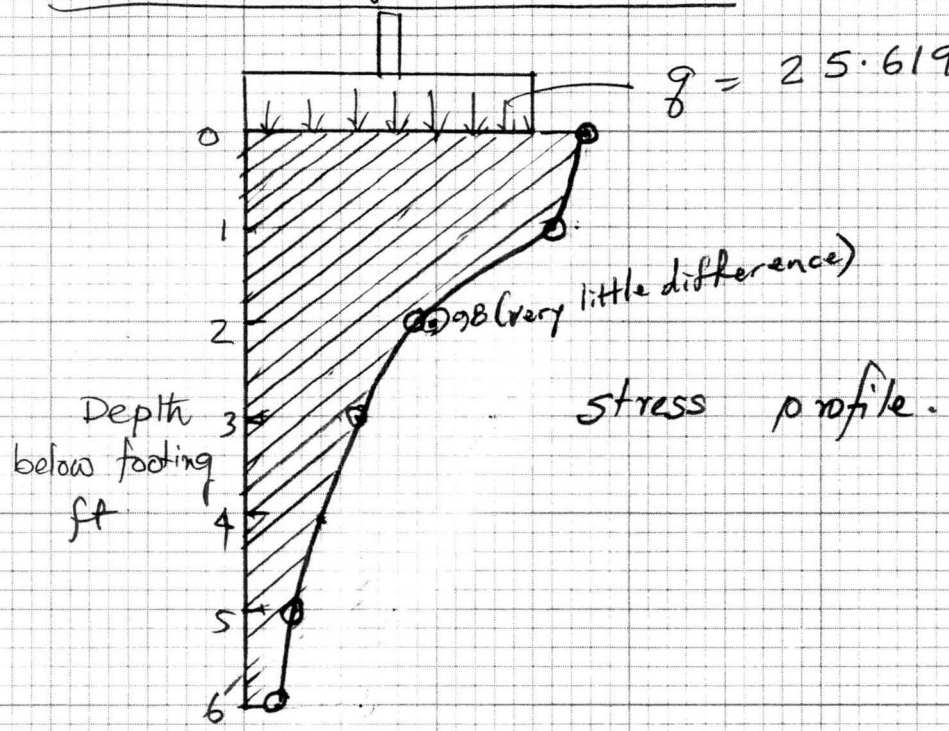
Project/Subject 16th Street and Glendale Avenue Bridge w/ strut, 36" ϕ piers, 4% steel Bearing Capacity and Settlement

Job No. 2128 J186

Date 1/31/89 Sheet 5 of 35

By E. Hoque

a. Stress Integration Method



ⓐ Calculate stress at depth of 1', 3', 5' and 6' below the foundation

Depth,	stress due to foundation.
0	178 psi
1	160 ✓
2	89 (98)
3	57.0 ✓
4.0	35.6 ✓
5	22.5 ✓
6	16.9 "

Sample calculation of stress: CI from Fig-5 chp. 4, NAVFAC DM 7.1

at: depth 3' below footing

$\frac{z}{r} = \frac{3}{1.5} = 2.0$; $\frac{x}{r} = 0$ ∴ $I = .32$

∴ $\sigma = 57$ (0.32×178) ✓

JMP 2/16/89

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Settlement is

$$\therefore S = \left(\frac{\Delta q}{E_s}\right)_{average} * L$$

Δq = Average increase in stress in the underlying strata

$$= \frac{1}{6}\left(\frac{178+16.9}{2} + 160 + 89 + 57 + 35.6 + 22.5\right)$$

$$= \boxed{76.9 \text{ psi}}$$

$$\therefore S = \frac{76.9 * 6 * 12}{20{,}000} = \boxed{0.28''}$$

(b) Normal Elastic half space method:

$E_s = 20000$ psi

$$\therefore S = qB \cdot \frac{1-\mu^2}{E_s} I_w$$

($I_w = 0.88$ for circular rigid foundation)

$$= 178 * 3 * 12 * \frac{1 - .32}{20{,}000} * 0.88$$

$$= \boxed{0.26''}$$

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Now apply 10 years creep correction to the estimated largest settlement of $\boxed{0.28''}$

The creep correction factor is

$$C_2 = 1 + 0.2 \log (10t)$$

t = elapsed time in years

$$\therefore C_2 = 1 + 0.2 \log (10 * 10)$$

$$= 1.4$$

$\therefore$ Total settlement at the end of 10 years is expected to be

$$S_{t_{10}} = 1.4 \times 0.28 = 0.39''$$

The above calculated settlement is for an equivalent shallow foundation. The deep foundation is expected to undergo a settlement of half of that calculated above i.e.

$$(S_{t_{10}})_{Deep} = 0.195 = 0.20''$$

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References: same as before.

Soil Parameters: same as before.

A. Bearing Capacity

a. Use NAVFAC method.

$q_{ult} = \bar{q} N_q$, $\phi = 37°$, $N_q = 38$

$= 10 * 125 * 38$

$= 47,500$ psf.

$q_{all} = 15.8 = 16$ ksf with a factor of safety of 3.0 ✓

b. Poulos and Davis book and use Berezantzev's Chart for N_q.

According to Berezantzev use angle of interfriction, $\phi_{used} = \phi - 3$ for drilled foundations

$\therefore N_q = 60$

$\therefore q_{ult} = 10 * 125 * 60 = 75000$ psf

$\boxed{q_{all} = 25 \text{ ksf}}$ ✓

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Total load = 144 Kips

Area = $\pi/4\, D^2$ = 4.906 □'

∴ Intensity of pressure = 29.4 ksf
> allowable bearing capacity.

Note: The foundation analyst anticipate that the proposed 2½ foot diameter drilled pier will support 144 kips total load without bearing capacity failure. The anticipation is based on existing subsurface conditions. Settlement is calculated below. ok ~~As~~ Factor of Safety @ 29.4 ksf = $3.0\left(\frac{25}{29.4}\right) = 2.55 \approx \boxed{2.6}$

B. <u>Settlement</u>:

Assume E_s = modulus of elasticity of soil
= 20,000 psi.

a. By using stress-integration techniques. Calculate stress for 1 foot increment to a depth of 5 feet.

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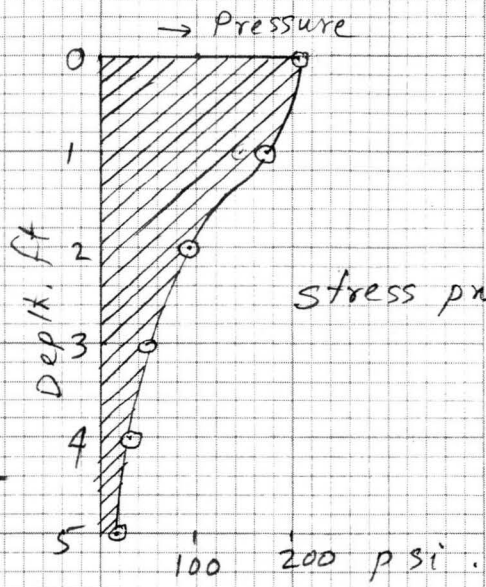
Depth below foundation bottom feet	Stress increment psi
0	204.2 ✓
1	171.5 ✓
2	91.9 ✓
3	47 ✓
4	28.6 ✓
5	19.4 ✓



stress profile.

Average incremental stress is

$$(\Delta q)_{av} = \frac{1}{5}\left(\frac{204.2+19.4}{2} + 171.5 + 91.9 + 47 + 28.6\right)$$

$$(\Delta q)_{av} = 90.16 \text{ psi} \checkmark$$

$$\therefore \text{Settlement, } S = (\Delta q)_{av} \cdot \frac{L}{E_s} ; \frac{90.2\,(5)(12)}{20{,}000}$$

$$= \boxed{0.27''} \checkmark$$

(b) Elastic half space method

$$S = qB \cdot \frac{1-\nu^2}{E_s} I_w$$

JAE 2/16/89

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$$\therefore S = 204.2 * 2.5*12 * \frac{.91}{20,000} * 0.88$$

$$= 0.245'' = \boxed{0.25''} \checkmark$$

(c) Also check settlement using Modulus of subgrade reaction procedure.

Assume $K_s = 350$ tons / cft

300 T/ft³ = 347 pci

$$= \frac{350 \times 2000}{1728} \text{ pci}$$

$$= 405 \checkmark \text{ pounds / cubic inches.}$$

∴ For Deep footing settlement @ D = ~~4B~~ ~~5B~~ 5B

$$\Delta H_i = \frac{2*204.2*(2.5\times12)^2}{347*((2.5+1)\times12)^2}$$

$$\frac{2qD^2}{K_s(D+1)^2}$$

$$= \cancel{0.51\ (0.51)}\ \ 0.60 @ K_s = 347 \text{ pci},\ 0.51 @ K_s = 405 \text{ pci}$$

D = 12.5'

Shallow D = B = 2.5'

$$\Delta H_i = \frac{4qB^2}{K_v(B+1)^2} = \cancel{1.0''}\ \ 1.2'' @ K_s = 347 \text{ pci},\ 1.0'' @ K_s = 405 \text{ pci}$$

Interpolating between ~~D = 4B and~~ D = 5B

∴ Actual = 0.60 @ K_s = 405 pci
0.75 @ K_s = 347 pci

Actual Ks may be higher. So settlement may be less than 1/2" OK

JHE 2/10/89

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Project/Subject 16th Street and Glendale Ave. Bridge W/struts, 36" dia Piers, 4% steel Slope Stability Within the Piers. Job No. 2128J186 Date 2/3/89 Sheet 13 of 35 By E. Hogue

Soil Parameters:

Use the direct shear test results on -#4 material within the proposed excavation showed the following soil parameters.

Glendale Avenue	16th Street
$\phi = 37^\circ$	$\phi = 35^\circ$
$c = 300$ psf	$c = 300$ psf
$\gamma = 125$ pcf	$\gamma = 125$ pcf

It seems that one analysis will suffice for both the cases.

Several different approaches has been used for this analysis. It is to be mentioned here that in this analysis, no credit is given to the angular cobbles and gravel sized particles which are calchified moderately and lightly. Recent excavation conditions were observed by the geotechnical engineer which is located within 1/4 mile of the project area. The vertical excavation on those sites in similar soil condition remained stable even when nearby excavation were blasted using explosives. The available literature (Reference: Douglas Carl Shank "Environmental Geology ~~of~~ In the Phoenix Mountains, Maricopa County, Az", ASU, Thesis, 1973) reveals that relatively steep slopes of moderate height will possibly remain stable for short time. Minor slumping is expected

JEH 2/6/89

Glendale Avenue Bridge

Direct shear test results on #4 material in this site shows that an angle of internal friction of 37° can be used for this site. In this perspective the calculations and hence bearing capacity and settlement will be of same magnitude.

JAE
2/16/89

The analysis

A. Janbu charts:

Factor of Safety, $F_S = N_{CF} \cdot \frac{C}{\gamma H}$

We have $\lambda_{c\phi} = \frac{\gamma H \tan\phi}{c}$

$c = 300$

$\therefore \lambda_{c\phi} = \frac{125 \times 30 \times \tan 35}{300} = 8.75$ ✓

We have $\lambda = 8$ maximum on figure

$b = 0$

$\therefore N_{CF} = 8.7$

$\therefore F_S = 8.7 * \frac{300}{125 \times 30} = 0.7$ ✓

(B) Sanglerat, Olivari, and Cambou

$\phi = 35°$, $c = 300$ psf

"Practical Problems in Soil Mechanics and Foundation Engineering - 2", 1985 Elsevier.

Page 208. Assume $\phi = 0$

for circular failure $F.S. = \frac{4.75 \times 300}{125 \times 30}$ ✓

JPE 2/16/89

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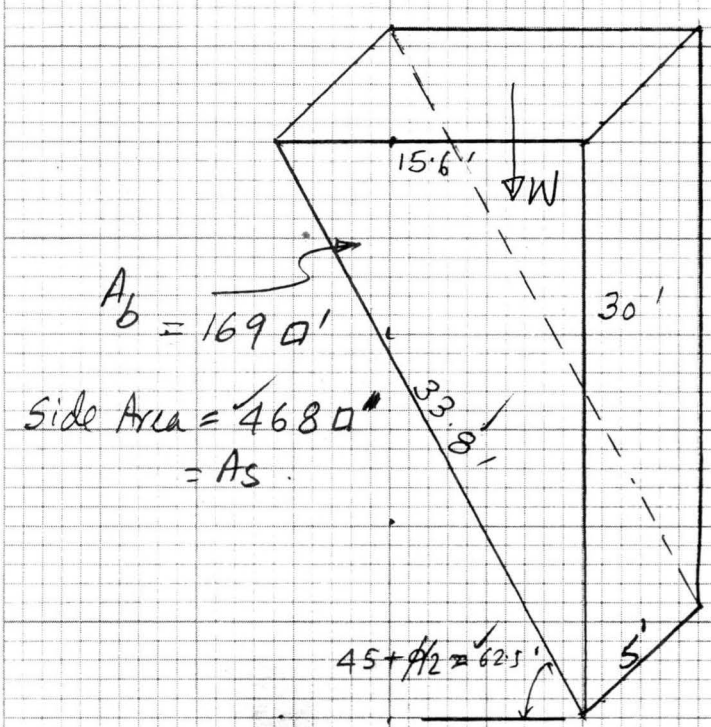
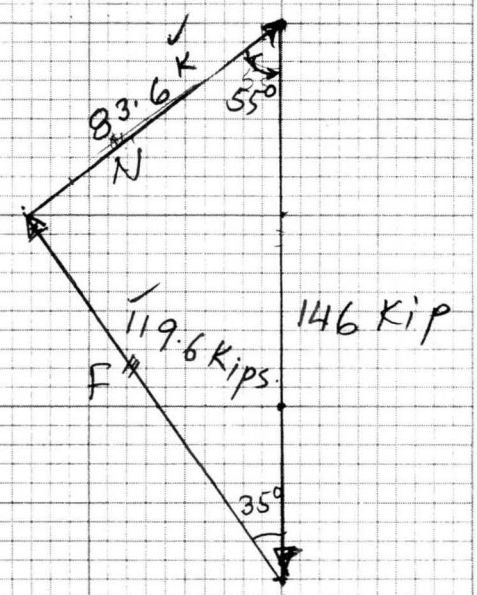
$F_s = 0.40 \ll 1.6$.

C Wedge Analysis.

Use wedge Analysis from Terzaghi, "Theoretical Soil Mechanics", 1943. P. 182





$W = \frac{1}{2} \times 5 \times 30 \times 15.6 \times 125$

$= 146$ kips.

No friction along sides.

$F_{max} = N \tan\phi + c A_b + (C_a * A_s)$ Not used here

$= 83600 * 0.70 + 300 * 169 = 109220$ lb

JAE 2/16/89

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Actual Friction that will develop due to the weight of the block = 119.6 Kips
= 120,000 lbs.

$\frac{109}{120}$

$\therefore$ F.S. = 0.91 < 1.0.

2.0

At the two extreme wedge, there will be some resistance in sides. Also due to the presence of the pier in between the wedges, there will be side friction.

2/6/89; at 4:00 p.m. talked to Mr. Nicholas P. Karan. He suggested or he has the opinion to use the resistance agaist failure along side only due to the cohesion or cementation. He also suggested to use a liberal safety factor — at least 2.0 in this case.

$\therefore$ Using side resistance:

F_{max} = 83600 × 0.70 + 300 × 169 + 468 × 300

= 249620 lb

PAC 2/16/89

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Project/Subject _____ Job No. _____

Date 2/6/89 Sheet 17 of 35

By _____

$$\therefore \quad F.S. = \frac{249620}{119600} = 2.09 = 2.10$$

O.K

JAE
2/10/89

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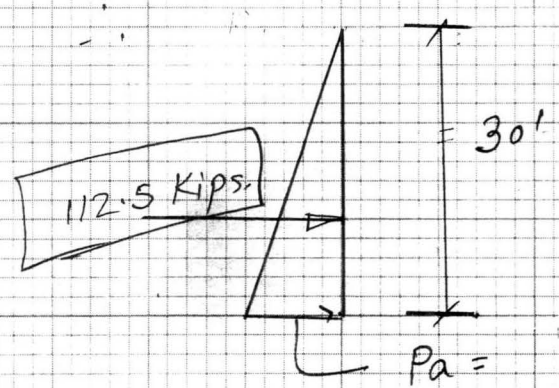
Given: Pile diameter = 36"
Moment of Inertia = 1.30×10^5 in^4
Area = 1020 in^2
Modulus of elasticity, $E = 4.03 \times 10^6$ psi
Spacing = 8'
Vertical Load, (Dead + Live) load = 181 Kips.

Soil Parameters:
Angle of internal friction = 37°
Unit weight of soil = 125 pcf.
Coefficient of Lateral subgrade reaction = 225 pci

A. Calculate Lateral pressure on Piles with free end. Active Lateral pressure will prevail.

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.249 \simeq 0.25$$



$P_a = \gamma \cdot h \cdot K_a = \times 125 \times 30 \times 0.25$
$= 937.5$ Pounds/ft.

$\therefore$ Total $P_a = \frac{1}{2} \cdot P_a \cdot 8 \cdot h =$ **112.5 Kips**

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B. Calculate Applied Moment

$M = P_a * L_m$ L_m = Moment arm

$= 112.5 \times 10 = \boxed{1125 \text{ k-ft}}$ ✓

C. Calculate relative stiffness factor

$$T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{4.03\times10^6 * 1.3\times10^5}{225}\right)^{1/5}$$

use $f = 60 T/ft^3 = 0.0347 \text{ k/in}^3$

from NAVFAC 7.2-236 Fig 9

This change made 2/28/89 MAE

$= \boxed{\cancel{74.7}\ 94.4}$ inch.

D. Check the criteria whether the piers will act as long piles or short piles.

L = depth of embedment

$= 20' = 20 \times 12 = 240$ inches.

Piers will behave as short pile if $L < 2T$; Long pile if $L > 4T$

$T = \cancel{74.7}\ 94.4$

$L/T = \cancel{3.2}\ 2.5$

$\boxed{\text{which falls in intermediate zone.}}$

∴ We can use NAVFAC charts for lateral analysis.

MAE 2/21/89

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E. Calculate Maximum deflection:

As these piers are free end, maximum deflection within the pier embedment is expected to occur at ground surface. Therefore deflection at ground surface is calculated.

Use Fig-11, NAVFAC, DM 7.2, Page 238.

$z = 0$;

Deflection: $\delta = (F_\delta)_P \cdot \frac{PT^3}{EI} + (F_\delta)_M \cdot \frac{MT^2}{EI}$

$$= \cancel{2.5}\ 3.5 * \frac{112500 * (\cancel{74.7}\ 94.4)^3}{4.03\times10^6 * 1.3\times10^5} + \cancel{1.7}\ 2.5 * \frac{1125000\times12 * (\cancel{74.7}\ 94.4)^2}{4.03\times10^6 * 1.3\times10^5}$$

$$= \cancel{0.220}\ \cancel{0.181}\ \cancel{0.452} + \cancel{0.24}\ \cancel{0.230}\ \cancel{0.390}\quad 0.632 + 0.574$$

$$= \cancel{0.468}\ \cancel{0.842}\ 1.206 \text{ inche} \simeq \boxed{\cancel{0.47}\ \cancel{0.84}\ 1.21''}$$

This deflection is expected to occur at the dredge line or along the canal bottom.
LPILE has deflection @ 0.56"; reasonably close (factor of 2) 2/20/89

F. Calculate Moment on pier due to Lateral soil pressure:

The moments are calculated at or below the proposed dredge line or canal bottom.

at surface: $z = 0$

$\therefore M = \boxed{1125000 \text{ lb-ft}}$ 2/22/89

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Project/Subject Glendale Avenue and 16th Street Bridges Job No. 212 8J186
36" ϕ, 8 ~~#~~% steel ~~with Strut~~, free end. Date 2/16/89 Sheet 21 of 35
Lateral Pile Analysis. By E. Hoque

It appears from the NAVFAC charts that maximum moment may occur at $\frac{z}{T} = 0.8$ to 1.2

∴ Moments are calculated at these locati-

(a) at $\frac{z}{T} = 0.8$; depth ≅ ~~5.0~~ 6.3 feet :

∴ $M = \cancel{0.69}\ 0.57 * 112500 * \frac{\cancel{74.7}\ 94.4}{12} + \cancel{0.9}\ 0.85 * 1125000 * 12$

$= \boxed{\cancel{1423}\ 1461 \quad \cancel{1496}\ \text{K-ft}}$ ✓

(b) at $\frac{z}{T} = 1.2$; depth ≅ ~~7.5~~ 9.4 feet :

∴ $M = \left(\cancel{0.75}\ 0.58 * 112500 * \frac{\cancel{74.7}\ 94.4}{12}\right) + \left(\cancel{0.75}\ 0.67 * 1125000 * 12\right)$

$= \boxed{\cancel{1508}\ 1267 \quad \cancel{1369}\ \text{K-ft}}$ ✓

~~LPILE~~ LPILE has max. moment @ 1553 k-ft ; reasonably close & depth = 6' MR 2/20/89

G Calculate Fixity :

Fixity is defined here to be a point where the moment is numerically zero. From the chart of NAVFAC it looks like moment will be zero around:

~~By insp~~ $\frac{z}{T} = 2.8$ and pier tip. By inspection : M = 0 @ pier base NAVFAC 7.2-238 Fig. 11

at $z/T = 2.8$ → z = 22.0 (not possible) calculate moment

Now $\left(M = 0.02 * 112.5 * \frac{\cancel{74.7}\ 94.4}{12} + .015 * 1125 = \cancel{31}\ 34.6\ \text{K-ft}\right)$

Depth = 17. feet MR 2/22/89

LPILE has M=0 @ base of pier (depth = 20 ft) ; MR 2/20/89

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at a ~~d~~ $z/T = 3.2$;

$M = 0 * PT + 0 * PT = 0$;

it appears that fixity is at about a depth of 18 feet below canal bottom. ✓

(H) Calculate Shear. (Maximum):

(a) at canal bottom i.e. dredge line

$V = 112.5$ Kips.

(b) at $z/T = 0.5$; depth = ~~8~~ 3.9 feet.

$V = \frac{0.75}{\cancel{0.8}} * 112.5 + \left(\frac{-0.23}{\cancel{-0.15}}\right) * \frac{1125 * 12}{\cancel{74.7}\ 94.4}$

$= \cancel{62.9}$ Kips ✓

(c) at $z/T = 1.5$ depth = ~~9.3~~ 11.8 feet.

$V = \left(\frac{-0.50}{\cancel{-0.25}} * 112.5\right) + \left(\frac{-0.69}{\cancel{-0.52}} * \frac{1125 * 12}{\cancel{74.7}\ 94.4}\right)$

$= \cancel{-122}$ Kips. ✓

(d) at $z/T = 2.0$ depth = ~~12.5~~ 15.7 feet

$V = \left(\frac{-0.28}{\cancel{-0.51}} * 112.5\right) + \left(\frac{-0.32}{\cancel{-0.50}} * \frac{1125 * 12}{\cancel{74.7}\ 94.4}\right)$

$= \cancel{-148}$ Kips ✓

TAE 2/22/89

LPILE has max. shear @ ~~-164.9 kips~~, reasonably close TAE 2/28/89

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Fill and Soil Properties are same as before.

Sectional Properties of Pile:

Dia = 36" = D.

Moment of Inertia of Pile = 1.02×10^5 in^4

Area of pier section = 1020 in^2

Modulus of elasticity $E = 4.03 * 10^6$ psi

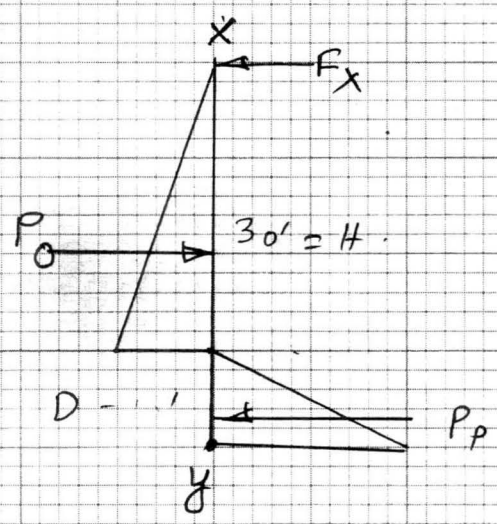
Free height above canal bottom = 30'

Depth of embedment = 10' below canal bottom.

Center to center spacing between piers = 8'

These piers will be restrained at the bridge girder level. Therefore, it is assumed that at-rest pressure condition will prevail.

$K_o = 1 - \sin\phi = 1 - \sin 37° = 0.398 \simeq 0.4$



$P_o = \frac{1}{2} * K_o * \gamma * h * h * 8$

$= 180$ kips ✓

$M_x = 180 * 20 = 3600$ k-ft

used 790 for computer runs, Di

$P_p = D * 500 * \frac{1}{2} \cdot D * 3$

$= 750 D^2$ ✓

$M_{px} = \left(30 + 2\frac{D}{3}\right) 750 D^2$ ✓

$= 22500 D^2 + 500 D^3$ ✓

@ D = 8': $M_{px} = 1696$ k-ft ✓

@ D = 10': $M_{px} = 2750$ k-ft ✓

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JHE 2/22/89

JHE 2/28/89

@ D = 12' : M_{px} = 4104 k-ft ✓

@ D = 11' M_{px} = 3388 k-ft ✓

@ D = 11.75' M_{px} = 3917 k-ft ✓

@ D = 11.5' M_{px} = 3736 k-ft ✓

Use 11.4 M_{px} = 3665 k-ft ✓

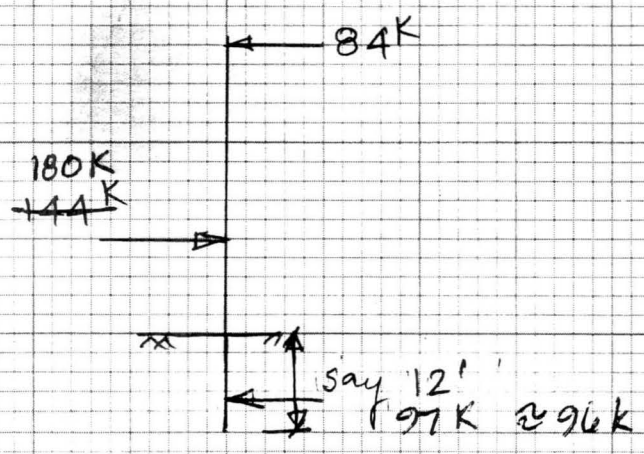
$$\therefore M_y = M_{py} + M_{ay} + M_{Fy} = 0$$

$$= 180 * 21.4 - 97.47 * \frac{11.4}{3} - F_x * 41.4 = 0$$

$$= 3852 - 370.39 - F_x\, 41.4$$

$$\therefore F_x = \frac{3852 - 370.39}{41.4} = 84 \text{ kips.}$$

Use 84 kips applied at bridge deck level to model fully restrained condition at top. The free-body diagram is:



2/23/89

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A. Calculate Relative Stiffness:

$$T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{4.03\times10^6 * 1.02\times10^5}{\cancel{225}\ 70}\right)^{1/5}$$

$$= \cancel{71.18}\ 89.9 \text{ inches.}$$

B. Caculate $L/T = \frac{10\times12}{\cancel{71.18}\ 89.9} = \cancel{1.68}\ 1.33 \approx \cancel{1.7}\ 1.3$

For embedment = 10'

$L/T < 2$ or $L < 2T$ so the pile will behave like a short pile.

For $L = 12'$: $L/T = \cancel{2.02}\ 1.60$

It is the designer's opinion that for short pier where $L < 2T$, Broms method to analyse the lateral load case is more suitable. However, both the Brom's method and method presented in NAVFAC are used in the following section.

C. Brom's Method (For 10' embedment)

Brom's method do not calculate the deflection — however calculate the ultimate lateral load and ultimate moment capacity.

$$\therefore K_p = \tan^2(45 + \phi/2) \quad \text{run } 5.83$$

45 computer

$$= \tan^2(45 + 37/2) = \boxed{4.02}$$

JHE 2/23/89
JHE 5/1/89

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Lateral Load Capacity, H_u

$$H_u = 1.5 \cdot d \cdot \gamma \cdot L^2 K_p$$

d = pile dia
γ = Unit weight of soil
L = depth of embedm

135 computer num 5.83

$$\therefore H_u = (1.5 * 3 * 125 * 10 * 10 * 4.02) \text{ lbs}$$

$$= 226 \text{ kips} \checkmark$$ 354 kips for comp parameters

$$\therefore H_{allowable} = 75 \text{ kip} \ (118k) \text{ with} \quad F.S. = 3.0 \checkmark$$

$$= 113 \text{ kips} \ (177k) \text{ "} \quad F.S. = 2.0 \checkmark$$

Using computer parameters for $P_o = 180k$ F.S. $= \frac{354}{180} = 2.0$

for $P_o = \cancel{144}\ 180$, F.S. $= \frac{226}{\cancel{144}\ 180} = \cancel{1.3}\ 6$ ~~1.3~~ 6 JAE 3/1/89

Yield Moment.

using comp. param. $\frac{H_u}{K_p d^3 \gamma} = 10$ using computer ~~data~~ param $\frac{180}{5.83 \times 27 \times 0.135} = 8.5$

$\cancel{(144/4 \times 27 \times 0.125) = 10.7}$ $\frac{180}{4 \times 27 \times 0.125} = 13.3$

$$M_{yield} = \cancel{9}\ 11 \ (9) * K_p \gamma \cdot d^4$$

$$= \cancel{9}\ 11 \ (9) * 4 * 125 \ (5.83) * (3 * \cancel{4})^4 \ (135)$$

$$= \boxed{\cancel{445.5}\ 574 \text{ k-ft}}$$

Depth to Point of Maximum moment

$$f = 0.82 * \left(\frac{226 \ (354)}{0.125 \ (0.135) * 3 \times 4 \ (5.83)} \right)^{0.5}$$

$$= 10 \text{ feet below}$$ ✓ JAE 2/23/89

same w/ ○ #'s JAE 3/1/89

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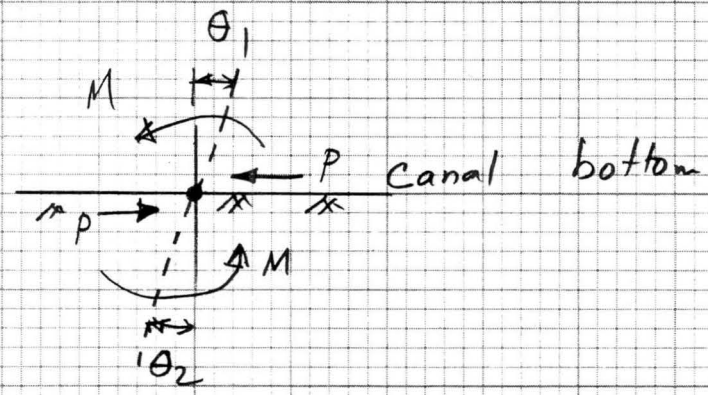
D. If the pile embedment is increased to more than or equal to 12', the pile will be intermediate pile and NAVFAC chart can be used.

L/T = 2.02 (say 2); for $L = 12'$, $T = 71.19$ in. (89.9)

Case is fixed at top and we have



an equivalent force of 84 kip at girder level

As per criteria given in NAVFAC, we have to calculate θ_1 as a function of the characteristics of the pier and θ_2 as a function of soil and pile properties as given in Figure 13, DM 7.2, Page 240.

E. Calculate Value of M to be developed due to the fixed rigid top. 2/23/89

$$\theta_1 = \frac{ML}{4EI} = \frac{M * 30 * 12}{4 * 4.03\times10^6 * 1.02\times10^5}$$

$\theta_1 = \frac{M}{4E}$

$$\therefore \theta_1 = 2.19\times10^{-10} M \checkmark \quad \text{—— (1)}$$

$$\theta_2 = F_{\theta_1} \cdot \frac{PT^2}{EI} + F_{\theta_2} \cdot \frac{MT}{EI}$$

~~(180-80) 89.9²~~ see computer conditional table

$$= -3.4 * \frac{(180-84) * 71.18^2}{4.03\times10^6 * 1.02\times10^5} - \frac{3.25 * M * 71.1}{4.03\times10^6 * 1.02\times}$$

$$= -4.02\times10^{-3} - 5.63\times10^{-10} M$$

Equate $\theta_1 = \theta_2$

7.82

$$\therefore 2.19\times10^{-10} M = -5.63\times10^{-10} M - 4.02\times10^{-3}$$

$$\text{or } 2.19\times10^{-10} M + 5.63\times10^{-10} M = -4.02\times10^{-3}$$

$$\therefore M = -\frac{4.02\times10^{-3}}{7.82\times10^{-10}}$$

$$= \boxed{-428 \text{ K-ft}} \checkmark$$

Now Calculate Deflection, Moment and Shear using $P = 180 - 84 = 96$ Kips. 2/23/89

$M = 428$ K-ft.

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E. Calculate Deflection at Canal Bottom:

$$\delta = \left(4.3 * \frac{96000 * 71.18^3}{4.03\times10^6 * 1.02\times10^5}\right) + \left(\frac{+3.3 * -428000 * 12 * (71.18)}{4.03\times10^6 * 1.02\times10^5}\right)$$

$$= 0.36 - 0.21 = 0.15''$$

F. Calculate Moment in Pier.

(i) at Dredge line $z = 0$

$$M_p = 0 - 1 * 428 = -428 \text{ k-ft}$$

(ii) at 3' below: $z = \frac{3\times12}{71.18} = 0.505$

$$\therefore M = 0.43 * \frac{96000 * 71.18}{12{,}000} + 0.93 * \frac{-428{,}000 * 12}{12{,}000}$$

$$= -153 \text{ k-ft}$$

(iii) at 6' below: $z = \frac{6*12}{71.18} = 1.01$

$$\therefore M = 0.49 * \frac{96}{12} * 71.18 + 0.64 * -428$$

$$= 5.1 \text{ k-ft}$$

(iii) at 8' below $z = 1.35$

$$M = 21 \text{ k-ft} \Rightarrow 0.27 * \frac{96}{12} \times 71.18 + 0.31 \times (-428)$$

ppc 2/23/89

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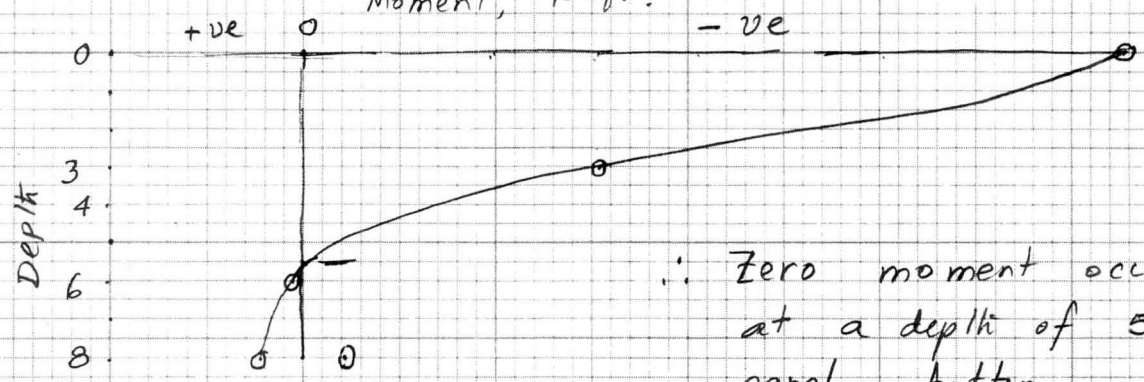
The zero moment may occur between 3' and 6'.



∴ Zero moment occurs at a depth of 5.5' below canal bottom.

G. Calculate Shear in Pier:

(i) At Canal bottom (dredge line):

V = 96 kip:

(ii) at 3' below canal bottom; z = 0.505

$V = 0.54 * 96 + \frac{(-428)*12}{71.18} * (-0.31) = 74$ kips. ✓

(iii) at 6' below, z = 1.01

$V = -0.28 * 96 + \left(\frac{-428*12}{71.18}\right) * (-0.8) = 30.8 \simeq 31$ kips. ✓

(iv) at 8' below; z = 1.35

$V = -0.65 * 96 + \left(\frac{-428*12}{71.18}\right) * (-0.93) = 5$ kips. ✓

EHE 2/23/89

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Pile Parameters Given:

diameter = 30"

I = moment of inertia = 5.16×10^4 in4

A = Area of pier x-section = 710 in^2

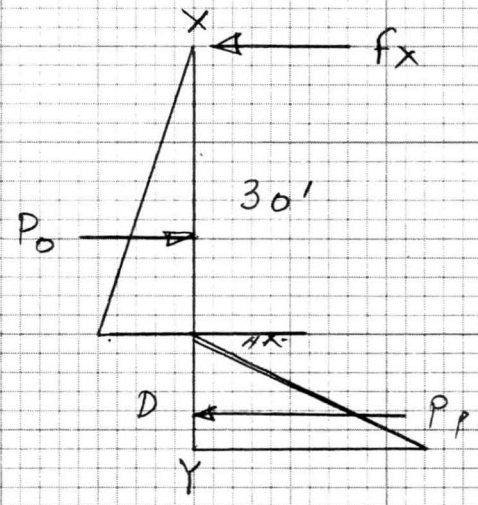
E = Mod of elasticity = 4.03×10^6 psi

Vertical DL + LL = 144 Kips

Assume no horizontal movement at top and an at-rest earth pressure exists. 6' center to center distance.

$$K_p = \frac{1+\sin\phi}{1-\sin\phi}$$

$$= 4.02$$



$P_o = 125 * 0.3982 * 30 \times \frac{1}{2} * 6$

$= 134$ Kips

$M_{ox} = 134 * 20 = 2686$ K-ft

$P_p = D * 503 * 2.5 * D * \frac{1}{2} = 628 D^2$

$M_{px} = \left(30 + 2\frac{D}{3}\right) . 628 D^2 = 18840 D^2 + 418 D^3$

@ D = 11'; M_{px} = 2836 K-ft

D = 10 ; M_{px} = 418 + 1884 = 2302

D = 10.8'; M_{px} = ~~2719~~ 2724 K-ft

D = 10.6 ; M_{px} = 2614 K-ft

JPE 2/23/89

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$D = 10.7'$;

$$M_y = 0 = (134 * 20.7) - \left(71.9 * \frac{10.7}{3}\right) - F_x * 40.7$$

or $-256 + 2774 = F_x (40.7)$

or $F_x = 61.86 = 62$ kips

Use 62 kips applied at bridge grade level against the soil.

A. Calculate Relative Stiffness:

$$T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{4.03 \times 10^6 * 5.16 \times 10^4}{\cancel{225}\ 70}\right)^{1/5}$$

$= \cancel{62.7}\ 78.4$ in

B. $L/T = \frac{120}{78.4} = 1.53 < 2.0$ ~~2.15 > 2.0~~

The pile will behave like an intermediate pile if depth of embedment is increased to at least 11'.

C. Calculate Applied Moment: JAE 2/23/89

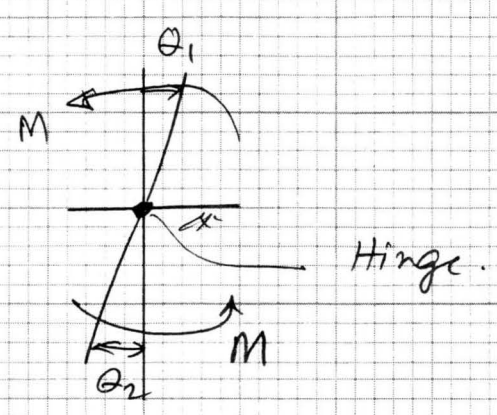
Assume a hinge at ground level and proceed as in previous case.

WESTERN TECHNOLOGIES INC.

Project/Subject 16th Street and Glendale Ave. Bridges — Job No. 2128 5186

30" ϕ, w/strut. 4% steel — Date 2/20/89 Sheet 33 of 35

Lateral Pile Analysis — By E. Hoque



$$** \theta_1 = \frac{ML}{4EI} = \frac{M * 30 * 12}{4 * 4.03 \times 10^6 * 5.16 \times 10^4} = \boxed{4.32 * 10^{-10} M} \quad \text{— (1)}$$

$$\theta_2 = -3.375 * \frac{(134-62) * (62.1)^2}{4.03 \times 10^6 * 5.16 \times 10^4} - \frac{3.125 * M * 62.1}{4.03 \times 10^{-6} * 5.16 \times 10^4}$$

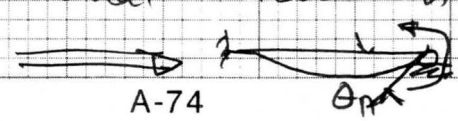
$$= -4.51 * 10^{-3} - 9.33 * 10^{-10} M \quad \text{— (2)}$$

$$13.65 * 10^{-10} M = -4.51 * 10^{-3}$$

$$\therefore \boxed{M = -275 \text{ k-ft.}}$$

AHB 2/23/89

** θ_1 is calculated using the structural properties of the top of pier. It is assumed that ~~due to~~ the pier above canal level is an popped cantilever beam.



$$M = \frac{4EI\theta}{L}$$

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D. Calculate Deflection:

at Canal bottom; $z = 0$

$$\delta = 4.25 * \frac{72{,}000 * (62.1)^3}{4.03 \times 10^6 * 5.16 \times 10^4} + 3.3 * \frac{-275 * 12000 * (62.1)^2}{4.03 \times 10^6 * 5.16 \times 10^4}$$

$$= 0.35'' - 0.202 = 0.15''$$

E. Calculate Moment:

(i) at canal bottom:

$M_p = 1 * 275 = -275$ k-ft.

(ii) at 3' below $z = \frac{3 \times 12}{62.1} = 0.58$

$$M = 0.5 * (72) * \frac{62.1}{12} + (-275) * 0.92$$

$$= -66.7$$

(iii) 6' below: $z = 1.16$

$$M = 0.43 * 72 * \frac{62.1}{12} + (-275) * 0.55$$

$$= 8.96 \simeq +9 \text{ k-ft}$$

(iv) at 5.5': $z = 1.06$:

$$M = 0.46 * 72 * \frac{62.1}{12} + (-275)(0.6)$$

$$= 6.4 \text{ k-ft}$$

(v) at 5': $M = 4.8'$

2/23/89

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(vi) at 4', M = -28 k-ft.

zero moment might have occured at 4'-9"

(vii) at 4'-9": @ -10 k-ft:

(F) Calculate Shear.

(i) at canal bottom: z = 0:

V = 72 kips.

(ii) 3 feet: $V = 0.5 * 72 + \left(\frac{-275 * 12}{62.1}\right) * (-.3)$

= 52 kips.

(iii) 6': $V = -0.38 * 72 + \frac{-278 * 12}{62.1} * (-.82)$

= 16 kips.

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Given: $\phi = 36''$

$I = 1.30 \times 10^5 \text{ in}^4$

$A = 1020 \text{ in}^2$

$E = 4.03 \times 10^6 \text{ psi}$

DL+LL = ~~181 kips~~ ; 0 for wingwalls

Height = 30' = 360"

c.c. space = 8'

Estimated or Assumed:

SOIL 1

$\phi = 37°$ (lab)

$\gamma = 125 \text{ pcf}$ (lab) $= 0.07 \text{ pci}$

$k_a = 31 \text{ psf/ft}$

$k_o = 50 \text{ psf/ft}$

$k_p = 500 \text{ psf/ft}$

SOIL 2

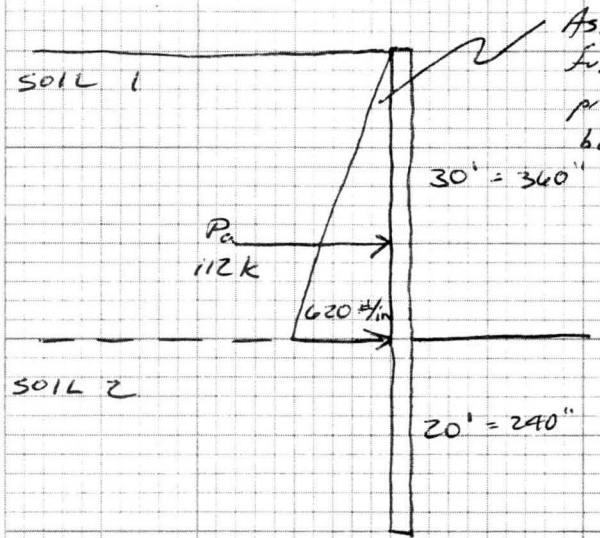
$\phi = 45°$ (based on max. lab value of 42°, avg. Becker blows of over 90/ft, and rock at 16th St.)

$\gamma = 135 \text{ pcf}$ (same as above) $= 0.08 \text{ pci}$

$k_a = 23 \text{ psf/ft}$

$k_o = 40 \text{ psf/ft}$

$k_p = 790 \text{ psf/ft}$



Assume active pressure, full load of 8' space on pier; pressure @ bottom = 30' × 8' × 31 psf/ft = 7440 lb/ft = 620 lb/in

SOIL 2 CONSIDERED CONSTANT TO DEPTH OF 1000"

Check w/ NAVFAC DM 7.2, pg. 234 ff : $f = 70 \text{ \#/in}^3$

$$T = \left(\frac{4.03 \times 10^6 \times 1.3 \times 10^5}{70}\right)^{0.2} = 94.4'' \quad ; L/T = 2.5$$

① $$\delta @ \text{canal} = 2.3\left(\frac{1120 \times 94.4^2 \times 12 \times 10^3}{4.03 \times 10^6 \times 1.3 \times 10^5}\right) + 3.0\left(\frac{112 \times 94.4^3 \times 10^3}{4.03 \times 10^6 \times 1.3 \times 10^5}\right) = 0.53'' + 0.54''$$

$= 1.1''$ vs. $0.56''$ (LPILE)

② LPILE $M_{max} = 1550$ k-ft @ 6' below canal; $z/T = 0.76$

$$M = 0.84\,(1.12 \times 10^6) + 0.57\,(1.12 \times 10^5 \times 94.4/12)$$

$= 941 \text{ k-ft} + 502 \text{ k-ft} = 1440 \text{ k-ft}$ vs. 1550 k-ft (LPILE)

③ LPILE $V_{max} = -165$ kips @ 13' below canal; $z/T = 1.65$

$$V = -0.7\,(1.12 \times 10^6 \times 12/94.4) + (-0.6)\,1.12 \times 10^5 = -100k + (-67.2)k = -167k$$

vs. $-165k$ (LPILE)

E. Hoag 3/3/89

WESTERN TECHNOLOGIES INC.

Project/Subject: Glendale Ave & 16th St. Bridges — Conditions for Computer Runs — Stitched w/ 36" ϕ Piers and 4% Steel

Job No. 212BJ186

Date 2/28/89 Sheet 2 of 8

By F. Costello

Given: $\phi = 36''$

$I = 1.02 \times 10^5 \text{ in}^4$

$A = 1020 \text{ in}^2$

$E = 4.03 \times 10^6 \text{ psi}$

$DL + LL = 181$ kips

Height $= 30' = 360''$

c.c. space $= 8'$

Estimated or Assumed:

SOIL 1

$\phi = 37°$ (lab)

$\gamma = 125$ pcf (lab) $= 0.07$ pci

$k_a = 31$ psf/ft

$k_o = 50$ psf/ft

$k_p = 500$ psf/ft

SOIL 2

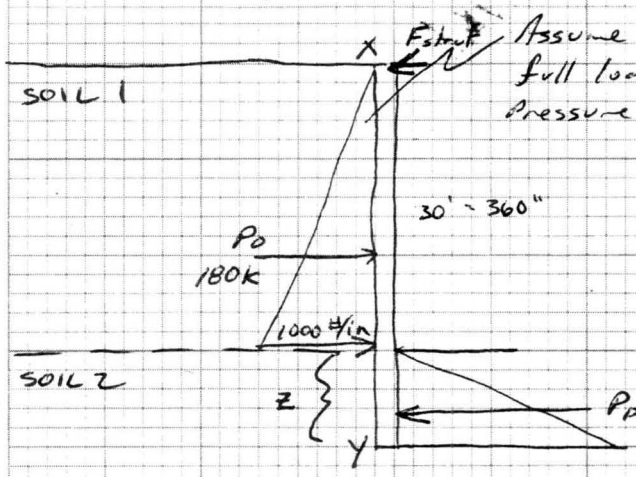
$\phi = 45°$ (based on lab, Becker blows, and rock @ 16th St)

$\gamma = 135$ pcf (same as above) $= 0.08$ pci

$k_a = 23$ psf/ft

$k_o = 40$ psf/ft

$k_p = 790$ psf/ft



Assume at-rest pressure, full load of 8' space on pier

Pressure @ bottom $= 30' \times 8' \times 50$ psf/ft $= 12{,}000$ lb/ft $= 1000$ lb/in

$P_p = \frac{1}{2}(z^2)(3)(.790) = 1.185\, z^2$

SOIL 2 considered constant to depth of 1000"

$M_x = 0 = 180(20) - 1.185 z^2 (30 + \frac{2z}{3}) = 3600 - 35.550 z^2 - .790 z^3$

$35.55 z^2 + 0.79 z^3 = 3600$

$z = 10'$; $3555 + 790 = 4345$ (hi)

$z = 9'$; $2880 + 576 = 3456$ (lo)

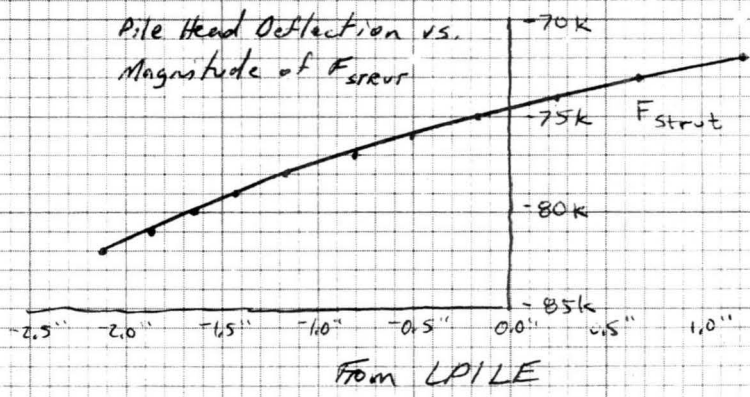
$z = 9.1'$; $2944 + 595 = 3539$ (lo)

$z = 9.15'$; $2976 + 605 = 3581$ (lo)

$z = 9.17'$; $2989 + 609 = 3598$ (ok)

$\therefore P_p = 1.185 (9.17)^2 = 100$ k

& $F_{strut} = 180 - 100 = 80$ k



From LPILE

E. Hogan 3/3/89

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checked by Broms by E. Hoque pp. 25-27

check by NAVFAC: $f = 70\ \#/in^3$; $T = \left(\frac{4.03\times10^6 \times 1.02\times10^5}{70}\right)^{1/5} = 89.9'' \cong 90$

$L/T = 120/89.9 = 1.33 < 2.0$ but try NAVFAC charts for 2.0 anyway

$$\theta_1 = \frac{mL}{4EI} = \frac{m\times360}{4\times4.03\times1.02\times10^{11}} = 2.19\times10^{-10}\,m$$

$$\theta_2 = \frac{F_{\theta_1} P T^2}{EI} + \frac{F_{\theta_2} M T}{EI}$$

$$= \frac{-3.4(180-80)(90)^2(10^3)}{4.1\times10^{11}} + \frac{-3.25(m)(90)}{4.1\times10^{11}}$$

$$= -0.00672 - 7.13\times10^{-10}\,m$$

@ canal $\theta_1 = \theta_2 \Rightarrow 2.19\times10^{-10}\,m = -6.72\times10^{-3} - 7.13\times10^{-10}\,m$

$$m = -\frac{6.72\times10^{-3}}{9.32\times10^{-10}} = -600\ k\text{-}ft$$

Estimate δ, M, S using $P = 100k$ & $M = -600\ k\text{-}ft$

① $$\delta = \frac{4.3(100{,}000)(90)^3}{4.1\times10^{11}} + \frac{3.3(-7{,}200{,}000)(90)^2}{4.1\times10^{11}} \cancel{= 0.76}$$

$= 0.76 - 0.47 = 0.29''$ vs $0.35''$ from LPILE

~~M =~~

② LPILE: $M_{max} = -970\ k\text{-}ft$ @ 11' above canal bottom

Try moment check where LPILE shows ~~M=0~~ M near 0 @ depth ≅ 60"

$M = 0.87(-7{,}200{,}000) + 0.49(100{,}000)(90)$

$= -1.85\times10^6$ in-lb vs -5.75×10^4 in-lb (LPILE)

try @ 72" $M = 0.78(-7{,}200{,}000) + 0.50(100{,}000)(90)$

$= -1.12\times10^6$ in-lb vs 2.75×10^5 in-lb (LPILE)

try @ 84" $M = 0.66(-7{,}200{,}000) + 0.49(100{,}000)(90)$

$= -3.42\times10^5$ in-lb vs 3.61×10^5 in-lb (LPILE)

WESTERN TECHNOLOGIES INC.

Project/Subject: Glendale Ave & 16th St. Bridges — Job No. 212B3186
Conditions for Computer Runs — Date 3/1/89 — Sheet 4 of 8
Strutted w/ 36" ϕ Piers & 4% Steel — By F. Costello

try 96" $M = 0.6(-7{,}200{,}000) + 0.47(100{,}000)(90)$
$= -9.0 \times 10^4$ in-lb vs 2.7×10^5 in-lb (LPILE)

try 108" $M = 0.5(-7{,}200{,}000) + 0.45(100{,}000)(90$
$= 4.5 \times 10^5$ in-lb vs 1.1×10^5 in-lb (LPILE)

LPILE shows $M=0$ between 5 and 6 feet below canal
NAVFAC shows $M=0$ between 8 and 9 feet below canal
Reasonably close considering L/T of 20 used when actual condition was L/T = 1.33

③ LPILE $P_{max} = 1.09 \times 10^5$ lb @ depth = 12"
by inspection of NAVFAC graphs P_{max} will be @ canal & be = 100 kips
check location $P=0$; from LPILE $P=0$ 1st time between depths of 72" and 84"
from NAVFAC

@ 12" $P = -0.02\left(\frac{-7{,}200{,}000}{90}\right) + 0.98(100{,}000)$
$= 100$ kips vs 109 kips (LPILE) Reasonably close

3/3/89

WESTERN TECHNOLOGIES INC.

Project/Subject: Glendale Ave. & 16th St. Bridges — Job No. 212B5186

Conditions for Computer Runs — Date 3/2/89 — Sheet 5 of 8

Strutted w/30" φ Piers & 4% steel — By F. Costello

Given: $\phi = 30''$

$I = 5.16 \times 10^4 \text{ in}^4$

$A = 710 \text{ in}^2$

$E = 4.03 \times 10^6 \text{ psi}$

DL+LL = 144 kips

c.c. space = 6'

Estimated or Assumed:

SOIL 1

$\phi = 37°$

$\gamma = 125 \text{ pcf} = 0.07 \text{ pci}$

$k_a = 31 \text{ psf/ft}$

$k_o = 50 \text{ psf/ft}$

$k_p = 500 \text{ psf/ft}$

SOIL 2

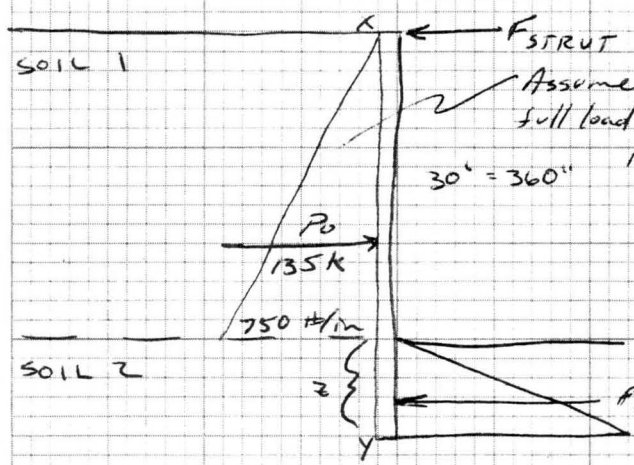
$\phi = 45°$

$\gamma = 135 \text{ pcf} = 0.08 \text{ pci}$

$k_a = 23 \text{ psf/ft}$

$k_o = 40 \text{ psf/ft}$

$k_p = 790 \text{ psf/ft}$



Assume at-rest pressure, full load of 6' space on pier

Pressure @ bottom = 30' × 6' × 50 psf/ft

9000 p/f

750 p/i

$P_p = \frac{1}{2}(z^2)(2.5)(0.79) = 0.99 z^2$

$M_X = 0 = 135(20) - z^2\left(30 + \frac{2z}{3}\right) = 2700 - 30z^2 - 0.66z^3$

$30z^2 + 0.66z^3 = 2700$

@ z = 8 ; 1920 + 338 = 2258

@ z = 9 ; 2430 + 481 = 2911

@ z = 8.9 ; 2376 + 465 = 2841

@ z = 8.8 ; 2323 + 450 = 2773

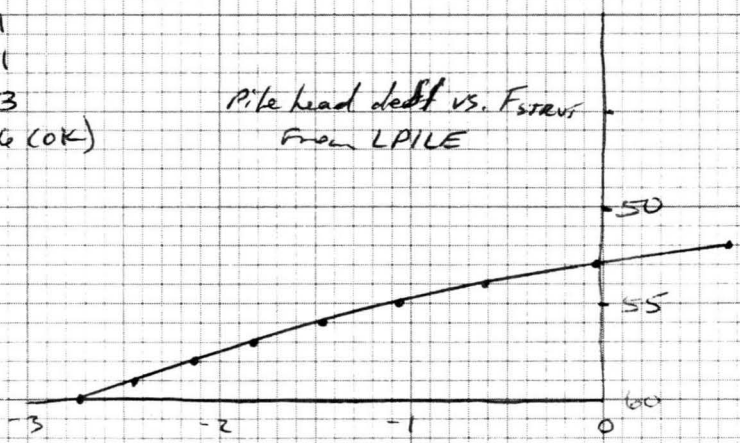
@ z = 8.7 ; 2271 + 435 = 2706 (OK)

@ z = 8.69 ; 2699

∴ $P_p = 75k$

& $F_{STRUT} = 60k$

Pile head defl vs. F_{STRUT} From LPILE



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Check by Broms:

$$T = \left(\frac{4.03\times10^6 \times 5.16\times10^4}{70}\right)^{0.2} = 78.4''$$

$L/T = 120/78.4 = 1.5 < 2$ rigid pile

Ultimate Lateral Resistance:

$$L/D = 10/2.5 = 4 \Rightarrow \frac{P}{K_p D^3 \gamma} = 20$$

try $\phi = 45°$ & $\gamma = 135$ pcf $\Rightarrow K_p = 5.83$

$$P = 20(5.83)(2.5)^3(135) = 246 \text{ kips} \quad \cancel{vs\ 82}$$

try $\phi = 40°$ & $\gamma = 125$ pcf $\Rightarrow K_p = 4.60$

$$P = 20(4.60)(2.5)^3(125) = 180 \text{ kips}$$

try $\phi = 35°$ & $\gamma = 125$ pcf $\Rightarrow K_p = 3.69$

$$P = 20(3.69)(2.5)^3(125) = 144 \text{ kips}$$

Range of F_s for P/shear @ ground surface from LPILE

$$\frac{246}{82} = 3.0 \; ; \; \frac{180}{82} = 2.2 \; ; \; \frac{144}{82} = 1.8$$

All considered adequate

Lateral Deflection: $\eta = \left(\frac{f}{EI}\right)^{0.2} = 0.0127$; $\eta L = 1.53$

$$0.7 = \frac{Y_0 (EI)^{0.6} (f)^{0.4}}{PL}$$

use P from LPILE $\Rightarrow$ 82k

$$Y_0 = \frac{0.7(82{,}000)(120)}{(4.03\times10^6 \times 5.16\times10^4)^{0.6}(70)^{0.4}}$$

$= 0.20''$ vs $0.42''$ from LPILE

3/31

WESTERN TECHNOLOGIES INC.

check by NAVFAC using $L/T = 2$, $P = 75k$, & $M = -450$ ft-kip

① $\delta = 3.3\left(\frac{-5.4\times10^6 \times 78.4^2}{4.03\times10^6\times5.16\times10^4}\right) + 4.3\left(\frac{75{,}000\times78.4^3}{4.03\times10^6\times5.16\times10^4}\right)$

$= \cancel{0.10}\ -0.53 + 0.75 = 0.22''$ vs 0.42 from LPILE

② LPILE shows $M=0$ between 5' and 6' below canal

by NAVFAC @ 96" $\Rightarrow 0.45(-5.4\times10^6) + 0.35(75{,}000)(78.4) = -3.72\times10^5$ in-lb

@ 108" $\Rightarrow 0.28(-5.4\times10^6) + 0.25(75{,}000)(78.4) = -4.2\times10^4$ in-lb

NAVFAC does not provide good check here

(2A) $\theta_1 = \frac{ML}{4EI} = \frac{M\times360}{4\times4.03\times5.16\times10^{10}} = 4.33\times10^{-10}\,M$

$\theta_2 = \frac{F\theta_1 PT^2}{EI} + \frac{F\theta_2 MT}{EI}$

$= \frac{-3.4(135-60)(10^3)(\cancel{90}\ 78.4)^2}{4.03\times5.16\times10^{10}} + \frac{-3.25(M)(\cancel{90}\ 78.4)}{4.03\times5.16\times10^{10}}$

$\cancel{= -9.61\times10^{-5}}$

$= -7.54\times10^{-3} - \cancel{1.23\times10^{-9}}\ 12.25\times10^{-10}\,M$

$\theta_1 = \theta_2$

$4.33\times10^{-10}\,M = -7.54\times10^{-3} - 12.25\times10^{-10}\,M$

$M = -\frac{7.54\times10^{-3}}{16.58\times10^{-10}} = -4.55\times10^6$ in-lb

subbing this $M = -4.55\times10^6$ in-lb into:

$\delta = 0.31''$ vs $0.42''$ from LPILE

$M_{@96''} = -10.5\times10^3$ in-lb

$M_{@108''} = 1.96\times10^5$ in-lb

$M=0$ between here vs 60" & 77" in LPILE; reasonably close

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The hand calculations using nondimensional curves (NAVFAC) and Brom's method substantiate the results obtained from the "LPILE" computer program. We believe the computer results are more precise than the hand calculations and should be used for design of the drilled shaft foundations.

For the free-end and strutted conditions highlights supporting our conclusions are presented below:

- Free-end: In this case, the nondimensional curves could be used appropriately. The moment and shear values obtained by hand calculations agreed very closely with the computer results. The calculated deflection at the ground surface (canal bottom level) was approximately twice the computer generated value (1.1" vs. 0.56"). This variation is not considered significant.

- Strutted: In this case, the nondimensional curves are not strictly appropriate because of the relatively shallow embedment resulting in L/T ratios of less than two. The nondimensional

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curves were used to estimate moment, shear, and deflections. The hand calculated estimates were reasonably close to the "LPILE" results considering the nonstandard use of the nondimensional curves.

- Brom's method: Brom's method for rigid pile were also used because the low L/T ratios. These calculations indicate that the strutted piers have adequate resistance against design lateral loads and applied moments. Adequate lateral resistance was obtained using ϕ angles from 35° to 45° and unit weights of 125 pcf to 135 pcf for soil. Lateral deflection at the canal bottom was approximately 1/2 of the computer generated value (0.20" vs. 0.42").
This is not considered significant.

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