CALIFORNIA DEPARTMENT OF TRANSPORTATION
Toll Bridge Program

DEPARTMENT'S POSITION PAPER FOR THE DISPUTES REVIEW BOARD

Notice of Potential Claim Number 10
"Differing Site Condition"
Damaged Temporary Spud Piles at Pier E5E

CONTRACT NUMBER: 04-012024
San Francisco-Oakland Bay Bridge East Span- Skyway Structures

STANDARD SPECIFICATIONS: JULY 1999
STANDARD PLANS: JULY 1999

CONTRACTOR:	Kiewit/FCI/Manson, a Joint Venture
CONSTRUCTION MANAGER:	PETE SIEGENTHALER
RESIDENT ENGINEER:	DOUGLAS COE
SENIOR BRIDGE ENGINEER:	MARK WOODS
REPORT PREPARED BY:	MARK WOODS
DATE OF REPORT:	November 4, 2004
DRB HEARING DATE:	November 19, 2004

TABLE OF CONTENTS

I. Description of Potential Claims

Kiewit/FCI/Manson – a Joint Venture (KFM) requests additional compensation for the damage of six - 42" pile-driving template piles by a latent or unknown physical condition encountered while driving the piles at pier E5E. Compensation due to time impact is not being sought in this potential claim.

II. Amount of Claims

KFM's estimated direct costs of the additional work as shown in NOPC #10 amounts to $200,000. KFM has not provided the Engineer with a detailed break down of what is included in the estimated amount (labor, equipment, materials, etc.)

III. Contract Chronology

a) Contract Award Date	January 17, 2002
b) Contract Approval Date	January 22, 2002
c) First Working Day	February 6, 2002
d) Contract Specified Duration	1000 Working Days
e) Original Computed Completion Date	February 14, 2006
f) Contractor Bid Amount	$1,043,541,000
g) Current Completion Date	November 29, 2006

IV. Chronology of Events and Key Documents

December 30, 2003 – Damaged template piles discovered at Pier E5E
January 6, 2004 – KFM-LTR-637 (Notice of differing site conditions)
January 9-26, 2004 – Permanent piles excavated and soils evaluated
January 12, 2004 – STL-3395

February 4, 2004 – STL-3531
February 13, 2004 – KFM-LET-689
May 12, 2004 – KFM-LET-800 (KFM submits Geotech report)
June 18, 2004- STL-4786
June 28, 2004- KFM-LET-854
July 22, 2004- STL-5037
August 6, 2004 –KFM-LET-887 (KFM submits NOPC 10)
August 23, 2004 – DRB hearing date set for November 18, 2004
October 1, 2004 – KFM-LET-972

V. Applicable Contract Documents

a. 1999 Standard Specifications Section 5-1.116, "Differing Site Conditions"
b. Special Provisions Section 5-1.012, "Differing Site Conditions"
c. Special Provisions Section 10-1.24, "Piling"
d. Project Plans- Sheet 939 of 978, Log of Test Borings, Boring 98-29
e. Material Information Handout, Boring 98-29

VI. Department's Understanding of Contractor's Position

The Contractor discovered that the pile driving template piles were damaged when they were removed at Pier E5E. The Contractor states in their NOPC that, "a latent or unknown subsurface physical condition most likely caused the damage." The Contractor also states, in KFM-LET-800, "Based on the actual pile damage, the conditions encountered at Pier 5E are materially different from those represented by the boring log" and that, "KFM does not take exception to Caltrans observations" (that no obstructions were observed in the material excavated from the 5E piles).

The Department understands that the Contractor's claim of a differing site condition (DSC) as described in NOPC 10 and KFM-LET-800 is based only upon the fact that the pile driving template piles were damaged in use and that no material difference between the contract documents and the site conditions was discovered during an investigation of the soils at Pier E5E.

VII. Background

Section 10-1.24, "Piling," of the Special Provisions required the Contractor to provide a pile driving template and pile handling plan in order to drive the battered 2.5m diameter piling at each of the footings on the San Francisco-Oakland Bay Bridge Skyway Project. The Contractor designed a pile driving template using six - 42" diameter temporary piles. This same template system was used at Piers E16E through E6E prior to being used at Pier E5E without incident. However, at Piers E3 through E6 longer temporary piles were required because the footings were constructed at water level. Additional lengths of pipe were added to the template piles for these locations. At Pier

E5E, one of the pile driving templates was installed over and around the temporary falsework piles (four 60" dia.), which were used to support the footing box. Upon removal of the temporary pile driving template piles, it was discovered that the lower 25 to 64 feet of the piles were collapsed and could not be reused.

VIII. Department's Position

The DSC clauses in the Special Provisions (section 5-1.012) and Standard Specifications (section 5-1.116) require that in order for a DSC to be present, It must be apparent to the Engineer, upon investigation, that either,

a) "subsurface or latent physical conditions are encountered at the site differing materially from those indicated in the contract" exist, or

b) "unknown physical conditions of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in the work provided for in the contract" exist.

In the Department's investigation, no evidence was found to suggest that either condition exists.

The Contractor designed and constructed their pile driving templates and upon removal of the template at Pier E5E, the template piles were discovered to be damaged and the Contractor claimed that he may have discovered a DSC. At this point, the Contractor and the Department investigated the site conditions and found that there was no material difference between the soils removed from the permanent piles and the soils shown on Boring 98-29, the boring on the Log of Test Borings closest to Pier E5 (sheet 939 of 978 of the Contract Plans). The Contractor concurred with this evaluation in KFM-LET-800, stating, "KFM does not take exception to Caltrans observations."

The blow counts for all of the temporary piles (42" template and 60" falsework) were less than 14 blows per foot using a Delmag D-80 pile hammer. This indicates that they did not face hard driving. This combined with the undamaged condition of the adjacent 60" temporary falsework piles, some of which were less than five feet away from the damaged template piles indicates that no obstruction was hit by the 60" falsework piles and that therefore no unknown physical condition exists.

Because the 60" falsework piles were not damaged less than 5 feet away, it appears that the pile driving template piles used at Pier E5E were damaged due to causes other than differing site conditions.

Potential reasons for the pile driving template damage other than a Differing Site Condition:

– All of the damage to the piles exists below a soil strata in which the undrained shear strength of the soil increased by more than 100% above the material in the strata above it, as shown on Boring 98-29 in the Log of Test Borings, sheet 939 of 978. Because this information is shown in the Contract, the Department is not responsible for any damage caused by this layer.

– Section 10-1.24, "Piling," of the Special Provisions, requires, in the "Pile Driving Template and Pile Handling Submittal" subsection, that the, "Pile handling shall conform to the recommendations in the American Petroleum Institute Recommended Practice 2A (API 2A)".

– The Contractor submitted calculations showing the design of the pile driving template. Contrary to good practice and the requirements of the Special Provisions, the pile-driving template piling were not sized based upon two of the three criteria set forth in API 2A. Section 6.10.6 of API 2A requires a minimum pile wall thickness in 42" diameter piles of 0.67" for local buckling. Table 12.5.7 requires a minimum wall thickness of $^{3}/_{4}$" in for driving energy. The **template piles were** designed with a ½" wall thickness and are therefore, **undersized**.

The 60" diameter falsework piles are sized thicker than the API 2A minimums for local buckling and driving energy and were undamaged, less than five feet away.

IX. Summary

- Excavated material from the six permanent piles show no evidence of material differing from that identified in the Contract, specifically the Logs of Test Borings.
- No unknown physical conditions were apparent to either KFM or Caltrans during the permanent pile excavation.
- No "subsurface or latent physical conditions are encountered at the site differing materially from those indicated in the contract" were encountered during pile excavation,
- No "unknown physical conditions of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in the work provided for in the contract" encountered.
- In undersizing the template pile wall thickness, the Contractor did not follow the requirements of Contract. Had the Contractor sized the template piles in accordance with API 2A, it is likely that no significant damage would have occurred.

Therefore, according to Section 5-1.012 of the Special Provisions, and Section 5-1.116 of the Standard Specifications no Differing Site Condition exists.

X. Exhibits

1. KFM-LTR-637 (Notice of differing site conditions) (January 6, 2004)
2. STL-3395 (January 12, 2004)
3. STL-3531 (February 4, 2004)
4. KFM-LET-689 (February 13, 2004)
5. KFM-LET-800 (KFM submits Geotech report) (May 12, 2004)
6. STL-4786 (June 18, 2004)
7. KFM-LET-854 (June 28, 2004)
8. STL-5037 (July 22, 2004)
9. KFM-LET-887 (KFM submits NOPC 10) (August 6, 2004)
10. 1999 Standard Specifications Section 5-1.116, "Differing Site Conditions"
11. Special Provisions Section 5-1.012, "Differing Site Conditions"
12. Sheet 939 of 978 of the Contract Plans (Log of Test Borings, Boring 98-29)
13. Special Provisions Section 10-1.24, "Piling"
14. American Petroleum Institute 2A, "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms." Section 6 and 12.
15. Material Information Handout, Final Marine Geotechnical Site Characterization, Volume 2D
16. Spud Pile calculations from KFM-SUB-000187, "Pile Driving Template & Pile Handling Submittal"

EXHIBIT 1

KFM

KIEWIT / FCI / MANSON, A JV

January 6, 2004

Serial Letter: KFM-LET-000637

California Department of Transportation
SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607

Attention: Douglas Coe, P.E.

Reference: Skyway Bridge Project (Caltrans Contract No. 04-012024)
KFM Job No. 364/3726

Subject: Differing Site Conditions at Pier E5E

Dear Doug:

In accordance with the "Differing Site Conditions" clauses found at Section 5.1.012 of the Special Provisions and 5.1.116 of the Standard Specifications, this letter provides notice that subsurface conditions were encountered at Pier E5E that differ substantially with those indicated by the information provided in the contract documents.

All six of the 42" pile driving template support piles were damaged by unknown obstructions at Pier E5E. This damage was observed when the piles were removed. A better understanding of the type of obstruction may be identified when pile excavation begins at Pier 5E later this week.

If you have any questions in this regard, please contact Scott Hanson at (510) 627-1045.

Sincerely,
KIEWIT/FCI/MANSON, a JV

A.T. (Tom) Skoro
Project Director

cc: file

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EXHIBIT 2

STATE OF CALIFORNIA-BUSINESS, TRANSPORTATION AND HOUSING AGENCY ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF TRANSPORTATION
SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607
Facsimile Number: (510) 622-5165

Flex Your Power
Be Energy Efficient!

January 12, 2004

KFM, a JV
220 Burma Road
Oakland, CA 94607

Contract: 04-012024
04-SF, Ala-80-13.9/14.3, 0.0/1.6
SFOBB Skyway Project
State Letter # 5.03.1-003395

Subject: Possible Differing Site Conditions at Pier E5 E

Dear Mr. Skoro,
Attention: Mr. Scott Hanson,

We acknowledge receipt of KFM-LET-000637, regarding "Differing Site Conditions at Pier E5E," on January 6, 2004. We have begun our investigation as to whether or not any subsurface conditions encountered in driving the temporary pile driving template support piles at Pier E5 E differed materially from those indicated in the "Materials Information," log of test borings, or other geotechnical data provided by the Department. As discussed last Tuesday and Friday, with Greg York and Kurt Hinkle, we agree that a better understanding of any possible difference in subsurface conditions may be identified when the permanent piles at Pier E5 E are excavated this week.

If you have any questions regarding this matter, please call Mark Woods at (510) 622-5107.

Sincerely,

Mark P. Woods
Foundation Structure Representative

for: Douglas B. Coe
Resident Engineer

cc: S. Mohan
D. Coe
K. Balan
R. Yost

file: 5.03.1

EXHIBIT 3

STATE OF CALIFORNIA-BUSINESS, TRANSPORTATION AND HOUSING AGENCY ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF TRANSPORTATION

SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607
Facsimile Number: (510) 622-5165

Flex Your Power
Be Energy Efficient!

February 4, 2004

KFM, a JV
220 Burma Road
Oakland, CA 94607

Contract: 04-012024
04-SF, Ala-80-13.9/14.3, 0.0/1.6
SFOBB Skyway Project
State Letter # 5.03.1-003531

Subject: Differing Site Conditions at Pier E5E

Dear Mr. Skoro,
Attention: Scott Hanson,

This letter is to follow STL 5.03.1-003395 regarding KFM-LET-000637, dated January 6, 2004, regarding "Differing Site Conditions at Pier E5E." We completed our subsurface investigation. We have logged our soil samples, which have been taken every 4.5 meters for a length of 60 meters in Piles #3 and #4 at Footings E6E and E5E during pile dredging activities. These soil samples have been compared to the Log of Test Borings, specifically bores numbered 98-29, 98-10, and 98-41 of the Contract Plans. We have determined that the soil samples taken in the field do not appear to differ materially from those indicated in the "Materials Information," log of test borings and other project geotechnical data. Therefore, the Engineer finds no apparent merit in the KFM-LET-000637 notice with respect to the definition of differing site conditions in sections 5-1.012 of the Special Provisions and 5-1.116 of the Standard Specifications. No additional compensation will be forthcoming.

If you have any questions regarding this matter, please contact Mark Woods at (510) 622-5107.

Sincerely,

Mark P. Woods
Foundation Structure Representative

for: Douglas B. Coe
Resident Engineer

cc: S. Mohan, D. Coe,
S. Abbas, P. Siegenthaler

file: 5.03.1

RY:ry

EXHIBIT 4

KFM

KIEWIT / FCI / MANSON, A JV

13-Feb-2004

Serial Letter: KFM-LET-000689

California Department of Transportation
SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607

Attention: Douglas Coe, P.E.

Reference: Skyway Bridge Project (Caltrans Contract No. 04-012024)
KFM Job No. 364/3726

Subject: Differing site conditions at Per E5E

Dear Doug:

Caltrans letter 3531 responded to KFM letter 637 that requested Caltrans recognize a differing site condition at pier 5E. Caltrans letter found no merit to KFM's request based on observations of material excavated from the permanent pile at pier 5E.

Per our Owner's meeting today, KFM plans to submit a more refined request for additional compensation resulting from the damage caused to the 42" spud pile. Caltrans further evaluation of this issue is appreciated.

Sincerely,
KIEWIT/FCI/MANSON, a JV

A.T. (Tom) Skoro
Project Director

EXHIBIT 5

KFM

KIEWIT / FCI / MANSON, A JV

12-May-2004

Serial Letter: KFM-LET-000800

California Department of Transportation
SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607

Attention: Douglas Coe, P.E.

Reference: Skyway Bridge Project (Caltrans Contract No. 04-012024)
KFM Job No. 364/3726

Subject: Differing site condition at pier 5E

Dear Doug:

Caltrans February 4, 2004 letter 3531 documented Caltrans findings regarding the damage to template spud piles at Pier 5E. Caltrans did not observe any obstructions in the material excavated from the 5E piles that caused the spud pile damage. KFM does not take exception to Caltrans observations. However, the fact remains that physical subsurface conditions collapsed the 42" spud pile driven at pier 5E. KFM has utilized the same means and methods to install and remove the spud pile without damage at many other footing locations. Per the attached layout, KFM has diligently examined all temporary and permanent pile locations and verified that there are no conflicts. Subsurface condition is the only remaining variable.

KFM concludes that, based on the actual pile damage, the conditions encountered at pier 5E are materially different from those represented by the boring log. KFM provides the attached GeoEngineers geotechnical report to support this conclusion. Based on this information and per the Special Provisions section 5-1.012, please issue a change order providing for compensation for the 5E pile damage.

Sincerely,
KIEWIT/FCI/MANSON, a JV

A.T. (Tom) Skoro
Project Director

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022834 MAY 18

Kiewit/FCI/Manson, A JV • 220 Burma Road • Oakland, CA 94607
P.O. Box 23223 • Oakland, CA 94623 • 510 419-0120 • Fax 510 839-0666

GEOENGINEERS

FAX TRANSMITTAL

1101 Fawcett Avenue, Suite 200, Tacoma, WA 98402 TELEPHONE: (253) 383-4940, FAX: (253) 383-4923 www.geoengineers.com

To: Kiewit/FCI/Manson JV

Date: 5/10/2004

File: 01092-030-00

Fax Number: (510) 839-0666

Attention: Greg York

Regarding: SFOBB East Span Seismic Safety Project No. 04-012024

Pages	Date	Description
1	5/10/2004	Fax Transmittal
3	5/10/2004	Memorandum - Pier E5E 42-inch Diameter Spud Pile Damage

Total Pages: 4

Remarks: Please call if you have questions.

Signed: Gail Boyer for
Gary W. Henderson
ghenderson@geoengineers.com

GEOENGINEERS

1101 FAWCETT AVENUE, SUITE 200, TACOMA, WA 98402, TELEPHONE: (253) 383-4940, FAX: (253) 383-4923 www.geoengineers.com

MEMORANDUM

TO:	Greg York – Kiewit/FCI/Manson JV
FROM:	Garry H. Squires, PE/Gary W. Henderson, PE
DATE:	May 10, 2004
FILE:	1092-030-00
SUBJECT:	Pier E5E 42-inch Diameter Spud Pile Damage SFOBB East Span Seismic Safety Project No. 04-012024 Bridge 34-006L/R District 4
CC:	George Atkinson – Kiewit/FCI/Manson JV Stewart Moore – Kiewit/FCI/Manson JV

INTRODUCTION

This memorandum summarizes our review and opinions regarding damage to the 42-inch diameter spud piles observed upon their removal at Pier E5E and supplements comments previously provided in our March 22, 2004 memorandum. Our understanding of pile installation, removal and damage is based on several discussions with you, and review of driving records, sketches and photographs you provided. We also reviewed the soil conditions described in the log for Boring 98-29 provided in the Caltran project documents for this pier.

DATA REVIEW

We understand all six of the 42-inch diameter spud piles were observed to be damaged upon removal at Pier E5E in December 2003. The piles are ½-inch wall thickness A572 Grade 60 steel and are 175 feet long. The spud pile layout comprises a four-pile group (Pile Numbers 1, 3, 4 and 6) with single piles located to the north (Pile Number 5) and south (Pile Number 2), opposite the mid-point of the group. Each of the four-pile group spuds are situated next to the 60-inch diameter pile box falsework piles. The piles to the north and south are not located near any of the pile box falsework piles. We understand the 60-inch diameter piles are installed before the 42-inch diameter piles. The 60-inch diameter piles have 1-inch thick walls.

The 60-inch diameter and the 42-inch diameter piles were driven to Elevation – 160 feet and Elevation – 126 feet, respectively, using the Delmag D80 pile hammer. You provided driving log data for the 42-inch diameter piles that indicates relatively easy driving conditions for the last approximately 50 feet of drive to tip elevation. In general, the logs indicate blow counts were in the range of 3 to 8 blows per foot. We noted slightly higher blow counts in the range of 11 to 13 blows per foot in the last 8 feet of drive for Pile Number 2.

Based on the sketches and photographs provided, the damaged portion of the 42-inch diameter piles ranges from the lower 25 feet to 64 feet of the pile. The observed damage ranges from an apparent inward deformation of a few inches along one side of the pile, parallel to the pile axis, to severe "flattening" and deformation of the pile to an approximately oval perimeter. One pile (Pile Number 2) appears to exhibit buckling in addition to severe flattening.

The log for Boring 98-29 in the project documents indicates the mud line is at about Elevation –32 feet. Soft clay (CH) is reported to a depth of about 46 feet (Elevation –78 feet). A stiff to very stiff clay (CH) layer is reported from 46 feet to 54 feet depth (Elevation –86 feet) and is underlain by very stiff clay (CH) to a depth of about 112 feet (Elevation –144 feet).

DISCUSSION AND CONCLUSIONS

Based on the reported relatively easy pile driving conditions, and the soil conditions disclosed in Boring 98-29 it is our opinion the piles should not have been damaged. The observed pile damage and driving behavior does not appear to us to be consistent with encountering a dense or hard soil layer, and no dense or hard layers apart from a thin (approximately 2-feet thick) layer at a depth of about 25 feet were disclosed on the boring log within the depth range for the 42-inch diameter spud piles.

We understand that you did not observe damage to any of the 60-inch diameter falsework piles upon removal at Pier E5E. Based on the pile damage sketches provided we also noted that one of the more heavily damaged 42-inch diameter spud piles (Pile Number 2) was not located near the falsework piles. We also understand that damage to the 42-inch diameter spud piles at other pier locations has not been encountered in installations completed prior to and subsequent to the Pier E5E installation.

We understand that you performed a review of available historical records of the area before construction of the existing Bay Bridge, and also reviewed some construction photographs for the Bay Bridge. Based on your review you did not find anything that would indicate prior construction activity in the vicinity of Pier E5E.

Considering all of the above it appears the spud piles at Pier E5E encountered some kind of obstruction that caused the change. It is unknown whether the possible obstruction is a natural feature or was man-placed. In our opinion the observed damage to the spud piles suggests a near vertically-oriented feature could have obstructed the pile rather than a more horizontally-oriented dense or hard soil layer or zone.

Please call if you have questions or require additional information regarding any of the above.

GHS:GWH:jm

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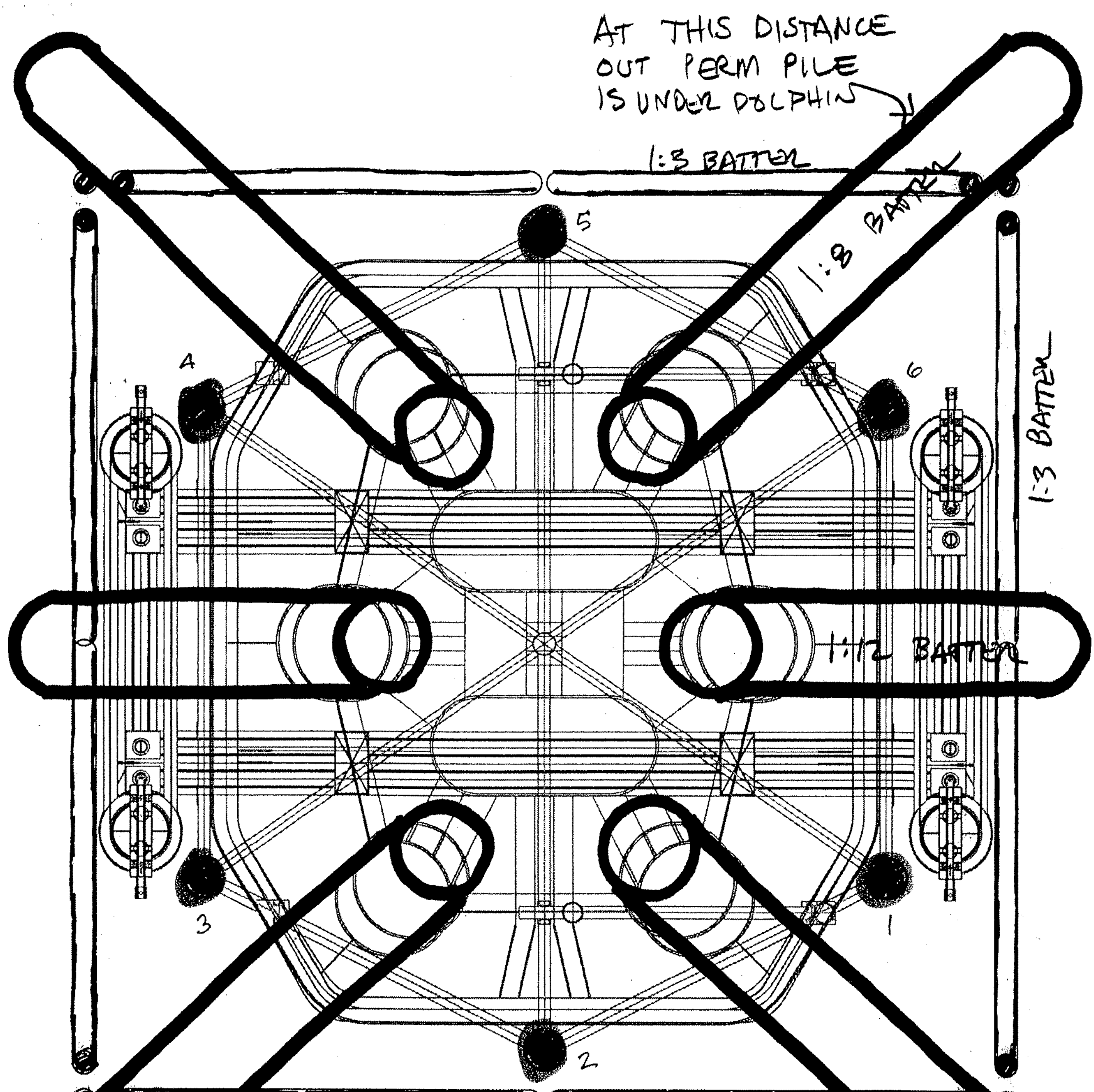
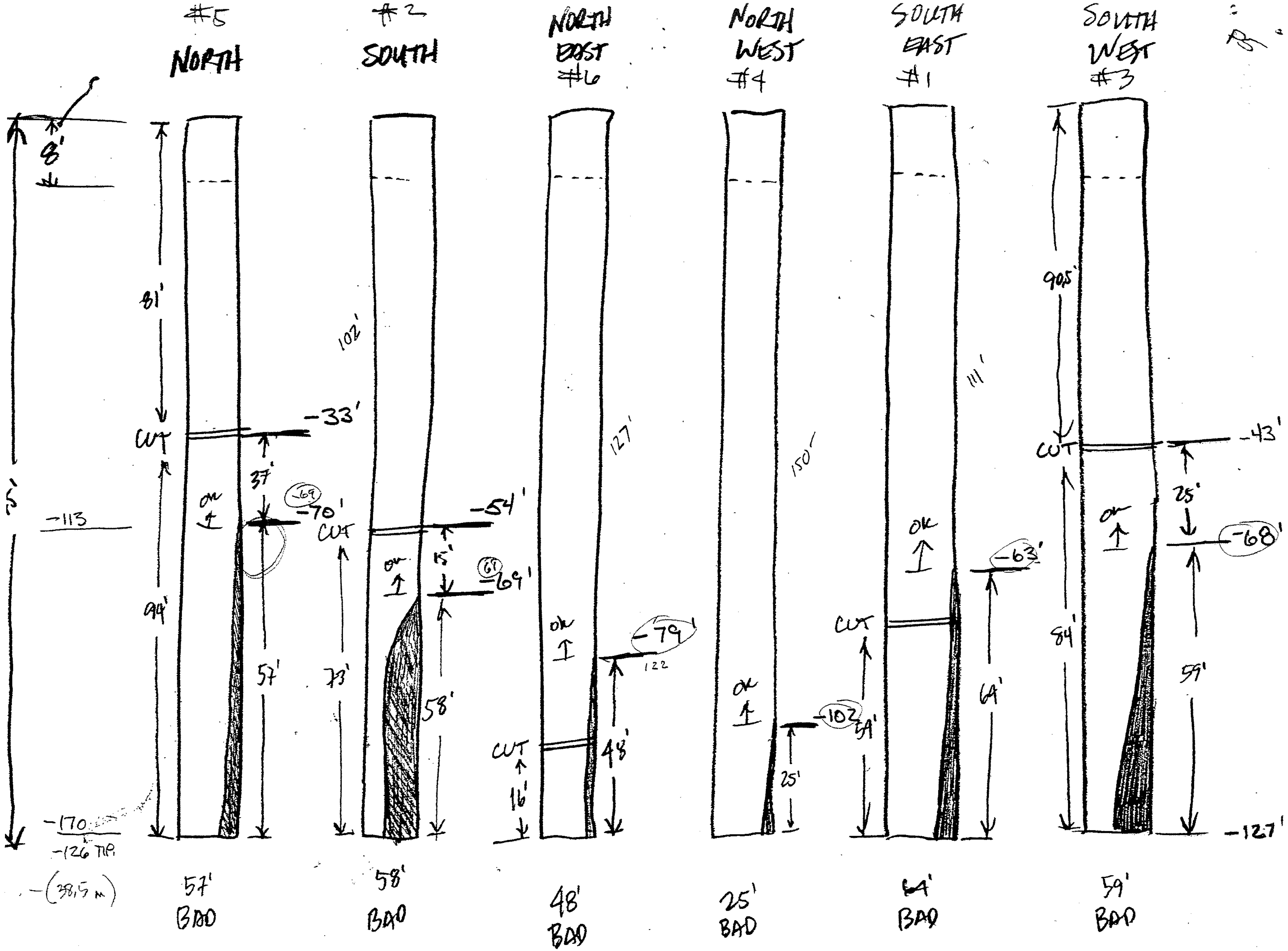


P5E: PILE INTERFERANCE PLAN VIEW

	INSTALLED
60" FALSEWORK PILES	2ND
42" TEMPLATE PILES	3RD
24" DOLPHIN PILES	1ST
2.5 M PERMANENT	LAST

NORTH

PILE INTERFERANCE		SHEET
BY: MOORE	DATE: 4/21/04	1 OF 1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NORTH
#2
SOUTH
NORTH
EAST
#6
NORTH
WEST
#4
SOUTH
EAST
#1
SOUTH
WEST
#3
8'
81'
CUT
-33'
37'
-69
-70'
OK
-113
94'
57'
-170
-126 TIP
(38.5 m)
57'
BAD
102'
CUT
-54'
15'
OK
68
-69'
73'
58'
58'
BAD
127'
OK
-79'
122
CUT
16'
48'
48'
BAD
150'
OK
-102
25'
25'
BAD
111'
OK
-63'
CUT
64'
64'
BAD
90.5'
CUT
-43'
25'
OK
-68'
84'
59'
-127'
59'
BAD

EXHIBIT 6

STATE OF CALIFORNIA-BUSINESS, TRANSPORTATION AND HOUSING AGENCY ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF TRANSPORTATION

SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607
Facsimile Number: (510) 622-5165

Flex Your Power
Be Energy Efficient!

June 18, 2004

KFM, a JV
220 Burma Road
Oakland, CA 94607

Contract: 04-012024
04-SF, Ala-80-13.9/14.3, 0.0/1.6
SFOBB Skyway Project
State Letter # 5.03.1-004786

Subject: Differing Site Conditions at Pier E5E

Dear Mr. Skoro,

Attention: Scott Hanson,

KFM Letter 800 and the attached geotechnical report do not offer any information to further the assertion that was already addressed in State Letters 3395 and 3531. Letter 800 describes classic buckling in the piles and suggests that because the piles buckled, there must be a condition which meets the requirements in section 5-1.012, "Differing Site Conditions," of the Special Provisions. This section states in part, that a differing site condition may arise if "... subsurface or latent conditions are encountered at the site differing materially from those indicated in the "Materials Information," log of test borings, other geotechnical data obtained by the Department's investigation of subsurface conditions...". State Letter 3531, states that "the soil samples taken in the field do not appear to differ materially from those indicated in the "Materials Information," log of test borings and other project geotechnical data." This remains the case. Also, Log of Test Boring sheet 8 of 47, (boring 98-29) shows at least a 100 percent increase in undrained shear strength for the soils at an elevation above the highest point of buckling shown on the pile damage drawing attached to the GeoEngineers' May 10, 2004 memorandum to Greg York.

The GeoEngineers memorandum describes the damage to the piles as "buckling," "inward deformation" and "severe flattening". We concur with this assessment and we direct your attention to the practice according to which your pile driving template piles were designed, American Petroleum Institute Recommended Practice 2A-WSD (API 2A). API 2A includes recommendations for minimum pile wall thickness. According to section 6.10.6 and table 12.5.7 of API 2A, the piles for the template were undersized. This is shown in the following table.

		Temporary Pile Type	
		Pile driving template	Falsework
API 2A recommended wall thickness based upon Buckling - section 6.10.6, Minimum Wall Thickness			
Pile Diameter (in)		42	60
Actual Pile Wall Thickness (in)		1/2	1
From API 2A: t=0.25+D/100		2/3	7/8
Wall thickness difference (in)		**undersized**	OK
		0.17	1/8
API 2A recommended wall thickness based upon Driving energy -- Table 12.5.7			
Pile Diameter (in)		42	60
Actual Pile Wall Thickness (in)		1/2	1
Tip Elevation (ft)		-126	-126
Hammer		Delmag D-80	Delmag D-80
Maximum Energy (ft-kips)		186-212	186-212
Guideline Wall Thickness (in)		3/4	7/8
Wall thickness difference (in)		**undersized**	OK
		1/4	1/8

It should be noted that the other large temporary structure located at Pier E5 E, the Footing Box Falsework is acceptable with respect to the recommendations of this practice for wall thickness. These piles, located adjacent to the six damaged template piles and as few as five feet away, do not exhibit any damage.

The GeoEngineers conclusion that "the spud piles at Pier E5E encountered some kind of obstruction… … a near vertically-oriented feature…" which would be able to damage these six piles while leaving undamaged, the falsework piles only a few feet away seems to be highly unlikely. It remains our determination that the damage to the six template piles was not caused by a differing site condition.

If you have any questions regarding this matter, please contact Mark Woods at (510) 622-5107.

Sincerely,

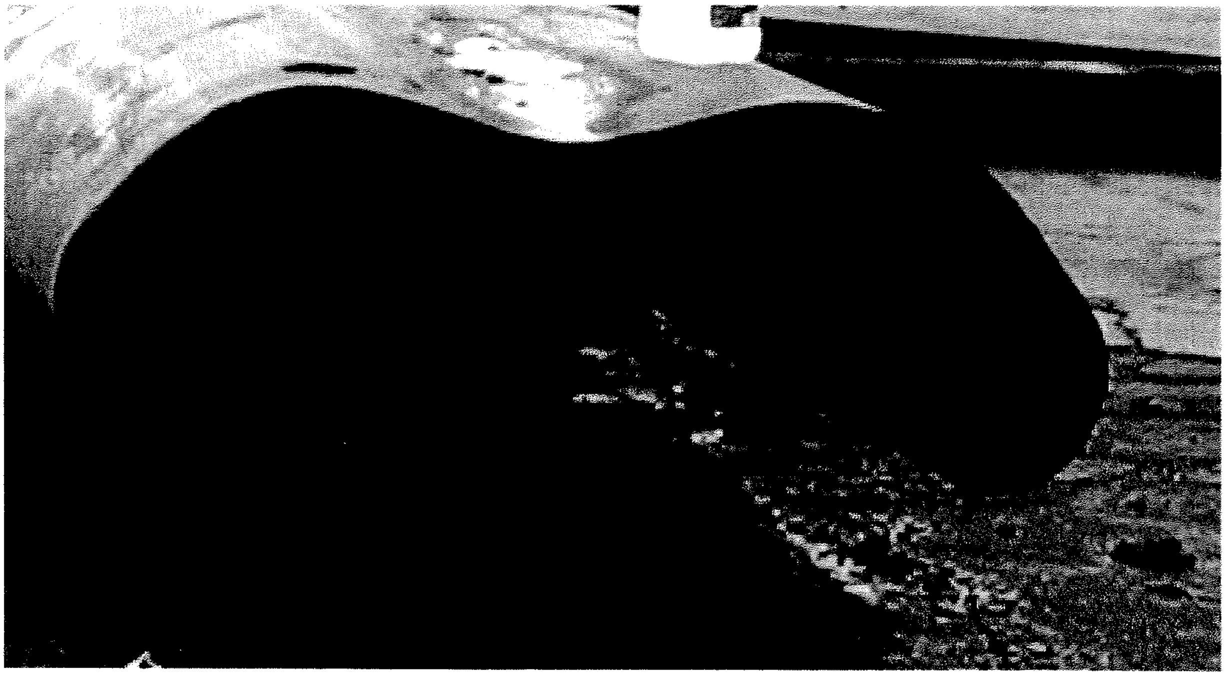
Mark P. Woods
Foundation Structure Representative

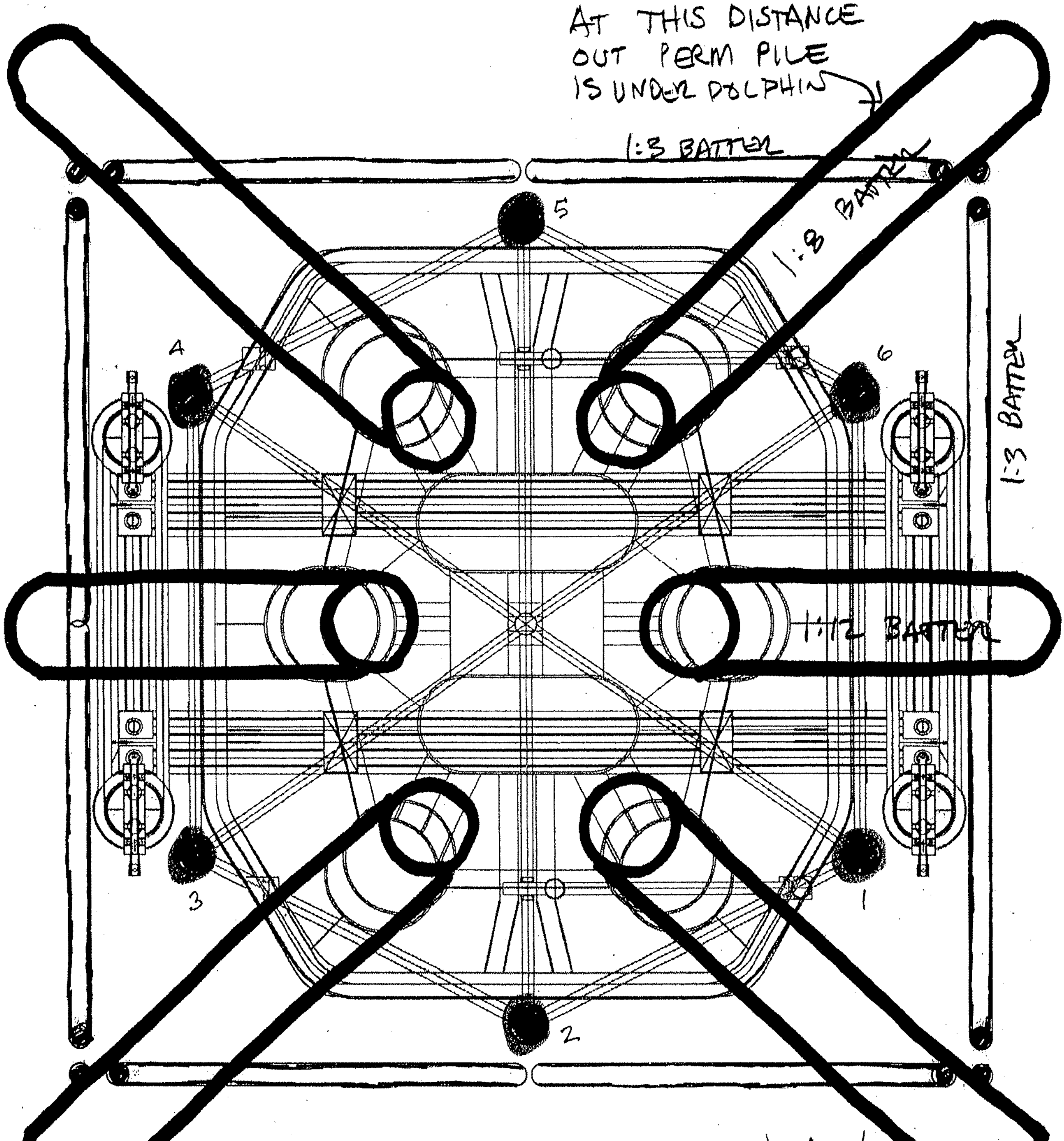
for: Douglas B. Coe
Resident Engineer

cc: D. Coe,
P. Siegenthaler,
D. Salladay

file: 5.03.1, 62.00

Buckled pile tips for 42" diameter 1/2" thick pile driving template spud piles used at Pier E5 E



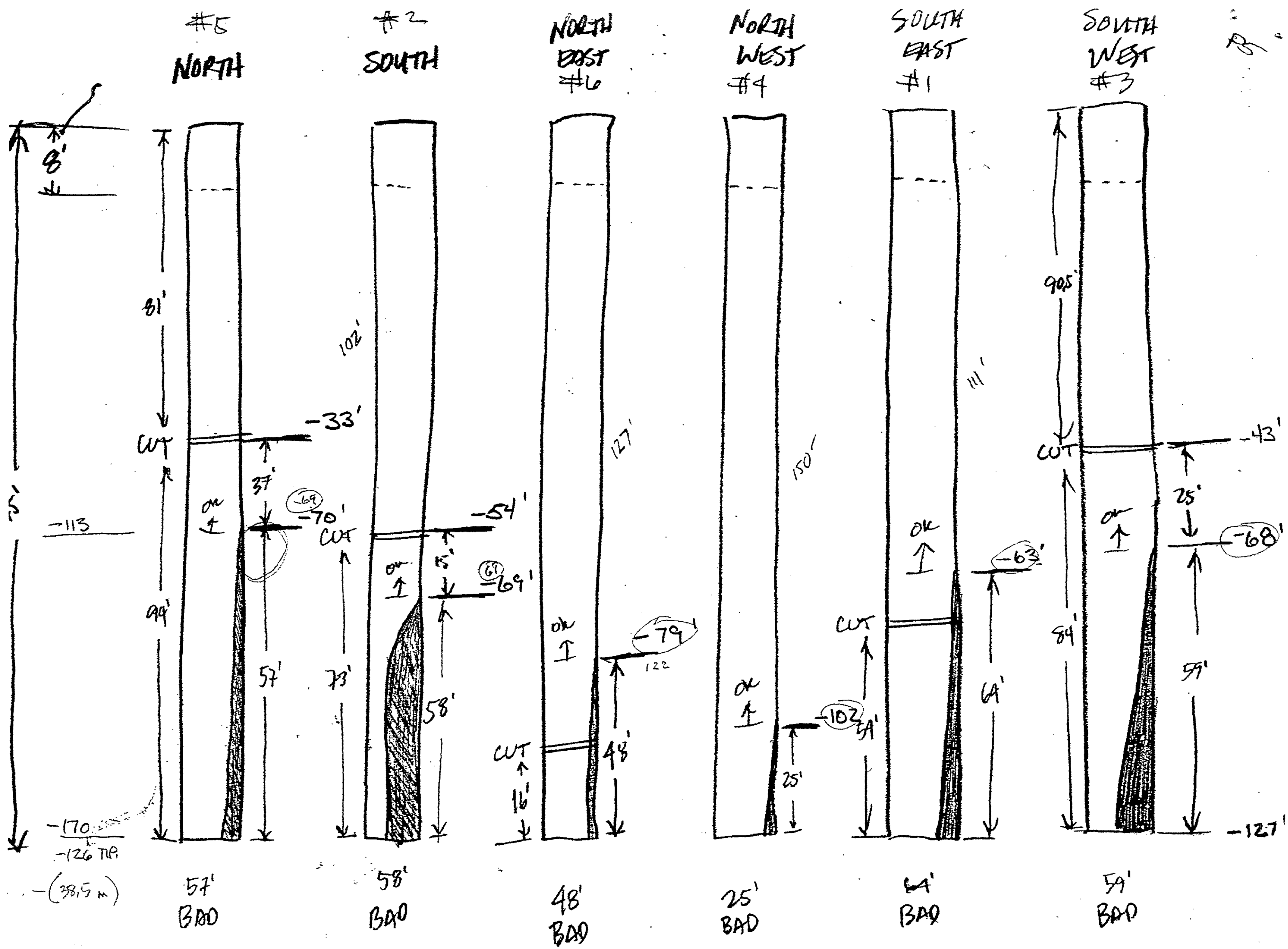


P5E: PILE INTERFERANCE PLAN VIEW

	INSTALLED
60" FALSEWORK PILES	2ND
42" TEMPLATE PILES	3RD
24" DOLPHIN PILES	1ST
2.8 M PERMANENT	LAST

NORTH

PILE INTERFERANCE		SHEET
BY: MOORE	DATE: 4/21/04	1 OF 1



#5
NORTH
#2
SOUTH
NORTH EAST #6
NORTH WEST #4
SOUTH EAST #1
SOUTH WEST #3
8'
81'
CUT
-33'
37'
-70'
OK
-113
99'
57'
-170
-126 TIP
(38.5 m)
57' BAD
102'
-54'
CUT
15'
-69'
73'
58'
58' BAD
127'
OK
-79'
122
CUT
16'
48'
48' BAD
150'
OK
-102
25'
25' BAD
111'
OK
-63'
CUT
64'
64' BAD
90.5'
CUT
-43'
25'
-68'
84'
59'
-127'
59' BAD

EXHIBIT 7

28-Jun-2004 Serial Letter: KFM-LET-000854

California Department of Transportation
SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607

Attention: Douglas Coe, P.E.

Reference: Skyway Bridge Project (Caltrans Contract No. 04-012024)
KFM Job No. 364/3726

Subject: Differing site condition -- Spud pile collapse

Dear Doug:

Caltrans letter 4786 responds to KFM letter 800 requesting a change order for the differing site condition that collapsed the spud pile at pier 5E. Caltrans refused KFM's request based on two incorrect conclusions.

First, Caltrans notes a 100 percent increase in undrained shear strength "above the point of buckling." This conclusion is incorrect. Damage to the spud piles extends above this interface between the soft and more dense clay layers. Regardless, the undrained shear strength, of any soils type identified in the borings, is insufficient to cause buckling of this size pile as indicated by the documented low blow counts necessary to achieve tip elevation.

Second, Caltrans states that the ½" wall piles were undersized per API 2A. This conclusion is incorrect. API is a general guideline often used in lieu of more detailed analysis to determine minimum performance criteria for piling. KFM consultants developed a full Wave Equation Analysis for Piles (WEAP) and the spud piles were sized accordingly. Based on the documented low blow counts necessary to achieve tip elevation, the pile stresses do not exceed design assumptions.

Please consider the above information and the attached geotechnical report and issue a change order accordingly.

Sincerely,
KIEWIT/FCI/MANSON, a JV

A.T. (Tom) Skoro
Project Director

cc: file

GeoEngineers

FAX TRANSMITTAL

1101 Fawcett Avenue, Suite 200, Tacoma, WA 98402 TELEPHONE: (253) 383-4940, FAX: (253) 383-4923 www.geoengineers.com

To: Kiewit/FCI/Manson JV
220 Burma Road
Oakland, California

Date: 6/24/2004

File: 01092-030-00

Fax Number: (510) 839-0666

Attention: Greg York

Regarding: SFOBB East Span Seismic Safety Project No. 04-012024

Pages	Date	Description
1	6/24/2004	Fax Transmittal
4	6/24/2004	Memorandum - Pier E5E 42-inch Diameter Spud Pile Damage

Total Pages: 5

Remarks: Please call if you have questions.

Signed: [signature] for
Garry H. Squires
gsquires@geoengineers.com

GEOENGINEERS

1101 FAWCETT AVENUE, SUITE 200, TACOMA, WA 98402, TELEPHONE: (253) 383-4940, FAX: (253) 383-4923 www.geoengineers.com

MEMORANDUM

TO:	Greg York – Kiewit/FCI/Manson JV
FROM:	Garry H. Squires, PE/Gary W. Henderson, PE
DATE:	June 24, 2004
FILE:	1092-030-00
SUBJECT:	Pier E5E 42-inch Diameter Spud Pile Damage SFOBB East Span Seismic Safety Project No. 04-012024 Bridge 34-006L/R District 4
CC:	George Atkinson – Kiewit/FCI/Manson JV Stewart Moore – Kiewit/FCI/Manson JV

INTRODUCTION

This memorandum summarizes our comments regarding Caltrans' June 18, 2004 (State Letter #5.03.1-004786) concerning damage to the 42-inch-diameter spud piles observed upon their removal at Pier E5E. You requested our comments on June 22, 2004. Specifically, our opinion regarding shear strength of soil as disclosed in Boring 98-29 of the contract documents, and Caltrans' comment pertaining to pile size.

SOIL SHEAR STRENGTH

Caltran stated in their letter that Boring 98-29 indicates "at least a 100 percent increase in undrained shear strength for the soils at an elevation above the highest point of buckling shown on the pile damage drawing". We interpret Caltrans' comment about shear strength to mean the depth at which the soil profile changes from soft to firm clay (undrained shear strength less than about 50 kPa) to stiff to very stiff clay (undrained shear strength greater than about 100 kPa).

The log for Boring 98-29 in the project documents indicates the mud line is at about Elevation – 10 m. Soft to firm clay (CH) is reported to a depth of about 46 feet corresponding to about Elevation – 24 m. A stiff to very stiff clay (CH) layer is reported from that depth to 54 feet depth and is underlain by very stiff clay (CH) to a depth of about 112 feet, equivalent to about Elevation – 44 m.

Based on the sketches and photographs provided, the damaged portion of the 42-inch-diameter piles ranges from the lower 25 feet to 64 feet of the pile. This corresponds to an elevation ranging between about Elevation – 30.8 m to Elevation – 18.9 m. It appears to us that the pile damage range does extend above the elevation of the top of the stiff clay.

PILE SIZE

Caltran states in their June 18, 2004 letter, that "piles for the template were undersized" with respect to wall thickness. The piles are 1/2-inch-wall thickness A572 Grade 60 steel and are 175 feet long. The 42-inch-diameter piles were driven to Elevation – 126 feet (Elevation – 38.5 m), using the Delmag D80 pile hammer. You provided driving log data for the 42-inch-diameter piles that indicates relatively easy driving conditions for the last approximately 50 feet of drive to tip elevation. In general, the logs indicate blow counts were in the range of 3 to 8 blows per foot. We noted slightly higher blow counts in the range of 11 to 13 blows per foot in the last 8 feet of drive for Pile No. 2.

Based on Wave Equation Analysis for Piles (WEAP) we previously performed, stresses in the pile during driving should have been relatively low for the observed blow counts within the stiff to very stiff clay. The observed pile damage and the reported driving behavior does not appear to us to be consistent with encountering a dense or hard soil layer. Accordingly, buckling should not have been expected, in our opinion. This conclusion is supported by the observation that the same size of pile has been successfully used at all pier locations except Pier E5E without appreciable damage. A copy of our prior WEAP analysis is attached.

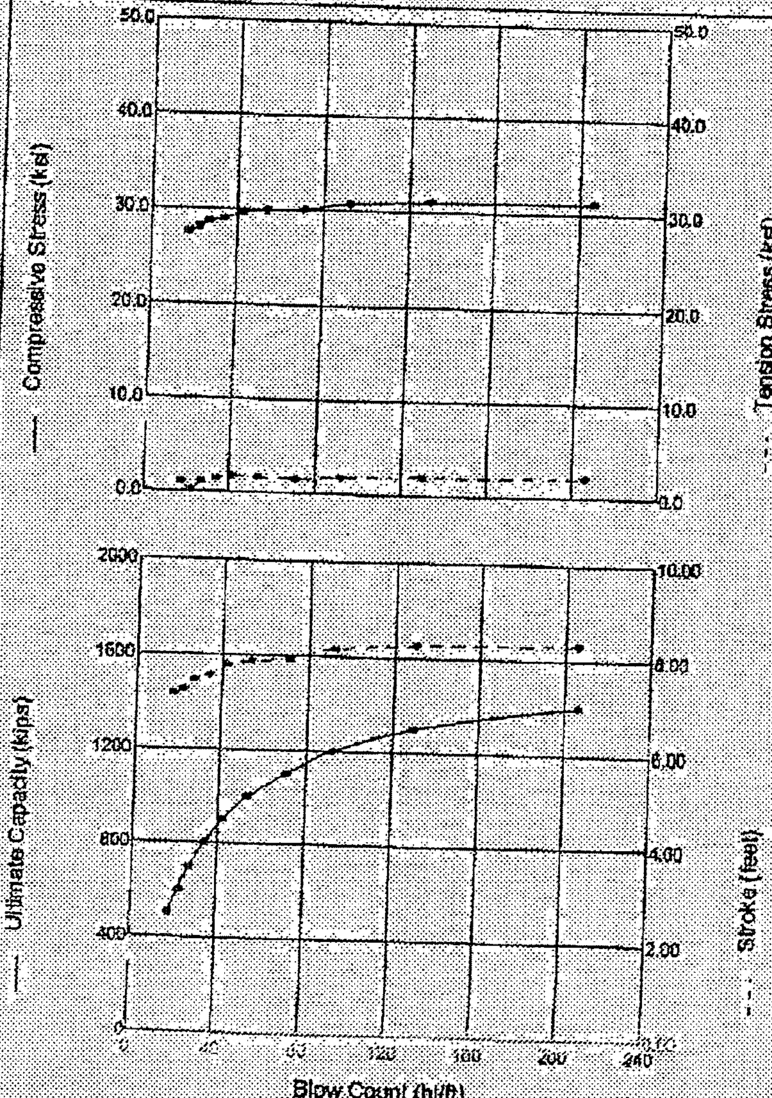
Please call if you have questions or require additional information regarding any of the above.

GHS:GWH:tt
TACO:\1\1092030\00\Finals\109203000M66PierE5E42InspudPileDamageResp.doc

Attachment:
WEAP Analysis

GeoEngineers, Inc.
120' Total Length -20'stickup

21-Jan-2003
GRLWEAP (TM) Version 1998-2

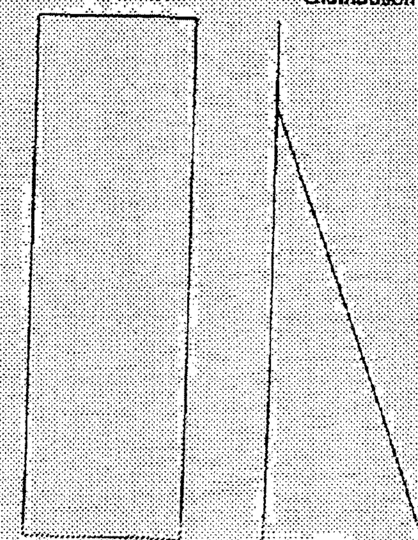


DELMAG D 80-23

Efficiency	0.800
Helmet	5.80 kips
Skin Quake	0.098 in
Toe Quake	0.098 in
Skin Damping	0.198 sec/ft
Toe Damping	0.152 sec/ft
Pile Length	120.00 ft
Pile Top Area	65.20 in2

Pile Model

Skin Friction
Distribution



Res. Shaft = 85 %
(Proportional)

GeoEngineers, Inc.
120' Total Length -20'stickup

21-Jan-2003
GRLWEAP(TM) Version 1998-2

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
500.0	27.702	0.989	17.9	7.15	74.83
599.9	28.128	0.139	22.4	7.24	74.24
699.9	28.752	1.022	27.4	7.45	76.29
799.9	29.105	1.406	34.1	7.56	77.16
899.9	29.696	1.591	42.0	7.78	79.51
999.9	30.035	1.601	53.5	7.88	80.34
1099.9	30.154	1.491	70.7	7.94	80.56
1199.9	30.852	1.686	92.0	8.16	83.10
1299.9	31.177	1.887	130.1	8.27	84.15
1399.9	31.327	2.092	206.5	8.32	84.59

EXHIBIT 8

STATE OF CALIFORNIA-BUSINESS, TRANSPORTATION AND HOUSING AGENCY ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF TRANSPORTATION

SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607
Facsimile Number: (510) 622-5165

Flex Your Power
Be Energy Efficient!

July 22, 2004

KFM, a JV
220 Burma Road
Oakland, CA 94607

Contract: 04-012024
04-SF, Ala-80-13.9/14.3, 0.0/1.6
SFOBB Skyway Project
State Letter # 5.03.1-005037

Subject: KFM-LET-854, regarding local buckling of spud piles at Pier E5 E

Dear Mr. Skoro,

Attention: Scott Hanson,

We acknowledge receipt of KFM Letter KFM-LET-000854. As stated in State letters 4786 and 3531, your request for a Change Order to address the alleged differing site condition is denied. We find no merit in the assertion that the buckling of the temporary piles at Pier E5 E was caused by a Differing Site Condition. No additional compensation will be forthcoming. Your attention is directed to the requirements of sections 9-1.04, "Notice of Potential Claim," of the Standard Specifications and 5-1.02, "Differing Site Conditions," of the Special Provisions. If you wish to pursue your objection to the Engineer's decision, please address the requirements of this section by August 6, 2004.

Furthermore, the two statements made in KFM-LET-854 are incorrect. First of all, there **is** a significant increase in undrained shear strength in two soil layers through which these temporary piles were driven, at approximate elevation -18m (-59') and at -24m (-79'). Based upon the drawings attached to KFM-LET-800, the damaged sections of pile extended only as high as elevation -63'. Secondly, API 2A is recommended practice for piling for the offshore industry and demonstrates that the pile thickness of the 42" spud piles was less than recommended for both driving and local buckling.

If you have any questions regarding this matter, please contact Mark Woods at (510) 622-5107.

Sincerely,

Mark P. Woods
Foundation Structure Representative

for: Douglas B. Coe
Resident Engineer

cc: D. Coe,
D. Salladay,
P. Siegenthaler

file: 5.03.1, 62.00

EXHIBIT 9

KFM
KIEWIT / FCI / MANSON, A JV

06-Aug-2004

Serial Letter: KFM-LET-000887

California Department of Transportation
SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607

Attention: Douglas Coe, P.E.

Reference: Skyway Bridge Project (Caltrans Contract No. 04-012024)
KFM Job No. 364/3726

Subject: NOPC 10 - Changed Condition – Collapsed spud pile at 5E

Dear Doug:

Please find attached NOPC 10 as requested by Caltrans letter 5037.

Sincerely,
KIEWIT/FCI/MANSON, a JV

for A.T. (Tom) Skoro
Project Director

cc: file

Kiewit/FCI/Manson, A JV • 220 Burma Road • Oakland, CA 94607
P.O. Box 23223 • Oakland, CA 94623 • 510 419-0120 • Fax 510 839-0666

STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION

NOTICE OF POTENTIAL CLAIM

CEM-6201 (REV 3/2001)

FOR STATE USE ONLY

TO	CONTRACT NUMBER	DATE
Doug Coe (resident engineer)	**04-012-024**	**8-2-04**

This is a Notice of Potential Claim for additional compensation under the provisions of Section 9-1.04 of the ***Standard Specifications.*** **The act of the engineer, or his/her failure to act, or the event, thing, occurrence, or other cause giving rise to the potential claim occurred on** DATE **7-22-04**

The particular circumstances of this potential claim are described in detail as follows:

KFM letters 637,689, and 800 describe damage to KFM's 42" pile template spud pile. The letters also describe the disagreement with the Engineer's determination and KFM's process of evaluating the cause of the damage concluding that a latent or unknown subsurface physical condition most likely caused the damage.

The reasons for which I believe additional compensation may be due are:

Additional compensation is due pursuant to Special Provision 5-1.012 and Standard Specification 5-1.116. These Contract sections state that if a latent or unknown physical condition causes an increase in the cost of the work, the Contractor is to be allowed an adjustment to the Contract price.

The nature of the costs involved and the amount of the potential claim are described as follows:
(If accurate cost figures are not available, provide an estimate, or describe the types of expenses involved.)

The cost of the additional work is estimated to be $200,000

This NOPC is also KFM's certification that the following were made in preparation of the bid: a review of the contract, a review of the "Materials Information," a review of the log of test borings and other records of geotechnical data to the extent they were made available to bidders prior to the opening of bids, and an examination of the conditions above ground at the site.

The undersigned originator (Contractor or Subcontractor as appropriate) certifies that the above statements are made in full cognizance of the California False Claims Act, Government Code sections 12650-12655. The undersigned further understands and agrees that this potential claim to be further considered unless resolved, must be restated as a claim in response to the states proposed final estimate in accordance with Section 9-1.07B of the Standard Specifications.

KFM

~~SUBCONTRACTOR~~ or CONTRACTOR
(Circle one)

[signature]

(Authorized Representative)

For subcontractor notice of potential claim

This notice of potential claim is acknowledged and forwarded by

PRIME CONTRACTOR

(Authorized Representative)

ADA Notice For individuals with sensory disabilities, this document is available in alternate formats. For information call (916) 263-2041 or TDD (916) 263-2044 or write Records and Forms Management, 1120 N Street, MS-89, Sacramento, CA 95814.

CEM6201

STATE OF CALIFORNIA-BUSINESS, TRANSPORTATION AND HOUSING AGENCY ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF TRANSPORTATION
SFOBB – Skyway Project
345 Burma Road
Oakland, CA 94607
Facsimile Number: (510) 622-5165

Flex Your Power
Be Energy Efficient!

July 22, 2004

KFM, a JV
220 Burma Road
Oakland, CA 94607

Contract: 04-012024
04-SF, Ala-80-13.9/14.3, 0.0/1.6
SFOBB Skyway Project
State Letter # 5.03.1-005037

Subject: KFM-LET-854, regarding local buckling of spud piles at Pier E5 E

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Attention: Scott Hanson,

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Furthermore, the two statements made in KFM-LET-854 are incorrect. First of all, there **is** a significant increase in undrained shear strength in two soil layers through which these temporary piles were driven, at approximate elevation -18m (-59') and at -24m (-79'). Based upon the drawings attached to KFM-LET-800, the damaged sections of pile extended only as high as elevation -63'. Secondly, API 2A is recommended practice for piling for the offshore industry and demonstrates that the pile thickness of the 42" spud piles was less than recommended for both driving and local buckling.

If you have any questions regarding this matter, please contact Mark Woods at (510) 622-5107.

Sincerely,

Mark P. Woods
Foundation Structure Representative

for: Douglas B. Coe
Resident Engineer

cc: D. Coe,
D. Salladay,
P. Siegenthaler

file: 5.03.1, 62.00

project, full compensation for any additional cost involved shall be considered as included in the contract price paid for the item of work involved and no additional compensation will be allowed therefor.

5-1.116 DIFFERING SITE CONDITIONS

- During the progress of the work, if subsurface or latent physical conditions are encountered at the site differing materially from those indicated in the contract or if unknown physical conditions of an unusual nature, differing materially from those ordinarily encountered and generally recognized as inherent in the work provided for in the contract, are encountered at the site, the party discovering those conditions shall promptly notify the other party in writing of the specific differing conditions before they are disturbed and before the affected work is performed.
- Upon written notification, the Engineer will investigate the conditions, and if the Engineer determines that the conditions materially differ and cause an increase or decrease in the cost or time required for the performance of any work under the contract, an adjustment, excluding loss of anticipated profits, will be made and the contract modified in writing accordingly. The Engineer will notify the Contractor of the Engineer's determination whether or not an adjustment of the contract is warranted.
- No contract adjustment which results in a benefit to the Contractor will be allowed unless the Contractor has provided the required written notice.
- No contract adjustment will be allowed under the provisions specified in this section for any effects caused on unchanged work.
- Any contract adjustment warranted due to differing site conditions will be made in conformance with the provisions in Section 4-1.03, "Changes," except as otherwise provided.

5-1.12 CHARACTER OF WORKERS

- If any subcontractor or person employed by the Contractor shall appear to the Engineer to be incompetent or to act in a disorderly or improper manner, they shall be discharged immediately on the request of the Engineer, and that person shall not again be employed on the work.

5-1.13 FINAL INSPECTION

- When the work has been completed, the Engineer will make the final inspection.

5-1.14 COST REDUCTION INCENTIVE

- The Contractor may submit to the Engineer, in writing, proposals for modifying the plans, specifications or other requirements of the contract for the sole purpose of reducing the total cost of construction. The cost reduction proposal shall not impair, in any manner, the essential functions or characteristics of the project, including but not limited to service life, economy of operation, ease of maintenance, desired appearance, or design and safety standards.
- Cost reduction proposals shall contain the following information:

1. A description of both the existing contract requirements for performing the work and the proposed changes.

H. At the completion of the contract, one compiled set of all approved working drawings (in electronic form and including all corrections and revisions) shall be furnished to the Engineer. The index shall be the first file on the CD.

I. At the completion of the contract, one set of reduced prints on 75-g/m2 (minimum) bond paper, 279 mm x 432 mm in size, of the corrected original tracings of all approved working drawings, including all corrections and revisions shall be furnished to the Engineer. Reduced prints that are common to more than one structure shall be submitted

---------- End of Page 19 in the original Special Provisions ----------

for each structure. An index prepared specifically for the drawings for each structure containing sheet numbers and titles shall be included on the first reduced print in the set for each structure. Reduced prints for each structure shall be arranged in the order of drawing numbers shown in the index

Working drawings shall be stamped and signed by an engineer who is registered as a Civil Engineer in the State of California. When independently checked calculations are required, these calculations shall be stamped and signed by another engineer who is registered as a Civil Engineer in the State of California.

Working drawings shall be submitted sufficiently in advance of the start of the affected work to allow time for review by the Engineer and correction by the Contractor of the drawings without delaying the work. The time shall be proportional to the complexity of the work, but in no case shall the time be less than the review time as specified for the type of working drawings as required elsewhere in these special provisions.

The Engineer will review a working drawing submittal for completeness. The Engineer will notify the Contractor in writing when a given working drawing submittal is determined to be complete, and the review period shall begin on that day.

Should the Engineer fail to review the complete working drawing submittal within the time specified, and the Contractor's controlling operation on the critical path is delayed (as determined by the Engineer) by the Engineer's failure to review within the time specified, an extension of time will be granted in conformance with the provisions in Section 8-1.09, "Right of Way Delays," of the Standard Specifications.

5-1.011 EXAMINATION OF PLANS, SPECIFICATIONS, CONTRACT, AND SITE OF WORK

The second paragraph of Section 2-1.03, "Examination of Plans, Specifications, Contract, and Site of Work," of the Standard Specifications is amended to read:

- Where the Department has made investigations of site conditions, including subsurface conditions in areas where work is to be performed under the contract, or in other areas, some of which may constitute possible local material sources, bidders or Contractors may, upon written request, inspect the records of the Department as to those investigations subject to and upon the conditions hereinafter set forth.

Attention is directed to "Differing Site Conditions" of these special provisions regarding physical conditions at the site which may differ from those indicated in "Materials Information," log of test borings or other geotechnical information obtained by the Department's investigation of site conditions.

5-1.012 DIFFERING SITE CONDITIONS

Attention is directed to Section 5-1.116, "Differing Site Conditions," of the Standard Specifications.

During the progress of the work, if subsurface or latent conditions are encountered at the site differing materially from those indicated in the "Materials Information," log of test borings, other geotechnical data obtained by the Department's investigation of subsurface conditions, or an examination of the conditions above ground at the site, the party discovering those conditions shall promptly notify the other party in writing of the specific differing conditions before they are disturbed and before the affected work is performed.

The Contractor will be allowed 15 days from the notification of the Engineer's determination of whether or not an adjustment of the contract is warranted, in which to file a notice of potential claim in conformance with the provisions of Section 9-1.04, "Notice of Potential Claim," of the Standard Specifications and as specified herein; otherwise the decision of the Engineer shall be deemed to have been accepted by the Contractor as correct. The notice of potential claim shall set forth in what respects the Contractor's position differs from the Engineer's determination and provide any additional information obtained by the Contractor, including but not limited to additional geotechnical data. The notice of potential claim shall be accompanied by the Contractor's certification that the following were made in preparation of the bid: a review of the contract, a review of the "Materials Information," a review of the log of test borings and other records of geotechnical data to the extent they were made available to bidders prior to the opening of bids, and an examination of the conditions above ground at the site. Supplementary information, obtained by the Contractor subsequent to the filing of the notice of potential claim, shall be submitted to the Engineer in an expeditious manner.

5-1.015 LABORATORY

When a reference is made in the specifications to the "Laboratory," the reference shall mean the Division of Materials Engineering and Testing Services and the Division of Structural Foundations of the Department of Transportation, or established laboratories of the various Districts of the Department, or other laboratories authorized by the Department to test materials and work involved in the contract. When a reference is made in the specifications to the "Transportation Laboratory," the reference shall mean the Division of Materials Engineering and Testing Services and the Division of Structural Foundations, located at 5900 Folsom Boulevard, Sacramento, CA 95819, Telephone (916) 227-7000.

---------- End of Page 20 in the original Special Provisions ----------

5-1.017 CONTRACT BONDS

Attention is directed to Section 3-1.02, "Contract Bonds," of the Standard Specifications and these special provisions.

5-1.018 EXCAVATION SAFETY PLANS

Section 5-1.02A, "Trench Excavation Safety Plans," of the Standard Specifications is amended to read:

5-1.02A Excavation Safety Plans

- The Construction Safety Orders of the Division of Occupational Safety and Health shall apply to all excavations. For all excavations 1.5 m or more in depth, the Contractor shall submit to the Engineer a detailed plan showing the design and details of the protective systems to be provided for worker protection from the hazard of caving ground during excavation. The detailed plan shall include any tabulated data and any design calculations used in the preparation of the plan. Excavation shall not begin until the detailed plan has been reviewed and approved by the Engineer.
- Detailed plans of protective systems for which the Construction Safety Orders require design by a registered professional engineer shall be prepared and signed by an engineer who is registered as a Civil Engineer in the State of California, and shall include the soil classification, soil properties, soil design calculations that demonstrate adequate stability of the protective system, and any other design calculations used in the preparation of the plan.
- No plan shall allow the use of a protective system less effective than that required by the Construction Safety Orders.
- If the detailed plan includes designs of protective systems developed only from the allowable configurations and slopes, or Appendices, contained in the Construction Safety Orders, the plan shall be submitted at least 5 days before the Contractor intends to begin excavation. If the detailed plan includes designs of protective systems developed from tabulated data, or designs for which design by a registered professional engineer is required, the plan shall be submitted at least 3 weeks before the Contractor intends to begin excavation.
- Attention is directed to Section 7-1.01E, "Trench Safety."

The third paragraph of Section 19-1.02, "Preservation of Property," of the Standard Specifications is amended to read:

- In addition to the provisions in Sections 5-1.02, "Plans and Working Drawings," and 5-1.02A, "Excavation Safety Plans," detailed plans of the protective systems for excavations on or affecting railroad property will be reviewed for adequacy of protection provided for railroad facilities, property, and traffic. These plans shall be submitted at least 9 weeks before the Contractor intends to begin excavation requiring the protective systems. Approval by the Engineer of the detailed plans for the protective systems will be contingent upon the plans being satisfactory to the railroad company involved.

5-1.02 LABOR NONDISCRIMINATION

Attention is directed to the following Notice that is required by Chapter 5 of Division 4 of Title 2, California Code of Regulations.

NOTICE OF REQUIREMENT FOR NONDISCRIMINATION PROGRAM

(GOV. CODE, SECTION 12990)

Your attention is called to the "Nondiscrimination Clause", set forth in Section 7-1.01A(4), "Labor Nondiscrimination," of the Standard Specifications, which is applicable to all nonexempt State contracts and subcontracts, and to the "Standard

EXHIBIT 12

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF, Ala	80	13.9/14.3, 0.0/1.6	939	978

GEOTECHNICAL PROFESSIONAL

6-4-01
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
T.W. McNeilan
No. 0583
Exp. 12/31/00
Geotechnical
STATE OF CALIFORNIA

FUGRO-EARTH MECHANICS,
A JOINT VENTURE.
7700, Edgewater Dr., Ste. 848,
Oakland, CA 94621
(510) 833-5100

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

Caltrans now has a web site! To get to the web site, go to: http://www.dot.ca.gov

Metric

SOIL AND ROCK TYPES

- Well graded GRAVEL (GW)
- Poorly graded GRAVEL (GP)
- GRAVEL with sand (GP or GW)
- GRAVEL with clay (GP or GW)
- Clayey GRAVEL (GC)
- GRAVEL with silt (GP or GW)
- Silty GRAVEL (GM)
- Well graded SAND (SW)
- Poorly graded SAND (SP)
- SAND with gravel (SP or SW)
- SAND with clay (SP-SC)
- Clayey SAND (SC)
- Silty SAND (SM)
- SAND with silt (SP-SM)
- Fat CLAY (CH)
- Sandy fat CLAY (CH)
- Lean CLAY (CL)
- Sandy lean CLAY (CL)
- Silty CLAY (CL-ML)
- Elastic SILT (MH)
- SILT (ML)
- Sandy SILT (ML)
- Clayey silt (ML/CL)
- Highly plastic ORGANICS (OH)
- Low plasticity ORGANICS (OL)
- SANDSTONE (Rx)
- SILTSTONE (Rx)
- CLAYSTONE (Rx)
- Interbedded Rock Strata (Rx)
- CONGLOMERATE (Rx)

SAMPLERS

- 76mm-OD, 72mm-ID Thin Walled Tube
- 57mm-OD, 54mm-ID* Thin Walled Driven Tube
- 83.5mm-OD, 54mm-ID Offshore Liner
- 76mm-OD, 60mm-ID Modified California Liner
- 51mm-OD, 35mm ID SPT Split Spoon Sampler
- Rock Core (inset symbol represents recovery (%))

* A variable-OD, 54-mm-ID tapered driven tube was used to sample coarse sand, gravel and rock

TUBE AND OFFSHORE LINER SAMPLERS

WOH	Offshore Liner sample advanced with the weight of an 80 kg hammer.
PUSH or SAVE	Pushed thin-walled 76mm-OD-tube.
15/60cm	Number of blows required to produce the indicated penetration using a 54mm-ID-tube sampler. The sampler was driven with an 80 kg downhole hammer dropped approximately 1.5 m.

STANDARD PENETRATION TEST AND MODIFIED CALIFORNIA LINER SAMPLERS

Samplers were driven with a 63.5 kg above deck hammer dropped approximately 760 mm.

20	Number of blows to produce 30 cm of penetration after the initial 15 cm of seating.
86/28cm	Number of blows required to produce the indicated penetration after an initial 15 cm seating.
Ref/8cm	50 blows produced the indicated penetration during the initial 15 cm interval.

Note: In rock coring interval recovery(%)-RQD are shown in the Blow Count column.

CLASSIFICATION TESTS BLOW COUNTS AND ROCK QUALITY

- PERCENT PASSING #200 SIEVE
- WATER CONTENT (%)
- SUBMERGED UNIT WEIGHT (KCF)
- THEORETICAL SUBMERGED UNIT WEIGHT (KCF)
- PLASTIC LIMIT — LIQUID LIMIT
- EQUIVALENT SPT BLOW COUNT

STRENGTH TESTS

- POCKET PENETROMETER
- TORVANE
- REMOTE VANE
- MINIATURE VANE (@ RESIDUAL VANE)
- UNCONSOLIDATED UNDRAINED TRIAXIAL
- SWEDISH FALL CONE
- UNCONFINED COMPRESSION (SOIL)
- ◇Δ (Open symbols indicate remolded tests)
- Soil Strength Exceeds Capacity of Measuring Device
- Shear Strength Interpreted from CPT Tip Resistance

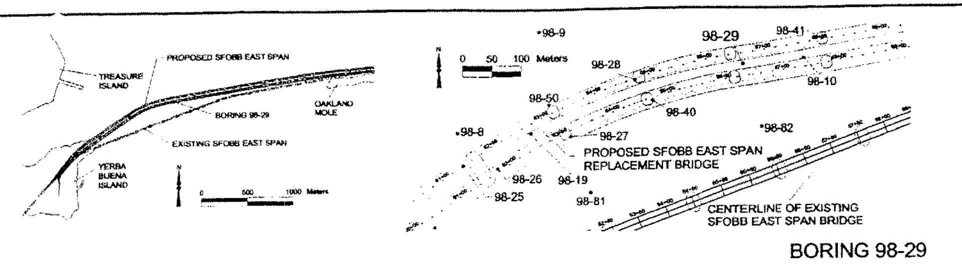
STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength kPa
Very Soft	less than 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	100 to 200
Hard	greater than 200

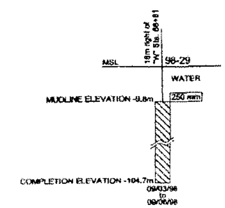
DENSITY OF GRANULAR SOILS

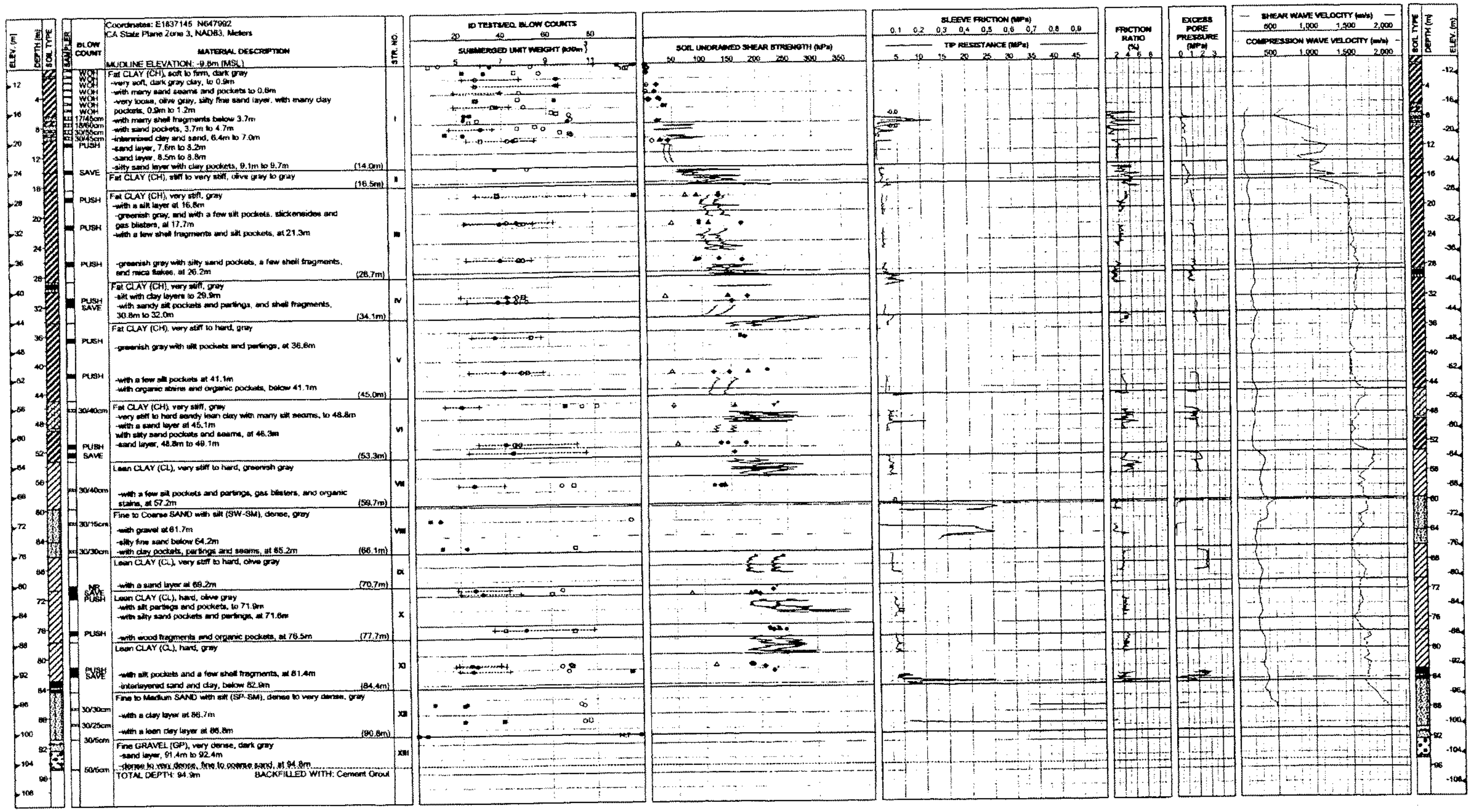
Descriptive Term	Relative Density (%)*
Very Loose	less than 15
Loose	15 to 35
Medium Dense	35 to 65
Dense	65 to 85
Very Dense	greater than 85

*Estimated from sampler driving record and CPT tip resistance.



BORING 98-29





DESIGN OVERSIGHT: REID BUELL, 11/17/00	DRAWN BY: SACHIN DESHPANDE	FIELD INVESTIGATOR: J.CHACKO/M.ZUEGER	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER: T.W.McNEILAN	BRIDGE NO. 34-0006	SAN FRANCISCO OAKLAND BAY BRIDGE EAST SPAN SEISMIC SAFETY PROJECT	
	CHECKED BY: J.CHACKO	DATE 06/14/99	ORIGINAL SCALE IN CENTIMETERS FOR REDUCED PLANS 0 1 2 3 4 5 6 7 8 9 10	CU 04 EA 012021	KM POST / POST MILE	LOG OF TEST BORINGS 8 OF 47	

DISREGARD PRINTS BEARING EARLIER REVISION DATES → REVISION DATES (PRELIMINARY STAGE ONLY) 7/19/99 2/15/00 8/02/00 11/17/00 SHEET 607 OF

100% PS&E

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A. Three sets of transparent drawings for each survey;
B. Three sets of computer sheet printouts or calculation sheets for dredging quantities for each survey; and
C. Three sets of cross-sections for each survey.

The Contractor shall submit for each survey, the ASCII file of raw and corrected survey data. Data shall be on 3 1/2" (1.44 MB) disks, operating under MSWindows 2000 or newer version. The files shall have hydrosurvey information, in both raw and adjusted format. The raw data shall be original data from the hydrosurvey computer. The adjusted data shall be corrected to National Ocean Survey NGVD datum. The record of raw data shall be comma delimited and consist of the following information: index, "x" coordinate; "y" coordinate; "z" elevation; and time. Each adjusted record shall consist of the following information: index; "x" coordinate; "y" coordinate; "z" elevation; time; and tide. The index shall be the first entry, representing the sequence that each point was taken. The index shall be numerical, beginning with the number "one" and continuing until a 24 hour work effort is completed. Each day shall be in one file (one or more disks). This convention is applicable for both raw and adjusted data. Time shall be reported in Gregorian day and military hours and seconds. (For example, "17 March 2001, 9:00 a.m." would be "170301, 090000"). The recording distance between the hydrosurvey points shall be 3 meters or less. All data recorded shall be in ASCII test. Other data collection formats will be considered if presented by the Contractor. Revisions in collection format will not be considered after the project has begun. All alternatives shall be approved by the Engineer.

The Contractor shall provide a complete listing of hydrographic equipment to be used on the project prior to the survey conference specified herein below.

At least five calendar days prior to performing any survey, the person responsible for that survey, the Contractor's chief surveyor and/or the independent surveyor, shall meet with the Engineer in a survey conference to outline the scope of survey and section interval. No survey work shall be performed until such conference has taken place.

The Department will retain an amount equal to 5 percent of the estimated value of the associated item of work performed during each estimate period in which the Contractor fails to complete the hydrographic surveys.

MEASUREMENT AND PAYMENT

Full compensation for Dredging Operation Plan preparation and updating; preparing and implementing Solid Debris Management Plan; overflow and leakage monitoring; implementing air quality requirements; performing control and monitoring surveys; preparation of disposal site verification logs; implementing SMMP requirements; and performing hydrographic surveys including data collection and preparation of drawings, cross-sections and calculations shall be considered as included in the contract prices paid for the items of work involved and no additional compensation will be allowed therefor.

10-1.24 PILING

GENERAL

Piling shall conform to the provisions in Section 49, "Piling," of the Standard Specifications, and these special provisions.

Foundation information is included in the "Information Handout" available to the Contractor as provided for in Section 2-1.03, "Examination of Plans, Specifications, Contract, and Site of Work," of the Standard Specifications.

Removal of underwater debris that is in conflict with construction work shown shall be as specified in "Existing Highway Facilities," of these special provisions.

Removal and relocation of materials from within the steel shells for construction of cast-in-steel shell concrete piling shall conform to the additional requirements of "Dredging" elsewhere in these special provisions.

Soil samples and rock cores are available for viewing. Contact the Toll Bridge Duty Senior at the office of the Toll Bridge Duty Senior at the District 4 Office, 111 Grand Avenue, Oakland, CA 94612, email: duty_senior_tollbridge_district04@dot.ca.gov, telephone (510) 286-5549.

Attention is directed to "Sound Control Requirements," and "Cost Reduction Incentive Proposals For Pile Driving Template," of these special provisions regarding pile driving.

Attention is directed to "Strong Motion Detection System," and "Pile Corrosion Monitoring System," of these special provisions regarding equipment to be installed in piling.

---------- Approximate End of Page 115 in the original Special Provisions ----------

Attention is directed to "Order of Work" and "Relations with United States Coast Guard" of these special provisions regarding redriving of existing test piles and certification procedures for welding.

The requirements in Section 49-1.03, "Determination of Length," of the Standard Specifications shall not apply.

Driven piling shall be installed and shall be of such length as required to obtain the specified pile tip elevation and to extend in to the pile cap, as shown on the plans.

All piles shall be clearly marked along their entire length in increments of 250 mm with more prominent markings every meter. Marking shall be made by white paint lines 50 mm wide. Markings shall be accurately placed on the pile with a tape measure that is at least 30 meters in length such that the intended measurement is true at the bottom of the marking. Markings shall be visible from all directions and shall indicate cumulative length from the pile toe.

Pile installation procedures shall consider the presence of soft soils that allow piles to penetrate significant distances under self-weight and the weight of the hammer, dense soils that result in hard driving, soils that gain strength during delays in driving, wind and wave excitation, pile batter, and tidal flow fluctuation.

DRIVING EQUIPMENT

Pile hammer energy input to the pile will be verified by the Engineer using dynamic monitoring.

Primary Hammer

The primary hammer shall be defined as a hydraulic impact hammer with a minimum manufacturer's rated energy of 1700 kJ. The Contractor shall maintain the primary hammer at the site and it shall be fully operational at all times during pile driving operations.

Secondary Hammer(s)

At the option of the Contractor (except for Piers E3 through E5), secondary impact hammers with a minimum manufacturer's rated energy of less than 1700 kJ may be used to install driven piling. Secondary impact hammers shall have a minimum manufacturer's rated energy of not less than 500 kJ for hydraulic hammers and not less than 750 kJ for air hammers. Said hammers shall have a penetration rate of not less than 3 mm per blow for continuous driving.

Secondary impact hammers shall have the capacity to operate with a minimum of 20 blows per minute at full stroke. Prototype impact hammers of a type still under development will not be permitted.

Diesel and steam impact hammers shall not be used to install driven piling.

Secondary hammers shall not be used to install driven piling at piers E3 to E5 below pile tip elevation –50 meters NGVD.

If piles do not meet the requirements of Pile Penetration Acceptance, as defined in this section, the Contractor shall replace the secondary hammer with the approved primary hammer and continue driving within 48 hours.

Jetting and drilling in conformance with Section 49-1.05, "Driving Equipment," of the Standard Specifications shall not be used.

At the option of the Contractor, vibratory hammers may be used to install piling to no deeper than pile tip elevation –35 meters NGVD.

PILE DRIVING TEMPLATE AND PILE HANDLING SUBMITTAL

The Contractor shall provide a pile driving template to maintain piling support and alignment during pile installation. Prior to installing driven piling, the Contractor shall submit to the Engineer for approval in accordance with the provisions in "Working Drawings," of these special provisions, working drawings for the pile driving template and pile handling procedures.

The Contractor shall allow the Engineer 50 working days after complete drawings and all supporting calculations are submitted for review of the pile driving template working drawings.

Pile handling shall conform to the recommendations in API RP2A "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms."

Working drawings for the pile driving template and pile handling procedures shall include the following:

A. Details for installation and removal of the template.
B. Locations for use, if there is more than one template.
C. Details and equipment used for handling of pile including the use of temporary lifting or handling attachments and supporting brackets.

---------- End of Page 116 in the original Special Provisions ----------

D. Details and methods for cutting the pile at the specified cut-off elevation and removing the pile head.
E. A list of all tasks required to install the piles and a written procedure for performing the work.
F. Details and equipment associated with the pile driving template including cranes and crane vessels, mooring systems and anchor patterns, transport barges, pile supports and fastenings.

Working drawings for the pile driving template and pile handling procedures shall be supplemented by calculations, and the calculations shall include the following:

A. Details and calculations demonstrating how the pile installation system will provide and maintain the specified axial and radial alignment of the pile, including pile batter, to within an angle of 1 in 100.
B. Details and calculations demonstrating adequate support and stability for the pile with the full operating weight and dynamic loading of the proposed hammer at the top of the pile.
C. Provisions to provide stability and maintain alignment during placement of the piles and in wind, wave and current conditions.
D. Provisions to prevent the pile from running under its own weight and the weight of the hammer, including, at a minimum, provisions to prevent the pile from penetrating below the top of the pile driving template or below water level.
E. Provisions for providing adequate work space for pile welding, cutting and inspection.
F. Provisions for ensuring the specified pile straightness, alignment, and support to prevent relative movement during field welding and to ensure that welding tolerances are met.
G. Method and equipment for monitoring pile alignment.
H. Calculation of pile stresses and deflections resulting from handling operations.

The pile driving template shall be removed after the installation of the piling. The pile driving template support piling shall be removed to at least 300 mm below original mudline. Procedures for installation and removal of the template and piles shall be included in the working drawing submittal.

DRIVING PILES

Pile heads to receive the hammer shall be square and smooth. The pile head face shall be perpendicular to the longitudinal axis of the pile. The maximum allowable deviation of any point on the pile head surface from a true perpendicular plane shall be 6 mm. Local deviations from a plane of best fit that are greater than 3 mm shall be ground smooth.

After driving, pile horizontal location, as measured at the centerline of the pile section, shall be within a radial distance of 150 mm, as measured at the cut-off elevation, from the location shown on the plans. Pile batter shall not deviate at a rate of more than 1 in 100 from the theoretical batter and the lateral alignment shown on the plans.

The Contractor shall survey each pile and record the top of pile location both vertically and horizontally and shall determine the pile's variance from the true line and design batter slope. Such variance shall be measured in two planes normal to each other. The top of pile location shall be surveyed immediately after each pile section is driven complete in place and also before a spliced pile section is driven. A second survey shall be carried out after all piles have been cut to their final elevations.

Pile surveying shall conform to "Construction Surveying," of these special provisions. All pile surveying data shall be submitted in writing to the Engineer at the completion of pile driving operations for a given work day.

A given pile that is driven to specified penetration and that fails to meet the alignment tolerances specified in this section will be rejected and replaced prior to driving further piles. Alternative corrective measures, if any, are subject to the prior approval of the Engineer and the cost of alternative measures shall be considered the sole responsibility of the Contractor.

Within 10 working days after the pile has been rejected, and prior to driving other piling, the Contractor shall revise his pile driving template. The Contractor shall submit to the Engineer for approval a plan for revised pile installation methods and this plan shall conform to the provisions in "Working Drawings," of these special provisions.

Revised pile template plans and pile handling shall also include the following:

A. A step by step description of the work to be performed, including revising pile driving template drawings and handling procedures as necessary.
B. A list of affected details and plan sheets.

The Engineer will notify the Contractor in writing when a complete plan has been received. The Contractor shall allow the Engineer 15 working days to review the revised pile installation plan after a complete submittal has been received.

REDRIVING EXISTING TEST PILES

At the locations shown on the plans, the Contractor shall redrive the three steel shell test piles previously installed on Contract No. 04-012084 as specified in "Order of Work," of these special provisions.

Redriving shall consist of using the primary hammer operating at its rated energy to advance the pile 250 mm or 670 blows, whichever occurs first.

Prior to redriving the existing test piles, the Contractor shall remove the navigation lighting assemblies and cover plates in accordance with "Existing Highway Facilities" of these special provisions. After redriving the piles, navigation lighting assemblies and cover plates shall be reattached to the piles and made fully operational.

PILE DRIVING REFUSAL

The requirements in Section 49-1.08, "Bearing Value and Penetration," of the Standard Specifications shall not apply.

Pile driving refusal shall be defined as the time when pile driving resistance, using the primary hammer operating at full rated energy according to the manufacturer's specifications meets one of the following conditions:

A. An average of 250 blows per 250 mm over a penetration of 1500 mm; or
B. 670 blows for 250 mm of penetration

If pile driving is interrupted for more than one hour, the above definition of refusal shall not apply until the pile has been driven 250 mm. During restart, or at any time, 670 blows in 125 mm shall be taken as pile driving refusal.

PILE DRIVING LOW RESISTANCE

If extremely low resistance to driving is experienced when driving within one meter of the specified pile tip elevation, the Engineer may modify the pile details shown on the plans.

When the Engineer directs the Contractor to modify the pile details shown on the plans, said work will be paid for as extra work as provided in Section 4-1.03D, "Extra Work," of the Standard Specifications.

PILE PENETRATION ACCEPTANCE

The requirements in Section 49-1.08 "Bearing Value and Penetration," of the Standard Specifications shall not apply.

Pile penetration acceptance shall be based on the following criteria:

A. Piles driven to the specified pile tip.
B. Piles that encounter refusal from continuous driving within 10 meters of the specified pile tip elevation for Piers E3, E4, and E5, and within 5 meters of the specified pile tip elevation for the remaining piers, will be accepted.

    1. Unless otherwise directed by the Engineer, piles shall be driven continuously throughout the final 10 meters above the specified pile tip elevation for Piers E3, E4, and E5, and throughout the final 5 meters above the specified pile tip elevation for the remaining piers. If driving refusal is encountered as a result of delays in pile driving occuring within these zones, the Contractor shall be responsible for soil plug removal down to an elevation not less than 7 meters above the specified pile tip elevation, splicing the pile (including all weld NDT) and any other measures necessary to advance the pile to the specified pile tip elevation shown on the plans.

C. For piles that encounter driving refusal at an elevation more than 10 meters above the specified pile tip elevation for Piers E3, E4, and E5, and at an elevation more than 5 meters of the specified pile tip elevation for the remaining piers, the Contractor shall remove the soil plug and continue driving the piles to the specified pile tip elevation. The Contractor's equipment and procedures shall be adequate to complete soil plug removal and resume driving within 48 hours. Soil plug removal shall not extend below an elevation that is 7 meters above the pile toe at the time of refusal.
D. Within 10 meters above the specified pile tip elevation for Piers E3, E4, and E5, and within 5 meters above the specified pile tip elevation for the remaining piers, if piles develop toe stresses in excess of 85 percent of the specified yield strength of the steel shell for Piers E3 through E5, or pile stresses in excess of 90 percent of the specified yield strength of the steel shell, as determined by the Engineer, pile driving shall be terminated and the pile will be accepted.
E. For piles that develop excessive driving stresses, as noted above, at an elevation more than 10 meters above the specified pile tip elevation for Piers E3, E4, and E5, and at an elevation more than 5 meters above the specified pile tip elevation for the remaining piers, the Contractor shall reduce the pile hammer stroke and continue driving the

pile to the specified pile tip elevation. When pile driving stresses are excessive and hammer stroke cannot be reduced without encountering refusal, the Contractor shall remove the soil plug and continue driving the piles to the

---------- End of Page 118 in the original Special Provisions ----------

specified pile tip elevation. Soil plug removal shall not extend below an elevation that is 7 meters above the pile toe at the time of refusal.

F. The Contractor shall provide a pile driving log at the completion of driving each pile or pile section. Upon completion of pile driving for a given pile, the Contractor shall allow the Engineer 48 hours to review the pile driving records. The Contractor shall not cut the top of the pile until the Engineer's review period is complete.

DRIVING SYSTEM SUBMITTAL

Prior to installing driven piling at any given pier, the Contractor shall provide a driving system submittal for driving at that pier, including driveability analysis, in conformance with the provisions in "Working Drawings," of these special provisions. A submittal shall be made for each eastbound and westbound pier. Technical data for all proposed driving systems (i.e., each hammer that may be brought onto the site) shall be included in the submittal.

The driving system submittal shall contain an analysis showing that the proposed driving systems will install piling to the specified pile tip elevation without soil plug removal and without overstressing the pile. Submittals shall include the following:

A. Complete description of soil parameters used, including soil quake and damping coefficients, skin friction distribution, percentage shaft resistance, and total soil resistance to driving.
B. List of all hammer operation parameters assumed in the analysis, including manufacturer's rated energy, fuel settings, stroke limitations, and hammer efficiency.
C. Driveability studies that are based on a wave equation analysis using a computer program that has been approved by the Engineer. Driveability studies shall model the Contractor's proposed driving systems, including the hammers, capblocks, pile cushions, and followers. The analyses shall consider a range of total soil resistance to driving and associated percentage shaft resistance for plugged and unplugged cases. The range of soil resistance to driving and percentage shaft resistance shall be determined for site conditions ranging from 10 meters above to 5 meters below the specified tip elevation shown on the plans. Separate analyses shall be completed at elevations above the specified pile tip elevations where difficult driving or pile splices are anticipated.
   Driveability analysis results shall include plots of the following:

   1. Maximum pile head and pile toe compressive stress versus blows per 250 mm.
   2.. Soils resistance to driving versus blows per 250 mm.

D. Details of equipment and procedures for removing the soil plug after successful pile driving and as a contingency in the case of driving refusal above an acceptable tip elevation.
E. Copies of all test results from any previous pile load tests, dynamic monitoring, and all driving records used in the analyses.
F. Completed "Pile and Driving Data Form," which is shown in these special provisions.
G. Estimated range of expected pile penetration due to self-weight and the weight of the hammer.
H. Written procedures for the pile driving and a pile installation schedule, including at a minimum the first and last pile at each footing.

The driving system submittal shall include the attached "Pile and Driving Data Form" completed for each hammer and driving system. The Contractor shall allow the Engineer 25 working days to review a driving system submittal.

The Contractor shall use the driving system and installation methods described in the approved driving system submittal for each pier location. Any change in hammers from those submitted and approved by the Engineer shall also meet the requirements for driving system submittals. Revised and new driving system submittals shall be approved by the Engineer prior to using corresponding driving systems on production piling. The Contractor shall allow the Engineer 25 working days to review each revised and each new driving system submittal after a complete set has been received, as determined by the Engineer.

Approval of pile driving equipment shall not relieve the Contractor of his responsibility to drive piling free of damage to the specified penetration.

---------- End of Page 119 in the original Special Provisions ----------

CALIFORNIA DEPARTMENT OF TRANSPORTATION
OFFICE OF TRANSPORTATION LABORATORY

PILE AND DRIVING DATA FORM

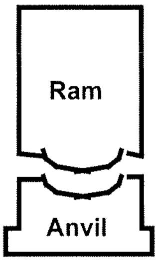
Structure Name : ____________________ Contract No.: ____________________

____________________ Project: ____________________

Structure No.: ____________________ Pile Driving Contractor or Subcontractor ____

Dist./Co./Rte./P.M.: ____________________ ____________________

(Pile Driven By)



Hammer

Manufacturer: ____________________ Model: ____________

Type: ____________________ Serial No.: ____________

Rated Energy: ____________ at ____________ Length of Stroke

Modifications: ____________________

____

____

____

____

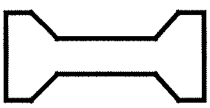
Capblock (Hammer Cushion)

Material: ____________________

Thickness: ____________ mm Area: ____________ mm^2

Modulus of Elasticity - E: ____________________ MPa

Coefficient of Restitution - e:



Pile Cap

Helmet
Bonnet
Anvil Block
Drivehead

Mass: ____________________ kg

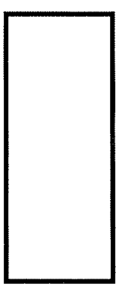
Pile Cushion

Material: ____________________

Thickness: ____________ mm Area: ____________ mm^2

Modulus of Elasticity - E: ____________________ MPa

Coefficient of Restitution - e: ____________________



Pile

Pile Type: ____________________

Length (In Leads): ____________________ m

kg/m.: ____________ Taper: ____________

Wall Thickness: ____________________ mm

Cross Sectional Area: ____________________ mm^2

Design Pile Capacity: ____________________ kN

Description of Splice: ____________________

____

Tip Treatment Description: ____________________

DISTRIBUTE one copy

- ☐ Translab, OSF Foundation Testing & Instrumentation
- ☐ Translab, OSF Structures Foundations
- ☐ Resident Engineer

Note: If mandrel is used to drive the pile, attach separate manufacturer's detail sheet(s) including mass (kg) and dimensions.

Submitted By: ____________________ Date: ____________

Phone No.: ____________

DYNAMIC MONITORING

Unless otherwise directed by the Engineer, the last 18 meters for each production pile, and each existing test pile will be monitored during driving (or redriving) for dynamic response to the driving equipment. Monitoring will be done by State forces using State-furnished dynamic pile analyzer monitoring instruments.

Monitoring attachments shall be fastened to the piles using the tapped holes shown on the plans. Each pile at piers E6 to E16 will be fitted with one set of attachments located on opposite sides of the pile. Each pile at piers E3 to E5 will be fitted with two such sets of attachments.

If the Contractor's driving system is such that monitoring instruments will be underwater, the Contractor shall notify the Engineer, in writing, at least 50 working days prior to commencement of driving operations.

The Contractor shall provide electric power (120-volt, 60 cycles stable power) for the State's monitoring equipment, access to the piles including a working platform, and shelter for State monitoring personnel and equipment.

Piles to be dynamically monitored shall be made available to State forces at least 8 hours prior to lifting. The pile shall be positioned so that State forces have safe access to the top 18 meters of the pile length for the installation of anchorages and control marks for monitoring. The Contractor shall rotate the piles on the blocks as directed by the Engineer.

Piles to be dynamically monitored shall be prepared and driven in accordance with the following:

A. The Engineer will determine if the Contractor's handling operations during lifting of the pile segment to be monitored are such that pile monitoring instrumentation can be bolted to the pile prior to lifting without damage to the instruments. If the Engineer determines that instruments cannot be mounted prior to lifting of the pile, operations shall be suspended for approximately 30 minutes before hammer placement. During this time the Contractor shall attach monitoring equipment onto the pile.
B. Prior to resuming driving operations, the Contractor shall connect electrical cables to the instrument package as approved by the Engineer.
C. Driving operations shall resume as directed by the Engineer. The Contractor's driving equipment shall provide sufficient clearances for monitoring instruments such that piles can be driven to the specified pile tip elevation without damage to the monitoring instruments.

Within 4 hours of completion of driving operations, the Contractor shall remove the cables and instrument package from the pile and deliver them to the Engineer. If monitoring instruments are underwater at the end of driving, the Contractor shall provide a diver and shall retrieve the cables and instruments.

The Contractor shall be responsible for damage to the State's cables and instruments caused by the Contractor's operations, and shall replace damaged cables or instruments in kind.

OPEN ENDED CAST-IN-STEEL-SHELL CONCRETE PILING

General

Cast-in-steel-shell concrete piling shall consist of open ended steel shells driven to the specified penetration and filled with reinforced cast-in-place concrete and shall conform to the provisions in Section 49-4, "Cast-in-Place Concrete Piles," of the Standard Specifications and these special provisions.

Attention is directed to "Steel Pipe Piling" of these special provisions.

In addition to driving, drilling or jetting within open ended steel shells to remove the soil plug may be necessary to obtain the specified penetration. The diameter of drilled holes shall be less than the clear inside diameter of the piling, including the shear rings. Jetting methods and procedures shall demonstrate that soil plug removal can be done at a predicted rate and in a controlled manner. Equipment or methods used for drilling or jetting shall not cause quick soil conditions, scouring, or caving of the hole and shall not damage the internal shear rings. If soil plug removal operations extend below the limit of the seal course concrete, as shown on the plans, the Contractor shall fill the void created by drilling or jetting with additional seal course concrete. Drilling or jetting shall not disturb the soil plug within 7 meters of the pile toe at any time during pile installation.

Concrete

At the Contractor's option, the Contractor may use either the 12.5-mm maximum combined aggregate grading or the 9.5-mm maximum combined aggregate grading. The grading requirements for the 12.5-mm maximum coarse aggregate and the 9.5-mm maximum coarse aggregate are shown in the following table:

---------- End of Page 121 in the original Special Provisions ----------

Sieve Sizes	Percentage Passing Primary Aggregate Nominal Size			
	12.5 mm x 4.75 mm		9.5 mm x 2.36 mm	
	Operating Range	Contract Compliance	Operating Range	Contract Compliance
19 mm	100	100		
12.5 mm	82 - 100	80 - 100	100	
9.5 mm	X ± 15	X ± 22	X ± 15	X ± 20
4.75 mm	0 - 15	0 - 18	0 - 25	0 - 28
2.36 mm	0 - 6	0 - 7	0 - 6	0 - 7

In the table above, the symbol X is the gradation which the Contractor proposes to furnish for the 9.5-mm sieve size.

The gradation proposed by the Contractor for the 12.5-mm x 4.75-mm primary aggregate or for the 9.5-mm x 2.36-mm primary aggregate shall be within the following percentage passing limits:

Primary Aggregate Nominal Size	Sieve Sizes	Limits of Proposed Gradation
12.5 mm x 4.75 mm	9.5 mm	40 - 78
9.5 mm x 2.36 mm	9.5 mm	50 - 85

The combined aggregate grading for the 12.5-mm x 4.75-mm primary aggregate nominal size or for the 9.5-mm x 2.36-mm primary aggregate nominal size shall be within the following limits:

Grading Limits of Combined Aggregate		
Sieve Sizes	Percentage Passing	
	12.5-mm Max.	9.5-mm Max.
19 mm	100	100
12.5 mm	90 - 100	90 - 100
9.5 mm	55 - 86	55 - 86
4.75 mm	45 - 63	45 - 63
2.36 mm	35 - 49	35 - 49
1.18 mm	25 - 37	25 - 37
600 µm	15 - 25	15 - 25
300 µm	5 - 15	5 - 15
150 µm	1 - 8	1 - 8
75 µm	0 - 4	0 - 4

The steel shells shall be installed open ended and shall have internal shear rings, as shown on the plans.

Steel Shells

The Contractor's attention is directed to "Steel Pipe Piling," of these special provisions.

Studs shall conform to "Steel Structures" of these special provisions.

Reinforcement

Reinforcement shall conform to the provisions in "Reinforcement," of these special provisions. Welded headed bar reinforcement shall conform to the provisions in "Welded Headed Bar Reinforcement," of these special provisions.

Construction

The Contractor shall submit to the Engineer for approval, a cleanout and inspection plan for open ended cast-in-steel-shell concrete piling. Care shall be taken during cleaning out of open ended steel shells to prevent disturbing the foundation material surrounding the pile and damaging the interior shear rings. The pile soil plug, as shown on the plans, shall not be cleaned out. Equipment or methods used for cleaning out steel shells shall not cause quick soil conditions or cause scouring or caving around or below the piles. The shell at the bottom of the concrete fill elevation shall be sealed in conformance with the provisions in Section 51-1.10, "Concrete Deposited Under Water," of the Standard Specifications. The sealed shell shall then be dewatered and cleaned out as specified herein. The Contractor shall maintain a hydrostatic head that is equal to the sea level on the soil plug until the seal course has been placed and allowed to cure for a period of time to be agreed by the Engineer.

---------- End of Page 122 in the original Special Provisions ----------

Open ended steel shells shall be free of any soil, rock or other material deleterious to the bond between concrete and steel prior to placing reinforcement and concrete. Interior surfaces of open ended steel shells and shear rings shall be 100% clean.

Verification of pile cleanout shall be demonstrated by a video camera capable of inspecting any location within the steel shell. At the option of the Engineer, verification of pile cleanout shall be by either real-time viewing of the inspection by the Engineer or by the Engineer viewing a recorded inspection of mutually agreed sections of the pile. Recordings shall indicate the azimuth and depth of the camera.

Drilling fluid, except for water, shall not be used inside the pile during the cleanout process.

The Contractor shall allow the Engineer 15 working days for review of the cleanout and inspection plan.

Reinforcement shall be placed and secured symmetrically about the axis of the pile and shall be securely blocked to clear the sides of the steel shell and the shear rings, and blocked or suspended to clear the top of the seal course.

Concrete fill for cast-in-place concrete piles shall be placed continuously. No construction joints will be permitted.

Concrete fill for cast-in-place piles shall be placed by use of a tremie tube or tubes, each of which are at least 250 mm in diameter. A hopper shall be attached to the tremie tube(s). Concrete pumps may be used to deliver concrete to a hopper that feeds to the tremie tube(s). Pumping concrete directly down the tremie pipe will not be permitted. Internal bracing for the steel reinforcing cage shall accommodate the delivery tube system.

Delivery tubes shall be capped with a watertight cap, or plugged with a good quality, tight fitting, moving plug. The cap or plug shall be designed to release as the tube is charged with concrete. The tremie tube shall extend to the bottom of the hole before charging the tube with concrete.

STEEL PIPE PILING

General

Steel pipe piling shall consist of steel shells for open ended cast-in-steel-shell concrete piling. Steel pipe piling shall conform to the provisions in Section 49-5, "Steel Piles," of the Standard Specifications and these special provisions.

Attention is directed to "Welding" of these special provisions regarding welding of steel pipe. Unless otherwise specified, welding of any work performed in conformance with the provisions in Section 49, "Piling," of the Standard Specifications, shall be in conformance with the requirements in AWS D1.1.

Wherever reference is made to the following American Petroleum Institute (API) specifications in the Standard Specifications, on the project plans, or in these special provisions, the year of adoption for these specifications shall be as follows:

API Codes	Year of Adoption
API SPEC 2B	1996

All requirements of the codes listed above shall apply unless specified otherwise in the Standard Specifications, on the plans or in these special provisions.

Handling devices may be attached to steel pipe piling. Welds attaching these devices shall be aligned parallel to the horizontal axis of the pile and shall conform to the requirements of field welding specified herein. All handling devices shall be removed from the permanent piling when no longer needed. All remaining welds shall be ground flush. Prior to making attachments, the Contractor shall submit a plan to the Engineer that includes the locations, handling and fitting device details, connection details, welding and removal procedures. Attachments shall not be made to the steel pipe piling until the plan is approved in writing by the Engineer. The Engineer shall have 5 working days to review the plan. Should the Engineer fail to complete the review within this time allowance and if, in the opinion of the Engineer, the Contractor's controlling operation is delayed or interfered with by reason of the delay in reviewing the plan, the delay will be considered a right of way delay as specified in Section 8-1.09, "Right of Way Delays," of the Standard Specifications.

For steel pipe piling, including any bar reinforcement in the piling, the time to be allowed for the Engineer to review the "Welding Report," specified in "Welding" of these special provisions, and respond in writing after all the required items have been received, shall be as follows:

Type of Welding	Review Time
Offshore field welding	24 hours
Bar reinforcement in piling	48 hours
All other pile welding	5 working days

Offshore field welding is defined as steel pipe pile splice welds made after stabbing the pile. No field welded steel pipe piling shall be installed, and no reinforcement in the piling shall be encased in concrete until the Engineer has approved the

---------- End of Page 123 in the original Special Provisions ----------

above requirements in writing. Should the Engineer fail to complete the review and provide notification within this time allowance, and if, in the opinion of the Engineer, the Contractor's controlling operation is delayed or interfered with by reason of the delay in notification, the delay will be considered a right of way delay in conformance with the provisions in Section 8-1.09, "Right of Way Delays," of the Standard Specifications.

At the Contractor's option, a steel pipe pile may be re-tapped to prevent pile set-up; however, the field welded splice shall remain at least one meter above the work platform until that splice is approved in writing by the Engineer.

The Contractor shall provide durable enclosures at field splice locations to allow welding during inclement weather conditions in accordance with the requirements in "Welding," of these special provisions.

Fabricated Steel Pipe

Fabricated steel pipe is defined as pipe produced at a permanent facility where a variety of steel fabrication including roll forming and welding steel plate into pipe is performed, where this pipe is at least 19 mm in wall thickness, where this pipe is produced in conformance with API SPEC 2B, and where this fabrication can be done on a daily basis. Fabricated steel pipe is a specifically engineered product. (i.e., Fabricated steel pipe is engineered for a specific project.)

Fabricated steel pipe used for steel pipe piling shall conform to API 2B and the following requirements:

A. An API site license and API monogram are not required.
B. Weld filler metal shall conform to the requirements in AWS D1.5 for the welding of ASTM Designation: A 709, Grade 50 steel, except that the qualification, pretest, and verification test requirements need not be conducted if certified test reports are provided for the consumables to be used.
C. Steel pipe piles and shear rings shall be fabricated from plate conforming to the requirements in ASTM A709, Grade 50. Steel pipe piles shall be fabricated in accordance with API Specification 2B.
D. The sulfur content of steel pipe piles shall not exceed 0.05 percent, except where through-thickness is designated on the plans. Where through-thickness is designated on the plans, steel shall conform to the low sulfur and 20% reduction of area requirements in AWS D1.5, Section 12.4.4.1.
E. The acceptance criteria for visual inspection of pile welds shall be AWS D1.1 criteria for statically loaded structures, except within the "Plastic Hinge Zone" designated on the plans, where the criteria for cyclically loaded structures subject to tensile stress shall apply.
F. The thickness transition between the pile sections with different wall thickness shall be no steeper than 1:3.

Field Welding

Field welding of steel pipe piling is defined as welding performed after the certificate of compliance has been furnished by the fabricator and shall conform to the following requirements:

A. Prior to positioning any 2 sections of steel pipe to be spliced by field welding, the Contractor shall minimize the offsets of the pipe ends to be joined .
B. Welds made in the flat position or vertical position (where the longitudinal pipe axis is horizontal) shall be single-vee or double-vee groove welds. Welds made in the horizontal position (where the longitudinal pipe axis is vertical, or near vertical) shall be single-bevel welds. Joint fit-ups shall conform to the requirements in AWS D1.1 and these special provisions.
C. The minimum thickness of the backing ring shall be 6 mm, and the ring shall be continuous. Splices in the backing ring shall be made by complete penetration welds. Radiographic or ultrasonic testing in conformance with the requirements in AWS D1.1, Section 6, shall be used on one out of each 10 splice welds to assure soundness of backing ring splices prior to final insertion into a pipe end. Attachment of backing rings to pipe ends shall be done using a continuous fillet weld on the inside of the pile. After fitting to the second pipe, tack welding shall be done in the root area of the weld splice or to spacers. Minimum size and length of tack welds shall be as defined by

AWS D1.1, Section 2.4.6. The gap between the backing ring and the steel pipe piling wall shall be no greater than 2 mm, except as follows:

1. Gaps greater than 2 mm, but not exceeding 6 mm may be seal welded using E7018 SMAW.
2. Gaps exceeding 6 mm shall be repaired by welding using E7018 SMAW, the weld groove shall be ground to provide the intended groove shape, and the area shall be inspected using magnetic particle testing prior to starting the groove weld.

Locations where fit-up gaps exceed 2 mm shall be marked so that they can be referenced during NDT. Backing rings shall have a minimum width of 3 times the thickness of the steel pipe piling to be welded so that the ring will not interfere with the interpretation of the NDT.

---------- End of Page 124 in the original Special Provisions ----------

D. For steel pipe with an outside diameter greater than 1.1 m, and with a wall thickness greater than 25.4 mm, the weld groove root opening tolerance may be increased to a maximum of 5 mm over the specified tolerance.
E. Weld filler metal shall conform to the requirements in AWS D1.5, Table 4.1 or 4.2, for the welding of ASTM Designation: A 709, Grade 50 steel, and shall be designated H4 or H8 by the manufacturer.
F. Prequalified welding procedures will not be permitted for pile splices. All field welding procedures shall be qualified by testing in conformance with the requirements in AWS D1.1 and these special provisions. Using the qualified WPS, a minimum of two additional weld mock-ups shall be required to qualify offshore field welding, and both shall use the State-Furnished pipe pile sections to simulate the field girth weld. All mock-up welding shall be performed outside in the enclosure that will be used during offshore installation. Both welds shall be made in the near horizontal position with the joint tending towards the overhead position to the maximum angle anticipated by pile batter. Each weld need not exceed 1 m in length, and all passes shall be stopped and restarted at the same location in the middle of the weld. The first weld shall be prepared and welded using the proposed production weld joint detail and welding parameters. The second weld shall simulate the most onerous combination of weld root opening, root face and backing ring gap anticipated for field fit-up, as agreed with the Engineer. The out-of-tolerance fit-up shall be repaired and accepted per these specifications before completing the weld. The completed welds shall be examined by the ultrasonic testing (UT) procedure proposed for production joints, and any significant indications shall be marked for sectioning to confirm the UT results prior to mechanical testing the weldment. Qualification tests shall include all tests required by AWS D1.1, macroetch sections of the center stop-start location and all areas marked during UT, and Charpy V-Notch tests at -18°C of the weld metal and heat affected zone. The tests shall meet 27 Joules minimum average and 20 Joules minimum individual.
G. The welding filler materials (wire/electrode and flux, if used) shall be considered an essential variable for welding procedure qualification. Any change in the filler material brand name or type shall require requalification of the welding procedure.
H. GMAW shall not be used for field welding.
I. For field welding, including attaching backing rings and making repairs, the preheat and interpass temperature shall be in conformance with AWS D1.1, Table 3.2, Category C; and the minimum preheat and interpass temperature shall be 66°C, regardless of the pipe wall thickness or steel grade. In the event welding is interrupted, preheating to 66°C must occur before welding is resumed. For welds with required preheat temperatures greater than 66°C, preheat temperatures shall be achieved and maintained using electric resistance heating bands for the entire length of the weld. The heaters shall be controlled by attached thermocouples at spacings not exceeding 2 m. For these welds, the minimum preheat temperature shall be maintained continuously from beginning to completion of the entire weld, even if welding is interrupted.
J. Welds shall not be water quenched. Welds shall be allowed to cool unassisted.
K. Stray current corrosion of the structure shall be avoided during installation at the site. Welding machines shall be placed on the structure being welded. Where this is not practical, the insulated welded power source output "ground" lead shall be connected directly to the work at a location close to the weld being made and shall not be permitted to touch the water. The minimum total cross sectional area of the return ground cable(s) shall be 645 circular mm per 1000 amperes per 30.5 m of cable. Grounding sufficiency shall be periodically monitored by simultaneously measuring the potential of the structure being welded and that holding the welding machines using a standard calomel electrode (SCE), Ag-AgCl or other reference electrode approved by the Engineer. A change in potential reading of 10% or more shall indicate insufficient grounding.

NONDESTRUCTIVE TESTING FOR STEEL PIPE PILING

Steel pipe piling shall receive nondestructive testing (NDT) in conformance with these special provisions.

Nondestructive Testing of Welds made at a Fabrication Facility

Twenty-five percent of each longitudinal and 100 percent of each circumferential weld made shall receive NDT by either radiographic, radioscopic, real time imaging systems or ultrasonic methods that are in conformance with the requirements in AWS D1.1. The acceptance and repair criteria shall conform to the requirements in AWS D1.1, Section 6, for statically loaded structures under tensile stress, except within the "Plastic Hinge Zone" designated on the plans, where the criteria for cyclically loaded nontubular connections subject to tensile stress shall apply. If repairs are required in a portion of a weld not 100 percent examined by NDT, additional NDT of the same type shall be performed. The additional NDT shall be made on both sides of the repair for a length equal to 10% of the length of the pipe outside circumference. After the additional NDT is performed, and if more repairs are required, then the entire splice weld shall receive NDT.

Nondestructive Testing of Field Welds

Prior to performing ultrasonic NDT on field welds, the Contractor's welding inspection personnel shall have passed Caltrans' Ultrasonic Test. Field welding is defined as welding performed after the Certificate of Compliance has been

---------- End of Page 125 in the original Special Provisions ----------

furnished to the Engineer by the fabricator or manufacture for said materials. Information regarding the Caltrans Ultrasonic (titled "Notification of California Department of Transportation Qualification Requirement for Ultrasonic Testing Personnel) is included in the "Information Handout," available to the Contractor as provided for in Section 2-1.03, "Examination of Plans, Specifications, Contract and Site of Work," of the Standard Specifications. This test includes both written and practical examinations.

Splices made by field welding steel pipe piling shall receive NDT as follows:

UT shall be used for each field weld, including splices that are made onto a portion of the steel pipe piling that has been installed and any repair made to a splice weld. UT shall be performed over the full length of weld. In addition, Magnetic Particle testing (MT) shall be used for 100% of the root pass of all field welds unless otherwise directed by the Engineer. The acceptance criteria shall conform to the requirements in AWS D1.1, Section 6, for statically loaded nontubular connections subject to tensile stress. UT shall be performed in accordance with a written procedure that shall be reviewed by the Engineer before use. The UT procedure shall address the unambiguous interpretation of indications from the weld root and backing and shall describe the treatment of root fit-up repairs. The procedure shall define all measurements and/or marking that may be required prior to the start of welding. This procedure shall be demonstrated during weld mock-up qualification to verify its effectiveness in differentiating root and repair conditions.

MEASUREMENT AND PAYMENT (PILING)

The first paragraph of Section 49-6.01, "Measurement," of the Standard Specifications shall not apply.

The length of furnish pile to be paid for shall be the total length of the pile, as shown on the plans, measured along the centerline, from the specified pile tip of the pile to the plane of the pile cut-off. If the Contractor elects to furnish piling longer than the piling shown on the plans, no adjustment will be made to the length of piling to be paid and payment will be based on the length of pile shown on the plans.

Payment for cast-in-place concrete piling shall conform to the provisions in Section 49-6.02, "Payment," of the Standard Specifications except that, when the diameter of cast-in-place concrete piling is shown on the plans as 600 mm or larger, reinforcement in the piling will be paid for by the kilogram as bar reinforcing steel (bridge).

The sixth paragraph of Section 49-6.02, "Payment," of the Standard Specifications shall not apply.

If steel shells are fabricated more than 480 airline kilometers from both Sacramento and Los Angeles, additional shop inspection expenses will be sustained by the State. Whereas it is and will be impractical and extremely difficult to ascertain and determine the actual increase in such expenses, it is agreed that payment to the Contractor for furnishing steel shells will be reduced $17 per meter of length of steel shell.

The contract unit price paid for redrive existing 2.438 meter steel pipe pile shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in redriving the existing steel shell test piles, including removing and reattaching navigation lighting and cover plates, and cutting off and disposing of the piles after redriving, as shown on the plans, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer.

Full compensation for cleaning out the open ended steel shells prior to installing reinforcement and filling with concrete, for disposing of materials removed from the inside of the pile, and for placing seal course concrete and dewatering the open ended steel shells, as shown on the plans, as specified in these special provisions, and as directed by the Engineer, shall be considered as included in the contract unit price paid for drive pile, and no additional compensation will be allowed therefor.

Full compensation for soil plug removal to obtain the specified penetration, for disposing of this material, and for filling the void created by soil plug removal with seal course concrete shall be considered as included in the contract unit price paid for drive pile of the types shown in the Engineer's Estimate and no additional compensation will be allowed therefor.

Full compensation for conforming to the provisions in "Steel Pipe Piling" and "Nondestructive Testing" of these special provisions shall be considered as included in the contract prices paid for the various items of work involved, and no additional compensation will be allowed therefor.

Full compensation for providing access and working platforms for the Engineer, additional pile length necessary for monitoring, dewatering during monitoring, and for installing and removing the instruments from the pile shall be considered as included in the contract unit price paid for drive pile of the types shown in the Engineer's Estimate and no separate payment will be made therefor.

Full compensation for providing access and working platforms for the Engineer, dewatering during monitoring, and for installing and removing the instruments from the pile shall be considered as included in the contract unit price paid for redrive existing 2.438 meter steel pipe pile and no separate payment will be made therefor.

Full compensation for driving system submittals shall be considered as included in the contract unit price paid for drive pile of the types shown in the Engineer's Estimate and no additional compensation will be allowed therefor.

Full compensation for furnishing and installing welded studs shall be considered as included in the contract unit price paid for drive pile of the types shown in the Engineer's Estimate and no additional compensation will be allowed therefor.

---------- End of Page 126 in the original Special Provisions ----------

Full compensation for supply of electrical power for dynamic monitoring shall be considered as included in the contract unit price paid for drive pile of the types shown in the Engineer's Estimate and no additional compensation will be allowed therefor.

10-1.24A MARINE PILE DRIVING ENERGY ATTENUATOR

This work shall consist of designing, furnishing, installing, operating, monitoring, maintaining, and removing an air bubble curtain system to attenuate underwater energy generated by driving 2.5 meter cast-in-steel shell concrete piling. For purposes of this specification, pile installation refers to the all activities involved with driving a single pile; pile driving refers to the time when the hammer is physically driving the pile.

Attention is directed to "Relations with United States Coast Guard," of these special provisions regarding navigation requirements.

Attention is directed to "Order of Work," of these special provisions regarding redriving of existing test piles.

The approved attenuator system shall be operating prior to beginning pile driving at any given pile location, including redriving of existing test piles. If the attenuator fails, as determined by the Engineer, pile driving shall immediately stop. Piling driving at any given location shall not resume until the attenuator system at that location is again operating in conformance with the requirements of this section, as determined by the Engineer.

The Contractor shall provide adequate means to prevent light from pile driving operations from shining directly into the water. At least 15 minutes prior to and during pile driving operations, the Contractor shall not shine light directly into the water in areas adjacent to piles being driven.

At the Contractor's option, cofferdams that conform to the following requirements may be used as a marine pile driving energy attenuator:

A. Cofferdams shall be continuous (no openings in the sides).
B. Cofferdams shall be made of concrete or steel members.
C. Cofferdams shall extend from Mean Higher High Water to at least 0.5 meters below the original mudline.
D. Cofferdams shall be dewatered prior to pile driving.

GENERAL

An air bubble curtain system is generally composed of an air compressor(s), supply lines to deliver the air, distribution manifolds or headers, perforated aeration pipes, and a frame. The frame facilitates transport and placement of the system, keeps the aeration pipes stable, and provides ballast to counteract the buoyancy of the aeration pipes in operation.

Air bubble curtain system shall conform to the following:

A. Air bubble system shall consist of multiple and concentric layers of perforated aeration pipes stacked vertically in accordance with the following:

EXHIBIT 14

6.10 PILE WALL THICKNESS

6.10.1 General

The wall thickness of the pile may vary along its length and may be controlled at a particular point by any one of several loading conditions or requirements which are discussed in the paragraphs below.

6.10.2 Allowable Pile Stresses

The allowable pile stresses should be the same as those permitted by the AISC specification for a compact hot rolled section, giving due consideration to Sections 3.1 and 3.3. A rational analysis considering the restraints placed upon the pile by the structure and the soil should be used to determine the allowable stresses for the portion of the pile which is not laterally restrained by the soil. General column buckling of the portion of the pile below the mudline need not be considered unless the pile is believed to be laterally unsupported because of extremely low soil shear strengths, large computed lateral deflections, or for some other reason.

6.10.3 Design Pile Stresses

The pile wall thickness in the vicinity of the mudline, and possibly at other points, is normally controlled by the combined axial load and bending moment which results from the design loading conditions for the platform. The moment curve for the pile may be computed with soil reactions determined in accordance with Section 6.8 giving due consideration to possible soil removal by scour. It may be assumed that the axial load is removed from the pile by the soil at a rate equal to the ultimate soil-pile adhesion divided by the appropriate pile safety factor from 6.3.4. When lateral deflections associated with cyclic loads at or near the mudline are relatively large (e.g., exceeding y_c as defined in 6.8.3 for soft clay), consideration should be given to reducing or neglecting the soil-pile adhesion through this zone.

6.10.4 Stresses Due to Weight of Hammer During Hammer Placement

Each pile or conductor section on which a pile hammer (pile top drilling rig, etc.) will be placed should be checked for stresses due to placing the equipment. These loads may be the limiting factors in establishing maximum length of add-on sections. This is particularly true in cases where piling will be driven or drilled on a batter. The most frequent effects include: static bending, axial loads, and arresting lateral loads generated during initial hammer placement.

Experience indicates that reasonable protection from failure of the pile wall due to the above loads is provided if the static stresses are calculated as follows:

1. The pile projecting section should be considered as a freestanding column with a minimum effective length factor K of 2.1 and a minimum Reduction Factor C_m of 1.0.

2. Bending moments and axial loads should be calculated using the full weight of the pile hammer, cap, and leads acting through the center of gravity of their combined masses, and the weight of the pile add-on section with due consideration to pile batter eccentricities. The bending moment so determined should not be less than that corresponding to a load equal to 2 percent of the combined weight of the hammer, cap, and leads applied at the pile head and perpendicular to its centerline.

3. Allowable stresses in the pile should be calculated in accordance with Sections 3.2 and 3.3. The one third increase in stress should not be allowed.

6.10.5 Stresses During Driving

Consideration should also be given to the stresses that occur in the freestanding pile section during driving. The sum of the stresses due to the impact of the hammer (the dynamic stress) and the stresses due to axial load and bending (the static stresses) should not exceed the minimum yield stress of the steel. A method of analysis based on wave propagation theory should be used to determine the dynamic stresses, (see 6.2.1). In general it may be assumed that column buckling will not occur as a result of the dynamic portion of the driving stresses. The dynamic stresses should not exceed 80 to 90 percent of yield depending on specific circumstances such as the location of the maximum stresses down the length of pile, the number of blows, previous experience with the pile-hammer combination and the confidence level in the analyses. Separate considerations apply when significant driving stresses may be transmitted into the structure and damage to appurtenances must be avoided. The static stress during driving may be taken to be the stress resulting from the weight of the pile above the point of evaluation plus the pile hammer components actually supported by the pile during the hammer blows, including any bending stresses resulting therefrom. Allowable static stresses in the pile should be calculated in accordance with Sections 3.2 and 3.3. The one third increases in stress should not be allowed. The pile hammers evaluated for use during driving should be noted by the designer on the installation drawings or specifications. When using hydraulic hammers it is possible that the driving energy may exceed the rated energy and this should be considered in the analyses. Also the static stresses induced by hydraulic hammers need to be computed with special care due to the possible variations in driving configurations, for example when driving vertical piles without lateral restraint and exposed to environmental forces, see also 12.5.7(a).

6.10.6 Minimum Wall Thickness

The *D/t* ratio of the entire length of a pile should be small enough to preclude local buckling at stresses up to the yield strength of the pile material. Consideration should be given to the different loading situations occurring during the installation and the service life of a piling. For in-service conditions, and for those installation situations where normal pile-driving is anticipated or where piling installation will be by means other than driving, the limitations of Section 3.2 should be considered to be the minimum requirements. For piles that are to be installed by driving where sustained hard driving (250 blows per foot [820 blows per meter] with the largest size hammer to be used) is anticipated, the minimum piling wall thickness used should not be less than

$$\left.\begin{array}{l} t = 0.25 + \dfrac{D}{100} \\ \text{Metric Formula} \\ t = 6.35 + \dfrac{D}{100} \end{array}\right\} \qquad (6.10.6\text{-}1)$$

where

t = wall thickness, in. (mm),

D = diameter, in. (mm).

Minimum wall thickness for normally used pile sizes should be as listed in the following table:

Minimum Pile Wall Thickness			
Pile Diameter		Nominal Wall Thickness, t	
in.	mm	in.	mm
24	610	$^1/_2$	13
30	762	$^9/_{16}$	14
36	914	$^5/_8$	16
42	1067	$^{11}/_{16}$	17
48	1219	$^3/_4$	19
60	1524	$^7/_8$	22
72	1829	1	25
84	2134	$1^1/_8$	28
96	2438	$1^1/_4$	31
108	2743	$1^3/_8$	34
120	3048	$1^1/_2$	37

The preceding requirement for a lesser *D/t* ratio when hard driving is expected may be relaxed when it can be shown by past experience or by detailed analysis that the pile will not be damaged during its installation.

6.10.7 Allowance for Underdrive and Overdrive

With piles having thickened sections at the mudline, consideration should be given to providing an extra length of heavy wall material in the vicinity of the mudline so the pile will not be overstressed at this point if the design penetration is not reached. The amount of underdrive allowance provided in the design will depend on the degree of uncertainty regarding the penetration that can be obtained. In some instances an overdrive allowance should be provided in a similar manner in the event an expected bearing stratum is not encountered at the anticipated depth.

6.10.8 Driving Shoe

The purpose of driving shoes is to assist piles to penetrate through hard layers or to reduce driving resistances allowing greater penetrations to be achieved than would otherwise be the case. Different design considerations apply for each use. If an internal driving shoe is provided to drive through a hard layer it should be designed to ensure that unacceptably high driving stresses do not occur at and above the transition point between the normal and the thickened section at the pile tip. Also it should be checked that the shoe does not reduce the end bearing capacity of the soil plug below the value assumed in the design. External shoes are not normally used as they tend to reduce the skin friction along the length of pile above them.

6.10.9 Driving Head

Any driving head at the top of the pile should be designed in association with the installation contractor to ensure that it is fully compatible with the proposed installation procedures and equipment.

6.11 LENGTH OF PILE SECTIONS

In selecting pile section lengths consideration should be given to: 1) the capability of the lift equipment to raise, lower and stab the sections; 2) the capability of the lift equipment to place the pile driving hammer on the sections to be driven; 3) the possibility of a large amount of downward pile movement immediately following the penetration of a jacket leg closure; 4) stresses developed in the pile section while lifting; 5) the wall thickness and material properties at field welds; 6) avoiding interference with the planned concurrent driving of neighboring piles; and 7) the type of soil in which the pile tip is positioned during driving interruptions for field welding to attach additional sections. In addition, static and dynamic stresses due to the hammer weight and operation should be considered as discussed in 6.10.4 and 6.10.5.

Each pile section on which driving is required should contain a cutoff allowance to permit the removal of material damaged by the impact of the pile driving hammer. The normal allowance is 2 to 5 ft. (0.5 to 1.5 meters) per section. Where possible the cut for the removal of the cutoff allowance should be made at a conveniently accessible elevation.

12.5.3 Lifting Methods

When lifting eyes are used to facilitate the handling of the pile sections, the eyes should be designed, with due regard for impact, for the stresses developed during the initial pick-up of the section as well as those occurring during the stabbing of the section. When lifting eyes or weld-on lugs are used to support the initial pile sections from the top of the jacket, the entire hanging weight should be considered to be supported by a single eye or lug. The lifting eyes or support lugs should be removed by torch cutting $^{1}/_{4}$ inch (6.4 mm) from the pile surface and grinding smooth. Care should be exercised to ensure that any remaining protusion does not prevent driving of the pile or cause damage to elements such as packers. If burned holes are used in lieu of lifting eyes, they should comply with the applicable requirements of this section and consideration should be given to possible detrimental effect during hard driving.

As an alternative to providing lifting eyes on the piles, pile handling tools may be used, providing they are the proper size and capacity for the piles being driven and the operating conditions anticipated. These tools should be inspected prior to each use to ensure that they are in proper working condition. They should be used in strict accordance with the manufacturer's instructions and/or recommendations. For installations which require the use of pile followers, the followers should be inspected prior to the first use and periodically during the installation, depending on the severity of pile driving.

12.5.4 Field Welds

The add-on pile sections should be carefully aligned and the bevel inspected to assure a full penetration weld can be obtained before welding is initiated. It may be necessary to open up the bevel or grinding or gouging. Welding should be in accordance with Section 10 of this Recommend Practice. Nondestructive inspection of the field welds, utilizing one or more of the methods referenced in Section 13, should be performed.

12.5.5 Obtaining Required Pile Penetration

The adequacy of the platform foundation depends upon each pile being driven to or near its design penetration. The driving of each pile should be carried to completion with as little interruption as possible to minimize the increased driving resistance which often develops during delays. It is often necessary to work one pile at a time during the driving of the last one or two sections to minimize "setup" time. Workable back-up hammers with leads should always be available, especially when pile "setup" may be critical.

The fact that a pile has met refusal does not assure that it is capable of supporting the design load. Final blow count cannot be considered as assurance of the adequacy of piling. Continued driving beyond the defined refusal may be justified if it offers a reasonable chance of significantly improving the capability of the foundation. In some instances when continued driving is not successful the capacity of a pile can be improved utilizing methods such as those described in clause 6.2.1. Such methods should be approved by the design engineer prior to implementation.

12.5.6 Driven Pile Refusal

The definition of pile refusal is primarily for contractual purposes to define the point where pile driving with a particular hammer should be stopped and other methods instituted (such as drilling, jetting, or using a large hammer) and to prevent damage to the pile and hammer. The definition of refusal should also be adapted to the individual soil characteristics anticipated for the specific location. Refusal should be defined for all hammer sizes to be used and is contingent upon the hammer being operated at the pressure and rate recommended by the manufacturer.

The exact definition of refusal for a particular installation should be defined in the installation contract. An example (to be used only in the event that no other provisions are included in the installation contract) of such a definition is:

Pile driving refusal with a properly operating hammer is defined as the point where pile driving resistance exceeds either 300 blows per foot (0.3 m) for five consecutive feet (1.5 m) or 800 blows per foot (0.3 m) of penetration (This definition applies when the weight of the pile does not exceed four times the weight of the hammer ram If the pile weight exceeds this, the above blow counts are increased proportionally, but in no case shall they exceed 800 blows for six inches (152 mm) of penetration.)

If there has been a delay in pile driving operations for one hour or longer, the refusal criteria stated above shall not apply until the pile has been advanced at least one foot (0.3 m) following the resumption of pile driving. However, in no case shall the blow count exceed 800 blows for six inches (152 mm) of penetration.

In establishing the pile driving refusal criteria, the recommendations of the pile hammer manufacturer should be considered.

12.5.7 Pile Hammers

12.5.7.a Use of Hydraulic Hammers

Hydraulic hammers tend to be more efficient than steam hammers, so that the energy transferred to the pile for a given rated energy may be greater. They can be used both above and below water, to drive battered or vertical piles, through legs or through sleeves and guides, or vertical piles through sleeves alone. In calculating pile stresses, full account should be taken of wave, current and wind forces, both during driving and during hammer stabbing (which may be either above or below water). Further, while for steam hammers the weight of the cage is generally held by crane, for hydraulic hammers the whole weight of the hammer is borne by the pile.

The energy output is generally varied by the contractor to maintain a fairly low blowcount. Thus, blowcounts do not give a direct guide to soil stratification and resistance. Since the ram is encased, hammer performance cannot be judged visually. It is therefore important that measurements are made to give a complete record of performance including for example, ram impact velocity, stroke, pressure of accelerating medium and blowrate. Reliable instrumentation of some piles may be also desirable, to verify the energy transferred to the pile to aid interpretation of soil stratification and to limit pile stresses.

Monitoring of underwater driving requires that easily identified, unambiguous datums, together with robust television cameras or remotely operated vehicles, capable of maintaining station, be employed. Alternatively, for shallow water sites, it is possible to extend the hammer casing so that blowcounts can be monitored above water.

Because no cushion block is used, there is no change in ram to anvil pile characteristics as driving progresses and no requirement for cushion changes. However, because of the steel to steel contact, particular attention should be paid to the design of the pile head.

In selecting hydraulic hammers for deeper water applications, account should be taken of possible decreases in efficiency due to increased friction between the ram and its surrounding air. Sufficient air should be supplied to the hammer so that water ingress is prevented and water in the pile should be able to escape freely.

It should be noted that hammer changes take much longer than for steam hammers.

12.5.7.b Selection of Pile Hammer Size

When piles are to be installed by driving, the influence of the hammers to be used should be evaluated as a part of the design process as set forth in Section 6.10. It is not unusual for alternate hammers to be proposed for use by the erector well after the design has been completed and reevaluation by the designer may not be feasible. In such an event, justification for the use of an alternate hammer shall include calculation of stresses in the pile resulting therefrom as set out in Section 6.10.

In lieu of an analytical solution for dynamic stress the guidelines in Table 12.5.7 may be used:

Table 12.5.7 is based on industry experience with up to 60 in. diameter piles and 300 ft-kip hammers.

When it is necessary to use a pile hammer to drive piles with less than the guideline wall thickness set out in the above table, or that determined by an analytical solution, the definition of refusal used should be reduced proportionally.

Table 12.5.7—Guideline Wall Thickness

Guideline Wall Thickness, In.						
Pile Outside Diameter in.	Hammer Size, Ft-Kips					
	36	60	120	180	300	500
24	1/2	1/2	7/8	—	—	—
30	9/16	9/16	11/16	—	—	—
36	5/8	5/8	5/8	7/8	—	—
42	11/16	11/16	11/16	3/4	1 1/4	—
48	3/4	3/4	3/4	3/4	1 1/8	1 3/4
60	7/8	7/8	7/8	7/8	7/8	1 3/8
72	—	—	1	1	1	1 1/8
84	—	—	—	1 1/8	1 1/8	1 1/8
96	—	—	—	1 1/4	1 1/4	1 1/4
108	—	—	—	—	1 3/8	1 3/8
120	—	—	—	—	1 1/2	1 1/2

Guideline Wall Thickness, mm						
Pile Outside Diameter mm	Hammer Size, KJ					
	36	60	120	180	300	500
610	13	13	22	—	—	—
762	14	14	18	—	—	—
914	16	16	16	22	—	—
1067	18	18	18	19	32	—
1219	19	19	19	19	29	44
1524	22	22	22	22	22	35
1829	—	—	25	25	25	29
2134	—	—	—	29	29	29
2438	—	—	—	32	32	32
2743	—	—	—	—	35	35
3048	—	—	—	—	38	38

Values above the solid line based upon minimum pile area in square inches equals to 50% of the rated energy of the hammer in ft-kips. Values below line controlled by Section 6.9.6.

12.5.8 Drilled and Grouted Piles

Drilling the hole for drilled and grouted piles may be accomplished with or without drilling mud to facilitate maintaining an open hole. Drilling mud may be detrimental to the surface of some soils. If used, consideration should be given to flushing the mud with circulating water upon completion of drilling, provided the hole will remain open. Reverse circu-

6.10 PILE WALL THICKNESS

6.10.1 General

The wall thickness of the pile may vary along its length and may be controlled at a particular point by any one of several loading conditions or requirements which are discussed in the paragraphs below.

6.10.2 Allowable Pile Stresses

The allowable pile stresses should be the same as those permitted by the AISC specification for a compact hot rolled section, giving due consideration to Sections 3.1 and 3.3. A rational analysis considering the restraints placed upon the pile by the structure and the soil should be used to determine the allowable stresses for the portion of the pile which is not laterally restrained by the soil. General column buckling of the portion of the pile below the mudline need not be considered unless the pile is believed to be laterally unsupported because of extremely low soil shear strengths, large computed lateral deflections, or for some other reason.

6.10.3 Design Pile Stresses

The pile wall thickness in the vicinity of the mudline, and possibly at other points, is normally controlled by the combined axial load and bending moment which results from the design loading conditions for the platform. The moment curve for the pile may be computed with soil reactions determined in accordance with Section 6.8 giving due consideration to possible soil removal by scour. It may be assumed that the axial load is removed from the pile by the soil at a rate equal to the ultimate soil-pile adhesion divided by the appropriate pile safety factor from 6.3.4. When lateral deflections associated with cyclic loads at or near the mudline are relatively large (e.g., exceeding y_c as defined in 6.8.3 for soft clay), consideration should be given to reducing or neglecting the soil-pile adhesion through this zone.

6.10.4 Stresses Due to Weight of Hammer During Hammer Placement

Each pile or conductor section on which a pile hammer (pile top drilling rig, etc.) will be placed should be checked for stresses due to placing the equipment. These loads may be the limiting factors in establishing maximum length of add-on sections. This is particularly true in cases where piling will be driven or drilled on a batter. The most frequent effects include: static bending, axial loads, and arresting lateral loads generated during initial hammer placement.

Experience indicates that reasonable protection from failure of the pile wall due to the above loads is provided if the static stresses are calculated as follows:

1. The pile projecting section should be considered as a freestanding column with a minimum effective length factor K of 2.1 and a minimum Reduction Factor C_m of 1.0.

2. Bending moments and axial loads should be calculated using the full weight of the pile hammer, cap, and leads acting through the center of gravity of their combined masses, and the weight of the pile add-on section with due consideration to pile batter eccentricities. The bending moment so determined should not be less than that corresponding to a load equal to 2 percent of the combined weight of the hammer, cap, and leads applied at the pile head and perpendicular to its centerline.

3. Allowable stresses in the pile should be calculated in accordance with Sections 3.2 and 3.3. The one third increase in stress should not be allowed.

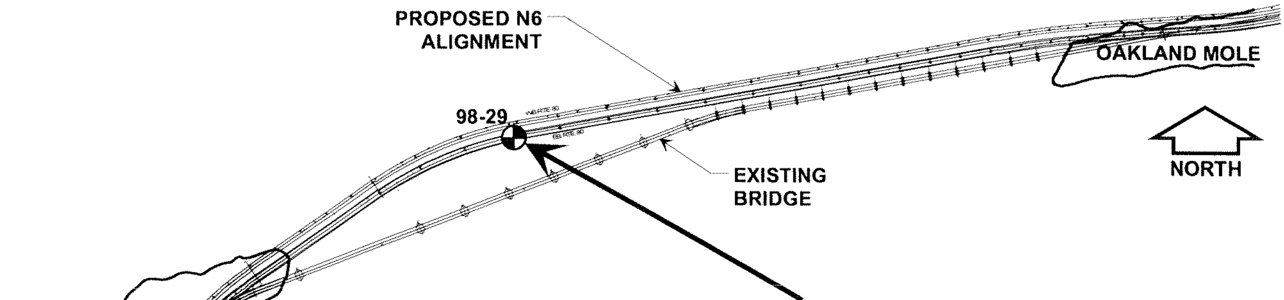
6.10.5 Stresses During Driving

Consideration should also be given to the stresses that occur in the freestanding pile section during driving. The sum of the stresses due to the impact of the hammer (the dynamic stress) and the stresses due to axial load and bending (the static stresses) should not exceed the minimum yield stress of the steel. A method of analysis based on wave propagation theory should be used to determine the dynamic stresses, (see 6.2.1). In general it may be assumed that column buckling will not occur as a result of the dynamic portion of the driving stresses. The dynamic stresses should not exceed 80 to 90 percent of yield depending on specific circumstances such as the location of the maximum stresses down the length of pile, the number of blows, previous experience with the pile-hammer combination and the confidence level in the analyses. Separate considerations apply when significant driving stresses may be transmitted into the structure and damage to appurtenances must be avoided. The static stress during driving may be taken to be the stress resulting from the weight of the pile above the point of evaluation plus the pile hammer components actually supported by the pile during the hammer blows, including any bending stresses resulting therefrom. Allowable static stresses in the pile should be calculated in accordance with Sections 3.2 and 3.3. The one third increases in stress should not be allowed. The pile hammers evaluated for use during driving should be noted by the designer on the installation drawings or specifications. When using hydraulic hammers it is possible that the driving energy may exceed the rated energy and this should be considered in the analyses. Also the static stresses induced by hydraulic hammers need to be computed with special care due to the possible variations in driving configurations, for example when driving vertical piles without lateral restraint and exposed to environmental forces, see also 12.5.7(a).

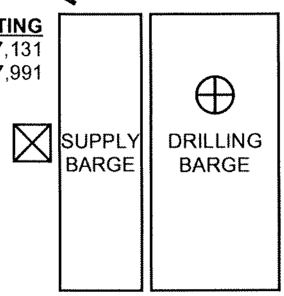
BORING 98-29

Date	Time From	Time To	Description of Activity
September 3, 1998	0600	0730	Move barge to location 98-29. Set 4 anchors and 2 spuds.
	0730	0900	Rig up for drilling. Lower drill pipe to mudline.
	0900	0930	Measure water depth of 10.0m (32.8 ft) using bottom sensor. Current tide level is approximately +0.2m (+0.8 ft) MSL. Calculate mudline elevation of -9.8m (-32 ft) MSL.
	0930	1230	Drill and sample from mudline to 9.6m (31.4 ft).
	****	****	
	1000	1215	Rig up Halibut vane equipment. Halibut Vane shear test at 0.6m (2 ft).
	****	****	
	1230	1315	Pull drill pipe to deck.
	1315	1330	Reposition barge.
	1330	2000	Service piston. Lower stinger to 2.1m (7 ft).
	2000	2035	Perform CPT pipe test.
	2035	2115	Drill to 3.1m (10 ft).
	2115	2230	CPT testing from 3.1m (10 ft) to 9.8m (32 ft). No data. Pull pipe to deck.
	2230	2400	Reconfigure casing.
September 4, 1998	0000	0445	Set casing.
	0445	0815	Drive stinger to 7.9m (26 ft).
	0815	1100	Mix drilling mud and shorten connection to mud recirculation system.
	1100	1345	Lower drill pipe to mudline and drill to 7.0m (23 ft).
	1345	2200	Drill sample and CPT testing from 7.0m (23 ft) to 36.9m (121 ft).
	2200	2330	Change CPT metering cylinder and cone.
	2330	2400	Drill and sample from 36.9m (121 ft) to 37.5m (123 ft).
September 5, 1998	0000	0200	Perform CPT test at 37.5m (123 ft). No data. Perform CPT pipe test.
	0200	1200	Drill, sample, and CPT testing from 37.5m (123 ft) to 70.1m (230 ft).
	1200	1615	Drill, sample, and CPT testing from 70.1m (230 ft) to 80.5m (264 ft).
	1615	2010	No pore pressure data. Change cone. Perform pipe test.
	2010	2400	Drill sample and CPT testing from 80.5m (264 ft) to 91.4m (300 ft).
September 6, 1998	0000	0230	Drill to 94.2m (309 ft). Perform maintenance on mud pump.
	0230	0330	Drill and sample from 94.2m (309 ft) to 94.9m (311.5 ft).
	0330	0530	Pull drill pipe to deck.
	0530	0750	P- and S-wave velocity logging from 88.0m (288.7 ft) to 7.0m (23 ft).
	0750	1130	Lower N-rod. Mix and circulate cement. Grout hole 98-29. Pull N-rod to deck.
	1130	1510	Pull casing to deck.
	1510	1730	Pull 2 spuds, 4 anchors, and move barge to location 98-30.

SUMMARY OF FIELD OPERATIONS
Boring 98-29
SFOBB East Span Seismic Safety Project



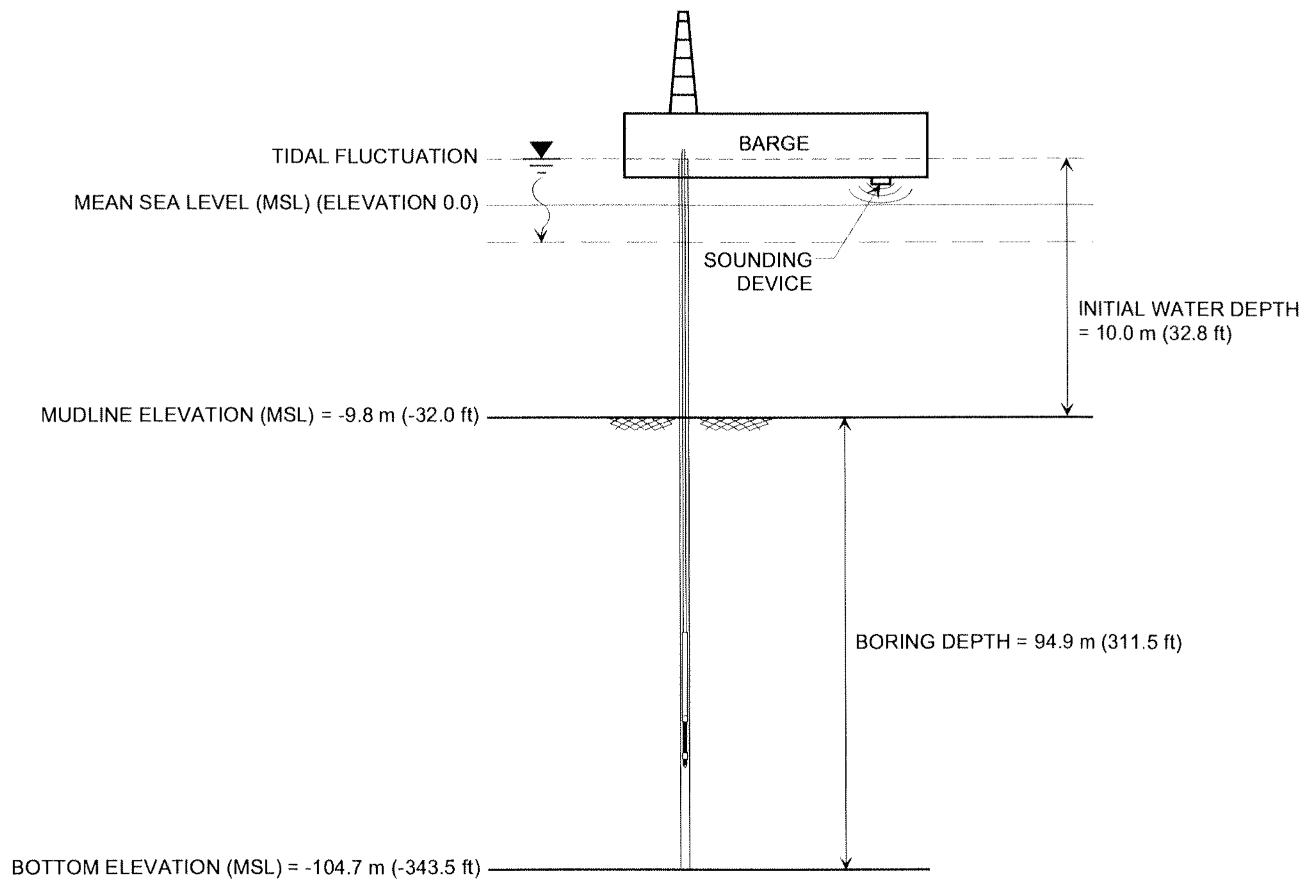
HALIBUT TESTING
EASTING 1,837,131
NORTHING 647,991



BORING 98-29

Soil and Rock Sampling Depth	In-Situ Testing Depth	Coordinates
0 m - 9.6 m (0 ft - 31.4 ft)		E = 1,837,133 N = 647,992
9.6 m - 94.9 m (31.4 ft - 311.5 ft)	3 m - 89.6 m (10 ft - 294 ft)	E = 1,837,145 N = 647,992

Coordinates in NAD83, CA Zone 3, meters.



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DEPTH AND LOCATION REFERENCE MAP
Boring 98-29
SFOBB East Span Seismic Safety Project

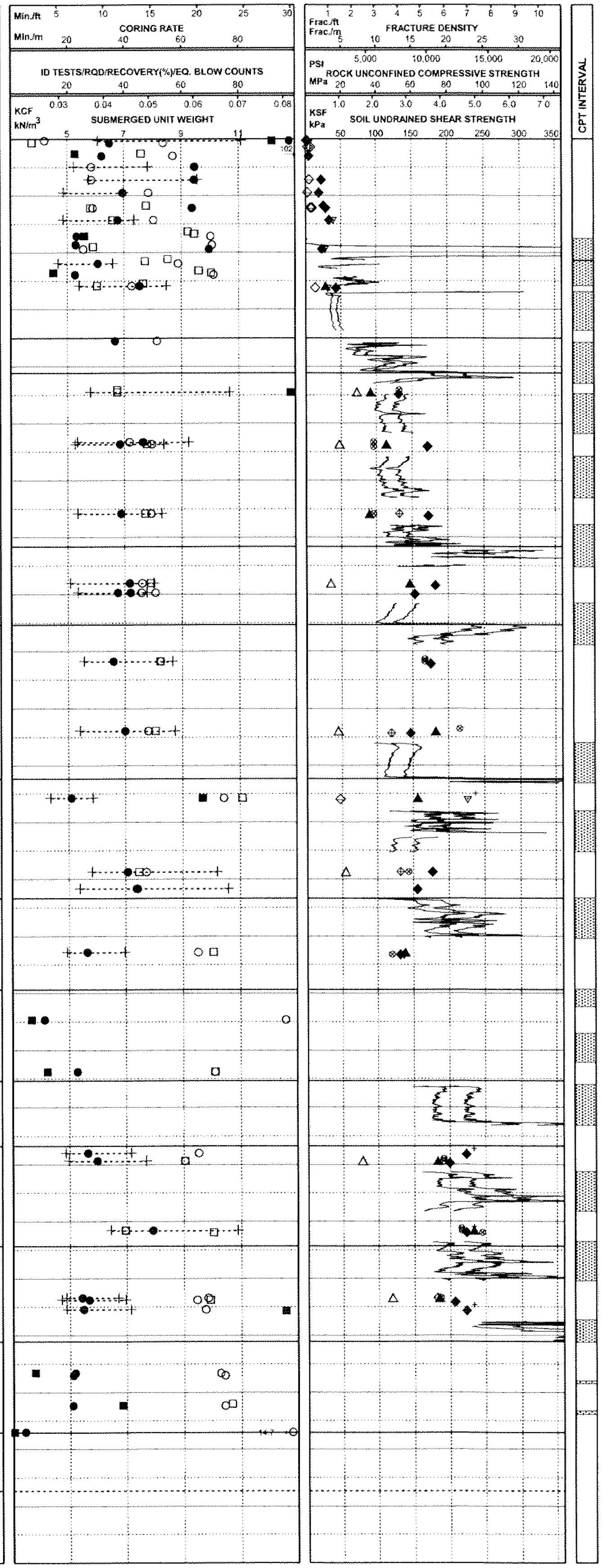
PROJECT NO: 98-42-0054	START DATE: 09/03/98	DRILLER:	Fugro-McClelland Marine Geosciences
BORING: 98-29 (Skyway - Frame 1)	COMPLETION DATE: 09/06/98	DRILLING METHOD:	Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -9.8m (MSL)

ELEVATION, m.	DEPTH, m.	BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
		WOH, WOH, WOH, WOH, WOH, WOH, WOH, 17/45cm, 18/60cm, 30/55cm, 30/45cm, PUSH	Fat CLAY (CH), soft to firm, dark gray -with shells and shell fragments -very soft, dark gray clay, to 0.9m -with many sand seams and pockets to 0.6m See Plate 98-29.6 for detailed stratigraphy -intermixed clay and sand, 6.4m to 7.0m -sand layer, 7.6m to 8.2m -sand layer, 8.5m to 8.8m -silty sand layer with clay pockets, 9.1m to 9.7m -sand layer at 10.1m (14.0m)	I
		SAVE	Fat CLAY (CH), stiff to very stiff, olive gray to gray -intermixed sand and clay with organic pockets, to 14.6m (16.5m)	II
		PUSH, PUSH, PUSH	Fat CLAY (CH), very stiff, gray -with a silt layer at 16.8m -greenish gray, and with a few silt pockets, slickensides and gas blisters, at 17.7m -with a few shell fragments and silt pockets, at 21.3m -greenish gray with silty sand pockets, a few shell fragments, and mica flakes, at 26.2m (28.7m)	III
		PUSH, SAVE	Fat CLAY (CH), very stiff, gray -silt with clay layers to 29.9m -with sandy silt pockets and partings, and shell fragments, 30.8m to 32.0m (34.1m)	IV
		PUSH, PUSH	Fat CLAY (CH), very stiff to hard, gray -greenish gray with silt pockets and partings, at 36.6m -with a few silt pockets at 41.1m -with organic stains and organic pockets, below 41.1m (45.0m)	V
		30/40cm, PUSH, SAVE	Fat CLAY (CH), very stiff, gray -very stiff to hard sandy lean clay with many silt seams, to 48.8m -with a sand layer at 45.1m with silty sand pockets and seams, at 46.3m -sand layer, 48.8m to 49.1m (53.3m)	VI
		30/40cm	Lean CLAY (CL), very stiff to hard, greenish gray -with a few silt pockets and partings, gas blisters, and organic stains, at 57.2m (59.7m)	VII
		30/15cm, 30/30cm	Fine to Coarse SAND with silt (SW-SM), dense, gray -with gravel at 61.7m -silty fine sand below 64.2m -with clay pockets, partings and seams, at 65.2m (66.1m)	VIII
			Lean CLAY (CL), very stiff to hard, olive gray -with a sand layer at 69.2m (70.7m)	IX
		NR, SAVE, PUSH, PUSH	Lean CLAY (CL), hard, olive gray -with silt partings and pockets, to 71.9m -with silty sand pockets and partings, at 71.6m -with wood fragments and organic pockets, at 76.5m (77.7m)	X
		PUSH, SAVE	Lean CLAY (CL), hard, gray -with silt pockets and a few shell fragments, at 81.4m -interlayered sand and clay, below 82.9m (84.4m)	XI
		30/30cm, 30/25cm	Fine to Medium SAND with silt (SP-SM), dense to very dense, gray -with a clay layer at 86.7m -with a lean clay layer at 88.8m (90.8m)	XII
		30/5cm, 50/5cm	Fine GRAVEL (GP), very dense, dark gray -sand layer, 91.4m to 92.4m dense to very dense, fine to coarse sand, at 94.8m	XIII

TOTAL DEPTH: 94.9m BACKFILLED WITH: Cement Grout



LOG OF BORING AND TEST RESULTS

BORING 98-29

SFOBB East Span Seismic Safety Project

Report Date: 04/30/99

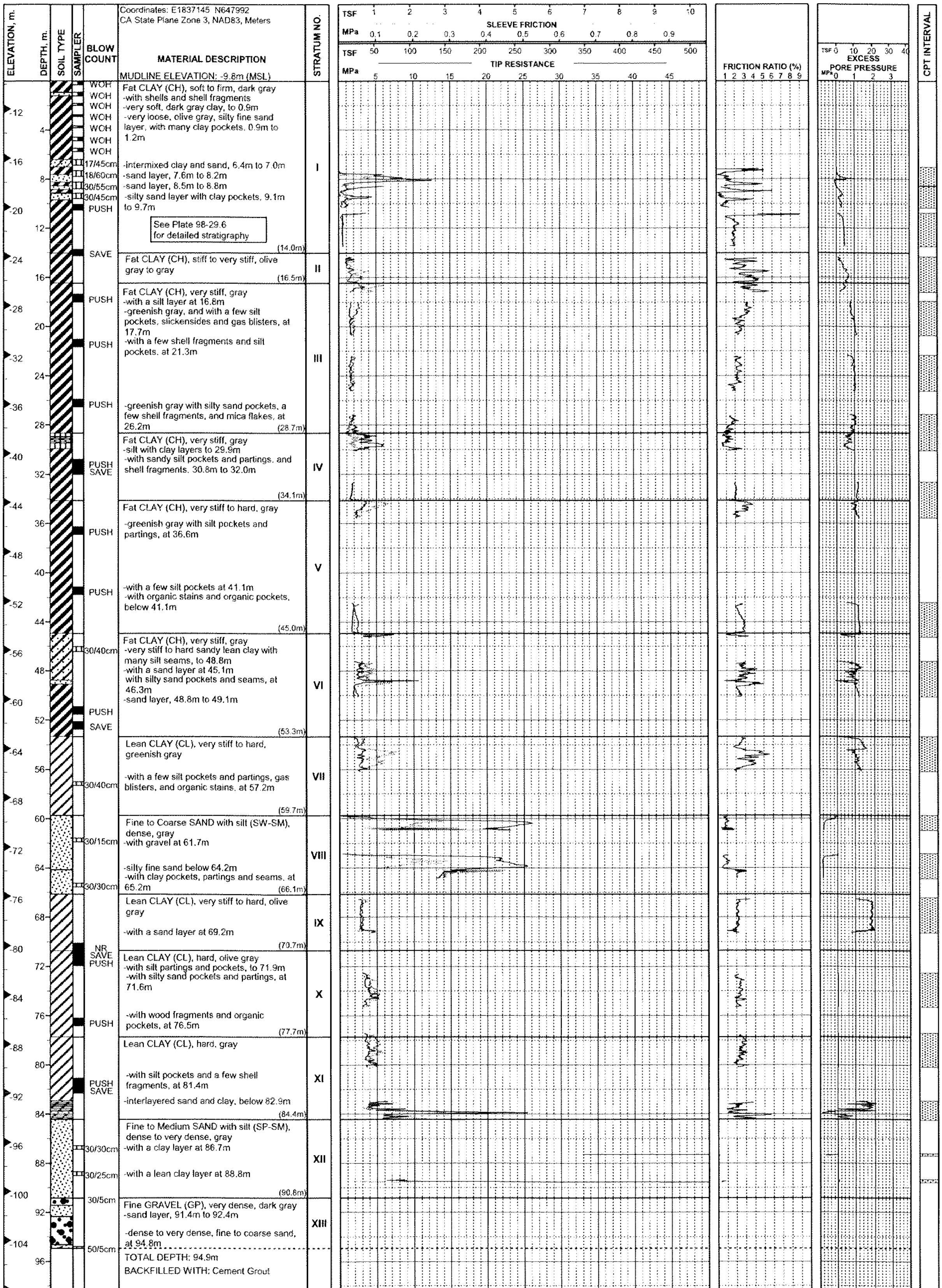
FUGRO

PROJECT NO:	98-42-0054	START DATE:	09/03/98	DRILLER:	Fugro-McClelland Marine Geosciences
BORING:	98-29 (Skyway - Frame 1)	COMPLETION DATE:	09/06/98	DRILLING METHOD:	Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -9.8m (MSL)

BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
WOH, WOH, WOH, WOH, WOH, WOH, WOH, 17/45cm, 18/60cm, 30/55cm, 30/45cm, PUSH, SAVE	Fat CLAY (CH), soft to firm, dark gray -with shells and shell fragments -very soft, dark gray clay, to 0.9m -very loose, olive gray, silty fine sand layer, with many clay pockets, 0.9m to 1.2m -intermixed clay and sand, 6.4m to 7.0m -sand layer, 7.6m to 8.2m -sand layer, 8.5m to 8.8m -silty sand layer with clay pockets, 9.1m to 9.7m See Plate 98-29.6 for detailed stratigraphy (14.0m)	I
	Fat CLAY (CH), stiff to very stiff, olive gray to gray (16.5m)	II
PUSH, PUSH, PUSH	Fat CLAY (CH), very stiff, gray -with a silt layer at 16.8m -greenish gray, and with a few silt pockets, slickensides and gas blisters, at 17.7m -with a few shell fragments and silt pockets, at 21.3m -greenish gray with silty sand pockets, a few shell fragments, and mica flakes, at 26.2m (28.7m)	III
PUSH SAVE	Fat CLAY (CH), very stiff, gray -silt with clay layers to 29.9m -with sandy silt pockets and partings, and shell fragments, 30.8m to 32.0m (34.1m)	IV
PUSH, PUSH	Fat CLAY (CH), very stiff to hard, gray -greenish gray with silt pockets and partings, at 36.6m -with a few silt pockets at 41.1m -with organic stains and organic pockets, below 41.1m (45.0m)	V
30/40cm, PUSH, SAVE	Fat CLAY (CH), very stiff, gray -very stiff to hard sandy lean clay with many silt seams, to 48.8m -with a sand layer at 45.1m with silty sand pockets and seams, at 46.3m -sand layer, 48.8m to 49.1m (53.3m)	VI
30/40cm	Lean CLAY (CL), very stiff to hard, greenish gray -with a few silt pockets and partings, gas blisters, and organic stains, at 57.2m (59.7m)	VII
30/15cm, 30/30cm	Fine to Coarse SAND with silt (SW-SM), dense, gray -with gravel at 61.7m -silty fine sand below 64.2m -with clay pockets, partings and seams, at 65.2m (66.1m)	VIII
	Lean CLAY (CL), very stiff to hard, olive gray -with a sand layer at 69.2m (70.7m)	IX
NR SAVE PUSH, PUSH	Lean CLAY (CL), hard, olive gray -with silt partings and pockets, to 71.9m -with silty sand pockets and partings, at 71.6m -with wood fragments and organic pockets, at 76.5m (77.7m)	X
PUSH SAVE	Lean CLAY (CL), hard, gray -with silt pockets and a few shell fragments, at 81.4m -interlayered sand and clay, below 82.9m (84.4m)	XI
30/30cm, 30/25cm	Fine to Medium SAND with silt (SP-SM), dense to very dense, gray -with a clay layer at 86.7m -with a lean clay layer at 88.8m (90.8m)	XII
30/5cm, 50/5cm	Fine GRAVEL (GP), very dense, dark gray -sand layer, 91.4m to 92.4m -dense to very dense, fine to coarse sand, at 94.8m	XIII
	TOTAL DEPTH: 94.9m BACKFILLED WITH: Cement Grout	



LOG OF BORING AND TEST RESULTS

BORING 98-29

SFOBB East Span Seismic Safety Project

Report Date: 05/03/99

PROJECT NO: 98-42-0054	**START DATE:** 09/03/98	**DRILLER:** Fugro-McClelland Marine Geosciences	
BORING: 98-29 (Skyway - Frame 1)	**COMPLETION DATE:** 09/06/98	**DRILLING METHOD:** Rotary Sample Boring (Wet)	

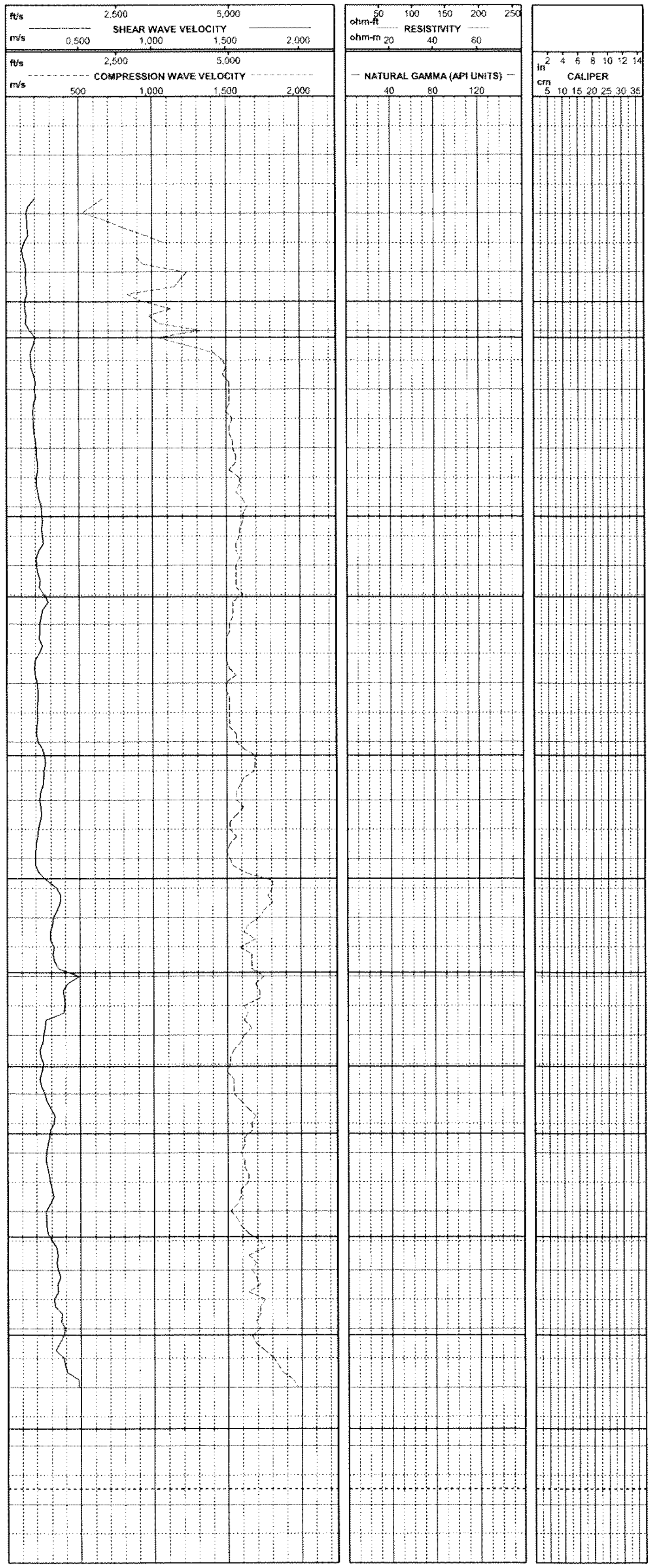
Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -9.8m (MSL)

BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
WOH, WOH, WOH, WOH, WOH, WOH, WOH, 17/45cm, 18/60cm, 30/55cm, 30/45cm, PUSH, SAVE	Fat CLAY (CH), soft to firm, dark gray -very loose, olive gray, silty fine sand layer, with many clay pockets, 0.9m to 1.2m See Plate 98-29.6 for detailed stratigraphy -intermixed clay and sand, 6.4m to 7.0m -sand layer, 7.6m to 8.2m -sand layer, 8.5m to 8.8m -silty sand layer with clay pockets, 9.1m to 9.7m (14.0m)	I
	Fat CLAY (CH), stiff to very stiff, olive gray to gray (16.5m)	II
PUSH, PUSH, PUSH	Fat CLAY (CH), very stiff, gray -with a silt layer at 16.8m -greenish gray, and with a few silt pockets, slickensides and gas blisters, at 17.7m -with a few shell fragments and silt pockets, at 21.3m -greenish gray with silty sand pockets, a few shell fragments, and mica flakes, at 26.2m (28.7m)	III
PUSH SAVE	Fat CLAY (CH), very stiff, gray -silt with clay layers to 29.9m -with sandy silt pockets and partings, and shell fragments, 30.8m to 32.0m (34.1m)	IV
PUSH, PUSH	Fat CLAY (CH), very stiff to hard, gray -greenish gray with silt pockets and partings, at 36.6m -with a few silt pockets at 41.1m -with organic stains and organic pockets, below 41.1m (45.0m)	V
30/40cm, PUSH, SAVE	Fat CLAY (CH), very stiff, gray -very stiff to hard sandy lean clay with many silt seams, to 48.8m -with a sand layer at 45.1m with silty sand pockets and seams, at 46.3m -sand layer, 48.8m to 49.1m (53.3m)	VI
30/40cm	Lean CLAY (CL), very stiff to hard, greenish gray -with a few silt pockets and partings, gas blisters, and organic stains, at 57.2m (59.7m)	VII
30/15cm, 30/30cm	Fine to Coarse SAND with silt (SW-SM), dense, gray -with gravel at 61.7m -silty fine sand below 64.2m -with clay pockets, partings and seams, at 65.2m (66.1m)	VIII
	Lean CLAY (CL), very stiff to hard, olive gray -with a sand layer at 69.2m (70.7m)	IX
NR SAVE PUSH, PUSH	Lean CLAY (CL), hard, olive gray -with silt partings and pockets, to 71.9m -with silty sand pockets and partings, at 71.6m -with wood fragments and organic pockets, at 76.5m (77.7m)	X
PUSH SAVE	Lean CLAY (CL), hard, gray -with silt pockets and a few shell fragments, at 81.4m -interlayered sand and clay, below 82.9m (84.4m)	XI
30/30cm, 30/25cm	Fine to Medium SAND with silt (SP-SM), dense to very dense, gray -with a clay layer at 86.7m -with a lean clay layer at 88.8m (90.8m)	XII
30/5cm, 50/5cm	Fine GRAVEL (GP), very dense, dark gray -sand layer, 91.4m to 92.4m -dense to very dense, fine to coarse sand, at 94.8m	XIII

TOTAL DEPTH: 94.9m

BACKFILLED WITH: Cement Grout



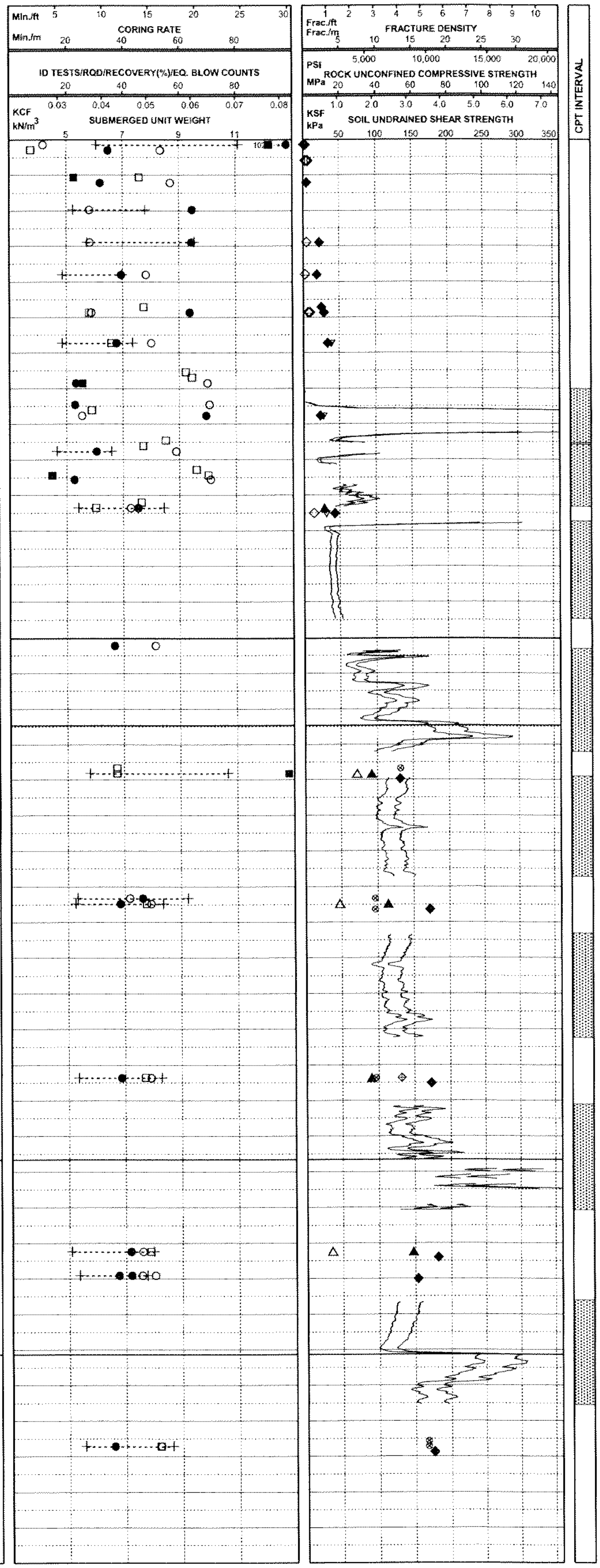
LOG OF BORING AND TEST RESULTS
BORING 98-29
SFOBB East Span Seismic Safety Project

Report Date: 05/05/99

PROJECT NO: 98-42-0054	START DATE: 09/03/98	DRILLER: Fugro-McClelland Marine Geosciences
BORING: 98-29 (Skyway - Frame 1)	COMPLETION DATE: 09/06/98	DRILLING METHOD: Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

ELEVATION, m	DEPTH, m	BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
			MUDLINE ELEVATION: -9.8m (MSL)	
-10		WOH	Fat CLAY (CH), soft to firm, dark gray -with shells and shell fragments -very soft, dark gray clay, to 0.9m	I
-11	1	WOH	-with many sand seams and pockets to 0.6m -very loose, olive gray, silty fine sand layer, with many clay pockets, 0.9m to 1.2m	
-12	2	WOH		
-13	3	WOH		
-14	4	WOH	-with many shell fragments below 3.7m -with sand pockets, 3.7m to 4.7m	
-15	5	WOH		
-16	6	WOH	-with sandy silt pockets at 5.5m	
-17	7	17/45cm	-intermixed clay and sand, 6.4m to 7.0m	
-18	8	18/60cm	-with sand seams at 7.3m -with gas blisters below 7.5m -sand layer, 7.6m to 8.2m	
-19	9	30/55cm	-sand layer, 8.5m to 8.8m	
		30/45cm	-silty sand layer with clay pockets, 9.1m to 9.7m	
-20	10	PUSH	-sand layer at 10.1m -with very few silt pockets and gas blisters, below 10.2m	
-21 to -23	11–13		(14.9m)	
-24	14	SAVE	Fat CLAY (CH), stiff to very stiff, olive gray to gray -intermixed sand and clay with organic pockets, to 14.6m	II
-25	15		-with silt seams, at 15.4m and 15.7m	
-26	16		(16.5m)	
-27	17		Fat CLAY (CH), very stiff, gray -with a silt layer at 16.8m	III
-28	18	PUSH	-greenish gray, and with a few silt pockets, slickensides and gas blisters, at 17.7m	
-29 to -30	19–20			
-31	21	PUSH	-with a few shell fragments and silt pockets, at 21.3m	
-32 to -35	22–25			
-36	26	PUSH	-greenish gray with silty sand pockets, a few shell fragments, and mica flakes, at 26.2m	
-37 to -38	27–28		(28.7m)	
-39	29		Fat CLAY (CH), very stiff, gray -silt with clay layers to 29.9m	IV
-40	30			
-41	31	PUSH	-with sandy silt pockets and partings, and shell fragments, 30.8m to 32.0m	
-42	32	SAVE		
-43 to -44	33–34		(34.1m)	
-44	34		Fat CLAY (CH), very stiff to hard, gray	V
-45 to -46	35–36			
-46	36	PUSH	-greenish gray with silt pockets and partings, at 36.6m	
-47 to -49	37–39			



LOG OF BORING AND TEST RESULTS

BORING 98-29

SFOBB East Span Seismic Safety Project

Report Date: 04/27/99

FUGRO

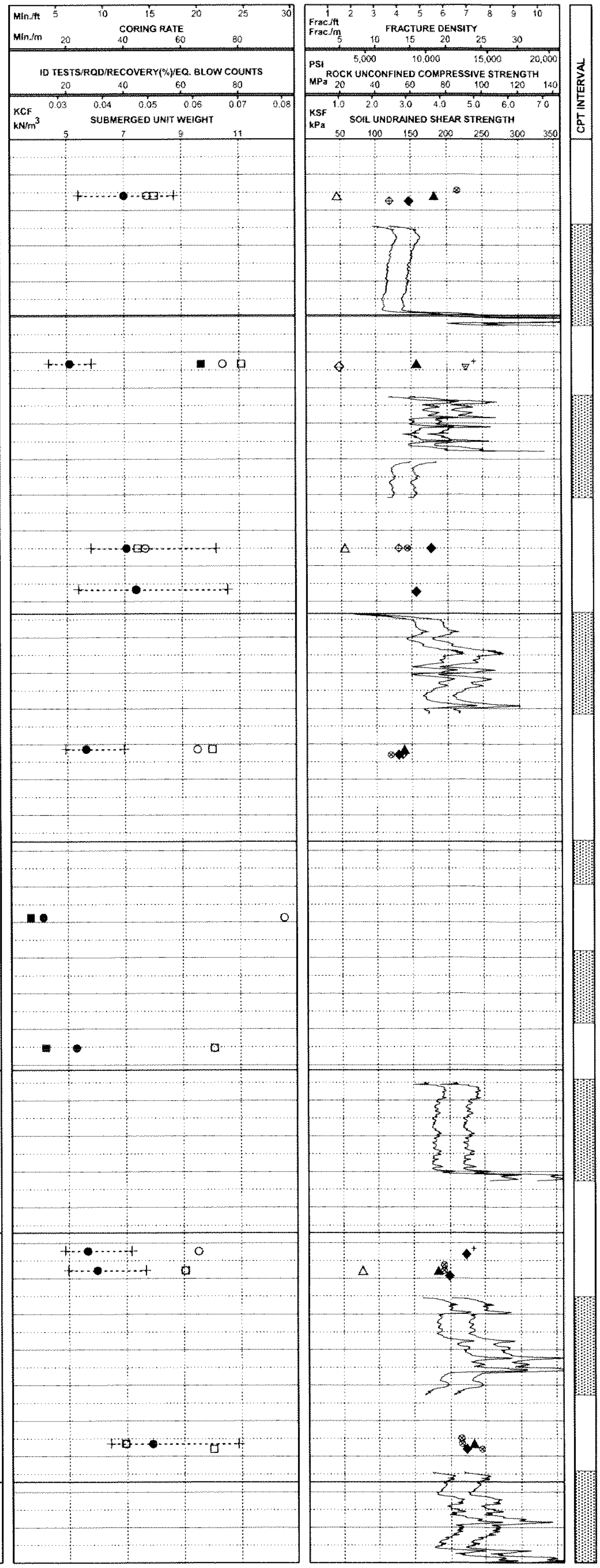
PLATE 98-29.6a

PROJECT NO: 98-42-0054	START DATE: 09/03/98	DRILLER:	Fugro-McClelland Marine Geosciences
BORING: 98-29 (Skyway - Frame 1)	COMPLETION DATE: 09/06/98	DRILLING METHOD:	Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -9.8m (MSL)

DEPTH, m	SAMPLER / BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
41–45.0	PUSH	Fat CLAY (CH), very stiff to hard, gray -with a few silt pockets at 41.1m -with organic stains and organic pockets, below 41.1m (45.0m)	V
45.0–53.3	30/40cm; PUSH; SAVE	Fat CLAY (CH), very stiff, gray -very stiff to hard sandy lean clay with many silt seams, to 48.8m -with a sand layer at 45.1m with silty sand pockets and seams, at 46.3m -sand layer, 48.8m to 49.1m (53.3m)	VI
53.3–59.7	30/40cm	Lean CLAY (CL), very stiff to hard, greenish gray -with a few silt pockets and partings, gas blisters, and organic stains, at 57.2m (59.7m)	VII
59.7–66.1	30/15cm; 30/30cm	Fine to Coarse SAND with silt (SW-SM), dense, gray -with gravel at 61.7m -silty fine sand below 64.2m -with clay pockets, partings and seams, at 65.2m (66.1m)	VIII
66.1–70.7	NR	Lean CLAY (CL), very stiff to hard, olive gray -with a sand layer at 69.2m (70.7m)	IX
70.7–77.7	SAVE; PUSH; PUSH	Lean CLAY (CL), hard, olive gray -with silt partings and pockets, to 71.9m -with silty sand pockets and partings, at 71.6m -with wood fragments and organic pockets, at 76.5m (77.7m)	X
77.7–		Lean CLAY (CL), hard, gray	XI



LOG OF BORING AND TEST RESULTS

BORING 98-29

SFOBB East Span Seismic Safety Project

Report Date: 04/27/99

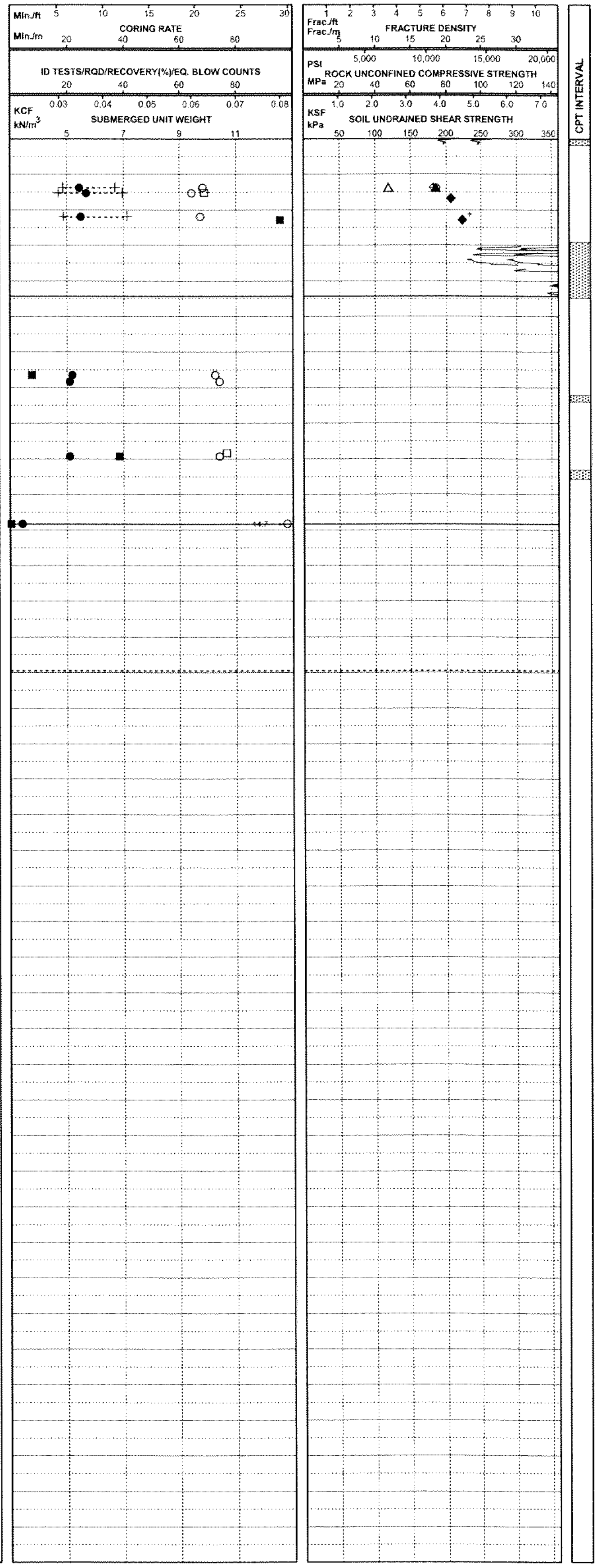
FUGRO

PROJECT NO: 98-42-0054	START DATE: 09/03/98	DRILLER: Fugro-McClelland Marine Geosciences
BORING: 98-29 (Skyway - Frame 1)	COMPLETION DATE: 09/06/98	DRILLING METHOD: Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

ELEVATION, m.	DEPTH, m.	BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
			MUDLINE ELEVATION: -9.8m (MSL)	
-90			Lean CLAY (CL), hard, gray	XI
-91	81	PUSH	-with silt pockets and a few shell fragments, at 81.4m	
-92	82	SAVE		
-93	83		-interlayered sand and clay, below 82.9m	
-94	84		(84.4m)	
-95	85		Fine to Medium SAND with silt (SP-SM), dense to very dense, gray	XII
-96	86			
-97	87	30/30cm	-with a clay layer at 86.7m	
-98	88			
-99	89	30/25cm	-with a lean clay layer at 88.8m	
-100	90		(90.8m)	
-101	91	30/5cm	Fine GRAVEL (GP), very dense, dark gray -sand layer, 91.4m to 92.4m	XIII
-102	92			
-103	93			
-104	94		dense to very dense, fine to coarse sand, at 94.8m	
-105	95	50/5cm	TOTAL DEPTH: 94.9m BACKFILLED WITH: Cement Grout	

Elevation scale continues -106 to -129; depth scale 96 to 119.



LOG OF BORING AND TEST RESULTS
BORING 98-29
SFOBB East Span Seismic Safety Project

Report Date: 04/27/99

FUGRO

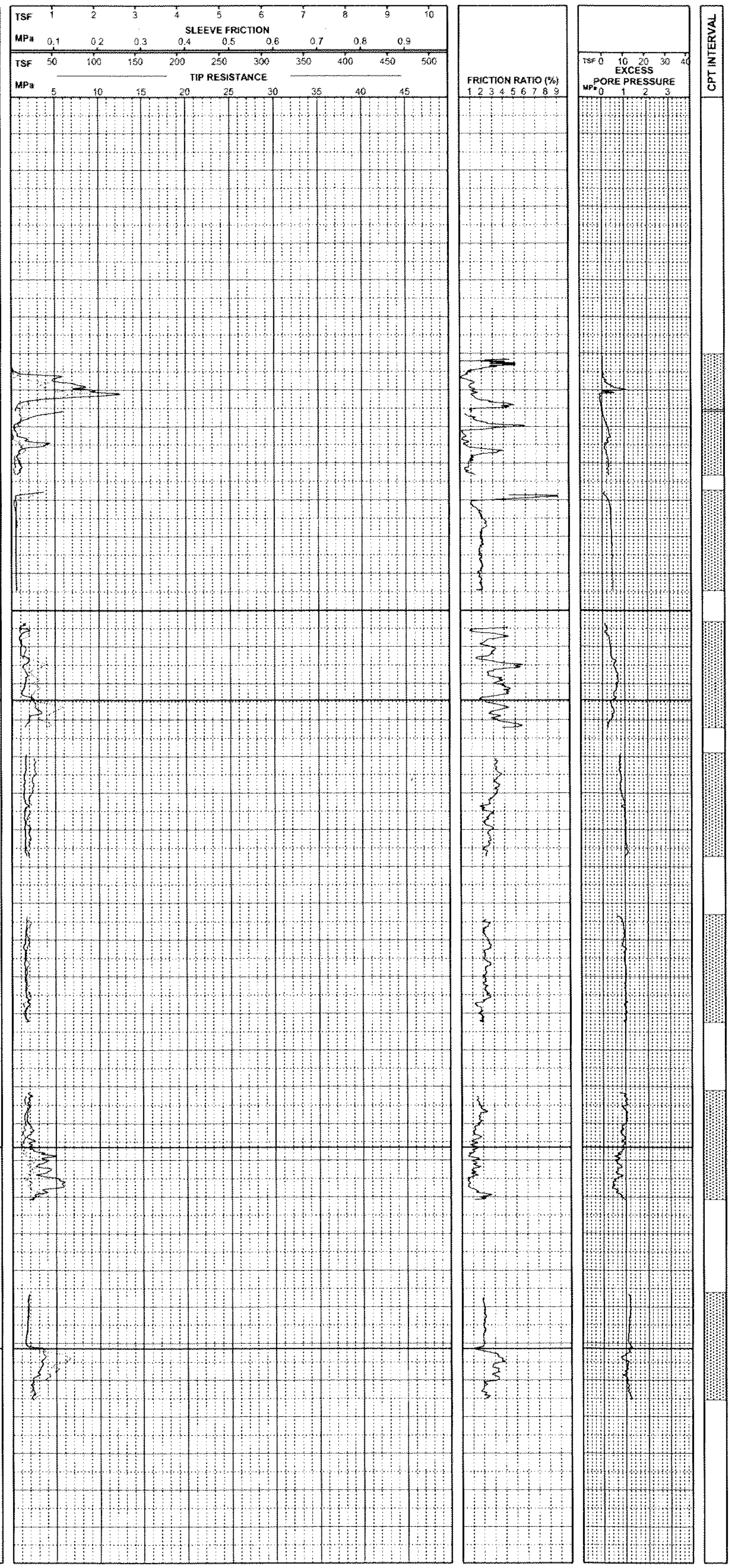
PLATE 98-29.6c

PROJECT NO:	98-42-0054	START DATE:	09/03/98
BORING:	98-29 (Skyway - Frame 1)	COMPLETION DATE:	09/06/98
DRILLER:	Fugro-McClelland Marine Geosciences	DRILLING METHOD:	Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -9.8m (MSL)

ELEVATION, m.	DEPTH, m.	BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
-10 to -23.8	0 to 14.0	WOH, WOH, WOH, WOH, WOH, WOH, WOH, 17/45cm, 18/60cm, 30/55cm, 30/45cm, PUSH, SAVE	Fat CLAY (CH), soft to firm, dark gray -with shells and shell fragments -very soft, dark gray clay, to 0.9m -with many sand seams and pockets to 0.6m -very loose, olive gray, silty fine sand layer, with many clay pockets, 0.9m to 1.2m -with many shell fragments below 3.7m -with sand pockets, 3.7m to 4.7m -with sandy silt pockets at 5.5m -intermixed clay and sand, 6.4m to 7.0m -with sand seams at 7.3m -with gas blisters below 7.5m -sand layer, 7.6m to 8.2m -sand layer, 8.5m to 8.8m -silty sand layer with clay pockets, 9.1m to 9.7m -sand layer at 10.1m -with very few silt pockets and gas blisters, below 10.2m (14.0m)	I
	14.0 to 16.5		Fat CLAY (CH), stiff to very stiff, olive gray to gray -intermixed sand and clay with organic pockets, to 14.6m -with silt seams, at 15.4m and 15.7m (16.5m)	II
	16.5 to 28.7	PUSH, PUSH, PUSH	Fat CLAY (CH), very stiff, gray -with a silt layer at 16.8m -greenish gray, and with a few silt pockets, slickensides and gas blisters, at 17.7m -with a few shell fragments and silt pockets, at 21.3m -greenish gray with silty sand pockets, a few shell fragments, and mica flakes, at 26.2m (28.7m)	III
	28.7 to 34.1	PUSH, SAVE	Fat CLAY (CH), very stiff, gray -silt with clay layers to 29.9m -with sandy silt pockets and partings, and shell fragments, 30.8m to 32.0m (34.1m)	IV
to -49	34.1 to 39+	PUSH	Fat CLAY (CH), very stiff to hard, gray -greenish gray with silt pockets and partings, at 36.6m	V



LOG OF BORING AND TEST RESULTS
BORING 98-29
SFOBB East Span Seismic Safety Project

Report Date: 04/28/99

FUGRO

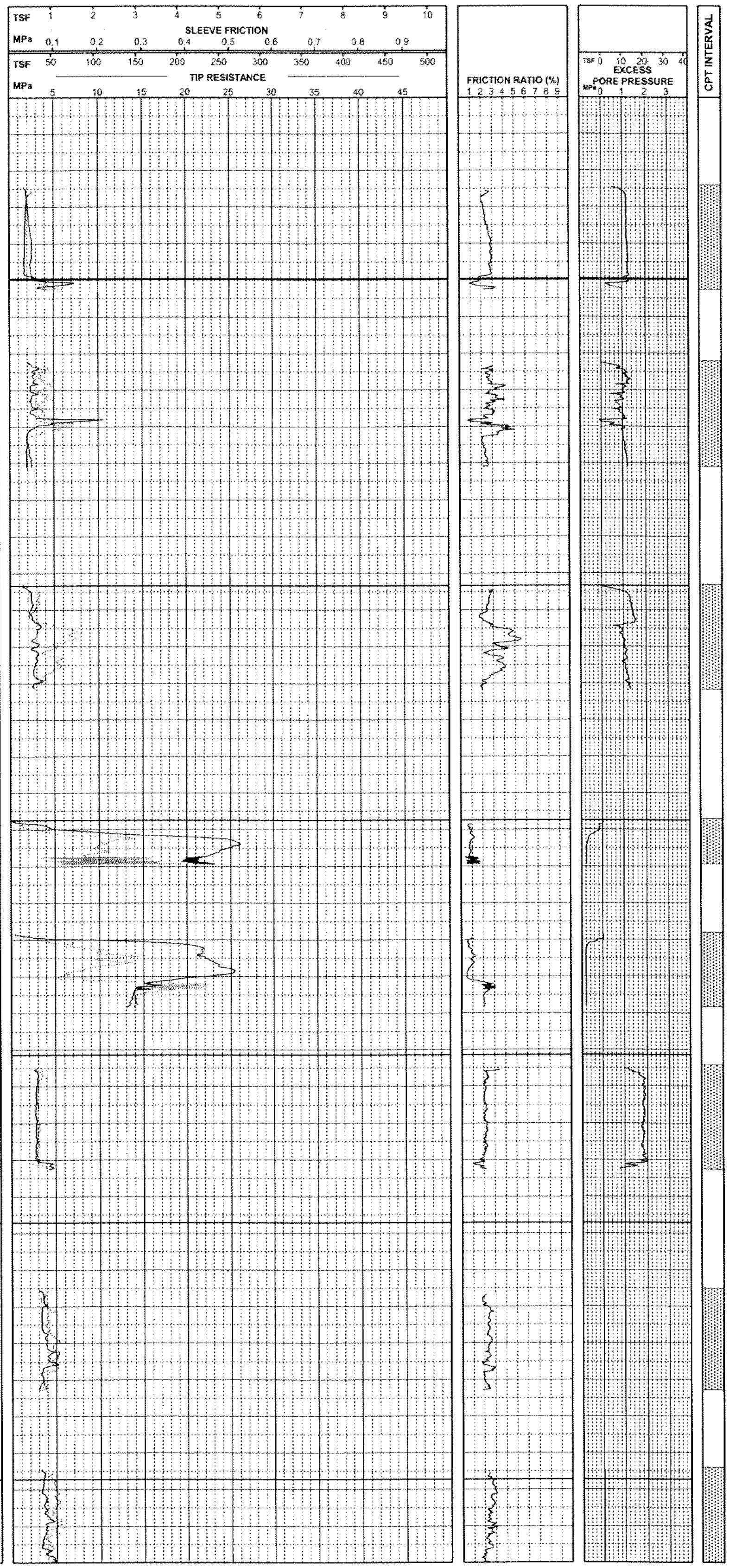
PLATE 98-29.7a

PROJECT NO:	98-42-0054	START DATE:	09/03/98	DRILLER:	Fugro-McClelland Marine Geosciences
BORING:	98-29 (Skyway - Frame 1)	COMPLETION DATE:	09/06/98	DRILLING METHOD:	Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -9.8m (MSL)

DEPTH, m.	SAMPLER / BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
40–45.0	PUSH (41m)	Fat CLAY (CH), very stiff to hard, gray -with a few silt pockets at 41.1m -with organic stains and organic pockets, below 41.1m (45.0m)	V
45.0–53.3	30/40cm (46m); PUSH (51m); SAVE (52m)	Fat CLAY (CH), very stiff, gray -very stiff to hard sandy lean clay with many silt seams, to 48.8m -with a sand layer at 45.1m with silty sand pockets and seams, at 46.3m -sand layer, 48.8m to 49.1m (53.3m)	VI
53.3–59.7	30/40cm (57m)	Lean CLAY (CL), very stiff to hard, greenish gray -with a few silt pockets and partings, gas blisters, and organic stains, at 57.2m (59.7m)	VII
59.7–66.1	30/15cm (61.7m); 30/30cm (65m)	Fine to Coarse SAND with silt (SW-SM), dense, gray -with gravel at 61.7m -silty fine sand below 64.2m -with clay pockets, partings and seams, at 65.2m (66.1m)	VIII
66.1–70.7	NR (70m)	Lean CLAY (CL), very stiff to hard, olive gray -with a sand layer at 69.2m (70.7m)	IX
70.7–77.7	SAVE (71m); PUSH (71.5m); PUSH (76.5m)	Lean CLAY (CL), hard, olive gray -with silt partings and pockets, to 71.9m -with silty sand pockets and partings, at 71.6m -with wood fragments and organic pockets, at 76.5m (77.7m)	X
77.7–		Lean CLAY (CL), hard, gray	XI



LOG OF BORING AND TEST RESULTS

BORING 98-29

SFOBB East Span Seismic Safety Project

Report Date: 04/28/99

PROJECT NO: 98-42-0054
BORING: 98-29 (Skyway - Frame 1)

START DATE: 09/03/98
COMPLETION DATE: 09/06/98

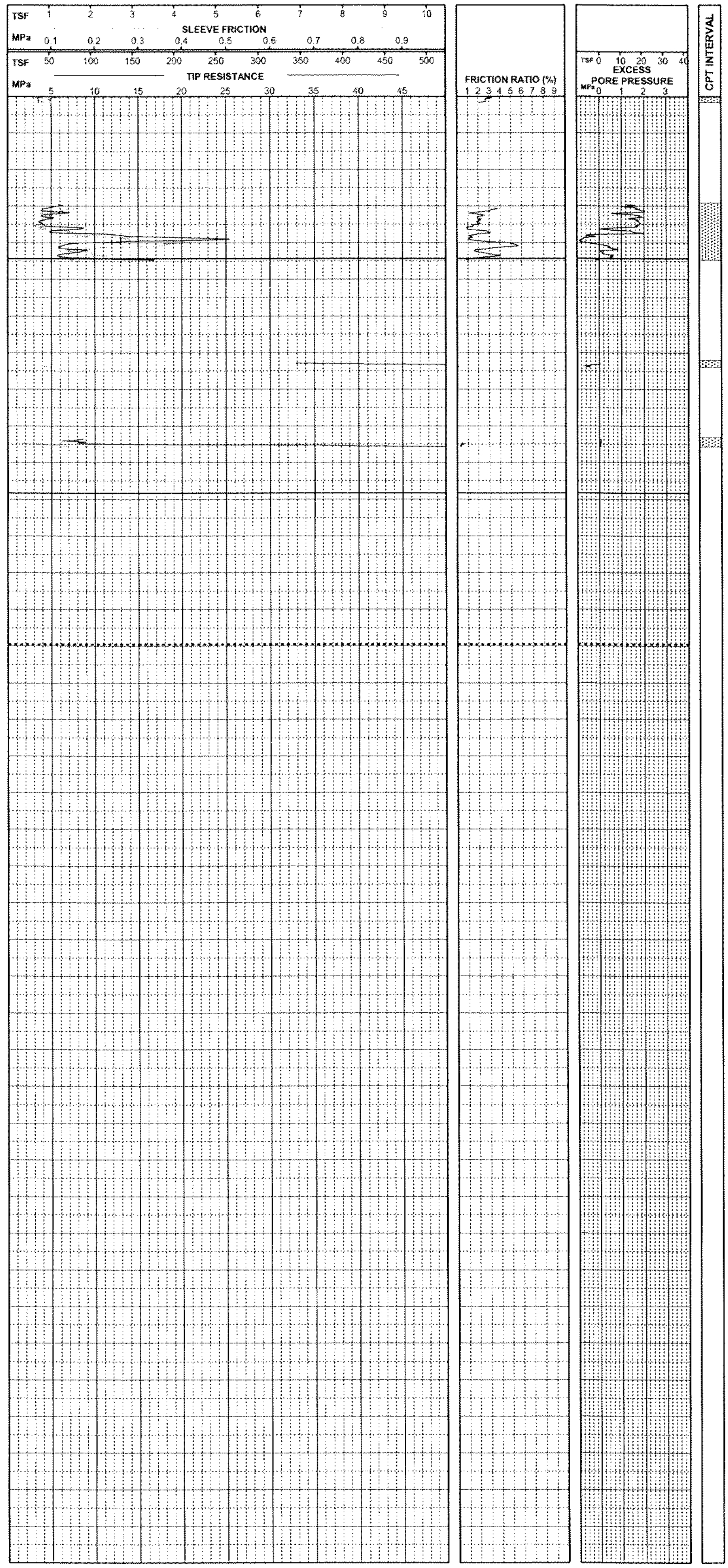
DRILLER: Fugro-McClelland Marine Geosciences
DRILLING METHOD: Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

ELEVATION, m.	DEPTH, m.	SOIL TYPE	SAMPLER	BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
					MUDLINE ELEVATION: -9.8m (MSL)	
-90 to -94	81–84			PUSH SAVE	Lean CLAY (CL), hard, gray -with silt pockets and a few shell fragments, at 81.4m -interlayered sand and clay, below 82.9m (84.4m)	XI
-94 to -100	85–90			30/30cm 30/25cm	Fine to Medium SAND with silt (SP-SM), dense to very dense, gray -with a clay layer at 86.7m -with a lean clay layer at 88.8m (90.8m)	XII
-101 to -104	91–95			30/5cm 50/5cm	Fine GRAVEL (GP), very dense, dark gray -sand layer, 91.4m to 92.4m -dense to very dense, fine to coarse sand, at 94.8m	XIII
					TOTAL DEPTH: 94.9m BACKFILLED WITH: Cement Grout	

Elevation scale: -90, -91, -92, -93, -94, -95, -96, -97, -98, -99, -100, -101, -102, -103, -104, -105, -106, -107, -108, -109, -110, -111, -112, -113, -114, -115, -116, -117, -118, -119, -120, -121, -122, -123, -124, -125, -126, -127, -128, -129

Depth scale: 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119



LOG OF BORING AND TEST RESULTS

BORING 98-29

SFOBB East Span Seismic Safety Project

Report Date: 04/28/99

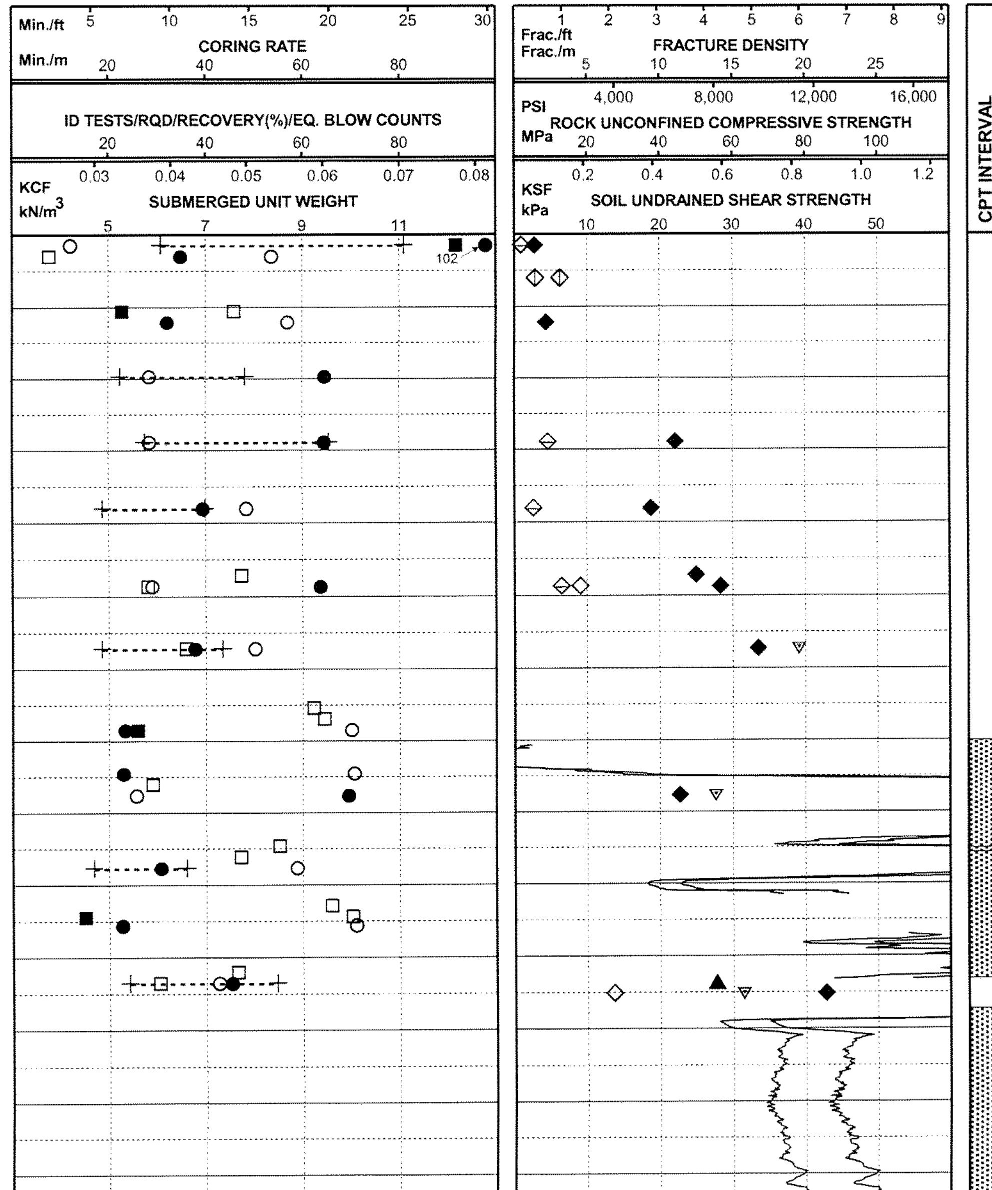
FUGRO

PLATE 98-29.7c

PROJECT NO:	98-42-0054	**START DATE:**	09/03/98	**DRILLER:**	Fugro-McClelland Marine Geosciences
BORING:	98-29 (Skyway - Frame 1)	**COMPLETION DATE:**	09/06/98	**DRILLING METHOD:**	Rotary Sample Boring (Wet)

Coordinates: E1837145 N647992
CA State Plane Zone 3, NAD83, Meters

ELEVATION, m.	DEPTH, m.	BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
			MUDLINE ELEVATION: -9.8m (MSL)	
-10		WOH	Fat CLAY (CH), soft to firm, dark gray -with shells and shell fragments -very soft, dark gray clay, to 0.9m -with many sand seams and pockets to 0.6m	
-11	1	WOH	-very loose, olive gray, silty fine sand layer, with many clay pockets, 0.9m to 1.2m	
-12	2	WOH		
-13	3	WOH		
-14	4	WOH	-with many shell fragments below 3.7m -with sand pockets, 3.7m to 4.7m	
-15	5	WOH		
	6	WOH	-with sandy silt pockets at 5.5m	
-16	7	17/45cm	-intermixed clay and sand, 6.4m to 7.0m	I
-17		18/60cm	-with sand seams at 7.3m -with gas blisters below 7.5m -sand layer, 7.6m to 8.2m	
-18	8	30/55cm	-sand layer, 8.5m to 8.8m	
-19	9	30/45cm	-silty sand layer with clay pockets, 9.1m to 9.7m	
-20	10	PUSH	-sand layer at 10.1m -with very few silt pockets and gas blisters, below 10.2m	
-21	11			
-22	12			
-23	13			

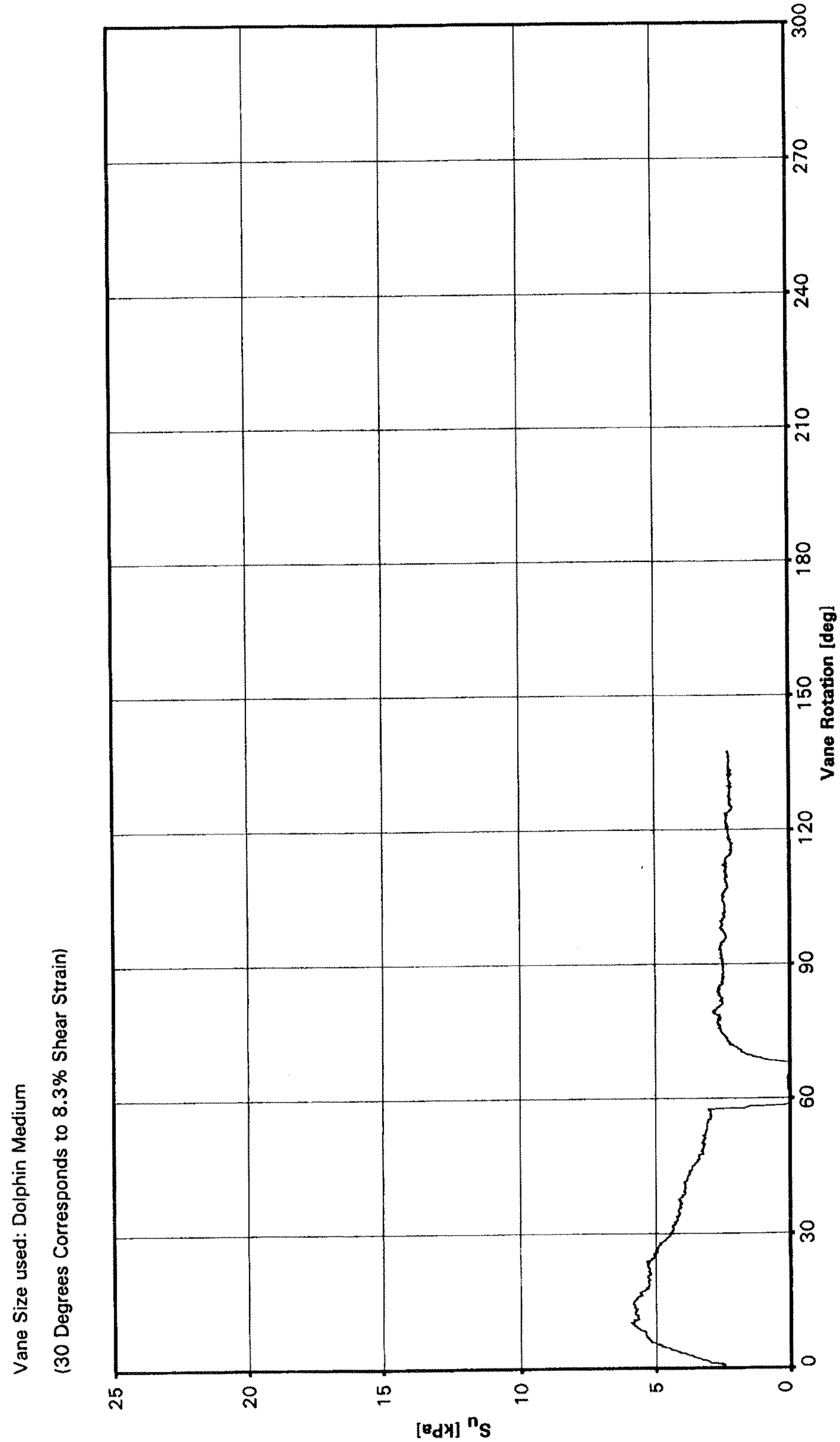


LOG OF BORING AND TEST RESULTS

BORING 98-29

SFOBB East Span Seismic Safety Project

Report Date: 04/30/99



REMOTE VANE (HALIBUT) TEST RESULTS
Test Depth: 0.6m
Boring 98-29
SFOBB East Span Seismic Safety Project

98-29		IDENTIFICATION TESTS						STRENGTH ESTIMATE			MINIATURE VANE TESTS			REMOTE VANE (kPa)	UU TRIAXIAL			MULTI-STAGE TRIAXIAL		DIRECT SHEAR TESTS				ADDITIONAL TESTS	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
DEPTH (m)	Sample No.	MC (%)	LL (%)	PL (%)	LI	SUW (kN/m3)	Fines (%)	Torvane (kPa)	Pocket Pen. (kPa)	Fall Cone (kPa)	Undist. (kPa)	Remold. (kPa)	Resid. (kPa)		Undist. (kPa)	Remold. (kPa)	e50 (%)	c (kPa)	phi (deg)	Peak c (kPa)	Peak phi (deg)	Post Peak c (kPa)	Post Peak phi (deg)			
0.2	1	102	81	31	1.41		92				2.8		0.9											H		
0.3	2	35				3.8																				
0.6	401													2.9												
0.6	501													6.2												
1.1	3					7.6	23																			
1.2	4	32									4.4															
2.0	5	65	48	22	1.63																					
2.9	6	64	66	28	0.97						22.2		4.6													
3.8	7	39	40	19	0.98						18.8		2.6													
4.7	8					7.7					25.0															
4.9	9	64				5.8					28.3	9.0	6.4													
5.7	10	38	44	19	0.77	6.6				39.3	33.5															
6.6	11					9.2																				
6.7	12					9.4																				
6.9	13	23					26																	H		

Identification Tests
MC = Moisture Content
LL = Liquid Limit
PL = Plastic Limit
LI = Liquidity Index

Identification Tests
SUW = Submerged Unit Weight
Fines = % Passing No. 200 Sieve

Strength Tests
UU = Unconsolidated Undrained
e50 = Strain at 50% Failure Stress
c = Effective Cohesion
phi = Effective Angle of Friction

Additional Tests
H = Hydrometer
C = Consolidation Test
RC = Resonant Column
CS = Cyclic Simple Shear

Additional Tests
K = Ko Consolidated Triaxial Test

SUMMARY OF LABORATORY TEST RESULTS
Boring 98-29
SFOBB East Span Seismic Safety Project

98-29		IDENTIFICATION TESTS						STRENGTH ESTIMATE			MINIATURE VANE TESTS			REMOTE VANE (kPa)	UU TRIAXIAL			MULTI-STAGE TRIAXIAL		DIRECT SHEAR TESTS				ADDITIONAL TESTS	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
DEPTH (m)	Sample No.	MC (%)	LL (%)	PL (%)	LI	SUW (kN/m3)	Fines (%)	Torvane (kPa)	Pocket Pen. (kPa)	Fall Cone (kPa)	Undist. (kPa)	Remold. (kPa)	Resid. (kPa)		Undist. (kPa)	Remold. (kPa)	e50 (%)	c (kPa)	phi (deg)	Peak c (kPa)	Peak phi (deg)	Post Peak c (kPa)	Post Peak phi (deg)			
7.5	14	23																								
7.6	15					5.9																				
7.8	16	69								27.8	22.6															
8.5	17					8.5																				
8.6	18					7.7																				
8.8	19	31	36	17	0.72																					
9.3	20					9.6																				
9.4	21					10.0	15											0.0	35							
9.6	22	23																								
10.2	23					7.6																				
10.4	24	45	55	24	0.69	6.0									27.7		2.8									
10.5	25									31.6	42.8	13.6														
14.2	27	37																								
17.7	28					6.7			131.7																	
17.8	29		77	28		6.7	98								91.4	71.7	0.6							H		

Identification Tests
MC = Moisture Content
LL = Liquid Limit
PL = Plastic Limit
LI = Liquidity Index

Identification Tests
SUW = Submerged Unit Weight
Fines = % Passing No. 200 Sieve

Strength Tests
UU = Unconsolidated Undrained
e50 = Strain at 50% Failure Stress
c = Effective Cohesion
phi = Effective Angle of Friction

Additional Tests
H = Hydrometer
C = Consolidation Test
RC = Resonant Column
CS = Cyclic Simple Shear

Additional Tests
K = Ko Consolidated Triaxial Test

SUMMARY OF LABORATORY TEST RESULTS
Boring 98-29
SFOBB East Span Seismic Safety Project

Earth Mechanics

FUGRO

98-29		IDENTIFICATION TESTS						STRENGTH ESTIMATE			MINIATURE VANE TESTS			REMOTE VANE (kPa)	UU TRIAXIAL			MULTI-STAGE TRIAXIAL		DIRECT SHEAR TESTS				ADDITIONAL TESTS	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
DEPTH (m)	Sample No.	MC (%)	LL (%)	PL (%)	LI	SUW (kN/m3)	Fines (%)	Torvane (kPa)	Pocket Pen. (kPa)	Fall Cone (kPa)	Undist. (kPa)	Remold. (kPa)	Resid. (kPa)		Undist. (kPa)	Remold. (kPa)	e50 (%)	c (kPa)	phi (deg)	Peak c (kPa)	Peak phi (deg)	Post Peak c (kPa)	Post Peak phi (deg)			
18.0	30										131.2															
21.3	31	46	63	23	0.58				95.8															C		
21.5	32	38	54	23	0.51	7.8									113.0	46.9	0.6									
21.6	33								95.8		172.0															
26.4	35	39	53	24	0.51	7.7		131.7	95.8						89.2		3.0									
26.5	36										173.3															
31.2	38	42	50	21	0.71	7.9									146.3	34.5	1.7									
31.4	39										181.9															
31.9	40	42																								
31.9	40	37	48	24	0.57																			C,K	2.73	
32.0	41										153.5															
36.6	42								167.6																	
36.7	43	36	57	26	0.33	8.2			167.6																	
36.9	44										175.8															
41.5	45								215.5																	

Identification Tests
MC = Moisture Content
LL = Liquid Limit
PL = Plastic Limit
LI = Liquidity Index

Identification Tests
SUW = Submerged Unit Weight
Fines = % Passing No. 200 Sieve

Strength Tests
UU = Unconsolidated Undrained
e50 = Strain at 50% Failure Stress
c = Effective Cohesion
phi = Effective Angle of Friction

Additional Tests
H = Hydrometer
C = Consolidation Test
RC = Resonant Column
CS = Cyclic Simple Shear

Additional Tests
K = Ko Consolidated Triaxial Test

SUMMARY OF LABORATORY TEST RESULTS
Boring 98-29
SFOBB East Span Seismic Safety Project

98-29		IDENTIFICATION TESTS						STRENGTH ESTIMATE			MINIATURE VANE TESTS			REMOTE VANE (kPa)	UU TRIAXIAL			MULTI-STAGE TRIAXIAL		DIRECT SHEAR TESTS				ADDITIONAL TESTS	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
DEPTH (m)	Sample No.	MC (%)	LL (%)	PL (%)	LI	SUW (kN/m3)	Fines (%)	Torvane (kPa)	Pocket Pen. (kPa)	Fall Cone (kPa)	Undist. (kPa)	Remold. (kPa)	Resid. (kPa)		Undist. (kPa)	Remold. (kPa)	e50 (%)	c (kPa)	phi (deg)	Peak c (kPa)	Peak phi (deg)	Post Peak c (kPa)	Post Peak phi (deg)			
41.6	46	40	57	24	0.47	8.0									181.7	44.6	0.7									
41.8	47							119.7			147.3															
46.3	48	21	29	14	0.48	11.1	67								156.3		5.0							H		
46.4	49									227.0			47.0													
51.5	50	41	72	28	0.28	7.4		131.7	143.6		177.0					54.3										
52.7	51	44	76	24	0.38																			C	2.76	
52.7	52										155.9															
57.2	53	26	40	19	0.33	10.0									138.0		3.1									
57.3	54							136.5	119.7		131.2															
61.9	55	11					7																		2.74	
65.5	56	23				10.1	12																			
71.2	57	26	42	19	0.33																			C		
71.3	58										222.8															
71.6	59								191.5																	
71.8	60	30	47	20	0.36	9.0			191.5						182.9	76.8	1.3									

Identification Tests
MC = Moisture Content
LL = Liquid Limit
PL = Plastic Limit
LI = Liquidity Index

Identification Tests
SUW = Submerged Unit Weight
Fines = % Passing No. 200 Sieve

Strength Tests
UU = Unconsolidated Undrained
e50 = Strain at 50% Failure Stress
c = Effective Cohesion
phi = Effective Angle of Friction

Additional Tests
H = Hydrometer
C = Consolidation Test
RC = Resonant Column
CS = Cyclic Simple Shear

Additional Tests
K = Ko Consolidated Triaxial Test

SUMMARY OF LABORATORY TEST RESULTS
Boring 98-29
SFOBB East Span Seismic Safety Project

Earth Mechanics

FUGRO

98-29		IDENTIFICATION TESTS						STRENGTH ESTIMATE			MINIATURE VANE TESTS			REMOTE VANE (kPa)	UU TRIAXIAL			MULTI-STAGE TRIAXIAL		DIRECT SHEAR TESTS				ADDITIONAL TESTS	SPECIFIC GRAVITY	ORGANIC CONTENT (%)
DEPTH (m)	Sample No.	MC (%)	LL (%)	PL (%)	LI	SUW (kN/m3)	Fines (%)	Torvane (kPa)	Pocket Pen. (kPa)	Fall Cone (kPa)	Undist. (kPa)	Remold. (kPa)	Resid. (kPa)		Undist. (kPa)	Remold. (kPa)	e50 (%)	c (kPa)	phi (deg)	Peak c (kPa)	Peak phi (deg)	Post Peak c (kPa)	Post Peak phi (deg)			
71.9	61										199.3															
76.5	62								215.5																	
76.7	63	49	79	35	0.32	6.9			216.1						233.1		0.8									
76.8	64					10.0			245.3		222.8															11
81.4	65	24	37	19	0.30			181.9	186.9						184.7	118.7	0.7									
81.5	66	27	40	17	0.42	9.9																				
81.7	67										206.7															
82.2	68	25	42	19	0.26																			C,K		
82.3	69						95				222.8															
86.7	70	22					8																			
86.8	71	21																								
88.8	72					10.7																				
88.9	73	21					38																			
90.8	74	4					< 1																			

Identification Tests
MC = Moisture Content
LL = Liquid Limit
PL = Plastic Limit
LI = Liquidity Index

Identification Tests
SUW = Submerged Unit Weight
Fines = % Passing No. 200 Sieve

Strength Tests
UU = Unconsolidated Undrained
e50 = Strain at 50% Failure Stress
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Additional Tests
H = Hydrometer
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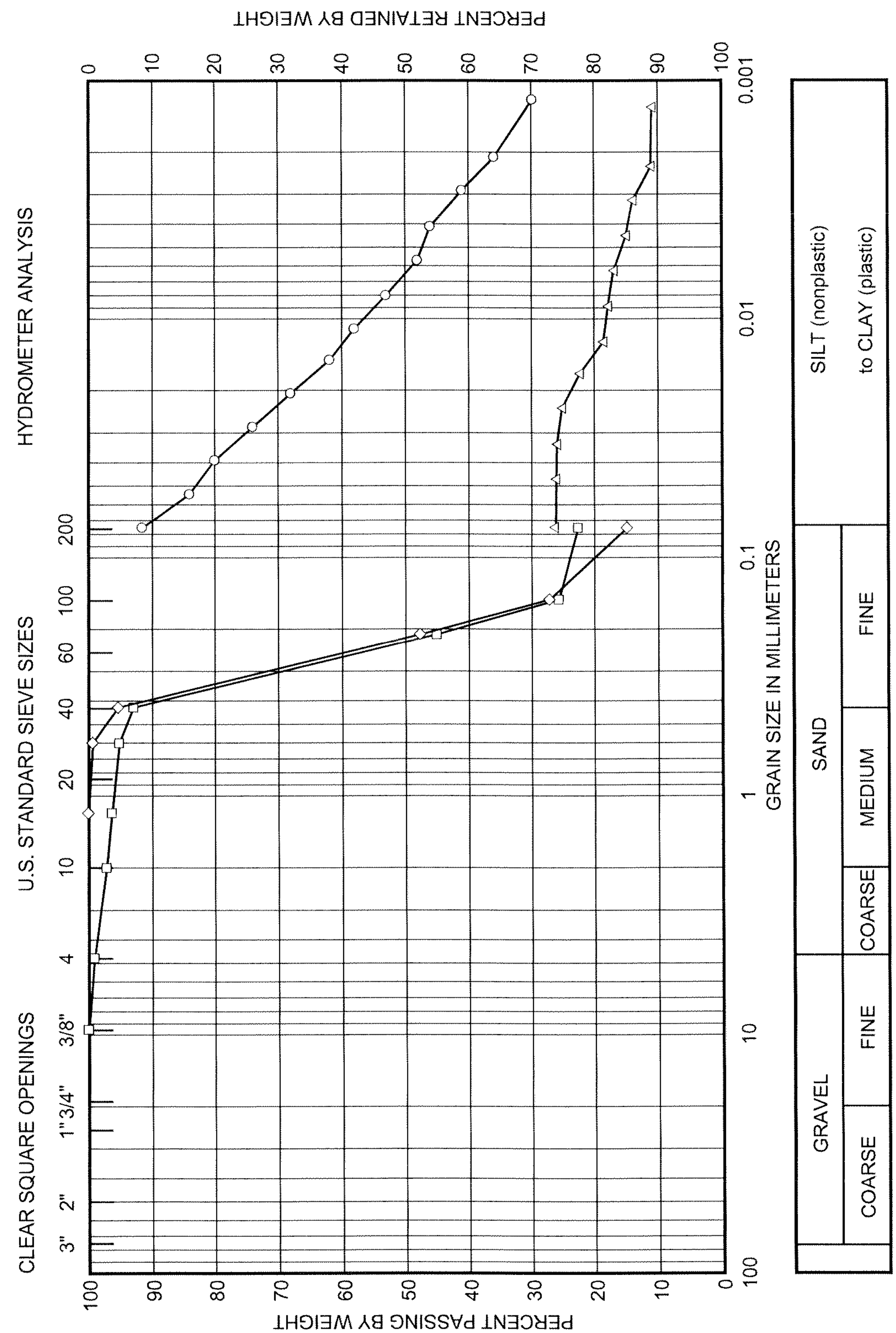
Additional Tests
K = Ko Consolidated Triaxial Test

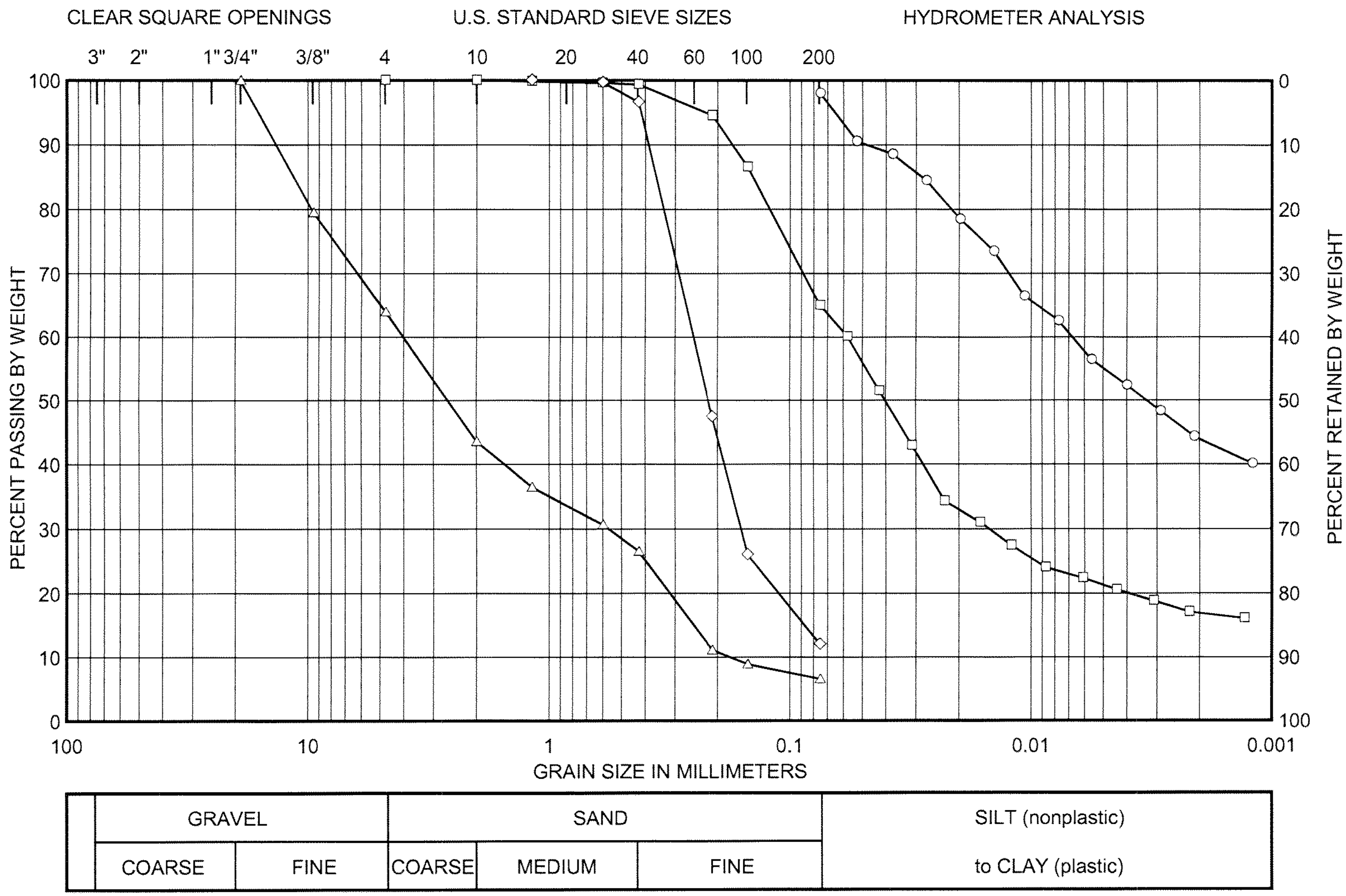
SUMMARY OF LABORATORY TEST RESULTS
Boring 98-29
SFOBB East Span Seismic Safety Project



SAMPLE NO.	DEPTH (m)	CURVE	CLASSIFICATION	Cc	Cu	D50 (mm)
1	0.2	○	FAT CLAY (CH)			0.0060
3	1.1	□	SILTY FINE SAND (SM) with a trace of medium sand			0.23
13	6.9	△	Intermixed SAND and CLAY			
21	9.4	◇	SILTY FINE SAND (SM)			0.22

GRAIN SIZE DISTRIBUTION CURVES
Boring 98-29
SFOBB East Span Seismic Safety Project

PLATE 98-29.11a

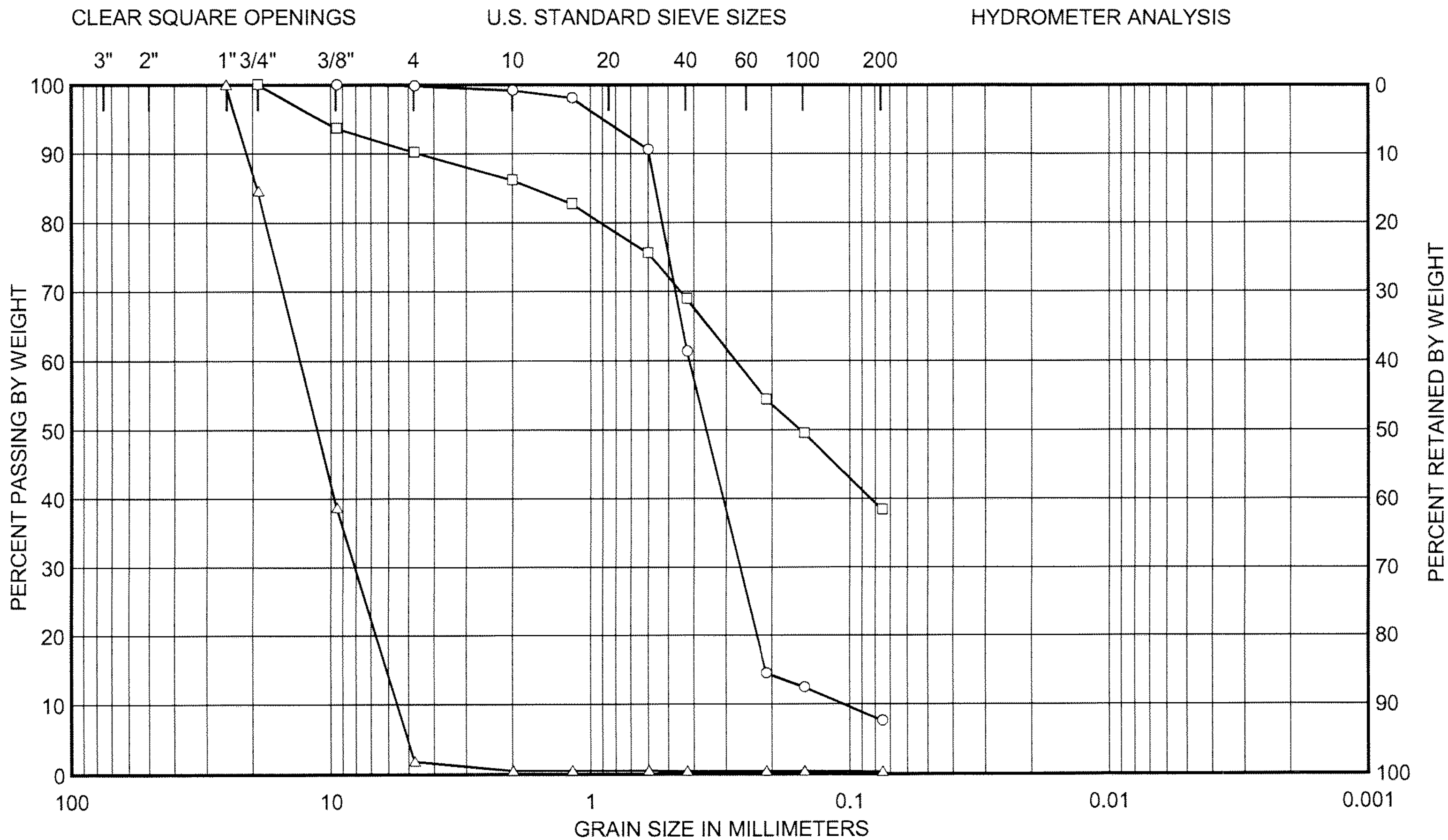


SAMPLE NO.	DEPTH (m)	CURVE	CLASSIFICATION	Cc	Cu	D50 (mm)
29	17.8	○—○	FAT CLAY (CH)			0.0030
48	46.3	□—□	SANDY LEAN CLAY (CL)			0.040
55	61.9	△—△	FINE TO COARSE SAND (SP-SM) with silt and fine gravel	0.4	22.6	2.6
56	65.5	◇—◇	SILTY FINE SAND (SM)			0.22

GRAIN SIZE DISTRIBUTION CURVES
Boring 98-29
SFOBB East Span Seismic Safety Project

Earth Mechanics

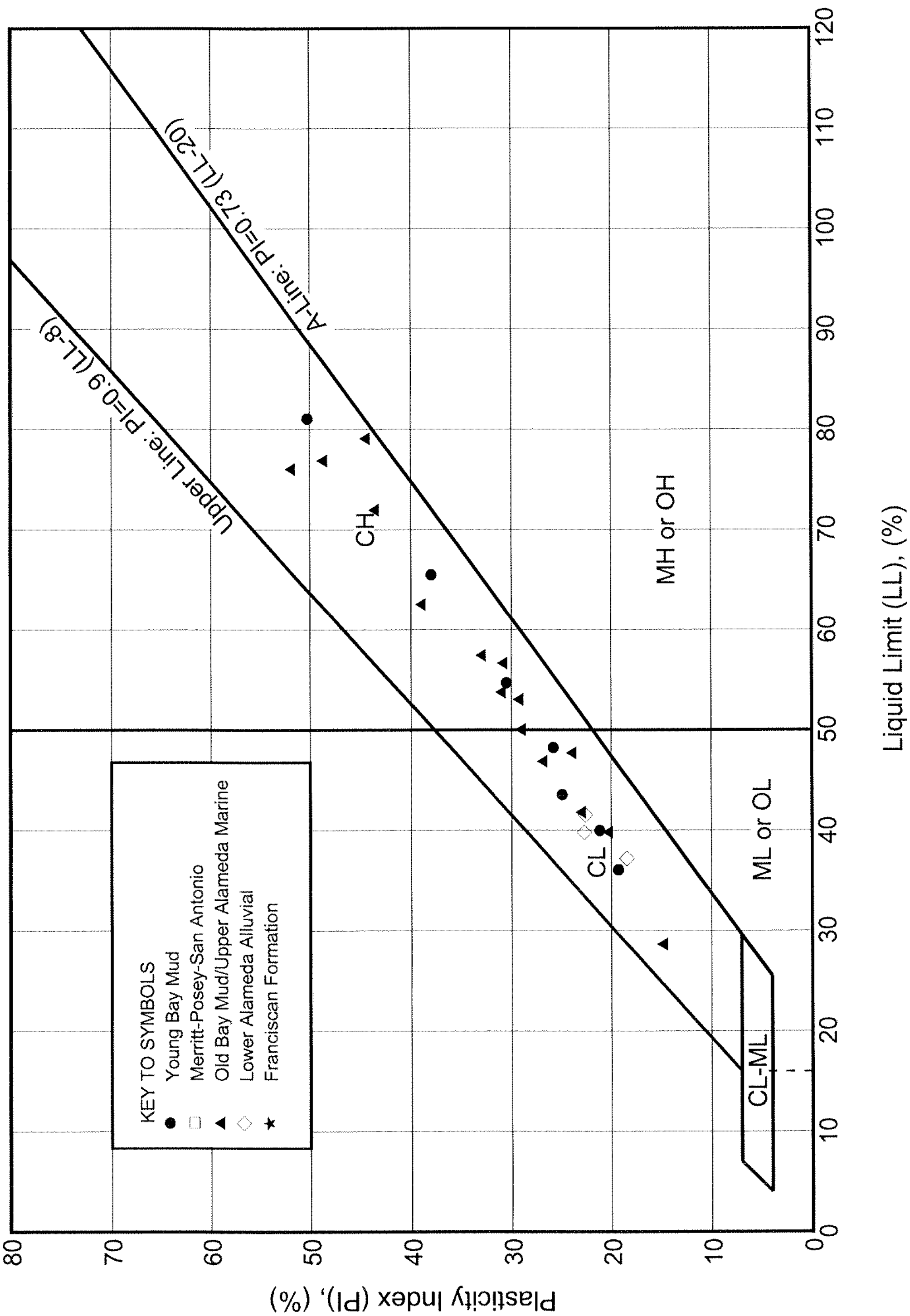
FUGRO



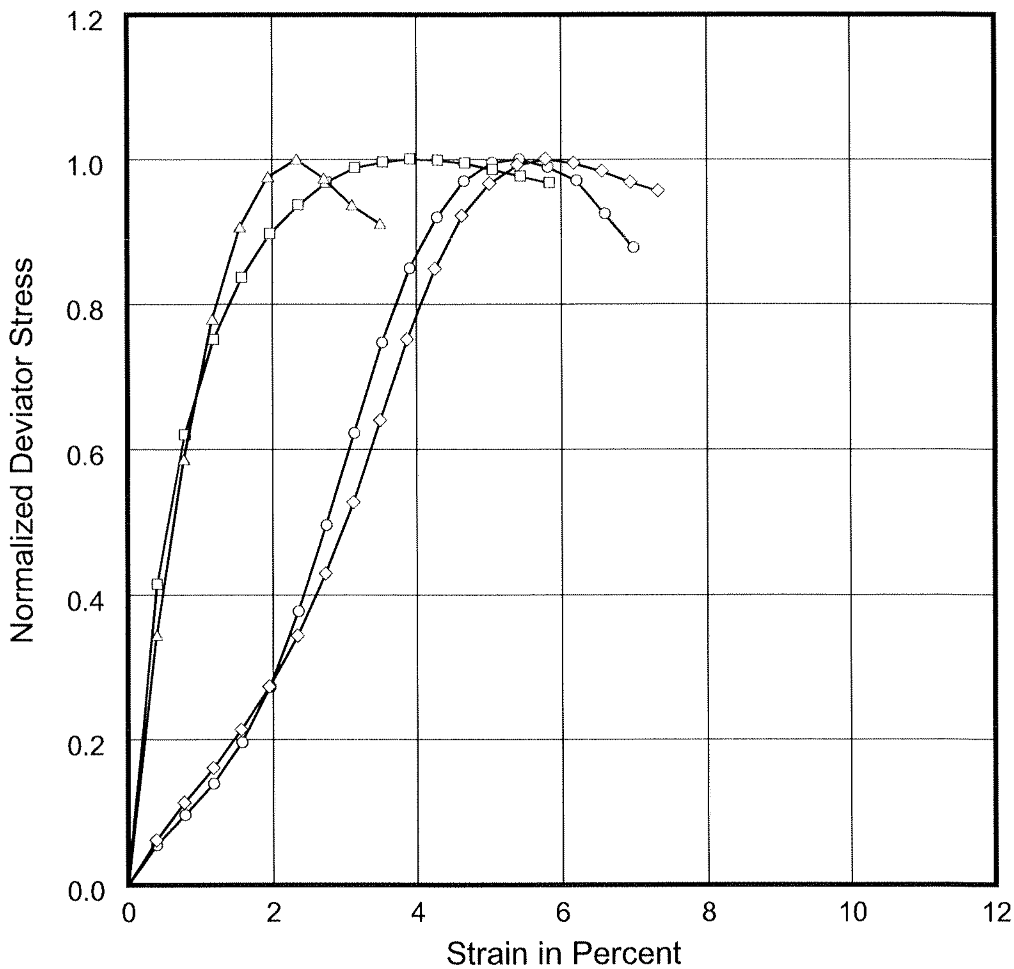
GRAVEL		SAND			SILT (nonplastic) to CLAY (plastic)
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	DEPTH (m)	CURVE	CLASSIFICATION	Cc	Cu	D50 (mm)
70	86.7	○—○	FINE TO MEDIUM SAND (SP-SM) with silt	1.6	3.9	0.36
73	88.9	□—□	SILTY FINE SAND (SM) with medium sand, a trace of coarse sand and clay layers			0.16
74	90.8	△—△	FINE GRAVEL (GP) with a trace of coarse gravel	0.9	2.4	11.3

GRAIN SIZE DISTRIBUTION CURVES
Boring 98-29
SFOBB East Span Seismic Safety Project



PLASTICITY CHART
Boring 98-29
SFOBB East Span Seismic Safety Project

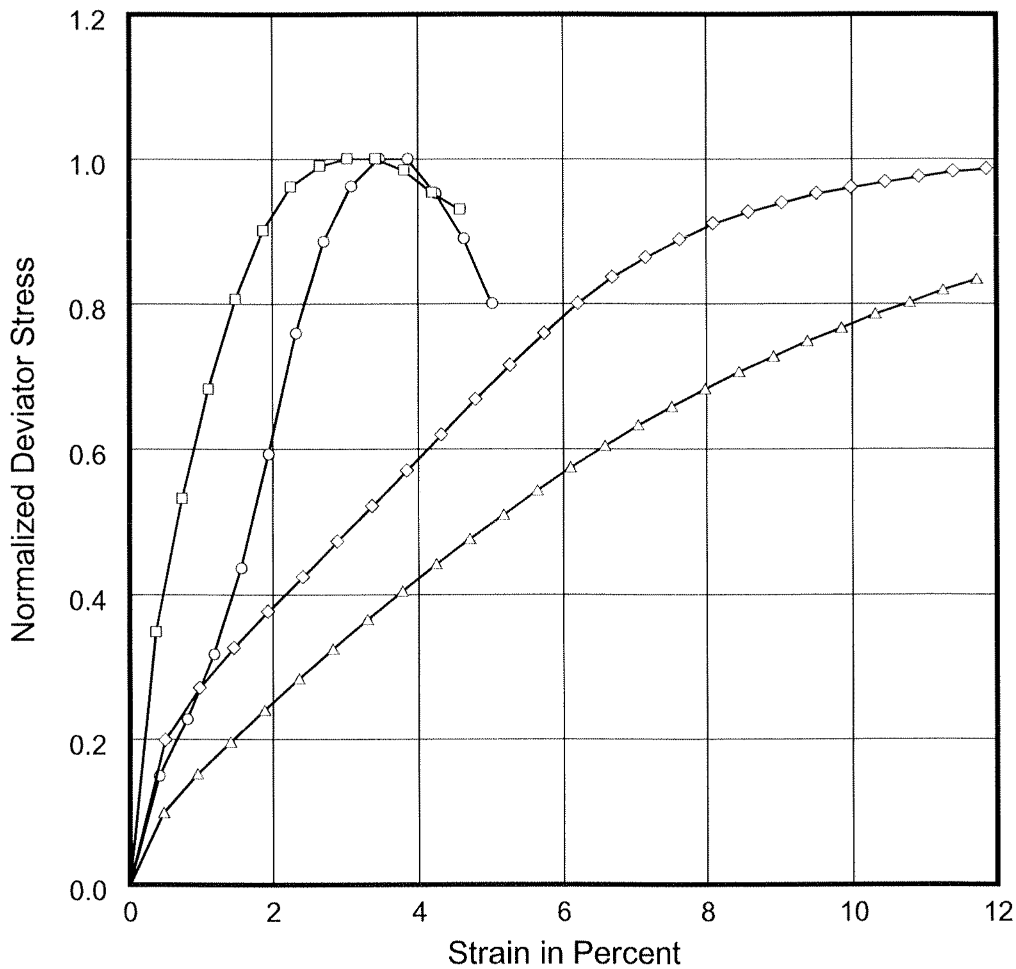


Curve	Sample No.	Depth (m)	Test Type	Confining Pressure (kPa)	Maximum Deviator Stress (kPa)	e50 (%)
○—○	24	10.4	UU	241	55	2.8
□—□	29	17.8	UU	827	183	0.6
△—△	32	21.5	UU	827	226	0.6
◇—◇	35	26.4	UU	827	179	3.0

Deviator stress normalized with respect to maximum deviator stress.

STRESS-STRAIN CURVES

Unconsolidated-Undrained Triaxial Compression Test
Boring 98-29
SFOBB East Span Seismic Safety Project

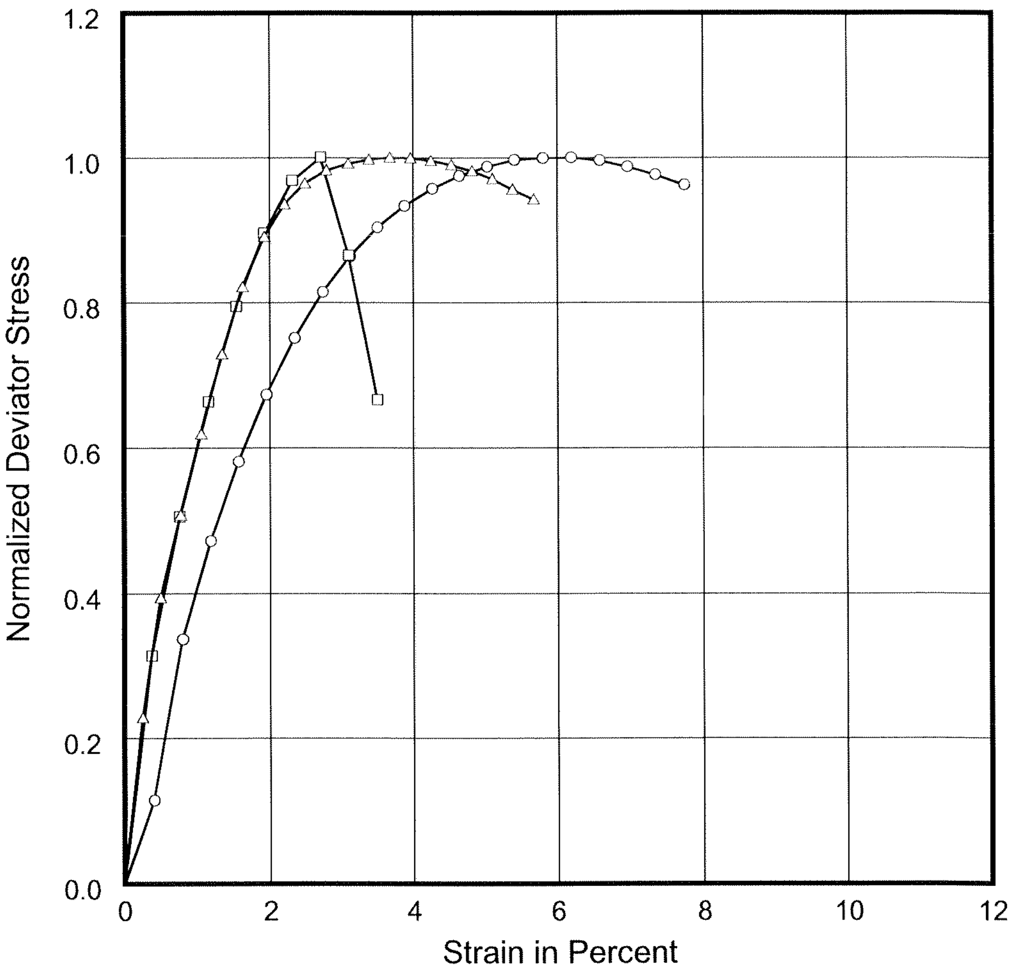


Curve	Sample No.	Depth (m)	Test Type	Confining Pressure (kPa)	Maximum Deviator Stress (kPa)	e50 (%)
○	38	31.2	UU	827	293	1.7
□	46	41.6	UU	931	363	0.7
△	48	46.3	UU	1034	313	5.0
◇	53	57.2	UU	1310	276	3.1

Deviator stress normalized with respect to maximum deviator stress.

STRESS-STRAIN CURVES

Unconsolidated-Undrained Triaxial Compression Test
Boring 98-29
SFOBB East Span Seismic Safety Project



Curve	Sample No.	Depth (m)	Test Type	Confining Pressure (kPa)	Maximum Deviator Stress (kPa)	e50 (%)
○—○	60	71.8	UU	1620	366	1.3
□—□	63	76.7	UU	1724	466	0.8
△—△	65	81.4	UU	1724	369	0.7

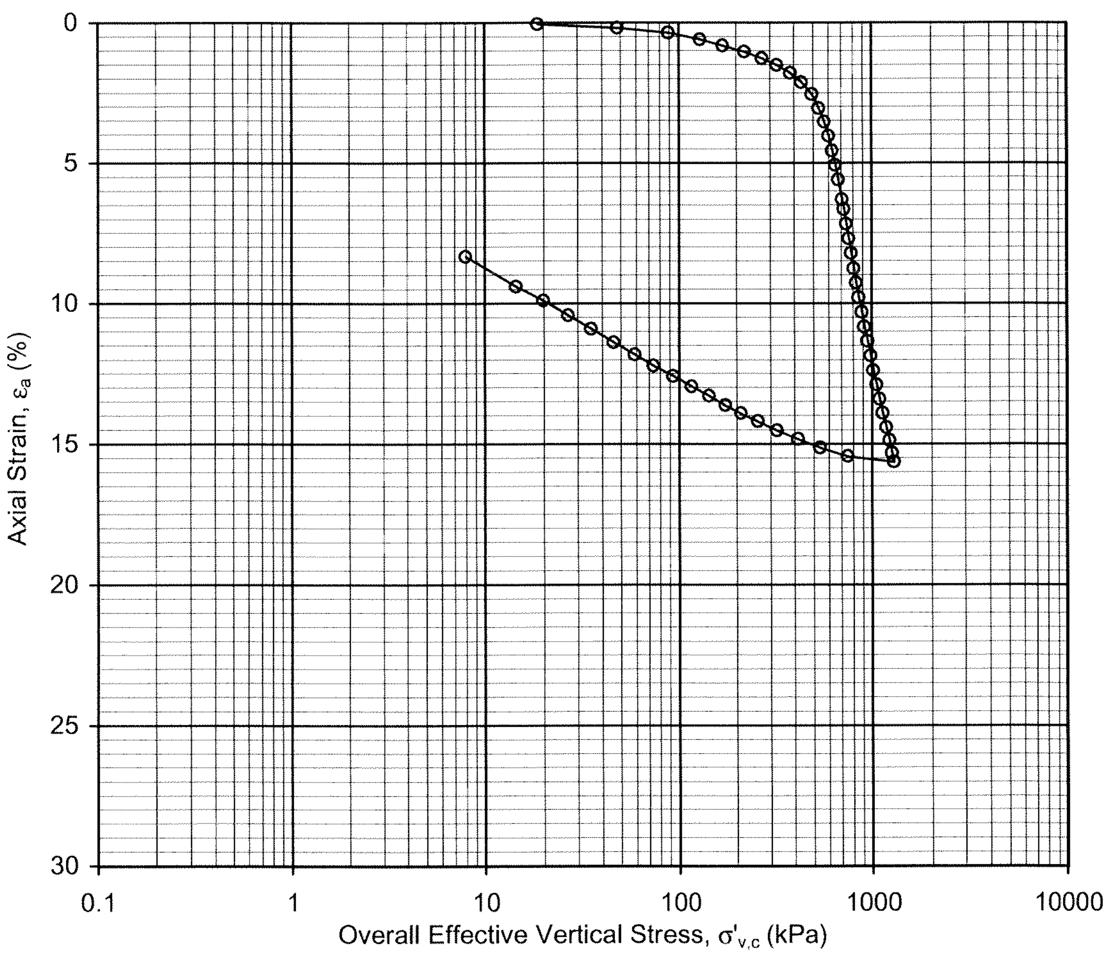
Deviator stress normalized with respect to maximum deviator stress.

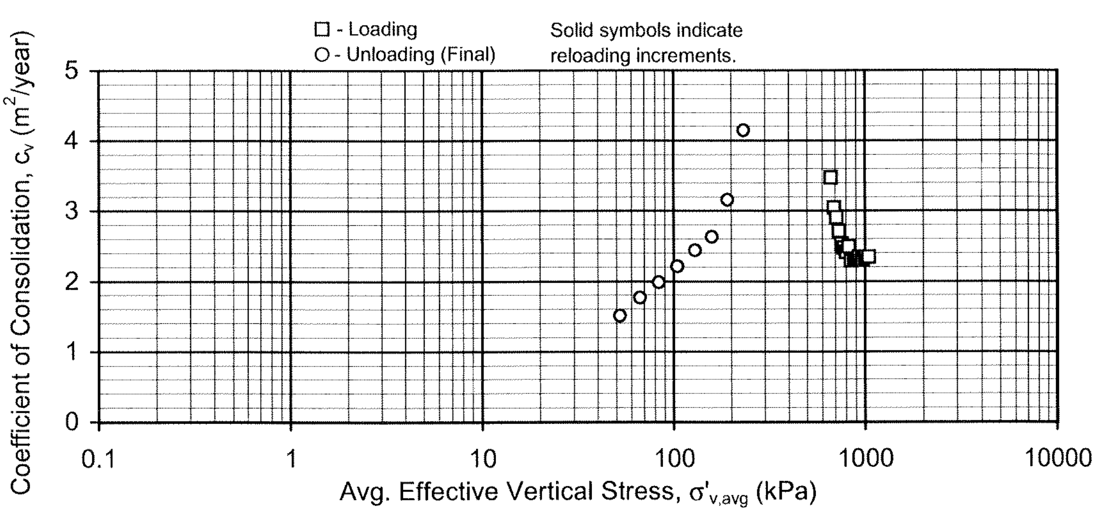
STRESS-STRAIN CURVES

Unconsolidated-Undrained Triaxial Compression Test

Boring 98-29

SFOBB East Span Seismic Safety Project



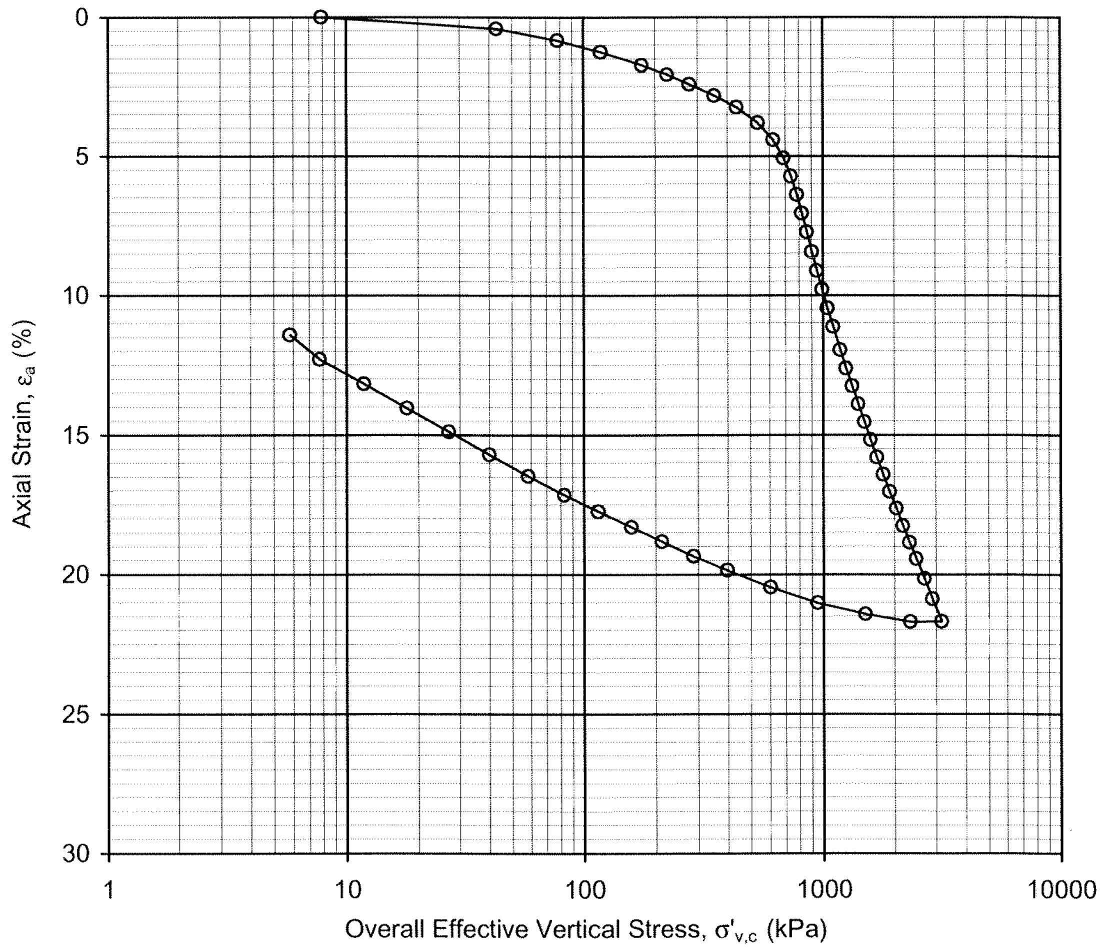


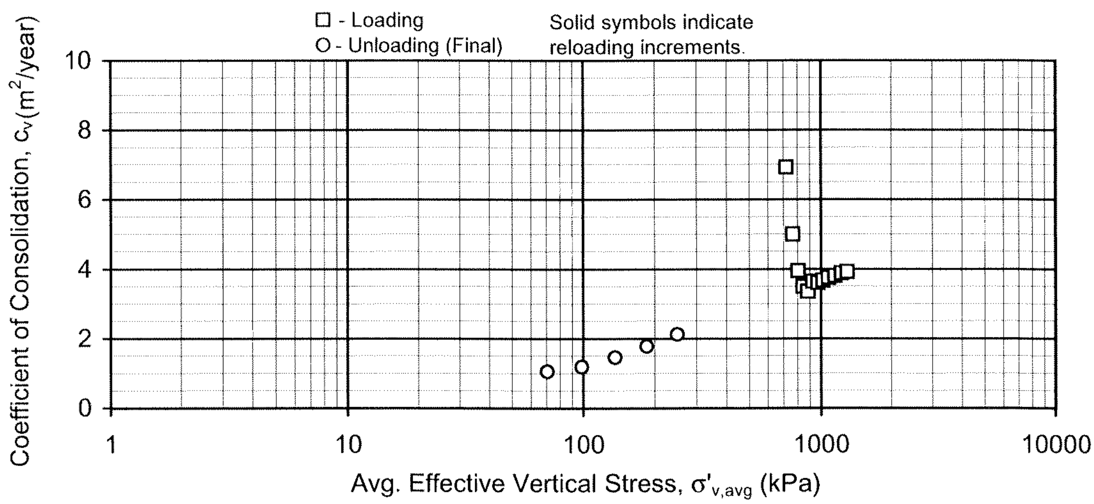
CRS CONSOLIDATION TEST RESULTS

Sample No. 31 - Depth: 21.3m

Boring 98-29

SFOBB East Span Seismic Safety Project



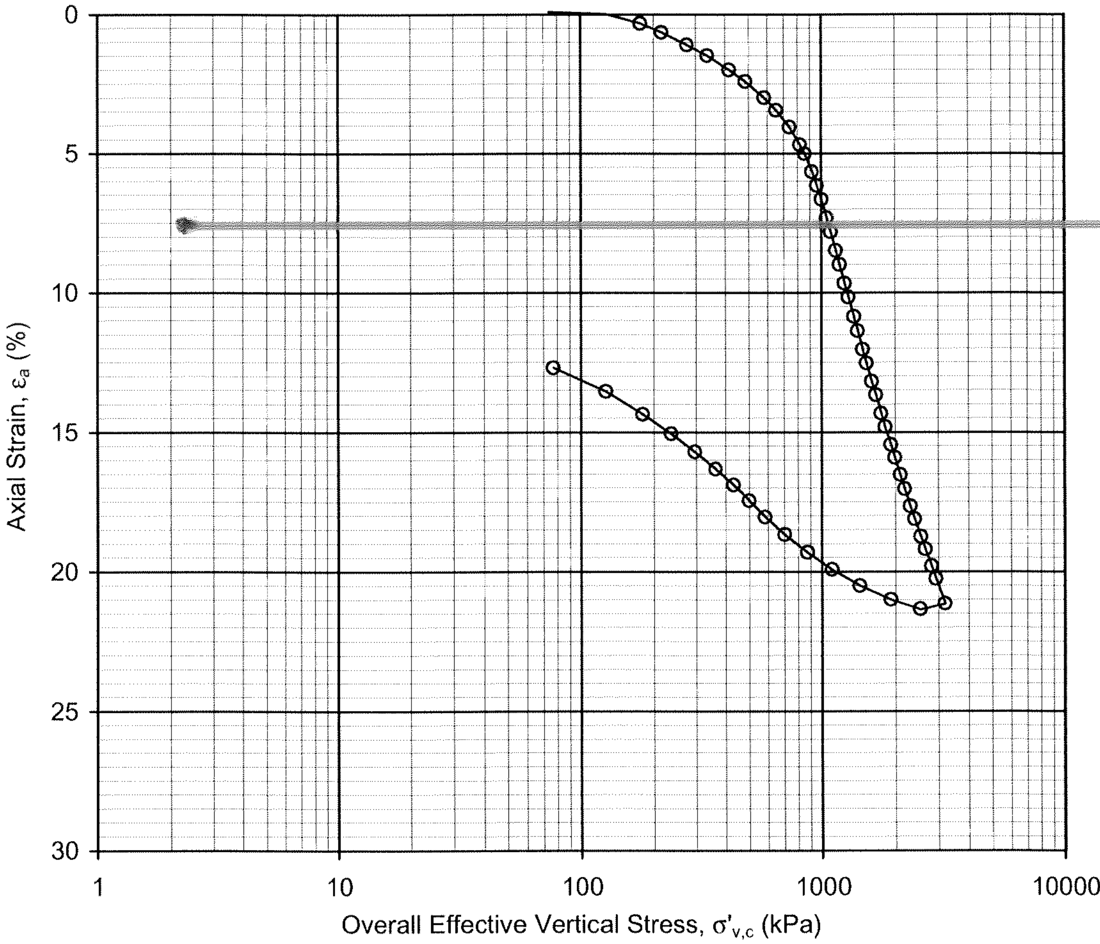


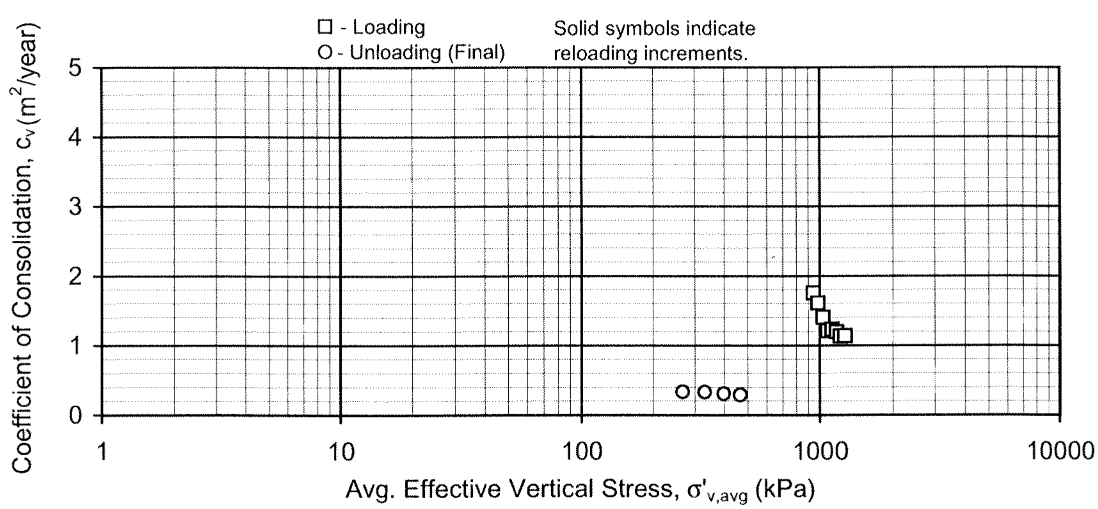
CRS CONSOLIDATION TEST RESULTS

Sample No. 40 - Depth: 31.9m

Boring 98-29

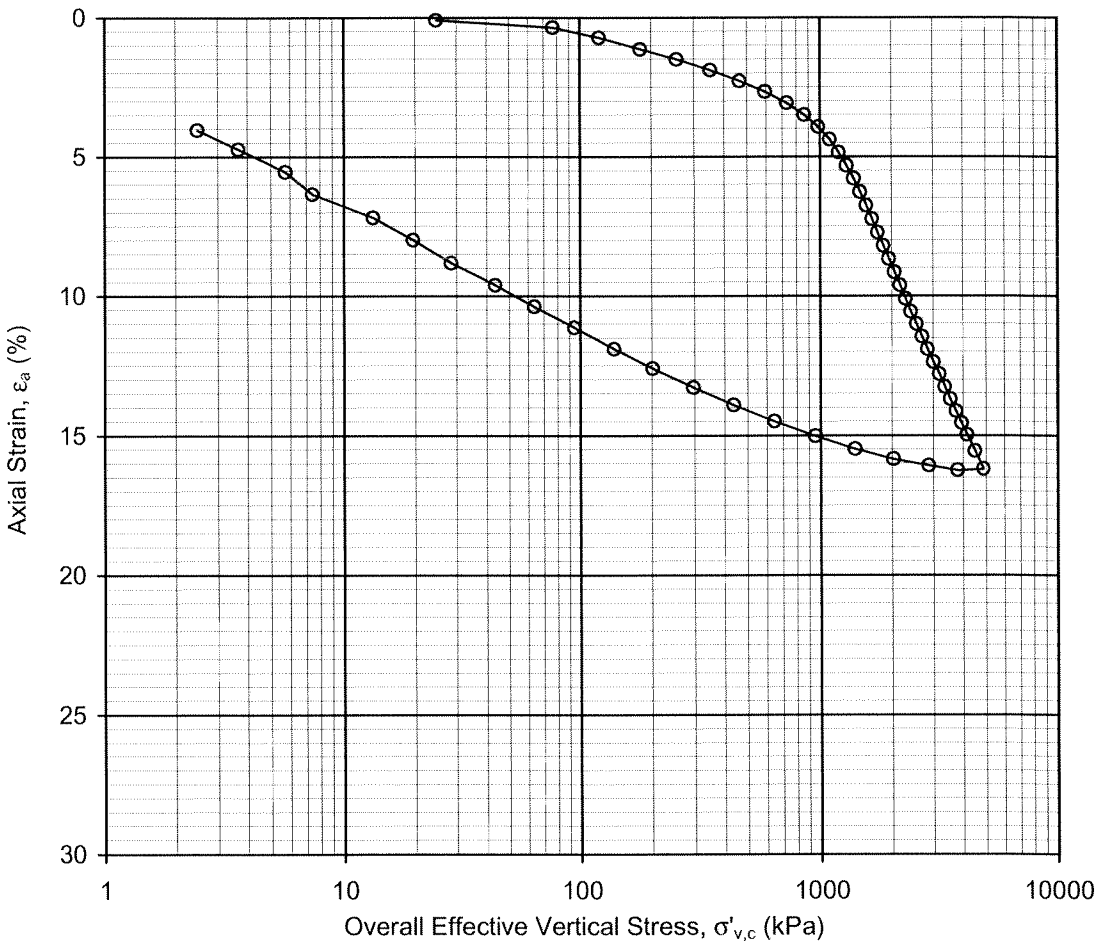
SFOBB East Span Seismic Safety Project

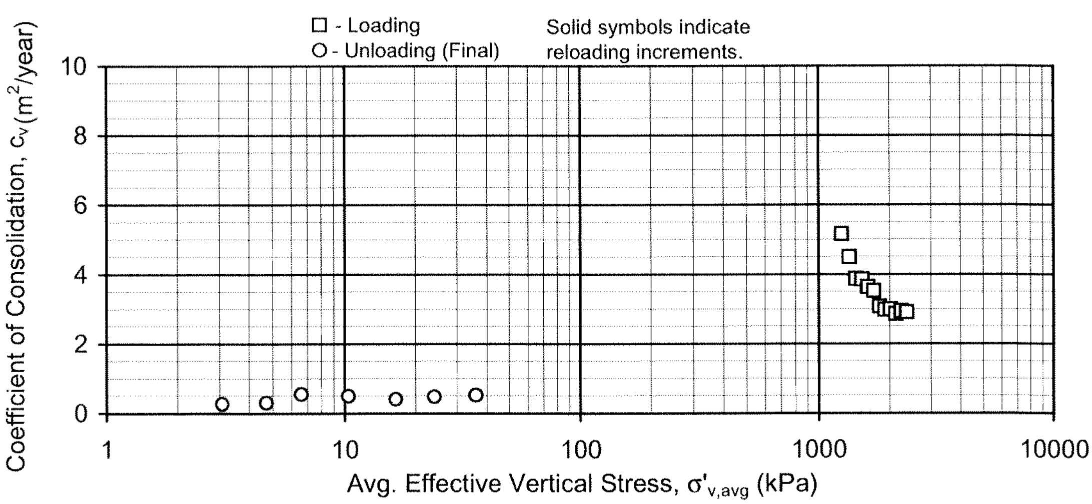




CRS CONSOLIDATION TEST RESULTS

Sample No. 51 - Depth: 52.7m
Boring 98-29
SFOBB East Span Seismic Safety Project



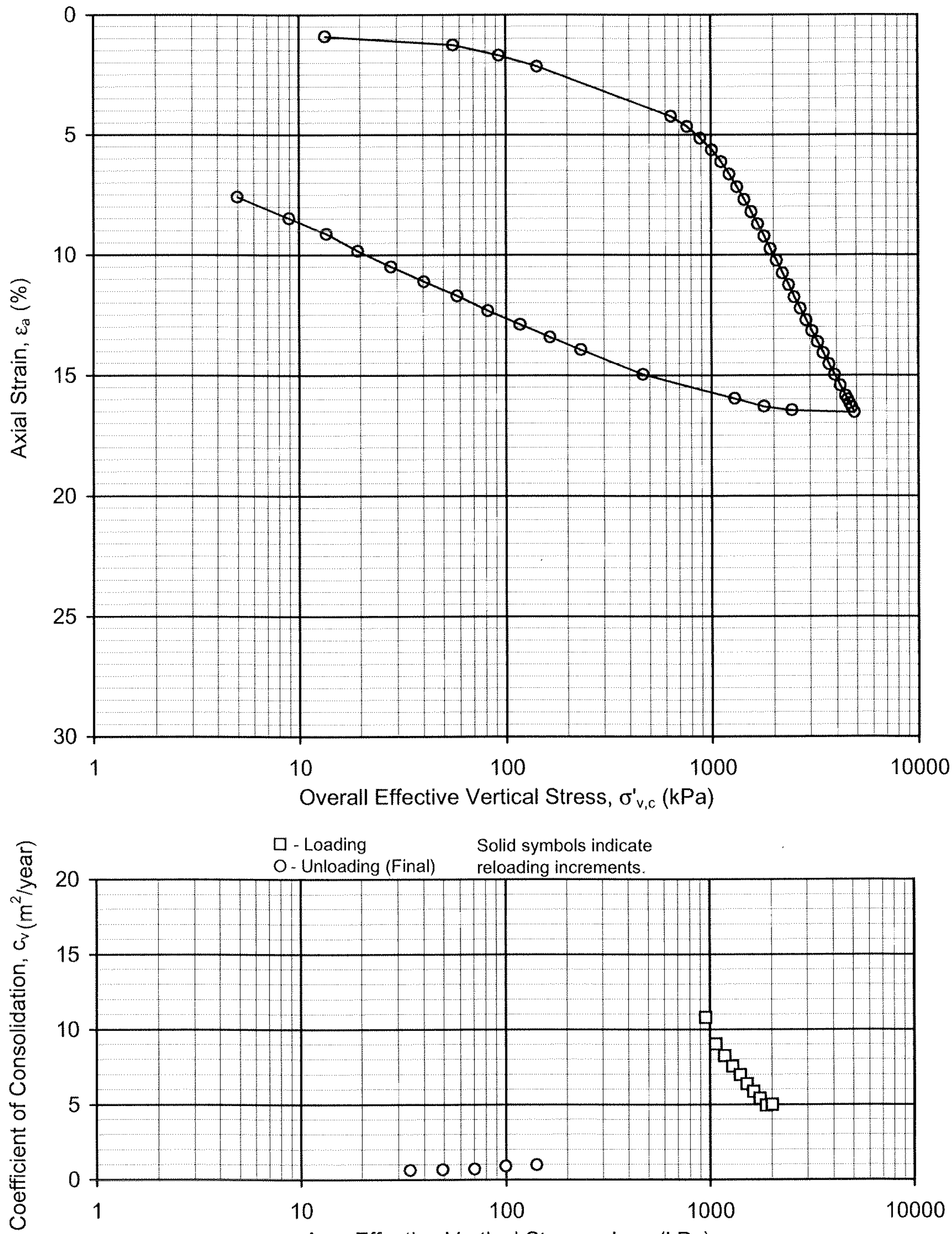


CRS CONSOLIDATION TEST RESULTS

Sample No. 57 - Depth: 71.2m

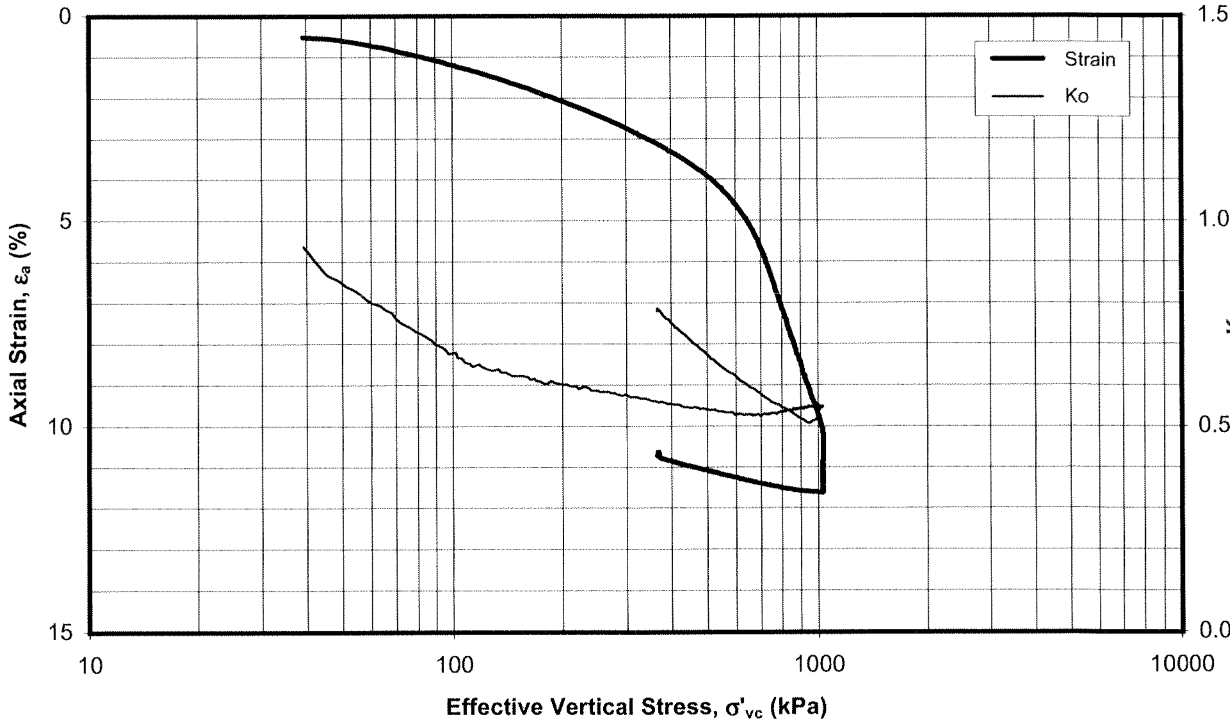
Boring 98-29

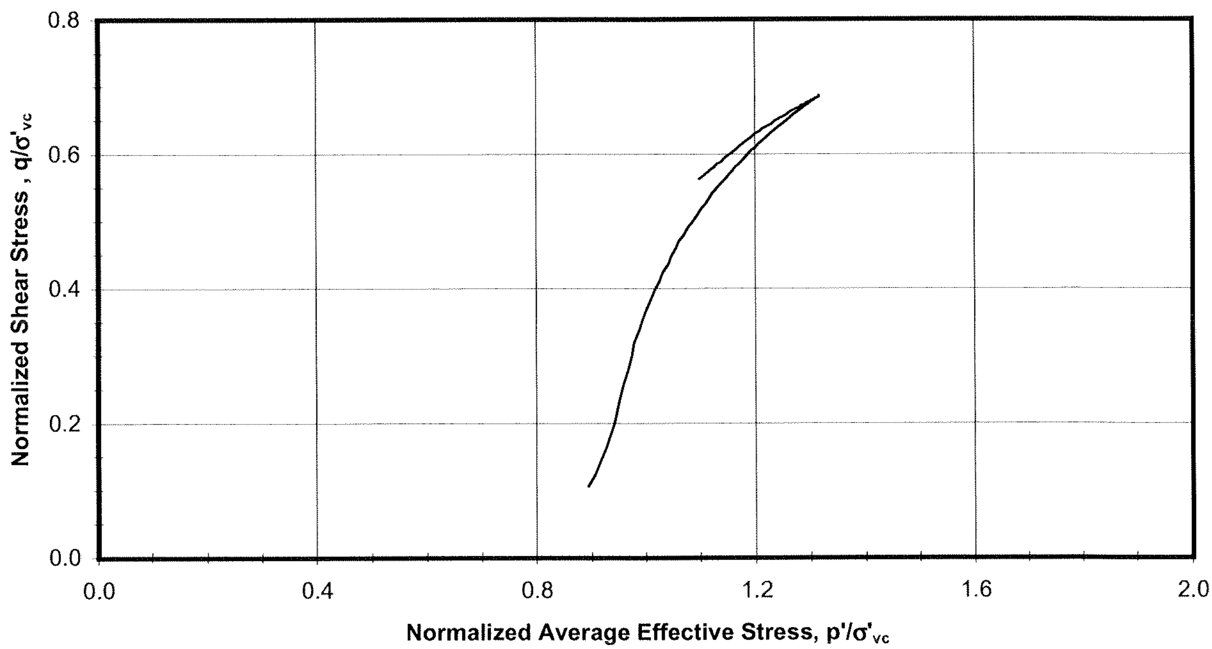
SFOBB East Span Seismic Safety Project



CRS CONSOLIDATION TEST RESULTS

Sample No. 68 - Depth: 82.2m
Boring 98-29
SFOBB East Span Seismic Safety Project



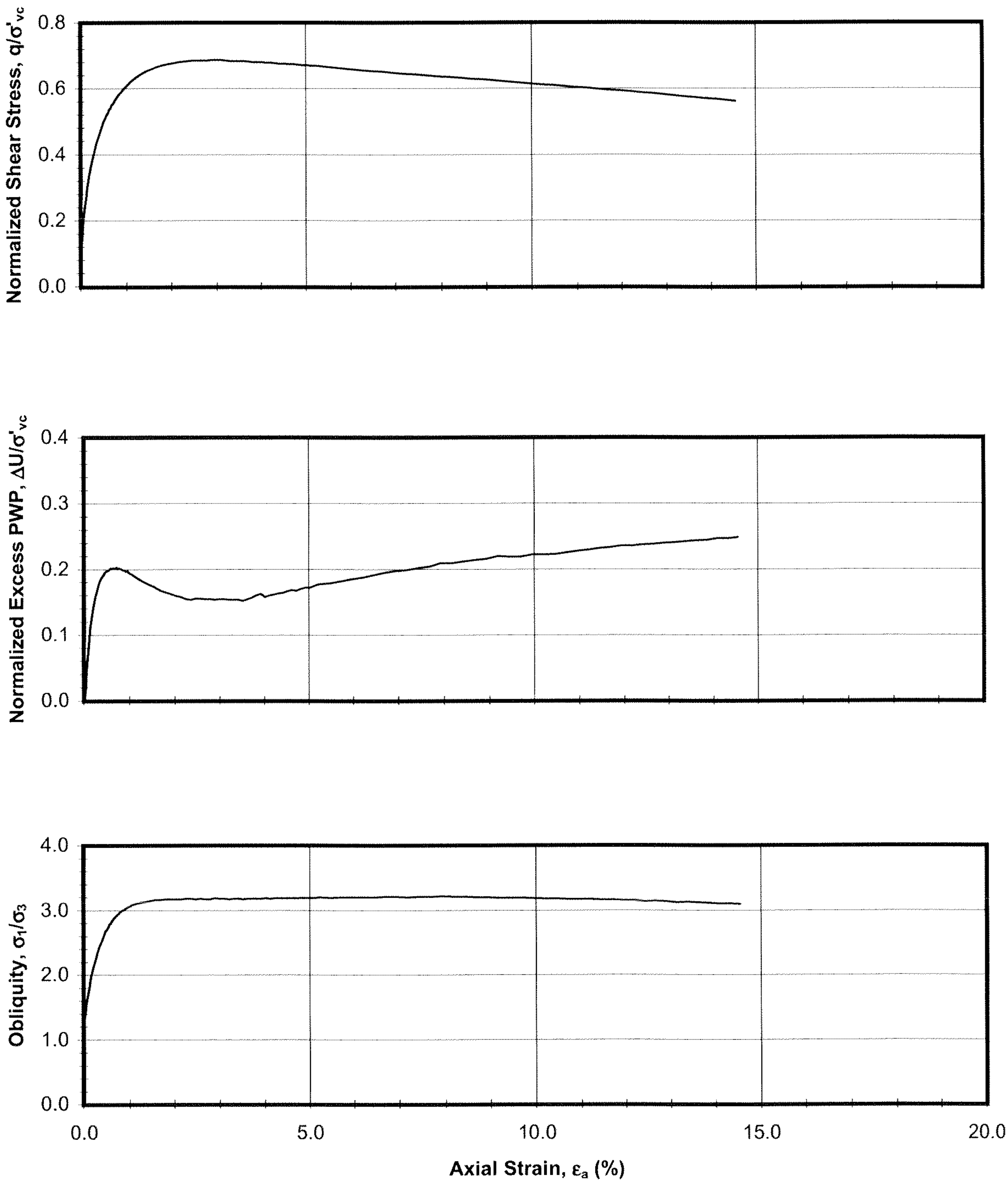


Ko CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

Sample No. 40 - Depth: 31.9m

Boring 98-29

SFOBB East Span Seismic Safety Project

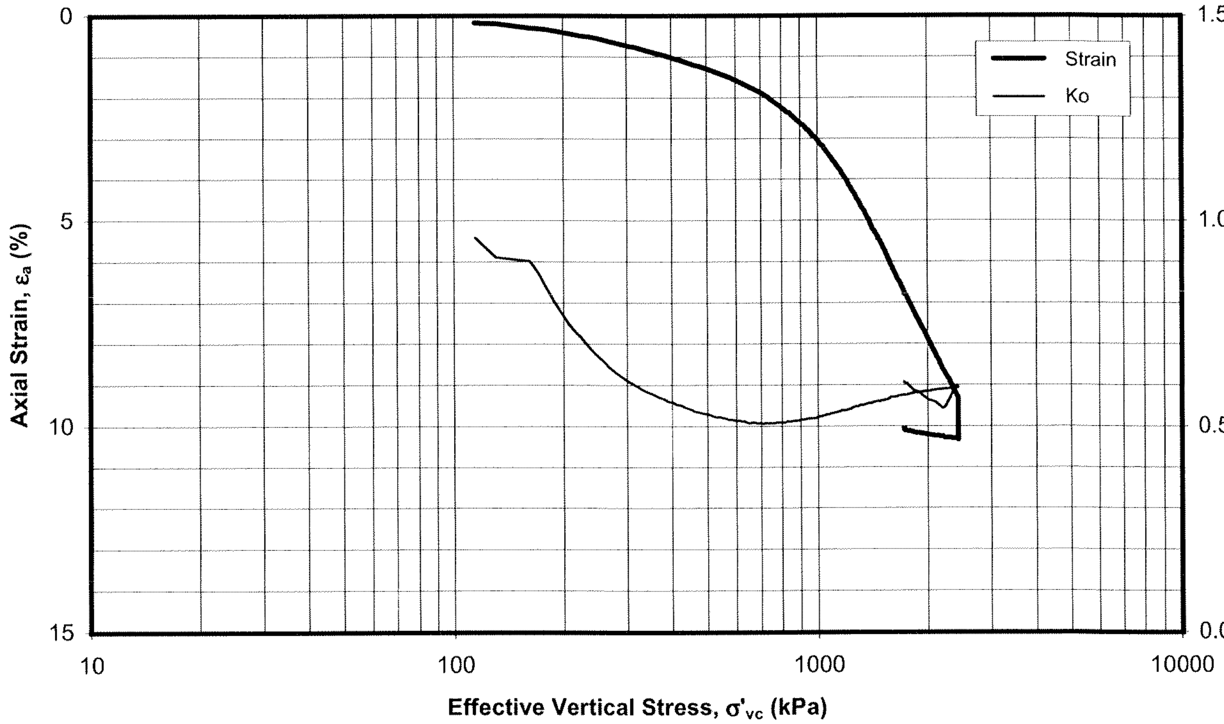


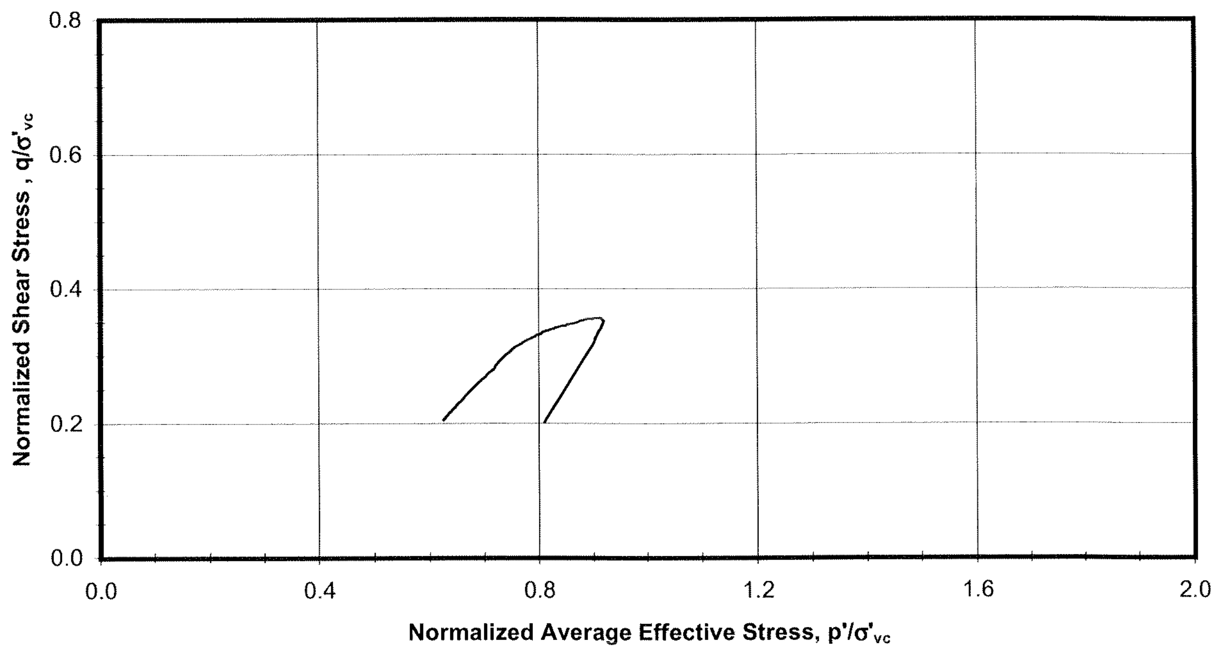
Ko CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

Sample No. 40 - Depth: 31.9m

Boring 98-29

SFOBB East Span Seismic Safety Project



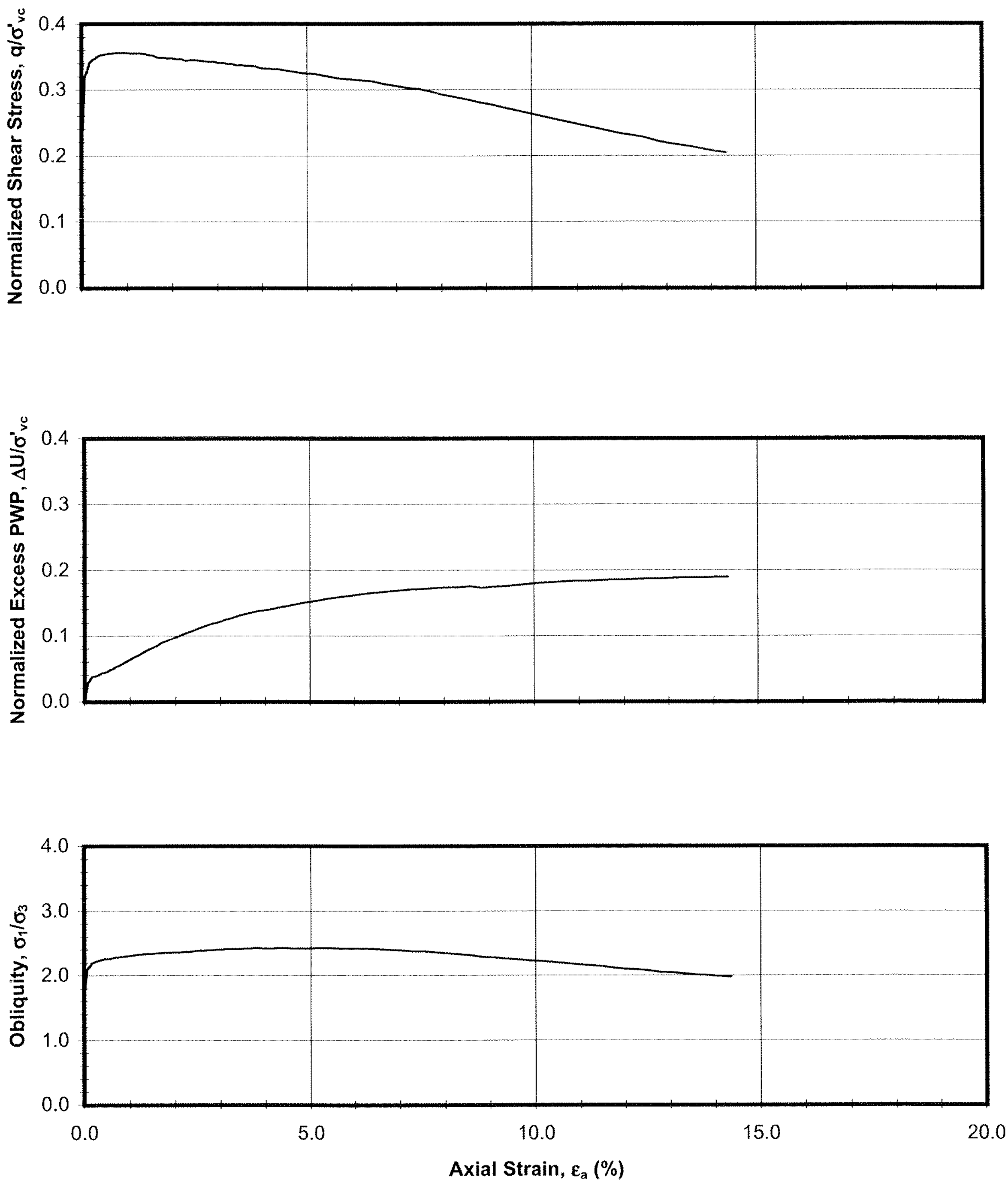


Ko CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

Sample No. 68 - Depth: 82.1m

Boring 98-29

SFOBB East Span Seismic Safety Project

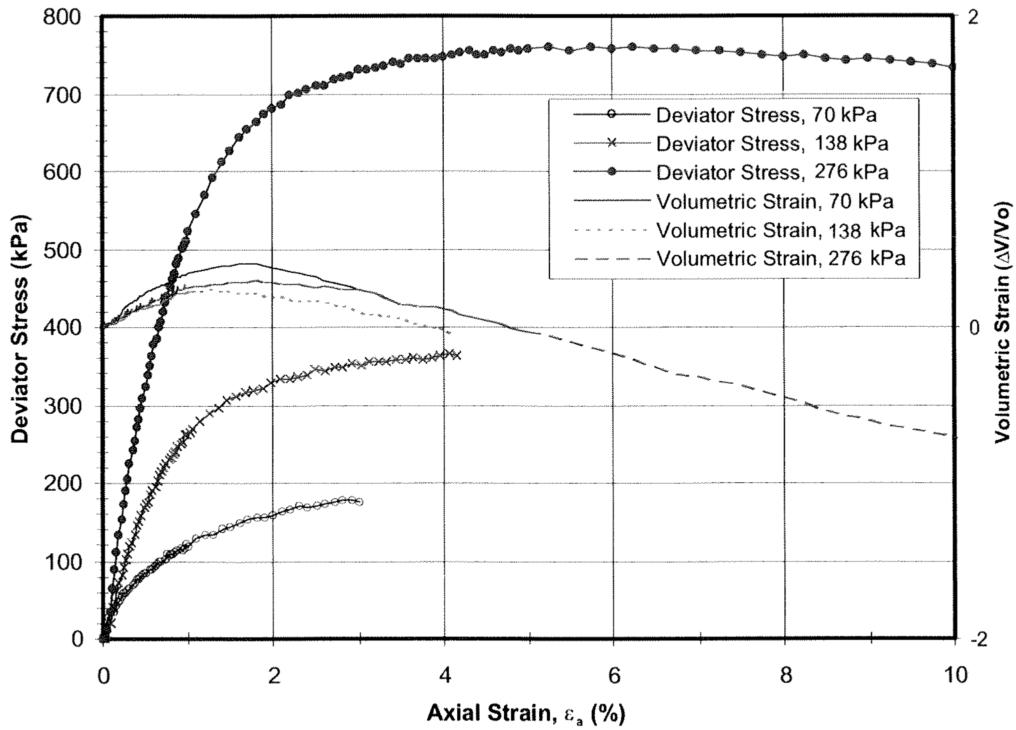


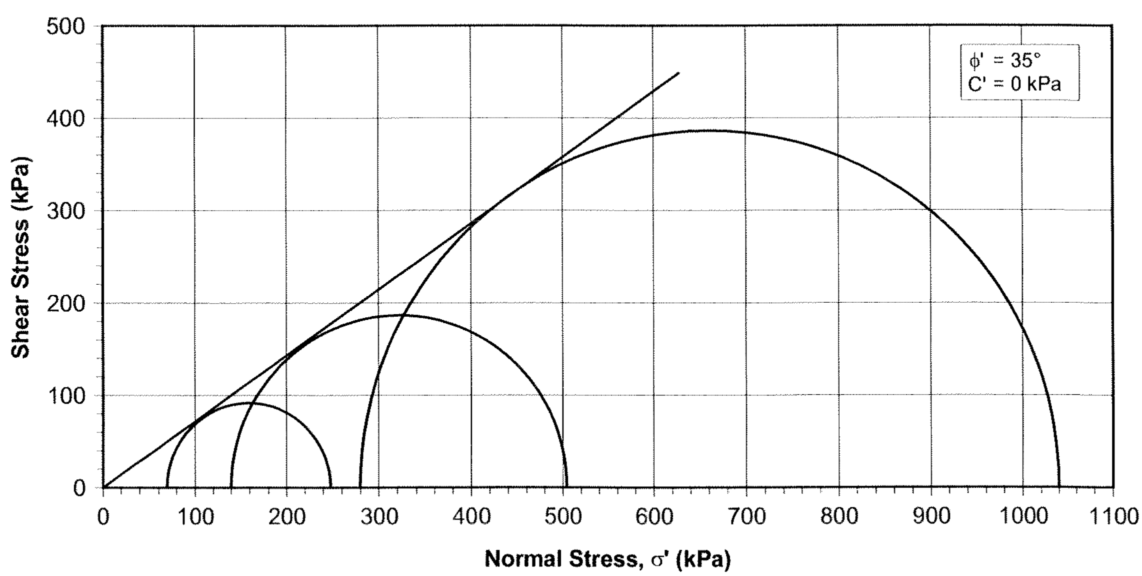
Ko CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

Sample No. 68 - Depth: 82.1m

Boring 98-29

SFOBB East Span Seismic Safety Project





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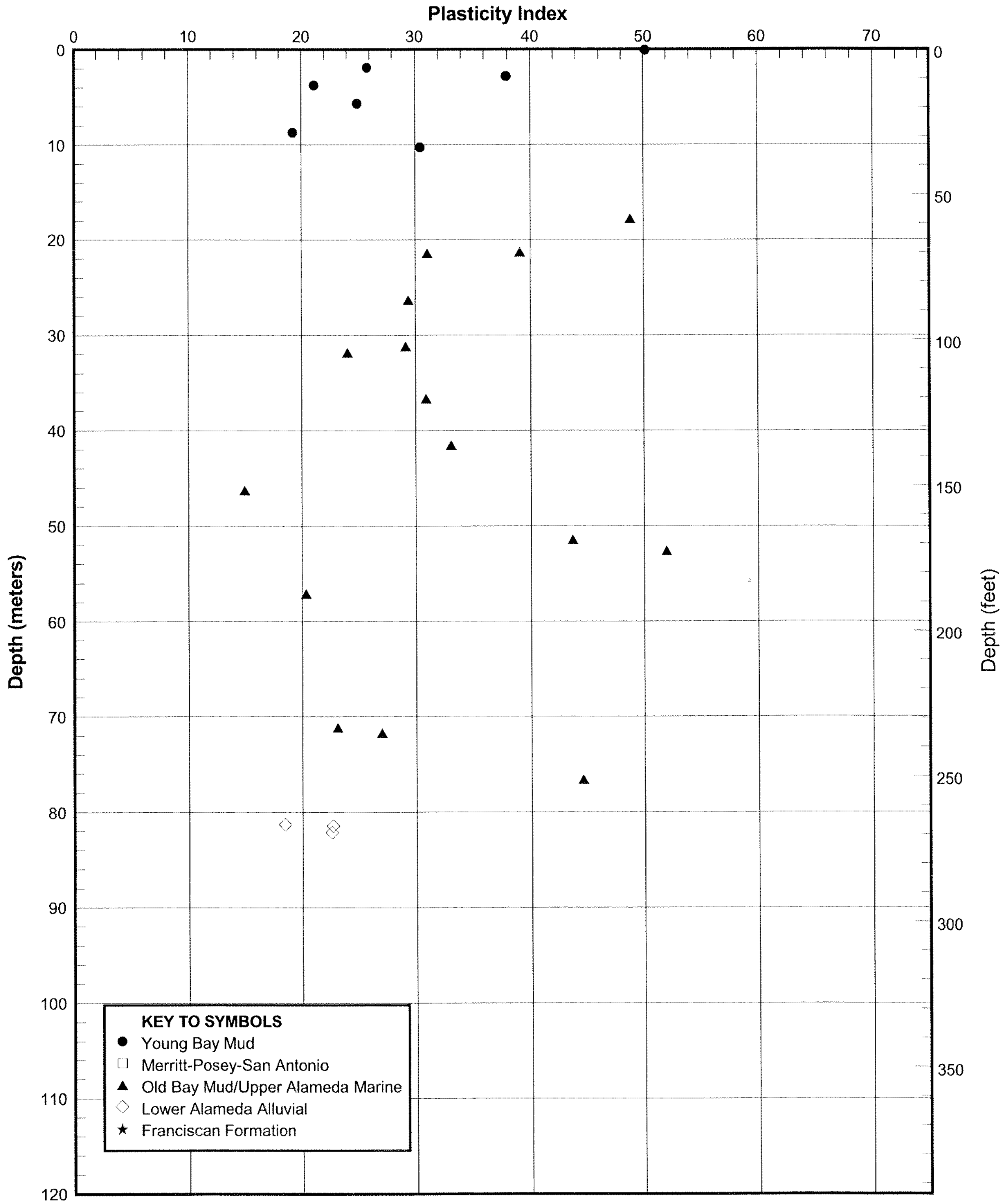
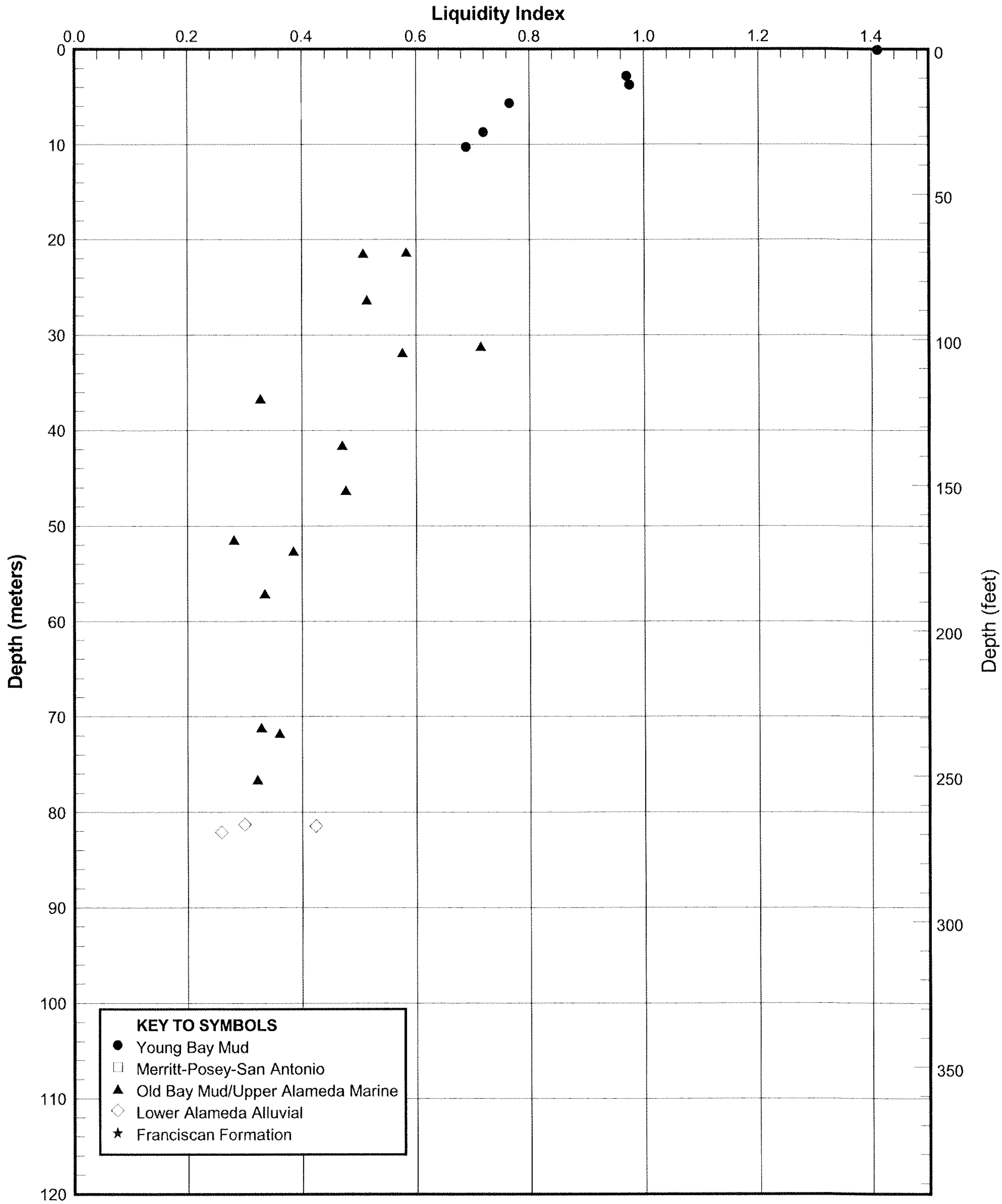
ISOTROPICALLY CONSOLIDATED-DRAINED TRIAXIAL COMPRESSION TEST

Sample 21 - Depth: 9.5m
Boring 98-29
SFOBB East Span Seismic Safety Project



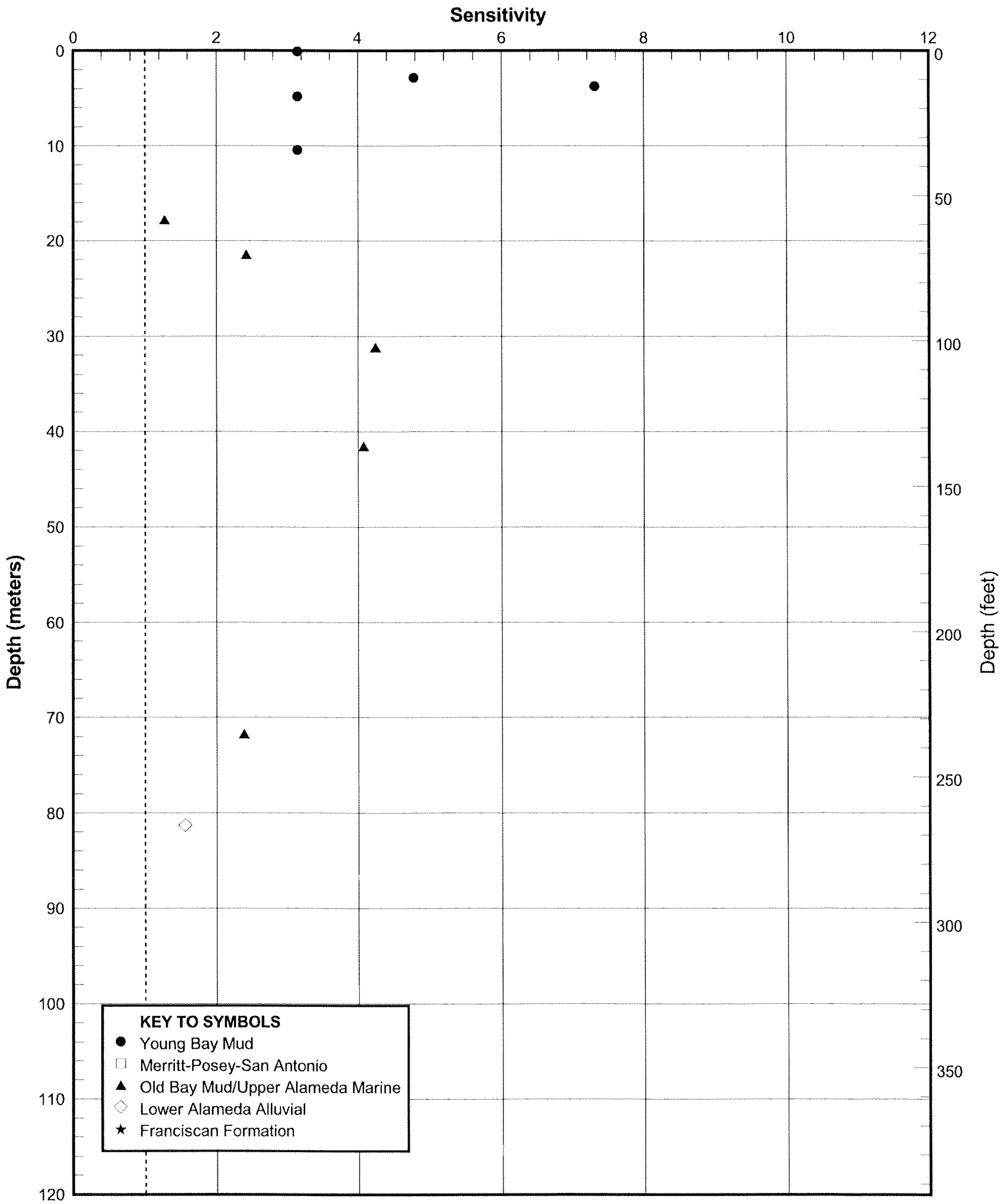
PLASTICITY INDEX PROFILE
Boring 98-29
SFOBB East Span Seismic Safety Project

PLATE 98-29.17

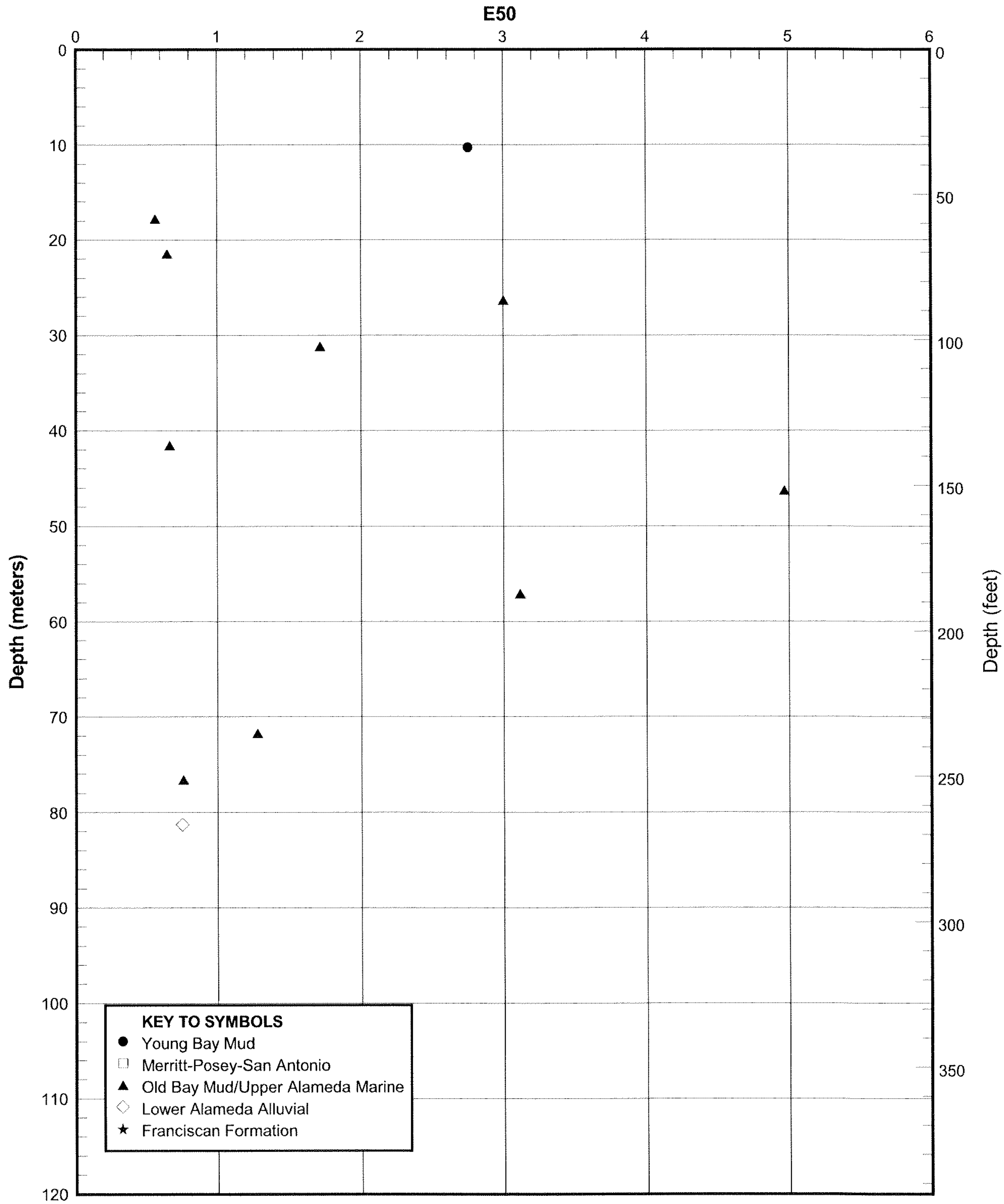


LIQUIDITY INDEX PROFILE
Boring 98-29
SFOBB East Span Seismic Safety Project

PLATE 98-29.18

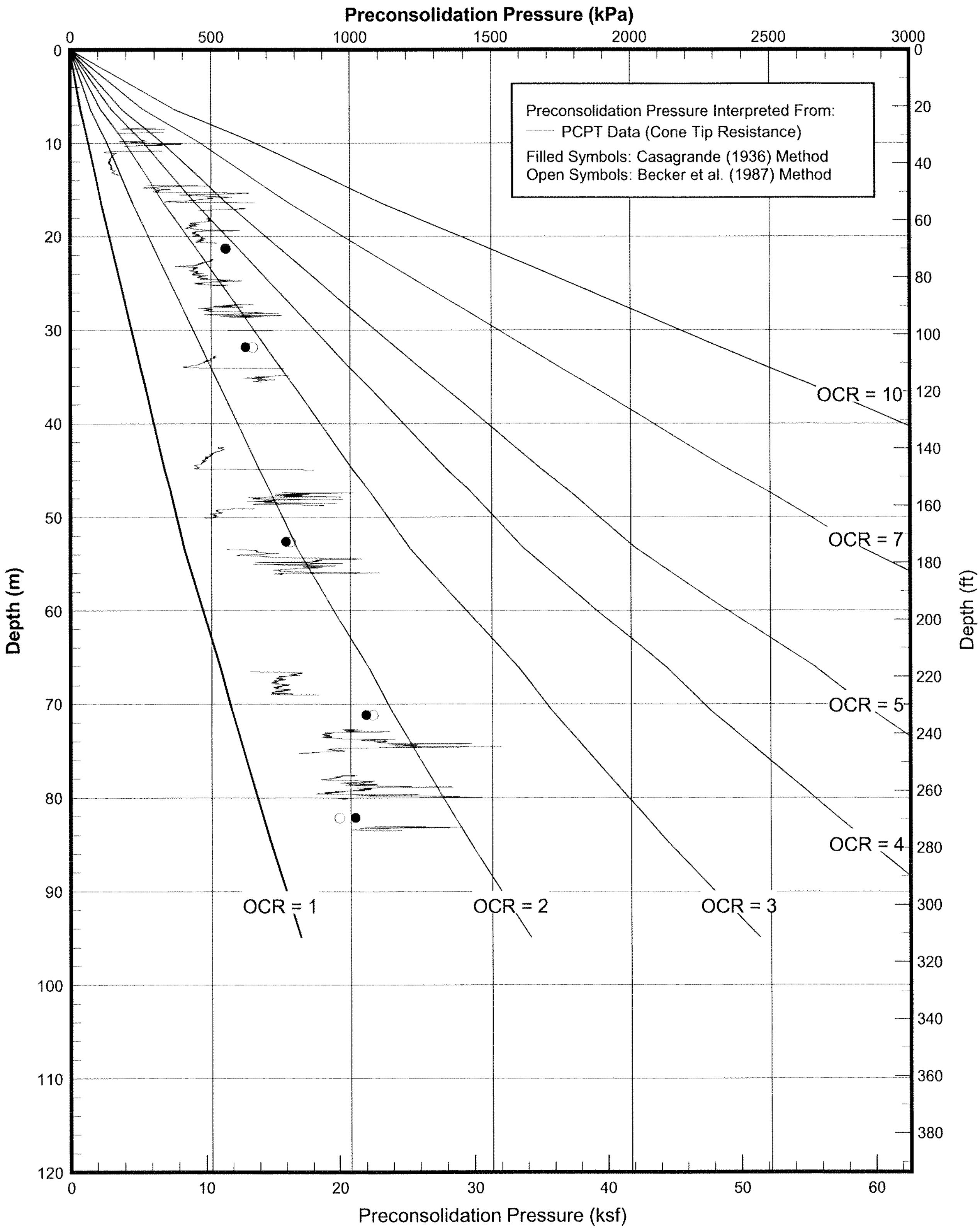


SENSITIVITY PROFILE
Boring 98-29
SFOBB East Span Seismic Safety Project



E50 PROFILE
Boring 98-29
SFOBB East Span Seismic Safety Project

PLATE 98-29.20

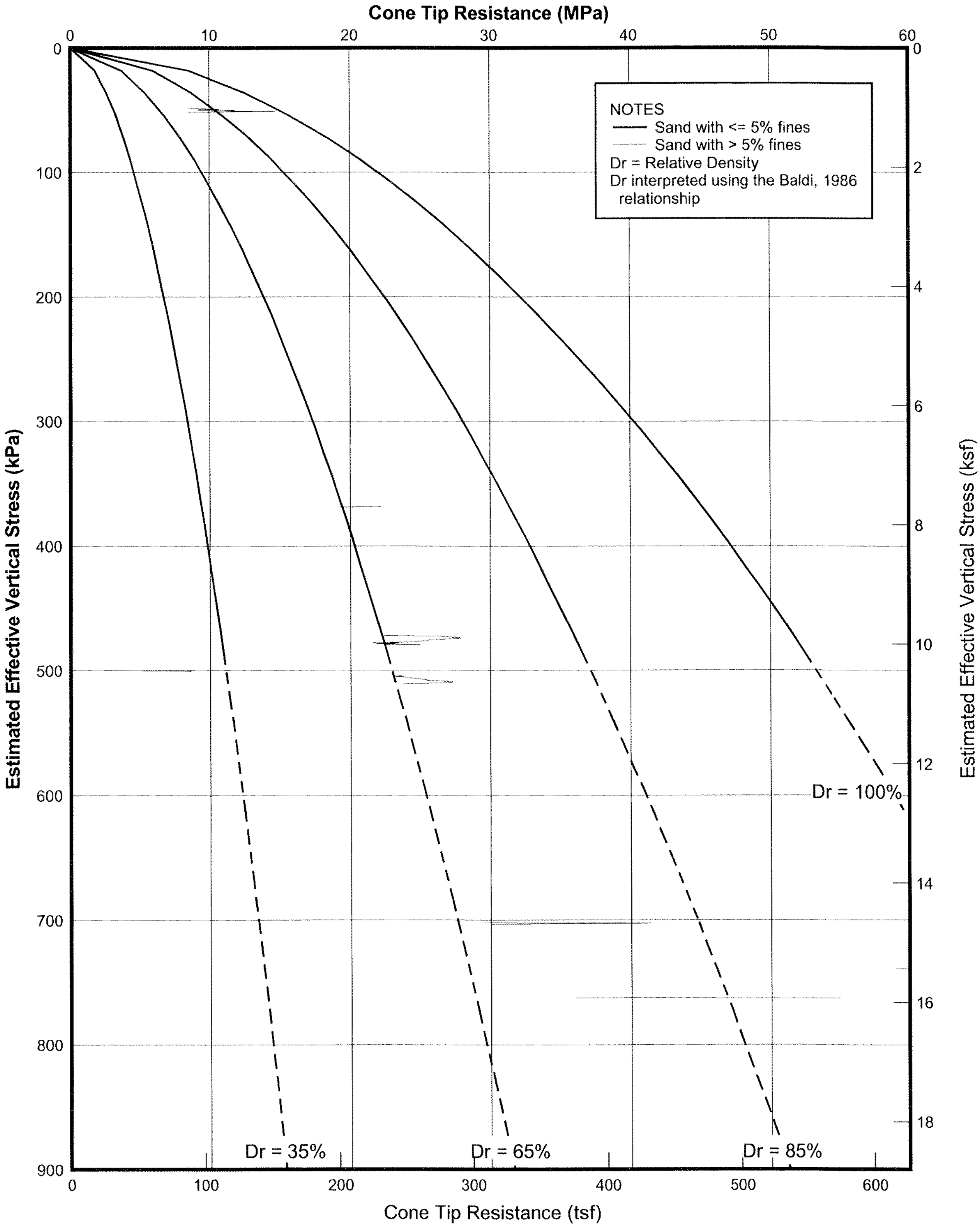


PRECONSOLIDATION PRESSURE INTERPRETED FROM CPT DATA

Boring 98-29

SFOBB East Span Seismic Safety Project

PLATE 98-29.21



RELATIVE DENSITY INTERPRETED FROM CPT DATA

Boring 98-29

SFOBB East Span Seismic Safety Project

EXHIBIT 16

SMITH & MONROE & GRAY ENGINEERS, INC.

CIVIL & STRUCTURAL ENGINEERS
MATERIAL HANDLING & MECHANICAL

CLIENT KFM
PROJECT SFOBB
PILE TMR's
BY AJMM DATE 6-24-02 REV.
JOB NO. 02-030A SHEET C2 OF

SPUD PILES - (60' FRAME TO FIXITY (40' IN MUD))

42"Ø x 1/2" WALL - A = 65.2 in², S = 669 in³
(GRADE 60)

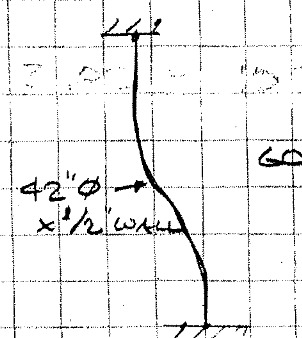
M_{MAX} + COMP. = $(822^2 + 49^2)^{1/2}$ = 823'K , 441 K

$D/t = \frac{42}{.5} = 84 < \frac{3300}{F_y} = 55$ ∴ USE EQN A-B5-9

$F_b = .60 F_y = 36$ ksi

$r = \frac{(42^2 + 41^2)^{1/2}}{4} = 14.67$ in

$\frac{Kl}{r} = \frac{1.2(60)(12)}{14.67} = 59$



F_a = 16.8 ksi OR $\frac{662}{84} + .4F_y = 31.9$ ksi ; $F'_{ex} = 42.9$ ksi
API (IDEAS BRL'G)

COMB. STRESSES -

$$\frac{f_a}{F_a} + \frac{f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} = \frac{\overset{6.74 \text{ ksi}}{441/65.2}}{16.8} + \frac{823(12)/669}{36\left(1 - \frac{6.76}{42.9}\right)}$$

$= .40 + .49 = .89 < 1.0$
∴ OK

SFOBB EAST SPAN SEISMIC SAFETY PROJECT

BR NO	DIST	CTY	RTE	KM POST, TTL, PROJ	CONT NO
34-006L/R	04	SF,ALA	80	13.9/14.3/01.6	04-12024

SPUD PILES (CONT)

CHECK ELASTIC & INELASTIC BUCKLING

PER A.P.I. 2A-WSD (7-1-93) SECT. 3.2.2.

F_a = AISC CHAPTER E, EXC. SUBST. F_{xe} OR

F_{xc} (BUCKLING STRESSES) FOR F_y IN EQN.'s

E2-1.

FOR 42"Ø x ½" WALL PILE, L = 60', K = 1.2

r = 14.67 in

$D/t = 42/.5 = \underline{84}$

$\boxed{\frac{Kl}{r} = 59}$

$F_{xe} = 2CE\frac{t}{D} = 2(.3)(29{,}000)\frac{1}{84} = 207$ KSI
(ELASTIC BUCKLING)

$F_{xc} = F_y\left[1.64 - .23(D/t)^{1/4}\right] = 36[1.64 - .69] = \underline{34 \text{ KSI}}$ ← USE
(INELAS. BK'G)

$\therefore \; C_c = \left[\frac{2\pi^2 E}{34}\right]^{1/2} = 129.8 \approx \underline{\underline{130}}$

$$\& \quad F_a = \frac{\left[1 - \frac{59^2}{2(130)^2}\right]F_{CE}}{\frac{5}{3} + \frac{3(59)}{8(130)} - \frac{(59)^3}{8(130)^3}} = \frac{.9(34)}{1.667 + .170 - .012}$$

$= \underline{\underline{16.76 \text{ KSI}}}$

($\approx$ 96% OF F_a AISC

$\therefore$ OK

SPUD PILES (CONT)

$42\phi \times 1/2$ - $F_y = 60 \text{ ksi}$

FIND $F_b \rightarrow$ @ $\frac{D}{t} = 84$

$\therefore \quad F_b = \left[.72 - .58 \frac{F_y}{E} \frac{D}{t}\right] F_y$

$= [.72 - .10]\, 60 = .62 F_y = 36 \text{ ksi}$

$\approx F_{b\,AISC}$

$\therefore$ OK