E2 / T1 PROJECT
KIEWIT / FCI / MANSON, A JV

April 05, 2006

Serial Letter: KFM-LET-000148

California Department of Transportation
SFOBB – E2T1 Project
333 Burma Road
Oakland, CA 94607

Attention: Pedro Sanchez

Reference: SAS E2/T1 Foundation Project
Caltrans Contract No 04-0120E4
KFM Job No. 364/4347
KFM Letter # 143
State Letters #05.003.01-000869 and 05.003.01-000964

Subject: Pier T1 CIDH Construction - Differing Site Condition Documentation

Dear Pedro:

On March 1, 2006, KFM notified the State that soil conditions encountered during the construction of the CIDH pile at pile #T1-1 were not as expected.

Based upon the frictional capacity of the soil as described in the contract boring logs and the subsequent test pile program, KFM's construction methods were developed with the engineering assumption that the Temporary Casings would be capable of supporting the 240 ton vertical load (casing and drill).

However, as the 2.75m diameter drilling continued 400mm past the temporary casing pile tip, the temporary casing continued to advance 200mm more under the vertical load. This was unexpected, considering the engineering analysis done using the attached documents confirmed that the soil conditions appeared adequate to sustain this 240 ton load. Note, the underreaming operation had not been commenced.

Please consider the facts and expert opinions presented herein when making your determination as to the validity of the Differing Site Conditions assertion.

Attached, please find:

1) Map of Pier T1 with pertinent Boring Locations shown
2) Boring Log #98-21, 98-22, 98-23 and 98-24
3) Results from KFM's Test Pile program conducted on 8/25 -27/04
4) ABE Engineering's Dynamic Pile Test Report dated 9/29/04 for Pile T1-9
5) Vertical Load calculation

6) GeoDrive Technology's Memo dated 2/3/06 re: Bearing Capacity of 3m Casing
7) InSituTech's Memo dated 2/7/06 re: Dynamic Monitoring Results for T1 Temp Casings
8) GeoEngineers' Memo dated 2/8/06 re: T1 Temporary Casing Friction Capacity
9) GeoEngineers' Memo dated 3/15/06 re: T1 Temporary Casing Friction Capacity
10) GeoDrive Technology's Letter dated 4/4/06 re: Bearing Capacity of 3m Casing
11) As-Built Locations of T1 Temporary Casing Locations relative to Soil Types

At this time KFM will continue to monitor the drilling and soil conditions at all pile locations of the Pier T1 footing. Additional information must be gathered during the remaining drilling operations to understand the soil conditions as a whole before making a final analysis. Therefore, KFM reserves its rights to further define the magnitude of the Differing Site Conditions at Pier T1.

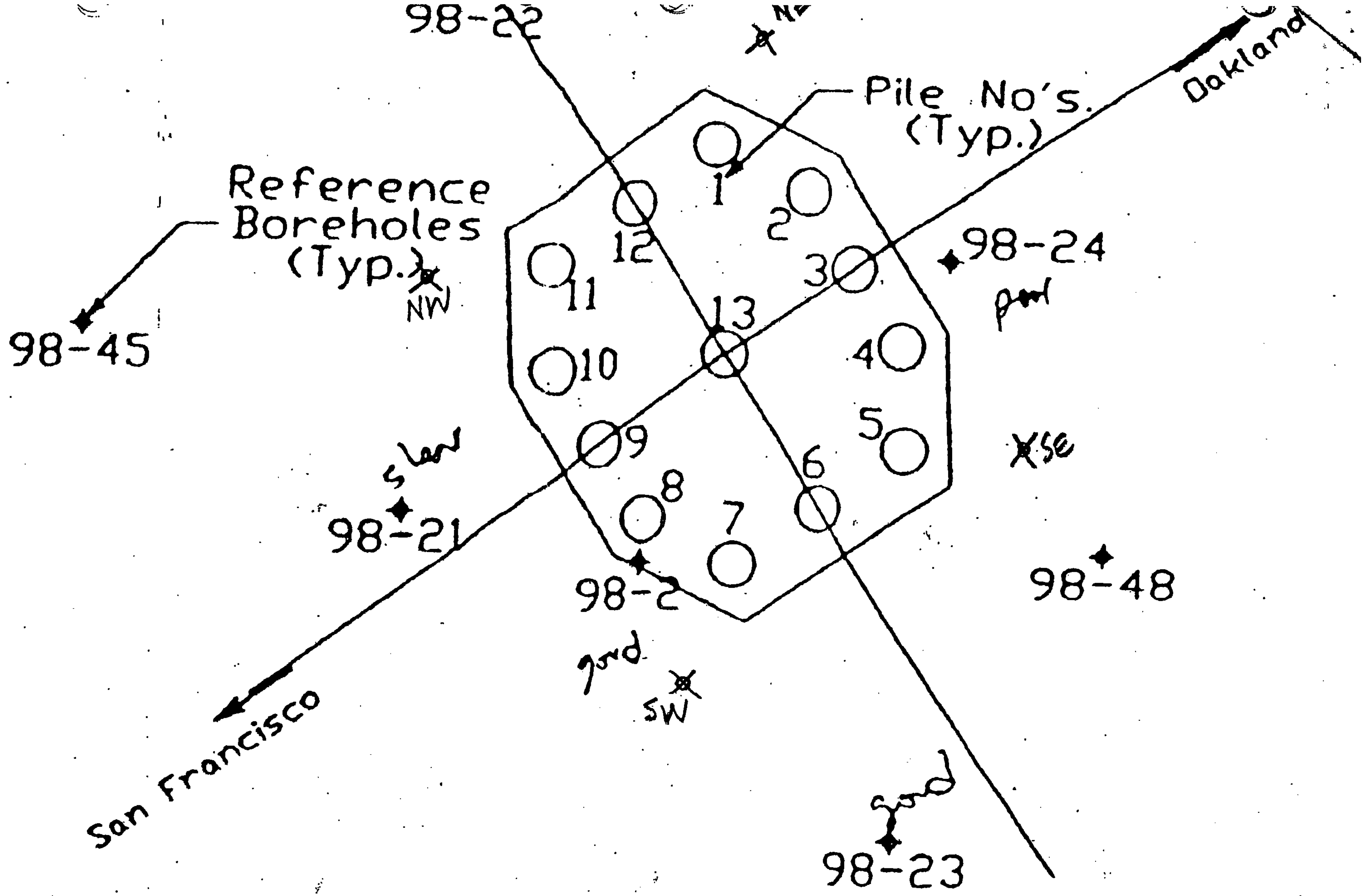
Furthermore, conditions at pile #T1-3, elevation -30 seem to indicate that the 'hard rock' layer has not yet been encountered as described in the contract documents. KFM requests that the State discuss the actual conditions at pile #T1-3, as observed by your Field Representative, and make an independent evaluation of the actual soil conditions encountered.

Sincerely,
KIEWIT/FCI/MANSON, a JV

for

Christopher J. Villa
Deputy Project Director

cc: file



Plan — Tower T1 Pile Cap

Scale 1:200

C1

SOIL AND ROCK TYPES

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

SAMPLERS

76mm-OD, 72mm-ID Thin Walled Tube	76mm-OD, 60mm-ID Modified California Liner
57mm-OD, 54mm-ID* Thin Walled Driven Tube	51mm-OD, 35mm-ID SPT Split Spoon Sampler
63.5mm-OD, 54mm-ID Offshore Liner	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 (kN/m³)
- THEORETICAL SUBMERGED UNIT WEIGHT (kN/m³)
- PLASTIC LIMIT __ LIQUID LIMIT
- EQUIVALENT SPT BLOW COUNT
- ROCK QUALITY DESIGNATION (RQD)
- ROCK RECOVERY PERCENT
- ROCK CORING RATE

STRENGTH TESTS

- POCKET PENETROMETER
- TORVANE
- REMOTE VANE
- MINIATURE VANE (RESIDUAL VANE)
- UNCONSOLIDATED UNDRAINED TRIAXIAL
- SWEDISH FALL CONE
- UNCONFINED COMPRESSION (SOIL)
- (Open symbols indicate remolded tests)
- Soil: Strength Exceeds Capacity of Measuring Device; Rock: Sample broke along discontinuity; Intact sample would have greater strength
- Shear Strength Interpreted from CPT Tip Resistance (Nk=12-15)
- POINT LOAD TEST (ROCK-INTACT SPECIMEN)
- POINT LOAD TEST (ROCK-ALONG WEAK PLANE)
- UNCONFINED COMPRESSION (ROCK)
- FRACTURE DENSITY (ROCK)

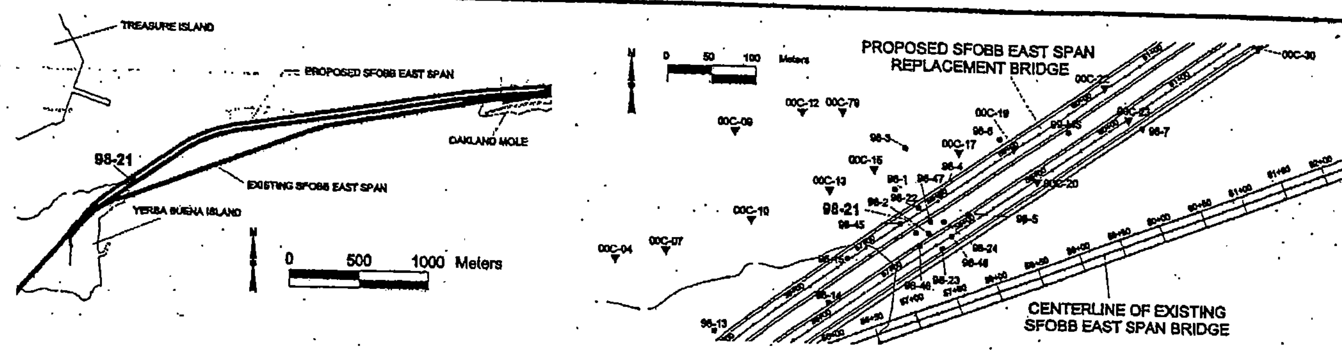
STRENGTH OF COHESIVE SOILS

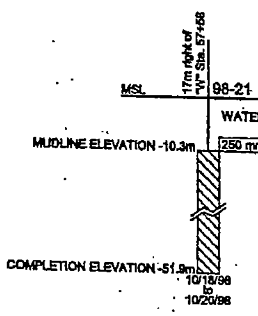
Consistency	Undrained Shear Strength kPa
Very Soft	less than 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	100 to 200
Hard	greater than 200

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.





Metric

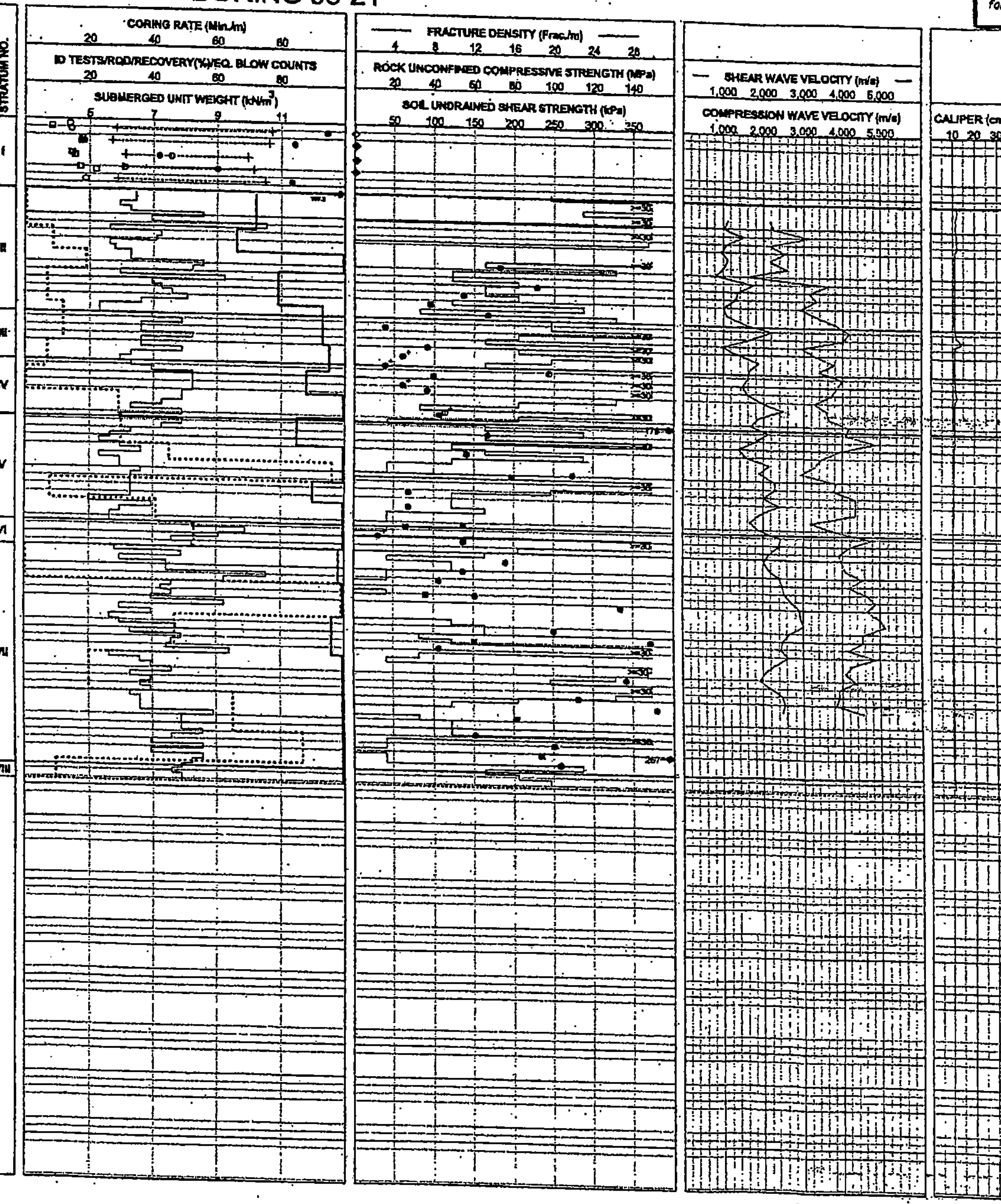
BORING 98-21

Coordinates: E1836360 N647585
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -10.3m (MSL)

BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
WOH, WOH, WOH, WOH, WOH, 50/5cm, 50/5cm	FAT CLAY (CH), dark olive gray to dark gray, very soft -subangular, moderate yellowish brown, sandstone gravel, below 4.3m (4.4m)	I
72-0, 66-9, 100-19, 79-7, 83-12	SANDSTONE (Graywacke) (Rx) with interbedded SILTSTONE (Rx), [10:1], moderate to dark yellowish brown, intensely to moderately weathered, moderately soft, poorly indurated, fine to coarse grained, moderately fractured -moderately weathered, subrounded to rounded coarse sandstone/conglomerate bed, at 6.8m -claystone interbed, slightly weathered, 9.4m to 9.8m -slightly weathered, moderately to intensely fractured sandstone, below 9.8m (12.2m)	II
95-7	SANDSTONE (Graywacke) (Rx), interbedded with SILTSTONE (Rx), [5:1], olive gray, slightly weathered, hard, intensely fractured to very intensely fractured (15.2m)	III
88-0, 100-29, 85-30, 100-45, 100-96	SILTSTONE (Rx), dark gray, fresh, hard, thinly bedded, very intensely to intensely fractured with clean, smooth to slightly rough fracture surfaces -very intensely sheared with interbedded sandstone, 17.1m to 17.5m -fractures with 25mm to 50mm spacing, dipping 30 to 40 degrees, with smooth, clean surfaces, 17.5m to 18.1m -sandstone, medium dark gray, fresh, hard, well indurated, moderately bedded, moderately fractured, 18.0m to 18.5m (18.8m)	IV
90-8, 100-41, 100-53, 98-0, 100-99	SANDSTONE (Graywacke) (Rx) with interbedded SILTSTONE (Rx), [10:1], moderate to dark yellowish brown, intensely to moderately weathered, moderately soft, poorly indurated, fine to coarse grained, moderately fractured (25.3m) -siltstone interbed, dipping 30 degrees, 19.2m to 19.6m -90mm thick siltstone interbed, very intensely fractured, 22.7m to 23.1m -siltstone interbed, slightly fractured, bedding dipping 35 degrees, 23.9m to 24.1m -siltstone interbed, dipping at 35 degrees, 24.2m to 24.8m (26.9m)	V, VI
	Interbedded SANDSTONE (Graywacke) (Rx) and SILTSTONE (Rx)	
96-47, 100-20, 100-65, 100-87, 100-10	SANDSTONE (Graywacke) (Rx), medium dark gray, fresh, hard, moderately indurated, fine to coarse grained, thickly bedded, intensely to moderately fractured with slightly rough, clean to very thinly coated surfaces -sandstone with interbedded siltstone, 31.9m to 38.6m -60mm thick, very intensely fractured zone, with moderately rough, calcite/quartz filled fractures, at 33.0m -very intensely fractured zone with numerous, closely spaced, healed, 8mm thick, calcite filled fractures, 33.9m to 34.2m -very thickly bedded, moderately to slightly fractured, at 36.9m -dark gray, hard, intensely to very intensely fractured, siltstone interbed with numerous, healed, 3mm thick, calcite filled fractures, 38.7m to 39.0m (40.7m)	VII
	SILTSTONE (Rx) with interbedded SANDSTONE (Graywacke) (Rx) [3:1] -sandstone below 41.5m	VIII
	TOTAL DEPTH: 41.6m BACKFILLED WITH: Cement Grout	

Note: Refer to multi-page logs presented in Volume 2 of the Final Marine Site Characterization Report for detailed soil and rock descriptions.



DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.4,13.8	91	118

REGISTERED GEOTECHNICAL ENGINEER

10-15-03
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
T.W. McNellan
No. 0583
Exp. 12/31/03
Geotechnical
STATE OF CALIFORNIA

FUGRO-EARTH MECHANICS,
A JOINT VENTURE.
7700, Edgewater Dr., Ste. 848,
Oakland, CA 94621
(510) 562-8833, FAX (510) 562-8858

SABA MOHAN GEOTECHNICAL DESIGN OVERSIGHT	DRAWN BY	SACHIN DESHPANDE	C. PRENTICE/J. BIANCHIN FIELD INVESTIGATOR DATE SEE ABOVE
S. M.	CHECKED BY	J.CHACKO	

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

T.W.McNEILAN
PROJECT ENGINEER

ORIGINAL SCALE IN CENTIMETERS FOR REDUCED PLANS 0 1 2 3 4 5 6 7 8 9 10

CU 04
EA 0120E1

BRIDGE NO.	34-0006L/R
KM POST	13.4/13.8
POST MILE	8.3/8.5

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

SELF-ANCHORED SUSPENSION BRIDGE (E2&T1)

LOG OF TEST BORINGS NO. 11

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 073 OF

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.4,13.8	94	118

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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
- 63.5mm-OD, 54mm-ID Offshore Liner
- 76mm-OD, 60mm-ID Modified California Liner
- 51mm-OD, 36mm-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

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

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STRENGTH TESTS

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- SWEDISH FALL CONE
- UNCONFINED COMPRESSION (SOIL)
- (Open symbols indicate remolded tests)
- Soil: Strength Exceeds Capacity of Measuring Device; Rock: Sample broke along discontinuity; intact sample would have greater strength
- Shear Strength Interpreted from CPT Tip Resistance (Nk=12-16)
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- POINT LOAD TEST (ROCK-ALONG WEAK PLANE)
- UNCONFINED COMPRESSION (ROCK)
- FRACTURE DENSITY (ROCK)

STRENGTH OF COHESIVE SOILS

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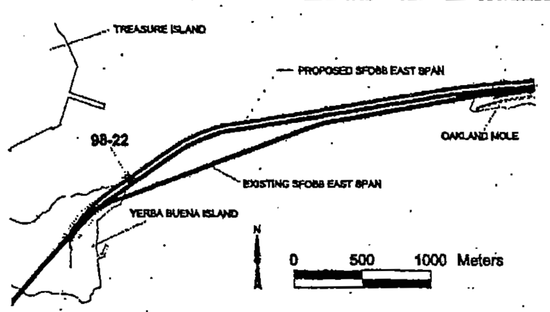
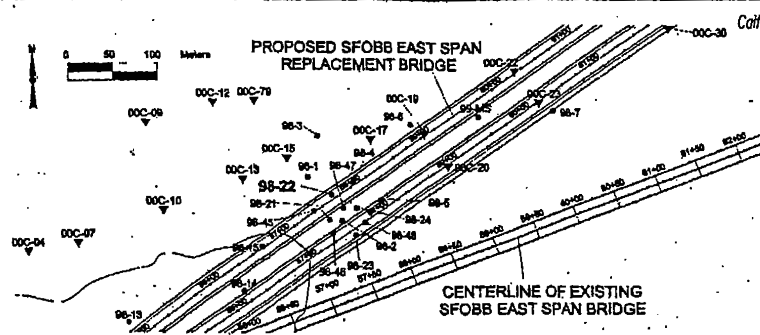
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	100 to 200
Hard	greater than 200

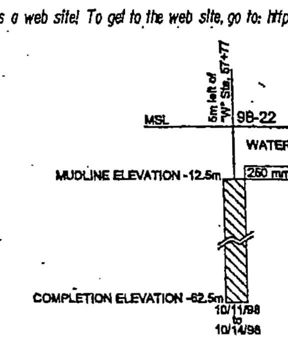
DENSITY OF GRANULAR SOILS

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 98-22

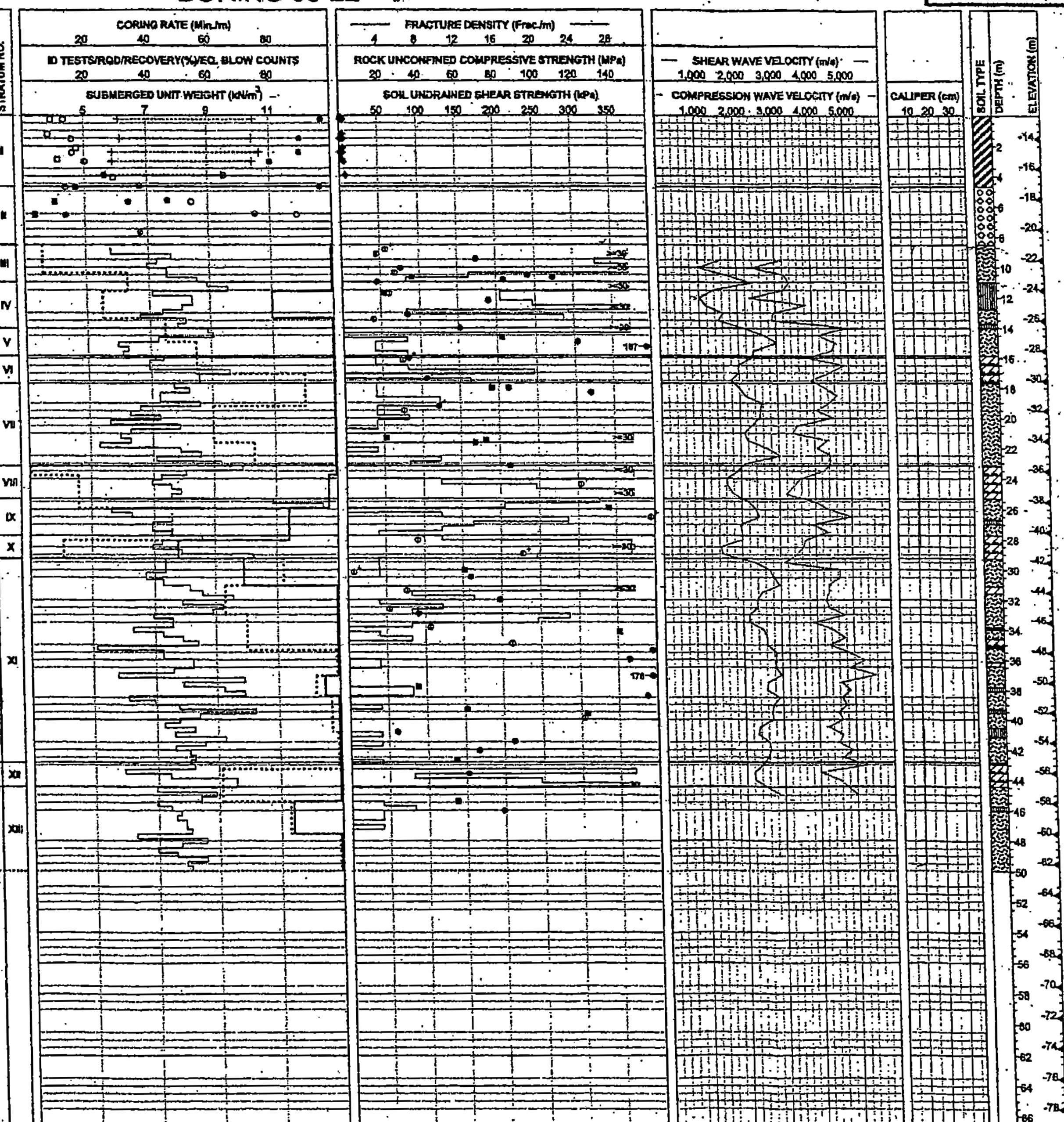
Coordinates: E1836363 N647614
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -12.5m (MSL)

BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
WOH, WOH, WOH, WOH, WOH	Fat CLAY (CH), olive gray to gray, very soft to soft (4.7m)	I
30/15cm, 50/20cm, 50/15cm, 30/15cm	ROCK FRAGMENTS with sand and fat clay, light brown -subangular gravel to cobble size sandstone rock fragments, origin uncertain, possibly intensely weathered rock (8.5m)	II
100-6	SANDSTONE (Graywacke) (Rx), light yellowish brown, intensely weathered, moderately soft to moderately hard, intensely fractured (11.0m)	III
100-33, 80-25, 100-45	SILTSTONE (Rx) with interbedded SANDSTONE (Rx) [4:1], moderately weathered, dark gray, moderately hard, thinly bedded, moderately to intensely fractured -sandstone, 12.8m to 13.7m (14.0m)	IV
100-55	SANDSTONE (Graywacke) (Rx), medium dark gray, slightly weathered to fresh, hard, thickly bedded, slightly fractured (15.8m)	V
100-90	SILTSTONE (Rx) with interbedded CLAYSTONE (Rx) and SANDSTONE (Graywacke) (Rx) [2:1:1], slightly weathered to fresh (17.7m)	VI
100-60, 100-73	SANDSTONE (Graywacke) (Rx) with minor thinly interbedded SILTSTONE (Rx) [10:1], medium gray, fresh, hard, thinly to thickly bedded, slightly to moderately fractured (23.2m)	VII
100-0, 97-16	SILTSTONE (Rx) with interbedded SANDSTONE (Graywacke) (Rx) [2:1], fresh, medium dark gray, moderately hard to hard, intensely fractured (25.3m)	VIII
84-46	SANDSTONE (Graywacke) (Rx) fresh, medium dark gray, hard, moderately fractured (27.7m)	IX
48-11	-siltstone, dark gray, hard, very thinly to thinly bedded, moderately to intensely fractured, 26.7m to 27.0m	X
69-82, 100-63	CLAYSTONE (Rx) with interbedded SANDSTONE (Graywacke) (Rx) [2:1], dark gray to black, fresh, hard, thinly bedded, slightly to intensely fractured (29.3m)	
100-70, 100-99, 95-92, 100-97, 100-99	SANDSTONE (Graywacke) (Rx) with minor thinly interbedded SILTSTONE (Rx) [10:1], medium gray, fresh, hard, thinly to thickly bedded, slightly to moderately fractured -interbedded sandstone and siltstone, 31.1m to 31.5m -siltstone, 33.8m to 34.0m -interbedded sandstone and siltstone, 34.5m to 35.2m -interbedded sandstone and siltstone, 36.1m to 36.4m -siltstone, 37.8m to 38.1m -dark gray siltstone, 39.3m to 39.6m -dark gray siltstone 40.7m to 40.8m -undulating siltstone bed dipping 30 degrees, at 41.1m (42.6m)	XI
100-61	SANDSTONE (Graywacke) (Rx) interbedded with SILTSTONE (Rx) [2:1], medium dark gray, hard, fresh, very slightly fractured (44.4m)	XII
84-83, 100-99	SANDSTONE (Graywacke) (Rx) with minor thinly bedded SILTSTONE (Rx) [10:1], medium dark gray, fresh, hard, medium to coarse grained, very thickly bedded, slightly fractured -siltstone dipping 32 degrees, 45.7m to 46.0m	XIII

TOTAL DEPTH: 50.0m BACKFILLED WITH: Cement Grout

Note: Refer to multi-page logs presented in Volume 2 of the Final Marine Site Characterization Report for detailed soil and rock descriptions.



SABA MOHAN GEOTECHNICAL DESIGN OVERSIGHT	DRAWN BY	SACHIN DESHPANDE	J.CHACKO/M.ZUEGER FIELD INVESTIGATOR	T.W.McNEILAN PROJECT ENGINEER
	CHECKED BY	J.CHACKO	DATE SEE ABOVE	

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

ORIGINAL SCALE IN CENTIMETERS FOR REDUCED PLANS 0 1 2 3 4 5 6 7 8 9 10

CU 04
EA 0120E1

BRIDGE NO.	KM POST	POST MILE
34-0006L/R	13.4/13.8	8.3/8.5

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

SELF-ANCHORED SUSPENSION BRIDGE (E2&T1)

LOG OF TEST BORINGS NO. 14

DISREGARD PRINTS BEARING EARLIER REVISION DATES → REVISION DATES (PRELIMINARY STAGE ONLY) SHEET OF

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.4,13.8	95	118

REGISTERED GEOTECHNICAL ENGINEER

10-15-03
PLANS APPROVAL DATE

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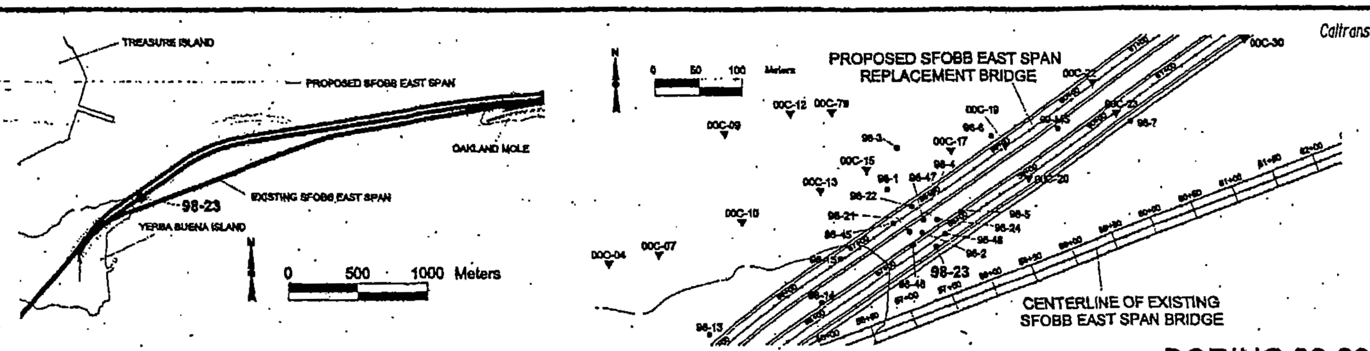
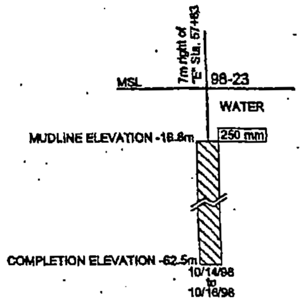
Metric





BORING 98-23

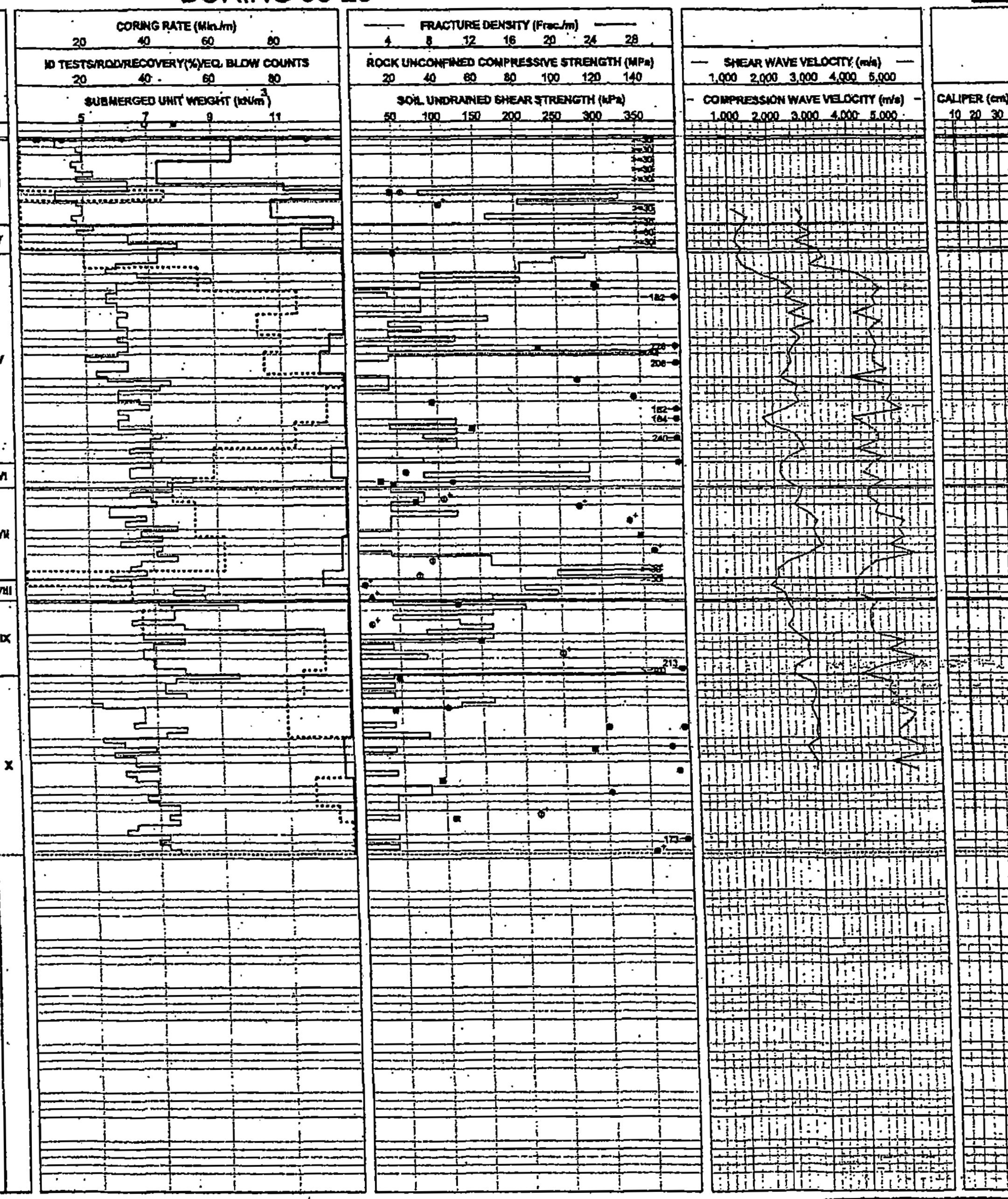
Coordinates: E1836389 N647567
CA State Plane Zone 3, NAD83, Meters

MATERIAL DESCRIPTION
MUDLINE ELEVATION: -16.8m (MSL)

BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
WOH 30/20cm	Fat CLAY (CH), gray, very soft (0.9m)	I
66-0	Fine GRAVEL (GP-GM) with silt, clay, sand, and coarse gravel, gray to olive gray (1.1m)	II
43-0, 82-0, 100-45, 78-0, 97-0	SANDSTONE (Graywacke) (Rx), olive gray, moderately to intensely weathered, moderately hard (6.4m)	III
87-0, 100-20	SILTSTONE (Rx) dark greenish gray, moderately to intensely weathered, moderately soft to moderately hard, poorly indurated, very thinly bedded, bedding dips 30 degrees, intensely fractured (8.2m)	IV
100-55, 100-65, 100-73, 95-60, 92-75, 100-99, 100-94, 100-84, 95-59	SANDSTONE (Graywacke) (Rx) with interbedded SILTSTONE (Rx) [10:1] -slightly to moderately rough fractures, slightly undulating, with very thin, dark yellowish brown iron oxide staining along 30, 60, and 80 degree fractures, and few healed fractures, below 9.1m -fine grained gravel (pebbly sandstone) interbed at 13.5m -siltstone, 16.1m to 16.5m -60mm thick, intensely fractured, claystone interbed, at 17.4m -siltstone with bedding dipping 25 degrees, 17.5m to 17.7m -fresh, hard, thinly bedded claystone, with bedding dipping 20 degrees, 18.7m to 18.8m -thinly bedded siltstone, dipping 20 degrees, 18.8m to 19.2m -iron oxide staining on steeply dipping (>60 degree) fractures, at 19.2m (21.3m)	V
100-46, 100-53	SANDSTONE (Graywacke) (Rx) interbedded with SILTSTONE (Rx) [1:1] -sandstone: medium dark gray, fresh, hard, moderately indurated, fine to medium grained (22.9m)	VI
95-62, 92-0	SANDSTONE (Graywacke) (Rx), moderate dark gray, fresh, hard -interbedded siltstone and sandstone, 23.2m to 23.4m -siltstone, dipping 20 degrees, pyrite mineralization along fracture plane, 23.8m to 24.0m (28.6m)	VII
100-33, 100-45, 100-36	-numerous vertical and near vertical healed, very thin (1mm), calcite/quartz filled fractures, 24.0m to 25.6m -dark gray, fresh, moderately hard claystone, 27.6m to 27.8m	VIII
100-92	CLAYSTONE (Rx), dark gray, fresh (29.9m) -very thinly bedded to laminated, bedding dipping 25 degrees, intensely fractured with smooth to slightly rough fracture planes, very thin calcite/quartz infilling	IX
100-85, 100-80, 97-97	SANDSTONE (Graywacke) (Rx) with interbedded CLAYSTONE (Rx) [2:1] -sandstone: medium dark gray, fresh, hard, moderately indurated, fine to medium grained with scattered coarse grains, thinly bedded, slightly fractured -claystone, intensely fractured, 33.4m to 33.7m -claystone with bedding dipping 20 degrees, below 34.2m (34.5m)	X
100-88, 100-95, 100-99	SANDSTONE (Graywacke) (Rx), medium dark gray, fresh, hard to very hard -moderately fractured with 60 degree to vertical fractures, at 35.2m -80mm thick, siltstone interbed, with bedding dipping 20 degrees, at 35.2m -moderately fractured, with slightly rough, calcite/quartz filled fractures, generally dipping 20 degrees, at 40.7m	

TOTAL DEPTH: 45.7m BACKFILLED WITH: Cement Grout

Note: Refer to multi-page logs presented in Volume 2 of the Final Marine Site Characterization Report for detailed soil and rock descriptions.



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
- 63.5mm-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 (kN/m³)
- THEORETICAL SUBMERGED UNIT WEIGHT (kN/m³)
- PLASTIC LIMIT LIQUID LIMIT
- EQUIVALENT SPT BLOW COUNT
- ROCK QUALITY DESIGNATION (RQD)
- ROCK RECOVERY PERCENT
- ROCK CORING RATE

STRENGTH TESTS

- POCKET PENETROMETER
- TORVANE
- REMOTE VANE
- MINIATURE VANE (RESIDUAL VANE)
- UNCONSOLIDATED UNDRAINED TRIAXIAL
- SWEDISH FALL CONE
- UNCONFINED COMPRESSION (SOIL)
- (Open symbols indicate remolded tests)
- Soil: Strength Exceeds Capacity of Measuring Device. Rock: Sample broke along discontinuity; intact sample would have greater strength
- Shear Strength Interpreted from CPT Tip Resistance (Nk=12-15)
- POINT LOAD TEST (ROCK-INTACT SPECIMEN)
- POINT LOAD TEST (ROCK-ALONG WEAK PLANE)
- UNCONFINED COMPRESSION (ROCK)
- FRACTURE DENSITY (ROCK)

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.

SABA MOHAN
GEOTECHNICAL DESIGN OVERSIGHT

DRAWN BY: SACHIN DESHPANDE
CHECKED BY: J.CHACKO

C. PRENTICE/ J. BIANCHIN
FIELD INVESTIGATOR
DATE SEE ABOVE

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

T.W.McNEILAN
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KM POST 13.4/13.8
POST MILE 8.3/8.5

ORIGINAL SCALE IN CENTIMETERS FOR REDUCED PLANS 0 1 2 3 4 5 6 7 8 9 10

CU 04
EA 0120E1

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

SELF-ANCHORED SUSPENSION BRIDGE (E2&T1)

LOG OF TEST BORINGS NO. 15

DISREGARD PRINTS BEARING EARLIER REVISION DATES →
REVISION DATES (PRELIMINARY STAGE ONLY)
SHEET 077 OF

SOIL AND ROCK TYPES

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

SAMPLERS

76mm-OD, 72mm-ID Thin Walled Tube	76mm-OD, 60mm-ID Modified California Liner
57mm-OD, 54mm-ID* Thin Walled Driven Tube	51mm-OD, 35mm-ID SPT Split Spoon Sampler
63.5mm-OD, 54mm-ID Offshore Liner	Rock Core (inset symbol represents recovery (%)

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- □ SUBMERGED UNIT WEIGHT (kN/m³)
- ○ THEORETICAL SUBMERGED UNIT WEIGHT (kN/m³)
- PLASTIC LIMIT LIQUID LIMIT
- ● EQUIVALENT SPT BLOW COUNT
- .... ROCK QUALITY DESIGNATION (RQD)
- —— ROCK RECOVERY PERCENT
- —— ROCK CORING RATE

STRENGTH TESTS

- ● POCKET PENETROMETER
- ◆ TORVANE
- ◈ REMOTE VANE
- ◆ MINIATURE VANE (◈ RESIDUAL VANE)
- ▲ UNCONSOLIDATED UNDRAINED TRIAXIAL
- ▼ SWEDISH FALL CONE
- ▣ UNCONFINED COMPRESSION (SOIL)
- ○△ (Open symbols indicate remolded tests)
- \+ Soil: Strength Exceeds Capacity of Measuring Device; Rock: Sample broke along discontinuity; intact sample would have greater strength
- Shear Strength Interpreted from CPT Tip Resistance (Nk=12-15)
- ● POINT LOAD TEST (ROCK-INTACT SPECIMEN)
- ⊕ POINT LOAD TEST (ROCK-ALONG WEAK PLANE)
- ■ UNCONFINED COMPRESSION (ROCK)
- —— FRACTURE DENSITY (ROCK)

STRENGTH OF COHESIVE SOILS

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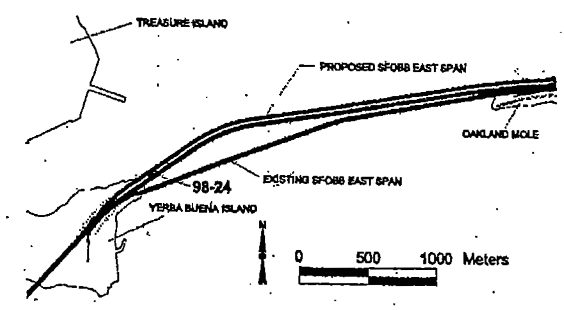
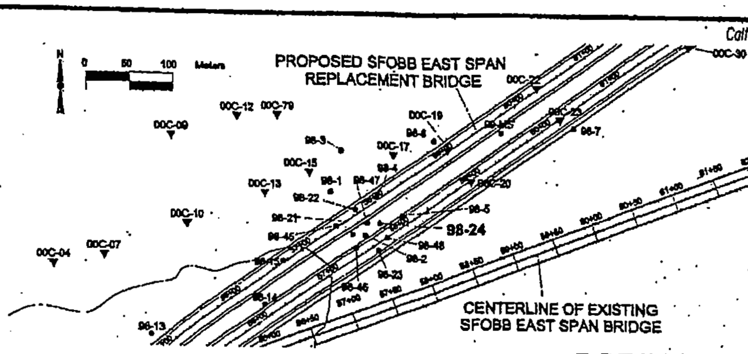
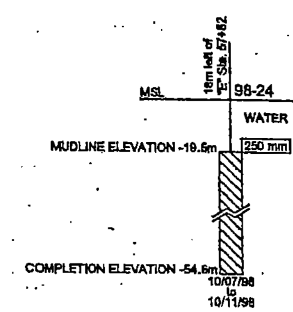
*Estimated from sampler driving record and CPT tip resistance.





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



DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.4,13.8	98	118

REGISTERED GEOTECHNICAL ENGINEER

10-15-03

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER T.W. McNeilan No. 0583 Exp. 12/31/03 Geotechnical STATE OF CALIFORNIA

FUGRO-EARTH MECHANICS, A JOINT VENTURE. 7700, Edgewater Dr., Ste. 846, Oakland, CA 94621 (510) 562-8833, FAX (510) 562-8858

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.

BORING 98-24

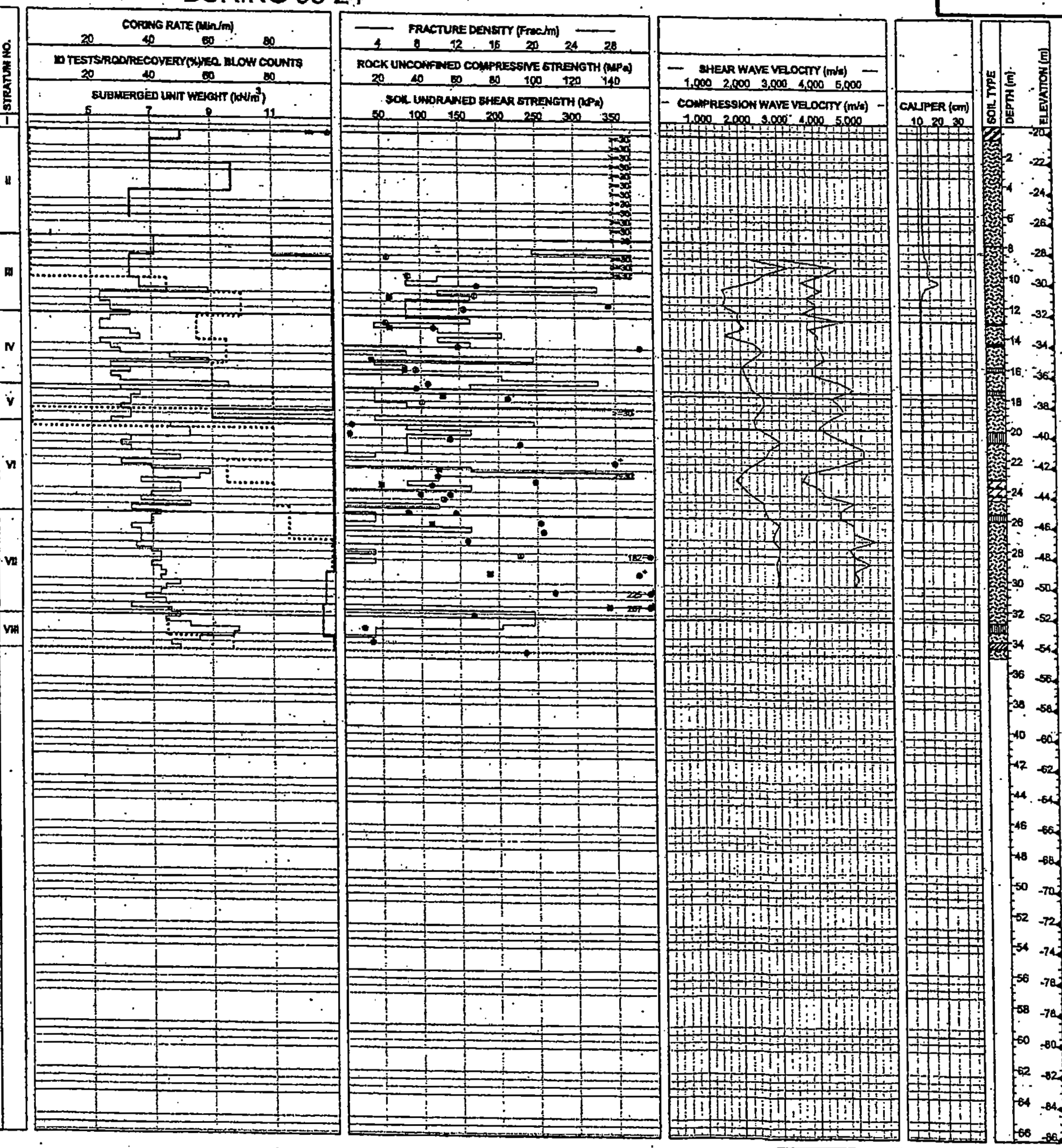
Coordinates: E1836391 N647599
CA State Plane Zone 3, NAD83, Meters

MUDLINE ELEVATION: -19.5m (MSL)

BLOW COUNT	MATERIAL DESCRIPTION	STRATUM NO.
WOH 60/10cm	Sandy CLAY (CH), dark olive gray, very soft, with shell fragments and gravel to 0.1m (0.9m)	I
50-0, 40-0, 67-0, 67-0, 33-0, 7-0	SANDSTONE (Graywacke) (Rx), dark greenish gray to light brown -intensely weathered, intensely fractured, gravel to cobble size, sandstone/rock fragments (7.8m)	II
80-0, 100-0, 100-45, 100-70	SANDSTONE (Graywacke) (Rx), yellowish brown, moderately weathered, moderately soft, intensely fractured, thinly to thickly bedded -grading to slightly weathered, moderately hard to hard, intensely to moderately weathered for 25mm on each side of fractures, at 10.7m (13.0m)	III
100-55, 100-65, 100-60	SANDSTONE (Graywacke) (Rx) with thinly interbedded SILTSTONE (Rx) [10:1], slightly weathered -sandstone: medium gray, hard, moderately fractured, slightly rough with thin yellowish orange iron oxide staining, fractures dipping 20, 30, 40 and 60 degrees (17.7m)	IV
100-80, 80-0	-siltstone, dark gray, moderately hard, thinly bedded, intensely fractured, bedding dipping 20 degrees, below 17.4m	V
100-0, 100-80	SANDSTONE (Graywacke) (Rx), medium gray, fresh, hard, slightly fractured (20.1m)	
100-65, 100-80, 100-85	SANDSTONE (Graywacke) (Rx) with thinly interbedded SILTSTONE (Rx) and CLAYSTONE (Rx) [1:1:0.5], fresh -sandstone: medium gray, hard, slightly fractured, with numerous, healed, closed, 1mm to 3mm thick, quartz/calcite filled fractures, dipping 60 to 80 degrees. -interbedded siltstone and claystone, 23.3m to 24.7m -sandstone, 24.7m to 25.6m (26.1m)	VI
100-99, 97-97	SANDSTONE (Graywacke) (Rx), medium gray, fresh, hard, slightly fractured -fractures: primarily 1mm thick, healed, dipping 60 degrees to near vertical, spacing typically 0.15m to 0.3m (32.8m)	VII
96-45, 100-67	SILTSTONE (Rx) with interbedded SANDSTONE (Graywacke) (Rx) and CLAYSTONE (Rx) [1.5:1:0.5], fresh -moderately hard, thinly bedded, intensely fractured -sandstone interbed, 33.4m to 34.0m -claystone interbed, 34.0m to 34.4m -sandstone below 34.4m	VIII

TOTAL DEPTH: 35.1m BACKFILLED WITH: Cement Grout

Note: Refer to multi-page logs presented in Volume 2 of the Final Marine Site Characterization Report for detailed soil and rock descriptions.



SABA MOHAN	DRAWN BY	SACHIN DESHPANDE	J.CHACKO, C. PRENTICE, J. PALMER, J. BIANCHIN	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	T.W.McNEILAN	BRIDGE NO. 34-0006L/R
GEOTECHNICAL DESIGN OVERSIGHT S. M...	CHECKED BY	J.CHACKO	FIELD INVESTIGATOR DATE SEE ABOVE		PROJECT ENGINEER	KM POST 13.4/13.8

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

SELF-ANCHORED SUSPENSION BRIDGE (E2&T1)

LOG OF TEST BORINGS NO. 18

ORIGINAL SCALE IN CENTIMETERS FOR REDUCED PLANS 0 1 2 3 4 5 6 7 8 9 10

CU 04
EA 0120E1

POST MILE

DISREGARD PRINTS BEARING EARLIER REVISION DATES

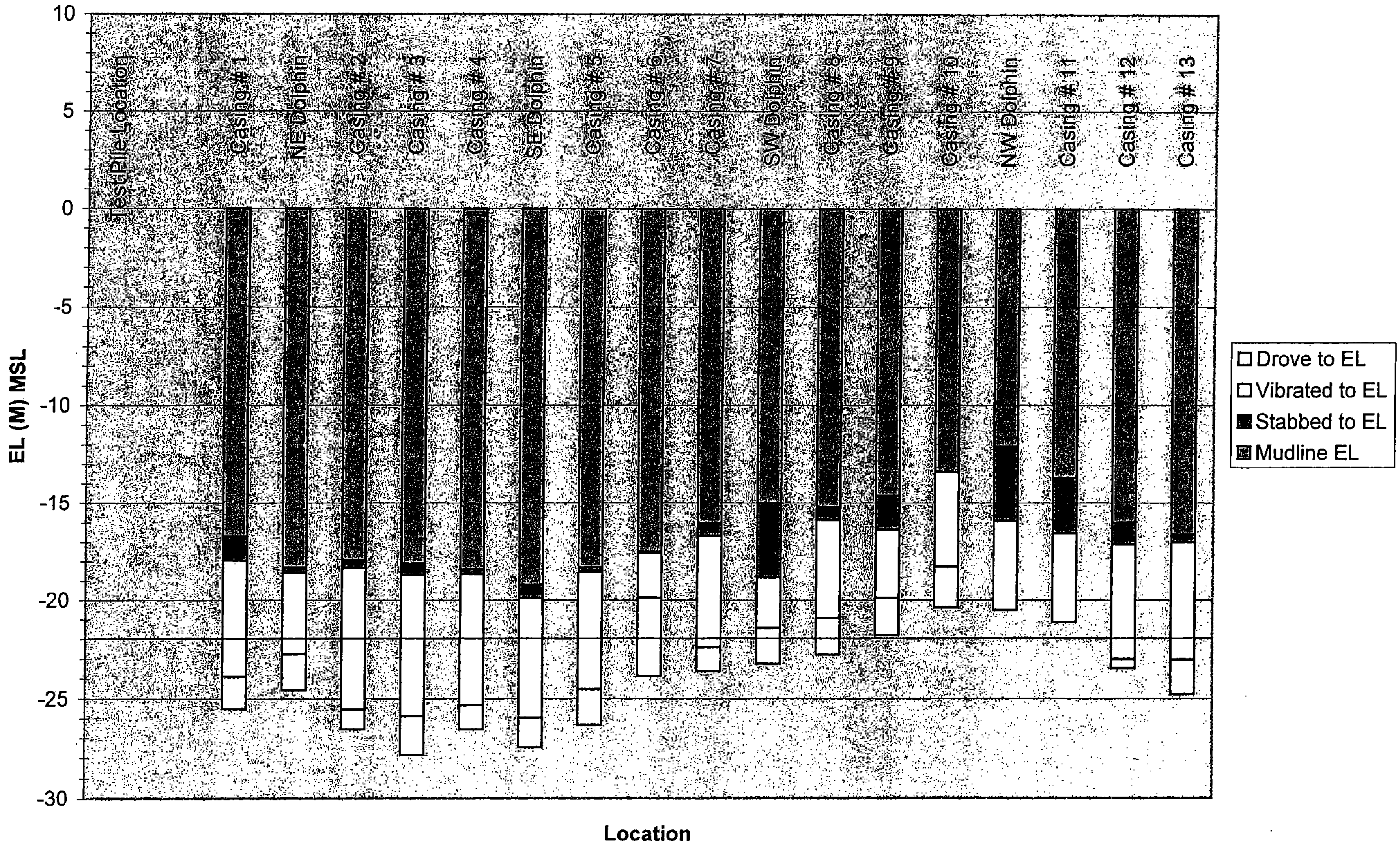
REVISION DATES (PRELIMINARY STAGE ONLY) SHEET OF

T1 Test Pile Program

Location: T1 Footing, Pile Locations #1-13 and 4 Dolphins
Casing Test Pile: 42" x 1/2" wall Test Pile, 1" top and tip
Dolphin Test Pile: 24" x 3/4" wall Test Pile (w/ & w/o stinger)
Date(s): 8/25/2004 - 8/27/04

Test Pile Location	Notes	Mudline EL (m) MSL	Stabbed to EL (m) MSL	Vibrated to EL (m) MSL	Drove to EL (m) MSL	Vibed In Ground (ft)	Total In Ground (ft)	Uplift Test Load (Tons)
Casing # 1		-16.70	-17.96	-23.88	-25.52	23.6	28.9	
NE Dolphin	2nd Stinger	-18.30	-18.57	-22.77	-24.58	14.7	20.6	held 145
Casing # 2		-17.90	-18.33	-25.53	-26.56	25.0	28.4	
Casing # 3		-18.10	-18.68	-25.88	-27.86	25.5	32.0	
Casing # 4		-18.40	-18.65	-25.32	-26.56	22.7	26.8	
SE Dolphin	No Stinger	-19.20	-19.88	-25.96	-27.46	22.2	27.1	held 145
Casing # 5		-18.30	-18.53	-24.51	-26.33	20.4	26.3	
Casing # 6		-17.50	-17.57	-19.87	-23.86	7.8	20.9	
Casing # 7		-16.00	-16.68	-22.40	-23.62	21.0	25.0	
SW Dolphin	No Stinger - w/ set up	-15.00	-18.83	-21.42	-23.26	21.1	27.1	pulled @ 140
Casing # 8		-15.20	-15.85	-20.92	-22.80	18.8	24.9	
Casing # 9		-14.60	-16.35	-19.88	-21.83	17.3	23.7	
Casing # 10		-13.40	NA	-18.29	-20.38	16.0	22.9	
NW Dolphin	1st Stinger	-12.10	-15.91	NA	-20.50		27.5	held 145
Casing # 11		-13.70	-16.55	-21.15	NA	24.4		
Casing # 12		-16.00	-17.13	-23.03	-23.48	23.1	24.6	
Casing # 13		-16.60	-17.02	-23.06	-24.79	21.2	26.9	

Pile Penetration



B2

ABE Engineering

RECIEVED
9/31/04
SM

2230 Lariat Lane, Walnut Creek, CA 94596
PHONE: 925-944-6363 FAX: 925-476-1588
EMAIL: SA@AbeEngineering.com

Dynamic Pile Test Report

cc: FRANK

Company:	KFM	**Date:**	September 29, 2004
Attn:	Mr. Stuart Moore	**From:**	Steve Abe
Re:	Bay Bridge San Francisco, CA August 13, 2004	PILE # 9	ABE Job No. 04041-1

This report presents dynamic test results obtained by Abe Engineering (AE) during driving of one test pile/casing (Location T1-9) for the project referenced above on August 25, 2004. The casing was driven to evaluate the depth of bedrock and to predict the driveability of future CIDH casings. The pile was dynamically tested for the final 7 ft of driving in the bedrock. The objectives of the testing were to evaluate driving stresses and soil/rock resistance at the time of testing. The dynamic testing was performed using a Model PAK Pile Driving Analyzer (PDA) according to the ASTM D4945 test standard. During dynamic monitoring, PDA calculations for soil resistance, hammer performance, and driving stresses were made according to the Case Method. Subsequent CAPWAP analysis was performed for EOD (End of Drive) test data to further evaluate pile capacity and soil resistance at the final pile depth.

The test pile was a 42-inch OD x 0.5-inch wall open-ended steel pipe pile. It was driven and tested with a Delmag D80-23 diesel hammer which has a maximum rated energy of 186 kip-ft. Further information regarding the pile material was not provided.

Soil Details

AE was not provided with a geotechnical report or soil boring logs for this site. The subsurface conditions were described to me at the site to generally consist of Bay Mud, overlying bedrock. Further details regarding the soil conditions or bedrock were not provided and are beyond the scope of this report.

DYNAMIC TEST RESULTS

The following PDA calculated Case Method results are printed versus blow number and pile penetration depth in Appendix A. The Case Method results are summarized BOR (Beginning of Restrike) and or EOR (End of Redrive) test conditions in Table 1.

RMX- the Case Method maximum soil resistance estimate
STK- hammer ram stroke as computed from the blow rate (BPM).
EMX- the maximum energy transfer to the pile.
CSX- the maximum axial compression stress at the sensor location, computed using the average of two strain transducer measurements.
FMX- The maximum impact force from the hammer.

A CAPWAP analysis was performed for selected test data at the final penetration depth. The CAPWAP analysis provides a more accurate capacity estimate than the Case Method estimate, which is based on an assumed damping value. Furthermore, the CAPWAP analysis also indicates the soil resistance distribution along the pile shaft and at the toe and the dynamic soil damping and quake parameters, and computes the pile stress distribution along the full length of the pile. The CAPWAP results are given in Appendix B.

DISCUSSION OF RESULTS

The PDA results at the final depth below water at the time of testing are summarized in the following table.

BPF	Depth	RMX	STK	EMX	CSX	CSI	FMX
Bl/ft	ft	Kips	ft	kip-ft	ksi	ksi	kips
80	77.00	1856	9.85	93	33.14	34.69	2156

The CAPWAP analysis result at the end of driving indicates a total ultimate resistance of 1880 kips consisting of 511 kips toe resistance and 1369 kips shaft resistance. The total shaft resistance at bottom of the pile (in the rock) was 558 kips or 82 kips/ft for the bottom 7 ft of the pile. Note that the CAPWAP shaft resistance is the total of both external and internal soil resistance inside the pipe. The maximum compression (CSX) stresses measured during driving were less than 34 KSI and the hammer stroke averaged 9.9 ft.

Please review the attached "LIMITATIONS AND CONSIDERATIONS REGARDING DYNAMIC TEST RESULTS". We appreciate the opportunity to assist you with this project. Please contact me if you have any questions regarding these results, or if we may be of further service.

Very truly yours,
ABE Engineering

Steve Abe, P.E.

REGISTERED PROFESSIONAL ENGINEER
STEVEN K. ABE
No. C 055277
Exp. 12/07
CIVIL
STATE OF CALIFORNIA

LIMITATIONS AND CONSIDERATIONS REGARDING DYNAMIC TEST RESULTS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing.** At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provides an estimate of the axial pile capacity. Increases (soil set-up) and decreases (relaxation) in the pile capacity with time typically occur after driving. Therefore, **restrike testing usually yields a better indication of long-term pile capacity than a test at the end of pile driving.** The appropriate waiting time depends, among other factors, on the permeability of the soil and the soil type.

Capacity results for open pile profiles

Larger diameter open-ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area. In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

- 90% of yield strength for steel piles
- 85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression
- 100%of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension
- 70% of the reinforcement strength for regularly reinforced concrete piles in tension

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- Additional pile loading from downdrag or negative skin friction,
- Lateral and uplift loading requirements
- Liquefiable soils
- Effective stress changes (due to changes in water table, excavations, fills or other changes in overburden, or construction activities)
- Long-term settlements in general and settlement from underlying weaker layers and/or pile group effects

These factors have not been evaluated by ABE Engineering and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Note that ABE Engineering is providing dynamic test data only; at the time the dynamic measurements were obtained. This data is to be used by the responsible project engineers (not ABE Engineering) to evaluate pile and hammer performance, soil response, and to assess the design assumptions made by others. ABE engineering is in no way assessing, warranting, or certifying the adequacy or performance of the piles or foundation, providing geotechnical evaluation or recommendations, or performing foundation design services on this project.

APPENDIX A

PDA Case Method Results

Pile: T1-9 Proj: KFM Pg1
Info: 42" OEP/ D80

RMX: Capacity - RMX
STK: Stroke (O.E.Diesels)
EMX: Max Transferred Energy
CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
FMX: Max Measured Force

BL# end	bl/ft	depth ft	TYPE	#Bls	RMX kips	STK ft	EMX kips-ft	CSX ksi	CSI ksi	FMX kips
10	34	70.26	AVG	10	816	6.99	58	24.92	25.14	1619
20	34	70.56	AVG	10	932	6.19	36	21.19	21.53	1377
30	34	70.85	AVG	10	1058	6.56	43	22.85	23.17	1486
40	57	71.09	AVG	10	1218	7.02	51	24.59	24.87	1597
50	57	71.26	AVG	10	1350	7.72	62	26.75	26.99	1738
60	57	71.44	AVG	10	1375	7.74	63	26.76	26.96	1740
70	57	71.61	AVG	10	1400	7.90	64	27.24	27.55	1770
80	57	71.79	AVG	10	1437	8.10	66	27.58	27.83	1791
90	57	71.96	AVG	10	1492	8.27	68	27.98	28.20	1819
100	57	72.14	AVG	10	1522	8.28	68	28.13	28.29	1828
110	57	72.32	AVG	10	1559	8.30	68	28.29	28.48	1840
120	57	72.49	AVG	10	1595	8.53	71	28.94	29.16	1882
130	57	72.67	AVG	10	1619	8.64	73	29.26	29.45	1903
140	57	72.84	AVG	10	1666	8.85	77	29.87	30.09	1943
150	57	73.02	AVG	10	1753	9.24	83	31.05	31.24	2019
160	57	73.19	AVG	10	1807	9.31	85	31.41	31.53	2042
170	57	73.37	AVG	10	1839	9.46	87	31.85	31.97	2070
180	57	73.54	AVG	10	1879	9.79	91	32.57	32.82	2119
190	57	73.72	AVG	10	1948	9.83	93	32.84	33.12	2137
200	57	73.89	AVG	10	1946	9.90	93	32.92	33.19	2141
210	57	74.07	AVG	10	1955	10.22	97	33.75	34.08	2195
220	57	74.25	AVG	10	1963	10.07	96	33.47	33.97	2179
230	57	74.42	AVG	10	1968	10.11	97	33.61	34.32	2188
240	57	74.60	AVG	10	1977	9.96	93	33.01	33.90	2147
250	57	74.77	AVG	10	1927	9.99	92	32.60	33.49	2119
260	57	74.95	AVG	10	1949	10.02	95	33.09	33.87	2153
270	57	75.12	AVG	10	1918	9.94	92	32.78	33.51	2133
280	57	75.30	AVG	10	1917	9.95	93	32.80	33.65	2134
290	57	75.47	AVG	10	1940	9.70	90	32.45	33.17	2111
300	57	75.65	AVG	10	1929	9.89	94	32.96	34.09	2145
310	57	75.82	AVG	10	1921	9.84	93	32.81	33.88	2134
320	57	76.00	AVG	10	1917	9.78	91	32.63	33.81	2122
330	80	76.13	AVG	10	1900	9.68	90	32.48	33.61	2114
340	80	76.25	AVG	10	1915	9.79	92	32.70	33.89	2128
350	80	76.38	AVG	10	1892	9.70	91	32.56	33.71	2117
360	80	76.50	AVG	10	1932	9.82	92	32.85	33.71	2136
370	80	76.63	AVG	10	1914	9.81	93	32.75	34.05	2130
380	80	76.75	AVG	10	1896	9.78	92	32.62	33.76	2122
390	80	76.88	AVG	10	1904	9.83	94	32.90	34.30	2140
400	80	77.00	AVG	10	1856	9.85	93	33.14	34.69	2156

DRIVEN (2004-Aug-25 : T1-9.Q01)

APPENDIX B

CAPWAP Analysis Results

ABE Engineering 29-Sep-2004

KFM□
Pile: T1-9□ Blow: 401 Data: 42" OEP/ D80□
Collected: 04-08-25 Operator: SA□ CAPWAP(R) Ver. 1997-1

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 1880.0; along Shaft 1369.0; at Toe 511.0 kips□

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit Resist. w. Respect to Depth kips/ft	Unit Resist. w. Respect to Area kips/f2	Smith Damping Factor s/ft	Quake inch
				1880.0					
1	50.8	19.3	4.1	1875.9	4.1	.61	.06	.124	.174
2	57.6	26.1	6.8	1869.1	10.9	1.00	.09	.124	.174
3	64.4	32.9	57.2	1811.8	68.2	8.44	.77	.124	.174
4	71.2	39.7	38.8	1773.1	106.9	5.72	.52	.124	.174
5	77.9	46.4	256.7	1516.4	363.6	37.87	3.44	.124	.174
6	84.7	53.2	447.2	1069.2	810.8	65.98	6.00	.124	.174
7	91.5	60.0	558.2	511.0	1369.0	82.36	7.49	.124	.174
Average Skin Values			195.6			22.82	2.62	.124	.174
	Toe		511.0				1135.56	.112	.242

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		1.458	.494-Smith Type
Unloading Quake	(% of loading quake)	40	49
Reloading Level	(% of Ru)	100	100
Soil Plug Weight	(kips)		.97

KFM□
Pile: T1-9□ Blow: 401 Data: 42" OEP/ D80□
Collected: 04-08-25 Operator: SA□ CAPWAP(R) Ver. 1997-1

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress kips/in2	max. Tension Stress kips/in2	max. Trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	2127.6	-389.6	32.637	-5.976	99.71	17.8	1.002
2	6.8	2129.1	-417.0	32.659	-6.396	98.49	17.8	.977
4	13.6	2132.3	-526.4	32.709	-8.075	95.93	17.7	.927
7	23.7	2137.8	-622.9	32.793	-9.556	92.57	17.6	.856
10	33.9	2144.9	-712.2	32.902	-10.925	86.77	17.5	.763
12	40.7	2150.6	-764.7	32.990	-11.730	83.79	17.5	.709
15	50.8	2167.9	-814.6	33.256	-12.495	77.34	17.3	.611
18	61.0	2203.3	-847.5	33.799	-13.001	70.16	16.6	.516
20	67.8	2111.7	-800.4	32.392	-12.278	61.72	16.0	.453
23	77.9	2366.8	-785.5	36.306	-12.050	53.29	13.0	.347
26	88.1	1491.7	-197.6	22.883	-3.031	22.91	9.7	.262
27	91.5	1361.4	-206.9	20.884	-3.174	8.14	11.0	.243
Absolute	77.9			36.306		(T=	25.4 ms)	
	64.4				-13.072	(T=	50.0 ms)	

CASE METHOD

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
RS1	2713.	2565.	2416.	2268.	2120.	1972.	1823.	1675.	1527.	1379.
RMX	2713.	2565.	2416.	2268.	2120.	1972.	1836.	1814.	1803.	1793.
RSU	2757.	2613.	2469.	2325.	2182.	2038.	1894.	1750.	1606.	1462.

RAU 1356. RA2 1456.

Current CAPWAP Ru= 1880.0; Corresponding J(Rs)= .56; J(Rx)= .57

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	EFN	RLT	REN
18.02	.29	2095.5	2100.7	2100.7	1.001	.094	100.6	69.1	2096.	4827.

PILE PROFILE AND PILE MODEL

Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3	Circumf. ft
.00	65.19	29972.0	.492	11.000
91.50	65.19	29972.0	.492	11.000

Toe Area .450 ft2

Top Segment Length 3.39 feet, Top Impedance 116.28 kips/ft/s

Pile Damping 1.0 %, Time Incr .202 ms, Wave Speed 16802.9 ft/s

Form 435 - Printed in U.S.A. 11/95

WORK SHEET

Project DRILLING T1 | Estimator FDA. | Item No.
Type of Work PHASE A (-30m) | Date 4/2/2006. | Sheet No. 1-2.

VERTICAL LOAD DURING UNDERREAMING (PHASE A)

- WEIGHT OF TEMPORARY CASING.

  32m x ∅3000mm x 38mm. → 99 tons.

- PIGTOP DRILLING MACHINE → ≈ 30 tons.

- TOTAL DRILL STRING AS PER ATTACHED SHEET → 110 tons.

≈ 240 tons.

E2 / T1 Foundations

Drilling Operations 13-Feb-06

Drillstring configuration for underreaming work

Part	*Description*	*Abreviation*	*Length - m*	*Weight - kg*	*Weight - lbs*	*Cum. Weight - kg*	*Cum. Weight - lbs*	*Cum. length - m*	*Remarks:*
1	Underreamer	UR 2.70/3.30	3.195	18,500	40,700	18,500	40,700	3.195	
2	Drill collar (balasted w/ lead)	DC	3.000	20,000	44,000	45,500	100,100	6.195	
3	BHA stabilizer (balasted w/ steel)	BHASTAB 2.90/2.	2.000	7,000	15,400	25,500	56,100	8.195	
4	Drill collar (balasted w/ steel)	DC	3.000	14,000	30,800	59,500	130,900	11.195	
5	Drill collar (balasted w/ steel)	DC	3.000	14,000	30,800	73,500	161,700	14.195	
6	Drill collar (empty)	DC	3.000	14,000	30,800	87,500	192,500	17.195	
7	BHA stabilizer (balasted w/ steel)	BHASTAB 2.90	2.000	7,000	15,400	94,500	207,900	19.195	
8	Cross over sub (2.0 m long)	COSUB	2.000	1,180	2,596	95,680	210,496	21.195	Total weight of bottom hole assembly
9	Drill pipe	DP	3.000	535	1,177	96,215	211,673	24.195	
10	Drill pipe	DP	3.000	535	1,177	96,750	212,850	27.195	
11	Drill pipe	DP	3.000	535	1,177	97,285	214,027	30.195	
12	Drill pipe stabilizer	DPSTAB 2.90	3.000	2,400	5,280	99,685	219,307	33.195	Total weight when adding stabilizer
13	Drill pipe	DP	3.000	535	1,177	100,220	220,484	36.195	
14	Drill pipe	DP	3.000	535	1,177	100,755	221,661	39.195	
15	Drill pipe	DP	3.000	535	1,177	101,290	222,838	42.195	
Note:	Actual weights have been verified in January 2006.			MAXIMUM CRANE PICK WILL 220000 lbs + 10000 lbs rigging = 232000 lbs ==> MAXIMUM RADIUS DB HAAKON = 115 feet.					

T1 DRILLING.
PHASE 4 (~30m).
2-2

C:\E2T1 foundations project\Drilling\Drill string weights.xls

BEARING CAPACITY OF 3000mm CASING IN OUSIDE FRICTION ONLY.

Frank,

First of all I would like to make a few comments:

1. It is very difficult to estimate the ultimate vertical load carrying capacity of the casing in just a couple of hours.
   First of all because there is very little know about the behaviour and strength properties of fragmented rock. .Secondly because there is a substantial amount of different information resulting from PDA measurements, static pull out tests.
2. I have tried to get a feeling for the range of capacities the casing could have. This may help in establishing the safety against creep when drilling.

I have analysed the following situations:

1. Estimation of ultimate bearing capacity based on general geotechnical (API) recommendations.
   I have used a very high angle of internal friction of 50 degrees, but this seems to me not to be very unlikely for the rock-fragments.
   This method will give a increasing unit friction with depth and a quadratic increase of ultimate capacity with depth.
   As an alternative I have assumed a constant skin friction of 2 ksf (100 kPa) resulting in a linear increase in ultimate capacity.
   The results are presented on page 4.
   The ultimate capacity ranges from 200 to 500 tons.

2. During the test phase a 42x0.5" test pile was driven by an Delmag D-80 hammer. PDA measurements were carried out by ABE engineering and reported on September 29, 2004 (ABE job nr. 04041-1)
   The reference number of this test pile is nr. 9.
   According to the presented results the ultimate load in friction is 1369 kips (internal + external friction). The toe resistance was calculated as 511 kips (ratio toe : shaft = 1:2.7)
   The penetration is about 40 ft. The average skin friction (internal + external) is calculated as 2.62 kips/sqft. Assuming inside and outside friction are identical, would results in an external friction of 1.3 kips/sqft. Taking the same number for the 3000mm casing and 25ft penetration would give an ultimate capacity of around 1000 kips (or 450 tons).

3. At the NE Dolphin Location a 24x3/4 inch test pile was statically pulled before it was extracted by the APE-300 vibratory hammer. The pile had a penetration of 88-60=28 ft. The maximum line pull of 145 tons did not move the pile at all. The pile weight is approximately 8 tons.

   Assumption 1:
   extraction resistance is caused by internal and external friction. The average friction can than be calculated as $(145-8)/(2*2*\pi*28)=0.38$ tons/sqft or 0.78 kips/sqft.
   Converted to the 3000mm casing and 25ft penetration results in a minimum ultimate load of 600 kips or 300 tons.

   Assumption 2:
   extraction resistance is caused by external friction and the weight of the pile and the soil inside the pile (soil inside the pile will move up with the pile). Pile weight and soil weight are approximately 30 tons together. The average friction results then in $(145-30)/(2*\pi*28)=0.65$ tons/sqft or 1.3 kips/sqft.

(FEB 03, 2006)

Again converted to the 3000mm casing would result in a capacity of 500 tons.

4. A wave equation runs has been made for the S-500 at 150 kJ energy setting. With similar soil input data as in my earlier runs a estimate was made of the expected blowcounts versus depth. This graph is given on page 3 having an expected blowcount of 60 bl/ft. These were also the final records of piles #2 and #3 (February 1st, 2006).
   The wave-equation programme has calculated this blowcount with a total skin friction of 1308 tons and a toe resistance of 472 tons. The ratio toe : shaft = 1:2.77, i.e. very similar to the values recorded by the PDA measurements with the D-80 hammer.
   A total skin friction resistance of 1308 tons and an even distribution between inside and outside friction would result in an outside capacity of 650 tons.
   Most likely this capacity will be more as the driving shoe at the casing will cause a friction reduction on the inside of the casing. In case of a 50% reduction on the inside of the pile would result in an outside frictional capacity of 850 tons.

Conclusions:

Although the standard geotechnical calculations show that the bearing capacity for a top-load of 240 tons is marginal (even assuming a very high angle of internal friction for the fragmented rock). all other analyses indicate that the actual bearing capacity is higher than calculate by the geotechnical formulas. Based on the back analyses of other available information it looks more likely that the casings will have a bearing capacity in the order of magnitude of 450-850 tons.

Torque from Drill.

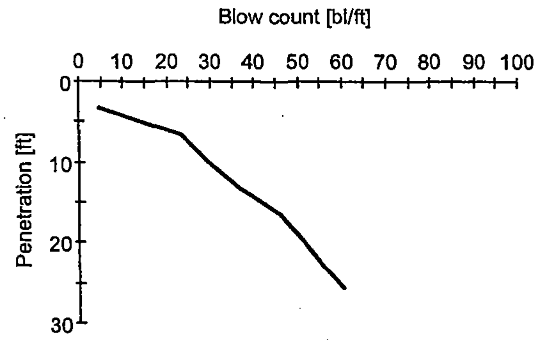
The drill has a maximum torque of 33 TN.meter. This will result in an average shear along the outer shaft (inside empty) of 33tons/628sqft=0.052 ton/sqft.
As most materials can only take friction once (only a very small load is required to push a spinning car on ice sideways) the above friction has to be 'translated' into a vertical shear or vertical friction.
The torque resistance of the soil is considered to be dependent on the Shear Modules and the vertical friction on the E-modulus of the soil. The relation between the two is approx. 1:3.
The 0.053 tons/sq/ft shear would than correspond with a vertical friction of 0.16 tons/sqft. This would 'consume' $0.16*\pi*10$(ft)*25(ft)=125 tons from the total available vertical carrying capacity.

Trust that above calculations are of any help to you. To prove the bearing capacity you could redrive the first pile after the drill has passed the toe of the casing. The blowcount will give an indication of the frictional bearing capacity, if you want to have the number more accurate a PDA measurement could be carried out.

Kind regards,
GeoDrive Technology BV,

Geert Jonker

S-500 - pile 3000x38/50/25mm

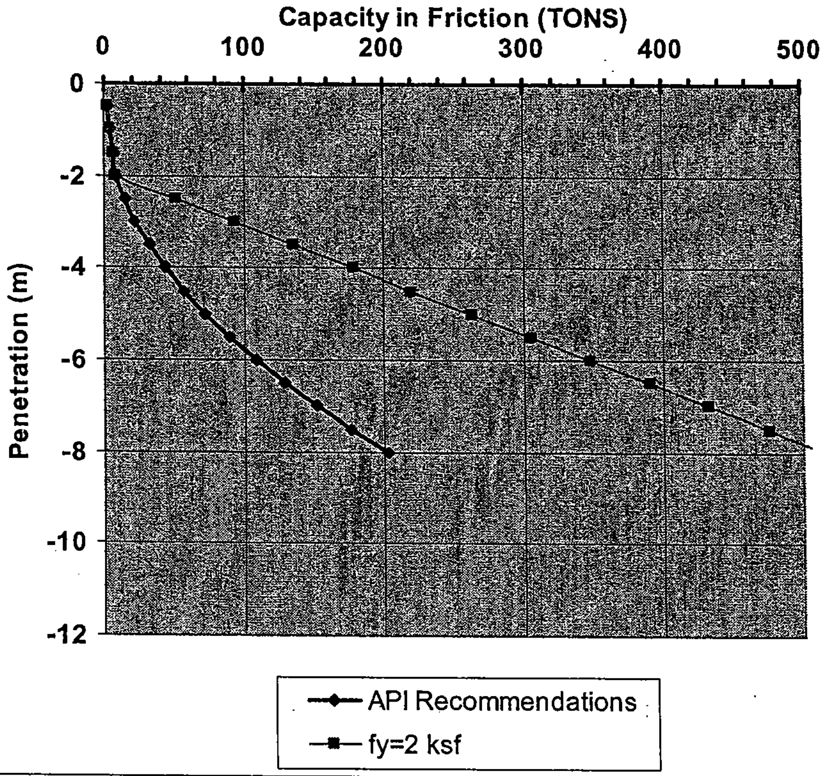


Blow count

Hammer type	S-500			
Rated Energy	369	[Kipsft]		
Impact Energy	111	[Kipsft]		
Transferred Energy	108	[Kipsft]		
Calculated set pile toe	0.2	[in]		
Maximum pile toe displ.	0.3	[in]		
Penetration pile toe	25.59	[ft]		
Blow count	**60.**	**[bl/ft]**		
Blow rate	72	[blows/min		
Total number of blows	966			
Total driving time	13	[min]		
Total driving resistance	**3562.08**	**[Kips]**		
Driving resistance shaft	**2616.62**	**[Kips]**	**=1308**	**[tons] inside+ outside**
Driving resistance shaft			**= 654**	**[tons] outside only.**
Driving resistance toe	**945.47**	**[Kips]**	**= 472**	**[tons]**
Maximum stresses in pile				
Compression stress	44.9	[ksi]		
at level	16.1	[ft]		
Tension stress	-7.1	[ksi]		
at level	10.2	[ft]		

SIMULATION ON 3000mm CASING USING IHC S-500 ON FOLOWER AT 150 kJ ENERGY

FRICTION CAPACITY 3000mm CASING



ACTUAL TEMP CASING DATA

MEMORANDUM

To: Reuben Zylstra, KFM JV
From: Mike Holloway
Date: February 7, 2006

Dynamic Monitoring Results
T1 Temporary Casing
San Francisco Bay Bridge Replacement

Dynamic monitoring results from the subject test program were used to investigate the performance of your IHC 500 double-acting hydraulic hammer during installation of 3-m-diameter x 38.1mm-thick-wall steel pipe casings. The casings are driven into underlying sedimentary bedrock for the T1 foundation. As the IHC 500 is designed to deliver 500 kJ per hammer blow, the target ceiling of 200 kJ represents a significantly lower energy level than that commonly used with this device. The use of a 79.3 tonne drive cap to adapt the hammer to this large diameter pipe provides another "large" variable in the hammer performance equation.

Three casings were installed on January 30th and 31st, with high strain dynamic testing performed at the end of installation of each of these, designated casings 2, 3 and 13. The PDAPLOTs of the monitored data are provided in Attachment 1. The figures contain: resistance to penetration (RMX); average and individual gage peak compression stress (CSX and CSI); and maximum transferred energy and blows per minute for each blow (EMX and BPM). The data are given in the respective panels versus blow number (on the y-axis). In some cases the rapid "restrike" of the hammer in this throttled-down mode resulted in blows undetected by the PDA system.

Pipe 13 was driven on January 30th to 28.75 m below the template deck (toe elevation -24.4 m). The hammer operated between 75 and 115 kJ over the last 100 blows, with erratic blow rate/energy transfer levels observed. The monitored transferred energy (EMX) levels were 15 to 20 percent higher than those detected by the kinetic energy sensor system provided for this hammer. Your observed blowcounts varied between 33 and 69 Blows/0.25m in the final 2 m penetration.

The next two casings (2 and 3) were driven in stages as the template system was adjusted with casing penetration levels below the deck, on January 31st. With casing 2 the final stage of driving was remarkably consistent from blow-to-blow, making for somewhat better agreement between our EMX values (80 to 115 kJ) and those given by the IHC data system. The blowcounts recorded over the final two meters ranged between 23 and 60 Blows per 0.25m, with a final penetration of 31.25 m (toe elevation -26.45 m).

Casing 3 was monitored only over the final meter and a half of driving, finishing at 30.75 m penetration (toe elevation -25.95 m). In this case the blowcount varied between 38 and 42 blows per 0.25 m, while the hammer was operating at an EMX level between 95 and 155 kJ. In this case (at higher throttle setting) the hammer blows oscillated from "upper to lower bound

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energy per blow" in the above range, with the PDA EMX values and the IHC monitoring system energy values differing by only a few percent.

In terms of hammer performance, the operating range applied in driving these casings remained well below the 200 kJ ceiling that was intended, achieving the greater penetrations into the bedrock that you intended. Clearly, the IHC readout of hammer energy is more accurate as the hammer runs closer to its normal operating level.

The peak (individual gage) compression stresses remained below 90 MPa, with peak average stresses about 15 MPa less than those from the individual gages. These values are substantially less than the yield strength of the steel (345 MPa). These relatively low impact stresses, coupled with the 2-m-long x 51mm thick wall toe reinforcement, make the chances of damaging the casing toe due to impact driving rather remote.

The resistance levels (RMX) shown in the PDAPLOTs deserve further comment. As none of these casings exhibited much shaft resistance during installation, the overwhelming share of soil resistance is attributable to that at the toe. The resistance obtained at the end of driving casings 2 and 13 were on the order of 12 MN and 10 MN, respectively, while that of casing 3 was just reaching 9 MN. There is certainly inherent variability in the bedrock materials described in the boring logs, which probably accounts for the differences between casings 2 and 13. The even lower resistance to penetration of casing 3 is probably also due to the appreciably larger blows that were applied in this case. It has been our experience with low-displacement piles in our sedimentary bedrock formations, that the larger hammer blows can be appreciably more effective in "cookie-cutting" through the materials at the pile toe.

We trust these findings suffice for your present needs. Should any questions arise, please call.

REGISTERED PROFESSIONAL ENGINEER
DAVID MICHAEL HOLLOWAY
No. 27028
Exp. 3/31/07
CIVIL
STATE OF CALIFORNIA

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Attachment 1

PDAPLOT Results

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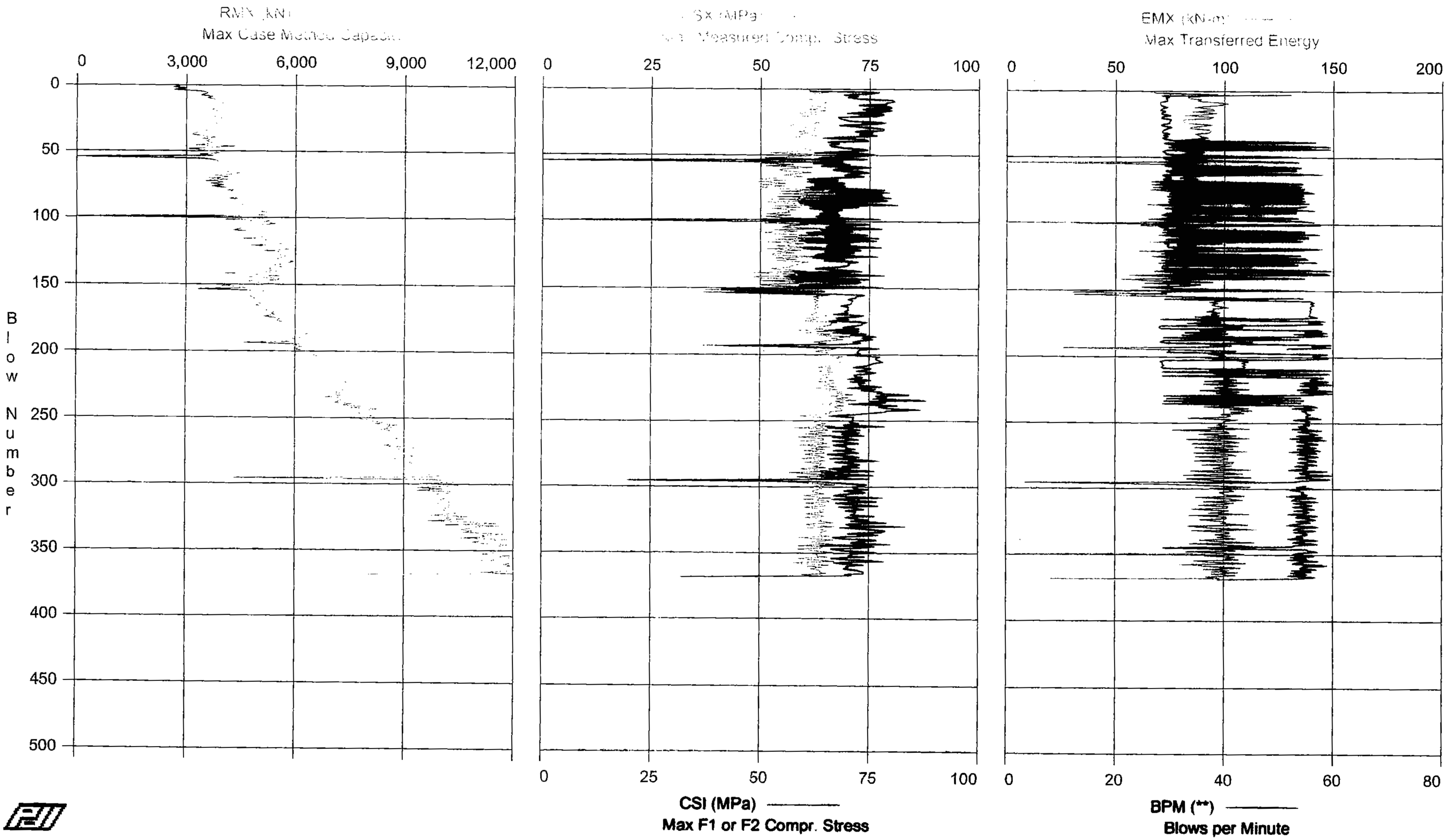
PDIPLOT Ver. 2005.1 - Printed: 3-Feb-2006

InSitu Tech, Ltd. - Case Method Results

Test date: 31-Jan-2006

San Francisco Bay Bridge T1 - Casing 2
IHC 500 Hydraulic

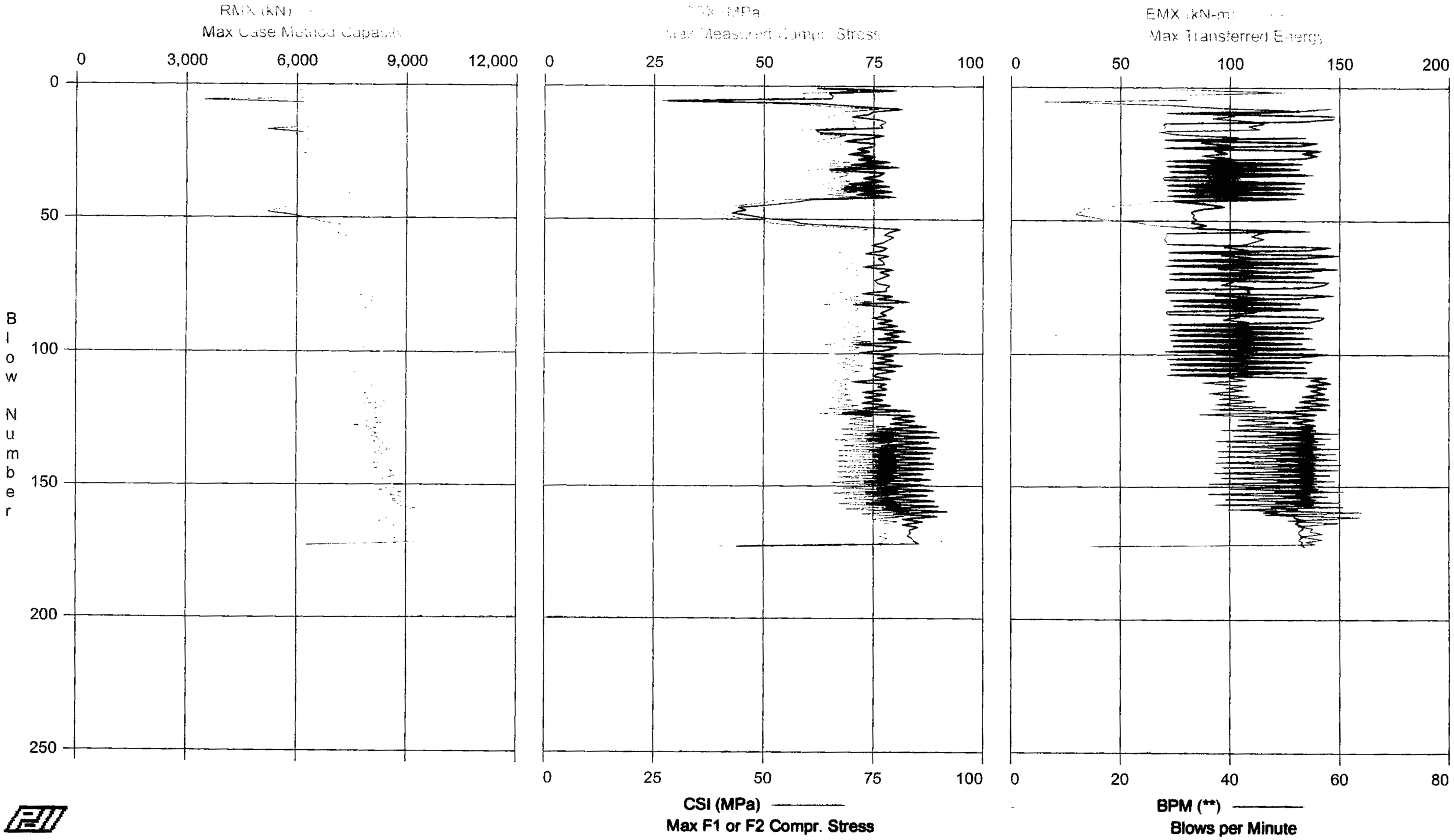


PDIPLOT Ver. 2005.1 - Printed: 3-Feb-2006

InSitu Tech, Ltd. - Case Method Results

Test date: 31-Jan-2006

San Francisco Bay Bridge T1 - Casing 3
IHC 500 Hydraulic

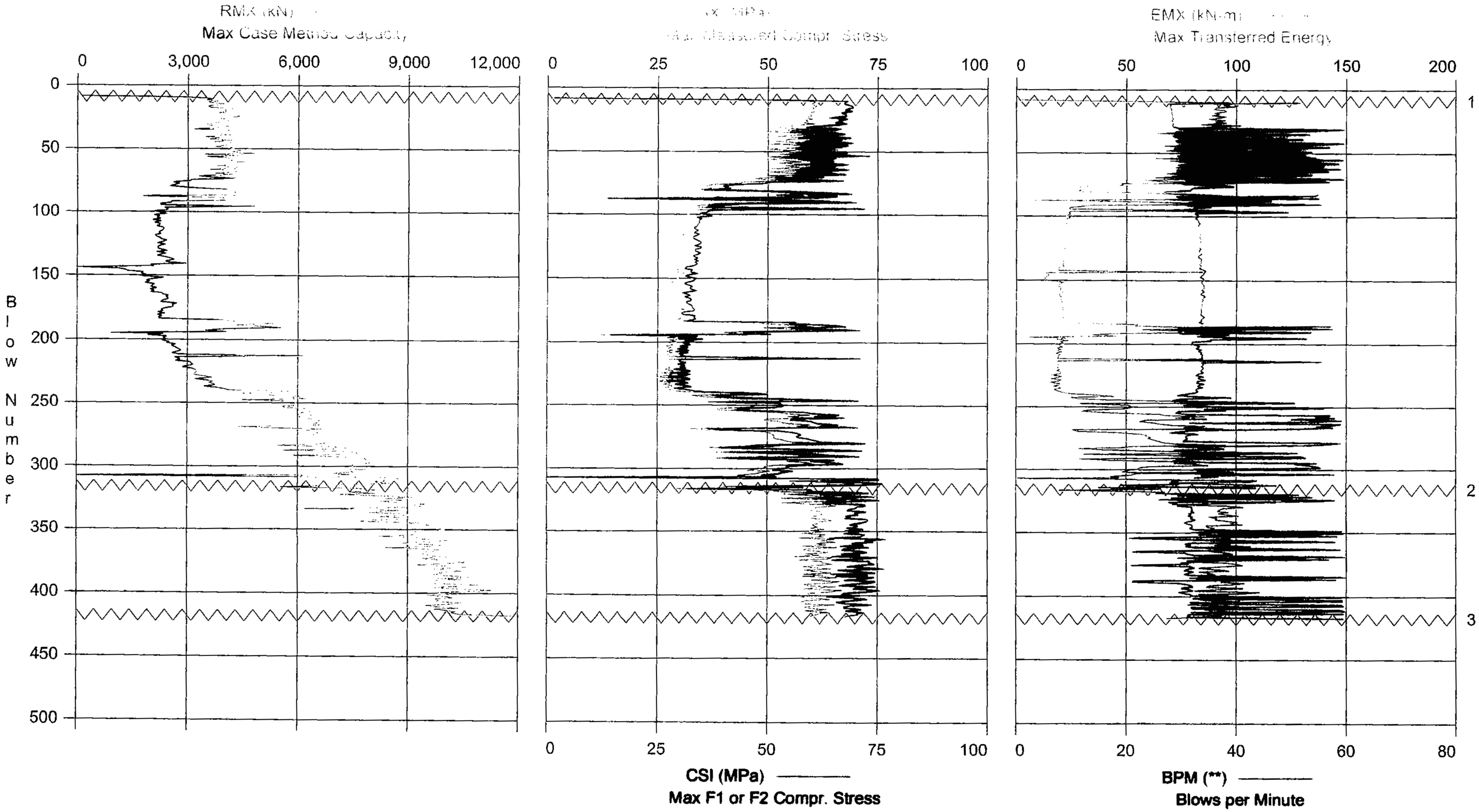


InSitu Tech, Ltd. - Case Method Results

PDIPLOT Ver. 2005.1 - Printed: 3-Feb-2006

Test date: 30-Jan-2006

San Francisco Bay Bridge T1 - Casing 13
IHC 500 Hydraulic



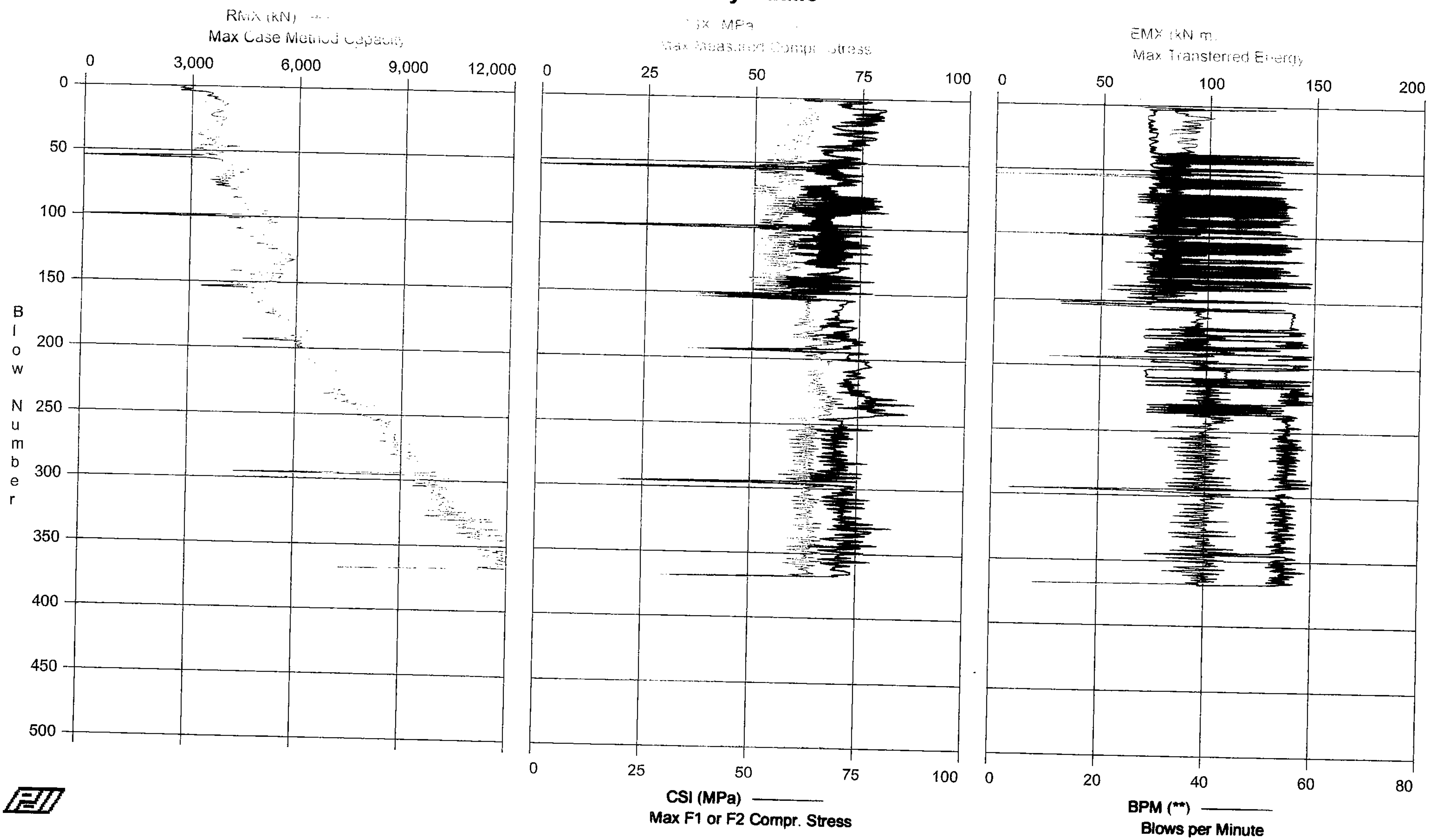
1 - Begin Monitoring 10:10am
2 - Stop 10:27am -- Resume Driving 12:36pm
3 - Complete Monitoring 12:40pm

PDIPLOT Ver. 2005.1 - Printed: 3-Feb-2006

InSitu Tech, Ltd. - Case Method Results

Test date: 31-Jan-2006

San Francisco Bay Bridge T1 - Casing 2
IHC 500 Hydraulic

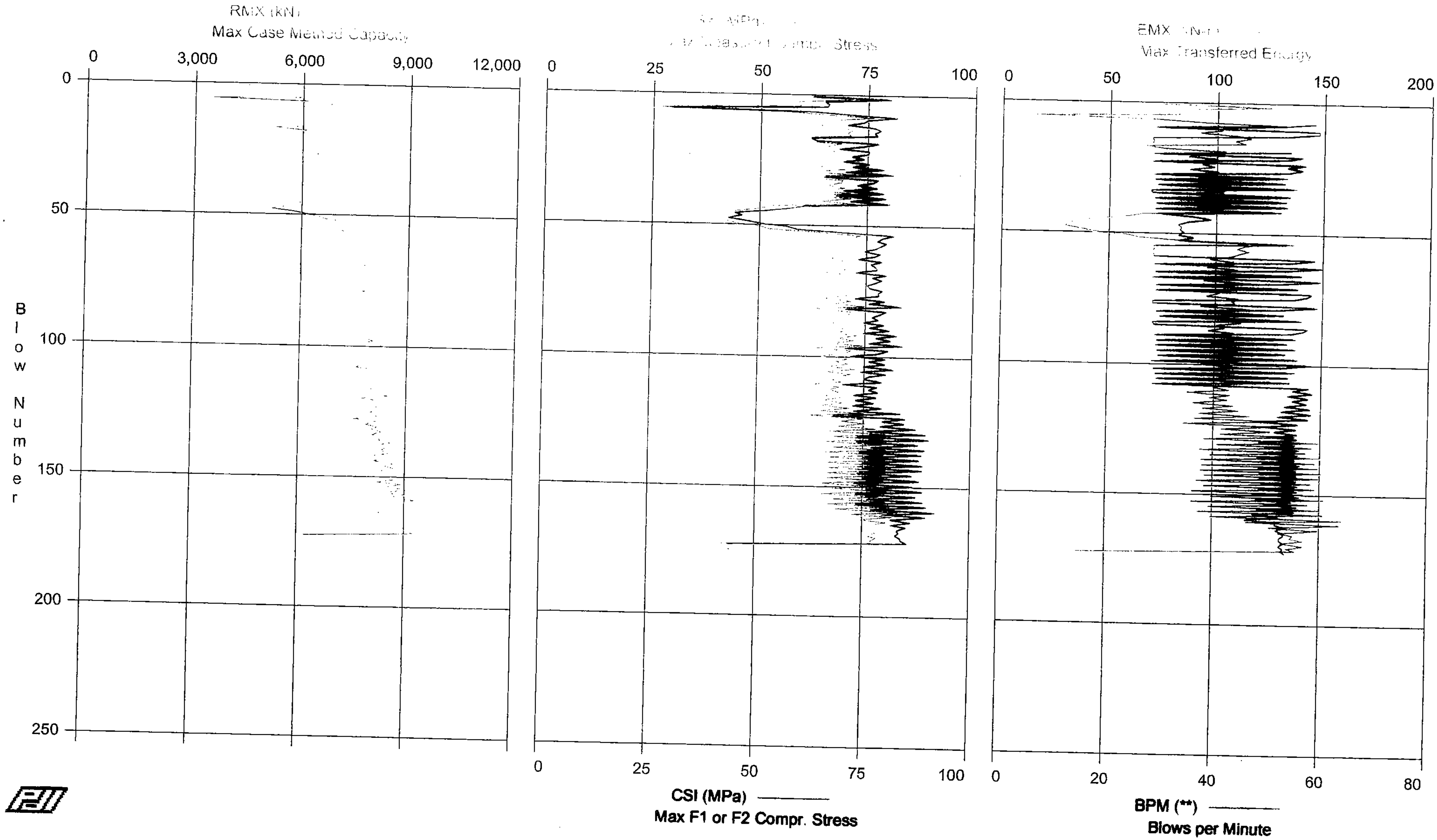


PDIPLOT Ver. 2005.1 - Printed: 3-Feb-2006

InSitu Tech, Ltd. - Case Method Results

Test date: 31-Jan-2006

San Francisco Bay Bridge T1 - Casing 3
IHC 500 Hydraulic

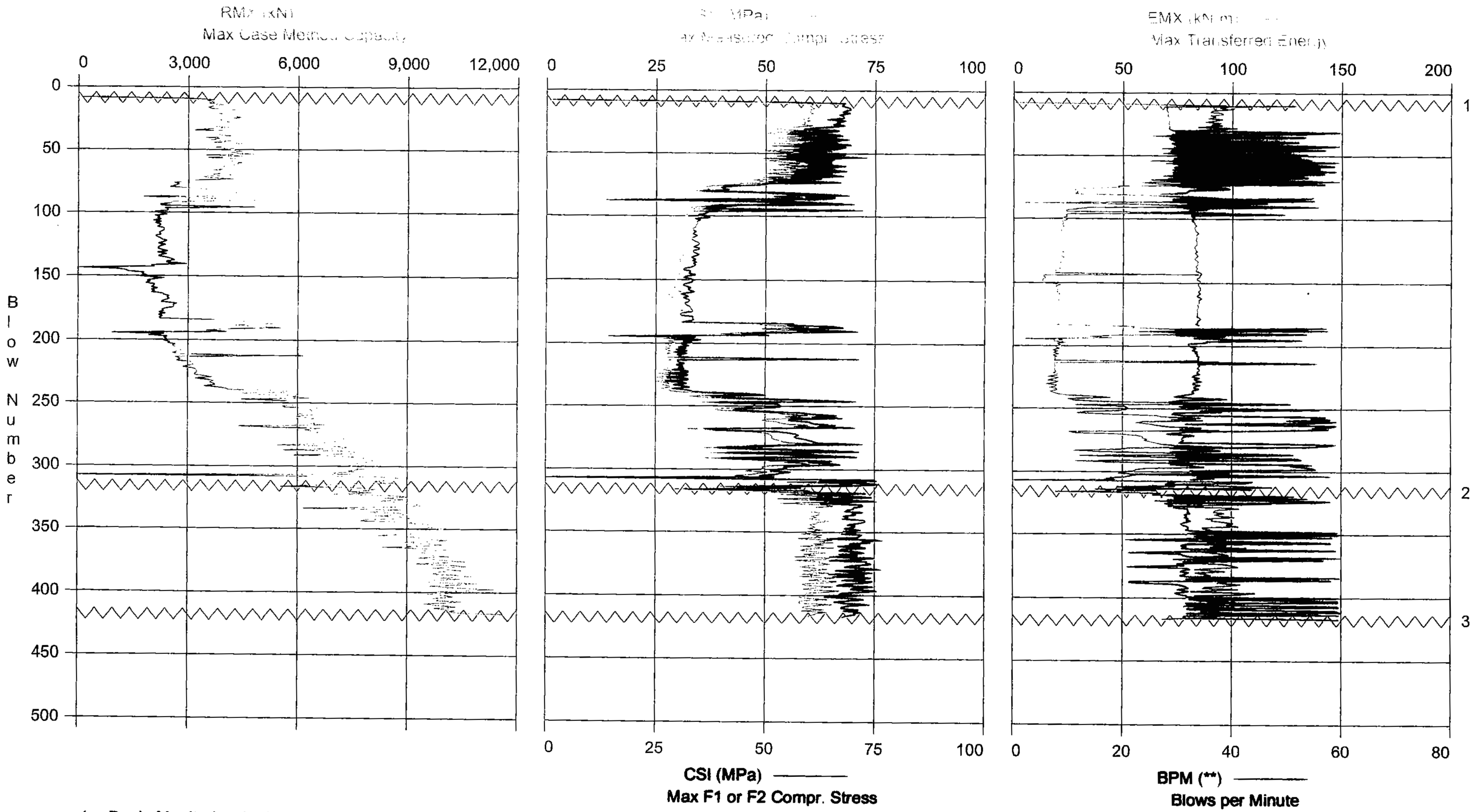


PDIPLOT Ver. 2005.1 - Printed: 3-Feb-2006

InSitu Tech, Ltd. - Case Method Results

Test date: 30-Jan-2006

San Francisco Bay Bridge T1 - Casing 13
IHC 500 Hydraulic



1 - Begin Monitoring 10:10am
2 - Stop 10:27am -- Resume Driving 12:36pm
3 - Complete Monitoring 12:40pm

T1 Temp Casing Installation Logs

Date: 1-30-06

Notes by: REUBEN ZYLSTRA

ELEVATION READ @ +4.8 MSL (NGVD)

Pile # 13

Notes:

9:30 AM - DEAD BLOW

- PLUMB IN BOTH DIRECTIONS
~~PLUMB~~ 0" IN 8'

1/2 m - 1ST DEAD BLOW

< 1/4 m - 2ND DEAD BLOW

< 1/4 m 3RD DEAD BLOW

TOTAL 卌 卌 ||||

FINISH DRIV

30

TIP -24.402

STAB TO → 22

Top of Template Meter Mark	Blows/ 0.25m	Time of Day	Hammer Energy
22	1		9:30 AM
.25	↓		
.50			
.75			
23			
.25			
.50			
.75	~~23~~		28kJ
24	~~21~~ 27		
.25	21		
.50	24		
.75	24		
25	25		
.25	31		
.50	29		
.75	30		
26	32		
.25	34		
.50	38		
.75	—		
27	36		
.25	44		
.50	59		
.75	60		
28	~~34~~ 69		10:30 AM / 12:15 PM
.25	65		
.50	62		
.75	33		12:45 PM
29			
.25			
.50			

80kJ (P

T1 Temp Casing Installation Logs

Date: 1-31-06

Notes by: Rewsew

Pile # 3

Top of Template Meter Mark	Blows/ 0.25m	Time of Day	Hammer Energy
24	1	8:30 AM	
.25	4		
24 .50	9		
.75	3		
25	~~13~~		
.25	13		
.50	13		
.75	14		
26	15		
.25	15		
.50	15		
.75	16		
27	19		
.25	19		
.50	19		
.75	18		
28	10 20		
.25	22		
.50	22		
.75	25		
29	37		
.25	51	9:00 AM	83 on Hammer
.50	8 10	11:31 AM / 12:48 PM	
.75	40		165kJ (PDA)
30	42		150-158 (Hammer)
.25	38		
.50	39	1:00 PM	
.75	20		
31			
.25			
.50			

Notes:

STAB TO →

~~24.75~~ 30.75
4.80

TIP 25.95 FINAL

E 14 3/16" 8
S 17

SURVEY TIP 26.197

30
- 4.8
25.2
24.5

8 15
~~24.80~~
4.80

TIP ~~24.70~~

RESTART DRIVING w/out RAM FRAME - CASING STARTED WALKING, (150 BLOWS TOTAL)

PUT RAM FRAME BACK ON - BEGIN DRIVING AGAIN

T1 Temp Casing Installation Logs

Date: 1-31-06

Notes by: Reuben

ELEVATION READ @ +4.8 MSL (NGVD)

Pile # 2	Top of Template Meter Mark	Blows/ 0.25m	Time of Day	Hammer Energy
Notes: STAB TO →	24	6	7:45 AM	
E 5	.25	9		
S 12	.50	7		
	.75	10		
	25	~~3~~ 12		
ASBUILT	.25	12		
W6 $\frac{1}{4}$" in 8'	.50	18		
S20	.75	17		
SURVEY -26.544mm	26	~~10~~ 17		
N S	.25	14		
	.50	15		
PUT PDA ON →	.75	16		
	27	17		
TEMPLATE	.25	17		
W3	.50	17		
S11	.75	17		
8 17 / 29.75 / - 4.80 / TIP 24.95 ; 30 15 / 31.5 / 4.8 / 26.7	28	16		
	.25	21		
	.50	20		
	.75	21		
	29	~~X~~ 23		
21 0 12	.25	30		
31.25	.50	38		
4.80	.75	~~23~~ ~~16~~ 39	8:15 AM / 10:00 AM	
TIP 26.45	30	~~38~~ 40		
	.25	~~4~~ 30		
	.50	37		
	.75	—		
TOP @ 31.25m	31	60	10:20 AM	80-85kJ (HAMMER)
→	.25	13		
	.50			

DPR - FYI
Please file in sub folder when complete

GEOENGINEERS

LETTER OF TRANSMITTAL

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

To: Frank Daams
Kiewit/FCI/Manson JV
P.O. Box 23223
Oakland, California 98623

Date: 2/8/2006

File: 10271-001-00

Regarding: E2/T1 Project - T1 Temporary Casing Friction Capacity

We are sending: ☑ Attached ☐ Under Separate Cover

Copies	Date	Description
1	2/8/2006	Memorandum

These are transmitted as checked below:

☑ For Your Use ☐ As Requested ☐ Returned

☐ For Review and Comment ☐ Other (see remarks)

We are sending via: US Mail

Remarks: Please call if you have questions.

Copy To: Meda Schultz
Kiewit/FCI/Manson JV
220 Burma Road
Oakland, California 94607

Signed: Amanda Welch 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: Frank Daams – Kiewit/FCI/Manson JV

FROM: Garry H. Squires, PE GHS

DATE: February 8, 2006

FILE: 10271-001-00

SUBJECT: E2/T1 Project – T1 Temporary Casing Friction Capacity

CC: Meda Schultz – Kiewit/FCI/Manson JV

This memorandum summarizes our comments and conclusions regarding expected vertical capacity of the 3m temporary casings for the T1 piers. The 13 casings will be driven through the overburden and fractured rock to planned tip elevations ranging between about Elevation -20.4m and Elevation -26.6m. Estimated embedment in fractured rock at these tip elevations is expected to range between about 2.1m and 9.2m. After driving to tip the overburden and fractured rock inside the casing will be removed and an oversize hole (3.3m diameter) will be extended below the casing tip elevation. The drilling equipment and tooling will be supported on the temporary casing during under reaming. Estimated total weight of the drilling equipment and casing is about 480 kips. You also provided test results of Pile Driving Analyzer (PDA) testing performed on 42-inch-diameter test piles at the T1 site in September 2004, and driving logs for 3 of the subject piles.

Based on our understanding of the site conditions and review of the PDA test data it appears there should be adequate frictional capacity on the outside of the temporary casing to support the drill, casing and tooling at most of the casing locations assuming they are driven to the estimated embedment into the fractured rock. However, it appears the embedments anticipated at the locations of casing numbers 9, 10 and 11 are significantly less than at the other casing locations. These casings therefore are likely to have significantly less frictional capacity. We estimate that casings 9 and 11 would have an ultimate capacity on the order of 480 to 500 kips. Casing 10 capacity could be substantially less than the required capacity unless additional embedment into the fractured rock is obtained. We can make a better estimate if provided with driving logs and PDA data (if available) for casings 9, 10 and 11.

We trust this provides the information that you require at this time. If you have any questions regarding the above, please call.

GHS:tt
TACO:\10\10271001\00\Finals\1027100100M11T1TempCasings.doc

GEOENGINEERS

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

LETTER OF TRANSMITTAL

To: Frank Daams
Kiewit/FCI/Manson JV
P.O. Box 23223
Oakland, California 98623

Date: 3/15/2006

File: 10271-001-00

Regarding: E2/T1 Project - T1 Temporary Casing Friction Capacity

We are sending: ☑ Attached ☐ Under Separate Cover

Copies	Date	Description
1	3/14/2006	Memorandum

These are transmitted as checked below:

☑ For Your Use ☐ As Requested ☐ Returned

☐ For Review and Comment ☐ Other (see remarks)

We are sending via: US Mail

Remarks: Please call if you have questions.

Copy To: Meda Schultz
Kiewit/FCI/Manson JV
220 Burma Road
Oakland, California 94607

SAS FOUNDATIONS E2/T1 PROJECT KIEWIT / FCI / MANSON, A JV	
DATE: 3/21/06	CO/JOB: 364-4347
ROUTED BY:	NO. 04-0120E4
TO:	SPECIAL NOTES:
INTERNAL KFM COPIES TO:	
EXTERNAL COPIES TO:	
SCANNED: Y N FILED TO:	

Signed: Amanda Mallet 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:	Frank Daams – Kiewit/FCI/Manson JV
FROM:	Garry H. Squires, PE GHS
DATE:	March 14, 2006
FILE:	10271-001-00
SUBJECT:	E2/T1 Project – T1 Temporary Casing Friction Capacity
CC:	Meda Schultz – Kiewit/FCI/Manson JV

This memorandum summarizes our comments regarding estimated shear strength and available side friction for temporary casings driven into the weathered rock at the location of the T1 piers based on review of information in the contract documents and Pile Driving Analyzer (PDA) test data provided for our review by KFM. GeoEngineers previously provided an opinion regarding friction capacity for the temporary casings in our February 8, 2006 memorandum. The 13 casings will be driven through the overburden and fractured rock to planned tip elevations ranging between about Elevation -20.4m and Elevation -26.6m. Estimated embedment in fractured rock at these tip elevations is expected to range between about 2.1m and 9.2m. You also provided test results of PDA testing performed on 42-inch-diameter test piles at the T1 site in September 2004, and driving logs for three of the subject piles.

We reviewed the boring logs for borings 98-21, 98-22, 98-23 and 98-24 available in the contract documents. The referenced borings typically describe weathered rock in the Elevation -20m to Elevation -26m depth range as interbedded sandstone and siltstone that varies from slightly to intensely weathered and fractured. Unconfined compressive strength data provided for samples within this approximate elevation range in boring 98-22 indicates a compressive strength in the 5,000 to 12,000 pounds per square inch range. Equivalent SPT N blow count values in the overlying soil unit (described as Rock Fragments) in the boring 98-22 log are typically around 40, which correlates to an estimated side friction of about 1.6 kips per square foot (ksf). Estimated unit friction data for the weathered rock based on CAPWAP analysis performed on 42-inch-diameter test piles installed in September 2004 ranged from about 3.4 to 7.5 ksf (combined inside and outside frictional resistance).

In our February 8, 2006 memorandum, we concluded that there should be adequate frictional capacity for all but three of the temporary casings to support the estimated loads at the design embedments. Considering all of the above it is our opinion there should have been at least 1 to 1.5 ksf of unit frictional capacity available for the portion of the temporary casings driven into the weathered rock, which should be adequate to support the casing with the applied loads.

We trust this provides the information that you require at this time. If you have any questions regarding the above, please call.

GHS:tt
TACO:\10\10271001\00\Finals\1027100100M12T1TempCasingfriction.doc

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1400 AG Bussum
The Netherlands

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F +31 - 35 698 94 95
E g.jonker@geodrive.nl
I www.geodrive.nl

Rabobank 37 22 39 641
CoC Nr. 32100320

KFM-JV,
220 Burma Road,
Oakland.
CA 94607
United States of America.
Attn. Mr. Frank Daams

Bussum, April 4, 2006.

Subj.: Bearing Capacity of 3000mm Diameter Driven Casings.

Frank,

Further to your request GeoDrive Technology BV, has analysed the expected outside frictional capacity of a 3000 mm diameter casing, driven into fragmented rock at the SFOBB East Span site. The construction of the permanent piles was to drive a temporary casing to into the harder rock, drill an oversized pilot hole into the rock and then grout the permanent pile in place.

The present calculations were based on information available during the bidding phase of the project, as e-mailed to us on March 30th, 2006. This comprised the boring logs of borings 98-21, 98-22, 98-23 and 98-24. Boring 98-23 is further outside the pier area as the other 3 and has therefore not been considered in this analysis.

All 3 boreholes show fragmented rock (Graywacke) overlain by very soft clay and underlain by Siltstone and Sandstone. The fragmented rock has a layer thickness varying from about 6m (20ft) to about 9m (30ft). At the location where this layer is thin (boring 98-22) it is underlain by Sandstone described as *'intensely weathered, moderately soft to moderately hard, intensely fractured.'*
This layer has a thickness of 3m and can be considered to be 'driveable' by an impact hammer.

The calculations on the frictional capacity of the casings have been carried out according to the API recommendations.
The friction angle in the fragmented rock (Graywacke) was taken as 45 degrees as recommended according to the attached sheet 'Strength Properties of Rock'.

The results of the calculations are presented as graphs showing the ultimate bearing capacity in friction as a function of penetration, from the mudline.
These capacities range from 230 to 320 metric tons and are thus sufficient for an operating load of approximately 240 tons at the time the project was bid.
Pile driving operations may further densify the fragmented rock layer resulting in higher unit friction values than based on API recommendations (maximum values at the toe of the casing in

above calculations were 1.1 to 1.4 ksf).

The above calculated frictional capacity can be verified by dynamic PDA measurements during the initial driving operations of the casings, before the casings are drilled out.

Conclusions:
Based on the results of the present calculations, taken into account that the assumed unit friction values can be checked during installation, it can be concluded that the selected construction method, from geotechnical point of view, is feasible.

Yours faithfully,
GeoDrive Technology BV,

Geert Jonker
Senior Geotechnical Engineer.

GeoDrive Technology

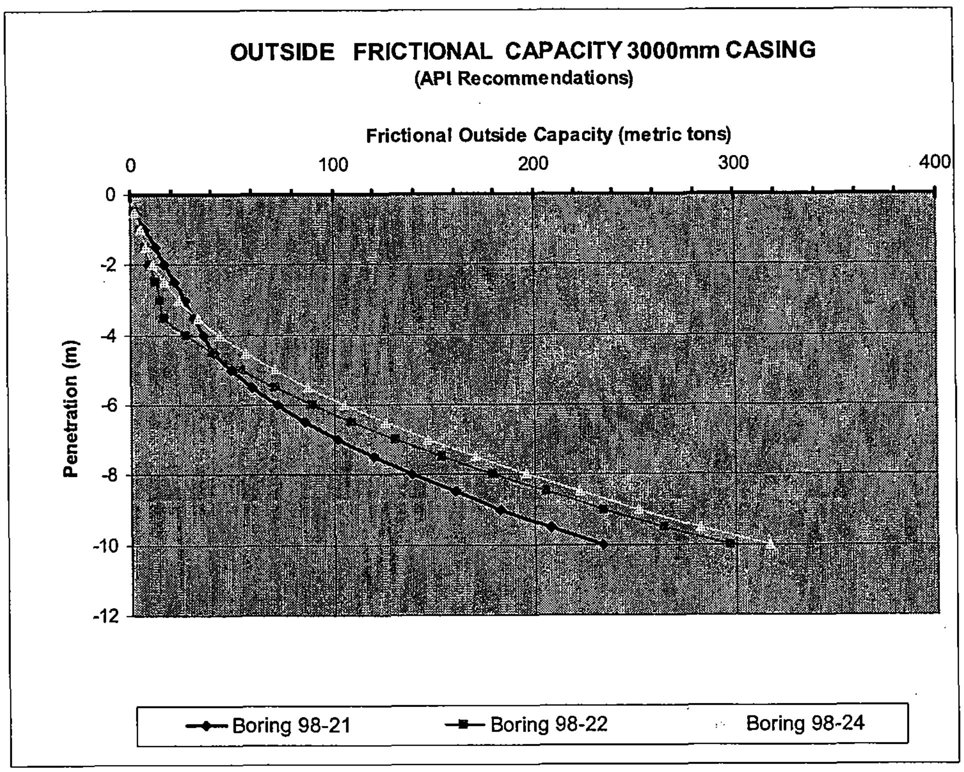
Figure 2.

Strength Properties of Rocks

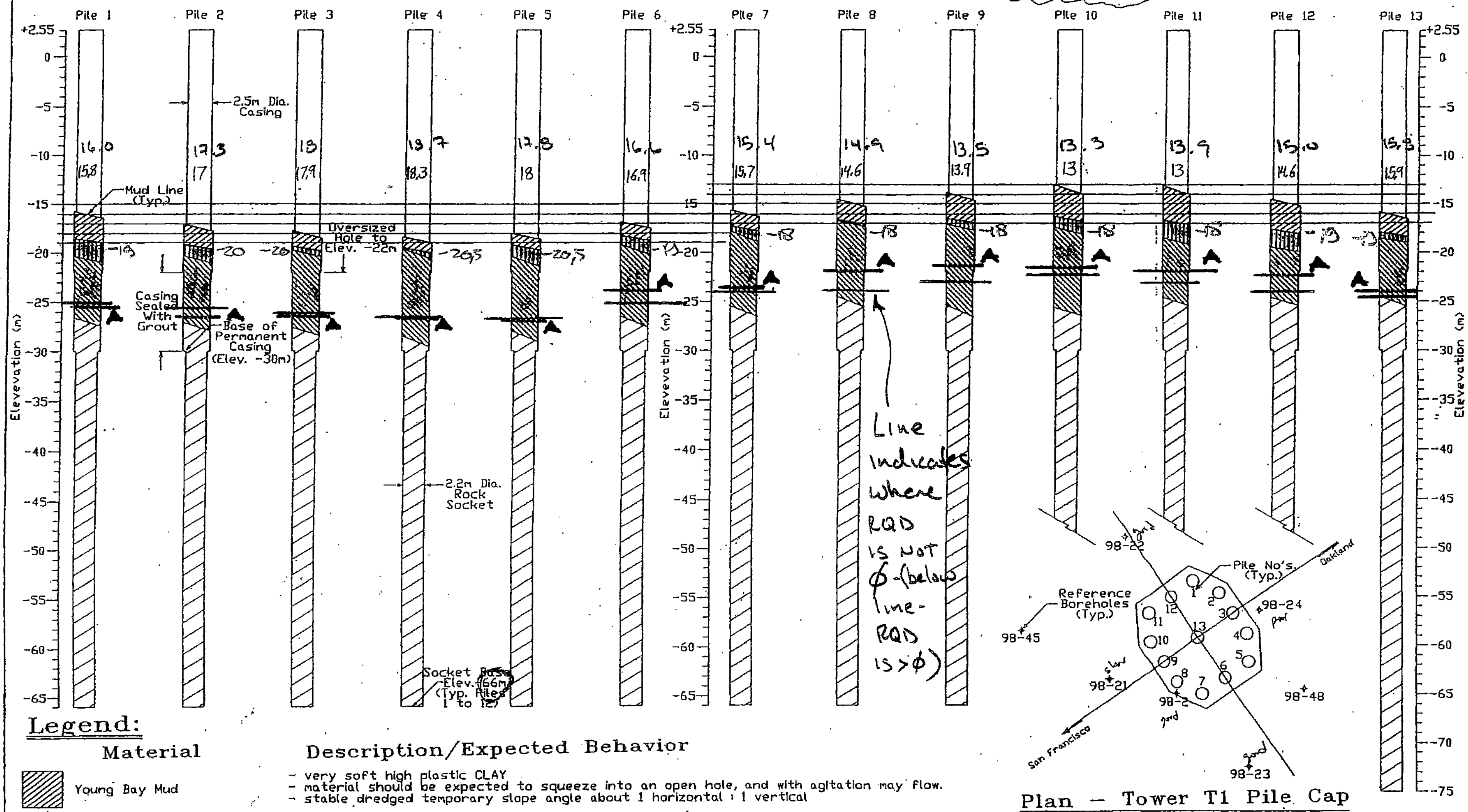
rock type	density dry t/m³	porosity %	dry UCS range MPa	dry UCS mean MPa	UCS saturated MPa	modulus of elasticity GPa	tensile strength MPa	shear strength MPa	friction angle φ°
Granite	2.7	1	50–350	200		75	15	35	55
Basalt	2.9	2	100–350	250		90	15	40	50
Greywacke	2.6	3	100–200	180	160	60	15	30	45
Sandstone – Carboniferous	2.2	12	40–100	70	50	30	5	15	45
Sandstone – Triassic	1.9	25	5–40	20	10	4	1	4	40
Limestone – Carboniferous	2.6	3	50–150	100	90	60	10	30	35
Limestone – Jurassic	2.3	15	15–70	25	15	15	2	5	35
Chalk	1.8	30	5–30	15	5	6	0.3	3	25
Mudstone – Carboniferous	2.3	10	10–50	40	20	10	1		30
Shale – Carboniferous	2.3	15	5–30	20	5	2	0.5		25
Clay – Cretaceous	1.8	30	1–4	2		0.2	2	0.7	20
Coal	1.4	10	2–100	30		10	2		
Gypsum	2.2	5	20–30	25		20	1		30
Salt	2.1	5	5–20	12		5			
Hornfels	2.7	1	200–350	250		80			40
Marble	2.6	1	60–200	100		60	10	32	35
Gneiss	2.7	1	50–200	150		45	10	30	30
Schist	2.7	3	20–100	60		20	2		25
Slate	2.7	1	20–250	90		30	10		25

These are mean or typical values, which can only be taken as approximate guidelines.
All values refer to intact rock which has not been weakened by weathering. Unquoted values indicate extreme variation related to orientation etc., or lack of adequate data.

Sedimentary rocks become stronger with age and tectonic stress; these values are typical for Britain and eastern USA; most rocks of similar ages are stronger in areas of plate boundary deformation such as Alpine Europe and western America.



— using high side for OG — (ave OG = <15.74>) per plans → <22> ⇒ 5.75



Plan — Tower T1 Pile Cap

Scale 1:200

Legend:

Material	Description/Expected Behavior
Young Bay Mud	- very soft high plastic CLAY - material should be expected to squeeze into an open hole, and with agitation may flow. - stable dredged temporary slope angle about 1 horizontal : 1 vertical
Gravel/Rock Fragments	- dense GRAVEL and weathered ROCK FRAGMENTS - only limited penetration likely with a vibratory hammer, may require impact hammer to drive a casing. - sloughing should be expected into an open hole, drilling would likely require a casing.
Intensely Fractured Bedrock	- weathered to moderately hard SANDSTONE and CLAYSTONE, intensely fractured - open hole drilling will present risk of collapse. - use fully cased excavation method or thick mud drilling.
Franciscan Formation	- hard SANDSTONE with siltstone and claystone interbeds - unconfined compresive strengths from about 5,000 to 30,000 psi, average about 15,000psi. - relative slow drilling with reverse circulation drill, mud not required.

—— A = ACTUAL TR.

KIEWIT ENGINEERING CO.
1000 KIEWIT PLAZA OMAHA NEBRASKA 68131

SCALE 1:400	DATE 3Mar03	DRAWN TJS	CHK BY JAM

SUBJECT Tower T1 Caissons Stratigraphic Cross Sections

PROJECT SFOBB, Skyway Bridge — DWG NO. G1

Survey/

2,3,4 <20> 8,9,10 <15.5> - <16.5>

5,6,7, <19>-<20> 11,12,1 <15.5> - <18.5>

F Daous 4/4/6.