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REVISED FINAL
OAKLAND SHORE APPROACH
GEOTECHNICAL SITE CHARACTERIZATION REPORT

SAN FRANCISCO-OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

VOLUME 4 - ADDITIONAL REPORTS
(Fugro-EM Project Memorandums and UC Berkeley Report)

Prepared for

CALIFORNIA DEPARTMENT OF TRANSPORTATION

March 2001

Earth Mechanics

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A JOINT VENTURE

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March 5, 2001
Project No. 98-42-0058/EMI Project No. 98-145

California Department of Transportation
Engineering Service Center
Office of Structural Foundations
5900 Folsom Boulevard
Sacramento, California 95819-0128

Attention: Mr. Mark Willian
Contract Manager

Final Oakland Shore Approach Geotechnical Site Characterization
SFOBB East Span Seismic Safety Project

Dear Mr. Willian:

The geologic and geotechnical studies for the San Francisco-Oakland Bay Bridge (SFOBB) East Span Seismic Safety Project are being conducted by Fugro-Earth Mechanics (a joint venture of Fugro West, Inc., and Earth Mechanics, Inc.) under California Department of Transportation (Caltrans) Contract 59A0053. The majority of the Oakland Mole exploration was conducted as part of the Phase 2, Task 5 (final site characterization phase) studies of the referenced contract, although three cone penetration test (CPT) soundings were conducted as part of the earlier Phase 1, Task 3 work scope (preliminary site characterization) of the referenced contract.

The field exploration conducted on the Oakland Mole in the Spring and Fall of 1998 as part of the referenced contract included 25 soil borings (designated in the subsurface database created for the project as 98-51 through 98-75) and 9 CPT soundings (designated as 98-16 through 98-18 and 98-101 through 98-110 [several planned locations were abandoned due to electrical interference]). A total of 53 offshore tethered Seascout CPT soundings were conducted in December 1998 during seasonal high tides on the tidal flat to the north of the Oakland Mole. Fifteen all-terrain CPT soundings were conducted on the Tidal Flat to the north of the Mole in March 1999 during low tide conditions. Two trench and two pit excavations were conducted in April 1999. In Fall 2000, 11 marine Seacalf CPT soundings were performed in the vicinity of the Oakland Shore Approach to the west and northwest of the western tip of the Mole. The Seacalf CPTs were conducted as part of the Phase 3 field investigation program.

This Oakland Shore Approach Geotechnical Site Characterization report summarizes the field exploration for the Oakland Shore Approach and describes our interpretation of the geotechnical conditions based on the information collected from the exploration. Our present interpretations are based on the field data from the borings, the results of laboratory tests completed in the onshore laboratories, and the CPT soundings.

This report is provided in four volumes. Volume 1 provides the interpretational text and illustrations. Volume 2 contains 27 appendices that provide boring logs and test data from 25 land borings on the Oakland Mole and 2 marine borings offshore from the northwest tip of the Oakland Mole. Volume 3 contains the results of the CPT soundings conducted on land, on the northern shore of the tidal flat, and offshore to the north and west of the Oakland Mole. Volume 4 contains four documents that include a preliminary study of approach fills at the Oakland Mole, studies on lateral spreading of fills at the Oakland Mole, findings from trench and pit excavations, and the advanced geotechnical laboratory studies conducted at the University of California at Berkeley.

On behalf of the project team, we appreciate the opportunity to contribute to Caltrans' design of the new bridge to replace the existing SFOBB East Span. Please call if we can answer any questions relative to the information presented in the enclosed report.

Sincerely,

FUGRO-EARTH MECHANICS, A Joint Venture

Roger Howard, Jr., P.E.
Project Engineer, Fugro West, Inc.

Jacob Chacko, P.E.
Project Engineer, Fugro West, Inc.

Mike Kapuskar, P.E.
Project Manager, Earth Mechanics, Inc.

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Vice President, Fugro West, Inc.

Attachment

Copies submitted:

Mr. Mark Willian, Caltrans
Mr. Saba Mohan, Caltrans
Mr. Robert Price, Caltrans
Dr. Brian Maroney, Caltrans
Ms. Sharon Naramore, Caltrans
Mr. Gerry Houlahan, TY Lin/M&N
Mr. Sajid Abbas, TY Lin/M&N
Mr. Al Ely, TY Lin/M&N

SAN FRANCISCO-OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
CALTRANS CONTRACT 59A0053

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VOLUME 4 - ADDITIONAL REPORTS

PROJECT MEMORANDUM
(DECEMBER 11, 1998)

PROJECT MEMORANDUM
PRELIMINARY STUDY OF APPROACH FILLS AT OAKLAND MOLE

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

MEMORANDUM

To: Dr. Brian Maroney / Caltrans
Mr. Ade Akinsanya / Caltrans
Mr. Reid Buell, Mr. John Thorne, Mr. Ronnie Gu / Caltrans
Mr. John Vitorelo / Caltrans
Mr. Hooshmand Nikoui / Caltrans

Copy: Mr. Al Ely, Mr. Gerry Houlahan / TYLin-Moffatt &Nichol
Mr. Jal Birdy / TYLin-Moffatt & Nichol
Mr. Tom McNeilan, Mr. Tim Dunne / Fugro-EMI
Mr. Tony Dover / Fugro-EMI

From: Hubert Law, Robert Pyke, Po Lam, M. Kapuskar / Fugro-EMI

Date: December 11, 1998

Subject: Preliminary Study of Approach Fills at Oakland Mole
SFOBB East-Span Replacement
Update with Response to Review Comments
Fugro-EMI T.O.#5 - Supplement 1

Gentlemen:

We are submitting this interim memo describing our preliminary study of the feasibility of using a wedge of fill on the north side of the Oakland Mole to support the west-bound approach. The purpose of this study is to determine the feasibility of the proposed fill option and to assist in a cost comparison of the slab-on-pile option versus the fill option. If the fill option is adopted by Caltrans, further studies will be needed to finalize design.

It has been recognized at the onset of the project that there are some key geotechnical issues (e.g. soil liquefaction, site stability and lateral spread) that need to be addressed which also require more quantitative evaluation. Presently, laboratory testing including consolidation testing and dynamic behavior testing of the soft bay mud soils are on-going which will aid these quantitative evaluations. Also, a special exploration program in the shallow waters at the north side of the Oakland Mole using the Fugro Seascout tethered CPT unit is scheduled for next week of December 14, 1998 which will provide key information needed for these evaluations.

Under our original contract, these items will be reported in a Draft Phase-2 Site Characterization Report to be submitted at the end of January or in February, 1999. In addition, more detailed pile survivability and pile capacity evaluations need to be conducted when pile design efforts of the structural designers have progressed further so that the pile type, pile details and loading demand are better defined and can be made available to Fugro-EMI. The issue of lateral spread and site stability was not meant to be an item to be addressed in this interim memo originally, because, to some extent they do not have a major impact on the cost of construction to either fill or structural alternative. The primary focus, from a geotechnical perspective was for EMI to assess the consolidation characteristics of the soft bay mud to appreciate the feasibility of the fill option at Oakland Mole and its impact on construction schedule for the Bay Bridge east span (which can impact cost).

The original memorandum on the present preliminary study of the Oakland Mole has been presented to Caltrans and the Civil and Structural Designers on November 20, 1998, which essentially established that it would be feasible (within the window of construction) to adopt a fill option for the approaches, and also provided some conceptual information for cost estimate evaluation by T.Y. Lin. Last Monday (December 9, 1998), we have received review comments from Mr. Al Ely of T.Y. Lin and Mr. John Vitrorelo and Mr. Hooshmand G-Nikoui of Caltrans District 4 regarding this memo. Many of the comments focused on the site stability and lateral spread issues which were not the original intent of the memorandum. In addition, we were requested to address the pros and cons of the two options in relation to the lateral spread issue. Since receipt of the above requests, we have stepped up our efforts to address site stability and lateral spread issues at the Oakland Mole. In the following update, we are providing additional geotechnical cross sections with shear strength and blow count data to Caltrans and the designers that can be used as the starting point to appreciate site stability issues (these section are preliminary and will be updated when Seascout tethered CPT data become available). Because of the dynamic nature of this topic and the fact that these issues are currently being discussed with various parties (Peer Review are scheduled for December 14), we have elected to provide the following update on the original memorandum to address the comments by T.Y. Lin and District 4 in part. Further comments regarding lateral spread issues and a list of the pros and cons for the two proposed design options will be provided shortly.

Construction of the Existing Mole

Based on Caltrans Annual Construction Reports, the construction of a northern extension to the then existing Key System mole started on Jan. 8, 1934. Figure 1 shows the proposed cross section of the northern extension. The westerly 2,000 feet was needed at an earlier date for building the approaches to the bridge proper and here fill was placed directly on the bay bottom. Elsewhere work began with dredging out about 10 ft of the mud from the bay bottom. Material for the new fill was dredged from the Oakland Outer Harbor. The new fill was built in three stages. The first-stage fill was placed to El. +3 ft. MLLW (0± ft MSL). Upon completion of the first-stage fill, a temporary bulkhead 4 ft in height was constructed during periods of low tide. This bulkhead served the dual purpose of reducing the amount of rock needed to retain further fill and retaining the second stage of the fill which was then placed to El. +7 ft. The second stage fill served as a roadway for the operation of the rock contractor. Core rock was moved into position beyond the temporary bulkhead to an elevation slightly above El. +7 ft. Face rock was then placed on the exposed face and the remainder of the core rock was placed to between El. +13 and +15 ft. This roughly shaped wall was then backed with sand to prevent excess leakage through it during the pumping of the third-stage fill. This final stage was placed to an elevation sufficiently above the grade line of El. +13 ft to allow for the anticipated settlement and consolidation that the fill would take in the years to come. The remainder of the face rock was then placed by dumping from the top of the fill. The dredged fill was completed on Dec. 28, 1934 with a total of 3,823,728 cubic yards having been pumped.

Settlement of Existing Fill

Figure 2 shows the settlement of the fill along the mole after 1, 5 and 20 years after the construction. It appears that the settlement varies almost directly with the thickness of the underlying Young Bay Mud. After 20 years, the average settlement is approximately 10 percent of the thickness of Young Bay Mud. The ultimate settlement could be more than that shown in Figure 2 because the settlement during the placement of fill was not recorded and the status of consolidation after 20 years is not known.

Failure of Mole Fill

During the construction of the mole fill north of the Toll Plaza in the winter of 1947-1948, the underlying mud failed. In order to understand the nature of the failure, a series of holes were bored through the area of failure. Figure 3 presents a cross section through this failed area. At the time of failure the fill sank at least 20 feet. In places the mud thrust itself upward through the previous sand fill.

Current Field Investigation

The current field investigation program on the Oakland Mole consists of 10 rotary wash borings and 6 completed CPT soundings as shown in the plan view map of Figure 4 and summarized in the soil cross section profile along the longitudinal direction of Figure 5B. Our review of the site data suggests that the site soil condition can be grouped into 4 segments (see Figure 4). The surficial site soil conditions (i.e. presence or absence of the Merritt sands) are affected by the presence of a paleochannel (Segment II) where the Merritt sands have been eroded and the Young Bay Mud would consequently be thickest. In Segments I and III, the Merritt sand is relatively thick, whereas in Segment IV, this layer is appears to thin out in the easterly direction. In the transverse direction, all the units generally dip in a northerly direction. Cross sections in the transverse direction, representative for each of the four segments are presented in Figures 5b through 5e.

The thickness of the existing fills is about 25 feet, and the thickness of Young Bay Mud is typically 30 ft in the area of interest. The soils investigation was recently completed during the week of Nov. 13. Soil samples retrieved from the Young Bay Mud are currently being tested and the results are not available at this time. Thus, our preliminary studies of settlement and consolidation were based on data available in the literature and our files. Further field data with particular relevance to edge stability will be obtained using a mini-cone rig.

Proposed Fill for West Bound Approach

A schematic construction sequence for fill placement is shown in Figure 6. Some schematic cross-sections for the west-bound approach fill is illustrated in Figures 6A and 6B. These figures are generic in nature and actual fill volume calculation will need to be based on the actual Caltrans topographic data which vary along the approach alignment. Figures 5B to 5E are provided (based on Caltrans topography data) to allow some appreciation of the actual fill volume at four selected station locations. Other schemes have been studied but this sequence represents the presently preferred scheme.

This scheme would involve the following steps:

1. Drive sheet piles to prevent unraveling of the existing mole during removal of the rock dike in Step 2;
2. Remove existing facing rock and dike;
3. Place geogrid and filter fabric, anchored to existing mole, with outer edge of filter fabric upturned to limit contamination of Bay waters;
4. Place first stage fill, topped by 1 foot thick gravel blanket that would provide both an all-weather working surface and drainage from the subsequent wick drain installation;
5. Install wick drains as indicated in Figures 6A and 6B;
6. Place second stage fill including surcharge;
7. Remove surcharge prior to roadway construction; and
8. Place rock on exterior face.

Provisions should be made in the estimate for monitoring the stability of the edge of the first stage fill and for monitoring the settlement of the surcharged fill.

We have examined settlements of the proposed fill for two cross sections, one at Sta. 88+80 and other at Sta. 90+00, as shown in Figures 7 and 8. At each section, six locations were selected for settlement analyses. We used a compression index (C_c) of 1.0 and a coefficient of consolidation (C_v) of 10 ft^2/year to compute the amount of settlement and the rate of settlement. The ultimate settlements shown in Figures 7 and 8 have been computed using the vertical stress increments that result from enlarging the existing fill. Tables 1 and 2 present the complete computation process. The ultimate settlement at the edge of the existing roadway is expected to be in the order of 6 inches.

Since the wick drains would be installed in a regular pattern, the rate of settlement can be estimated using the theory of radial consolidation for a single wick drain. Although the horizontal permeability is typically 2 to 5 times larger than the vertical permeability, we elected to use the same coefficient of consolidation in the radial direction as in the vertical direction. With this assumption we conservatively estimate that 90% primary consolidation would be reached within 1 year assuming that the wick drains are installed at 3-ft spacing. Because final consolidation is reached asymptotically, it would be advisable to add an additional 3 ft of surcharge above the final grade (as shown in Figures 6A and 6B). Removal of the surcharge at the end of the construction period would minimize further settlements. We understand that in fact 20 months might be available for the surcharge period. Surcharging for this period of time should eliminate most of future settlements due to primary consolidation. At the end of primary consolidation, there may still be secondary consolidation (which is more difficult to mitigate for a fill option). However, the amount of settlement associated with the secondary consolidation is typically at least one order of magnitude smaller than that of the primary consolidation. Further, any secondary consolidation would proceed at much the same rate as the secondary consolidation under the existing mole so the differential settlements would be minimal. Data from a nearby site (Watergate) suggests secondary consolidation rates on the order of ¼ inch per year.

Locations where the structure meets the roadway (which will exist for either the fill or slab approach alternative) are areas where concentration of differential displacements will occur and the designers would need to address this issue (e.g. use of approach slab, variable thickness light weight fills, etc. would be beneficial).

Proposed Fill for East Bound Approach

The settlement of the eastbound approach fill was estimated in a similar manner assuming use of ordinary-weight fill. Figure 9 shows the settlements for new fill assuming a maximum of 8 feet fill above the existing grade. The time for consolidation is the same as for the westbound fill using wick drains with a 3-ft spacing (i.e., 90% consolidation within 1 year). There should be no settlement associated with the primary consolidation where the fill touches down to the existing grade.

If lightweight fill such as cellular concrete were to be used, it would be possible to design the fill so that there is no net increase in vertical stress, and thus no primary consolidation would be expected. For example, using cellular concrete with a unit weight of 30 lb/ft^3 that is 4 times lighter than the existing fill, one could excavate 2 ft of the existing fill and replace it with 8 ft of lightweight fill.

The lightweight fill approach is technically the preferred option for the eastbound approach, and Caltrans has relevant experience using lightweight fill on several other projects. The settlement of the underlying clay is unavoidable if regular fill is used and this may be an important issue for the existing pavement because a major portion of the new abutment is situated at the existing westbound roadway.

ARS Criteria for Oakland Mole Slab on Pile Structures

The cost for the slab on pile structures will depend on the seismic loading criteria. Two modes of seismic loading need to be considered: (1) the conventional inertial structural load (i.e. ARS) and (2) kinematic earth pressure load induced by permanent ground movement. The later mode of loading relates to the displacement capacity of the piles (very often at the in-ground hinge zone) and specialized pile survivability analyses are needed to address this mode of loading. As discussed in the beginning of this memorandum, further follow-up studies are needed on this topic.

To develop the ARS criteria for the Oakland Mole, we have made use of the six sets of SEE motions (Earth Mechanics Seismic Ground Motion Report dated November 24, 1998) in site response analyses using the SHAKE program. For the small diameter piles (24-inch concrete piles from T.Y. Lin), force transfer from the ground to the structure will be at a shallow depth and freefield mudline motion would provide a good indication of the level of shaking for the slab-on-pile structure option at Oakland Mole. Figure 10 presents the spectra from the conducted site response analyses at Oakland Mole along with the recommended smooth ARS criteria. The Phase 1 ARS criteria adopted for design of the main bridge has also been shown for comparison. Figure 11 provides a tabulation of the corresponding ARS curve coordinates.

Attachments

(1) Excerpts from Annual Construction Reports, 1934, 1935, 1936, 1937
(2) Original Contract Documents for Dredger Fill, 1933

Table 1. Settlement Analysis at Oakland Mole West Bound Fill Approach at Station 88+80 11/13/98

		Control Location					
	Unit	1	2	3	4	5	6
Specific Gravity of Bay Mud, Gs		2.66	2.66	2.66	2.66	2.66	2.66
Saturated Unit Wt of Bay Mud	lb/ft^3	94.00	94.00	94.00	94.00	94.00	94.00
Compression Index, Cc		1.00	1.00	1.00	1.00	1.00	1.00
Coefficient of Consolidation (vert), Cv	ft^2/yr	10.00	10.00	10.00	10.00	10.00	10.00
Thickness of Bay Mud	ft	30.00	30.00	30.00	30.00	30.00	30.00
Depth Below Original Mudline	ft	20.00	20.00	20.00	20.00	20.00	20.00
Effective Unit wt of Bay Mud	lb/ft^3	31.60	31.60	31.60	31.60	31.60	31.60
Initial Vertical Effective Stress before Exist Fill	lb/ft^2	632	632	632	632	632	632
Unit Wt of Fill x Exist Fill Height	lb/ft^2	1906	1906	1906	1906	1906	1906
Normalized Stress under Fill		0	0	0.05	0.1	0.6	0.85
Stress Increment Due to Existing Fill	lb/ft^2	0	0	95	191	1144	1620
Stress at End of Existing Fill Construction	**lb/ft^2**	**632**	**632**	**727**	**823**	**1776**	**2252**
Initial Void Ratio Before Existing Fill		1.92	1.92	1.92	1.92	1.92	1.92
Final Void Ratio After Existing Fill		1.92	1.92	1.86	1.81	1.47	1.37
Ultimate Settlement due to Existing Fill	ft	0.00	0.00	0.63	1.18	4.61	5.67
Settlement in 1 year	ft	0.000	0.000	0.075	0.141	0.553	0.680
Settlement in 2 year	ft	0.000	0.000	0.107	0.200	0.784	0.964
Settlement in 6 year	ft	0.000	0.000	0.169	0.318	1.244	1.531
Settlement in 21 year	ft	0.000	0.000	0.363	0.682	2.673	3.289
Unit Wt of Fill x New & Exist Fill Height	lb/ft^2	1906	1906	1906	1906	1906	1906
Normalized Stress under New & Exist Fill		0.1	0.6	0.85	0.9	0.95	0.95
Stress Increment Due to New & Exist Fill	lb/ft^2	191	1144	1620	1715	1811	1811
Stress at End of New Fill Construction	**lb/ft^2**	**823**	**1776**	**2252**	**2347**	**2443**	**2443**
Initial Void Ratio Before New Fill		1.92	1.92	1.86	1.81	1.47	1.37
Final Void Ratio After New Fill		1.81	1.47	1.37	1.35	1.33	1.33
Ultimate Settlement due to New Fill	**ft**	**1.18**	**4.61**	**5.15**	**4.87**	**1.68**	**0.45**
Settlement in 3 months	ft	0.588	2.305	2.575	2.435	0.841	0.223
Settlement in 6 months	ft	0.941	3.687	4.121	3.896	1.345	0.358
Settlement in 9 months	ft	1.058	4.148	4.636	4.383	1.513	0.402
Settlement in 12 months	ft	1.082	4.240	4.739	4.480	1.547	0.411
Settlement in 15 months	ft	1.117	4.379	4.893	4.626	1.597	0.425

T	U
0.011	0.12
0.022	0.17
0.067	0.27
0.233	0.58

Wick Drain Spacing
3 ft

T	U
0.278	0.5
0.556	0.8
0.833	0.9
1.111	0.92
1.389	0.95

Table 2. Settlement Analysis at Oakland Mole West Bound Fill Approach at Station 90+00 11/13/98

		Control Location					
	Unit	1	2	3	4	5	6
Specific Gravity of Bay Mud, Gs		2.66	2.66	2.66	2.66	2.66	2.66
Saturated Unit Wt of Bay Mud	lb/ft^3	94.00	94.00	94.00	94.00	94.00	94.00
Compression Index, Cc		1.00	1.00	1.00	1.00	1.00	1.00
Coefficient of Consolidation (vert), Cv	ft^2/yr	10.00	10.00	10.00	10.00	10.00	10.00
Thickness of Bay Mud	ft	30.00	30.00	30.00	30.00	30.00	30.00
Depth Below Original Mudline	ft	20.00	20.00	20.00	20.00	20.00	20.00
Effective Unit wt of Bay Mud	lb/ft^3	31.60	31.60	31.60	31.60	31.60	31.60
Initial Vertical Effective Stress before Exist Fill	lb/ft^2	632	632	632	632	632	632
Unit Wt of Fill x Exist Fill Height	lb/ft^2	1906	1906	1906	1906	1906	1906
Normalized Stress under Fill		0	0.05	0.1	0.45	0.65	0.85
Stress Increment Due to Existing Fill	lb/ft^2	0	95	191	858	1239	1620
Stress at End of Existing Fill Construction	**lb/ft^2**	**632**	**727**	**823**	**1490**	**1871**	**2252**
Initial Void Ratio Before Existing Fill		1.92	1.92	1.92	1.92	1.92	1.92
Final Void Ratio After Existing Fill		1.92	1.86	1.81	1.55	1.45	1.37
Ultimate Settlement due to Existing Fill	ft	0.00	0.63	1.18	3.83	4.84	5.67
Settlement in 1 year	ft	0.000	0.075	0.141	0.459	0.581	0.680
Settlement in 2 year	ft	0.000	0.107	0.200	0.650	0.823	0.964
Settlement in 6 year	ft	0.000	0.169	0.318	1.033	1.307	1.531
Settlement in 21 year	ft	0.000	0.363	0.682	2.219	2.809	3.289
Unit Wt of Fill x New & Exist Fill Height	lb/ft^2	1906	1906	1906	1906	1906	1906
Normalized Stress under New & Exist Fill		0.05	0.2	0.65	0.85	0.87	0.9
Stress Increment Due to New & Exist Fill	lb/ft^2	95	381	1239	1620	1658	1715
Stress at End of New Fill Construction	**lb/ft^2**	**727**	**1013**	**1871**	**2252**	**2290**	**2347**
Initial Void Ratio Before New Fill		1.92	1.86	1.81	1.55	1.45	1.37
Final Void Ratio After New Fill		1.86	1.72	1.45	1.37	1.36	1.35
Ultimate Settlement due to New Fill	**ft**	**0.63**	**1.51**	**3.82**	**2.11**	**1.08**	**0.23**
Settlement in 3 months	ft	0.313	0.755	1.908	1.057	0.538	0.114
Settlement in 6 months	ft	0.501	1.209	3.053	1.691	0.861	0.182
Settlement in 9 months	ft	0.564	1.360	3.434	1.902	0.968	0.205
Settlement in 12 months	ft	0.577	1.390	3.511	1.945	0.990	0.210
Settlement in 15 months	ft	0.595	1.435	3.625	2.008	1.022	0.217

T	U
0.011	0.12
0.022	0.17
0.067	0.27
0.233	0.58

Wick Drain Spacing
3 ft

T	U
0.278	0.5
0.556	0.8
0.833	0.9
1.111	0.92
1.389	0.95

Table 3. Settlement Analysis at Oakland Mole East Bound Fill Approach at Station 87+00 11/13/98

		Control Location					
	Unit	1	2	3	4	5	6
Specific Gravity of Bay Mud, Gs		2.66	2.66	2.66	2.66	2.66	2.66
Saturated Unit Wt of Bay Mud	lb/ft^3	94.00	94.00	94.00	94.00	94.00	94.00
Compression Index, Cc		1.00	1.00	1.00	1.00	1.00	1.00
Coefficient of Consolidation (vert), Cv	ft^2/yr	10.00	10.00	10.00	10.00	10.00	10.00
Thickness of Bay Mud	ft	30.00	30.00	30.00	30.00	30.00	30.00
Depth Below Original Mudline	ft	20.00	20.00	20.00	20.00	20.00	20.00
Effective Unit wt of Bay Mud	lb/ft^3	31.60	31.60	31.60	31.60	31.60	31.60
Initial Vertical Effective Stress before Exist Fill	lb/ft^2	632	632	632	632	632	632
Unit Wt of Fill x Exist Fill Height	lb/ft^2	1906	1906	1906	1906	1906	1906
Normalized Stress under Fill		0.03	0.1	0.55	0.8	0.9	0.9
Stress Increment Due to Existing Fill	lb/ft^2	57	191	1048	1525	1715	1715
Stress at End of Existing Fill Construction	**lb/ft^2**	**689**	**823**	**1680**	**2157**	**2347**	**2347**
Initial Void Ratio Before Existing Fill		1.92	1.92	1.92	1.92	1.92	1.92
Final Void Ratio After Existing Fill		1.88	1.81	1.50	1.39	1.35	1.35
Ultimate Settlement due to Existing Fill	ft	0.39	1.18	4.36	5.48	5.85	5.85
Settlement in 1 year	ft	0.046	0.141	0.524	0.657	0.703	0.703
Settlement in 2 year	ft	0.066	0.200	0.742	0.931	0.995	0.995
Settlement in 6 year	ft	0.104	0.318	1.178	1.479	1.581	1.581
Settlement in 21 year	ft	0.224	0.682	2.531	3.177	3.396	3.396
Unit Wt of Fill x New & Exist Fill Height	lb/ft^2	1906	1906	1906	1906	1906	1906
Normalized Stress under New & Exist Fill		0.1	0.65	0.8	0.9	0.9	0.9
Stress Increment Due to New & Exist Fill	lb/ft^2	191	1239	1525	1715	1715	1715
Stress at End of New Fill Construction	lb/ft^2	823	1871	2157	2347	2347	2347
Unit Wt of Fill x Abutment Height	lb/ft^2	880	880	880	880	880	880
Normalized Stress under Abutment Fill		0	0	0.1	0.65	0.65	0.25
Stress Increment Due to Abutment Fill	lb/ft^2	0	0	88	572	572	220
Stress at End of New & Abutment Fill Constr	**lb/ft^2**	**823**	**1871**	**2245**	**2919**	**2919**	**2567**
Initial Void Ratio Before New Fill		1.88	1.81	1.50	1.39	1.35	1.35
Final Void Ratio After New Fill		1.81	1.45	1.37	1.26	1.26	1.31
Ultimate Settlement due to New Fill	**ft**	**0.80**	**3.82**	**1.51**	**1.65**	**1.21**	**0.50**
Settlement in 3 months	ft	0.400	1.908	0.756	0.826	0.604	0.248
Settlement in 6 months	ft	0.640	3.053	1.210	1.322	0.967	0.397
Settlement in 9 months	ft	0.720	3.434	1.361	1.487	1.088	0.447
Settlement in 12 months	ft	0.736	3.511	1.391	1.520	1.112	0.457
Settlement in 15 months	ft	0.760	3.625	1.437	1.570	1.149	0.472

T	U
0.011	0.12
0.022	0.17
0.067	0.27
0.233	0.58

Wick Drain Spacing
3 ft

T	U
0.278	0.5
0.556	0.8
0.833	0.9
1.111	0.92
1.389	0.95

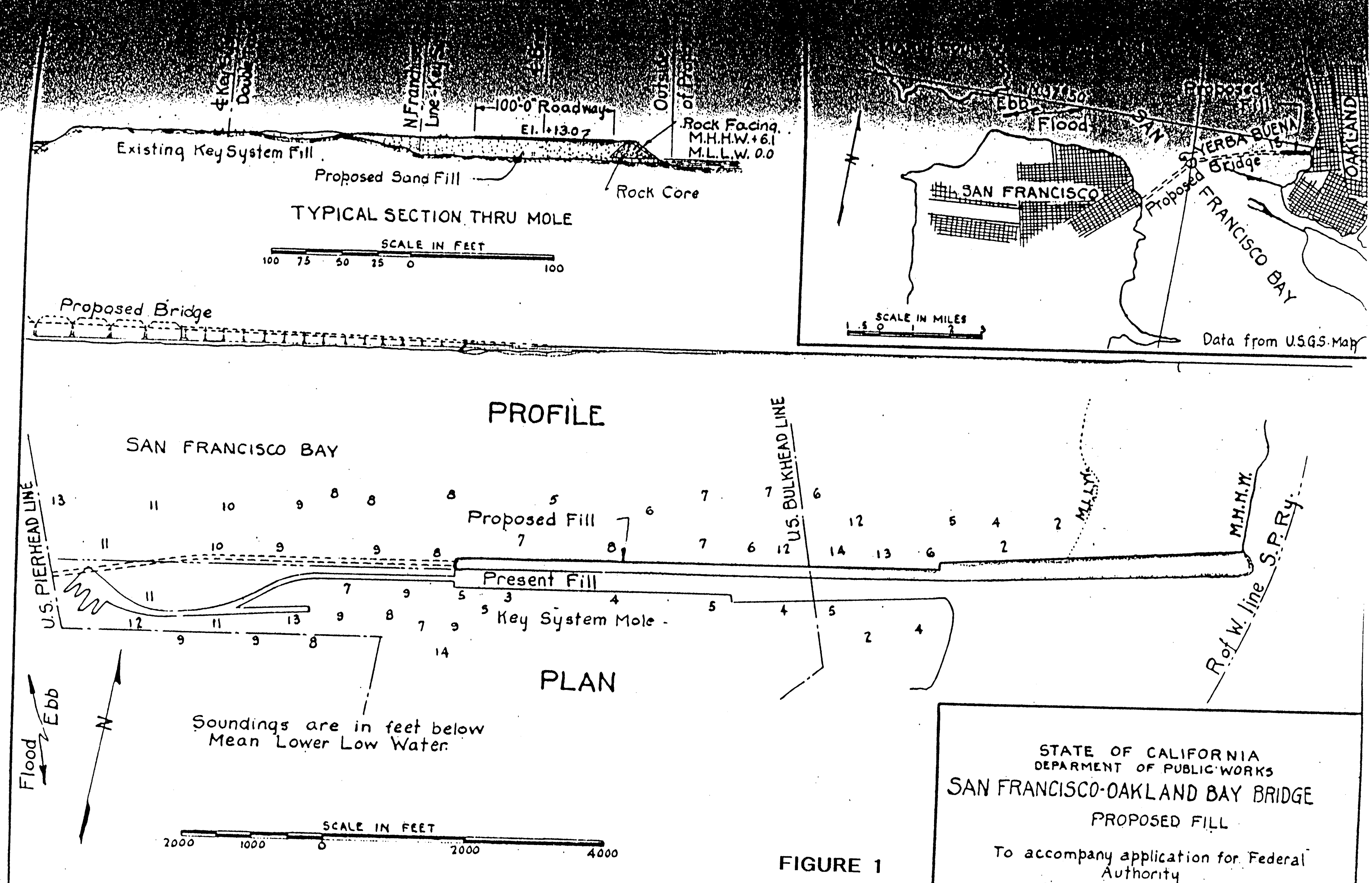


FIGURE 1

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
SAN FRANCISCO-OAKLAND BAY BRIDGE
PROPOSED FILL

To accompany application for Federal Authority
Dated August 26, 1932

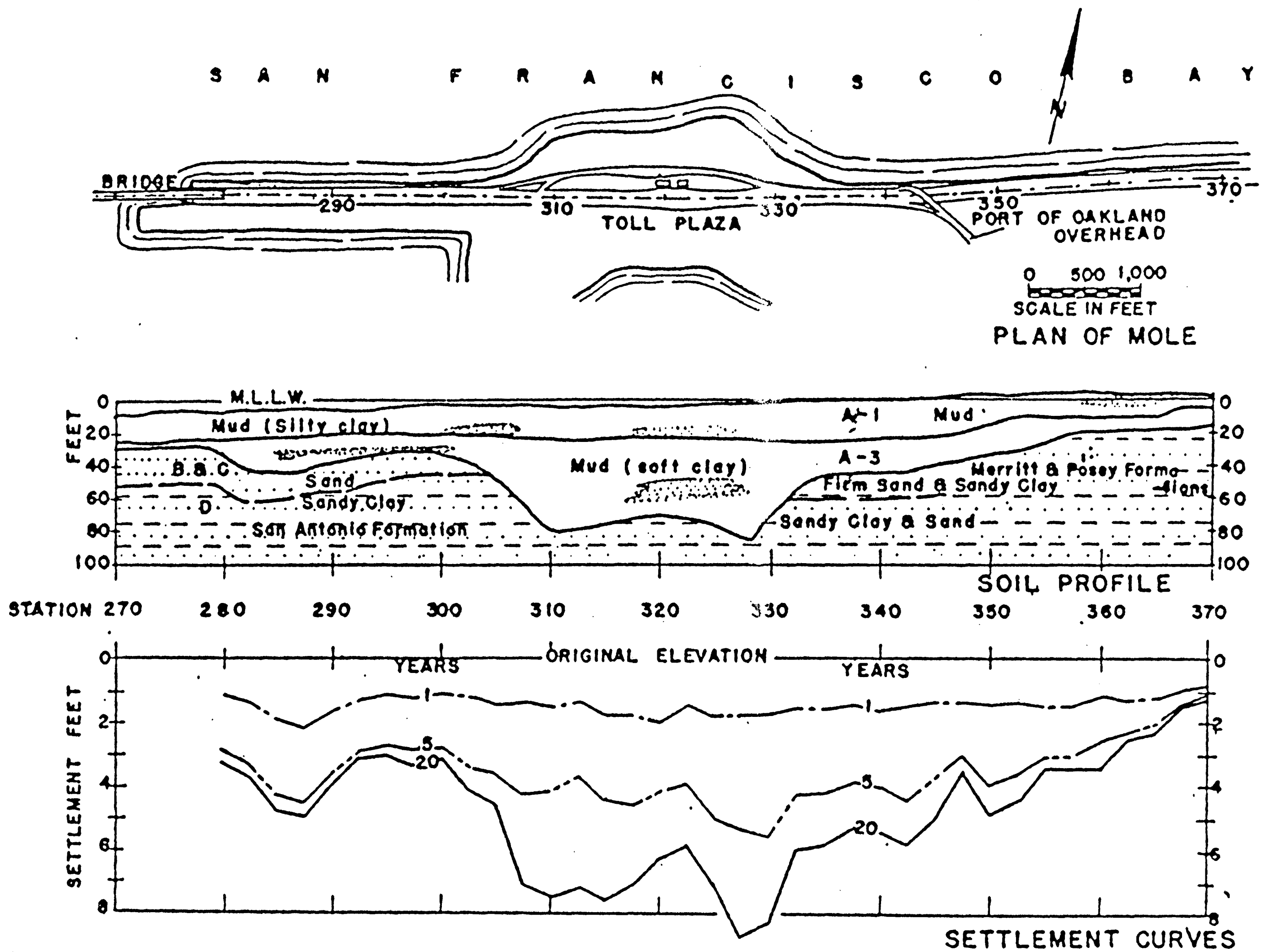


FIGURE 2 —SOIL PROFILE AND SETTLEMENT CURVES, EAST BAY MOLE, SAN FRANCISCO-OAKLAND BAY BRIDGE

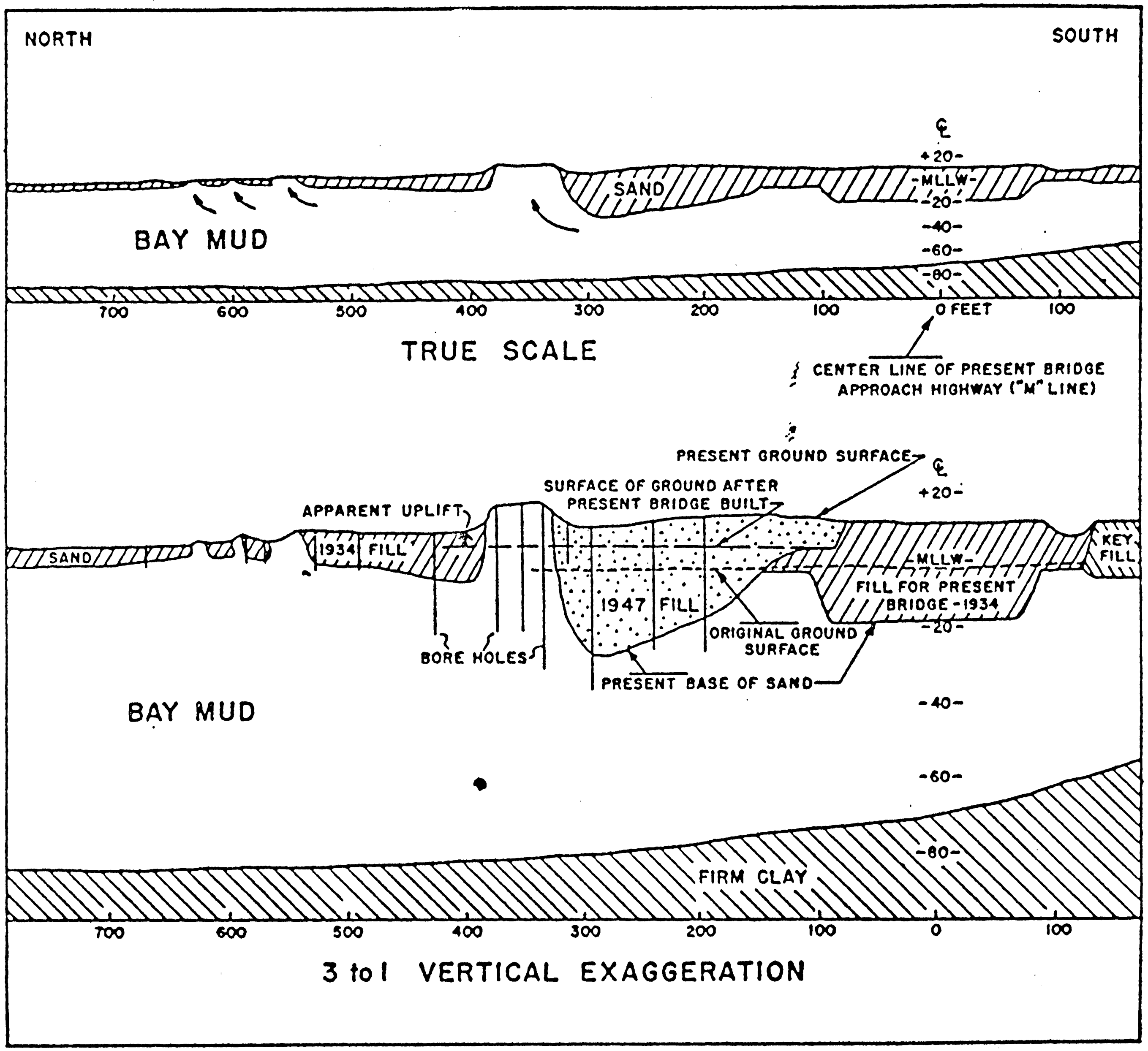


FIGURE 3 .—PROFILE OF SEDIMENTS NEAR PLACE OF FAILURE, TOLL PLAZA, SAN FRANCISCO-OAKLAND BAY BRIDGE

Scale in feet.

AS-DRILLED BOREHOLE AND CPT LOCATIONS
OAKLAND MOLE

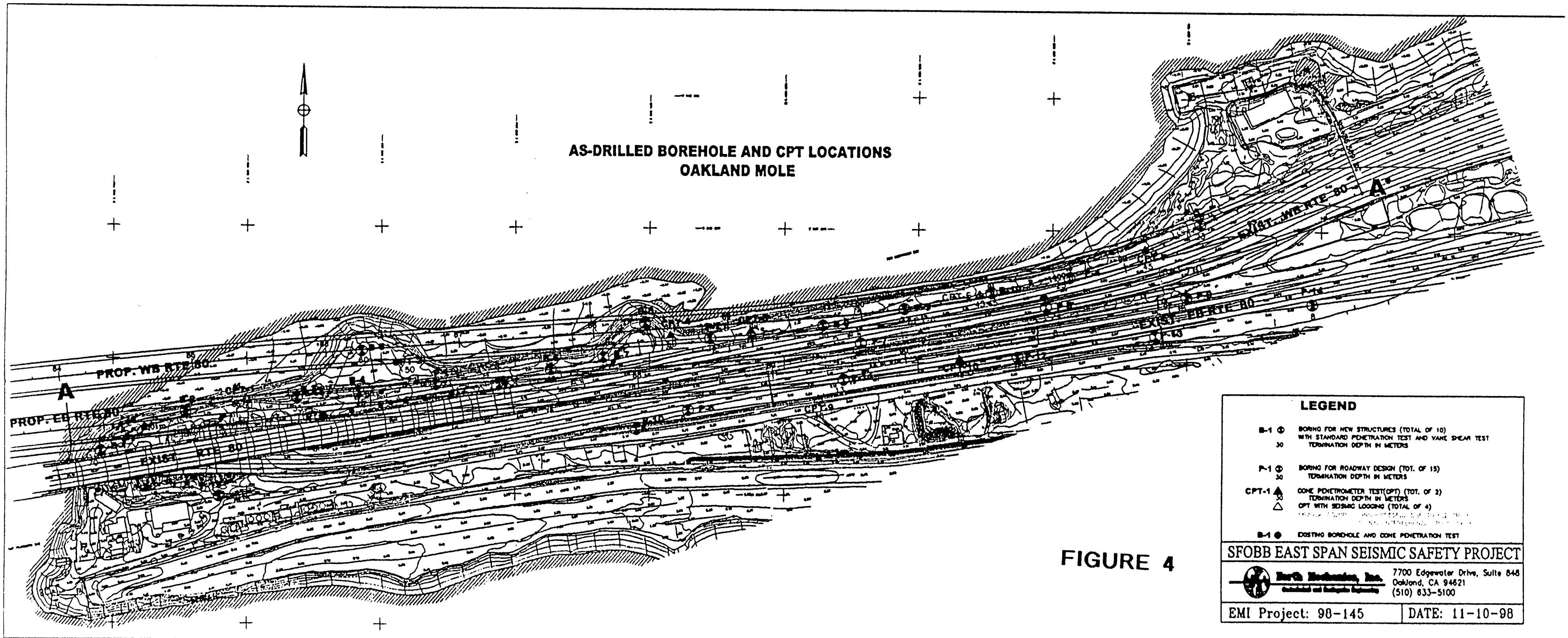
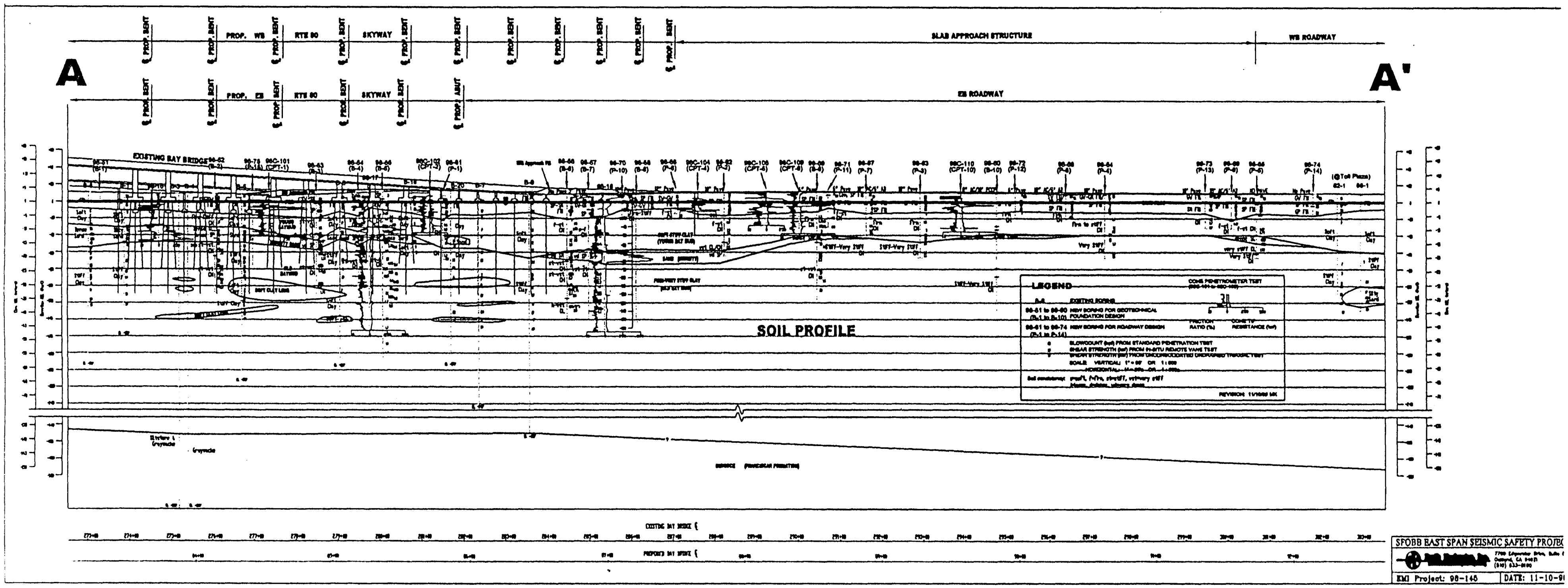
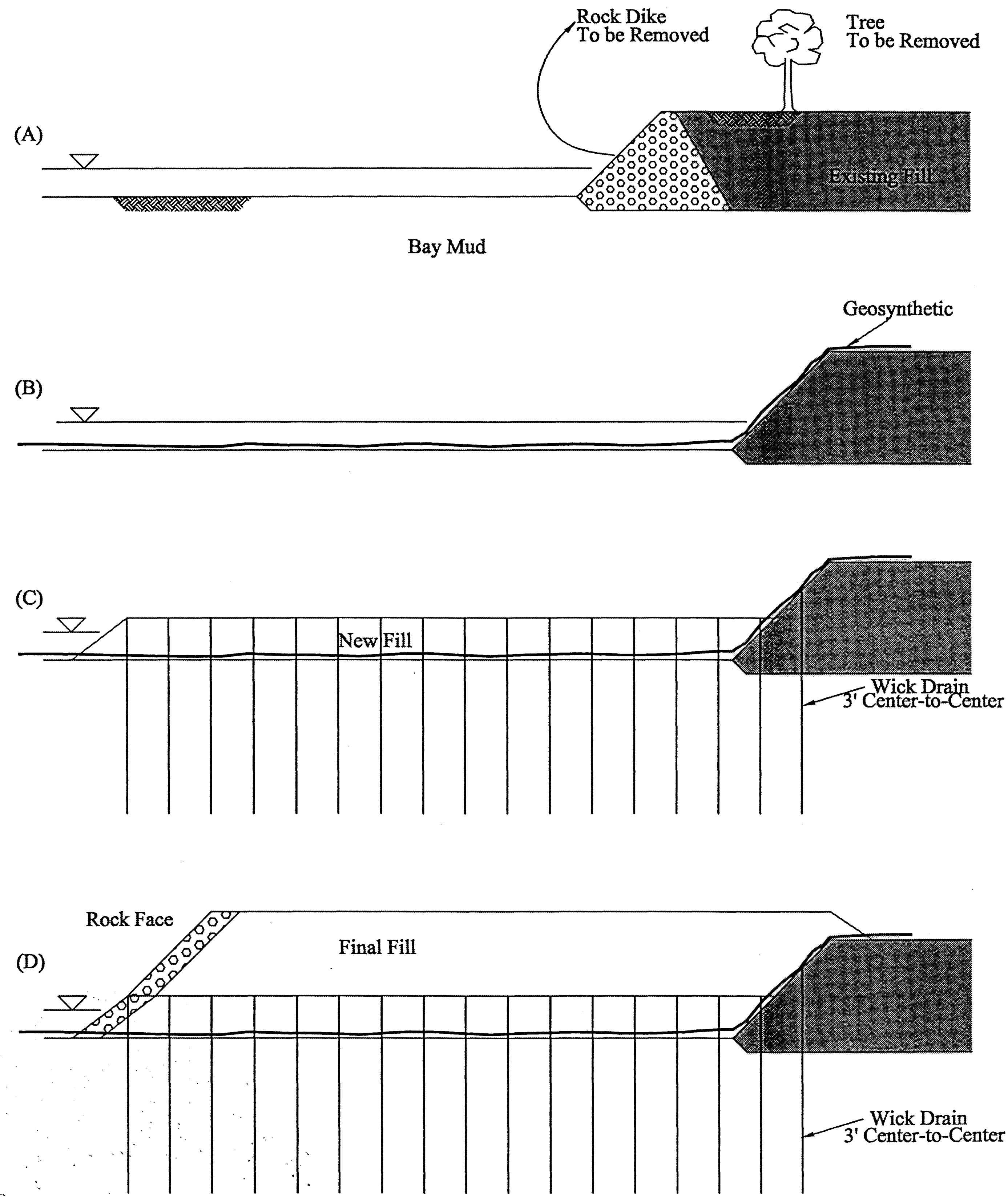


FIGURE 4



A
A'
SLAB APPROACH STRUCTURE
WB ROADWAY
EB ROADWAY
SKYWAY
EXISTING BAY BRIDGE
SOIL PROFILE
LEGEND
SFOBB EAST SPAN SEISMIC SAFETY PROJEC
EMI Project: 98-145
DATE: 11-10-9

FIGURE 5



NEW S.F.-OAKLAND BAY BRIDGE	POSSIBLE CONSTRUCTION SEQUENCE FIGURE 6
Earth Mechanics, Inc. Geotechnical & Earthquake Engineering	Project No. 98-107 Date: 11-10-98

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering
17660 Newhope Street, Suite E, Fountain Valley, CA 92708
(714) 751-3826, Fax (714) 751-3928

Project SFOBB. OAKLAND MOLE Project No. ______

By Hubert Date 12-2-98 Checked By ______ Date ______ Sheet ____ of ____

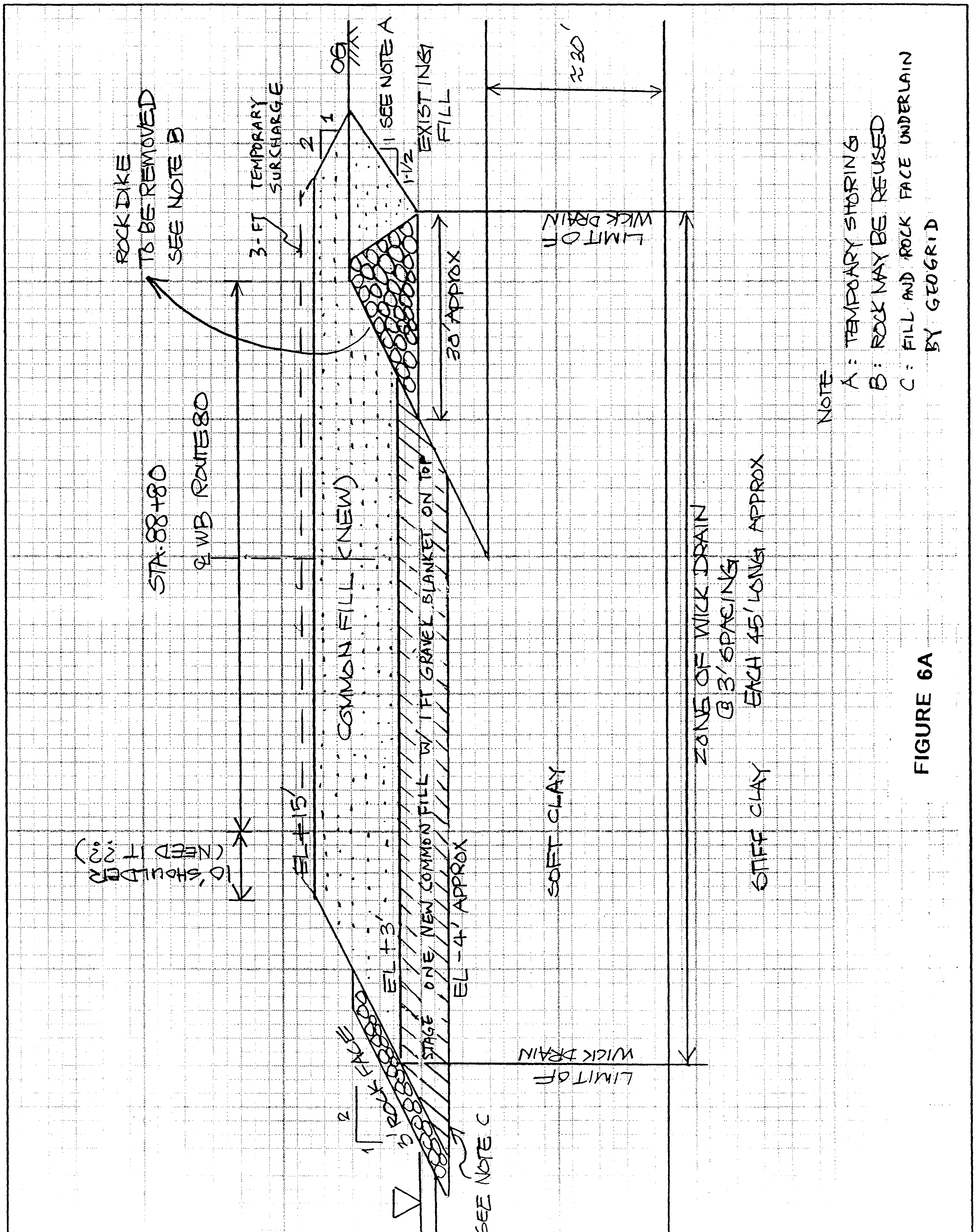


FIGURE 6A

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

17660 Newhope Street, Suite E, Fountain Valley, CA 92708
(714) 751-3826, Fax (714) 751-3928

Project SFOBB OAKLAND MOLE   Project No. ______

By HUBERT   Date 12-2-98   Checked By ______   Date ______   Sheet ____ of ____

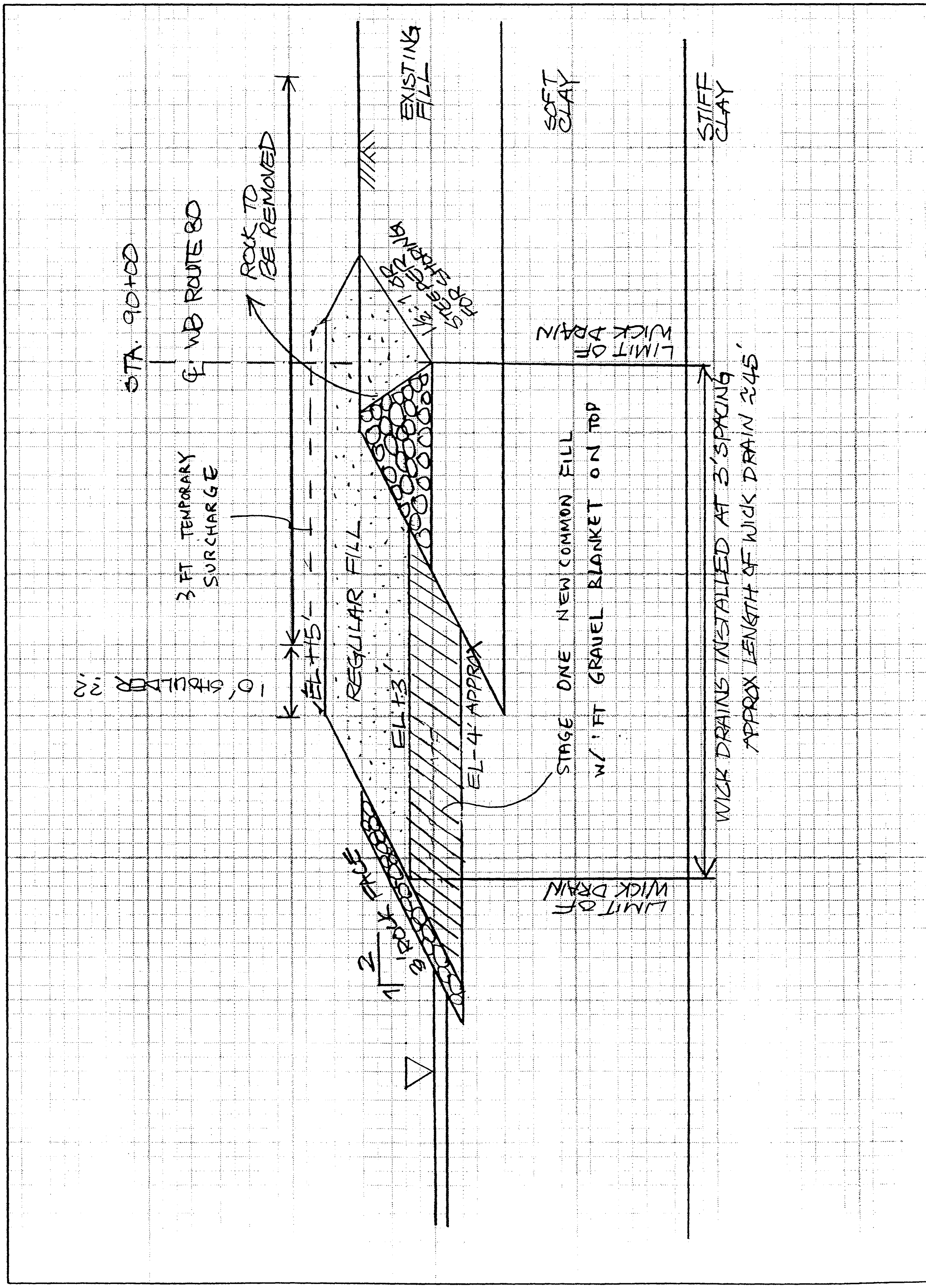
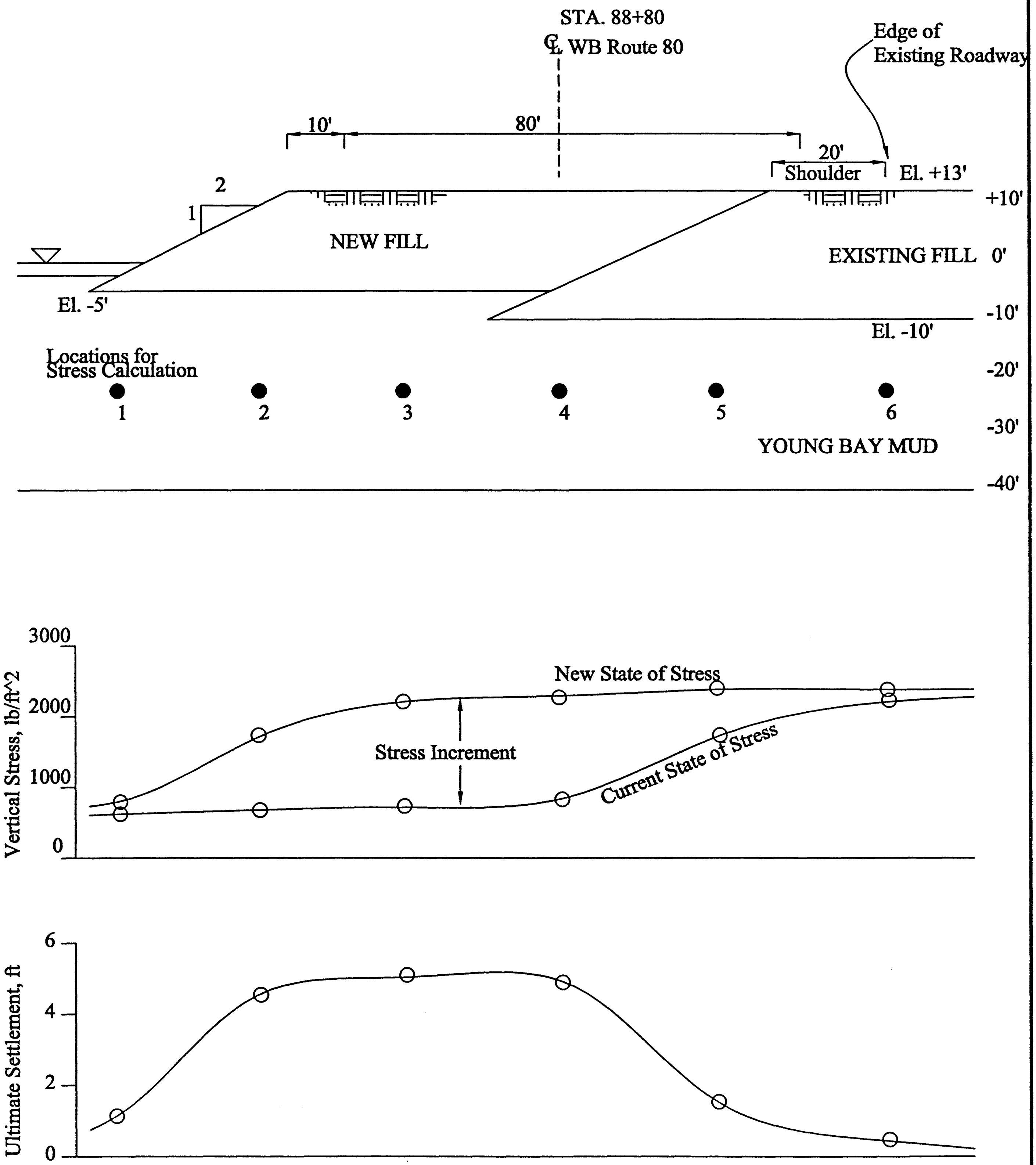
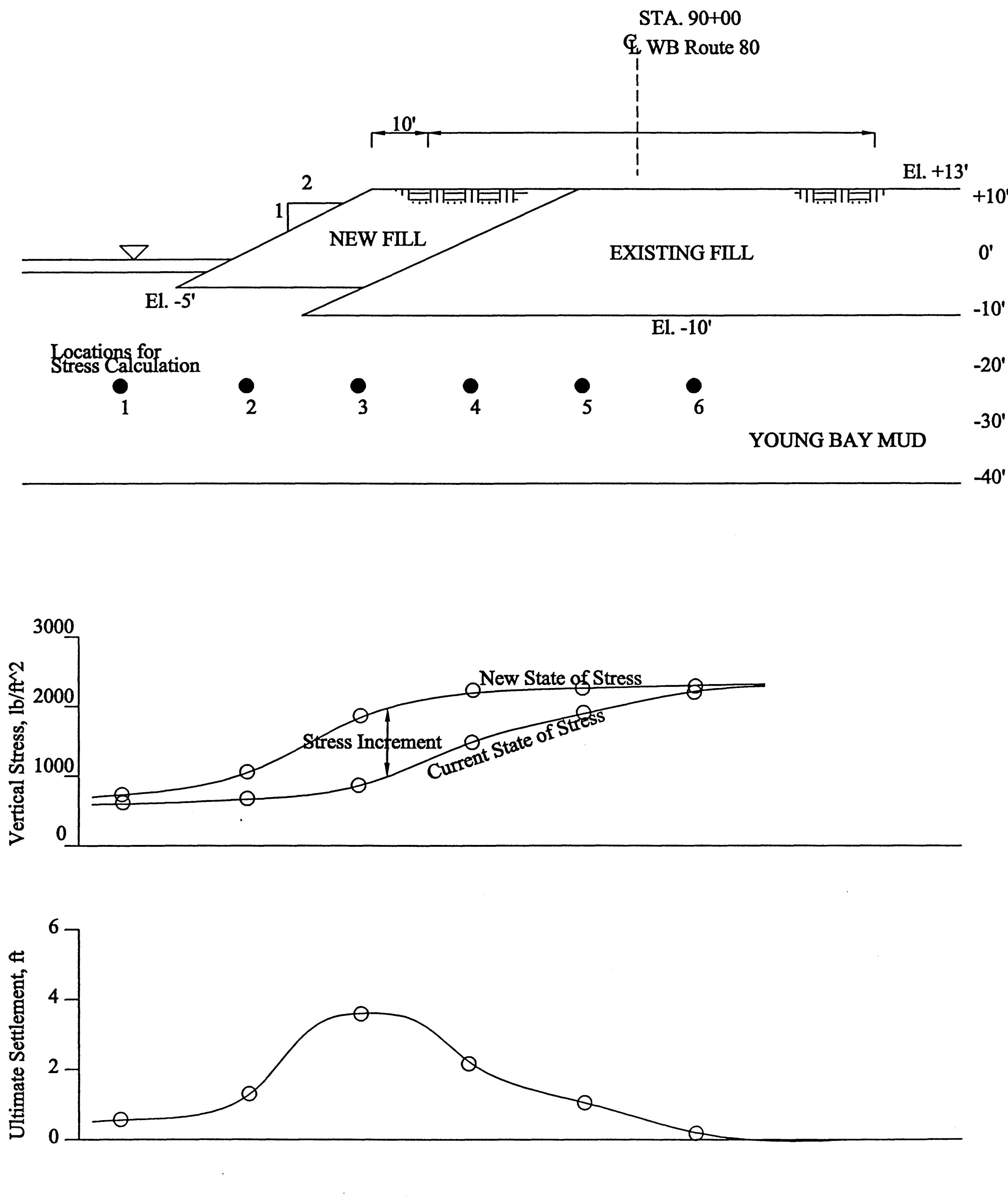


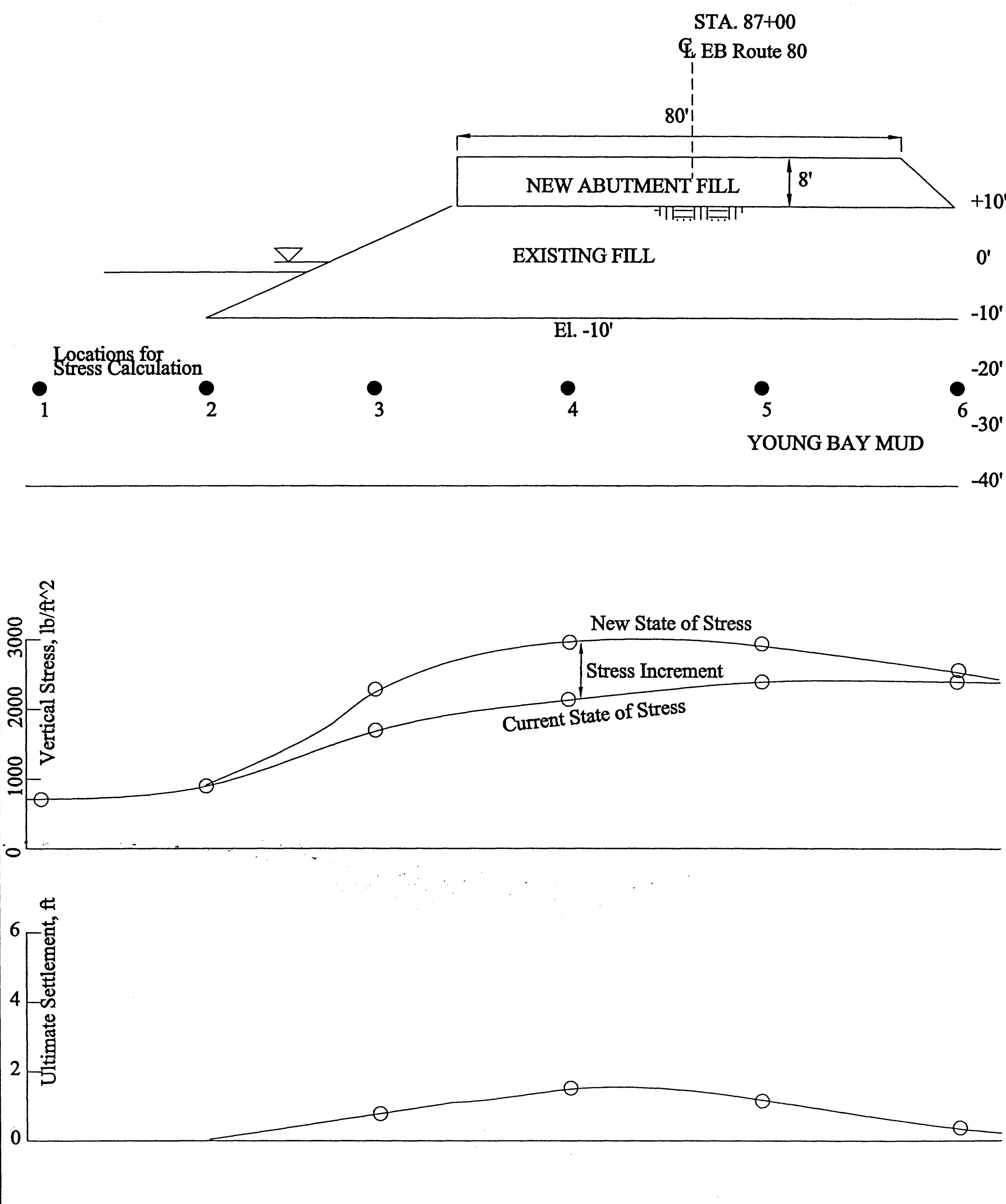
FIGURE 6B



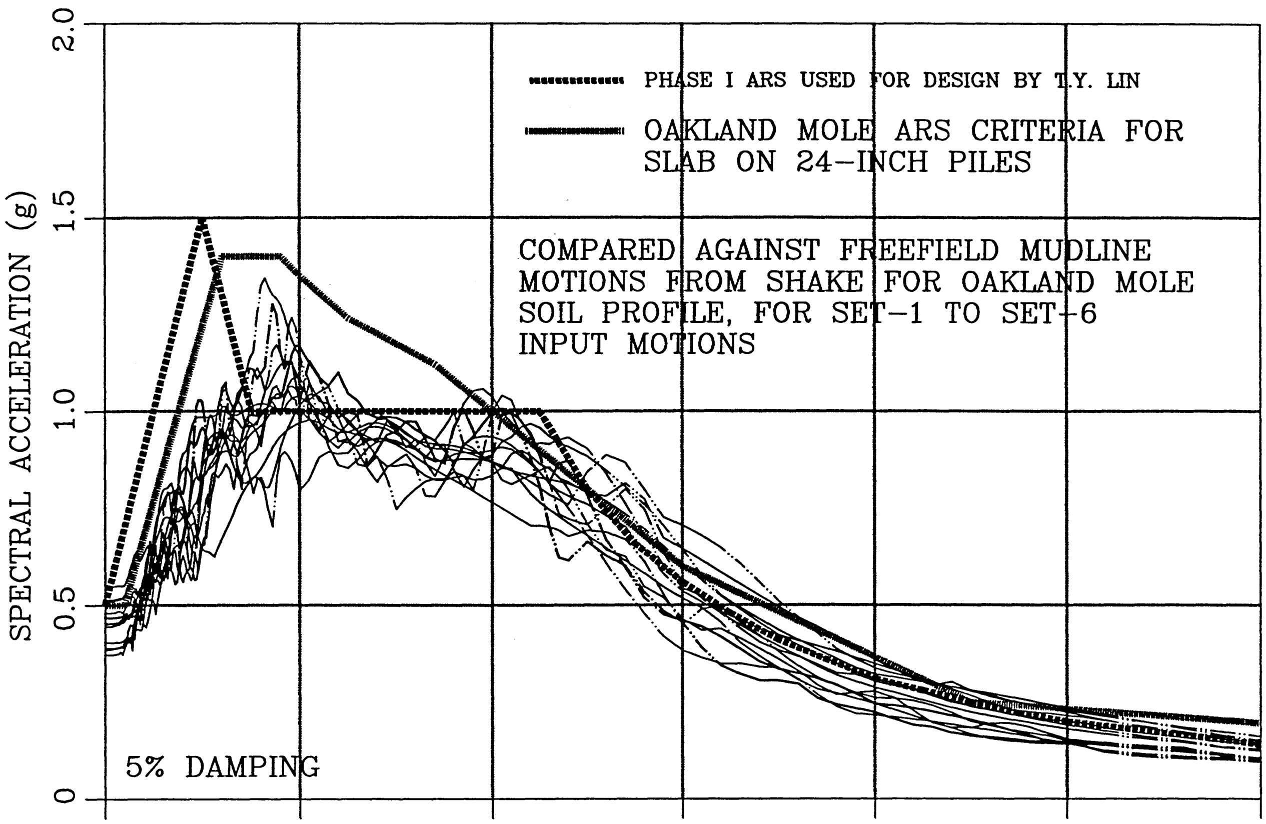
NEW S.F.-OAKLAND BAY BRIDGE	CONSOLIDATION OF WB NEW FILL @ STA 88+80 OAKLAND MOLE FIGURE 7	
Earth Mechanics, Inc. Geotechnical & Earthquake Engineering	Project No. 98-107	Date: 11-10-98

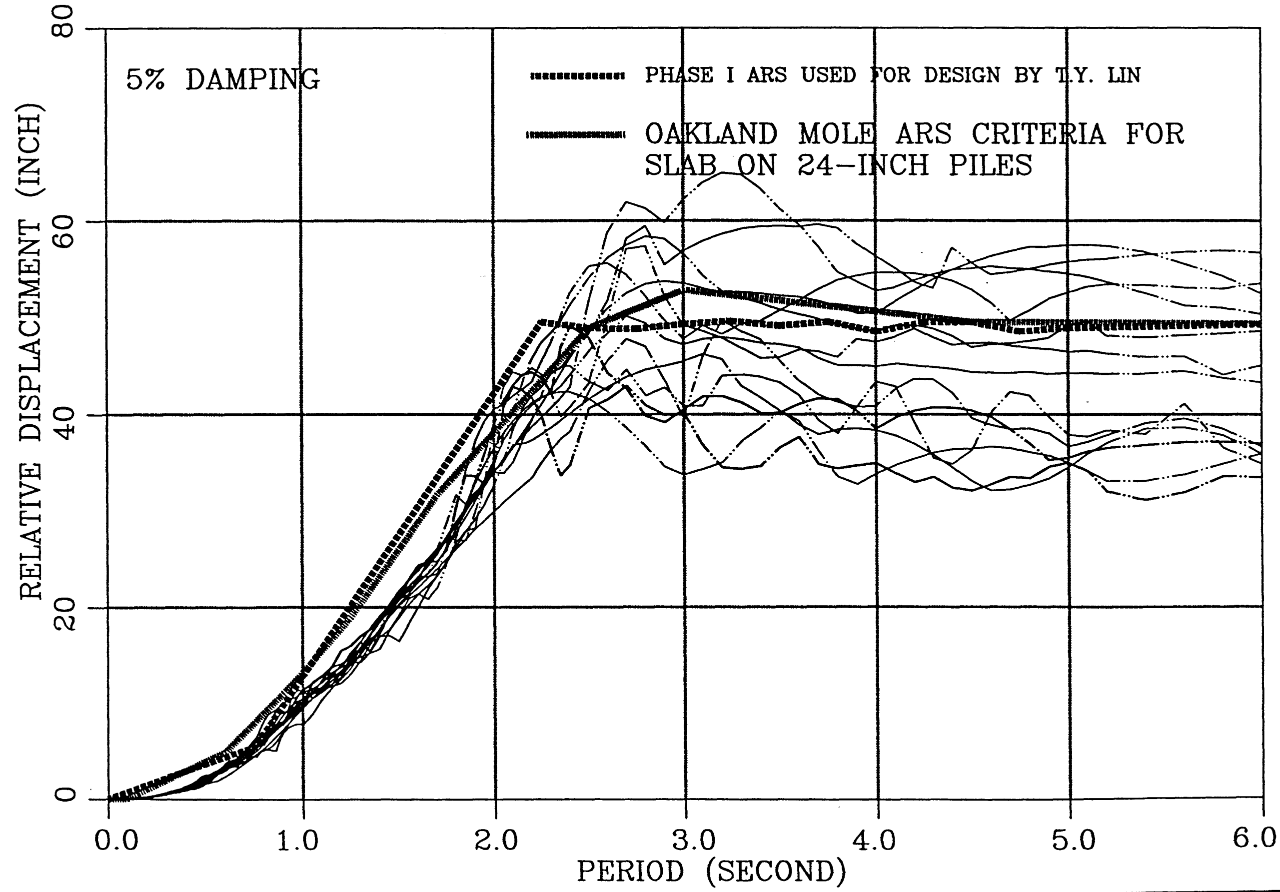


NEW S.F.-OAKLAND BAY BRIDGE	CONSOLIDATION OF WB NEW FILL @ STA 90+00 OAKLAND MOLE
Earth Mechanics, Inc. Geotechnical & Earthquake Engineering	FIGURE 8
Project No. 98-107	Date: 11-10-98



NEW S.F.-OAKLAND BAY BRIDGE	CONSOLIDATION OF EB NEW FILL @ STA 87+00 OAKLAND MOLE
Earth Mechanics, Inc. Geotechnical & Earthquake Engineering	FIGURE 9
	Project No. 98-107 / Date: 11-10-98

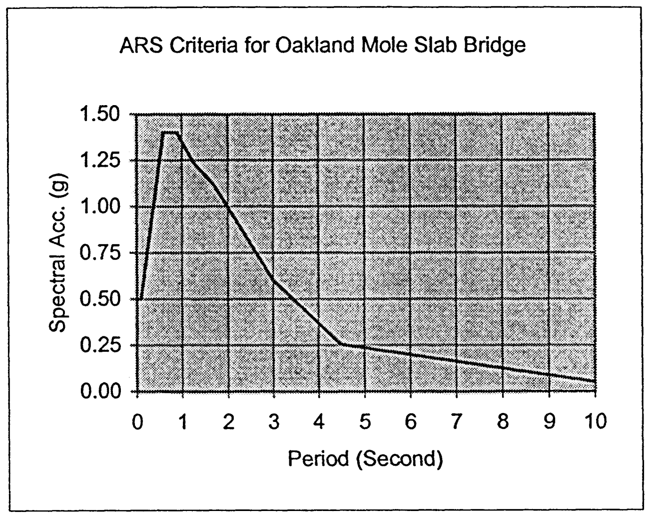




Earth Mechanics, Inc. NEW BAY BRIDGE	ARS CRITERIA FOR OAKLAND MOLE
EMI Project No.: 97-146 / Date: 8/22/98	FIGURE 10

ARS Criteria for Oakland Mole Slab Bridge

N	Period (Second)	PSA (g)	Rd (cm)	Rd (in)
1	0.01	0.50	0.0	0.00
2	0.10	0.50	0.1	0.05
3	0.60	1.40	12.5	4.93
4	0.90	1.40	28.2	11.10
5	1.00	1.35	33.6	13.21
6	1.25	1.24	48.2	18.96
7	1.70	1.12	80.2	31.57
8	2.50	0.80	124.3	48.94
9	3.00	0.60	134.3	52.89
10	4.50	0.25	127.4	50.17
11	10.00	0.05	124.3	48.94



N	Period (Second)	Rd (in)
1	0.01	0.00
2	0.10	0.05
3	0.60	3.43
4	0.90	11.10
5	1.00	13.21
6	1.25	18.96
7	1.70	31.57
8	2.50	48.94
9	3.00	52.89
10	4.50	50.17
11	10.00	48.94

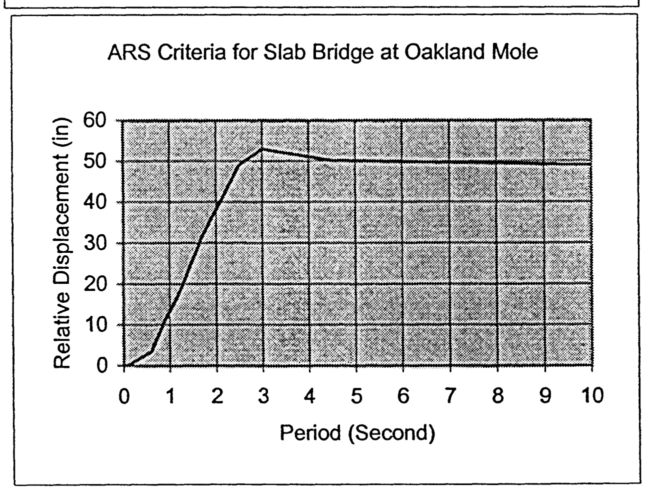


FIGURE 11

Annual Construction Reports
1934, 1935, 1936, 1937

FIRST ANNUAL PROGRESS REPORT SAN FRANCISCO OAKLAND BAY BRIDGE

JULY 1, 1934

Extracted From 1st Annual Report, July 1, 1934

The concrete is placed with electric vibrators, only a minor amount of hand tamping, etc., being employed.

This general setup has been found to work very dependably and satisfactorily on the whole.

PROGRESS AND STATUS

	Start Work	Excavation Complete	Piles Complete	Sealed	July 1, 1934 % Pier Complete
E23	2- 5-34	3-20-34	4-25-34	5- 9-34	80%
E23A	4-13-34	4-25-34	5- 4-34	5-23-34	50%
E24	4- 5-34	5-14-34	5-24-34	6- 5-34	75%
E24A	4-25-34	5-14-34	6-15-34	6-18-34	40%
E25	(Crib for cofferdam partly built—started 6-19-34)				
E25A—No work					

The schedule of the work (revised) calls for the completion of the work in three stages.

	Pier E23	Pier E33	Entire Contract
Scheduled date	June 25, 1934	October 23, 1934	February 21, 1934
Probable date	July 10, 1934	November 10, 1934	December 10, 1934

While the preliminary stages of the contract are somewhat behind schedule, it appears that the contract as a whole will be completed considerably ahead of time.

PRESENT STATUS OF WORK

Pier	Cofferdam	Excavation	Piling	Seal	Elev. Concrete	Per Cent Complete
E23	Complete being pulled	Complete	Complete	Comp.	36′ above sea level	80%
E23A	Complete	Complete	Complete	Comp.	5.2′ below sea level	50%
E24	Complete	Complete	Complete	Comp.	to 10′ above	75%
E24A	Complete	Complete	Complete	Comp.		40%
E25	Guide Frame built. No other piers started.					
Value of contract bid items						$253,665

The value of work done to last monthly estimate, June 20, 1934, was $55,849.27, or 22 per cent of the whole, with 34 per cent of the time elapsed. This lag in time should be rapidly compensated during the future progress of the work, the most difficult stages having been completed.

Contractor is the Clinton Construction Co. of California, the officers of which are:

W. B. Brinker, President.

Albert Huber, Vice President.

Wm. Joyner, Secretary-Treasurer.

Approach Construction

The progress on the approaches to the San Francisco-Oakland Bay Bridge during its first year of construction work was confined to that section which traverses portions of San Francisco Bay along the northerly side of the Key System mole from its westerly end toward the Oakland shore and northerly along this shore to the foot of Folger Avenue, Berkeley. Work was begun on this section of the project first due to the advisability of giving these fills as long a time as possible in which to attain the major portion of the settlement expected.

In order that a more stable fill might be constructed, the Berkeley Water Front Co. was awarded a contract under which they made 16 three-inch borings along the center line of the approach adjacent to the Key Mole. Undisturbed samples of the underlying strata to a depth of from 40 to 90 feet were taken on these borings and tests made upon them by the Sacramento testing laboratory to determine the weight per cubic foot, percentage moisture, percentage consolidation under various loadings, coefficient of friction and mechanical analysis of the materials. From this and earlier data taken from observations, obtained by District IV, Division of Highways on various fills placed for highway routes under its jurisdiction along the shores of San Francisco Bay, it was possible to design a fill which would be more stable and cause less difficulty from settlement than fills made previously.

Borings Test Location of Fill

Rock Wall Protects Sand Fill

On December 14, 1933, the American Dredging Co. was awarded the contract for making the dredger sand fill at a cost of $869,063.32. It was hoped that a contract could be awarded at this same time for the construction of a rock wall protection for the sand fill, but the bid totals were so high that it was deemed inadvisable to accept any bid because of the necessity of keeping the cost of the approaches to the bridge within the $6,600,000 set up for them by the State Legislature, and accepted by the Reconstruction Finance Corporation.

New bids were called on a revised typical section and decreased rock quantities, and the contract was awarded on December 29, 1933, to Fredrickson and Watson Construction Co., Fredrickson Bros. and the Basalt Rock Co. at a cost of $274,687.50. The dates for completion of the above contracts are June 14, 1935, and June 27, 1935, respectively, although present progress indicates actual completion dates of February or March, 1935.

The work being done under the dredger fill and rock wall contracts are inseparable in regard to economy and practicability of construction. Economical placing of rock could not be done from barges nor could it be placed upon a semi-liquid foundation such as the bay bottom found in this close vicinity.

Rock and Sand Fill Jointly

In turn a dredger sand fill could not be placed to an elevation of 13 feet above mean lower low water without a restraining barrier to keep it within bounds, and prevent its being washed out by the tides after placing. For these reasons it was imperative that the contractors so arrange their operations that there would be no conflict or delay in the work. It was so arranged, and to date everything has worked out very smoothly.

3500 hp. Dredger

Actual construction was started on January 8, 1934, when the dredge "Harris" started pumping material for the approach fill. It is a 24-inch suction dredge, one of the most powerful in the United States, and has 750 hp. at the cutter, with an overload capacity of 1600 hp. It develops 3000 hp. at the pump and is capable of 3500 hp. The dredge is a very economical plant, having an all electric "Scherbins Control," which gives wide pumping latitude and can be operated at a pressure up to 155 pounds per square inch. With a discharge velocity of 14 feet per second it pumps an average of 11 per cent solids. On June 19th a record output was obtained of 25,400 cubic yards in 23 hours pumping time, or about 1100 cubic yards per hour. This was accomplished with a 12,000-foot or 2¼-mile length of pipe line without a booster.

The material for the fill is being taken from an area in the Oakland Outer Harbor north of the present channel, and is giving the Port of Oakland an excellent turning basin and channel varying in width from 1200 to 1800 feet. The material taken from this area contains an average of 75 per cent fine sand and makes an ideal fill. The fill is built in three stages or lifts; the bay bottom having been dredged of a portion of the mud under the highway proper before filling is begun. In general, the first operation on the work has been to dredge out about 10 feet of the mud from bay bottom, except at the end of the Key System mole and in places where a good foundation was found on the surface. The best design indicated that it would be desirable to have the dredging carried to a depth greater than 10 feet where old, mud-filled channels crossed the highway but limitation of the amount of money available for use on this work made it impossible to do this.

Sand Fill in Three Lifts

The westerly 2000 feet of the fill was needed, on February 1st, for the use of the bridge contractor building the steel and concrete approach spans to the bridge proper, and so was built up as fast as possible, without danger of slides, and delivered to the above contractor in February, 1934.

Dredging out of the mud and placing it alongside the highway was started at the end of the above section, and has been completed throughout the remainder of the mole and to a point northerly along the bay shore 400 feet north of the Judson Manufacturing Company's property, 846,500 cubic yards of mud or 90 per cent of this contract item, having been dredged to date.

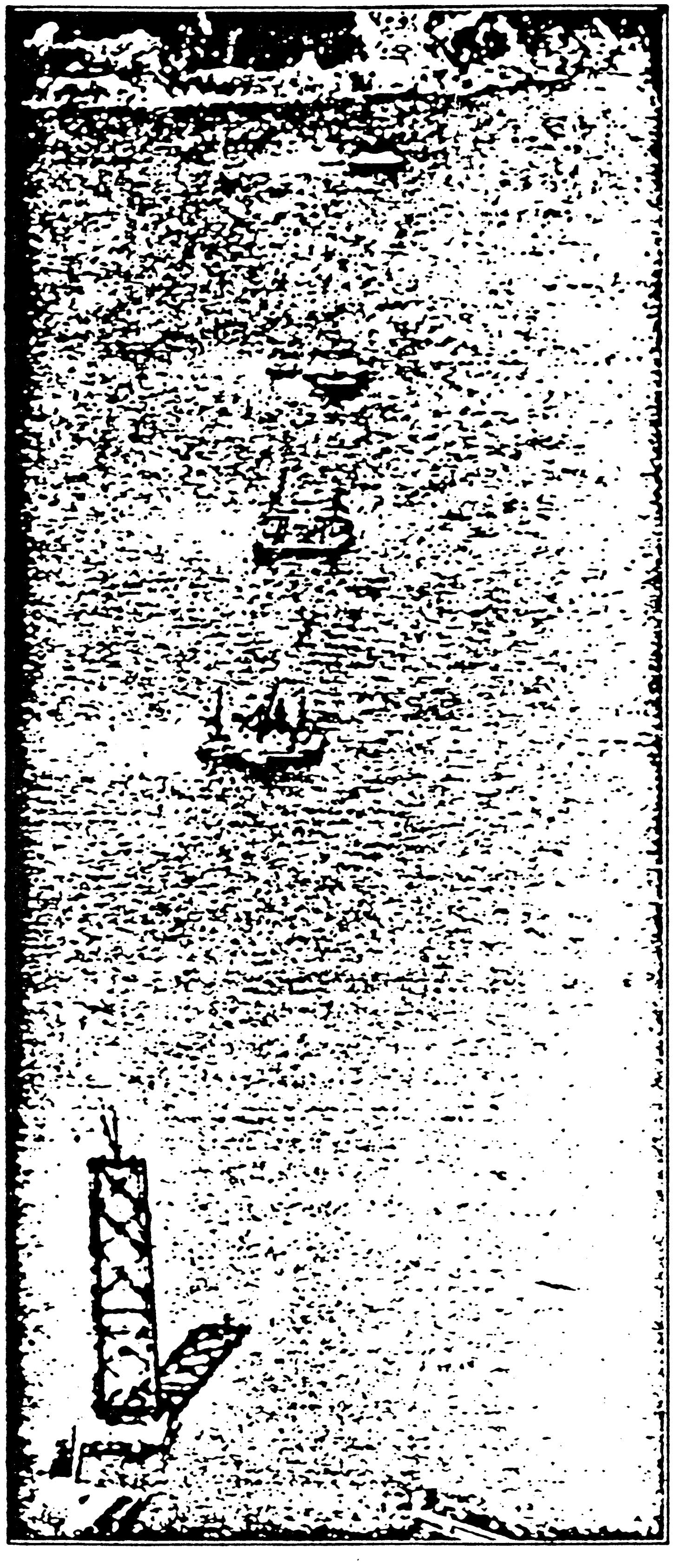
In the process of this work, it was necessary to remove a 38-foot section of the Twenty-second and Thirty-sixth streets lateral sewers, above the flow line, in order to allow passage of the dredge into the portion of the highway between the Oakland outfall sewer and the present mole. These were rebuilt upon completion of the dredging. This work was done by the dredge *Richmond* which is an 18-inch all electric suction dredge.

The first stage of fill was placed to an elevation of +3 mean lower low water. Suspended mud and light clays washed from the sand pumped to the fill and redeposited from the dredging out operation formed mud waves ahead of the toe of the fill, at times, and caused considerable trouble at the outlet to the Oakland outfall sewer, making it necessary to keep the dredge *Richmond* near on standby service to keep the channel open until this section of the fill was complete. Every precaution was also taken to keep the outfall sewer from being moved laterally. This stage of the fill has been placed throughout the mole section to date.

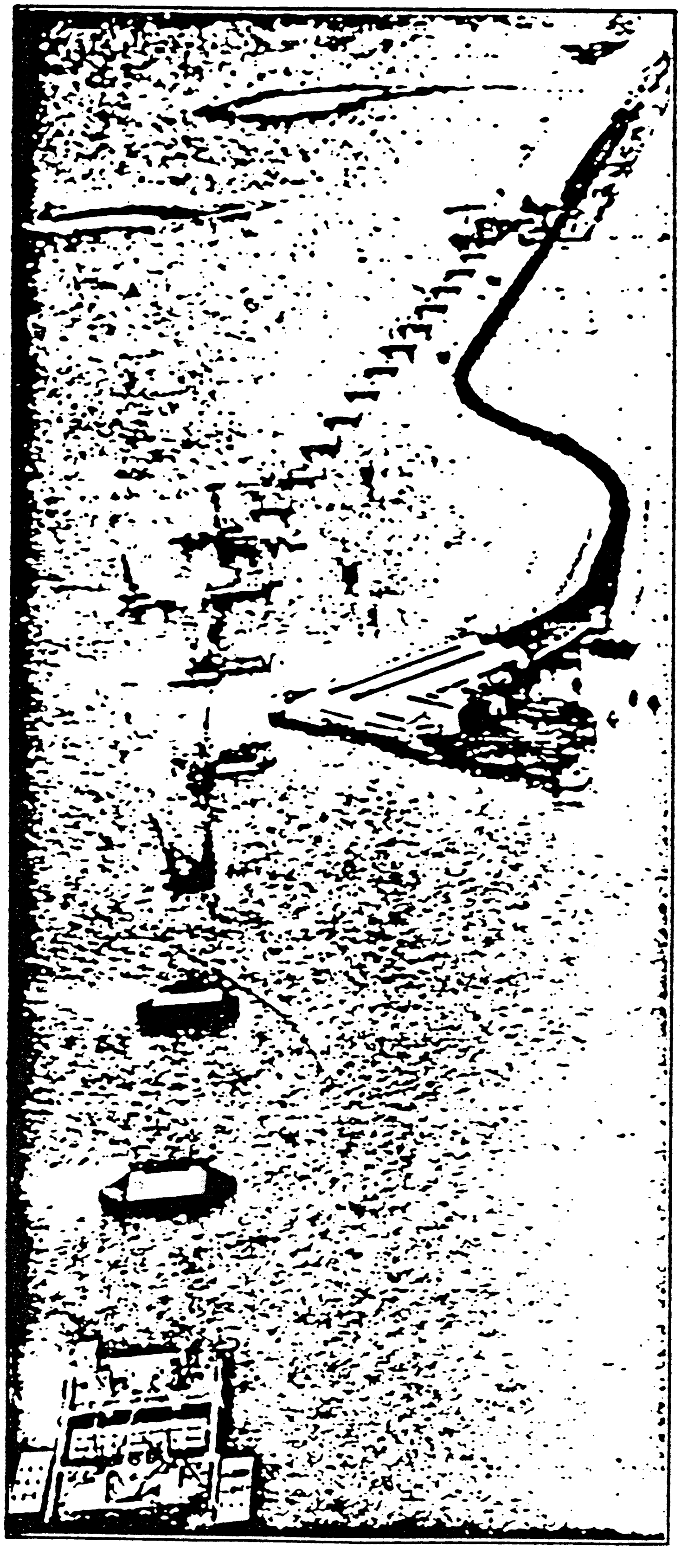
Temporary Bulkhead for Sand

Upon completion of the first stage of the fill a temporary bulkhead, four feet in height, was constructed during the low stages of the tides. This bulkhead served the dual purpose of reducing the amount of core rock necessary in the rock wall facing for the fill and retaining the second stage of the fill which was then placed to an elevation of +7. This second stage in turn served as a roadway for the operation of the rock contractor, and over which 75 hp. caterpillars and 25 to 30-ton capacity Le Tourneau wagons, equipped with eight oversize pneumatic tires, were able to operate a few hours after pumping on this stage had ceased.

Core rock was moved into position beyond the temporary bulkhead to an elevation slightly above +7 by four Le Tourneau wagons, and bulldozed into shape.



Status of West
Bay Crossing
July 1, 1934



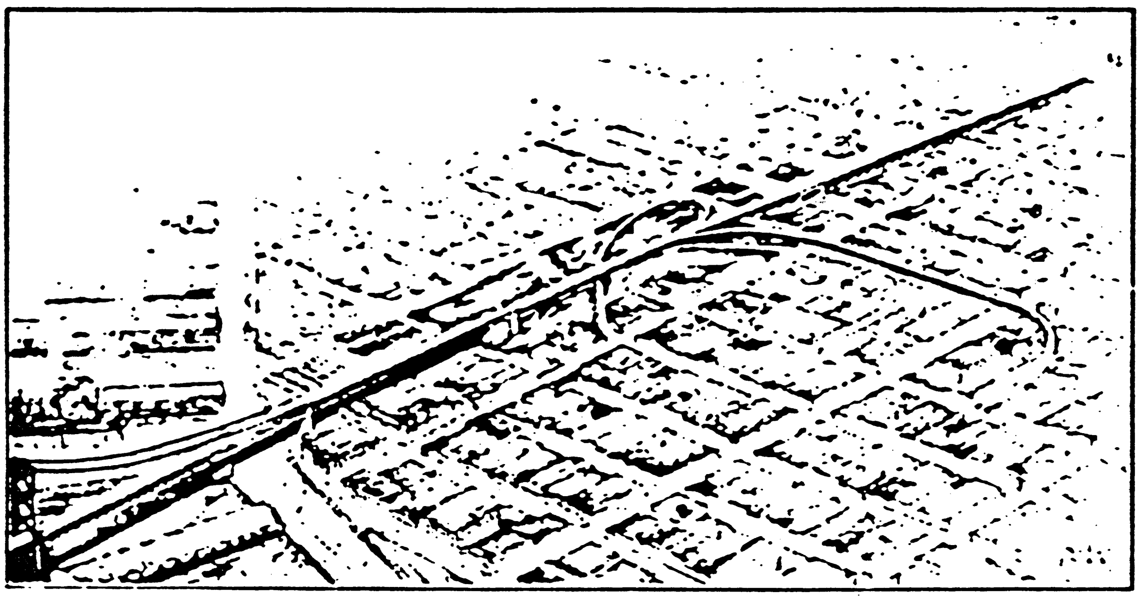
Status of East Bay Crossing July 1, 1934

When this was completed, face rock was placed on the exposed face, and then the remainder of the core rock needed for the wall was placed to the final grade elevation between +13 and +15. This roughly shaped wall was then backed with sand to prevent excess leakage through it during the pumping of the third stage of filling.

The placement of the remainder of the face rock to complete the wall was deferred until the third or final stage of fill had been pumped to grade in order that the rock might be placed by dumping from Le Tourneau wagons rather than by the more expensive method of hand placing by crane or shovel.

Rock on Fill

The rock contractor has furnished rock on a guaranteed maximum specific weight basis of 145 pounds for core rock and 150 pounds for face rock. This rock has been furnished from the Napa quarry of the Basalt Rock Co. and a portion of the core rock from the Richfield Quarry in Richmond. The rock was shipped by barge and unloaded into bunkers on a temporary timber pile dock by means of a 2½-yard floating clamshell derrick.



Drawing of San Francisco automobile approaches

Rock Fill
Complete

The contractor has placed the entire section of wall from the beginning of the fill, at the end of the mole, to the outfall sewer outlet including a short section necessary to maintain a sewage outlet channel. This section contains about 56,700 tons of core rock and 29,200 tons of face rock and represents 40 per cent of the work to be done under this contract. It was completed on May 19, 1934, 26 per cent of the contract time having elapsed. The contractor's equipment has been removed from the job until the dredger fill shall have been completed along the mole section before resuming operations, as no wall will be placed along the outfall sewer.

Placing of the third or final stage of the fill was delayed as long as possible after completion of the second stage; not sufficiently long, however, to conflict with the well-timed and planned schedule of the dredging operations. This stage was placed to an elevation sufficiently above the grade line of +13 feet to allow for the anticipated settlement and consolidation that the fill will take in the years to come.

This portion of the fill was made by placing cross dykes along the fill about 2200 feet apart, which with the dykes along either side of the fill, formed a series of settling basins.

The dredger pipe was set at one cross dyke and a spillway outlet of four 24-inch pipes laid at the next dyke to allow the water from the dredger pipe line to escape after the sand which it carried had been deposited in the basin formed. In several places along the line of the rock wall the concentration of this weight added to the wet sand under it caused the material to slip out and leave a low spot in the fill.

These holes were filled later by bulldozing material into them when it was certain that the sand had stabilized. This final stage has been completed along the entire mole section.

2,408,000 Cubic Yards of Sand Dumped Into Fill

The dredging contractor has pumped 2,408,000 cubic yards of sand from the Oakland Outer Harbor to the approach fill which represents 66 per cent of the contract quantity. The entire contract is 63 per cent complete and only 37 per cent of the contract time elapsed.

As a part of the right of way agreement with the Paraffine Companies in Emeryville, where the highway cuts through their property, it was agreed that a double 9 foot by 9 foot concrete subway would be built to allow passage of their trucks from the plant to the waterfront property and export docks. The contract for this job was let on April 7, 1934, to Healy-Tibbitts Construction Co. for $26,433.50.

To date the cofferdam, excavation, driving of piles and forms for the floor slab have been completed. Pouring of concrete will start within the next week. As regards cost this structure is 38 per cent complete, while 70 per cent of the contract time has elapsed.

In the drafting room plans are now being prepared for the traffic distribution structure in the east bay in the hope that it may be advertised for contract in November, 1934. Plans for the San Francisco approaches are getting well along for advertisement at an early date.

The approaches to the San Francisco-Oakland Bay Bridge are being financed from the State Highway Construction Fund, Primary North, from the appropriation of $6,600,000 made by the State Legislature in 1933 to be expended at the rate of $1,650,000 per year.

SECOND ANNUAL PROGRESS REPORT SAN FRANCISCO OAKLAND BAY BRIDGE

JULY 1, 1935

East Bay Approaches

[Division of Highways Contracts Nos. 64TC16, 84TC1–64TC26 and 64TC29; and Bridge Contract Nos. 20 and 20-A]

Extracted from 2nd Annual Report

The East Bay approaches to the San Francisco-Oakland Bay Bridge involve the following contracts:

64TC6 [No. 20], The American Dredging Company, $869,063.32, for the construction of the dredger fill from the Key Mole east to Emeryville and thence northerly to Berkeley; and

64TC8 [No. 20-A], Fredrickson & Watson Construction Co., and Fredrickson Brothers, $274,687.50, for the rock retaining wall for Contract No. 20; and

64TC16, Healy-Tibbitts Construction Company, $26,433.50, for subway beneath the fill opposite Emeryville.

84TC1–64TC26, Barrett & Hilp, $1,026,780, for the East Bay interlacing distribution viaducts at Emeryville.

64TC29, J. F. Knapp, $117,478, for the construction of a concrete subway under the Southern Pacific tracks at Folger Avenue, Berkeley.

(In the first Annual Report the first three contracts mentioned above were described in detail.)

During the past year, the approach construction work has progressed at a rapid pace, with projects under way on both sides of the Bay.

The dredger fill, which was described at considerable length in the previous report, was completed on December 28, 1934, with a total of 3,823,728 cubic yards having been pumped.

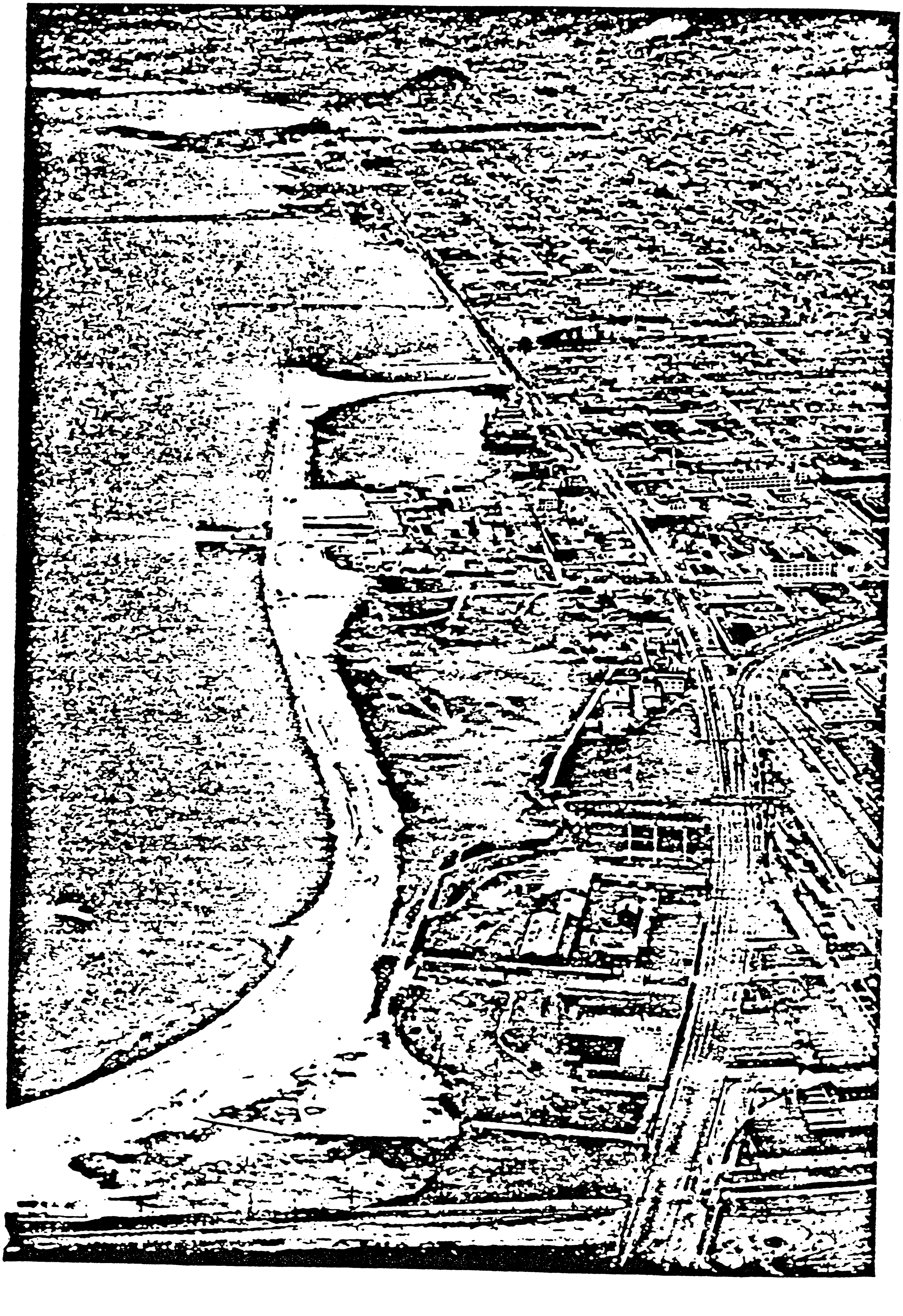
Considerable attention was given to subsidence studies, to determine the rate of settlement, especially along the mole line, where firm material was at varying depths, in some cases being quite deep. The settlement rates were plotted on charts, showing the rates with each stage of fill placed. Each additional stage increased the rate of settlement, due to the added load. when

From these studies it was possible to compute the future rate of settlement, and estimate the additional height necessary to provide sufficient material, so that when the completed project reached a state of equilibrium with the major subsidence complete, the top of the fill would be approximately to the ultimate grade. In some locations, where the underlying soft material was quite deep, the fill was placed as much as eight feet above the ultimate grade. Settlement records taken after completion of the project until the present time, have confirmed the studies made, and the results have checked the computations very closely.

The Rock Wall contract was completed on April 5, 1935, and the final quantity of rock placed amounted to 101,750 tons of core rock, and 95,403 tons of face rock. This work was prosecuted in an orderly fashion, and was completed well ahead of the contract limit.

The access subway for the Paraffine Companies, under contract to Healy-Tibbitts Construction Company, will be finished within the next month, being at the present time about 95 per cent complete.

On May 22, 1935, Barret and Hilp commenced work on the Distribution Structure, which crosses the tracks of the Key System, Southern Pacific, and Santa Fe Railways at the east end of the mole.



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This interlacing structure, which contains 1.6 miles of roadway, also affects sixteen grade separations. All travel on the Distribution Structure will be unidirectional, or one-way traffic, which minimizes the driving hazards.

On June 8, 1935, J. F. Knapp commenced work on the Folger Avenue Subway. This structure crosses under the tracks of the Southern Pacific Company at Folger Avenue in the city of Berkeley, providing a 44-foot width of pavement.

Falsewook is being installed for supporting the railroad tracks during construction, and excavation for the structure will soon commence.

Plans for the central branch of the East Bay Approaches are under way, and the San Pablo Avenue undergrade crossing will soon be advertised for bids. As soon as possible thereafter, the plans for the south branch will be completed and that project advertised for contract.

In addition to the approaches to the bridge, which are financed by the $6,600,000 appropriated for that purpose by the State Legislature, the Division of Highways is building a system of extensions, which includes Harrison, Bryant, Tenth and Potrero Streets in San Francisco, and Moss Avenue and the East Shore Highway in Alameda County. These improvements are financed from regular gas tax funds allocated to the projects.

Bridge Railway

The San Francisco-Oakland Bay Bridge will provide facilities for the transportation of more than 30,000,000 train passengers annually, a large part of whom will be commuters, numbering about 50,000, who live in Alameda County and now go to San Francisco by the combination East Bay electric railways and ferryboats. In the future the transportation will be by the same East Bay electric lines, which, however, are to be extended over the bridge to a terminal in San Francisco. The bridge was designed as much for this great mass of commuting traffic as for the motorist traffic.

The Citizens' Financial Advisory Committee to the California Toll Bridge Authority, a committee of representative citizens of the cities around the Bay region, recommended to the Toll Bridge Authority that the authority provide for an electric railway over the bridge on the same self-liquidating basis upon which the bridge proper is being constructed, and that the California Toll Bridge Authority should acquire statutory power to own, operate and lease cars, trains and other equipment over and in connection with the bridge; and, further, that the existing East Bay lines be given leases on the Authority's railway over the bridge so that the present East Bay cars might, instead of terminating at the ferry slips on the east shore, cross the bridge to a terminal in San Francisco and return.

It was recommended that contracts be drawn between the California Toll Bridge Authority and the Key System and Southern Pacific Companies, owners of the East Bay lines, at a toll sufficient to pay interest on and progressively amortize the capital investment.

Several sites for the terminal in San Francisco were proposed and were given thorough engineering analyses. Considerations in the location of the terminal were: passenger comfort and convenience, capital investment, proximity to the destinations of the commuters, traffic congestion, damage to adjoining property,

THIRD ANNUAL PROGRESS REPORT SAN FRANCISCO OAKLAND BAY BRIDGE

. . JULY 1, 1936 . .

side roads were 77 per cent completed. Selected base material was placed on part of the side roads, with paving scheduled to start early in July. The contract is scheduled for completion by September 1, 1936.

Extracted from 3rd Annual Report July 1, 1936

PERSONNEL

The contract for this work is held by J. F. Knapp of Oakland, California, of which Otto Parlier is Associate and Superintendent. V. A. Endersby is Resident Engineer.

Mole Paving (Contract No. 84TC4)

This contract covers the paving of the mole fill, extending from the east end of the bridge to the distribution structure and thence extending north to the west end of the Folger Avenue underpass. East of the toll plaza the cross-section has a 60-foot pavement to accommodate traffic from the upper deck of the bridge, and two 22-foot pavements, one on either side of the roadway, separated from the main roadway by curbed parkways five and one-half feet in width. East of the toll plaza the present contract includes two 32-foot asphalt concrete pavements, separated by a five and one-half-foot curbed parkway strip in the center. Space is provided for future widening of these pavements when necessary. Between the distribution structure and the Folger Avenue underpass to the north, there are two 21-foot asphalt concrete, one-way pavements, separated by a center parkway six feet in width.

Sixty-foot Pavement

Ten-foot wide rock shoulders, oil treated, are to be provided adjacent to all outside lanes of all the roadways involved.

Work started on this contract in November of 1935, and was 55 per cent completed on June 30, 1936.

PERSONNEL

The contract for this work is held by the Hanrahan-Wilcox Corporation. E. G. Poss is District Construction Engineer with L. G. Marshall as Resident Engineer.

Ashby Avenue Connection (Contract No. 84TC8)

This project involves the construction of a concrete pavement 47 feet wide and 56 feet wide between concrete curbs with eight-foot sidewalk spaces on either side, extending from the east end of the Folger Avenue underpass to a connection with Ashby Avenue at Ninth Street in Berkeley—a total length of 0.3 mile.

Work started on this contract in May of 1936, and was approximately 35 per cent completed on June 30, 1936.

The contract is held by L. C. Seidel. E. G. Poss is District Construction Engineer with L. G. Marshall as Resident Engineer.

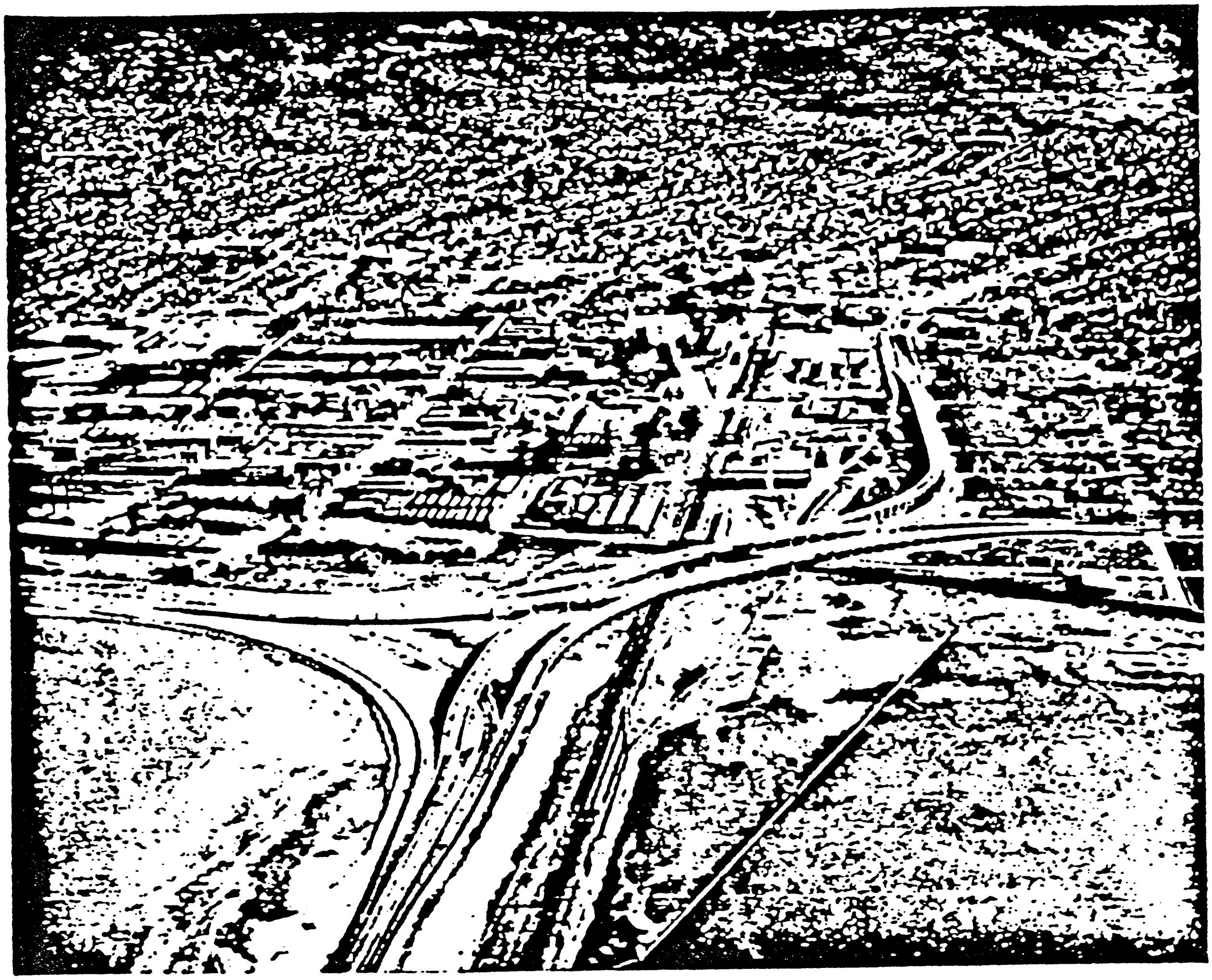
Cypress Street Improvement (Contract 84TC10)

This project covered paving from the southerly approach to the distribution structure at Thirty-fourth Street to the intersection of Seventh and Cypress Streets in Oakland, a distance of 1.4 miles. Beween Thirty-fourth and Eighteenth Streets the paving involves an asphalt concrete surface 40 feet in width, two eight-foot oiled and screened rock borders. Between Eighteenth and Seventh Streets the asphalt concrete pavement is 60 feet in width, with eight-foot Portland cement concrete gutters and curbs on either side, providing a completed 76-foot roadway between curbs.

FOURTH ANNUAL PROGRESS REPORT SAN FRANCISCO OAKLAND BAY BRIDGE

.. JULY 1, 1937 ..

Extracted From 4th Annual Report July 1, 1937



Aerial View of
I bution Structure,
December 1, 1936

San Pablo Underpass (Contract 84TC3)

At the beginning of the fiscal year, with this project 84 per cent complete, the only remaining work consisted of the paving of the side roads adjacent to the Underpass. This was completed on August 4, 1936.

Mole Paving (Contract 84TC4)

By June 30, 1936, this project was approximately 55 per cent complete. Rapid progress was continued throughout the early month of the fiscal year, with the major portion of the work completed in time for the opening of the bridge. The final cleanup was deferred until after the opening, so that all efforts could be directed toward consummation of the paving work. The project was recommended for acceptance on November 25, 1936, two weeks after the official bridge opening.

Ashby Avenue Connection (Contract 84TC8)

This project was 35 per cent complete at the end of the fiscal year. The balanc of the work was rapidly prosecuted and completed on August 27, 1936.

Cypress Street Improvement (Contract 84TC10)

This project was briefly described in Volume III. At the beginning of the fisca year actual construction work had not been started. Bids were taken on June 24 1936, and the contract was awarded to the Hanrahan Company.

Original Contract Document for Dredger Fill for the Existing Fill
1933

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

SPECIAL PROVISIONS PROPOSAL AND CONTRACT

FOR CONSTRUCTING A

DREDGER FILL

IN

ALAMEDA COUNTY BETWEEN THE WESTERLY END OF THE KEY MOLE FILL AND THE FOOT OF FOLGER AVE., BERKELEY; DISTRICT IV, EAST BAY APPROACH, SAN FRANCISCO-OAKLAND BAY BRIDGE.

For Use in Connection With Standard Specifications Dated January, 1930.

CONTRACT No.______

1933

CALIFORNIA STATE PRINTING OFFICE
HARRY HAMMOND, STATE PRINTER
SACRAMENTO, 1933

7817

Alameda County
East Bay Bridge Approach

STATE OF CALIFORNIA

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

NOTICE TO CONTRACTORS

Sealed proposals will be received at the office of the State Highway Engineer, Public Works Building, Sacramento, California, until 2 o'clock p.m. on November 15, 1933, at which time they will be publicly opened and read, for construction in accordance with the specifications therefor, to which special reference is made, of portions of State Highway as follows:

Alameda County, between the westerly end of the Key Mole fill, and the foot of Folger Ave. in Berkeley, East Bay Approach, San Francisco-Oakland Bay Bridge, about four and one-tenth (4.1) miles in length, to be graded by dredging and placing selected dredger material fill.

STATE HIGHWAY ENGINEER'S ESTIMATE.

Item 1.	100	lineal feet removing and reconstructing existing 6′ 0″ by 6′ 2″ sewer.
Item 2.	50	lineal feet removing and reconstructing existing 3′ 6″ by 3′ 6″ sewer.
Item 3.	946,000	cubic yards dredging.
Item 4.	3,625,000	cubic yards dredger sand fill.
Item 5.	7,990	cubic yards structure excavation.
Item 6.	498	cubic yards Class "A" Portland cement concrete (structures).
Item 7.	47,600	pounds bar reinforcing steel (structures).
Item 8.	2,560	lineal feet furnishing untreated Douglas fir piles.
Item 9.	84	each, driving piles.
Item 10.	430	lineal feet 8″ vitrified sewer pipe.
Item 11.	976	lineal feet 12″ vitrified sewer pipe.
Item 12.	140	lineal feet 24″ vitrified sewer pipe.
Item 13.	520	lineal feet 30″ vitrified sewer pipe.
Item 14.	356	lineal feet 18″ corrugated metal pipe.
Item 15.	366	lineal feet 24″ corrugated metal pipe.

Item 16. 190 lineal feet 36″ corrugated metal pipe.

Item 17. 4 each, cutting openings in existing concrete sewer.

Item 18. 215 stations finishing roadway.

SUPPLEMENTAL ITEM.

(Note: This item will not be included when comparing bids.)

Item 19. 140,000 cubic yards dredger sand fill (private fill on property of Paraffine Companies, Inc.).

The State will furnish corrugated metal pipe and tidegates as more explicitly set forth in the special provisions.

In accordance with the provisions of Chapter 397, Statutes of 1931, the Department of Public Works has ascertained the general prevailing rate of wages applicable to the work to be done to be as follows:

Classification	Rate per hour
Laborer	$0.60
Tractor driver (30 h.p.)	0.96
Tractor driver (60 h.p. to 80 h.p.)	1.10
Carpenter	1.00
Carpenter's helper	0.80
Truck driver (15,500 or less)	0.82
Truck driver (over 15,500)	0.96
Power shovel or crane operator	1.25
Oiler	0.82
Grader operator	0.82
Roller operator	1.25
Driller	0.82
Blacksmith	1.00
Blacksmith's helper	0.80
Powderman	0.82
Reinforcing steel worker	1.125
Mechanic (trouble shooter)	0.88
Handyman	0.60
Leverman	1.10
Machinist	1.10
Machinist's helper	0.80
Rigger	1.10
Cranesman	1.10
Assistant engineer	0.95
Assistant electrician	0.85
Fireman	0.80
Levee foreman	0.85
Levee man	0.70
Deck hand	0.80
Mate	0.80
Towboat operator	0.80
Launchman	0.80
Confidential clerk	0.75
Any classification omitted herein not less than	0.60
Overtime	one and one-half (1½) times the above rates
Sundays and holidays (except watchmen, guards and flagmen)	double the above rates

The Director of the Department of Industrial Relations has determined that the classifications of leverman, mate, towboat operator, confidential clerk, levee foreman, assistant engineer and assistant electrician enumerated above shall be exempt from the application of the provisions of an act establishing a thirty hour week on public works as more specifically set forth in Section 4, article (c), "Laws to be Observed," of the special provisions.

The foregoing quantities are approximate only, being given as a basis for the comparison of bids, and the Department of Public Works does not, expressly or by implication, agree that the actual amount of work will correspond therewith, but reserves the right to increase or decrease the amount of any class or portion of the work, as may be deemed necessary or expedient by the said Department of Public Works.

All bids are to be compared on the basis of the State Highway Engineer's estimate of the quantities of work to be done.

Proposal forms will be issued only to those Contractors who have furnished a verified statement of experience and financial condition in accordance with the provisions of Chapter 788, Statutes of 1933, and whose statements so furnished are satisfactory to the Department of Public Works.

Bids will not be accepted from a Contractor who has not been licensed in accordance with the provisions of Chapter 791, Statutes of 1929, as amended, or to whom a proposal form has not been issued by the Department of Public Works.

The attention of bidders is directed to the special provisions covering subletting and assigning the contract and to the use of domestic materials.

Plans may be seen, and forms of proposal, bonds, contract and specifications may be obtained at the said office, and they may be seen at the offices of the District Engineers at Los Angeles and San Francisco, at the office of the San Francisco-Oakland Bay Bridge, and at the office of the Associated General Contractors in San Francisco.

Contractors are urged to investigate the location, character and quantity of work to be done.

Detailed information concerning the proposed work may be obtained from the San Francisco-Oakland Bay Bridge office.

No bid will be received unless it is made on a blank form furnished by the State Highway Engineer. The special attention of prospective bidders is called to the "Proposal Requirements and Conditions" annexed to the blank form of proposal, for full directions as to bidding, etc.

The Department of Public Works reserves the right to reject any or all bids.

DEPARTMENT OF PUBLIC WORKS,
DIVISION OF HIGHWAYS,

C. H. PURCELL,
State Highway Engineer.

Dated November 3, 1933.

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

PROPOSAL REQUIREMENTS AND CONDITIONS.

(a) Contents of Proposal Forms.—Prospective bidders will be furnished with proposal forms which will state the location and description of the contemplated construction and will show the approximate estimate of the various quantities and kinds of work to be performed or materials to be furnished, with a schedule of items for which unit bid prices are asked. All special provisions will be grouped together and attached to the proposal form.

(b) Approximate Estimate.—The quantities given in the notice to Contractors, proposal and contract forms are approximate only, being given as a basis for the comparison of bids, and the Department of Public Works does not, expressly or by implication, agree that the actual amount of work will correspond therewith, but reserves the right to increase or decrease the amount of any class or portion of the work, or to omit portions of the work as may be deemed necessary or expedient by the Engineer.

(c) Examination of Plans, Specifications, Special Provisions and Site of Work.—The bidder is required to examine carefully the site of, and the proposal, plans, specifications and contract forms for the work contemplated, and it will be assumed that the bidder has investigated and is satisfied as to the conditions to be encountered, as to the character, quality, and quantities of work to be performed and materials to be furnished, and as to the requirements of the Standard Specifications, the special provisions and the contract. It is mutually agreed that submission of a proposal shall be considered prima facie evidence that the bidder has made such examination.

(d) Proposal Forms.—All proposals must be made upon blank forms to be obtained from the State Highway Engineer at his office, Public Works Building, Sacramento. Proposal forms are numbered serially and are not transferable. The bidder must submit his proposal on the form furnished to him. Proposals submitted on forms other than the one issued to the bidder will be disregarded. All proposals must give the prices proposed, both in writing and figures, and must be signed by the bidder, with his address. If the proposal is made by an individual, his name and post-office address must be shown. If made by a firm or partnership, the name and post-office address of each member of the firm or partnership must be shown. If made by a corporation, the proposal must show the name of the State under the laws of which the corporation was chartered and the names, titles, and business addresses of the president, secretary, and treasurer.

(e) Rejection of Proposals Containing Alterations, Erasures or Irregularities.—Proposals may be rejected if they show any alterations of form, additions not called for, conditional or alternate bids, incomplete bids, erasures, or irregularities of any kind.

(f) Bidder's Guaranty.—All bids shall be presented under sealed cover and shall be accompanied by cash, cashier's check or a certified check made payable to the Director of Public Works, for an amount equal to at least ten per cent (10%) of the amount of said bid and no bid shall be considered unless such cash, cashier's check or certified check is inclosed therewith.

(g) **Time of Opening.**—Sealed proposals, addressed to the State Highway Engineer, and endorsed:

"Proposal for Constructing a Portion of the East Bay Approach to the San Francisco-Oakland Bay Bridge in Alameda County—Dredging"

will be received by the State Highway Engineer and will be publicly opened and read at the time and place stated in the notice to Contractors.

(h) **Withdrawal of Proposals.**—Any bid may be withdrawn at any time prior to the hour fixed in the notice to Contractors for the opening of bids, provided that a request in writing executed by the bidder or his duly authorized representative for the withdrawal of such bid is filed with the Director of Public Works or the State Highway Engineer. The withdrawal of a bid shall not prejudice the right of a bidder to file a new bid.

(i) **Public Opening of Proposals.**—Proposals will be opened and read publicly at the time and place indicated in the notice to Contractors. Bidders or their authorized agents are invited to be present.

(j) **Disqualification of Bidders.**—More than one proposal from an individual, a firm or partnership, a corporation or an association under the same or different names, will not be considered. Reasonable ground for believing that any bidder is interested in more than one proposal for the work contemplated will cause the rejection of all proposals in which such bidder is interested. If there is reason for believing that collusion exists among the bidders, none of the participants in such collusion will be considered in future proposals. Proposals in which the prices obviously are unbalanced may be rejected. Contracts will be awarded only to responsible bidders capable of performing the class of work contemplated.

(k) **Competency of Bidders.**—Before receiving proposal forms, prospective bidders must file with the Department of Public Works a statement of their financial condition and previous construction experience on a standard form which can be secured from the State Highway Engineer at his office, Public Works Building, Sacramento, in accordance with the provisions of Chapter 788, Statutes of 1933. Proposal forms will not be furnished to prospective bidders who have not qualified by submitting the above statement of their financial condition and construction experience.

Proposal forms will not be furnished to any prospective bidder who has not been prequalified as above provided at least five (5) days prior to the date fixed for opening bids for the work on which said bidder intends to submit a bid.

Bidders will not be required to submit the plan and equipment statement specified in Section 2, article (k), of the Standard Specifications with their proposals for the work herein contemplated.

(l) **Material Guaranty.**—Before any contract is awarded, the bidder may be required to furnish a complete statement of the origin, composition, and manufacture of any or all materials to be used in the construction of the work, together with samples, which samples may be subjected to the tests provided for in the Standard Specifications or in the special provisions to determine their quality and fitness for the work.

AWARD AND EXECUTION OF CONTRACT.

(a) **Award of Contract.**—The right is reserved to reject any and all proposals. The award of the contract, if it be awarded, will be to the lowest responsible bidder whose proposal complies with all the requirements prescribed. The award, if made, will be made within thirty (30) days after the opening of the proposals.

(b) **Return of Bidders' Guaranties.**—Within ten (10) days after the award of the contract, the Department of Public Works will return the proposal guaranties accompanying such of the proposals which are not to be considered in making the award. All other proposal guaranties will be held until the contract has been finally executed, after which they will be returned to the respective bidders whose proposals they accompany.

(c) **Contract Bonds.**—The Contractor shall furnish two good and sufficient bonds executed as surety by a corporation or corporations authorized to issue surety bonds in the State of California, as provided by Chapter 788, Statutes of 1933. Each of the said bonds shall be executed in a sum equal to at least one-half of the contract price. One of the said bonds shall guarantee the faithful performance of the said contract by the Contractor; and the other of the said bonds shall be furnished as required by the terms of an act entitled

> "An act to secure the payment of the claims of persons employed by contractors upon public works, and the claims of persons who furnish materials, supplies, teams, implements or machinery used or consumed by such contractors in the performance of such works, and prescribing the duties of certain public officers with respect thereto," approved May 10, 1919, as amended.

Whenever any surety or sureties on any such bonds, or on any bonds required by law for the protection of the claims of laborers and material men, becomes insufficient, or the Director of Public Works has cause to believe that such surety or sureties has become insufficient, he may demand in writing of the Contractor such further bond or bonds or additional surety, not exceeding that originally required, as in his opinion is necessary, considering the extent of the work remaining to be done. Thereafter no payment shall be made upon such contract to the Contractor or any assignee of the Contractor until such further bond or bonds or additional surety has been furnished.

(d) **Execution of Contract.**—The contract shall be signed by the successful bidder and returned, together with the contract bonds, within eight (8) days, not including Sundays, after the bidder has received notice that the contract has been awarded. No proposal shall be considered binding upon the State until the execution of the contract.

(e) **Failure to Execute Contract.**—Failure to execute a contract and file acceptable bonds as provided herein within eight (8) days, not including Sundays, after the bidder has received notice that the contract has been awarded shall be just cause for the annulment of the award and the forfeiture of the proposal guaranty. If the successful bidder refuses or fails to execute the contract, the Director of Public Works may award the contract to the second lowest responsible bidder. If the second lowest responsible bidder refuses or fails to execute the contract, the Director of Public Works may award the contract to the third lowest responsible bidder. On the failure or refusal of the second or third lowest responsible bidder, to whom any such contract is so awarded, to execute the same such bidders' guaranties shall be likewise forfeited to the State. The work may then be readvertised or may be constructed by day labor as the Director may decide.

STATE OF CALIFORNIA

DEPARTMENT OF PUBLIC WORKS

DIVISION OF HIGHWAYS

SPECIAL PROVISIONS FOR CONSTRUCTING A PORTION OF A STATE HIGHWAY IN THE COUNTY OF ALAMEDA, BETWEEN THE WESTERLY END OF THE KEY MOLE FILL AND THE FOOT OF FOLGER AVE., BERKELEY; DISTRICT IV, EAST BAY APPROACH, SAN FRANCISCO-OAKLAND BAY BRIDGE.

(Annexed to Contract No.______________)

Section 1. Plans and Specifications.—The work embraced herein shall be done in accordance with the Standard Specifications of the Department of Public Works, Division of Highways, dated January, 1930, in so far as the same may apply and in accordance with the following special provisions.

In case of conflict between the Standard Specifications and these special provisions, the special provisions shall take precedence over and be used in lieu of such conflicting portions.

Identical plans will cover the work of this contract and a coincident contract for constructing rock wall and riprap. All work shown upon these plans, excepting the rock wall and riprap, shall apply to and come under the provisions of this contract, subject to the modifications of these special provisions.

Section 2. Progress of the Work and Time for Completion.—The Contractor shall begin work within fifteen (15) days after receiving notice that the contract has been approved by the Attorney General or the attorney appointed and authorized to represent the Division of Highways, and shall diligently prosecute the same to completion before the expiration of

four hundred fifty (450)

consecutive days (Sundays and legal holidays excluded) from the date of said approval.

SECTION 3. GENERAL.

(a) Subletting and Assignment.—The Contractor shall give his personal attention to the fulfillment of the contract and shall keep the work under his control. The Contractor shall perform with his own organization and with the assistance of workmen under his immediate superintendence, work of a value not less than eighty per cent (80%) of the value of all work embraced in the contract. The value of the work sublet or subcontracted shall be determined by multiplying the number of units sublet of any contract item by the unit price as set forth in the contract. If any subdivision of a contract unit is sublet or subcontracted, the entire unit shall be considered as sublet or subcontracted.

Before any work is started on a subcontract, the Contractor shall file with the State Highway Engineer at his office, Public Works Building, Sacramento, a written statement showing the work to be sublet or subcontracted, giving the names of the subcontractors and the description of each portion of the work to be so sublet or subcontracted.

The above provisions shall supplement the provisions of Section 8, article (a), "Subletting and Assignment," of the Standard Specifications.

(b) Repair of Equipment.—The work of installing, assembling, repairing or reconditioning, or other work of any nature on machinery, equipment or tools used in or upon the work to be performed under this contract shall be considered a part of the work to be performed hereunder and any laborers, workmen or mechanics working on such machinery, equipment or tools, unless employed by bona fide commercial repair shops, garages, blacksmith shops or machine shops, which have been established and operating on a commercial basis for a period of at least two months prior to the award of the contract, shall be subject to the requirements relating to wages, hours of labor and selection of labor set forth in these special provisions.

(c) Construction Code.—The Contractor will be expected to comply with any code of fair competition governing the construction industry, approved under the authority of the provisions of the National Industrial Recovery Act, and any State statutes relating thereto, and any violation of such code or statute will be reported to the proper authorities.

(d) Material Plants.—The construction, erection and operation of material production, proportioning, or mixing plants from which material is used wholly on this contract or on contracts under the supervision of the Department of Public Works shall be considered a part of the work to be performed under the contract and any laborers, workmen or mechanics working on such plants shall be subject to the requirements relating to wages, hours of labor and selection of labor set forth in these special provisions.

SECTION 4. LAWS TO BE OBSERVED.

(a) Alien Labor.—The Contractor shall forfeit as penalty to the State of California ten dollars ($10) for each alien knowingly employed in the execution of the contract, by him or by any subcontractor under him, upon any of the work herein mentioned, for each calendar day, or portion thereof, during which such alien is permitted or required to labor in violation of the provisions of an act entitled

> "An act to prohibit the employment of aliens by contractors and subcontractors on all public work, except in certain cases of extraordinary emergency, providing for the reporting of such cases of extraordinary emergency and the keeping of records of the citizenship of workers employed upon public work and the inspection of such records by the proper officials, providing for a forfeiture for each calendar day, or portion thereof, any alien is knowingly permitted to work on public work and for a stipulation to this effect in the contract, and providing other penalties for violation of the provisions thereof," approved May 25, 1931.

(b) Prevailing Wage.—The Contractor shall forfeit as penalty to the State of California ten dollars ($10) for each laborer, workman, or mechanic employed, for each calendar day, or portion thereof, such laborer, workman, or mechanic is paid less than the general prevailing rate of wages hereinafter stipulated for any work done under the attached contract, by him, or by any subcontractor under him in violation of the provisions of an act entitled

> "An act to provide for the payment of not less than the general prevailing rate of wages on public works, and not less than the general prevailing rate of wages for legal holiday and overtime work on public works, providing for the ascertainment of such general prevailing rate by the public body awarding the contract and its insertion in the contract and call for bids for the contract, providing for the keeping of records of the wages paid all workers engaged in public work and the inspection of such records by the proper public officials, providing for a forfeiture for each calendar day, or portion thereof, any worker is paid less than the said rate and for a stipulation to this effect in the contract, and providing other penalties for violation of the provisions thereof." Chapter 397, Statutes of 1931.

In accordance with the provisions of Chapter 397, Statutes of 1931, as quoted by title above, the Department of Public Works has ascertained the general prevailing rate of wages applicable to the work to be done to be as follows:

Classification	Rate per hour
Laborer	$0.60
Tractor driver (30 h.p.)	0.96
Tractor driver (60 h.p. to 80 h.p.)	1.10
Carpenter	1.00
Carpenter's helper	0.80
Truck driver (15,500 or less)	0.82
Truck driver (over 15,500)	0.96
Power shovel or crane operator	1.25
Oiler	0.82
Grader operator	0.82
Roller operator	1.25
Driller	0.82
Blacksmith	1.00
Blacksmith's helper	0.80
Powderman	0.82
Reinforcing steel worker	1.125
Mechanic (trouble shooter)	0.88
Handyman	0.60
Leverman	1.10
Machinist	1.10
Machinist's helper	0.80
Rigger	1.10
Cranesman	1.10
Assistant engineer	0.95
Assistant electrician	0.85
Fireman	0.80
Levee foreman	0.85
Levee man	0.70
Deck hand	0.80
Mate	0.80
Towboat operator	0.80
Launchman	0.80
Confidential clerk	0.75
Any classification omitted herein not less than	0.60
Overtime	one and one-half (1½) times the above rates
Sundays and holidays (except watchmen, guards and flagmen)	double the above rates

The Director of the Department of Industrial Relations has determined that the classifications of leverman, mate, towboat operator, confidential clerk, levee foreman, assistant engineer, and assistant electrician enumerated above shall be exempt from the application of the provisions of an act establishing a thirty hour week on public works as more specifically set forth in Section 4, article (c), "Laws to be Observed," of these special provisions.

The fifth paragraph of Section 7, article (a), "Laws to be Observed," of the Standard Specifications in words and figures following:

> "The minimum compensation to be paid for labor on the work to be performed by the contractor or by any subcontractors on the work shall not be less than two dollars ($2) per day,"

is hereby canceled, in accordance with Chapter 396, Statutes of 1931, approved May 25, 1931, repealing "An act fixing the minimum rate of compensation for labor on public work," approved March 9, 1897.

(c) **Hours of Labor.**—The Contractor shall forfeit as penalty to the State of California ten dollars ($10) for each laborer, workman, or mechanic, except as otherwise noted in article (b) above, employed in the execution of the contract by him or by any subcontractor under him for each calendar day of each week during which such laborer, workman or mechanic is required or permitted to labor more than thirty hours in violation of the provisions of an act entitled

"An act to add a new section to the Penal Code to be numbered 653c-1, relating to the hours of labor on public works during the present emergency whether such work is done by contract or otherwise, providing penalties for violations of its provisions, and declaring the urgency thereof," Chapter 632, Statutes of 1933.

Attention is called to Chapter 1144, Statutes of 1931, approved June 19, 1931, amending section 653c of the Penal Code relating to the hours of labor on public work.

Attention is also called to Chapter 41, Statutes of 1893, approved February 27, 1893, providing that every person employed in any occupation of labor shall be entitled to one day's rest in seven.

(d) Registration of Contractors.—Before submitting bids, Contractors shall be licensed in accordance with the provisions of Chapter 791, Statutes of 1929, as amended.

(e) Domestic Materials.—Only such unmanufactured articles, materials and supplies as have been mined or produced in the United States, and only such manufactured articles, materials and supplies as have been manufactured in the United States, substantially all from articles, materials and supplies mined, produced or so manufactured, as the case may be, in the United States, shall be used in the performance of the contract in accordance with the provisions of an act entitled

"An act to require the use of materials and supplies substantially produced in the United States in public works and for public purposes," Chapter 226, Statutes of 1933.

Any person, firm or corporation who fails to comply with the provisions of the act shall not be awarded any contract to which the act applies for a period of three years from date of violation.

(f) Codes of Fair Competition.—In accordance with the provisions of the California Industrial Recovery Act, Chapter 1039, Statutes 1933, the Contractor shall give a preference of fifteen per cent (15%) to supplies, articles and materials mined, produced, manufactured, or supplied in observance of any code of fair competition approved, prescribed or issued under Title I of the National Industrial Recovery Act, or under the California Industrial Recovery Act, or in pursuance of any agreement entered into or approved under said laws, or in conformity with the terms prescribed in any licenses issued under said laws, as the case may be, and which concerns such trade or industry and subdivision thereof as may be involved.

Any Contractor who furnishes materials or supplies for public use or on public works in violation of the above provision shall not be entitled to payment for the materials or supplies furnished to the extent of such violation.

(g)) Delay or Default of Contractor.—If at any time in the opinion of the Director of Public Works the Contractor has failed to supply an adequate working force, or material of proper quality, or has failed in any other respect to prosecute the work with the diligence and force specified in and by the terms of the contract, said Director may, after written notice of at least five days to the Contractor, specifying the defaults to be remedied, provide any such labor or materials and deduct the cost thereof from any moneys due or thereafter to become due to the Contractor under the contract.

If said Director shall consider that such failure is sufficient ground for such action, he may give written notice to the Contractor and the Contractor's surety or sureties, of at least five days, that if such default or defaults be not remedied the Contractor's control over the said work will be terminated, and in case such default is not so remedied within the time specified in said notice, the Contractor's control shall terminate as of said time, and thereupon the said Director, or his duly authorized representatives, may take possession of all or any part of the Contractor's materials, tools, equipment, and appliances upon the premises and use the same for the purpose of completing said contract, and may complete said work, doing all or any part thereof by day's labor or by employing another Contractor or Contractors on informal contracts, or both. Such informal contracts shall only be awarded after a proposal form has been prepared, a copy thereof served upon the said Contractor, and three days allowed thereafter so that said Contractor may have opportunity to cause others to bid. Any person may bid on such informal contracts who is prequalified therefor as provided herein. None of the provisions of Chapter 788, Statutes of 1933, except as to prequalification, shall apply to the award of such informal contracts.

In case the control of the Contractor over such contract is terminated, or in case the Contractor abandons the work, he shall not be entitled to receive any further balance of the amount to be paid under the contract until the same shall be fully completed, at which time, if the unpaid balance shall exceed the amount expended by the State in finishing the work, together with all damages sustained or to be sustained by the State, the excess not otherwise required by law to be retained shall be paid to the Contractor, but if such expense and damages exceed the unpaid balance, the Contractor and his surety shall be liable to the State therefor.

On the completion of the contract, the original Contractor shall be entitled to the return of all his unused materials, and his equipment, tools, and appliances, except that he shall have no claim on account of usual and ordinary depreciation, loss, and wear and tear.

Any notice above provided for may be served on the Contractor personally, or on his agent having charge of the work, or by registered mail addressed to such Contractor or agent, or if such Contractor or agent can not be located or their addresses are unknown, then by posting such notice in a conspicuous place upon the premises of the work.

SECTION 5. STATE FURNISHED MATERIALS.

The State will furnish corrugated metal pipe and tidegates for use under these special provisions.

The materials above specified will be delivered by freight or truck to any available freight siding or water shipment point designated by the Contractor in Berkeley, Oakland or Emeryville.

DESCRIPTION OF WORK.

Section 6. The work to be done consists, in general, of constructing a graded roadbed of variable width, involving dredging out of underlying mud to depths of from six feet (6′) to fifteen feet (15′) below the mud surface level, and wasting within the limits and at the locations shown on the plans, or designated by the Engineer; placing a dredger fill from a specified source of supply to a completed average height of approximately thirteen feet (13′) above mean lower low water, to be retained upon the waterward side by a rock retaining wall which is to be placed under a separate contract progressively with the work herein contemplated.

Sections of Oakland's Twenty-second Street, Thirty-sixth Street and Yerba Buena Avenue outfall concrete sewers may require removal and reconstruction to provide dredger access for the work proposed.

Corrugated metal pipe culverts are to be installed, existing and new sewers are to be connected and extended through the highway fill, tidegates are to be installed and concrete culvert structures with and without piles are to be constructed, all as shown on the plans, as hereinafter specified, and as directed by the Engineer.

SECTION 7. COOPERATION, RESPONSIBILITY FOR DAMAGE AND PERMITS.

(a) Cooperation and Responsibility.—The Contractor's attention is directed to Section 5, article (f), "Cooperation of Contractor," and to Section 7, article (h), "Responsibility for Damage," of the Standard Specifications. Sections of highway adjacent to this contract will be under construction during the progress of the work herein contemplated. Contractors shall cooperate with each other to the end that any hindrance or delay to the work may be avoided.

(b) Permits.—The State will obtain all government permits necessary for the prosecution of this work, but all permits necessary for construction access or operations required by municipalities, port authorities, commercial firms or private property owners shall be arranged for and obtained by the Contractor.

CONSTRUCTION DETAILS.

SECTION 8. MISCELLANEOUS ITEMS.

(a) Other Coincident Construction Within the Limits of This Contract:

(1) The Contractor on Contract No. 7 of the San Francisco-Oakland Bay Bridge for superstructure has been promised the completed fill and wall (except for subsidence) between Stations M 273+16 and M 290 by February 1, 1934, for storage space, and yard layout. There will later be let between Station M 273+16 and M 284 other contracts for portions of bridge work during the time limit set for completion of this work; also contracts for Toll Plaza construction, Stations M 320 to 324.

(2) At the Paraffine Companies Inc. plant, approximately at Stations C 33 to C 40, there will be a concrete subway under construction and rearrangement of plant facilities under separate contract or contracts, before, during which, and after, it will be essential to keep the Paraffine Companies' water shipment channel, wharves, docks and water supply lines free and clear from obstruction and operating at all times.

(3) There will be rearrangement, reconstruction, or new construction of other plant facilities on the MC and C lines under separate contract.

(4) A rock retaining wall contract, with identical limits as to general location and time, will progress with and become an integral part of the dredger fill herein specified.

The Contractor shall cooperate completely, and coordinate his work so as to cause no delay to, or interference with, any of the coincident work outlined, and shall assist in every way possible in planning his work so as to promote the progress of the project as a whole. The Engineer will act as an advisory coordinator on the project as a whole, and in case of dispute his decision shall be final.

In case it should be feasible or desirable to include any of the work outlined under paragraphs (2) and (3) in this contract, the Contractor will be required by the Engineer to perform portions or all of said work as extra work as provided in Section 9, article (d) of the Standard Specifications.

(b) Order of Work and Progress.—The Contractor will be required to complete to finished grade elevation, subject to future subsidence, that section of work between the beginning of the project and Station M 290 by February 1, 1934.

The work shall progress by stages or lifts as hereinafter specified continuously from Station M 290 to the end of the project, excepting at the

Paraffine Companies' plant where it may be necessary to delay the completion of the fill for rearrangement of plant facilities. That section of fill north from Station C 40 to the end of the project may be placed at any time within the time limit specified for completion of the contract, after the construction of the rock wall, and necessary culverts and sewers have been provided for.

The Contractor's attention is directed to the fact that certain sections of the fill and rock wall will subside greatly during the construction period. The Contractor will be required to place additional fill material from time to time at locations designated by the Engineer.

The Contractor will be required to furnish a plant of sufficient capacity to place an average amount of dredger fill of two hundred fifty thousand (250,-000) cubic yards per month.

The plant for dredging in the roadway section shall be of such type and capacity as to complete the project as planned without hindrance or delay to the orderly progress of the fill construction and completion of the project as a whole.

The Contractor's attention is directed to Section 4, article (g), of these Special Provisions, "Delay or Default of Contractor."

(c) Removal or Replacement of Excess Material.—Should the Contractor, during the progress of the work, remove excess material or place excess material outside the limits as staked by the Engineer, subject to the allowable variations specified herein for the class of work being performed, it shall be at the Contractor's expense and shall be subject to such penalties as provided herein. Should the Contractor lose, dump, throw overboard, or otherwise place any portions of plant, equipment or material which may be dangerous to or obstruct navigation, he shall recover and remove such obstructions immediately at his own expense, and shall absolve the State from any and all claims for damages to municipalities, port authorities, commercial firms, private property owners and the Federal government.

(d) Navigation.—The Contractor shall comply with all requirements of the Federal Government with regard to navigation in transporting materials and equipment to and from the site of the work. Proper lights shall be kept upon all floating plants connected with the work, and the Contractor shall be responsible for all damages resulting from any neglect or failure in this respect.

(e) Free Movement of Tides Across Right of Way.—The Contractor's attention is directed to the tide flat areas lying easterly of the highway between Station C 6+00 to Station C 22+00 and Station C 44+00 to Station C 72+25. There are large operating commercial plants adjacent to the shore line confronting these tide flat areas which obtain supplies of salt water at stages of high tide for use in their manufacturing processes. Culvert openings under the contract have been planned sufficient in size to maintain large reservoirs of water in these areas until such time as the land may be reclaimed.

The Contractor shall so plan his work in the construction of culvert structures and fill as to insure free movement of tide waters from one side of the highway to the other equal to or greater than that provided by the structures contemplated herein under this contract, subject to revision or enlargement as determined by the Engineer.

(f) Obstructions.—Minor obstructions within the highway right of way shall be removed at the Contractor's expense. The Port of Oakland pipe and trestle at Station M 314 and all miscellaneous buildings adjacent to the Oakland outfall sewer between Stations M 346 and M 363 and the boat house Station C 71+00 shall be removed and disposed of as shown on the plans. The cost of removing these obstructions shall be considered as included in the prices paid for the other items of work and no additional allowance will be made therefor. An advertising sign at Station MC_2 381+00, an old pile and timber dock at Station NC 375 an 8″ salt water line on frame bent trestle Station C 65, and all obstructions except at the Paraffine Companies plant, will be removed by others, or by the State, prior to the time that the adjacent highway area is required for construction under this contract. In case these have not been removed by such time, in order that there be no delay to the work contemplated herein, they shall be removed by the Contractor, if so ordered by the Engineer, and the removal thereof will be paid for as extra work as provided in Section 9, article (d) of the Standard Specifications.

(g) **Temporary Bulkhead at Toll Plaza.**—A temporary bulkhead for retaining the dredger sand fill at the location, and according to the general design shown on the plans, or other acceptable design submitted by the Contractor and approved by the Engineer, shall be constructed and maintained as required during the construction of the fill at the Toll Plaza location until such time as a contract has been let by the State for construction of the Toll Plaza buildings and structures, at which time the Contractor will be relieved from any further responsibility for said bulkhead and fill at this specific location. The cost of constructing this bulkhead shall be considered as included in the prices paid for the other items of work and no additional allowance will be made therefor.

(h) **Protection of Oakland Outfall Sewer.**—The Contractor shall take every precaution during dredging of roadway areas, and during construction of dredger fill, toward protecting from displacement or damage all portions of the main Oakland outfall sewer from Station M 343+60 to the easterly shoreline, including the Twenty-second Street, Thirty-sixth Street and Yerba Buena Avenue sewers. The general construction of these sewers is shown on the plans.

The State will assume full responsibility for damage to the sewers provided the Contractor does not deviate from current instructions in force from the Engineer as to the manner in which the construction work is to be prosecuted adjacent to said sewers.

The plan and method for constructing the highway section adjacent to these sewers as herein outlined and specified, is thought to be the safest and most economical manner for prosecuting this work. Should it become apparent during the progress of the work, that damage to the sewer be imminent, the plan and method of constructing this section of highway may be altered at the direction of the Engineer and the Contractor shall conduct his operations accordingly.

Such changes or alterations as may be required involving contract units of work will be paid for at the unit prices as bid, unless, in the opinion of the Engineer, such chanages materially increase or decrease the cost of performing the work, in which case a supplemental and equitable agreement of such cost between the Contractor and the Engineer will be made.

If, through carelessness or negligence on the part of the Contractor, or failure to follow current instructions, then in effect from the Engineer, the sewer be damaged, the Contractor shall immediately repair the damaged portions, and shall maintain the sewer in operation, without cost to the State.

Should the sewer be damaged due to no fault, neglect, or failure to follow the current instructions of the Engineer, the Contractor shall immediately repair the damaged portions and maintain the sewer in full operation, in which case such work will be paid for as extra work as provided in Section 9, article (d) of the Standard Specifications.

(i) **Soundings of Mud Depths Under Highway Area.**—The State has made soundings and taken samples of the mud underlying the highway area and has taken soundings and samples from the proposed sources of supply for dredger sand fill.

Any information or data obtained by the State will be made available to the Contractor, but the State does not guarantee, or assume any liability for, any properties or physical characteristics. The Contractor is assumed to have investigated thoroughly, prior to submitting his bid, the characteristics of these materials, their depths and location, and shall base his work upon the data so obtained.

SECTION 9. REMOVAL AND RECONSTRUCTION OF EXISTING SEWERS (FOR DREDGING).

(a) Items Involved.—Items 1 and 2 of this contract provide for the removal and reconstruction of sections of the existing Twenty-second Street, Thirty-sixth Street and Yerba Buena Avenue, Oakland outfall sewers at approximately Stations M 346+82, M 363+26 and MC_1 377+40.

(b) Payment in Lieu of Reconstructing Sewers.—The State would prefer that these sewers be not disturbed if possible, and should the Contractor be able to so plan his work as to accomplish the results desired without disturbing any portion or all of these sewers, then, and in that event, the State will allow the payment of the amount bid by the Contractor for that portion of the work which the Contractor has eliminated from these items. The allocation made in the Engineer's estimate upon which such payment will be based, is as follows:

Item 1. 50 lineal feet of 22nd Street sewer M 346 + 82.
50 lineal feet of 36th Street sewer M 363 + 26.

Item 2. 50 lineal feet of Yerba Buena Ave. sewer MC_1 377 + 40.

Should the Contractor elect to do the necessary work by means not requiring the removing and reconstructing of any portion or all of these sewers, the payment allowed by the State as specified herein, for such portion or portions of the work so omitted, shall be in lieu of any and all costs to the Contractor involved in rearranging his plant, equipment, materials, and construction methods and no additional allowance will be made therefor.

Should the Contractor, in altering his plant, equipment, material, or construction methods for elimination of any portion or portions of the work as herein specified, damage any portion or portions of these sewers, such damage shall be repaired by the Contractor at his own expense and the payment allowed to the Contractor by the State for the elimination of the portion or portions of the work involved shall be full compensation for all such damages and all claims arising therefrom.

(c) Removal and Reconstruction as Planned.—Should the Contractor elect to remove portions or all of these sections of sewers he shall remove only such lengths as will enable his equipment to safely pass through the sewer involved. The Contractor shall maintain in operation any or all of such sewers removed to the full satisfaction of the Engineer, and the City of Oakland, and at such time as the occupancy of his plant in this area shall have been terminated, he shall replace the section so removed to the full satisfaction of the Engineer and the City of Oakland, or he shall reconstruct the sewers in accordance with the design shown on the plans of Class "A" concrete, reinforced as shown, to the full satisfaction of the Engineer and the City of Oakland.

Payment for removal and reconstruction of the section or sections of sewers involved shall be based on the actual length removed and shall include full compensation for all labor, plant, equipment, piles, concrete, reinforcing and all other materials required for removing, maintaining in operation, and reconstruction of such sewers.

SECTION 10. STRUCTURES.

(a) General.—Subject to the requirements of Section 8, article (e) of these special provisions, or the maintaining in operation of present sewers as herein specified, installation of pipe and structures shall be made at such stage of the work as may be directed by the Engineer, but shall, in general, be deferred until the highway fill has been completed to at least mean higher high water, or until the major portion of anticipated subsidence has been realized.

(b) **Structure Excavation.**—Structure excavation and backfill shall conform to the provisions of Section 11, articles (o) to (r) inclusive, of the Standard Specifications as herein modified.

Payment will be based upon the combined quantities of excavation and backfill for all pipes, sewers, and structures contemplated under this contract at the unit price per cubic yard bid by the Contractor for this work.

(c) **Installing Corrugated Metal Pipe.**—Corrugated metal pipe, band couplers, fittings, and tidegates will be furnished by the State and shall be installed by the Contractor at the locations shown on the plans, or designated by the Engineer, in accordance with the requirements of Section 36, articles (l), (m) and (p) of the Standard Specifications, except as herein modified.

Those sections of pipe to be placed through the rock wall shall be installed by the Contractor under this contract in cooperation with the Contractor for the rock work during such rock wall construction, to the elevations determined by the Engineer to be proper considering probable subsidence to ultimate desirable flow line grade. The end toward the fill shall be plugged temporarily until such time as the fill shall have been completed to a stage where the Engineer deems it advisable to, and orders the connection to, and installation of, the remaining portion of the pipe through the fill.

Full compensation for hauling and placing tidegates shall be considered as included in the prices paid for the other items of work, and no additional allowance will be made therefor.

(d) **Concrete Structures.**—Concrete shall be Class "A" as specified in Section 29, "Portland Cement Concrete," of the Standard Specifications.

Concrete structures shall be constructed in accordance with the provisions of Section 26, "Concrete Structures," of the Standard Specifications. There is a fresh water main along the Key Route Mole fill, and fresh water connections are located at the Judson Manufacturing and Paraffine Companies' plants. The Contractor shall make all arrangements for obtaining a satisfactory water supply.

(e) **Reinforcing Steel.**—Reinforcing steel shall conform to the requirements of Section 30, "Reinforcement," of the Standard Specifications, as herein modified.

Intermediate grade steel shall be used for bar reinforcement in lieu of structural grade as specified in Section 30, article (b), of the Standard Specifications.

The following table shall supersede and be used in lieu of the table in Section 30, article (l), of the Standard Specifications for calculating the weights of reinforcing steel to be paid for:

Size	Weight per lineal foot, deformed square
½"	0.86
1"	3.44
1⅛"	4.35
1¼"	5.37

Size	Weight per lineal foot, deformed round
¼"	0.17
⅜"	0.38
½"	0.68
⅝"	1.06
¾"	1.52
⅞"	2.07
1"	2.70

The following paragraph shall supplement Section 30, article (d), of the Standard Specifications.

The mill shall tag each bundle of reinforcing steel with an identifying mill tag, showing the name of the mill and the melt or heat number. This tag shall be preferably a metal tag attached with a lead seal and placed in an exposed position for easy identification by the Inspector.

(f) Timber Piles.—Timber piles shall be Douglas Fir, untreated, and shall be furnished and driven in accordance with the provisions of Section 39, "Timber Piles," of the Standard Specifications.

(g) Sewers.—Vitrified clay pipe shall conform to the requirements set forth in Section 38, articles (a) and (b) of the Standard Specifications as herein modified.

The dimensions for double strength vitrified salt-glazed sewer pipe of standard manufacture shall be substituted for and used in lieu of the dimensions specified in Section 38, article (b), of the Standard Specifications.

Joints shall conform to the requirements of Section 38, article (d) of the Standard Specifications.

Sewers, the placing of which may be delayed until the highway fill has reached a stage of completion, or near completion, shall be laid in accordance with the provisions of Section 38, article (c) of the Standard Specifications.

Those sewers which involve connection to, and extension of, existing sewers which must be maintained in continuous operation shall be laid to conform to the following general provisions: Where dredging of roadway area is involved, they shall not be laid or extended until such dredging has been completed, and the highway fill constructed to the maximum elevation possible without clogging or interfering with present sewer operation. The sewer shall then be laid through the fill on a timber cradle and grout bed as shown on the plans. Where dredging of the roadway is not involved the Contractor shall first excavate underlying mud to firm foundation as determined by the Engineer, and shall then backfill with selected coarse sand to a height sufficient to thoroughly imbed the timber cradle in the sand, after which the pipe shall be laid in a grout bed as shown on the plans. Selected sand backfill will be paid for as extra work as provided in Section 9, article (d) of the Standard Specifications in lieu of the unit price bid for structure backfill.

Full responsibility shall be assumed by the Contractor for any or all disturbances or damage to the sewer in completing the highway fill.

The price paid per lineal foot for the various sizes of vitrified sewer pipe in place shall include full compensation for furnishing and placing the pipe, cementing the joints, furnishing and placing the timber cradle and grout bed, maintaining the flow of existing sewers, and all incidental work connected therewith. The excavation and backfill of the trench will be measured and paid for as provided in Section 11, articles (o) to (r), of the Standard Specifications, except that the furnishing and placing of selected sand backfill will be paid for as extra work as above provided.

(h) Drop Inlets and Cutting Sewer Openings.—Drop inlets shall be of the side inlet type with concrete cover as shown on the plans.

At locations shown on the plans or designated by the Engineer for drop inlet connections to the Twenty-second Street, Thirty-sixth Street, or Yerba Buena outfall sewers, round openings eighteen inches in diameter shall be cut into the top or side of these sewers. Openings shall be neat and uniform with reinforcing steel cut off or bent into adjacent new concrete or grout. Pipe or inlet connection shall be plastered with 1 to 2 cement sand mortar and flared in the direction of sewer flow. Payment will be made at the unit price bid per opening and shall include full compensation for furnishing all materials, equipment and labor involved in cutting the openings as herein specified.

SECTION 11. DREDGING.

(a) Scope of Work.—Dredging, within the highway area, shall be at the locations and to the depths and dimensions shown on the plans and cross-sections, subject to the conditions herein specified.

(b) Plant and Progress.—The plant and progress of the work shall conform to the requirements of Section 8, article (b), of these special provisions.

(c) Obstructions.—Obstructions above the water line are shown on the plans. The State assumes no responsibility for any obstruction below the water and mud line such as sunken vessels, equipment, piles, and so forth.

(d) Alteration of Quantities Due to Adjacent Existing Improvements.—Should it become apparent that the dredging work as planned is endangering or undermining the adjacent Key Mole fill, or the adjacent Oakland outfall sewer, the State reserves the right to decrease the width of, or eliminate the dredging herein contemplated in order to protect these improvements, and payment will be based upon the decreased quantities as set forth in article (b), of the attached Proposal Requirements and Conditions.

(e) Overdepth Dredging and Side Slopes.—To cover unavoidable inaccuracies of dredging processes, material actually removed to a depth of not more than one foot below the depth specified will be estimated and paid for at the contract price.

Side slopes shall be approximately one vertical to two horizontal. The Contractor will not be required to dredge the side slopes strictly to the theoretical line, but may approximate the side slopes by a series of steps between the bottom of the excavation and the mud surface level, the material actually removed within these steps to provide the required side slopes. An overdepth of one foot measured vertically on a parallel slope will be allowed in computing limiting amounts of side slope dredging. Material from beyond the limits herein described will be deducted from the amount dredged as excessive dredging and will not be paid for.

Dredging quantities shown on the plans and included in the estimate are computed to the neat lines as shown on the profile and cross-sections and do not include the allowable overdepth quantities.

(f) Disposal of Excavated Material.—Excavated material shall be disposed of in the areas shown on the plans. Adjacent to and northerly of the Oakland outfall sewer the material excavated from the M line shall be deposited so as to provide a fill of uniform height with flat natural slopes toward the sewer, with the approximate toe of slope at the sewer. High point of this waste fill should be above high water.

After placing the highway fill on both sides of the sewer as shown on the typical section, the excavated material from the MC and C lines shall be placed between the northerly slope of the highway fill and the high ridge of waste from the M line as herein specified and as shown on the typical section for this portion of work adjacent to the sewer.

The Contractor shall take every precaution to insure against water pockets adjacent to and endangering the Oakland outfall sewer in disposing of excavated material and shall keep the outlet and channel to open water clear at all times.

All provisions as to dredging and disposal herein shall be subject to the requirements of Section 8, article (h) of these special provisions.

A portion of the material excavated from the MC line shall be disposed of adjacent to the Key Route fill and easterly shore line as shown on the plans.

(g) Dredged Area to Be Filled Immediately.—The Contractor shall so conduct his operations as to fill the dredged highway area with dredger sand fill to mean lower low water as closely behind the dredging as plant operation will permit.

(h) Measurement and Payment.—The material removed will be measured by the cubic yard in place by means of soundings or sweepings taken shortly before and shortly after dredging. Monthly deductions for excessive overdepth and side slope dredging will be made and monthly progress payments will be based upon these measurements.

In case unstable formations are encountered and the methods herein outlined do not appear to give reliable results, average hourly rates of pumping as determined from similar operating conditions for other portions of the work, for a period of 48 hours, will be applied through the unstable section, at the discretion of the Engineer, and will be based on actual time operated as determined from Inspector's daily reports.

The price paid per cubic yard for dredging shall include full compensation for furnishing all labor, materials, tools and equipment and doing all the work involved in dredging and disposing of material as herein specified.

SECTION 12. DREDGER SAND FILL.

(a) Source of Supply.—The material for this contract item shall be obtained within the general limits as shown on the plans. The State reserves the right to select the best quality material available within this area, dredging to depth of 45 feet below mean lower low water in case the class of material so obtained justifies this depth. In dredging this area the Contractor shall make every effort to obtain an average material from the bottom of the excavation to the mud line.

(b) Plant and Progress.—The plant and progress of the work shall conform to the requirements of Section 8, article (b) of these special provisions.

(c) Obstructions.—The State assumes no responsibility for any obstructions, submerged or otherwise, within this area.

(d) Construction of Fill in Stages.—The Contractor shall construct the fill in lifts or stages, the order of which shall vary on different sections of the work, as tentatively shown on the typical sections.

From the beginning of the project to the westerly end of the Oakland outfall sewer the first stage shall be to place the fill on a gradually northerly descending slope so that the elevation at the outer toe of the rock wall, subsequently to be placed, will be at approximate mean lower low water elevation. The rock contractor will then place the first stage of rock wall to slightly above mean higher high water elevation, after which the second lift or stage of the fill shall be placed to the level of this portion of wall. Successive stages thereafter, for both fill and rock wall, shall be in lifts of from four feet to seven feet depending upon the amount of subsidence obtained or anticipated.

For the MC and C line sections, where dredging of highway area is first involved, the stage construction will be similar to that of the outer Mole line as shown on the typical sections. From approximate Station C 20 + 00 to the Paraffine Companies' channel the fill shall be constructed in two lifts only. From approximate Station C 40 + 00 to the northerly end of the project the fill may be constructed to its full height in a single lift.

The first stage of fill construction along the Oakland outfall sewer shall be to place material discharging directly on top of the sewer with slopes not to exceed one vertical to ten horizontal northerly of the sewer coincident with placing stabilizing material south of the sewer. Successive stages will thence progress southerly as shown on the typical sections.

(e) Subsidence and Construction Grade Line.—The stage construction herein outlined has been specified in order to promote subsidence through the orderly consolidation of the underlying strata of soft material without the loss of any of the rock wall or fill material through lateral displacement. It is highly desirable that as much time as possible be allowed to elapse between the successive stages of construction, consistent with the required progress of the project as a whole, in order to produce the utmost stability possible. The Contractor shall give every assistance possible toward determining the actual subsidence that has taken place through consolidation between these stages of construction. Any expense to the Contractor in this connection will be paid for as extra work as provided in Section 9, article (d), of the Standard Specifications.

During the progress of the work the State may elect to do, either under separate contract or as extra work under this contract, experimental work with vertical drains, for promoting orderly consolidation and stabilization of the fill, but such work will not be allowed to interfere with the regular contract work.

Due to anticipated heavy subsidence on sections of this fill, the fill may be constructed to a high grade line varying in height with the anticipated ultimate total subsidence over a period of years. This actual construction grade line will depend upon the consolidation obtained during the stage construction of the fill consistent with a factor of safety due to possible overloading and consequent lateral heave or displacement of the fill. The theoretical grade line shown on the plans will be used as a guide in construction only.

(f) Quantities Shown on the Plans.—Quantities shown on the plans are anticipated embankment quantities obtained by increasing the theoretical embankment section for anticipated consolidation and possible displacement. The loss ratio applied to the quantities shown on the plans has been estimated at twenty per cent, since it is desirable to eliminate all the clay possible from the sandy clay source deposit.

(g) Side Slopes on Shoreward Side and Riprap. The side slopes on the shoreward side of the C line shall be constructed one vertical to four horizontal. As soon as the shoreward side has been constructed as specified to a height above mean higher high water the rock Contractor shall be permitted to place the necessary riprap on these slopes.

(h) Measurement and Payment.—The material dredged from the source of supply will be measured by the cubic yard in place by means of soundings or sweepings taken shortly before and shortly after dredging and monthly progress payments will be based upon these measurements.

The price paid per cubic yard for dredger sand fill shall include full compensation for furnishing all labor, materials, tools and equipment and doing all the work involved in dredging, transporting, and placing the material in the highway fill as herein specified.

SECTION 13. PRIVATE FILL OF PARAFFINE COMPANIES, INC.

Pursuant to the right of way agreement with Paraffine Companies, Inc., the State has agreed to obtain a bid for constructing a private fill under this contract, between the highway fill and the Paraffine Companies' present fill and bulkhead. The bid for constructing the private fill may be accepted or rejected at the discretion of the Paraffine Companies, Inc. This item is not being used as a basis for comparison of bids but the State reserves the right to include it in or exclude it from the contract.

If the State elects to include this work in this contract, the fill may be constructed coincident with the highway fill, provided such construction is performed in a manner that will not cause upheaval or displacement of soft underlying material from the Paraffine Companies' property into the highway right of way.

In accordance with the agreement, the material shall be obtained from the same source of supply as the highway fill. It has been agreed that the yardage involved in this fill is one hundred forty thousand (140,000) cubic yards and in case this work is performed by the Contractor, payment will be made by the State and will be based upon that quantity at the unit price bid by the Contractor per cubic yard for this item of work. Any variations or differences as to actual quantities involved in this private property area will be absorbed in the highway fill, the one hundred forty thousand (140,000) cubic yards being deducted from the total quantities dredged for combined highway and Paraffine Companies' fill, payment by the State for highway fill being based upon the net balance.

SECTION 14. FINISHING ROADWAY.

After the fill has been completed the Contractor shall, with a road grader, blade the surface over the entire roadway width to a smooth, slightly crowned and well drained section to the satisfaction of the Engineer. No hand work will be required.

Payment will be based on the unit price bid per station of 100 feet for finishing roadway and shall include full compensation for all labor, materials, tools, implements, and equipment, and performing all incidental work necessary to finish the roadway as specified.

PROJECT MEMORANDUM
(AUGUST 25, 1999)

PROJECT MEMORANDUM
STUDIES ON LATERAL SPREADING OF FILLS AT OAKLAND MOLE

MEMODRANUM

To: Mr. Thomas Shantz, Mr. John O'Leary/ Caltrans
Dr. Brian Maroney/ Caltrans
Mr. Ade Akinsanya/ Caltrans
Mr. Reid Buell, Mr. John Thorne, Mr. Ronnie Gu / Caltrans
Dr. Abbas Abghari, Dr. Mahmoud Khojasteh / Caltrans

Copy: Dr. I. M. Idriss / Peer Review
Mr. Al Ely, Mr. Gerry Houlahan / TYLin-Moffatt&Nichol
Mr. Jal Birdy / TYLin-Moffatt&Nichol
Mr. Tom McNelian, Mr. Tim Dunne / Fugro-EMI
Dr. Robert Pyke/Fugro-EMI

From: Hubert Law

Date: August 25, 1999

Subject: Studies on Lateral Spreading of Fills at Oakland Mole
SFOBB East-Span Replacement

Gentlemen:

This memo summarizes geotechnical studies conducted so far on lateral spreading of the existing fills and proposed new fills at the Oakland Mole subjected to earthquake loading. The analyses only considered free-field conditions without presence of nearby structures at vicinity of the slope. Several cross sections were chosen for the lateral spreading studies, namely, Station 88+00 (existing fill), Station 86+89 (proposed light weight fill), and Station 86+34 (proposed light weight fill).

Two possible failure modes were postulated in these studies: one mode of failure mechanism is shearing deformation in the Young Bay Mud and the other relates to soil liquefaction and associated lateral spreading in the existing fill. Separate studies were conducted to address potential settlements associated with consolidation of the Young Bay Mud under the proposed new fill, and the findings have been documented in our draft memo dated November 20, 1998 and the final memo dated December 11, 1998.

Subsurface Conditions

The phase II site investigation program on the Oakland Mole consists of 25 soil borings and 9 conventional CPT soundings conducted on land and more than 40 tethered Seascout CPT soundings conducted in the tidal flat to the north of the existing mole. Figure 1 shows the locations of all the borings. A large area immediately next to the shoreline appears to be covered by surficial beach sands. Though the Seascout CPTs were conducted within as close as 30 meters to the edge of the existing rock dike in a shallow water, the extend and nature of this sand were not fully explored by the Seascout CPTs. Additional CPTs were probed in this area using an all-terrain vehicle that is track and buggy mounted. Also, trenching within the beach sands immediately next to the mole took place in the first week of March to explore an extent of the material. Subsurface conditions near Stations 88+00, 86+34 and 86+89 are depicted in Figures 2 to 4.

Stress-Strain Behavior of the Young Bay Mud

Six soil sampling tubes from the Young Bay Mud have been tested at the University of California, Berkeley to investigate a stress-strain behavior under large cyclic strains and post-earthquake properties. The experimental program consisted of testing pairs of specimens from a given sampling tube. One of the specimens was sheared monotonically from a virgin state, while the other was cyclically loaded with a specified strain amplitude and number of cycles and then monotonically sheared again without allowing to reconsolidate the specimen. The purpose is to understand the stress-strain behaviors before and after an earthquake.

All the tests were performed using wire-reinforced membranes to provide lateral restraint. Table 1 summarizes the test results. The monotonic loading was conducted at a loading rate of 1% of shear strain per minute. The cyclic loading was conducted for 25 strain-controlled cycles at a frequency of 0.2 Hz with single amplitude shear strains ranging from 1% to 3%. Figures 5 to 10 show the stress strain curves for monotonic shearing (before and after cyclic test) and the cyclic stress histories. Some samples have in-situ measurements to compare against.

Table 1 Summary of Cyclic Shear Testing Program on YBM

Boring No.	Sample Pair	Depth	Static Shear Strength (Pre-Cyclic)	Cyclic Strain	Initial Cyclic Stress	Final Cyclic Stress @ 25th Cycle	Static Shear Strength (Post Cyclic)
(-)	(-)	(m)	(kPa)	(%)	(kPa)	(kPa)	(kPa)
98-55	1	10.98	45	1	40	33	42
98-52	2	10.98	50	2	52	33	50
98-59	3	9.15	47	1	45	33	56
98-60	4	10.98	48	1.5	45	33	48
98-56	5	18.60	72	3	73	45	60
98-62	6	12.50	51	3	52	33	47

In general, the monotonic tests conducted after cyclic shearing show comparable or smaller static shear strengths compared to the tests conducted on the virgin specimens except the sample pair No 3. The reduction in static shear strength is less than 17%, but the effect is not very obvious for those samples with the cyclic amplitude of less than 3% shear strain. All the tests reveal a flatter initial slope of the stress-strain curve for the specimens that have a prior cyclic stress history.

Idriss et al., (1980) have studied stress-strain-strength behavior of clays under earthquake and wave loading conditions to understand strain-rate effects on Icy Bay Mud from Alaska. They used two frequencies (1 Hz and 0.05 Hz) to conduct cyclic loading in a laboratory and reported that the cyclic shear stress with a 1 Hz frequency is about 68 % higher than the static shear stress at 2 percent shear strain. Even with a 0.05 Hz frequency, their tests show a 46% increase over the static shear stress.

The strain-rate effects were indeed inherent in our cyclic simple shear tests conducted at the U. C. Berkeley as some increase in cyclic shear stress can be seen when it is compared with the monotonic test at the same strain level. However, very limited numbers of cyclic tests were conducted on the Young Bay Mud to draw meaningful conclusions on the strain-rate effects from our own set of tests. We propose to use 40% increase from the static shear strength to account for strain-rate effects for the Young Bay Mud in dealing with site stability and lateral spreading issues.

Lateral Spreading in the Young Bay Mud

The framework of analysis to compute seismically-induced ground deformations has been based on the Newmark's sliding method. The procedure consists of three steps as shown in Figure 11 illustrating an earth slope that is free to move in the direction of the bluff. These three steps can be summarized as:

(1) Calculation of a yield acceleration for the block under investigation using shear strength acting at the bottom of the shear plane (resisting force)

(2) Calculation of a seismic coefficient time history of the block subjected to seismic loading taking into account the geometry and compliance of the slope (driving force)

(3) Double integration of the difference between the seismic coefficient and the yield acceleration to obtain permanent deformations

We have considered three potential failure blocks, and all the sliding planes pass through the Young Bay Mud. The sizes of the block are 25 feet, 50 feet and 100 feet measured from the slope face. Limit equilibrium analysis was adopted to compute yield acceleration using the following strength scenarios:

- Using static undrained shear strengths as measured in the field

- Increasing the static undrained shear strengths by 40% to account for the strain rate effects
- Using remolded strengths to account for degradation

The seismic coefficient time histories of the potential failure blocks were computed for the six reference rock motions developed for the project. We have used QUAD4M computer program to compute the seismic coefficient time histories, which were taken as the shear force acting on the bottom surface of the pre-defined block divided by the total weight of the block.

Newmark's integration was then performed for all six motions. Since the procedure only allows for one-sided movement, permanent deformations were also evaluated by changing the polarity of the earthquake motions. Effectively, twelve permanent displacement time histories were computed for each block. The following different yield accelerations were considered in the Newmark's integration:

- Using a constant yield acceleration corresponding to static undrained strengths
- Using a constant yield acceleration corresponding to 40% increase over the static undrained shear strength to account for the strain rate effects
- Starting with a yield acceleration corresponding to 1.0 times or 1.4 times static undrained strengths and then switching to a lower yield acceleration corresponding to remolded shear strengths when the block displacement exceeds 6 inches

Liquefaction Potential of the Existing Fill

A liquefaction potential of the fill was studied using the relationship between cyclic stress ratio causing liquefaction and standard penetration resistance $(N_1)_{60}$. The in-situ blow count measurements at the Oakland Mole were taken with a cathead system and typical rope with two turns about the cathead that would deliver approximately 60% of the theoretical free fall hammer energy to the drill stem. Therefore the blow-counts as measured in the field are already $(N)_{60}$-values, but they must be corrected to account for effective overburden stress to develop the final standard and corrected penetration resistance $(N_1)_{60}$ at a hypothetical overburden stress of 1 ton per square foot.

Cone penetration tests were also conducted at several locations in the existing fills. Equivalent blow counts can be obtained roughly by dividing the cone tip resistance Q_t in terms of tons per square foot with a factor of 4 (i.e., $N \approx Q_t/4$). From all the in-situ measurements in the existing fills, the typical values of $(N_1)_{60}$ range from 10 to 20 blows per foot. For magnitude 7.5 earthquakes, the fills are considered susceptible to liquefaction.

Lateral Spreading of the Existing Fill

The procedure to evaluate lateral spreading in the fill was similar to the method adopted for the Young Bay Mud. As in the case of sliding in the Young Bay Mud, three soil blocks were analyzed for sliding deformation in the fills. The yield accelerations of the three failure blocks were computed with the drained shear strengths as well as the undrained residual strengths of the fill. A shear strength of 600 psf was selected as the undrained residual strength using on the curve relating undrained residual strength versus equivalent clean sand SPT blow counts as suggested by Idriss (1998) based on case histories (See Figure 12). Some parametric studies were also conducted with a residual undrained strength of 800 psf for a few cases.

Presentation of Results

The results of lateral spreading studies are presented in the appendices as follows:

Appendix	Location	Soil Conditions
Appendix A	STA 88+00	Existing Fills (Grade @ El +13')
Appendix B	STA 86+89	Proposed Light Weight Fill, γ_{fill} = 40 pcf (Grade @ El +20')
Appendix C	STA 86+34	Proposed Light Weight Fill, γ_{fill} = 40 pcf (Grade @ El +23')
Appendix D	STA 86+34	Proposed Light Weight Fill, γ_{fill} = 31 pcf (Grade @ El +23')

Each station was analyzed to two failure mechanisms; sliding in the Young Bay Mud and in the fills. Three soil blocks with six ground motions were examined for different strength scenarios.

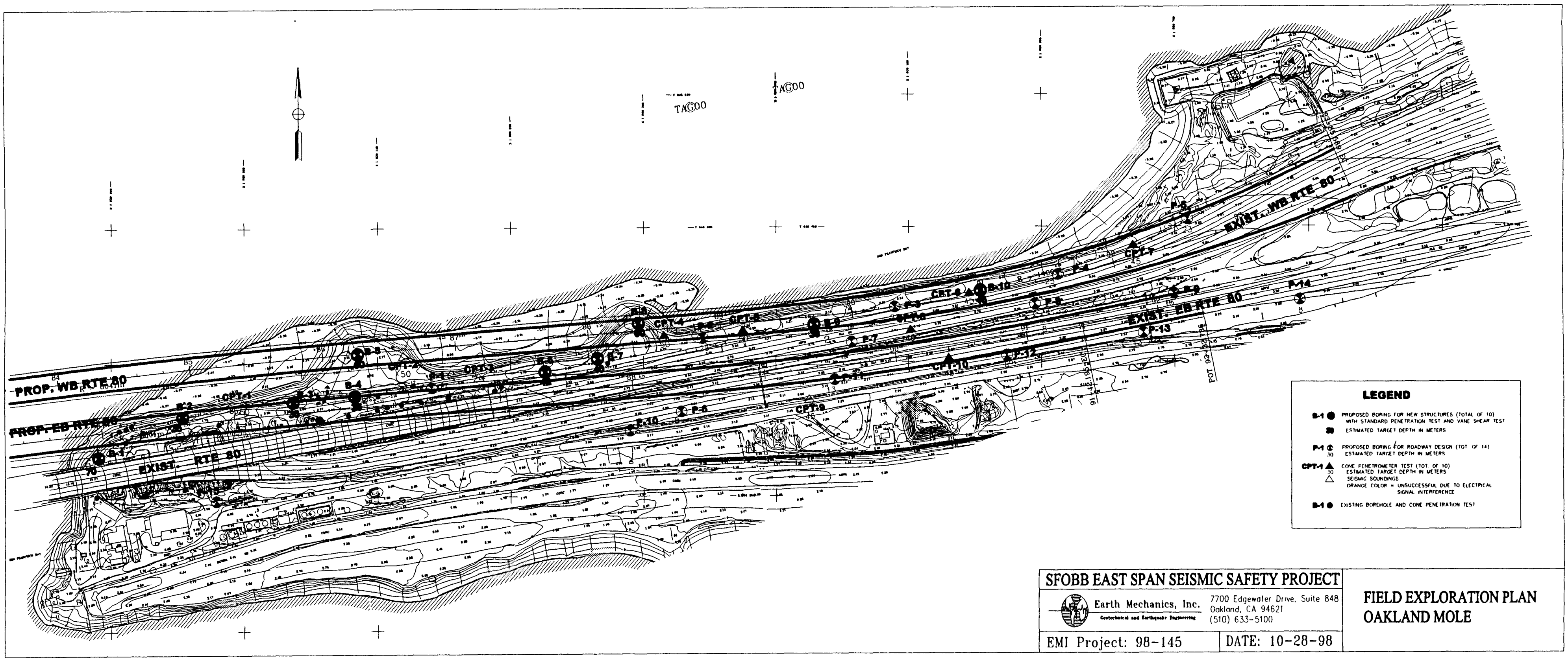

LEGEND
B-1 PROPOSED BORING FOR NEW STRUCTURES (TOTAL OF 10) WITH STANDARD PENETRATION TEST AND VANE SHEAR TEST
ESTIMATED TARGET DEPTH IN METERS
P-1 PROPOSED BORING FOR ROADWAY DESIGN (TOT OF 14)
ESTIMATED TARGET DEPTH IN METERS
CPT-1 CONE PENETROMETER TEST (TOT. OF 10)
ESTIMATED TARGET DEPTH IN METERS
SEISMIC SOUNDINGS
ORANGE COLOR = UNSUCCESSFUL DUE TO ELECTRICAL SIGNAL INTERFERENCE
B-1 EXISTING BOREHOLE AND CONE PENETRATION TEST
PROP. WB RTE 80
PROP. EB RTE 80
EXIST. RTE 80
EXIST. WB RTE 80
EXIST. EB RTE 80
TAGOO
SFOBB EAST SPAN SEISMIC SAFETY PROJECT
Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering
7700 Edgewater Drive, Suite 848
Oakland, CA 94621
(510) 633-5100
FIELD EXPLORATION PLAN
OAKLAND MOLE
EMI Project: 98-145
DATE: 10-28-98

Figure 1

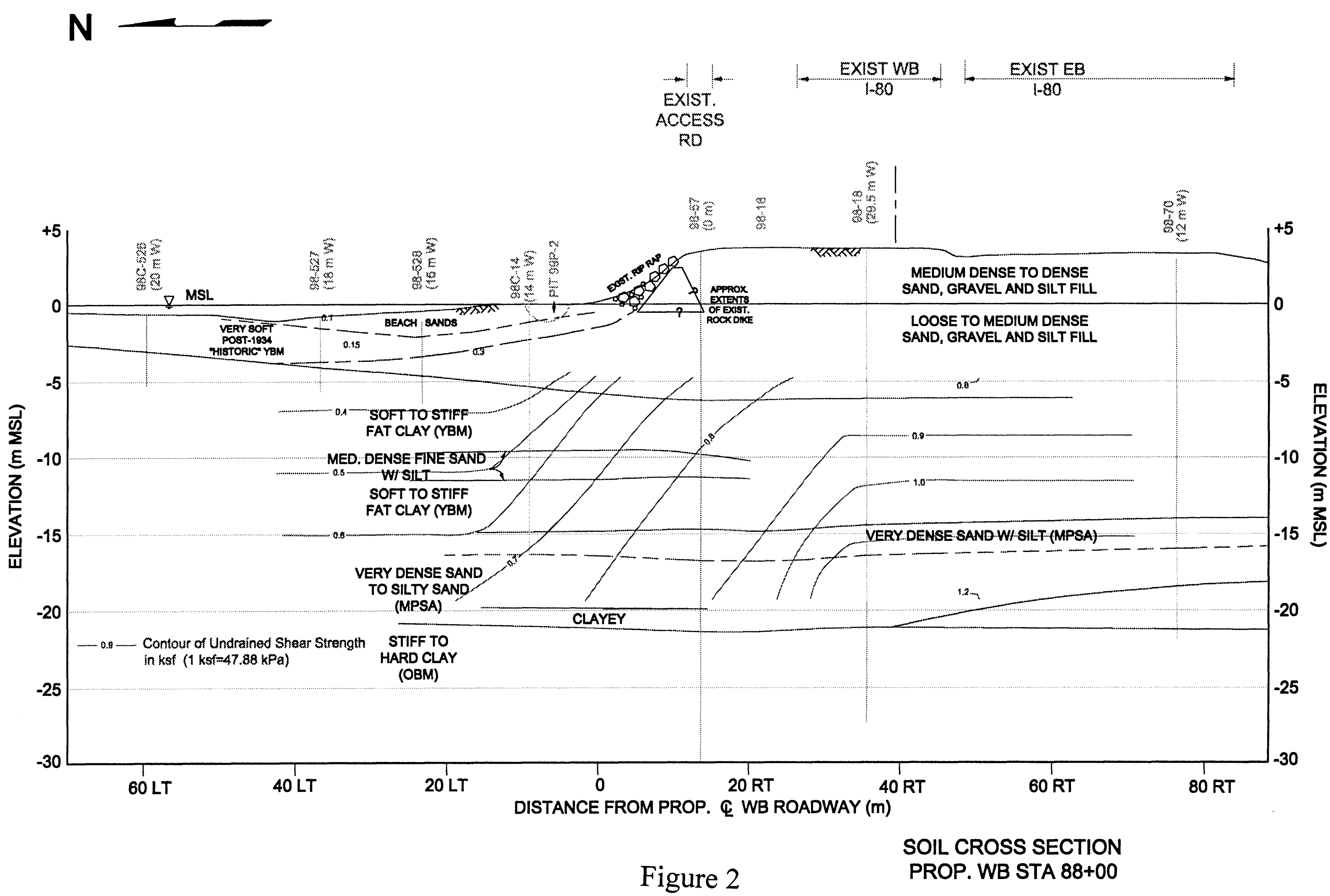

N
EXIST. ACCESS RD
EXIST WB I-80
EXIST EB I-80
98C-526 (20 m W)
98-527 (18 m W)
98-528 (15 m W)
98C-14 (14 m W)
PIT 99P-2
98-57 (0 m)
98-18
98-18 (29.5 m W)
98-70 (12 m W)
MSL
VERY SOFT POST-1934 "HISTORIC" YBM
BEACH SANDS
EXIST. RIP RAP
APPROX. EXTENTS OF EXIST. ROCK DIKE
MEDIUM DENSE TO DENSE SAND, GRAVEL AND SILT FILL
LOOSE TO MEDIUM DENSE SAND, GRAVEL AND SILT FILL
SOFT TO STIFF FAT CLAY (YBM)
MED. DENSE FINE SAND W/ SILT
SOFT TO STIFF FAT CLAY (YBM)
VERY DENSE SAND W/ SILT (MPSA)
VERY DENSE SAND TO SILTY SAND (MPSA)
CLAYEY
STIFF TO HARD CLAY (OBM)
0.9 Contour of Undrained Shear Strength in ksf (1 ksf=47.88 kPa)
ELEVATION (m MSL)
DISTANCE FROM PROP. ℄ WB ROADWAY (m)
60 LT
40 LT
20 LT
0
20 RT
40 RT
60 RT
80 RT
SOIL CROSS SECTION
PROP. WB STA 88+00

Figure 2

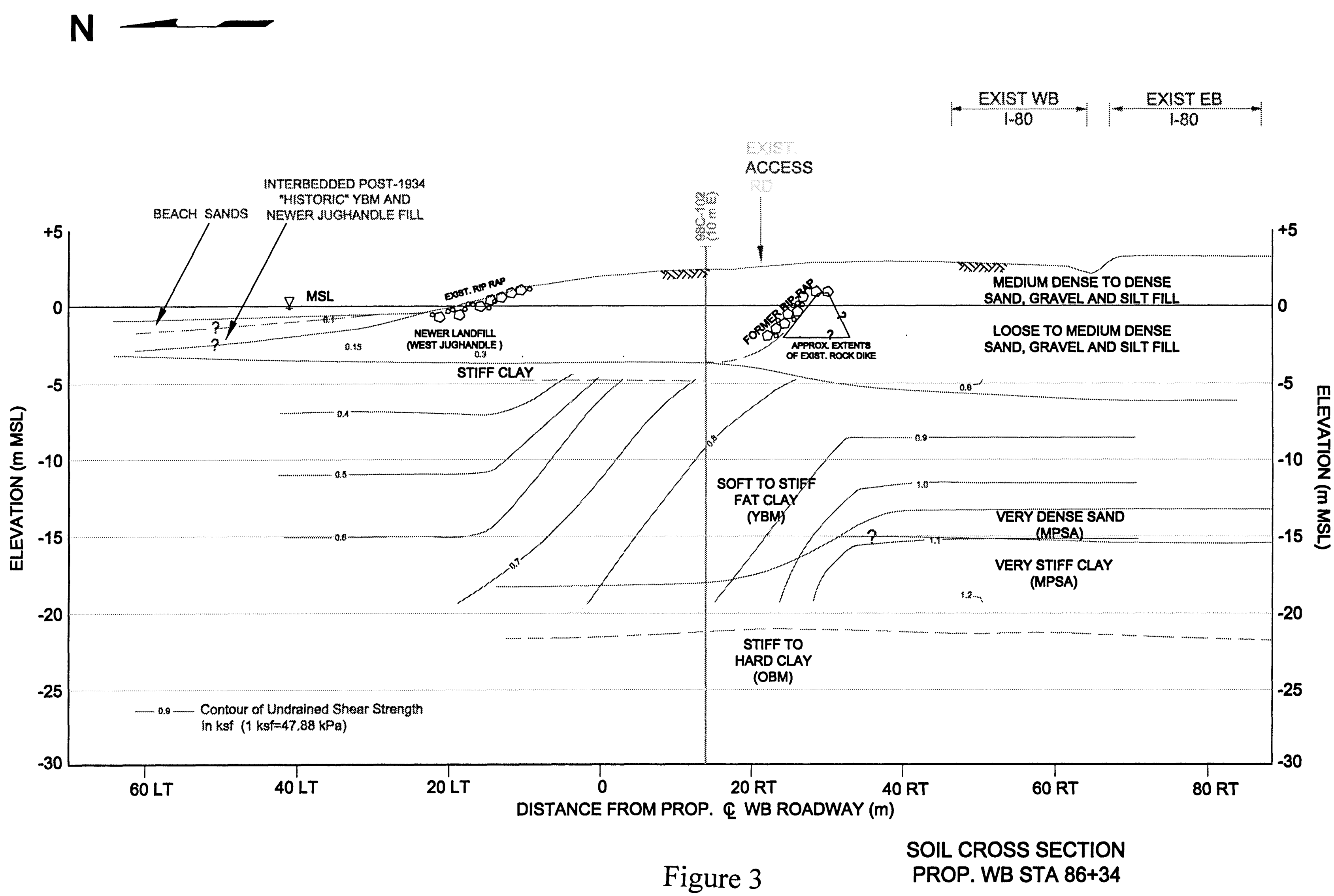

N
EXIST WB
I-80
EXIST EB
I-80
EXIST.
ACCESS
RD
98C-102
(10 m E)
INTERBEDDED POST-1934
"HISTORIC" YBM AND
NEWER JUGHANDLE FILL
BEACH SANDS
MSL
EXIST. RIP RAP
NEWER LANDFILL
(WEST JUGHANDLE)
STIFF CLAY
FORMER RIP-RAP
APPROX. EXTENTS
OF EXIST. ROCK DIKE
MEDIUM DENSE TO DENSE
SAND, GRAVEL AND SILT FILL
LOOSE TO MEDIUM DENSE
SAND, GRAVEL AND SILT FILL
SOFT TO STIFF
FAT CLAY
(YBM)
VERY DENSE SAND
(MPSA)
VERY STIFF CLAY
(MPSA)
STIFF TO
HARD CLAY
(OBM)
0.9 Contour of Undrained Shear Strength
in ksf (1 ksf=47.88 kPa)
ELEVATION (m MSL)
DISTANCE FROM PROP. ℄ WB ROADWAY (m)
60 LT
40 LT
20 LT
0
20 RT
40 RT
60 RT
80 RT
+5
0
-5
-10
-15
-20
-25
-30
SOIL CROSS SECTION
PROP. WB STA 86+34

Figure 3

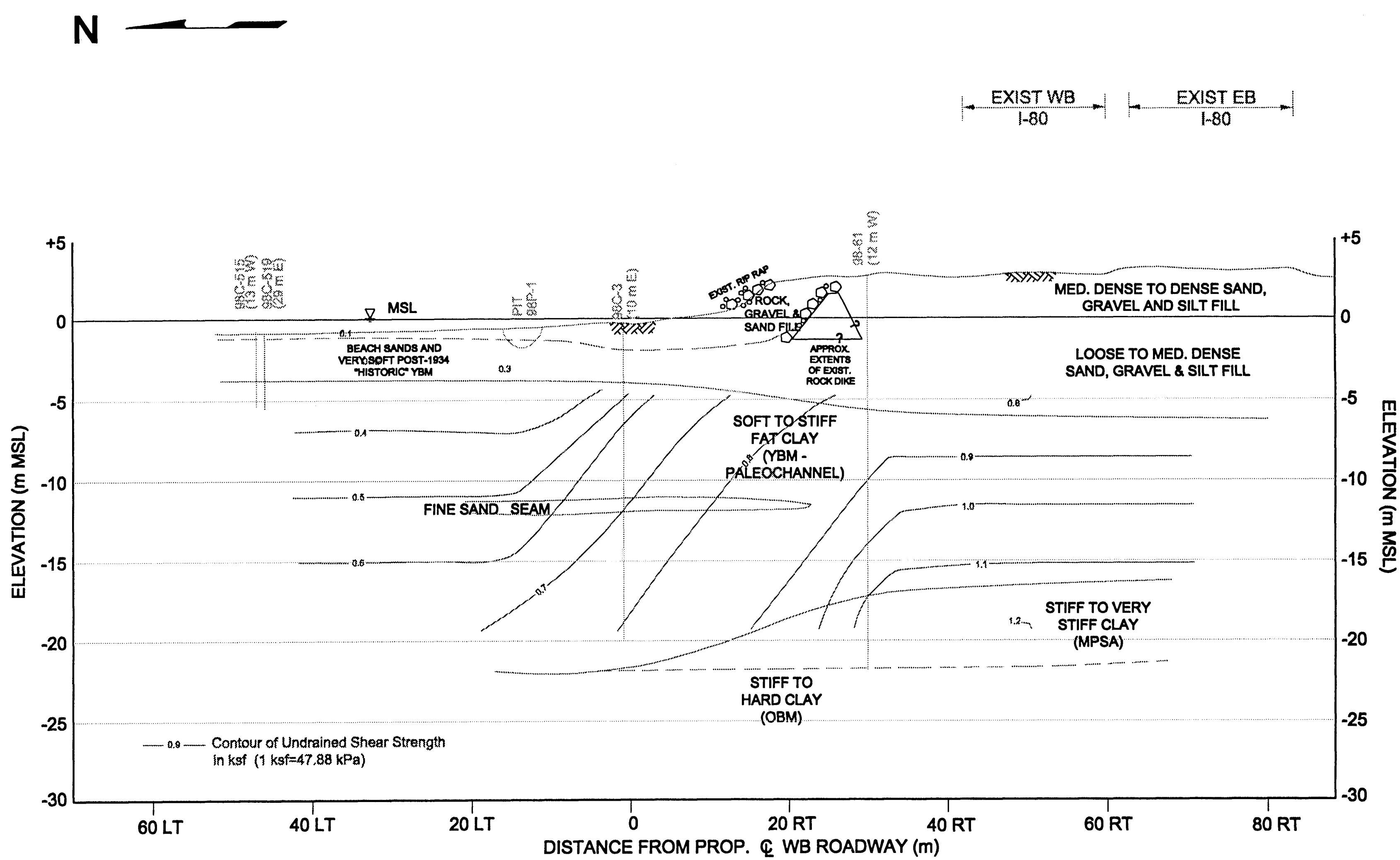
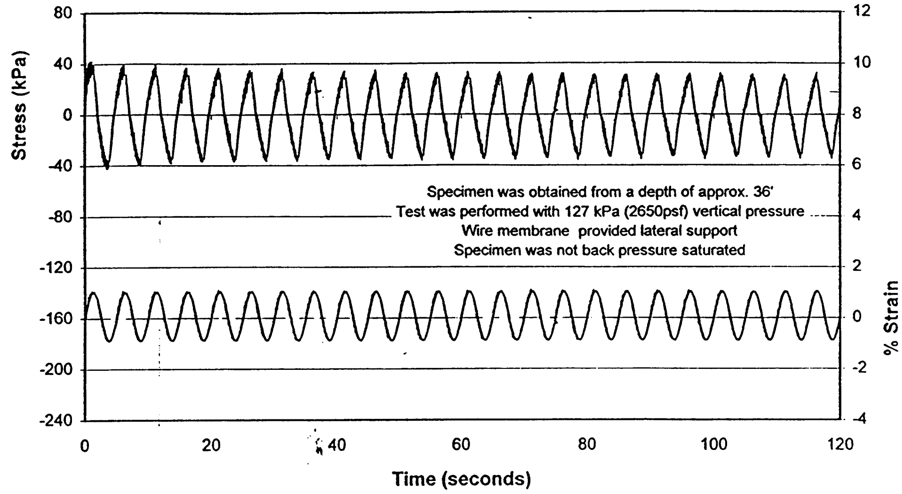


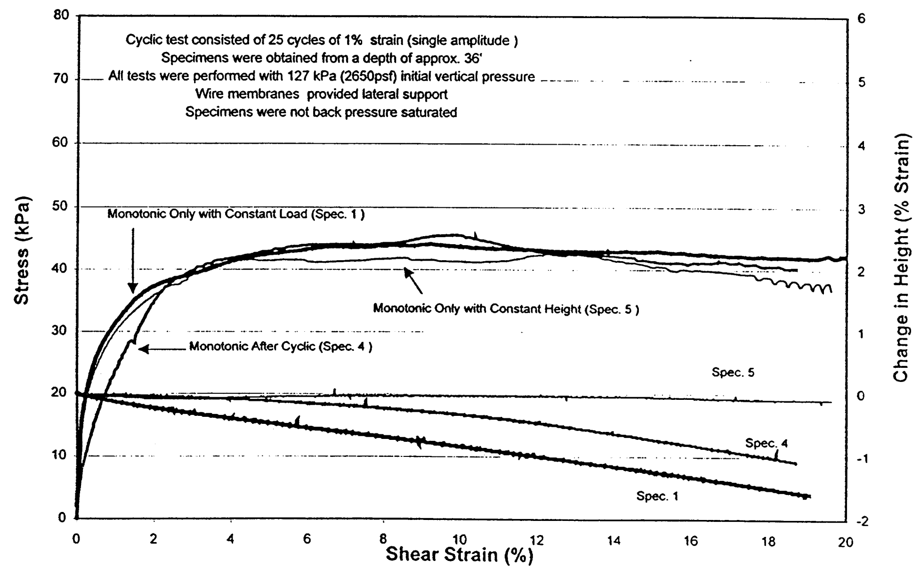
Figure 4

SOIL CROSS SECTION
PROP. WB STA 86+89

98-55

SFOBB Boring B-5 Specimen 4
Cyclic Test to 1% Strain (Single Amplitude)





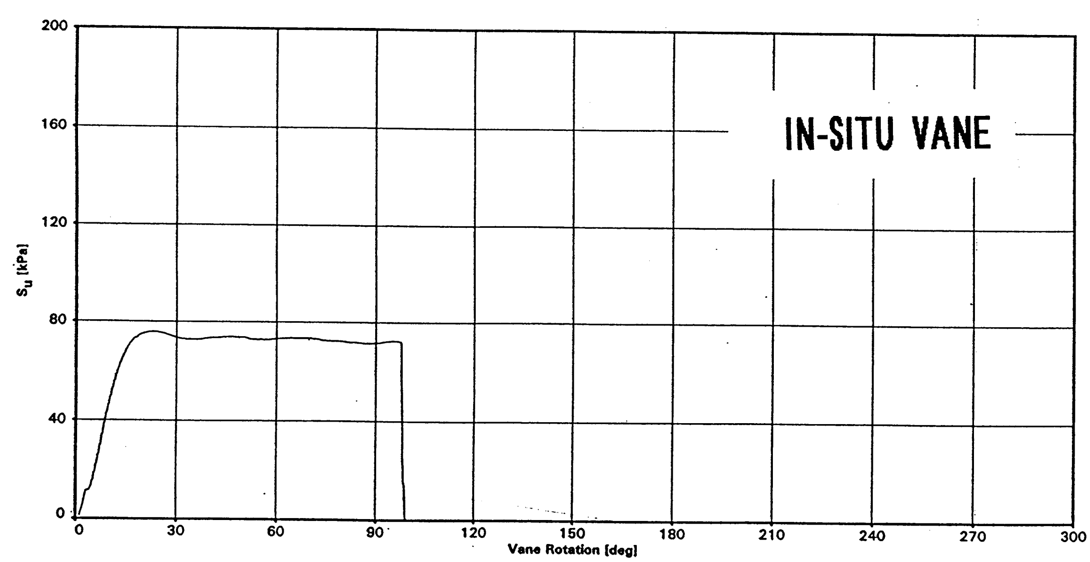
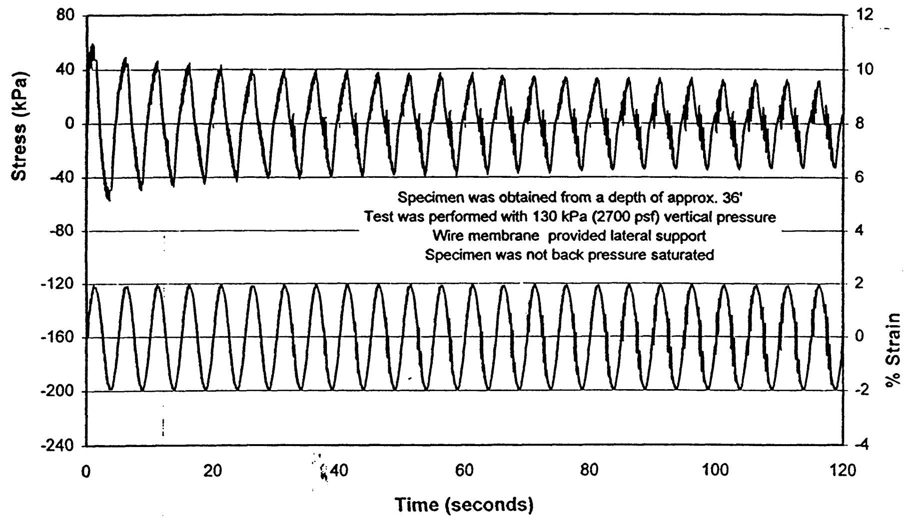


Figure 5

98-52

SFOBB Boring ~~B-2~~ Specimen 8
Cyclic Test to 2% Single Amplitude Strain



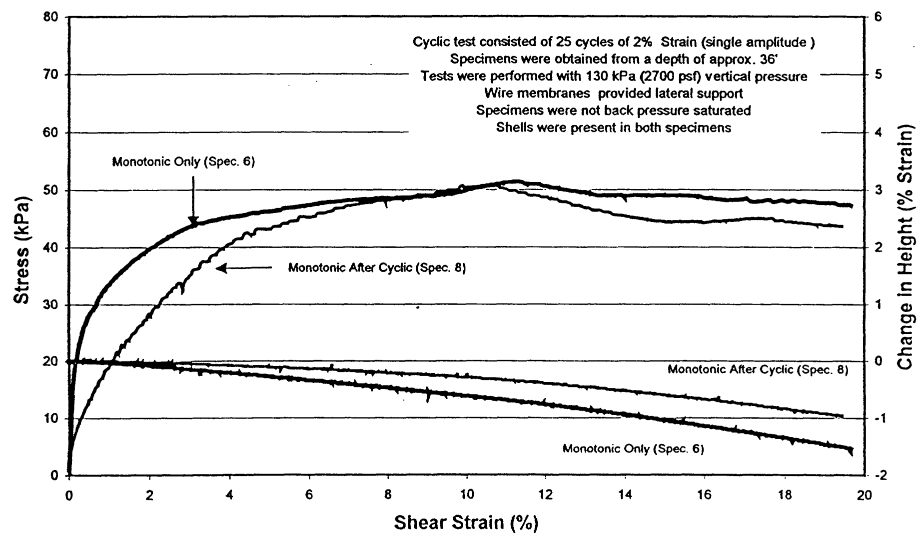
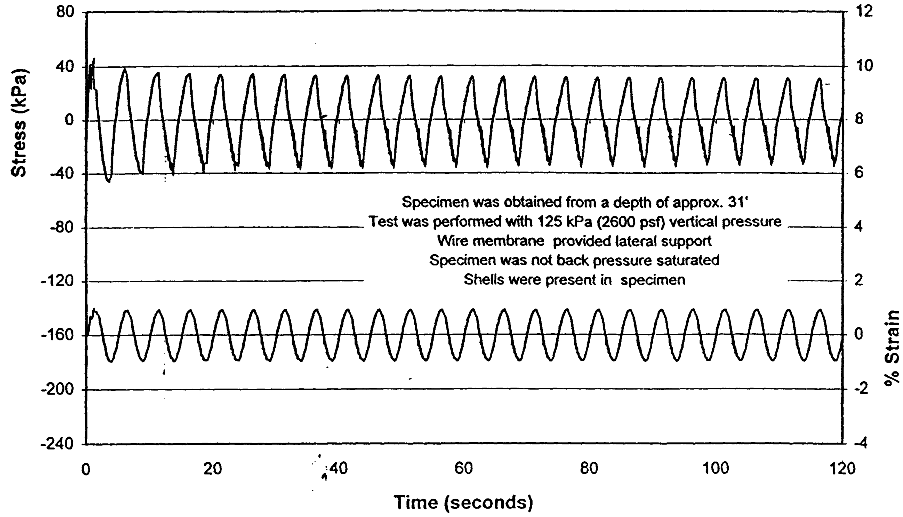


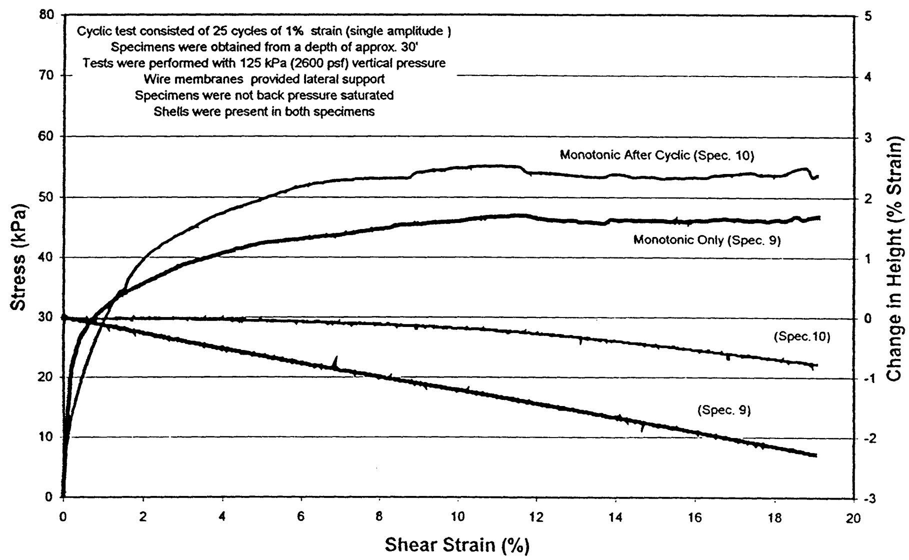
Figure 6

98-59

SFOBB Boring ~~B-9~~ Specimen 10

Cyclic Test to 1% Single Amplitude Strain





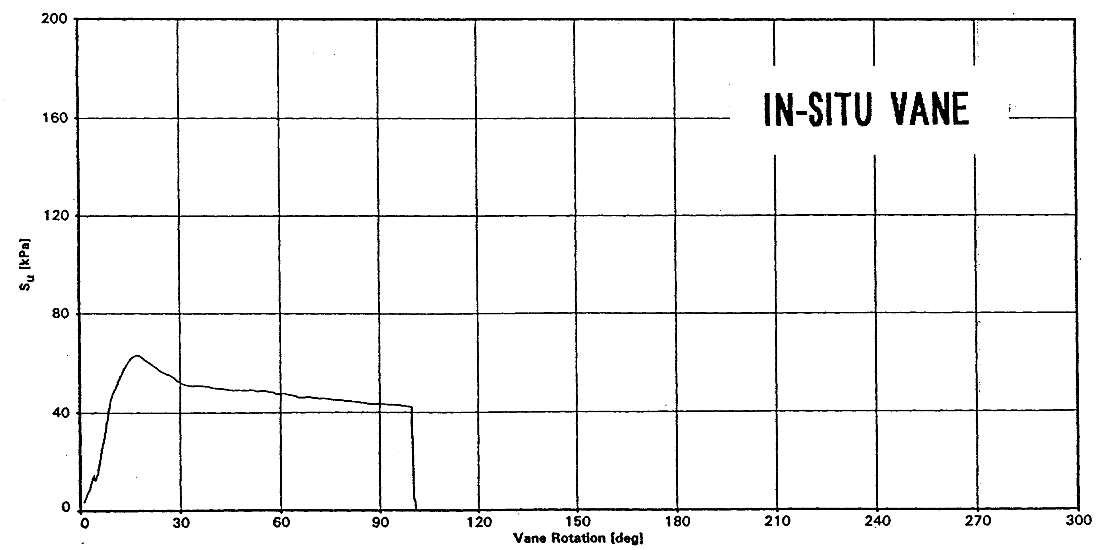
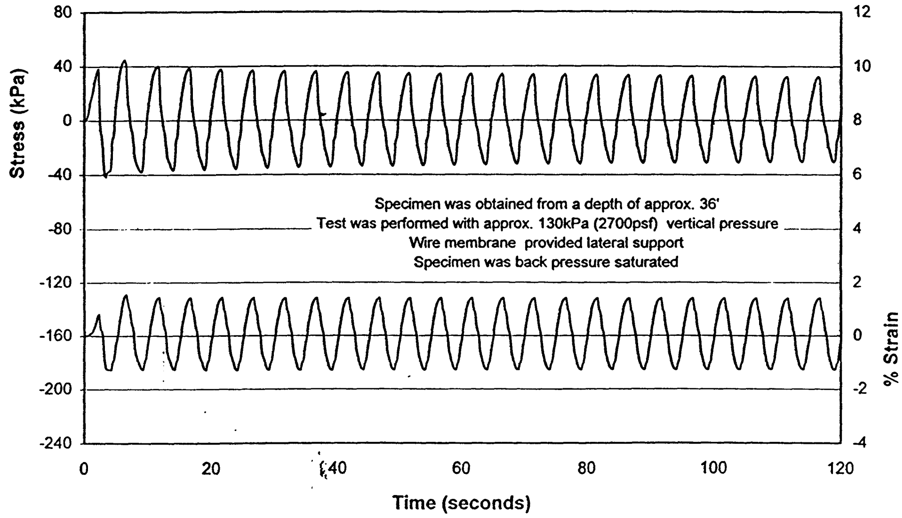


Figure 7

98-60
SFOBB Boring ~~B-10~~ Specimen 13
Cyclic Test to 1.5% Strain (Single Amplitude)



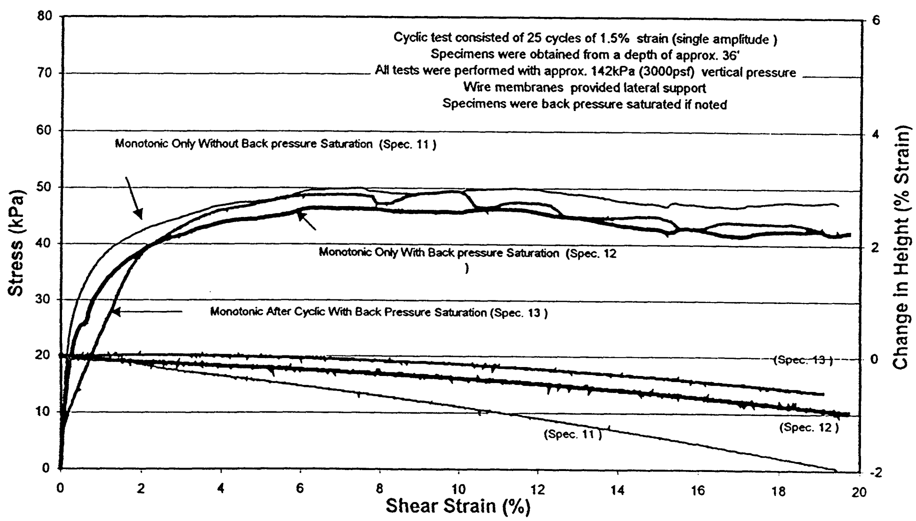
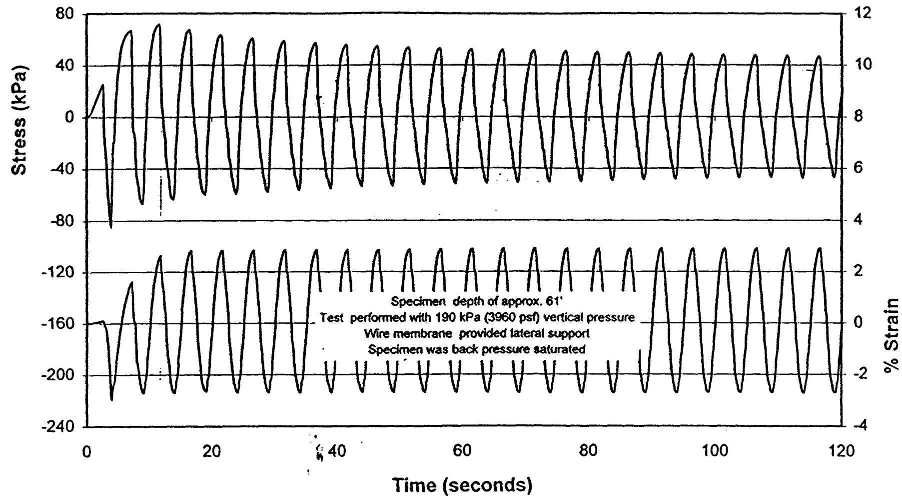
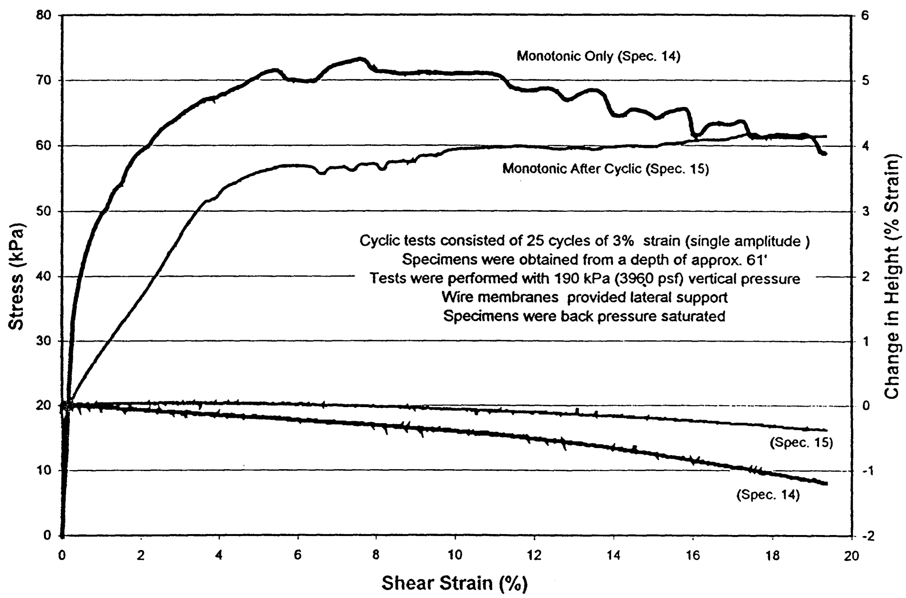


Figure 8

98-56

SFOBB Boring ~~B-6~~ Specimen 15
Cyclic Test to 3% Single Amplitude Strain





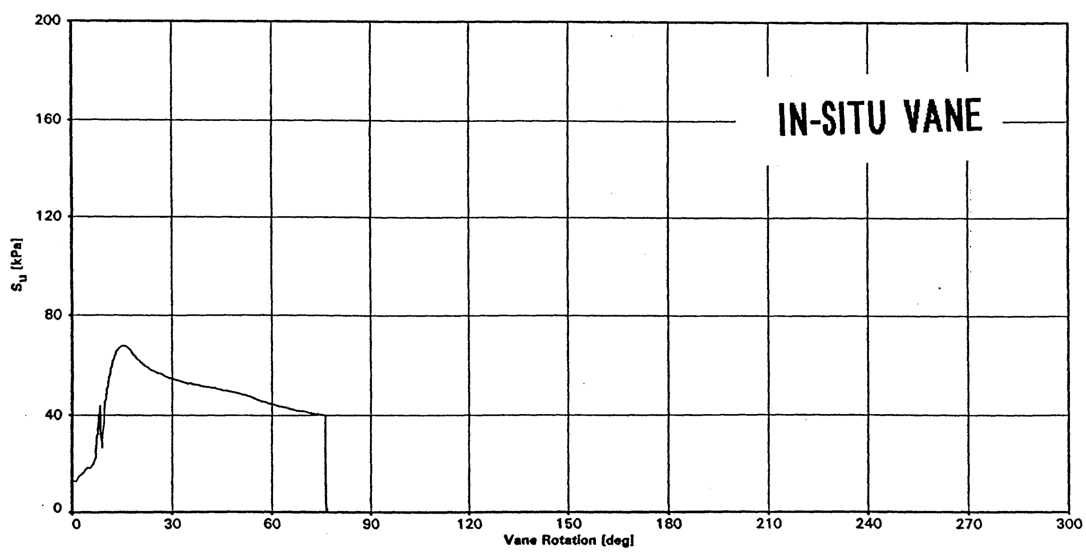
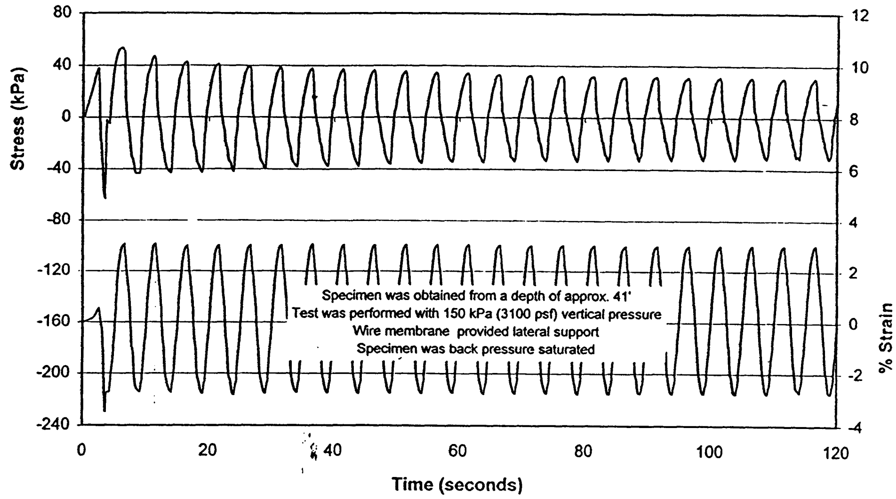
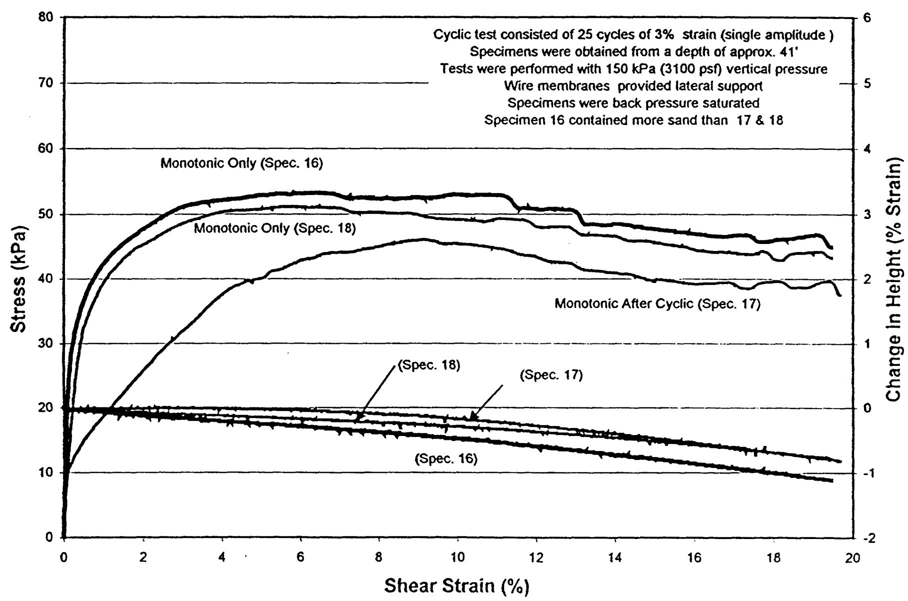


Figure 9

98-62
SFOBB Boring ~~P-2~~ Specimen 17
Cyclic Test to 3% Single Amplitude Strain





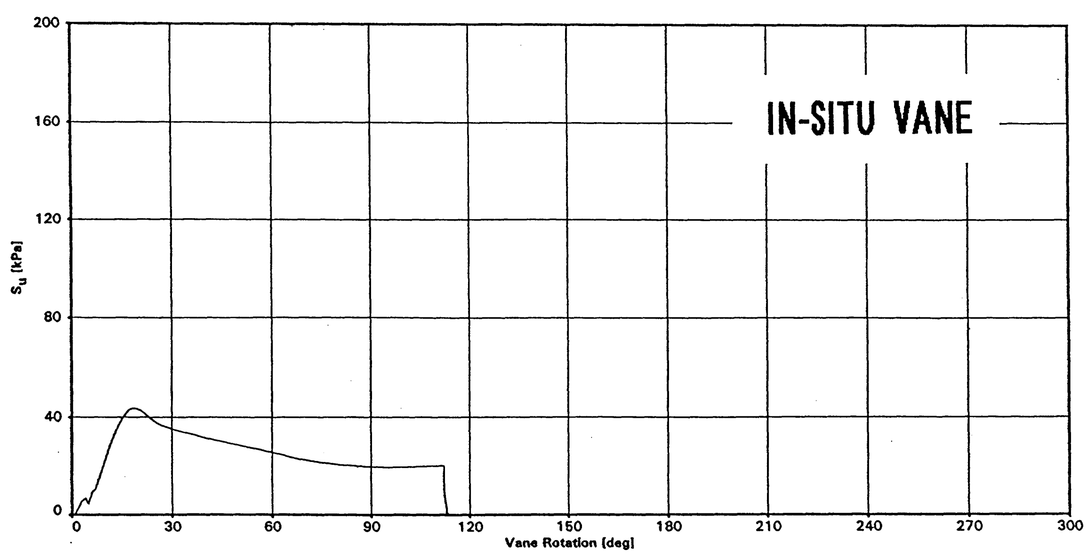
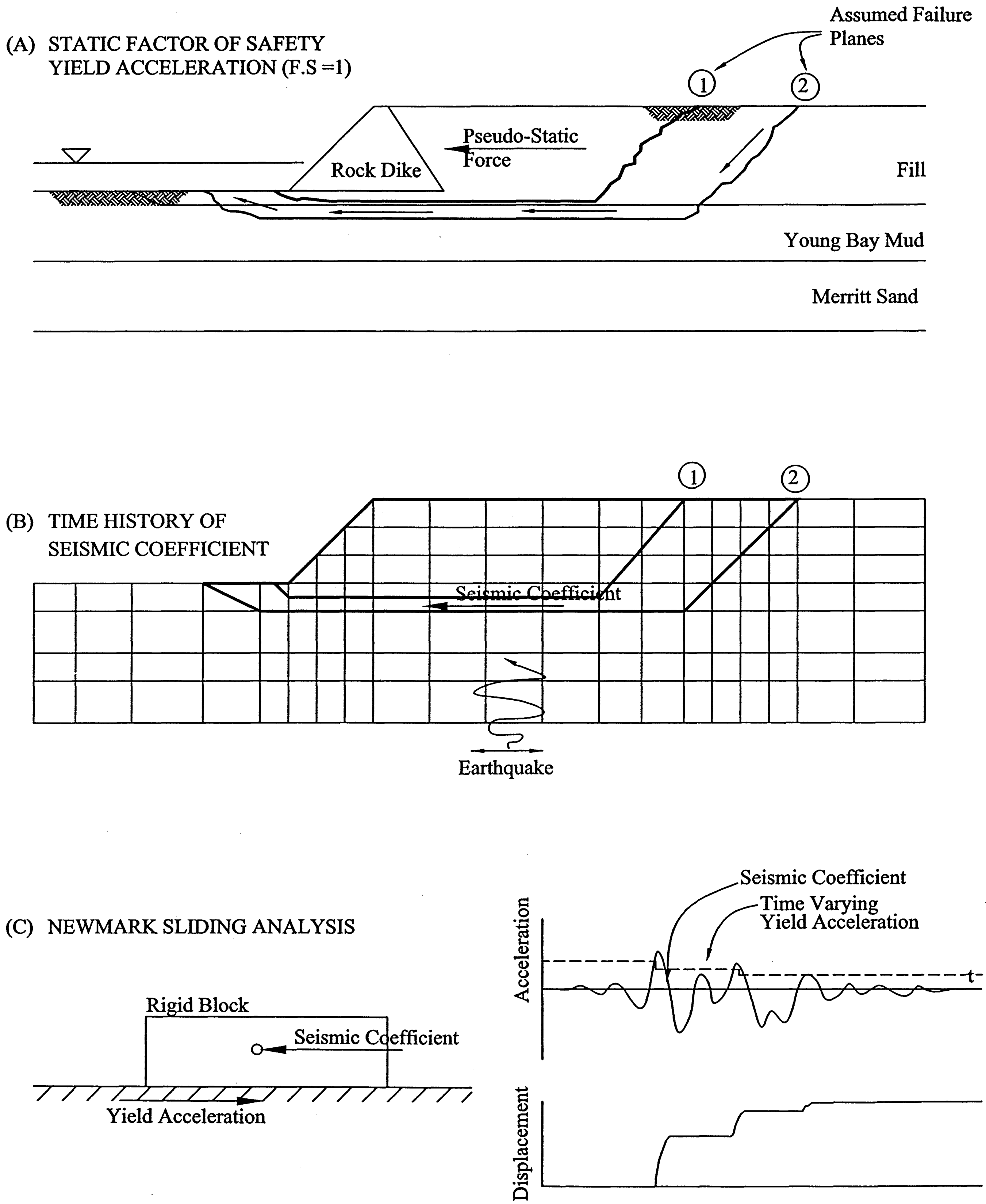


Figure 10



NEW S.F.-OAKLAND BAY BRIDGE

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

PROPOSED PROCEDURE FOR EVALUATION OF OAKLAND MOLE SITE STABILITY

Figure 11

Project No. 98-107

Date: 1-12-99

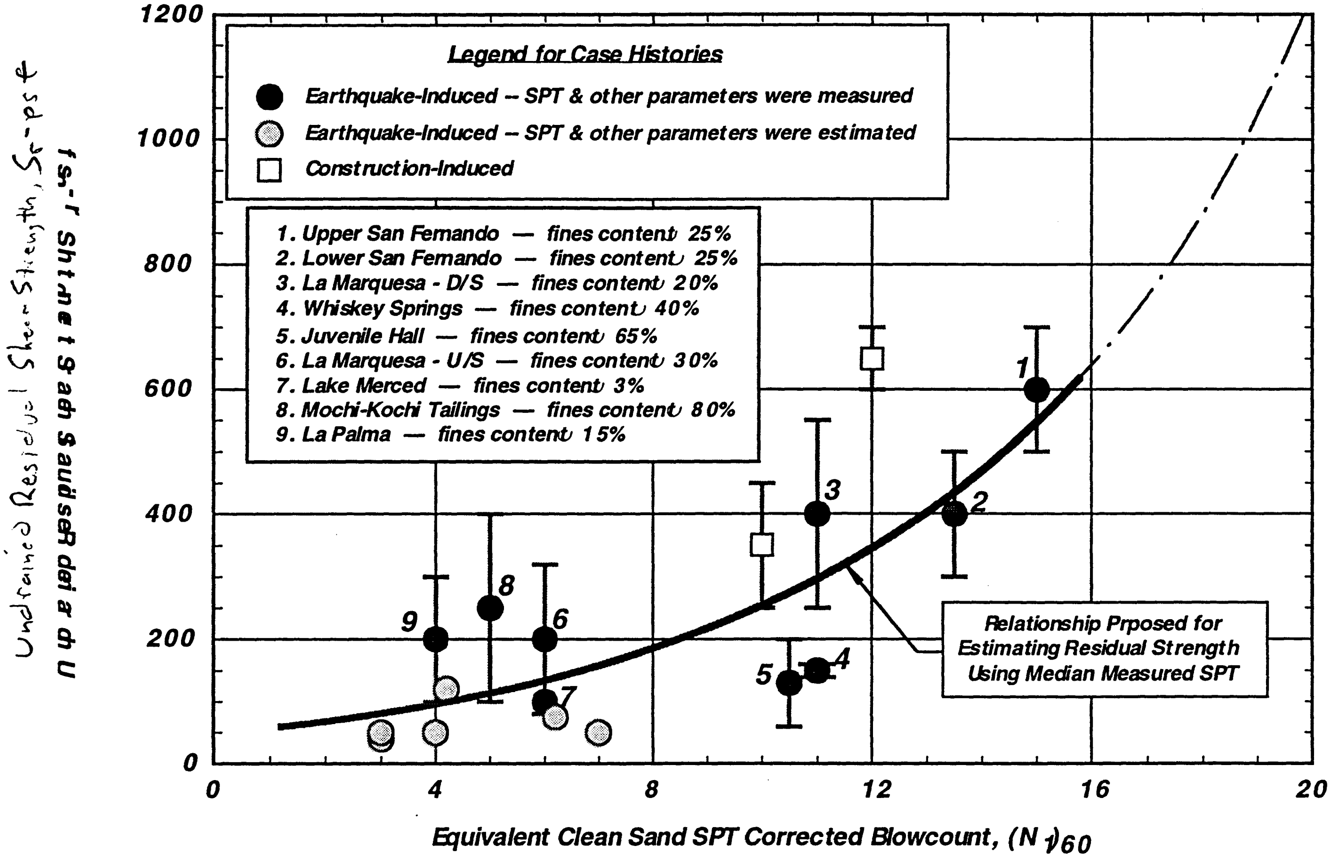
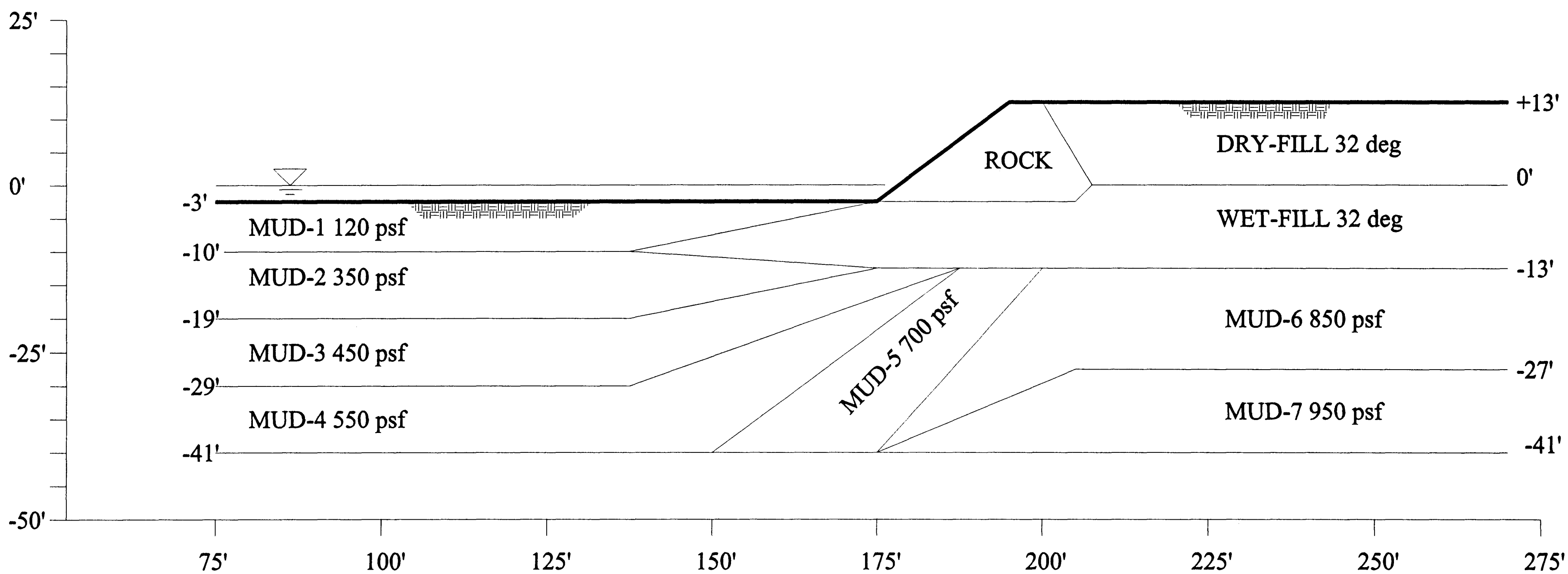


Fig. 14 Undrained Residual Strength, Sr, versus Equivalent Clean Sand SPT Corrected Blowcount Based on Field Case Studies Published by Seed (1987) and by Seed & Harder (1990)

Note: ***The undrained residual shear strength should not exceed the drained strength.***

Figure 12

Appendix A
Station 88+00
Existing Fill (Grade At El. +13')



Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	NEW S.F.- OAKLAND BAY BRIDGE Project No. 98-107 Date: 6-10-98	IDEALIZED SOIL PROFILE AT OAKLAND SHORE APPROACH STA. 88+00 Figure

TABLE 1 Summary of Pseudo Static Analyses for YBM 4/25/99

Block	Block Size, Measured from Shore	1.0*Su (1)		1.4*Su (2)		0.33*Su (3)		0.80*Su (4)	
		Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g
Block 1	25'	2.06	0.22	2.8	0.33	1.3	0.05	1.97	0.17
Block 2	50'	2.75	0.22	3.52	0.32	1.43	0.053	2.38	0.17
Block 3	100'	3.83	0.22	4.87	0.31	1.78	0.057	3.21	0.17

Note:
(1) Using static shear strength
(2) Using dynamic shear strength, i.e., strain rate effects (increase shear strength by 40%)
(3) Using residual shear strength (reduce shear strength to 33%)
(4) Using residual shear strength (reduce shear strength to 80%)

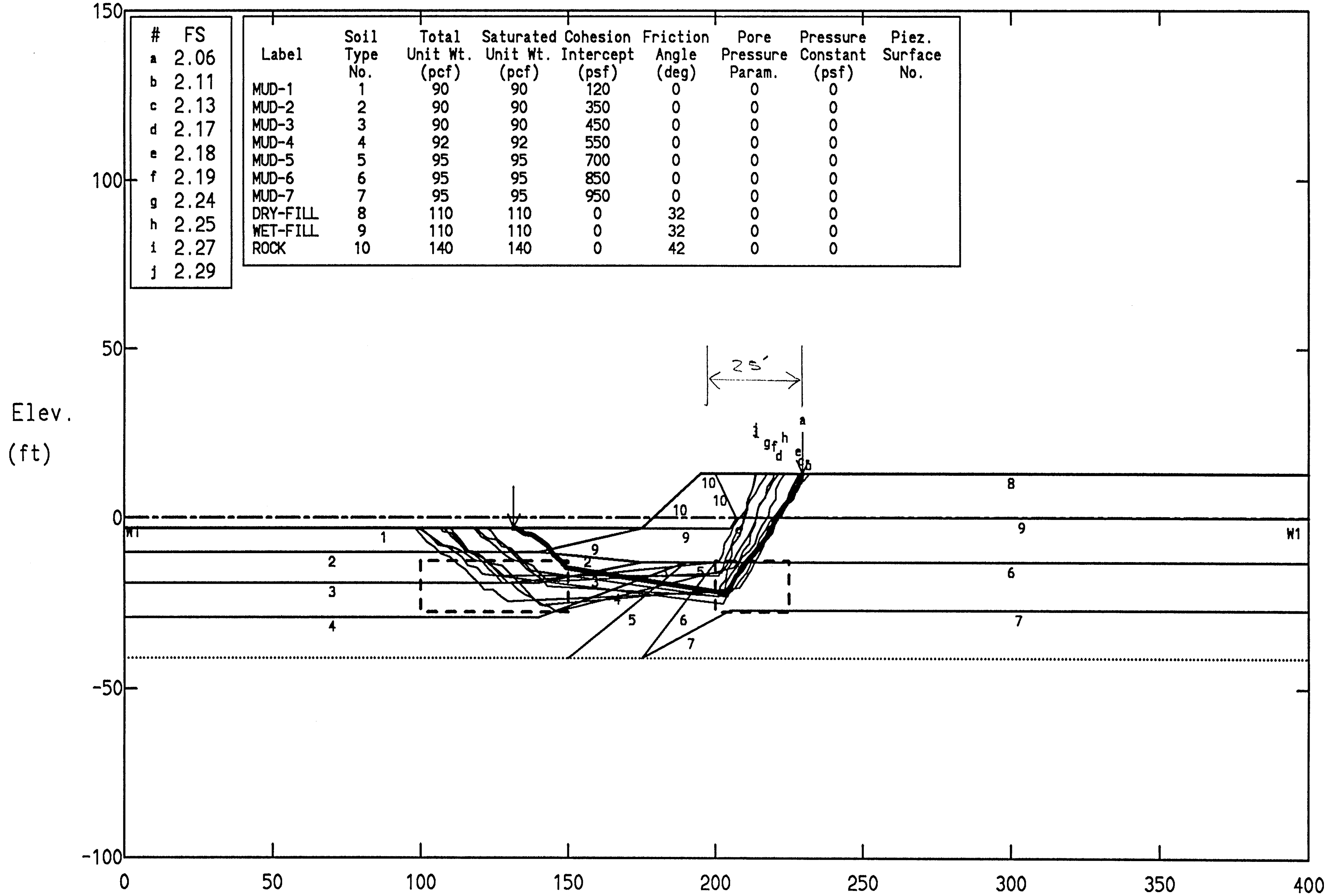
TABLE 2 Summary of Newmark Sliding Analyses for YBM

Ground Motion	Permanent Displacement, inches														
	Static Strength (1)			Static Followed By Residual (2)			Dynamic Strength (3)			Dynamic Followed By Residual (4)			Dynamic Followed By Residual (5)		
	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3
SET-1 (+)	4	3	2	4	3	2	0	0	0	0	0	0	0	0	0
SET-1 (-)	9	7	6	62	50	33	1	1	1	1	1	1	1	1	1
SET-2 (+)	8	7	6	95	75	56	2	2	2	2	2	2	2	2	2
SET-2 (-)	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0
SET-3 (+)	7	6	5	130	107	5	1	1	1	1	1	1	1	1	1
SET-3 (-)	4	3	2	4	3	2	0	0	0	0	0	0	0	0	0
SET-4 (+)	20	18	16	87	74	60	9	8	7	64	54	42	12	11	9
SET-4 (-)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SET-5 (+)	1	1	0	1	1	0	0	0	0	0	0	0	0	0	0
SET-5 (-)	3	2	2	3	2	2	0	0	0	0	0	0	0	0	0
SET-6 (+)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SET-6 (-)	14	13	11	63	50	36	4	3	3	4	3	3	4	3	3
minimum	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
maximum	20	18	16	130	107	60	9	8	7	64	54	42	12	11	9
average	6	5	4	37	30	16	1	1	1	6	5	4	2	2	1

Note:
(1) Using constant static strength
(2) Using static strength followed by residual strength (0.33*Su) when deformation exceeds 6 inches
(3) Using dynamic strength
(4) Using dynamic strength followed by residual strength (0.33*Su) when deformation exceeds 6 inches
(5) Using dynamic strength followed by residual strength (0.80*Su) when deformation exceeds 6 inches

OAKLAND MOLE, SECTION E-E', 2-9-99 LATERAL SPREADING STUDIES

Ten Most Critical. D:CLAY1.PLT 02-09-99 4:26pm



#	FS
a	2.06
b	2.11
c	2.13
d	2.17
e	2.18
f	2.19
g	2.24
h	2.25
i	2.27
j	2.29

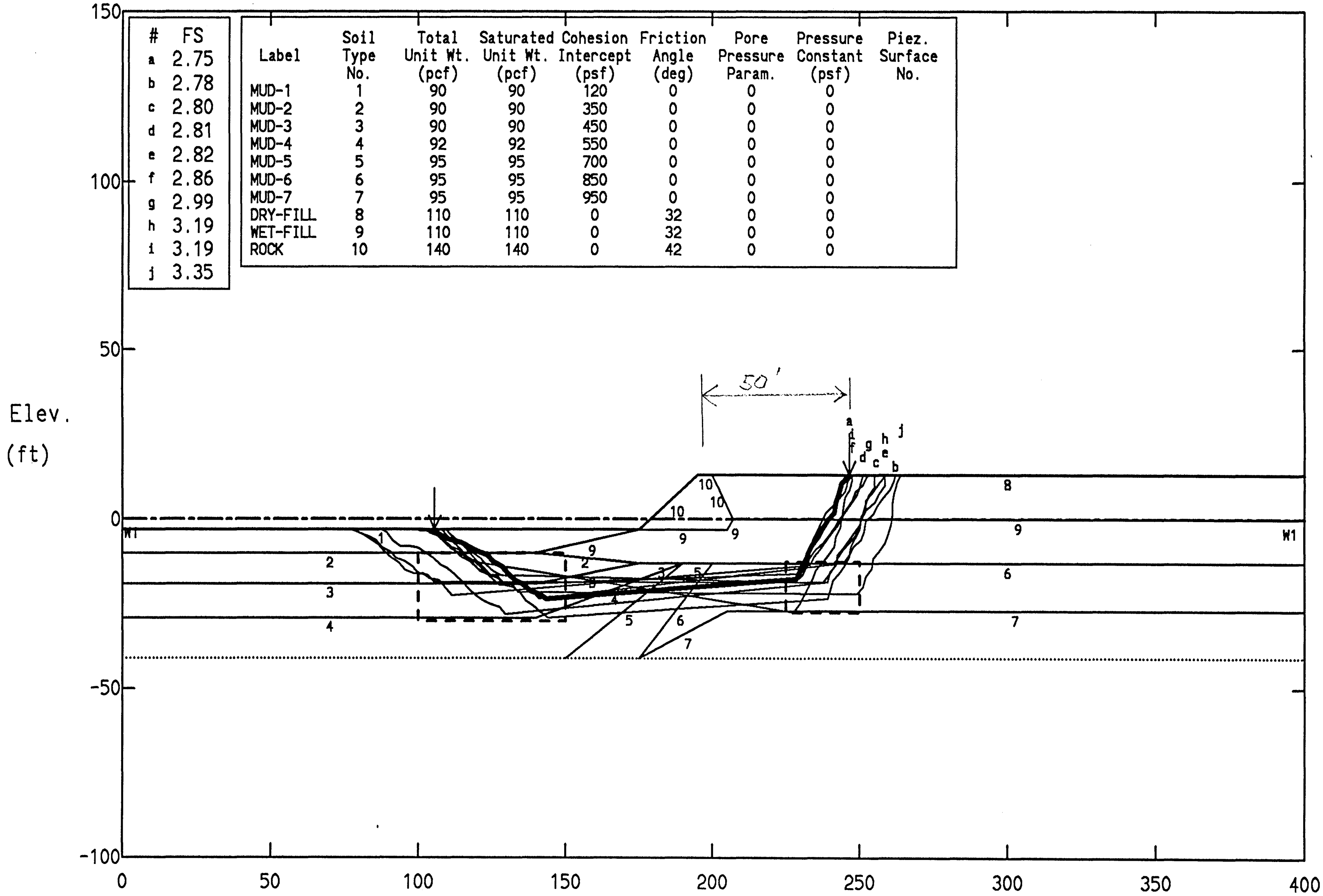
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
MUD-1	1	90	90	120	0	0	0	
MUD-2	2	90	90	350	0	0	0	
MUD-3	3	90	90	450	0	0	0	
MUD-4	4	92	92	550	0	0	0	
MUD-5	5	95	95	700	0	0	0	
MUD-6	6	95	95	850	0	0	0	
MUD-7	7	95	95	950	0	0	0	
DRY-FILL	8	110	110	0	32	0	0	
WET-FILL	9	110	110	0	32	0	0	
ROCK	10	140	140	0	42	0	0	

PCSTABL5M FSmin=2.06 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

OAKLAND MOLE, SECTION E-E', 2-9-99 LATERAL SPREADING STUDIES

Ten Most Critical. D:CLAY2.PLT 02-09-99 4:27pm



#	FS
a	2.75
b	2.78
c	2.80
d	2.81
e	2.82
f	2.86
g	2.99
h	3.19
i	3.19
j	3.35

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
MUD-1	1	90	90	120	0	0	0	
MUD-2	2	90	90	350	0	0	0	
MUD-3	3	90	90	450	0	0	0	
MUD-4	4	92	92	550	0	0	0	
MUD-5	5	95	95	700	0	0	0	
MUD-6	6	95	95	850	0	0	0	
MUD-7	7	95	95	950	0	0	0	
DRY-FILL	8	110	110	0	32	0	0	
WET-FILL	9	110	110	0	32	0	0	
ROCK	10	140	140	0	42	0	0	

PCSTABL5M FSmin=2.75 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

OAKLAND MOLE, SECTION E-E', 2-9-99 LATERAL SPREADING STUDIES

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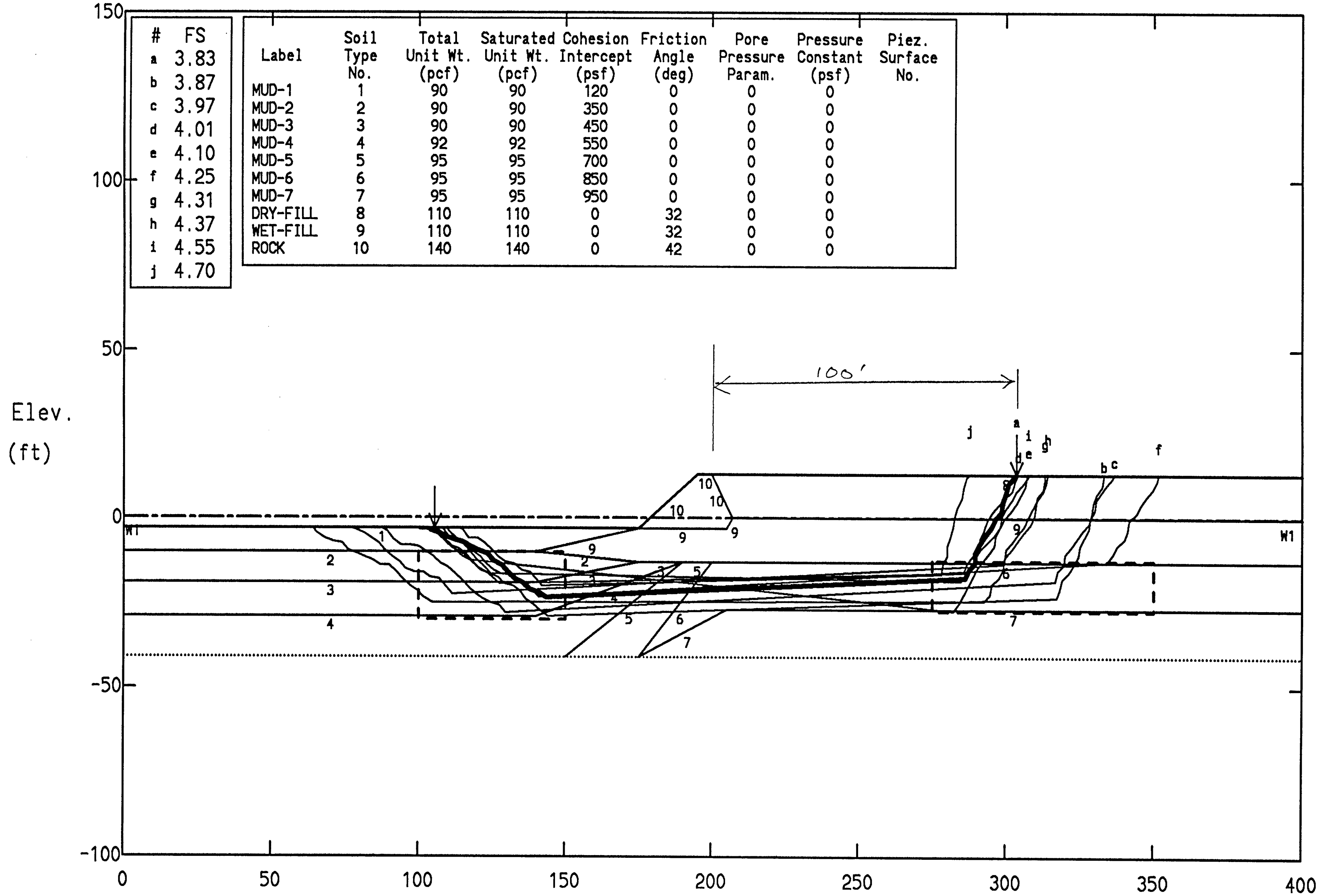
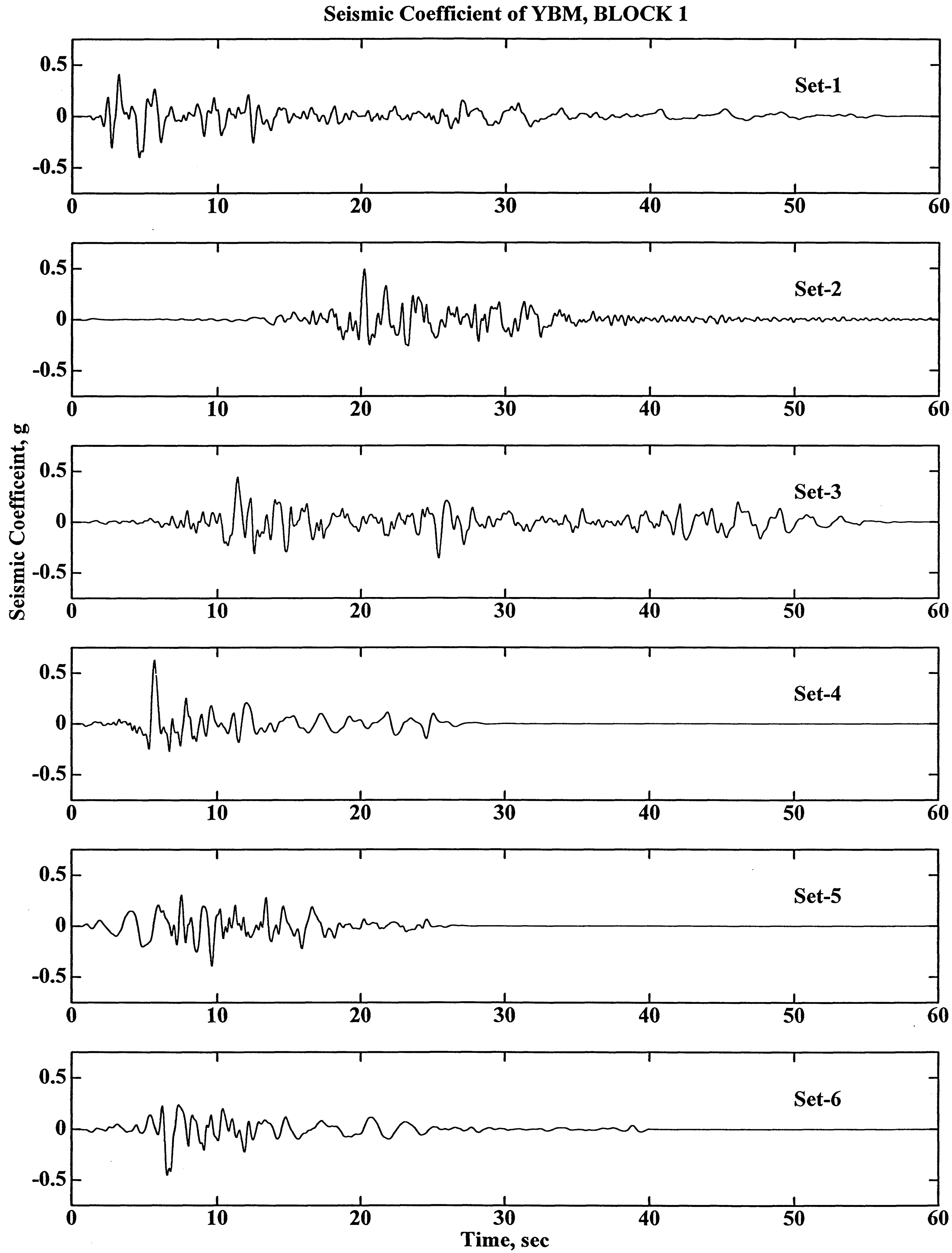


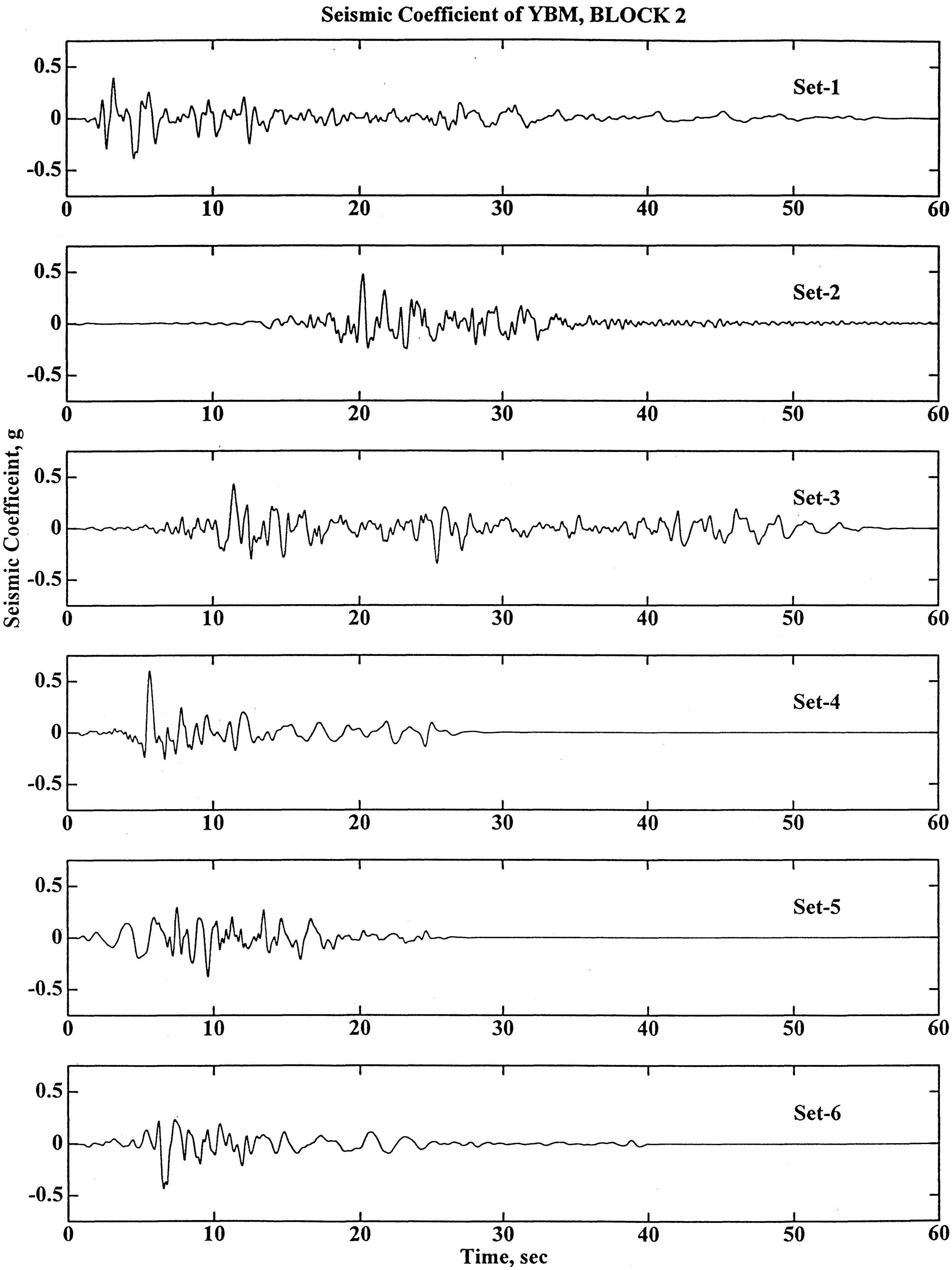
#	FS
a	3.83
b	3.87
c	3.97
d	4.01
e	4.10
f	4.25
g	4.31
h	4.37
i	4.55
j	4.70

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
MUD-1	1	90	90	120	0	0	0	
MUD-2	2	90	90	350	0	0	0	
MUD-3	3	90	90	450	0	0	0	
MUD-4	4	92	92	550	0	0	0	
MUD-5	5	95	95	700	0	0	0	
MUD-6	6	95	95	850	0	0	0	
MUD-7	7	95	95	950	0	0	0	
DRY-FILL	8	110	110	0	32	0	0	
WET-FILL	9	110	110	0	32	0	0	
ROCK	10	140	140	0	42	0	0	

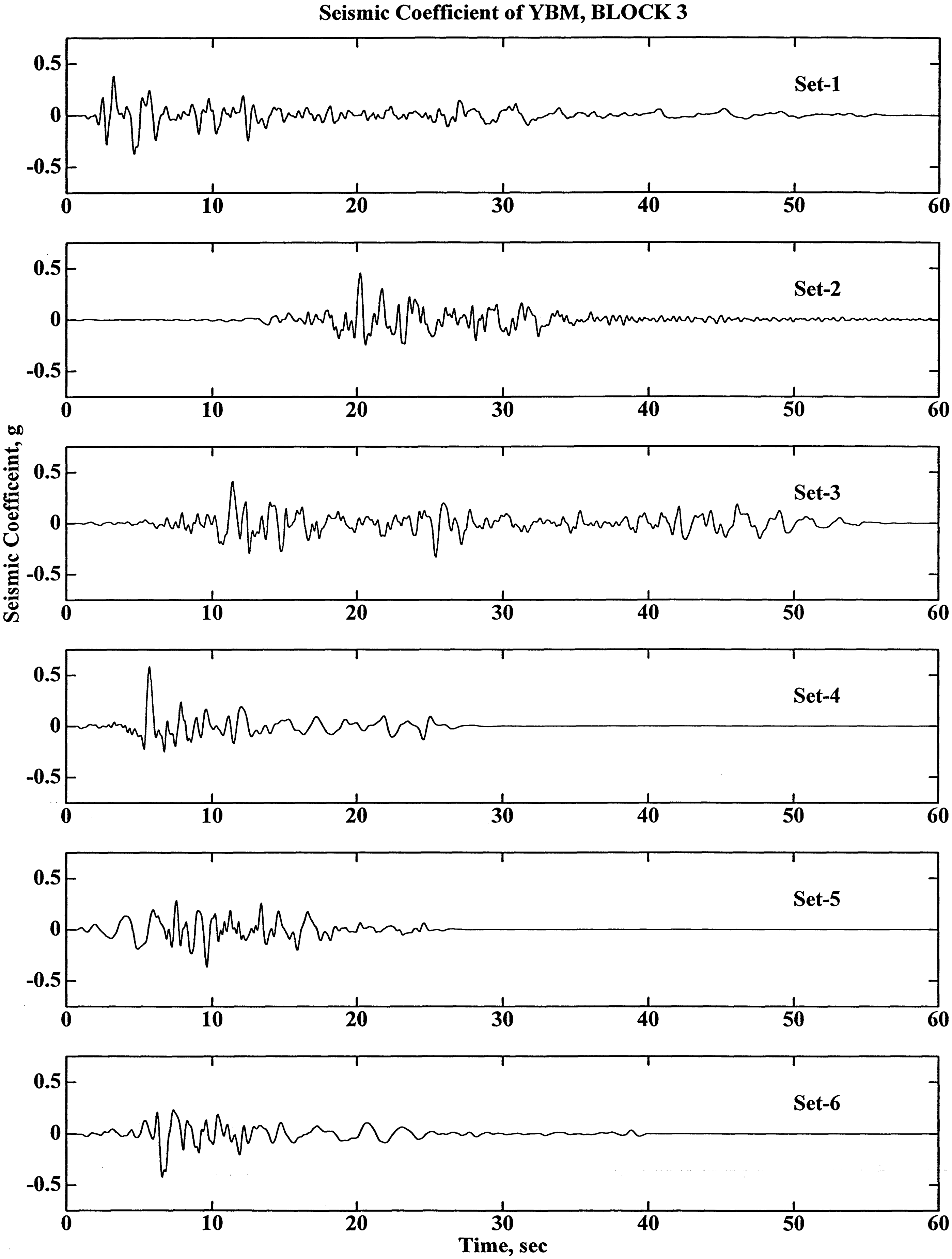
PCSTABL5M FSmin=3.83 X-Axis (ft)

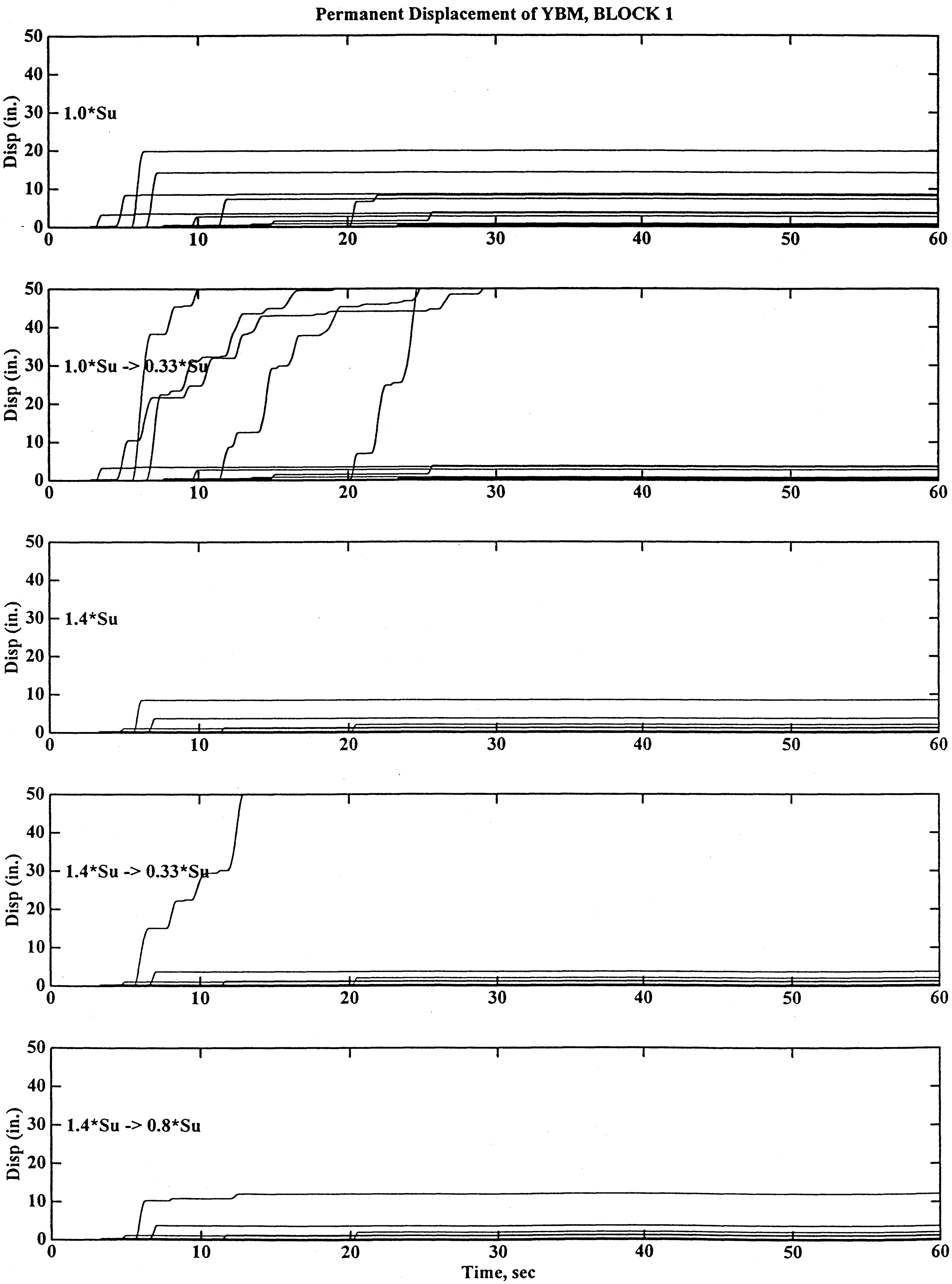
Factors Of Safety Calculated By The Modified Janbu Method


Seismic Coefficient of YBM, BLOCK 1
Set-1
Set-2
Set-3
Set-4
Set-5
Set-6
0.5
0
-0.5
0
10
20
30
40
50
60
Seismic Coefficeint, g
Time, sec

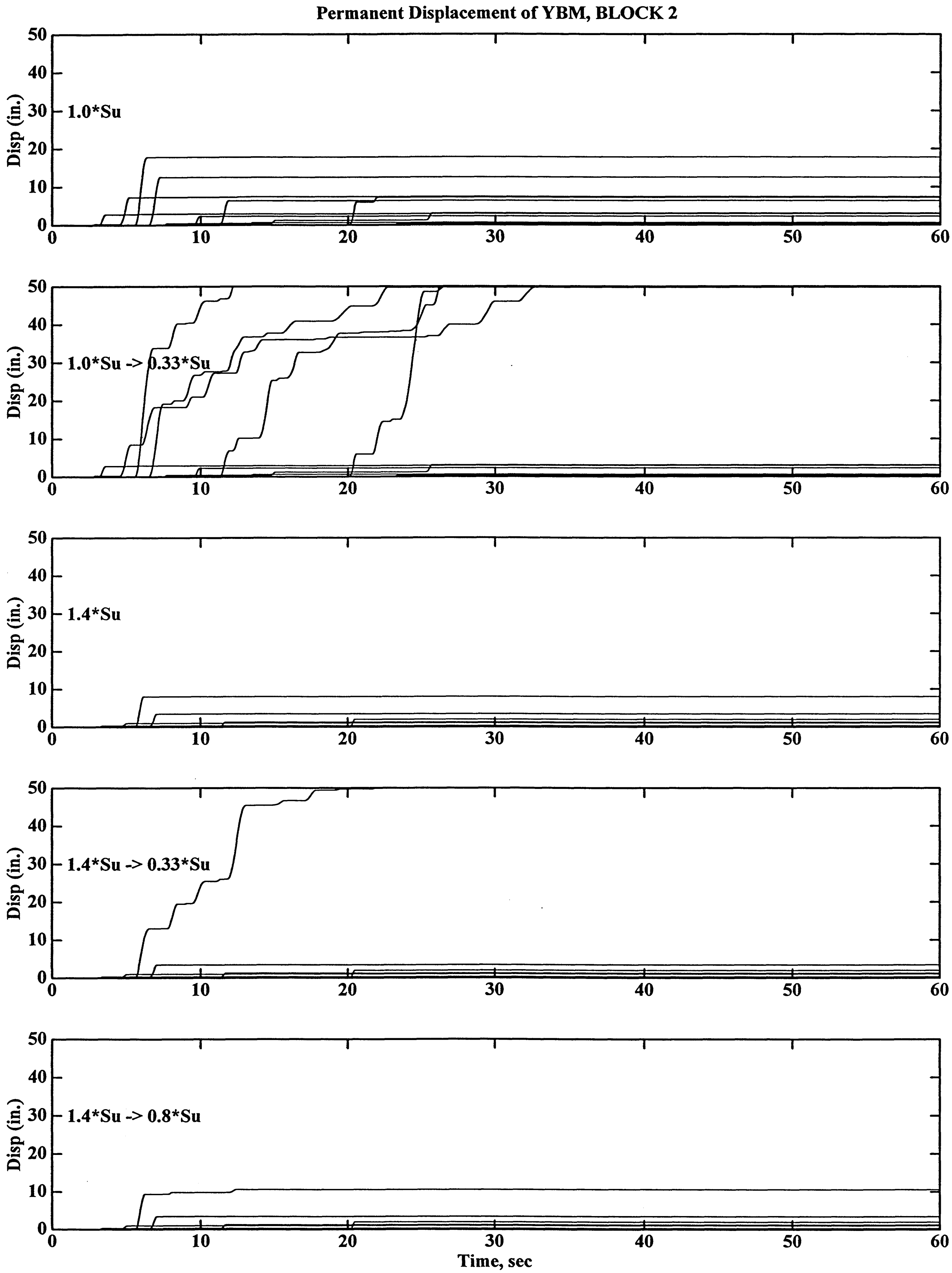


Seismic Coefficient of YBM, BLOCK 2
Set-1
Set-2
Set-3
Set-4
Set-5
Set-6
Seismic Coefficeint, g
Time, sec
0.5
0
-0.5
0
10
20
30
40
50
60


Seismic Coefficient of YBM, BLOCK 3
Set-1
Set-2
Set-3
Set-4
Set-5
Set-6
Seismic Coefficeint, g
Time, sec
0.5
0
-0.5
0
10
20
30
40
50
60



Permanent Displacement of YBM, BLOCK 1
1.0*Su
1.0*Su -> 0.33*Su
1.4*Su
1.4*Su -> 0.33*Su
1.4*Su -> 0.8*Su
Disp (in.)
Time, sec
0
10
20
30
40
50
60



Permanent Displacement of YBM, BLOCK 2
Disp (in.)
1.0*Su
1.0*Su -> 0.33*Su
1.4*Su
1.4*Su -> 0.33*Su
1.4*Su -> 0.8*Su
Time, sec

Permanent Displacement of YBM, BLOCK 3

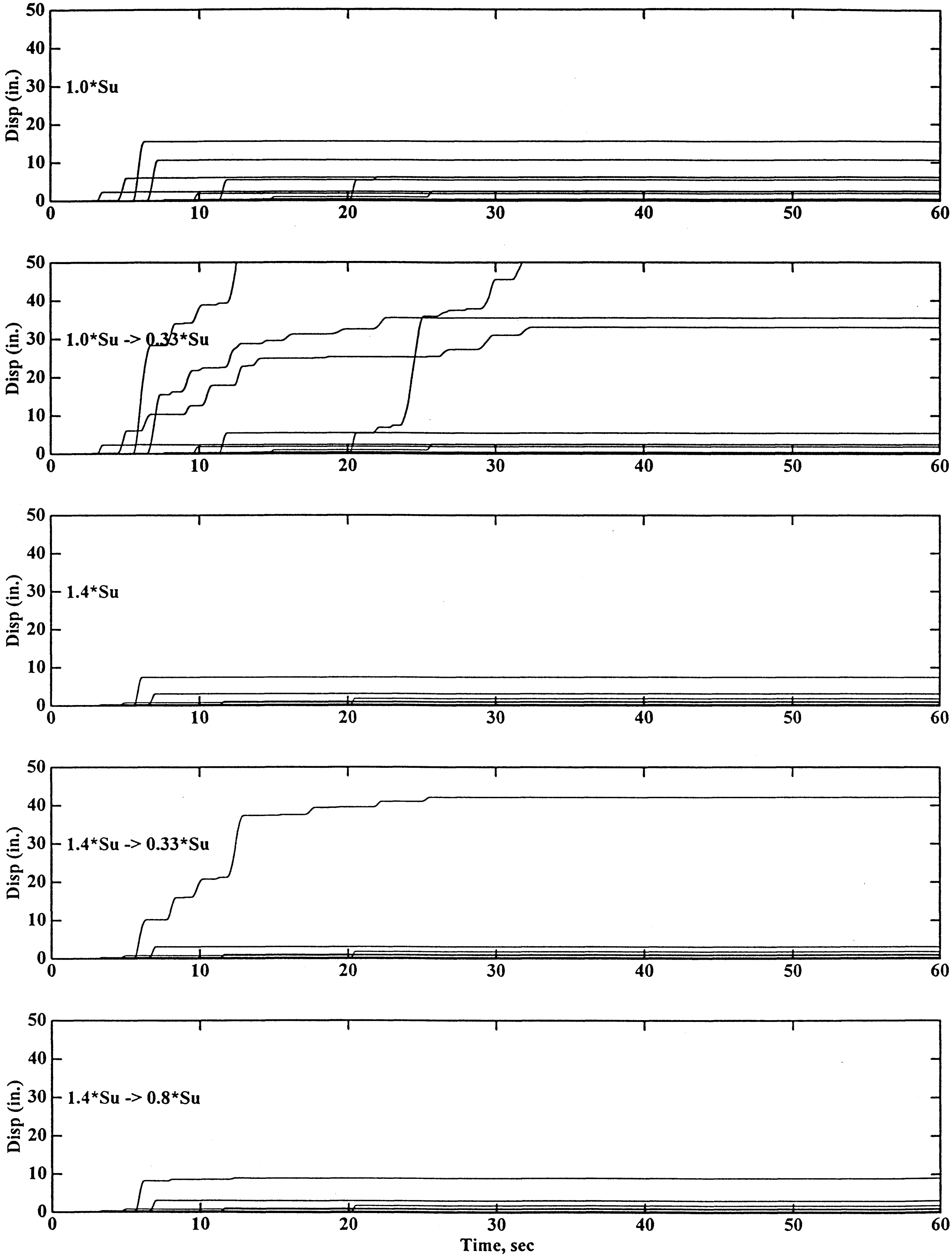


TABLE 3 Summary of Pseudo Static Analyses for FILL

Block	Block Size, Measured from Shore	Drained Strength (1)		Residual Strength (2)	
		Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g
Block 1	25'	3.65	0.49	2.53	0.26
Block 2	50'	4.91	0.47	3.03	0.25
Block 3	100'	7.98	0.43	4.48	0.24

Note:
(1) Using drained shear strength of sand
(2) Using residual shear strength of sand in fill below watertable (Su=600 psf)

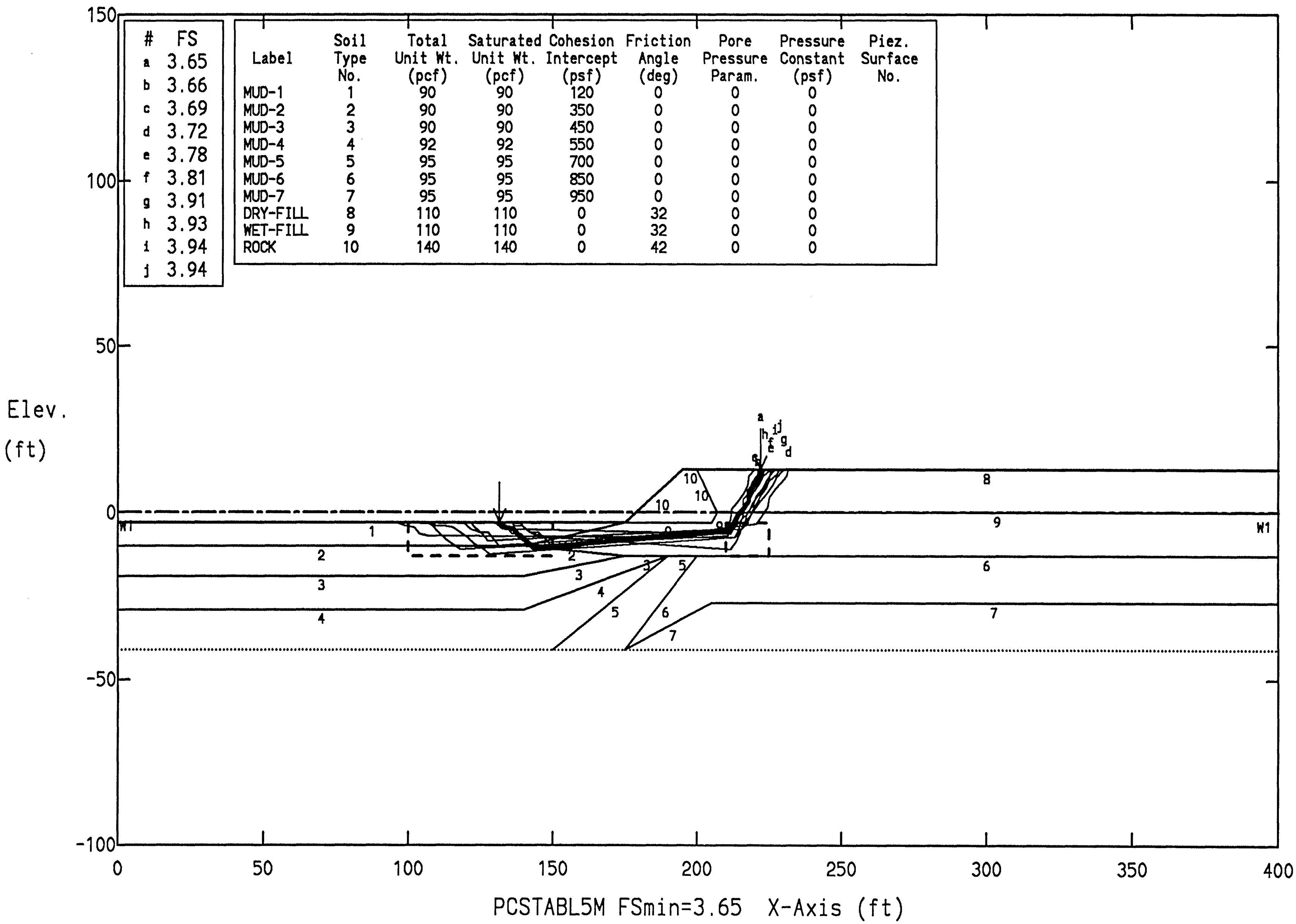
TABLE 4 Summary of Newmark Sliding Analyses for FILL

Ground Motion	Permanent Displacement, inches					
	Drained Strength (1)			Residual Strength (2)		
	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3
SET-1 (+)	0	0	0	5	4	3
SET-1 (-)	0	0	0	12	10	8
SET-2 (+)	1	0	0	10	9	8
SET-2 (-)	0	0	0	1	1	0
SET-3 (+)	0	0	0	9	8	7
SET-3 (-)	0	0	0	5	4	3
SET-4 (+)	5	4	4	26	23	19
SET-4 (-)	0	0	0	0	0	0
SET-5 (+)	0	0	0	1	1	1
SET-5 (-)	0	0	0	4	3	2
SET-6 (+)	0	0	0	0	0	0
SET-6 (-)	0	0	0	19	17	13
minimum	0	0	0	0	0	0
maximum	5	4	4	26	23	19
average	1	0	0	8	7	5

Note:
(1) Using constant drained shear strength of sand (unliquefied strength)
(2) Using constant residual shear strength of sand (liquefied strength)

OAKLAND MOLE, SECTION E-E', 2-9-99 LATERAL SPREADING STUDIES

Ten Most Critical. D:FILL1.PLT 02-11-99 1:49pm



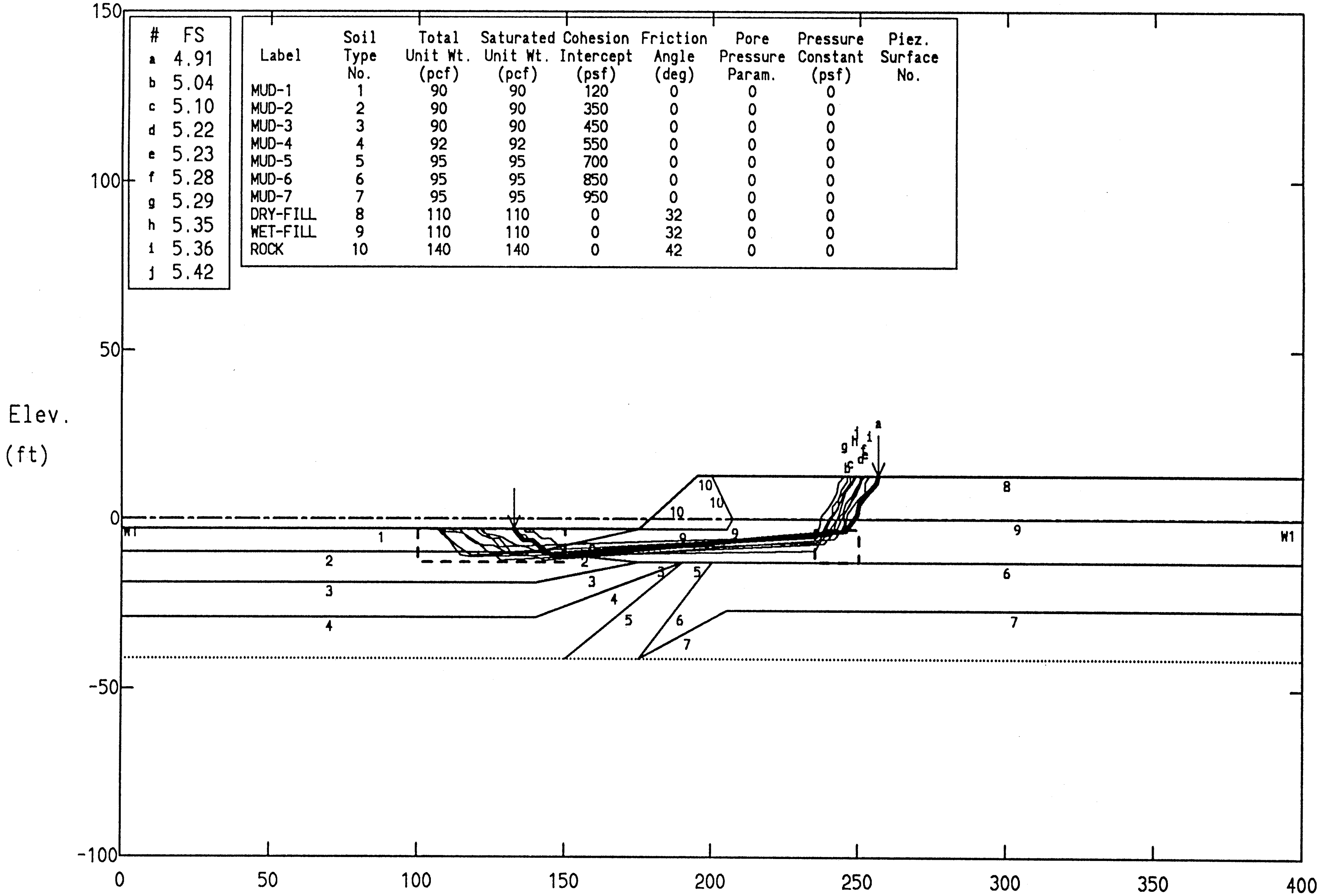
#	FS
a	3.65
b	3.66
c	3.69
d	3.72
e	3.78
f	3.81
g	3.91
h	3.93
i	3.94
j	3.94

Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
MUD-1	1	90	90	120	0	0	0	
MUD-2	2	90	90	350	0	0	0	
MUD-3	3	90	90	450	0	0	0	
MUD-4	4	92	92	550	0	0	0	
MUD-5	5	95	95	700	0	0	0	
MUD-6	6	95	95	850	0	0	0	
MUD-7	7	95	95	950	0	0	0	
DRY-FILL	8	110	110	0	32	0	0	
WET-FILL	9	110	110	0	32	0	0	
ROCK	10	140	140	0	42	0	0	

Factors Of Safety Calculated By The Modified Janbu Method

OAKLAND MOLE, SECTION E-E', 2-9-99 LATERAL SPREADING STUDIES

Ten Most Critical. D:FILL2.PLT 02-11-99 1:51pm



#	FS
a	4.91
b	5.04
c	5.10
d	5.22
e	5.23
f	5.28
g	5.29
h	5.35
i	5.36
j	5.42

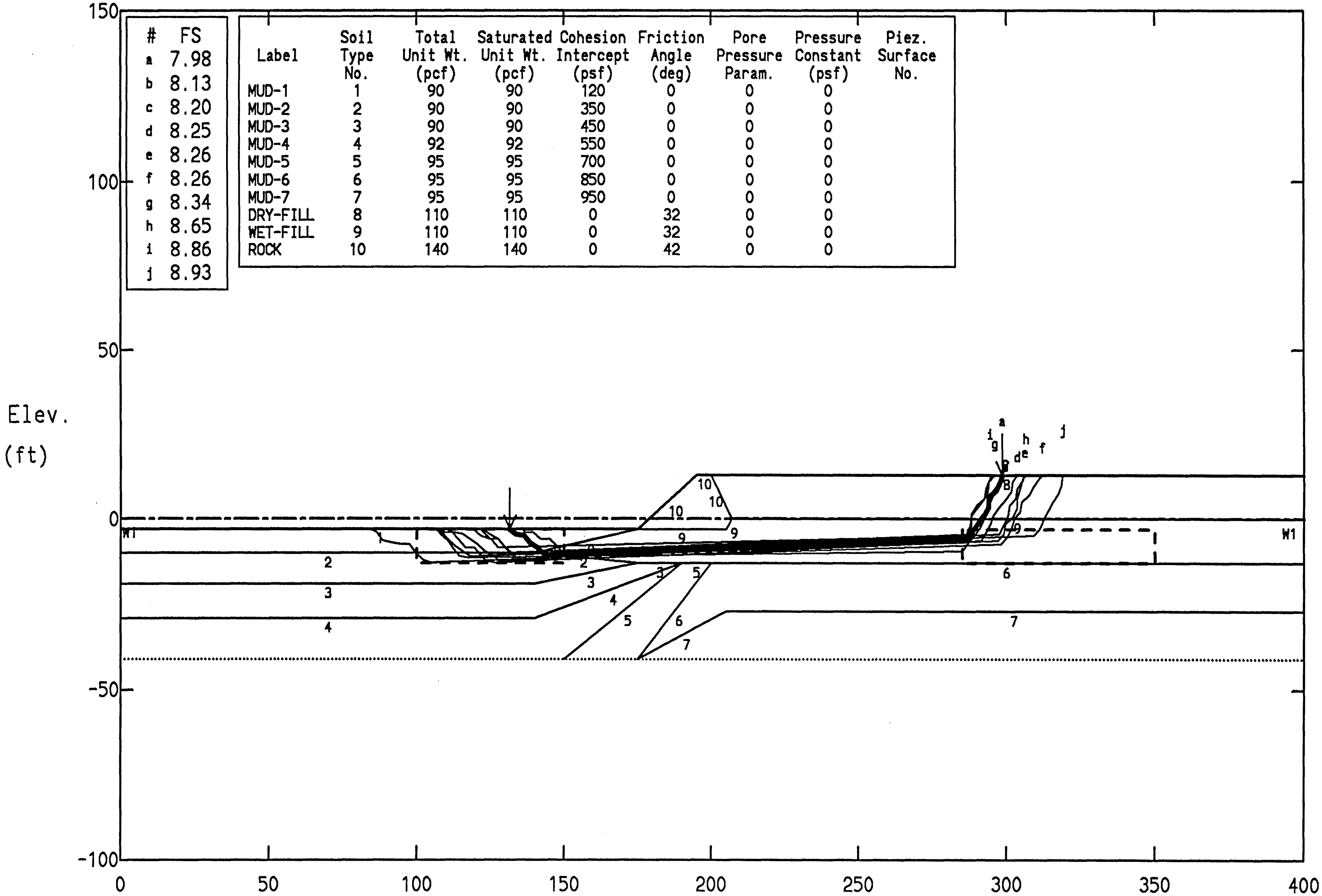
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
MUD-1	1	90	90	120	0	0	0	
MUD-2	2	90	90	350	0	0	0	
MUD-3	3	90	90	450	0	0	0	
MUD-4	4	92	92	550	0	0	0	
MUD-5	5	95	95	700	0	0	0	
MUD-6	6	95	95	850	0	0	0	
MUD-7	7	95	95	950	0	0	0	
DRY-FILL	8	110	110	0	32	0	0	
WET-FILL	9	110	110	0	32	0	0	
ROCK	10	140	140	0	42	0	0	

PCSTABL5M FSmin=4.91 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

OAKLAND MOLE, SECTION E-E', 2-9-99 LATERAL SPREADING STUDIES

Ten Most Critical. D:FILL3.PLT 02-11-99 1:52pm

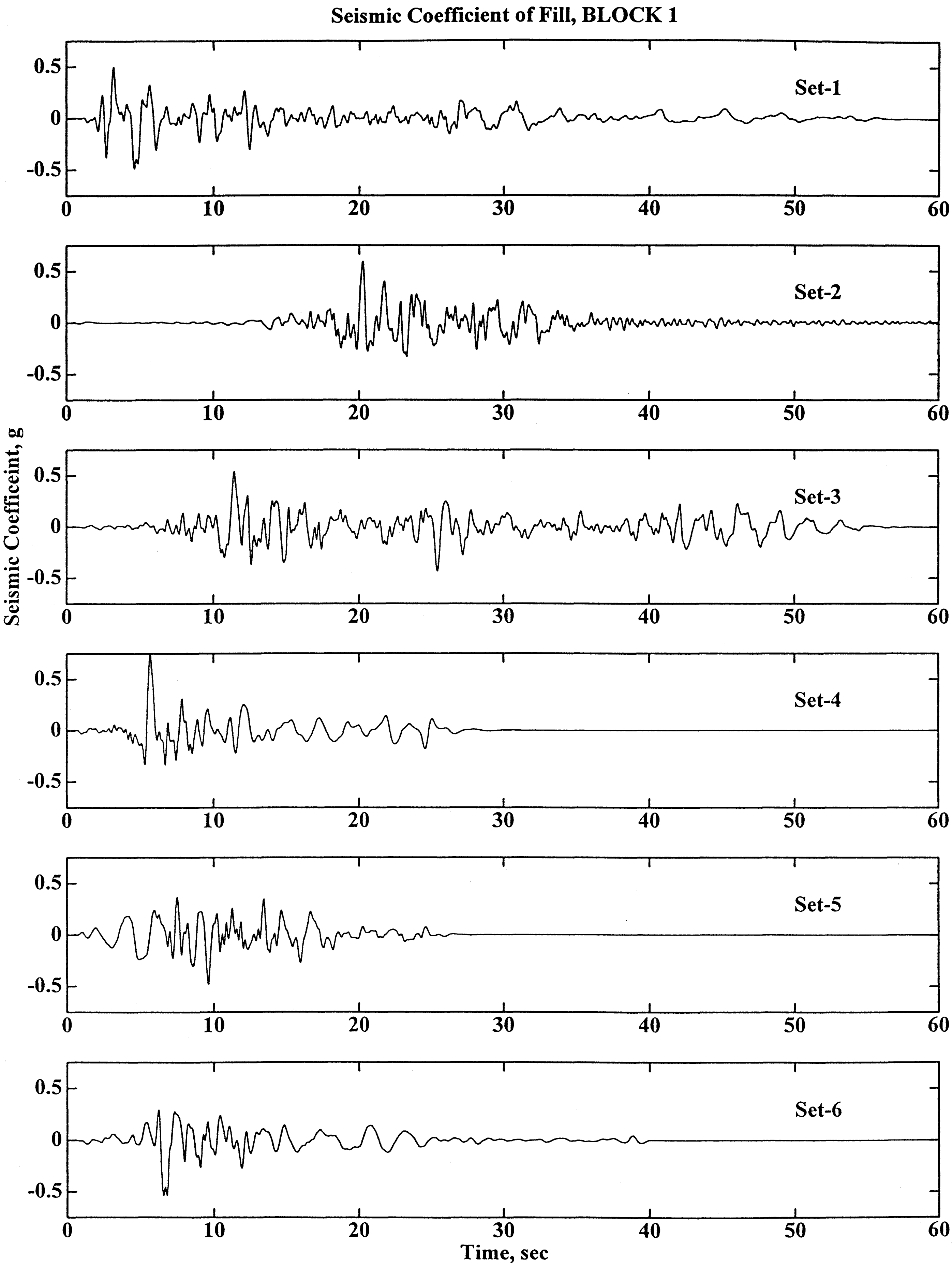


#	FS
a	7.98
b	8.13
c	8.20
d	8.25
e	8.26
f	8.26
g	8.34
h	8.65
i	8.86
j	8.93

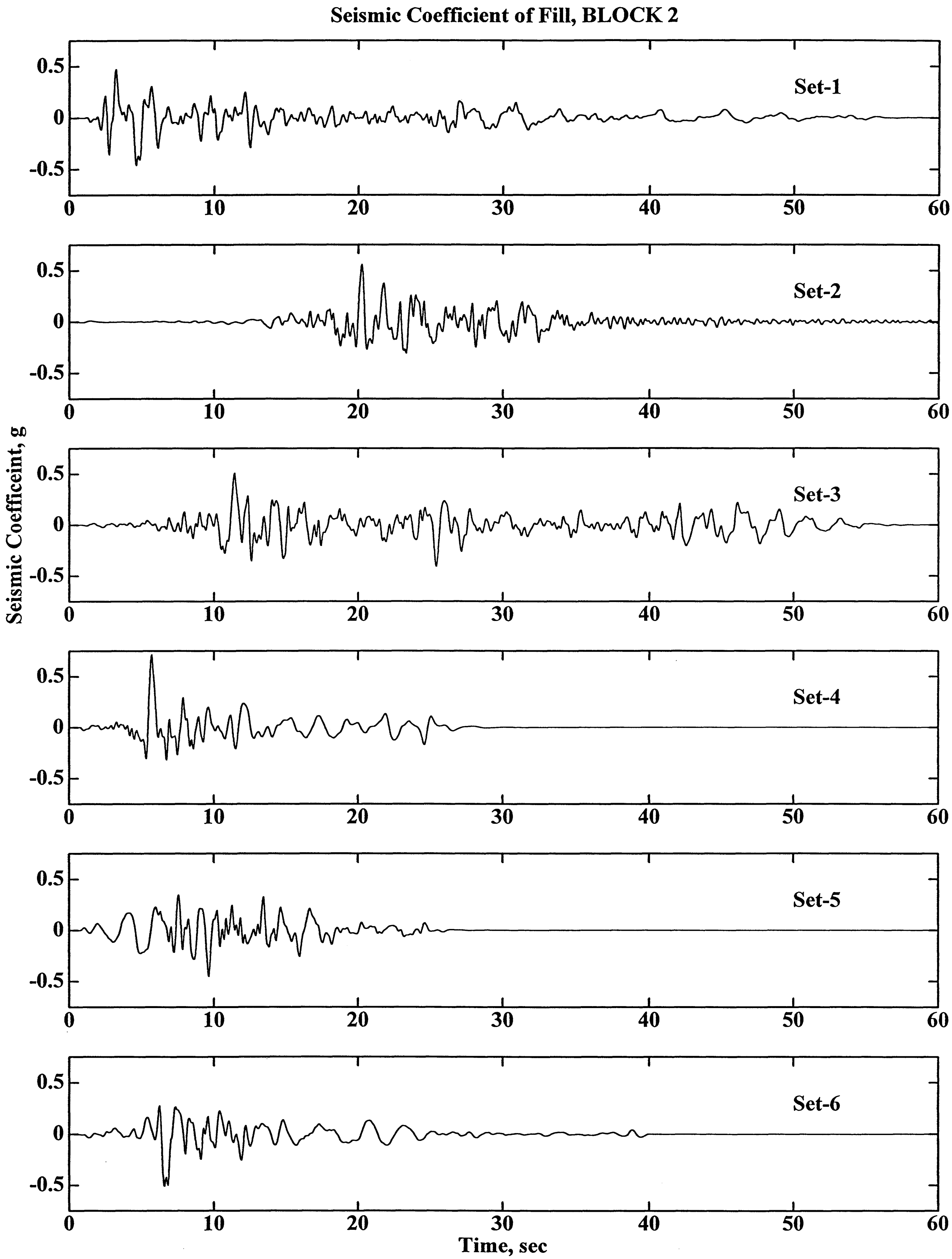
Label	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
MUD-1	1	90	90	120	0	0	0	
MUD-2	2	90	90	350	0	0	0	
MUD-3	3	90	90	450	0	0	0	
MUD-4	4	92	92	550	0	0	0	
MUD-5	5	95	95	700	0	0	0	
MUD-6	6	95	95	850	0	0	0	
MUD-7	7	95	95	950	0	0	0	
DRY-FILL	8	110	110	0	32	0	0	
WET-FILL	9	110	110	0	32	0	0	
ROCK	10	140	140	0	42	0	0	

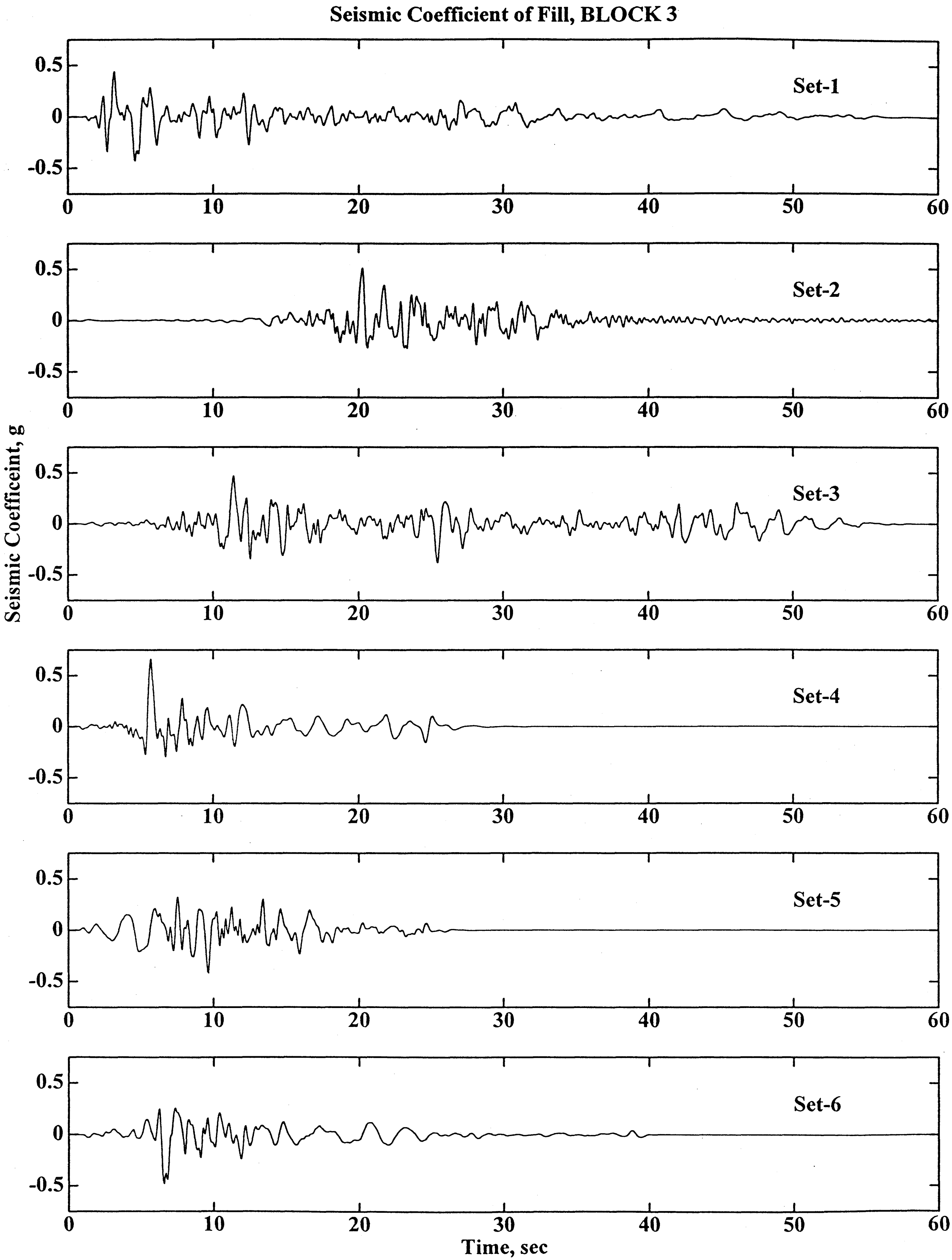
PCSTABL5M FSmin=7.98 X-Axis (ft)

Factors Of Safety Calculated By The Modified Janbu Method

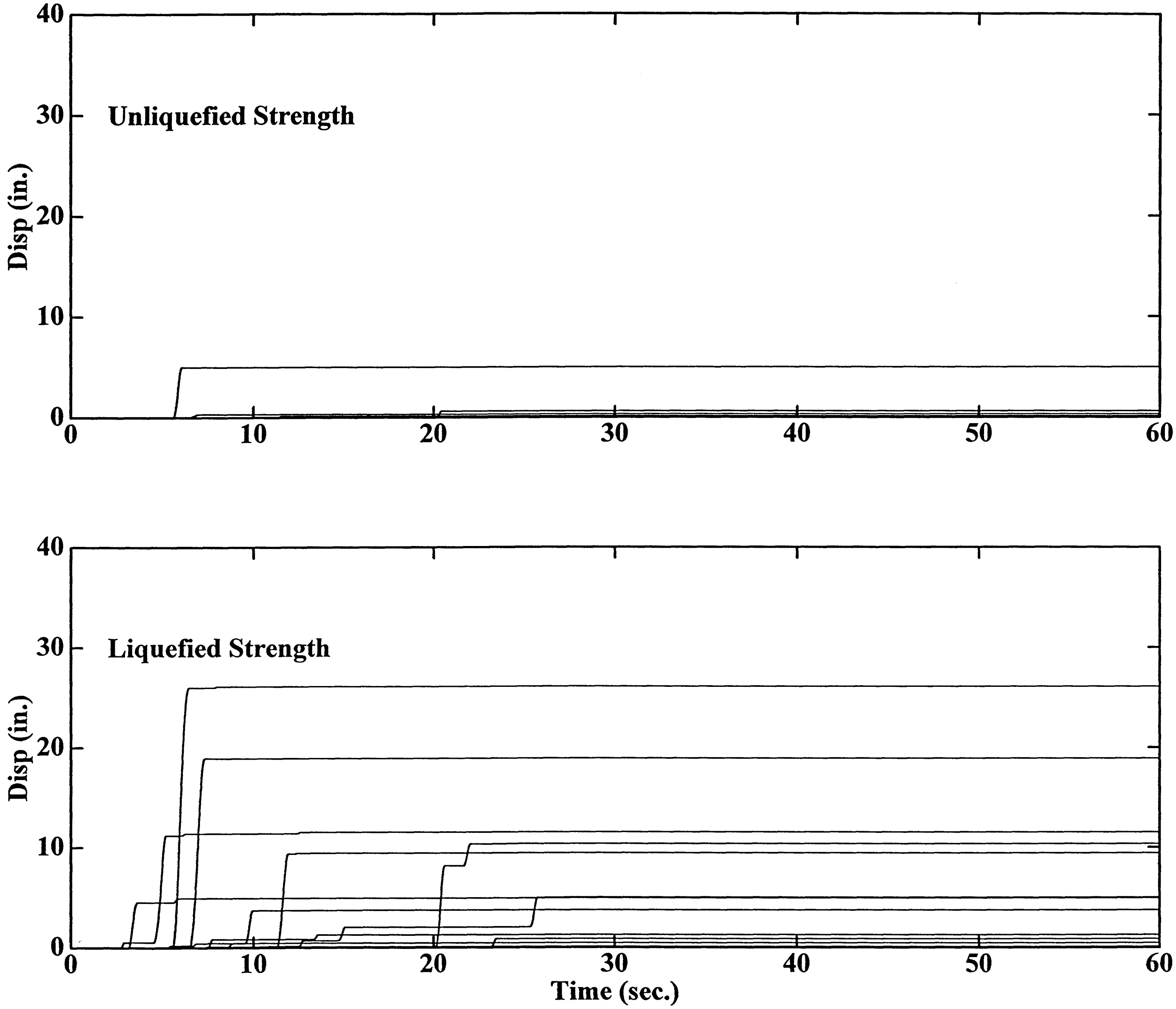


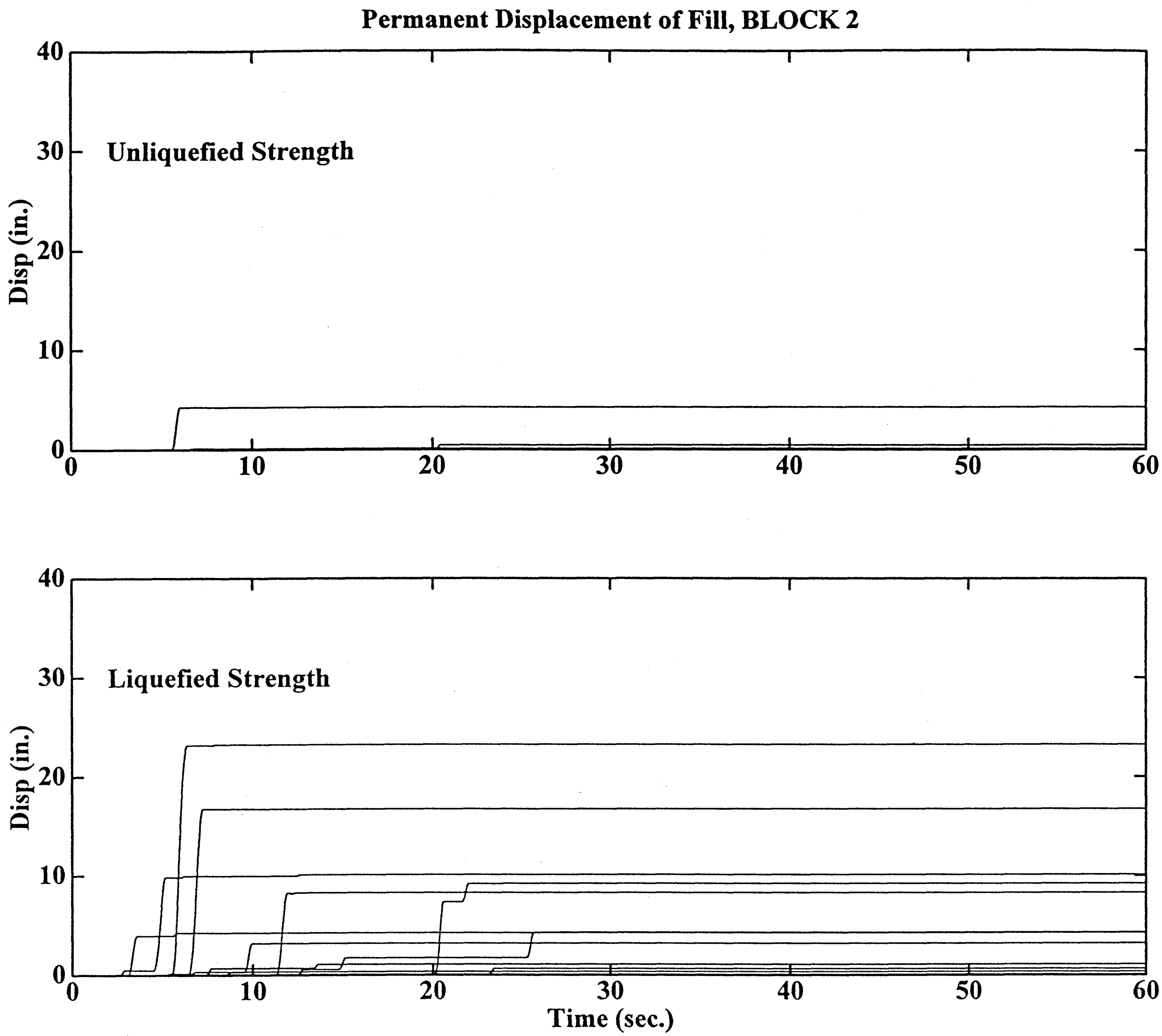
Seismic Coefficient of Fill, BLOCK 1
Set-1
Set-2
Set-3
Set-4
Set-5
Set-6
Seismic Coefficeint, g
0.5
0
-0.5
0
10
20
30
40
50
60
Time, sec


Seismic Coefficient of Fill, BLOCK 2
Set-1
Set-2
Set-3
Set-4
Set-5
Set-6
Seismic Coefficeint, g
Time, sec
0.5
0
-0.5
0
10
20
30
40
50
60


Seismic Coefficient of Fill, BLOCK 3
Set-1
Set-2
Set-3
Set-4
Set-5
Set-6
0.5
0
-0.5
0
10
20
30
40
50
60
Seismic Coefficeint, g
Time, sec

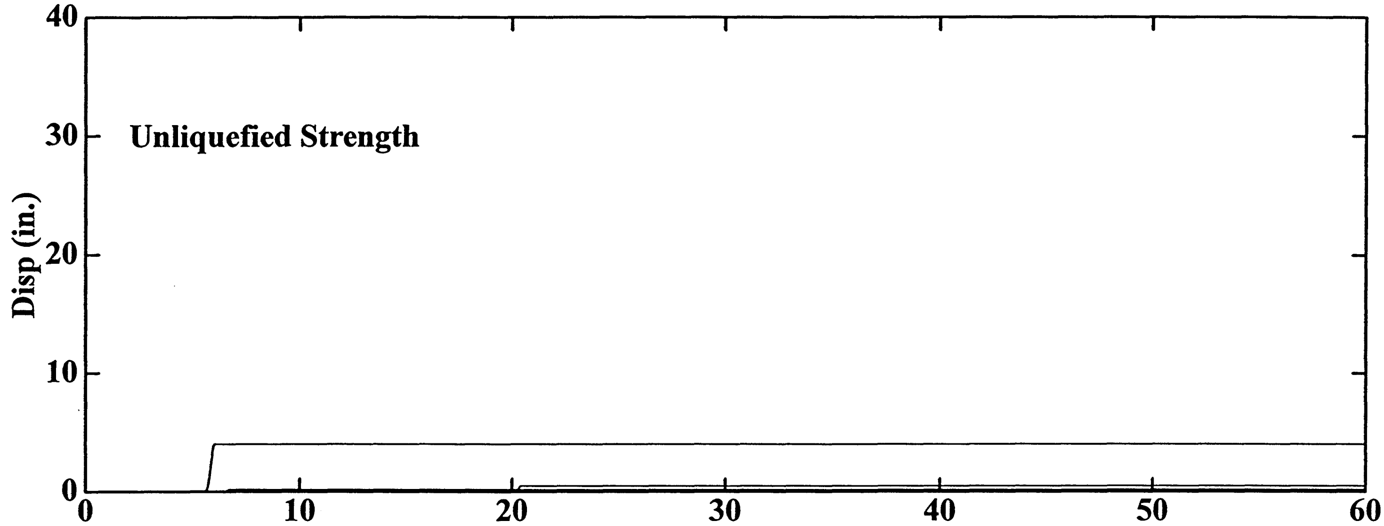
Permanent Displacement of Fill, BLOCK 1

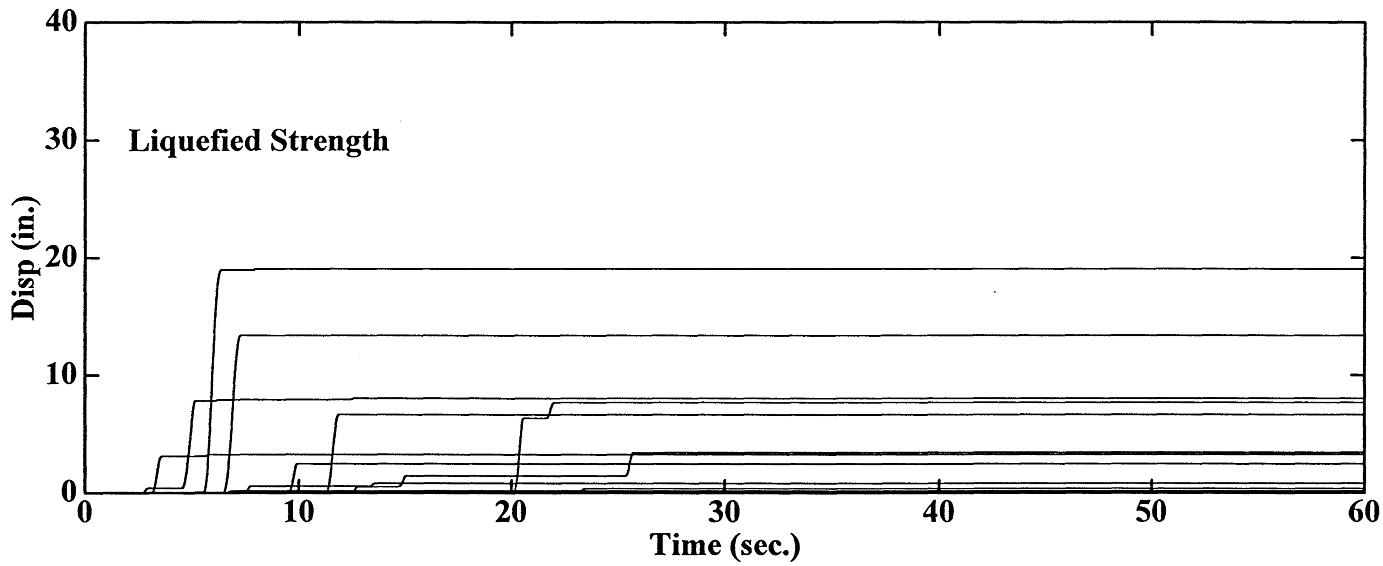




Permanent Displacement of Fill, BLOCK 2
Unliquefied Strength
Disp (in.)
40
30
20
10
0
0
10
20
30
40
50
60
Liquefied Strength
Disp (in.)
40
30
20
10
0
0
10
20
30
40
50
60
Time (sec.)

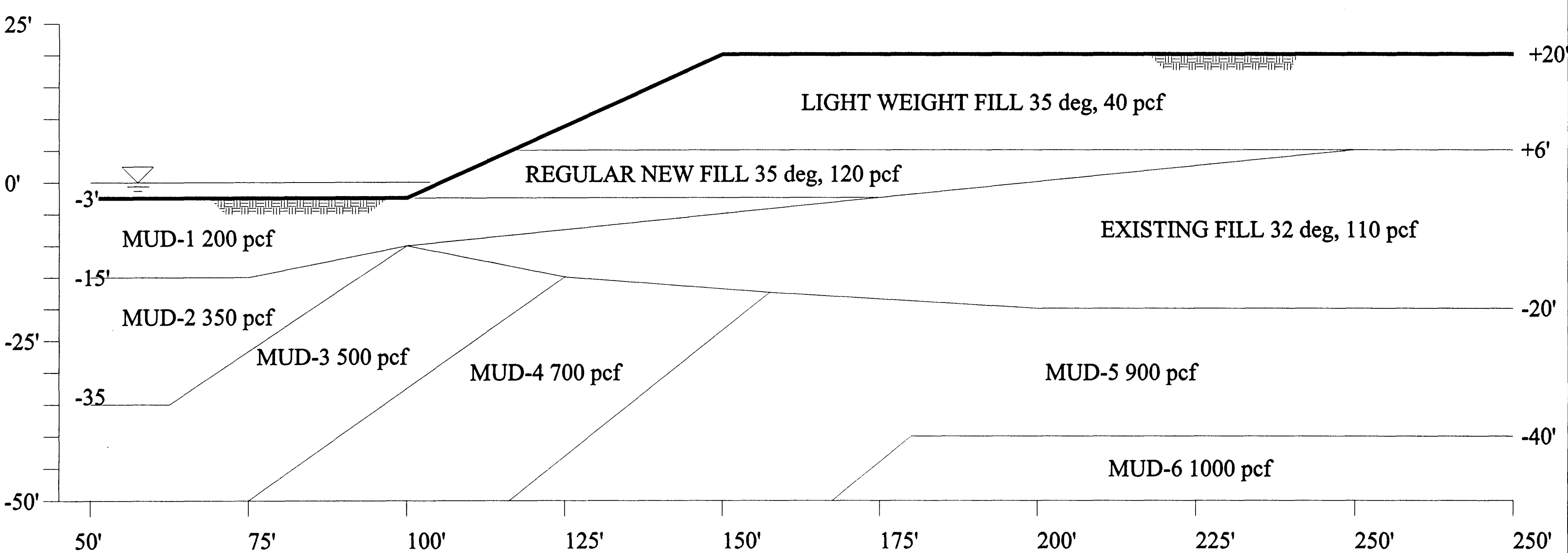
Permanent Displacement of Fill, BLOCK 3





APPENDIX B

Appendix B
Station 86+89
Proposed Light Weight Fill, γ_{fill} = 40 pcf (Grade at El. +20')



Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

NEW S.F.- OAKLAND BAY BRIDGE

Project No. 98-107

Date: 6-10-98

IDEALIZED SOIL PROFILE AT
OAKLAND SHORE APPROACH
STA. 86+89

Figure

TABLE 1 Summary of Pseudo Static Analyses for YBM (with light weight fill) 4/19/99

Block	Block Size, Measured from Shore	1.0*Su		1.4*Su		0.33*Su	
		Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g
Block 1	25'	2.36	0.19	2.9	0.27	1.31	0.05
Block 2	50'	2.97	0.19	3.56	0.27	1.52	0.06
Block 3	100'	3.36	0.19	4.34	0.26	1.72	0.06

Note:
(1) Using static shear strength
(2) Using dynamic shear strength (increase shear strength by 40%)
(3) Using residual shear strength (reduce shear strength to 33%)

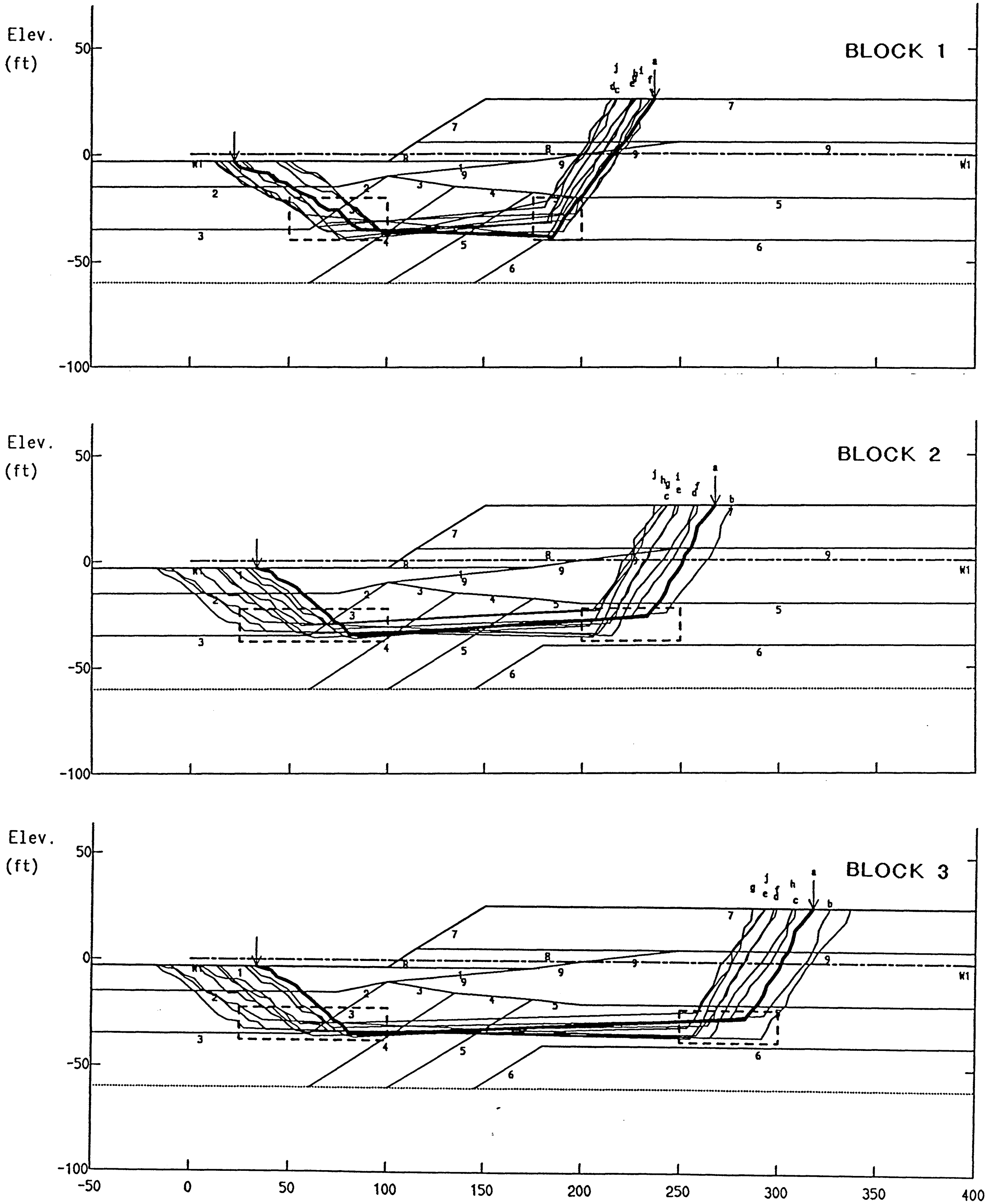
TABLE 2 Summary of Newmark Sliding Analyses for YBM (w/ light weight fill)

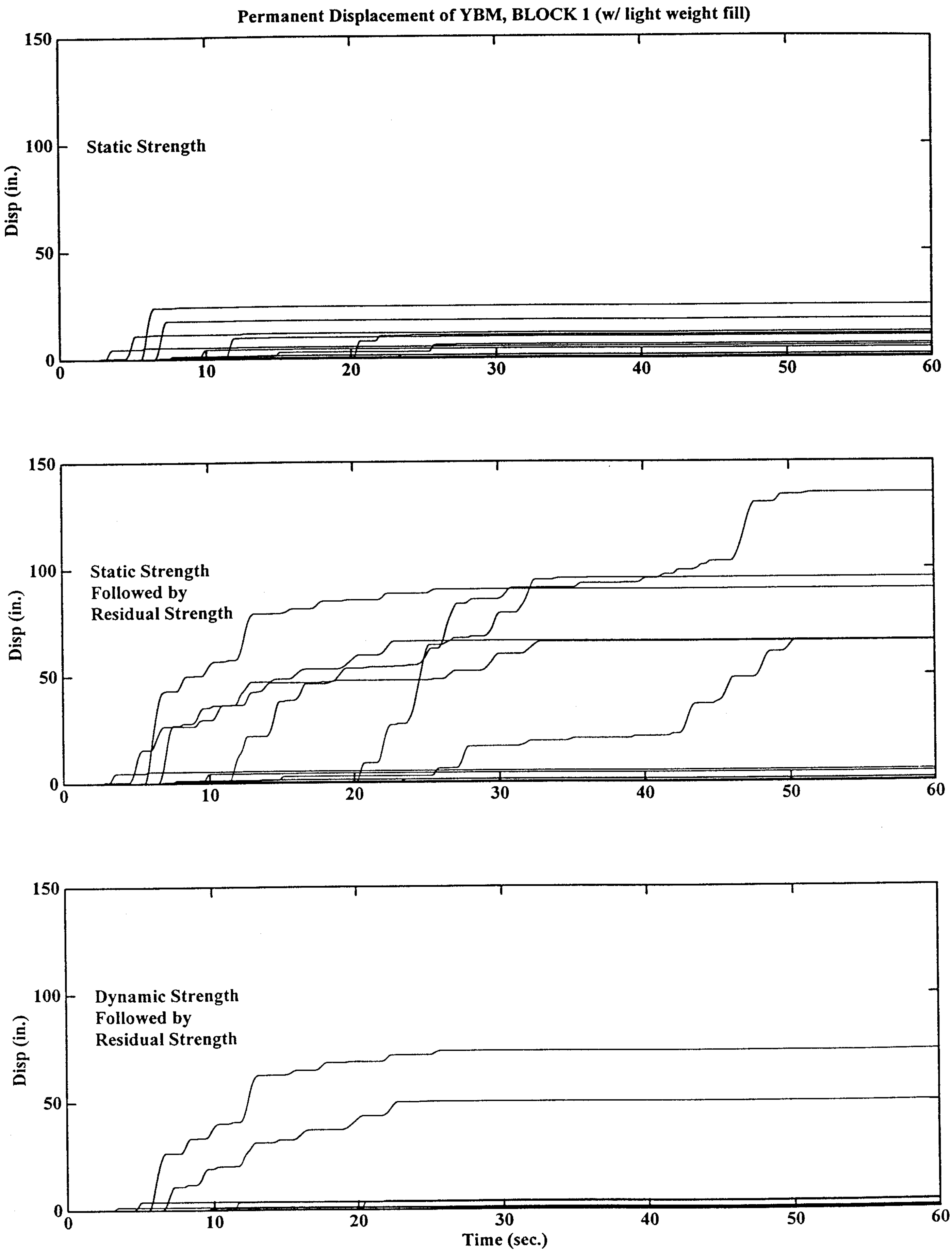
Ground Motion	Permanent Displacement, inches								
	Static Strength (1)			Static Followed By Residual (2)			Dynamic Followed By Residual (3)		
	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3
SET-1 (+)	5	5	4	5	5	4	1	1	1
SET-1 (-)	12	11	10	66	45	40	4	3	3
SET-2 (+)	11	10	9	95	74	67	4	4	4
SET-2 (-)	2	1	1	2	1	1	0	0	0
SET-3 (+)	10	9	8	135	94	82	4	3	3
SET-3 (-)	6	6	5	66	6	5	1	1	1
SET-4 (+)	24	23	20	90	70	64	74	55	50
SET-4 (-)	1	0	0	1	0	0	0	0	0
SET-5 (+)	2	1	1	2	1	1	0	0	0
SET-5 (-)	4	4	3	4	4	3	1	1	1
SET-6 (+)	1	1	1	1	1	1	0	0	0
SET-6 (-)	18	16	15	66	45	39	50	30	26
minimum	1	0	0	1	0	0	0	0	0
maximum	**24**	**23**	**20**	**135**	**94**	**82**	**74**	**55**	**50**
average	8	7	6	44	29	26	12	8	7

Note:
(1) Using constant static strength
(2) Using static strength followed by residual strength when deformation exceeds 6 inches
(3) Using dynamic strength followed by residual strength when deformation exceeds 6 inches

LATERAL SPREADING IN YBM

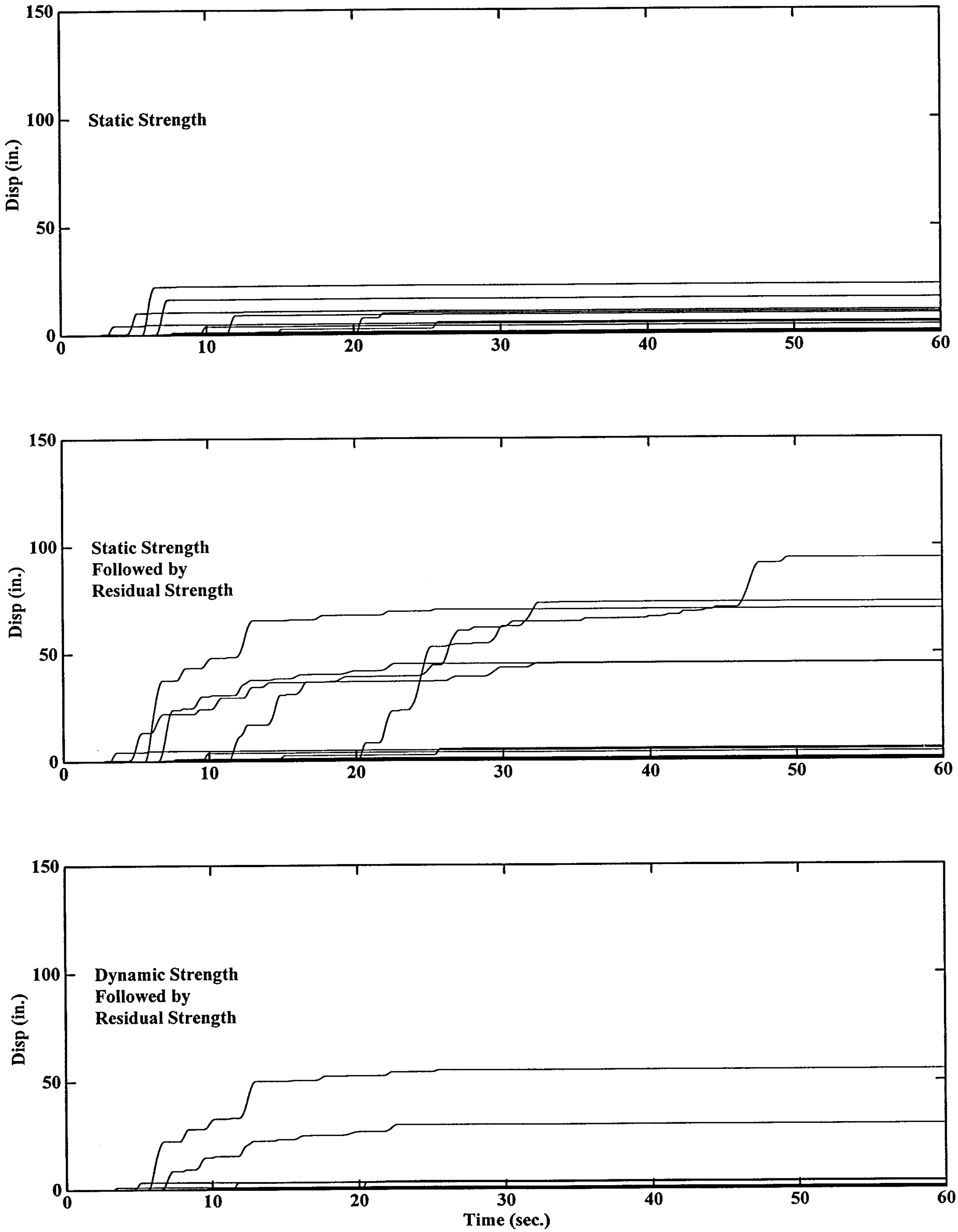
WITH LIGHT WEIGHT FILL OPTION

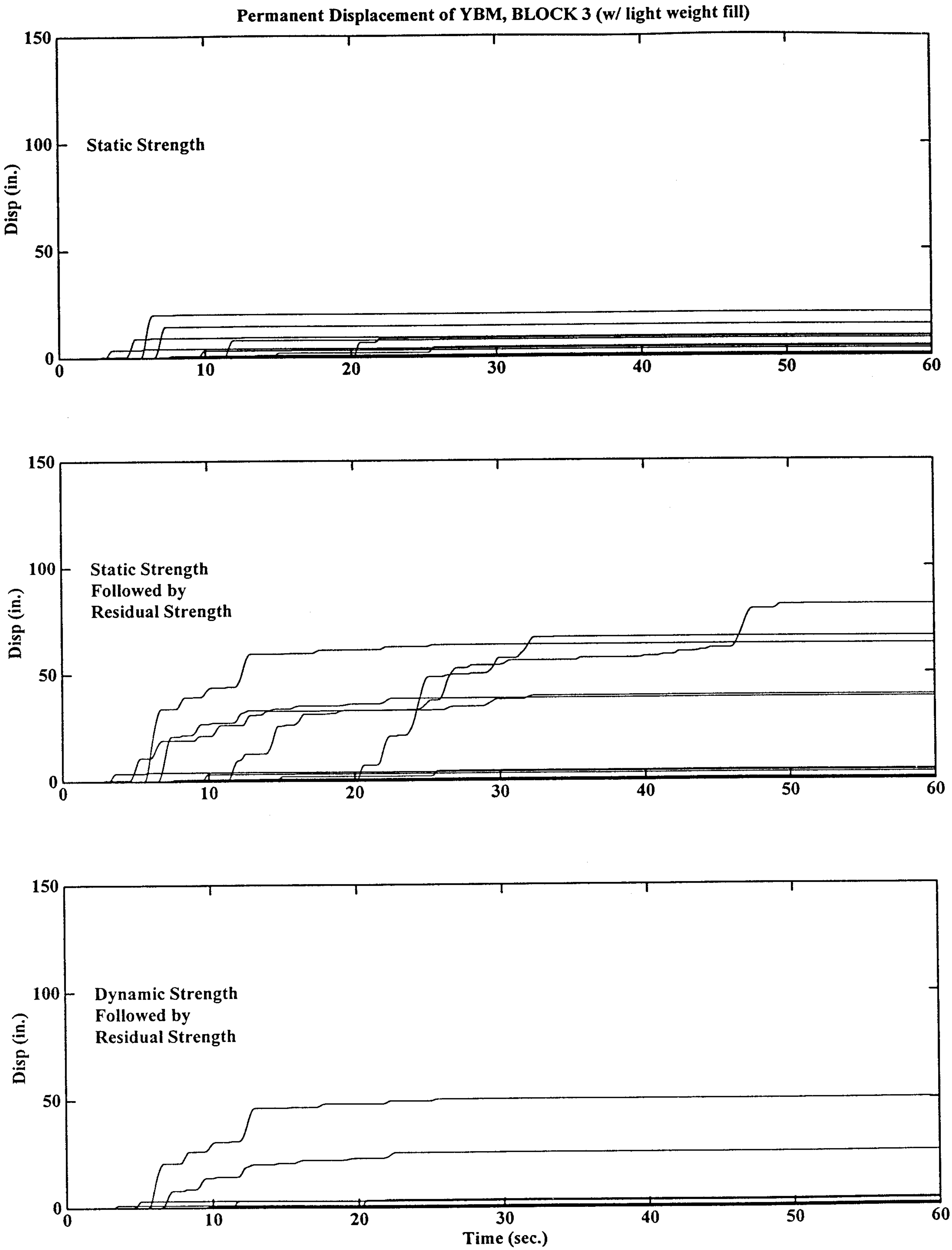




Permanent Displacement of YBM, BLOCK 1 (w/ light weight fill)
Static Strength
Static Strength
Followed by
Residual Strength
Dynamic Strength
Followed by
Residual Strength
Disp (in.)
Time (sec.)
0
50
100
150
0
10
20
30
40
50
60

Permanent Displacement of YBM, BLOCK 2 (w/ light weight fill)





Permanent Displacement of YBM, BLOCK 3 (w/ light weight fill)
150
100
50
0
Disp (in.)
Static Strength
0
10
20
30
40
50
60
Static Strength
Followed by
Residual Strength
Dynamic Strength
Followed by
Residual Strength
Time (sec.)

TABLE 3 Summary of Pseudo Static Analyses for FILL (w/ light weight fill)

Block	Block Size, Measured from Shore	Drained Strength (1)		600 psf (2)		800 psf (3)	
		Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g
Block 1	25'	4.33	0.39	2.73	0.22	3.26	0.28
Block 2	50'	5.28	0.4	3.2	0.22	3.93	0.28
Block 3	100'	6.99	0.41	4.03	0.21	5.08	0.28

Note:
(1) Using drained shear strength of sand
(2) Using residual shear strength of sand in fill below watertable (Su=600 psf)
(2) Using residual shear strength of sand in fill below watertable (Su=800 psf)

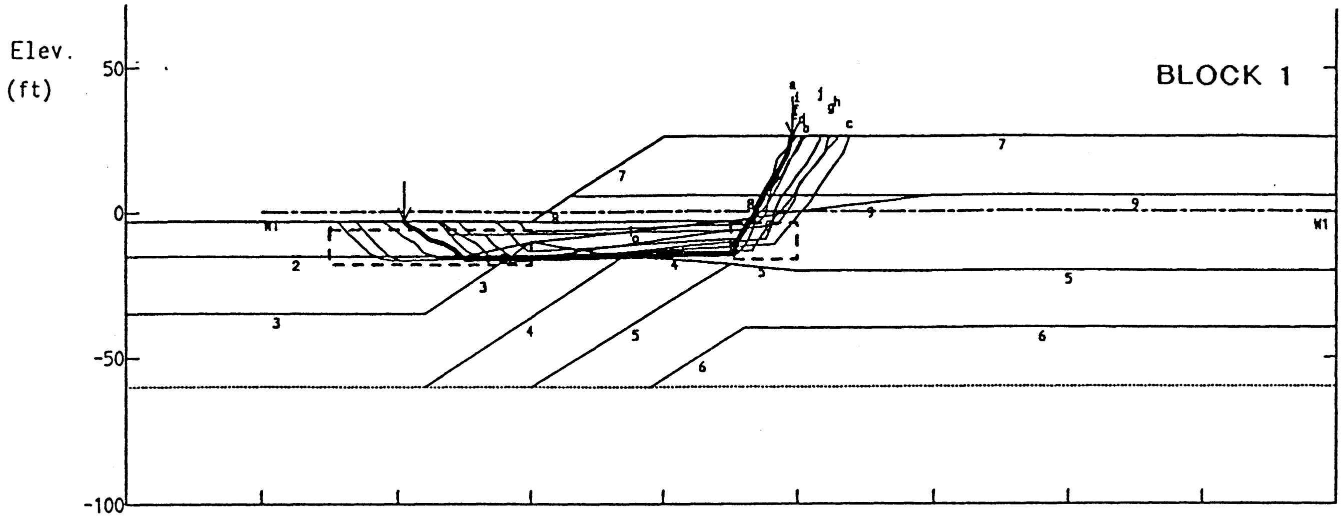
TABLE 4 Summary of Newmark Sliding Analyses for FILL (w/ light weight fill)

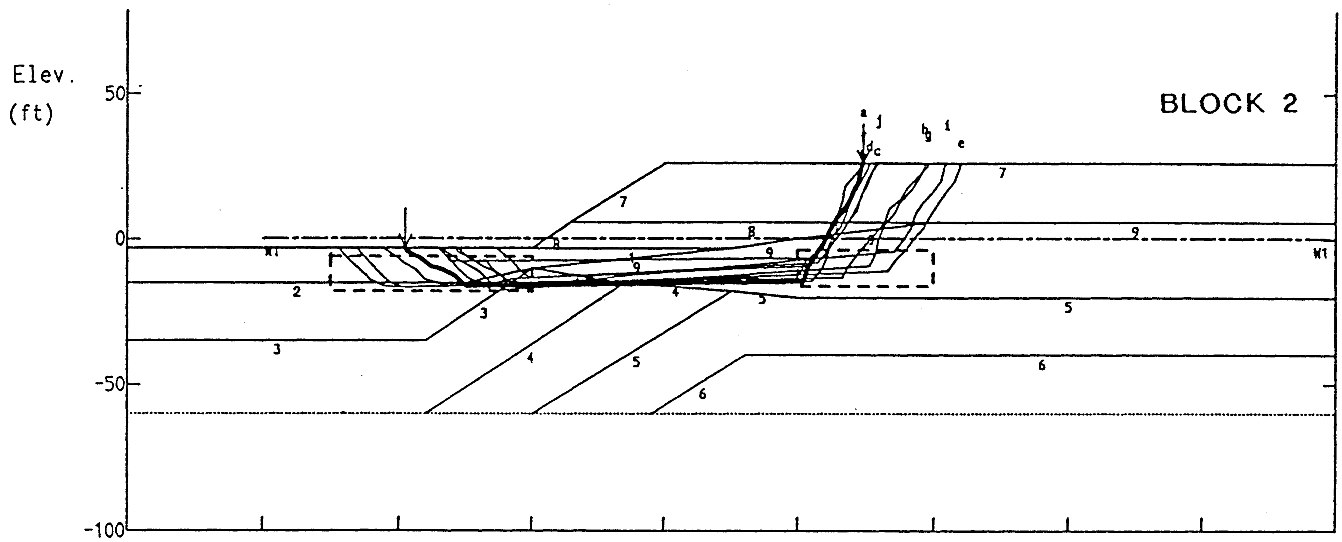
Ground Motion	Permanent Displacement, inches								
	Drained Strength (1)			Residual Strength of 600 psf (2)			Residual Strength of 800 psf (3)		
	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3
SET-1 (+)	0	0	0	8	7	5	3	3	2
SET-1 (-)	2	0	0	16	14	12	9	7	5
SET-2 (+)	2	2	1	15	12	11	8	7	5
SET-2 (-)	0	0	0	2	2	1	0	0	0
SET-3 (+)	2	1	0	14	12	10	7	6	4
SET-3 (-)	0	0	0	9	7	6	3	2	1
SET-4 (+)	10	8	5	34	29	25	22	19	15
SET-4 (-)	0	0	0	1	1	1	0	0	0
SET-5 (+)	0	0	0	2	2	2	1	0	0
SET-5 (-)	0	0	0	7	5	4	2	2	1
SET-6 (+)	0	0	0	2	1	1	0	0	0
SET-6 (-)	5	3	1	25	21	18	15	13	9
minimum	0	0	0	1	1	1	0	0	0
maximum	**10**	**8**	**5**	**34**	**29**	**25**	**22**	**19**	**15**
average	2	1	1	11	9	8	6	5	4

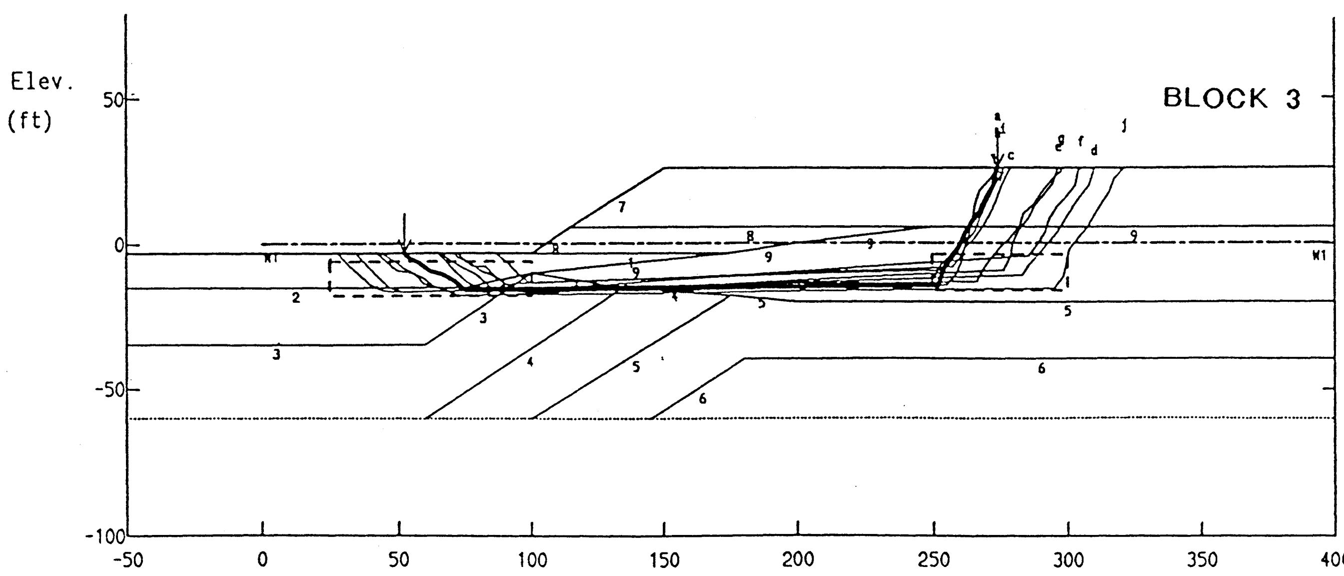
Note:
(1) Using constant drained shear strength of sand (unliquefied strength)
(2) Using constant residual shear strength of sand (liquefied strength of 600 psf)
(3) Using constant residual shear strength of sand (liquefied strength of 800 psf)

LATERAL SPREADING IN FILL

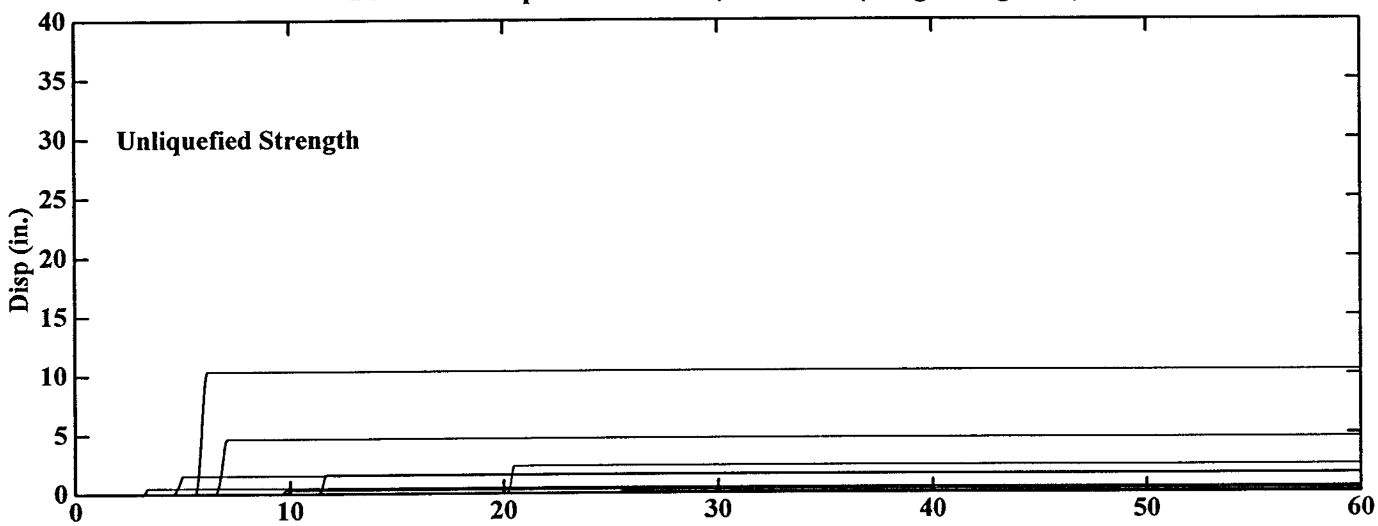
WITH LIGHT WEIGHT FILL OPTION

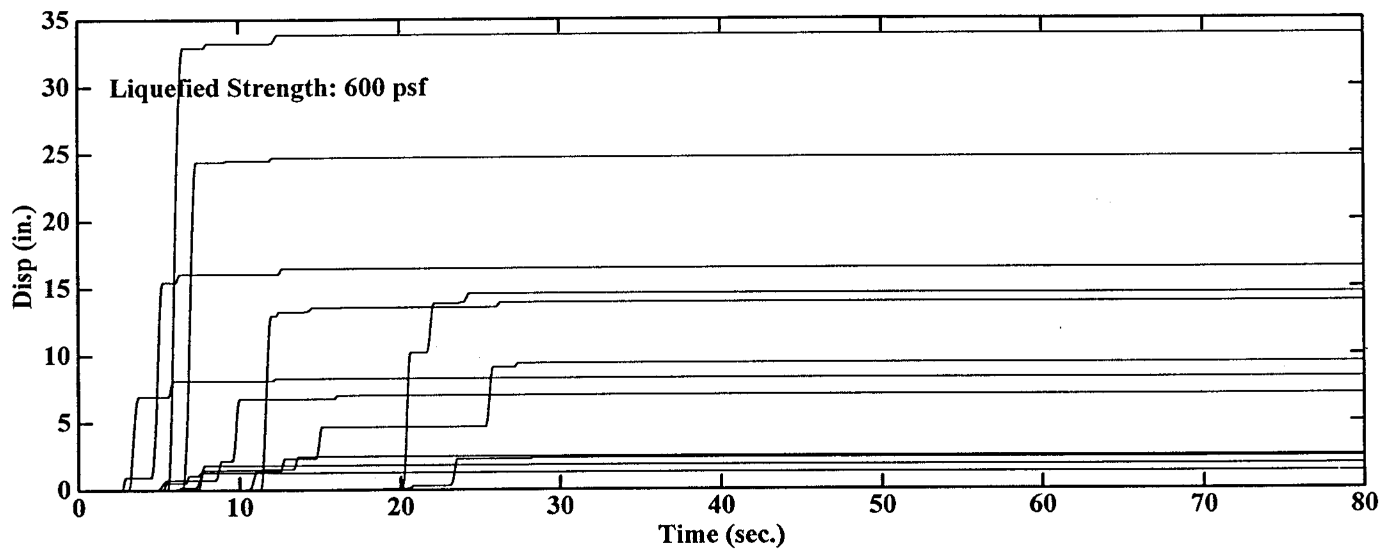


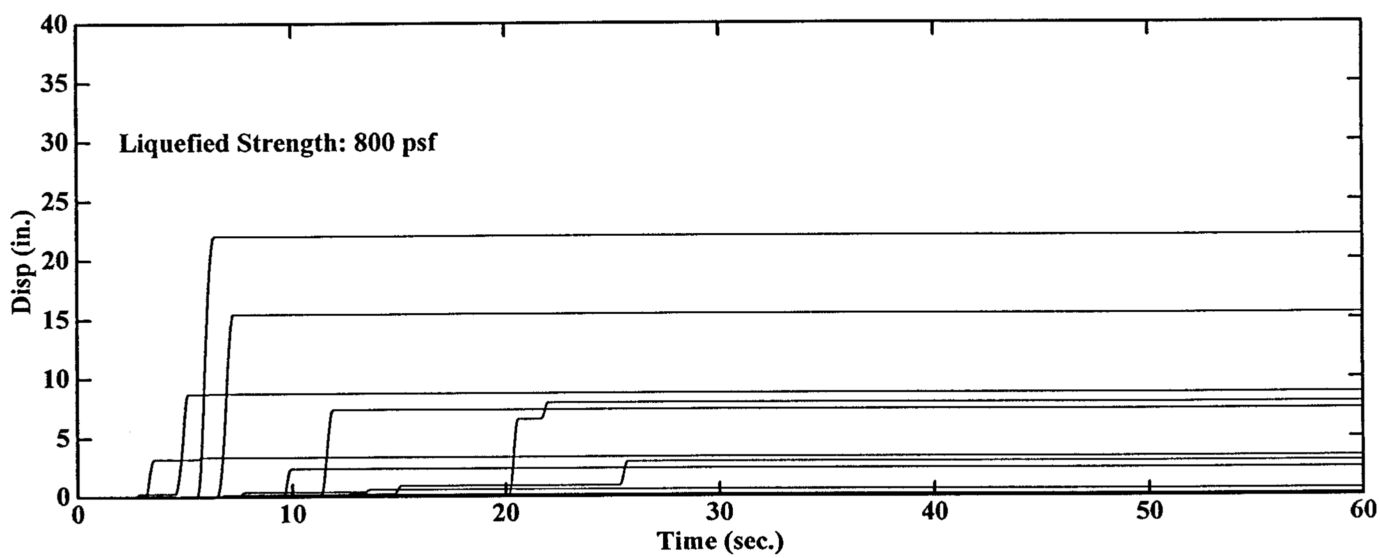




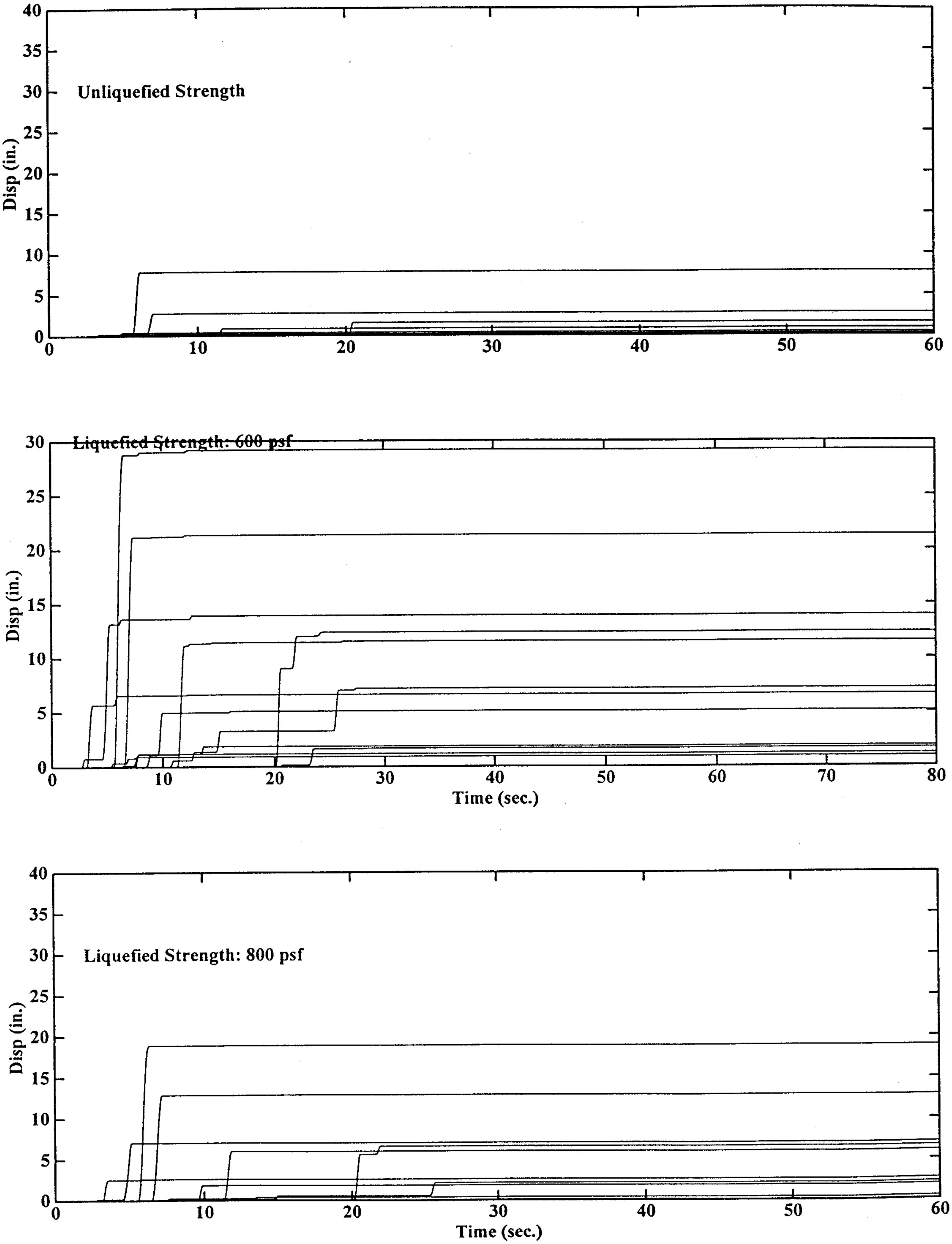
Permanent Displacement of Fill, BLOCK 1 (w/ light weight fill)

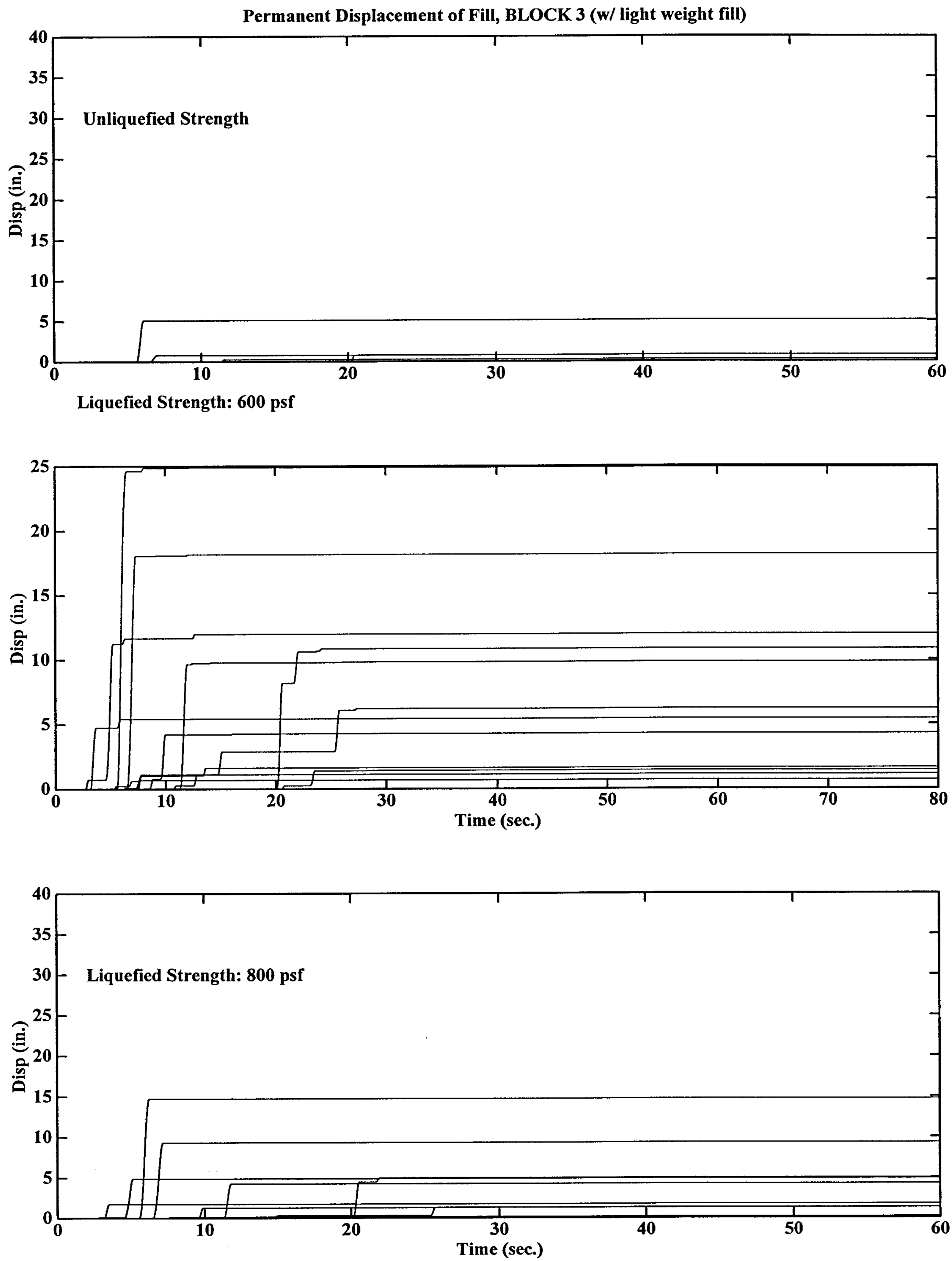






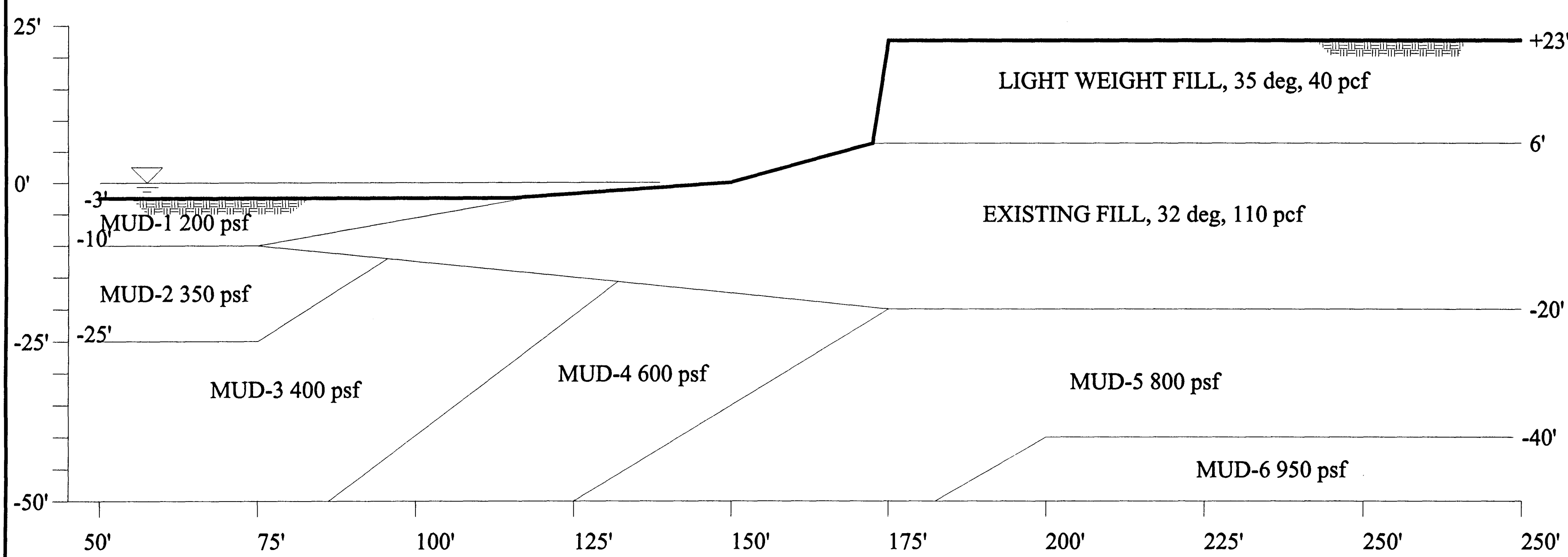
Permanent Displacement of Fill, BLOCK 2 (w/ light weight fill)




Permanent Displacement of Fill, BLOCK 3 (w/ light weight fill)
Unliquefied Strength
Disp (in.)
0
5
10
15
20
25
30
35
40
0
10
20
30
40
50
60
Liquefied Strength: 600 psf
Disp (in.)
0
5
10
15
20
25
0
10
20
30
40
50
60
70
80
Time (sec.)
Liquefied Strength: 800 psf
Disp (in.)
0
5
10
15
20
25
30
35
40
0
10
20
30
40
50
60
Time (sec.)

APPENDIX C

Appendix C
Station 86+34
Proposed Light Weight Fill, γ_{fill} = 40 pcf (Grade at El. +23')



Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

NEW S.F.- OAKLAND BAY BRIDGE

Project No. 98-107 | Date: 6-10-98

IDEALIZED SOIL PROFILE AT
OAKLAND SHORE APPROACH
STA. 86+34

Figure

TABLE 1 Summary of Pseudo Static Analyses for YBM (w/ light weight fill at Sta. 86+34) 5/13/99

Block	Block Size, Measured from Shore	1.0*Su (1)		1.4*Su (2)		0.33*Su (3)		0.80*Su (4)	
		Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g
Block 1	25'	2.08	0.16	2.56	0.23	1.28	0.04	1.84	0.13
Block 2	50'	2.36	0.16	2.95	0.23	1.37	0.04	2.07	0.13
Block 3	100'	2.87	0.155	3.7	0.23	1.49	0.04	1.84	0.13

Note:
(1) Using static shear strength
(2) Using dynamic shear strength, i.e., strain rate effects (increase shear strength by 40%)
(3) Using residual shear strength (reduce shear strength to 33%)
(4) Using residual shear strength (reduce shear strength to 80%)

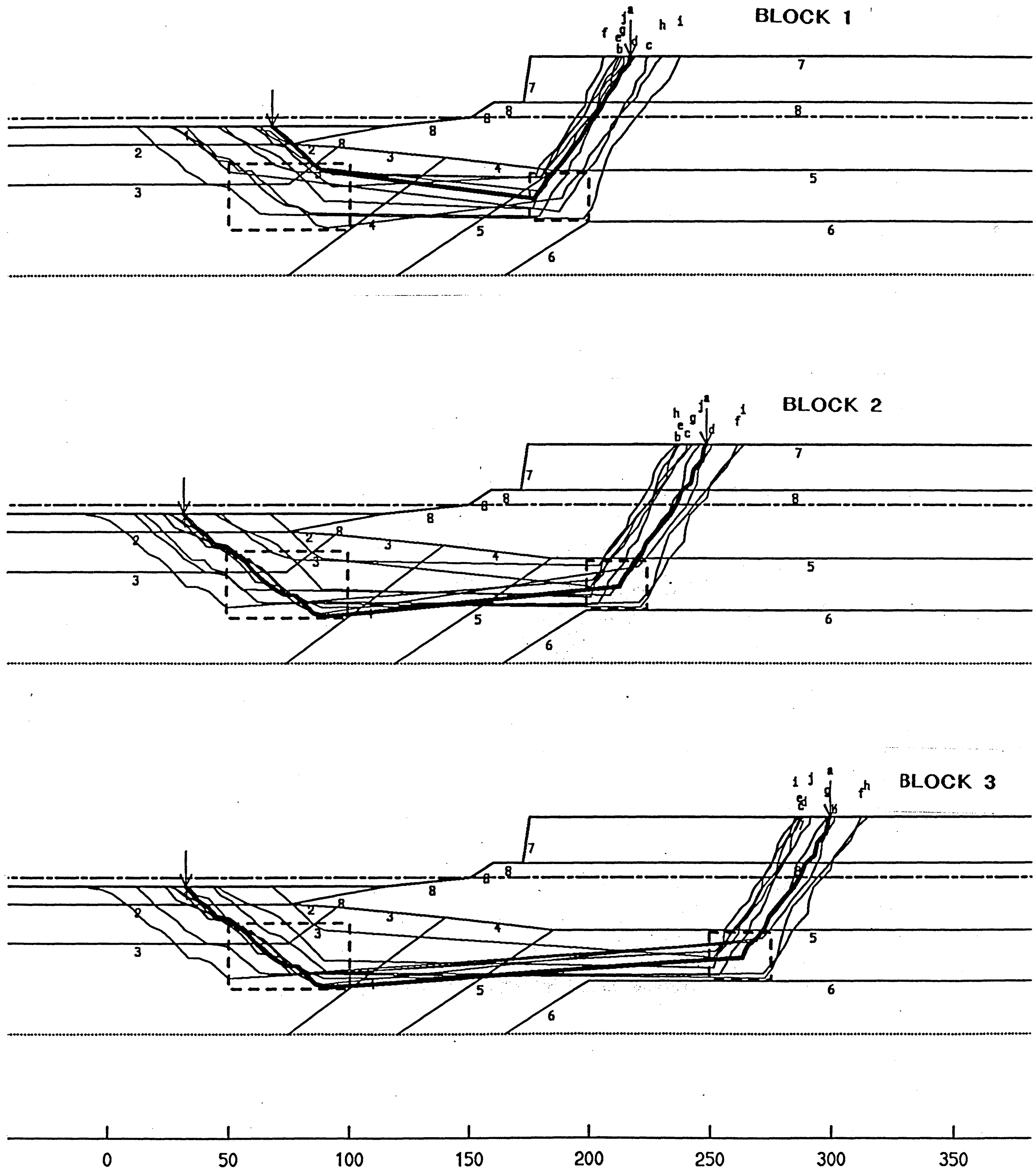
TABLE 2 Summary of Newmark Sliding Analyses for YBM (w/ light weight fill at Sta. 86+34)

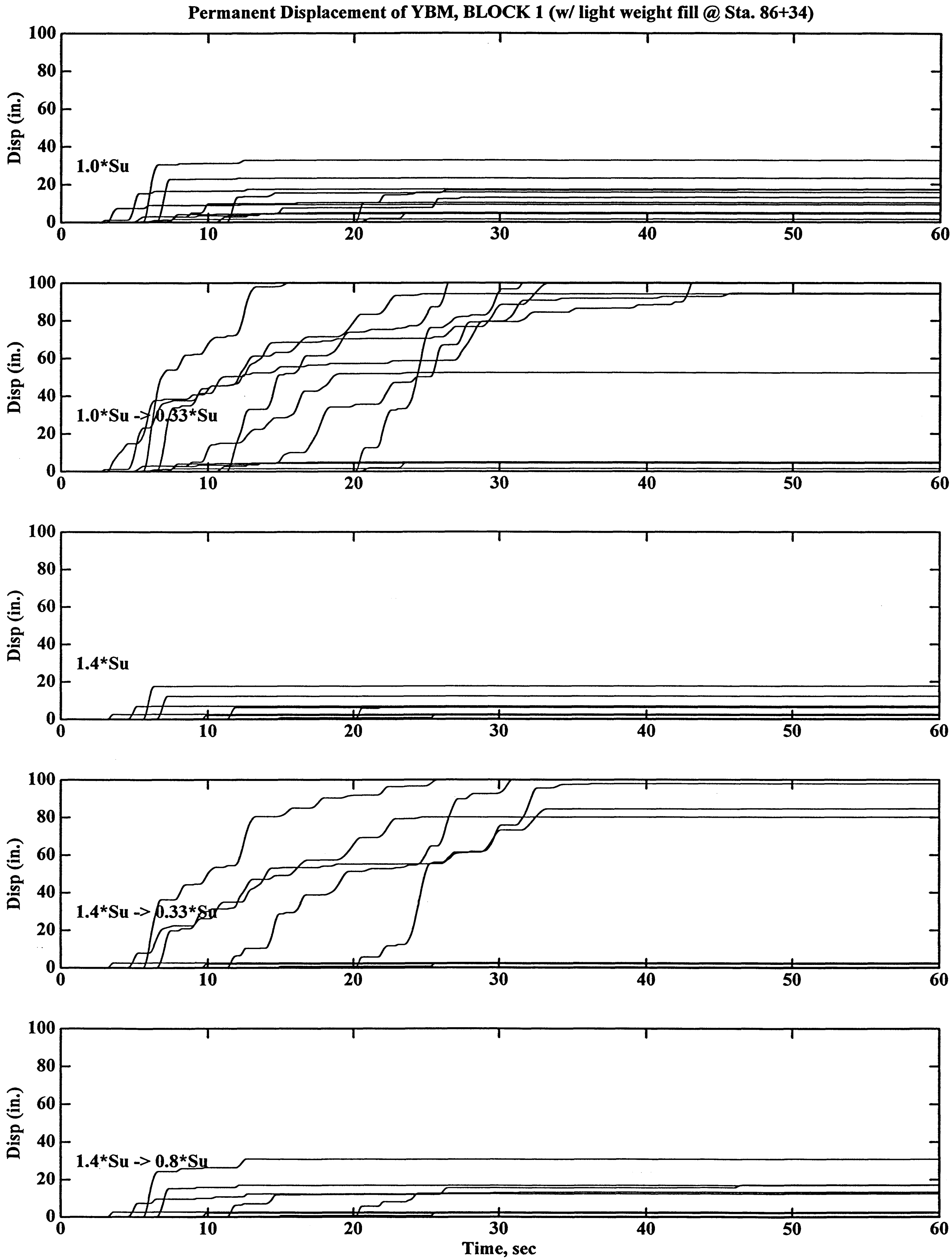
Ground Motion	Permanent Displacement, inches														
	Static Strength (1)			Static Followed By Residual (2)			Dynamic Strength (3)			Dynamic Followed By Residual (4)			Dynamic Followed By Residual (5)		
	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3
SET-1 (+)	9	8	8	95	86	76	3	2	2	3	2	2	3	2	2
SET-1 (-)	17	16	15	100	92	83	7	6	5	85	77	5	12	11	5
SET-2 (+)	16	14	14	119	112	104	7	6	5	98	89	5	13	11	5
SET-2 (-)	5	4	4	5	4	4	0	0	0	0	0	0	0	0	0
SET-3 (+)	17	15	15	186	174	159	6	6	5	164	6	5	17	6	5
SET-3 (-)	13	11	11	149	137	125	3	2	2	3	2	2	3	2	2
SET-4 (+)	33	30	28	118	110	100	18	16	15	100	93	84	31	28	24
SET-4 (-)	2	1	1	2	1	1	0	0	0	0	0	0	0	0	0
SET-5 (+)	5	4	4	5	4	4	0	0	0	0	0	0	0	0	0
SET-5 (-)	10	9	8	52	47	42	2	2	1	2	2	1	2	2	1
SET-6 (+)	4	4	4	4	4	4	0	0	0	0	0	0	0	0	0
SET-6 (-)	23	22	20	94	87	78	12	11	10	80	73	64	17	15	13
minimum	2	1	1	2	1	1	0	0	0	0	0	0	0	0	0
maximum	33	30	28	186	174	159	18	16	15	164	93	84	31	28	24
average	13	12	11	77	71	65	5	4	4	45	29	14	8	6	5

Note:
(1) Using constant static strength
(2) Using static strength followed by residual strength (0.33*Su) when deformation exceeds 6 inches
(3) Using dynamic strength
(4) Using dynamic strength followed by residual strength (0.33*Su) when deformation exceeds 6 inches
(5) Using dynamic strength followed by residual strength (0.80*Su) when deformation exceeds 6 inches

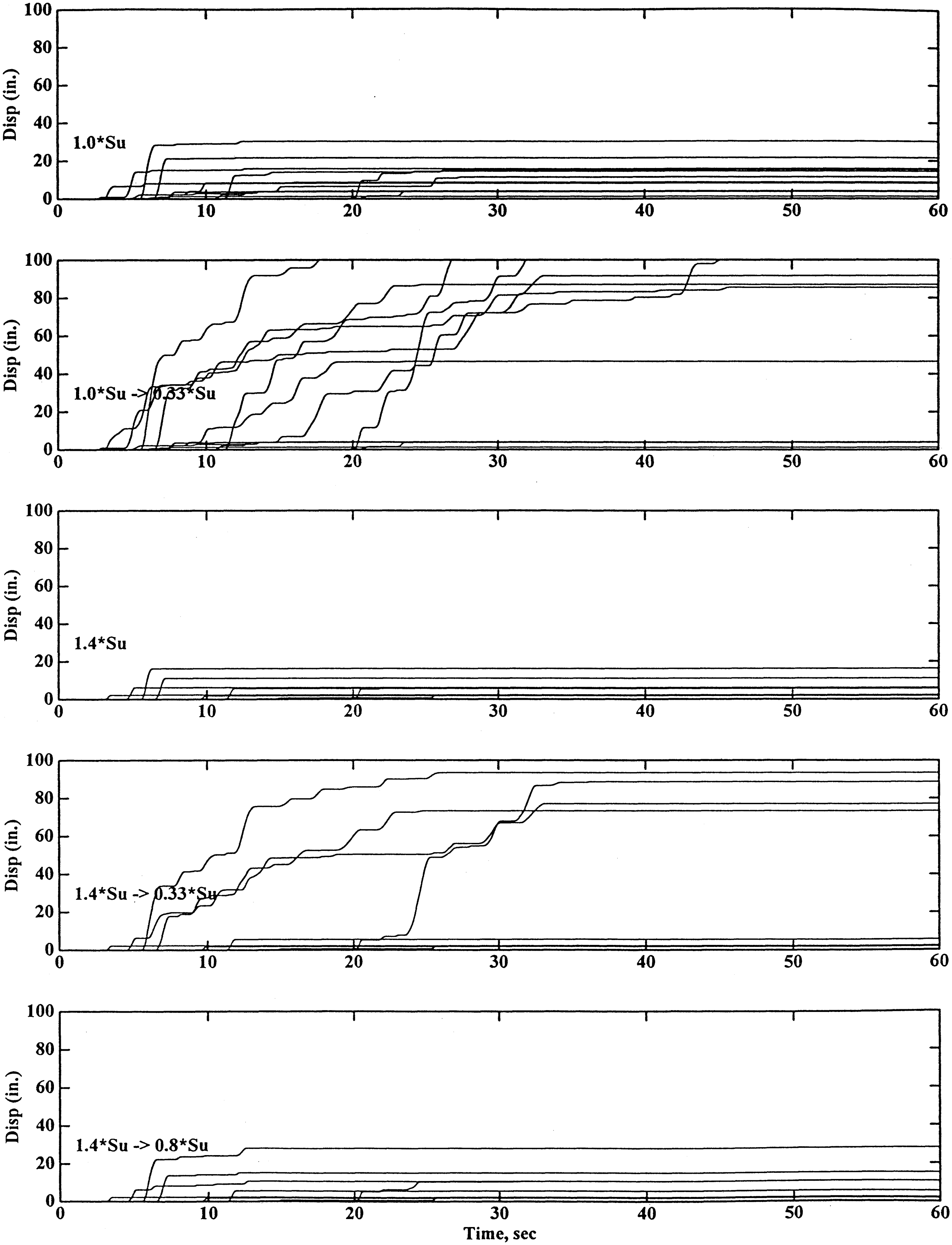
LIGHT WEIGHT FILL @ STA. 86-34

LATERAL SPREADING IN YBM




Permanent Displacement of YBM, BLOCK 1 (w/ light weight fill @ Sta. 86+34)
Disp (in.)
1.0*Su
1.0*Su -> 0.33*Su
1.4*Su
1.4*Su -> 0.33*Su
1.4*Su -> 0.8*Su
Time, sec

Permanent Displacement of YBM, BLOCK 2 (w/ light weight fill @ Sta. 86+34)



Permanent Displacement of YBM, BLOCK 3 (w/ light weight fill @ Sta. 86+34)

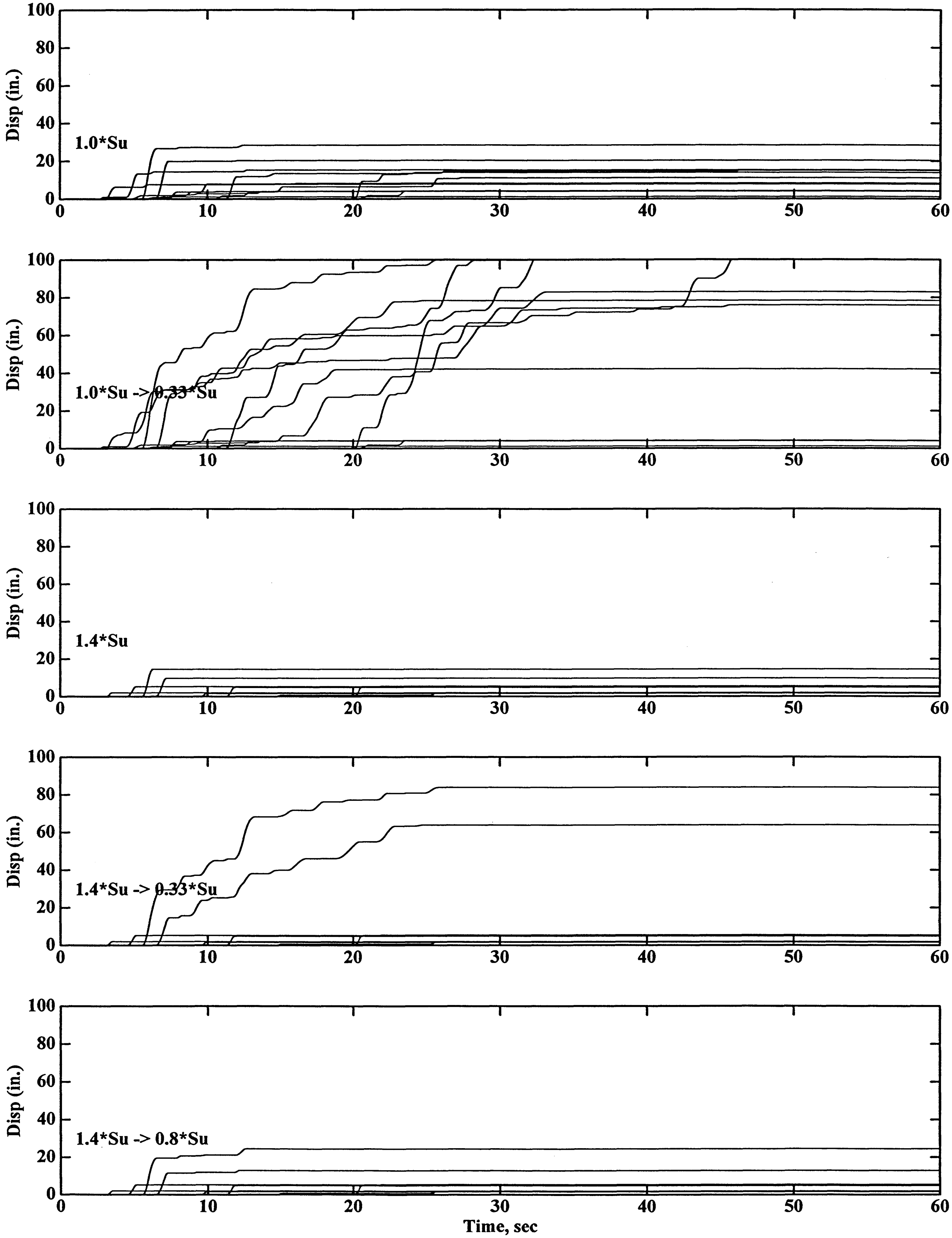


TABLE 3 Summary of Pseudo Static Analyses for FILL (w/ light weight fill at Sta. 86+34)

Block	Block Size, Measured from Shore	Drained Strength (1)		600 psf (2)		800 psf (3)	
		Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g
Block 1	25'	4.17	0.45	2.34	0.25	2.92	0.32
Block 2	50'	5.11	0.45	2.78	0.23	3.5	0.3
Block 3	100'	6.78	0.41	3.6	0.22	4.6	0.28

Note:
(1) Using drained shear strength of sand
(2) Using residual shear strength of sand in fill below watertable (Su=600 psf)
(2) Using residual shear strength of sand in fill below watertable (Su=800 psf)

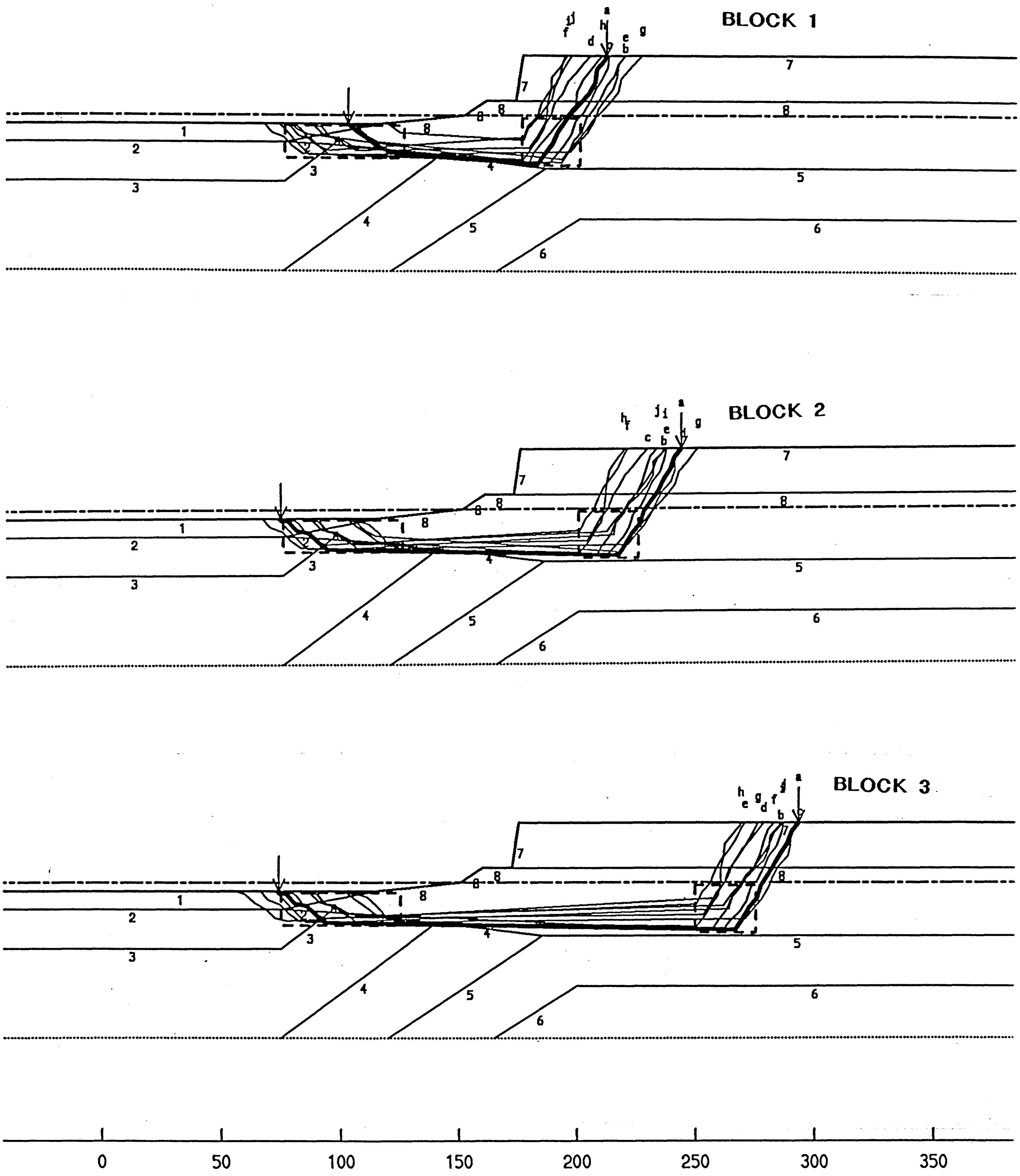
TABLE 4 Summary of Newmark Sliding Analyses for FILL (w/ light weight fill at Sta. 86+34)

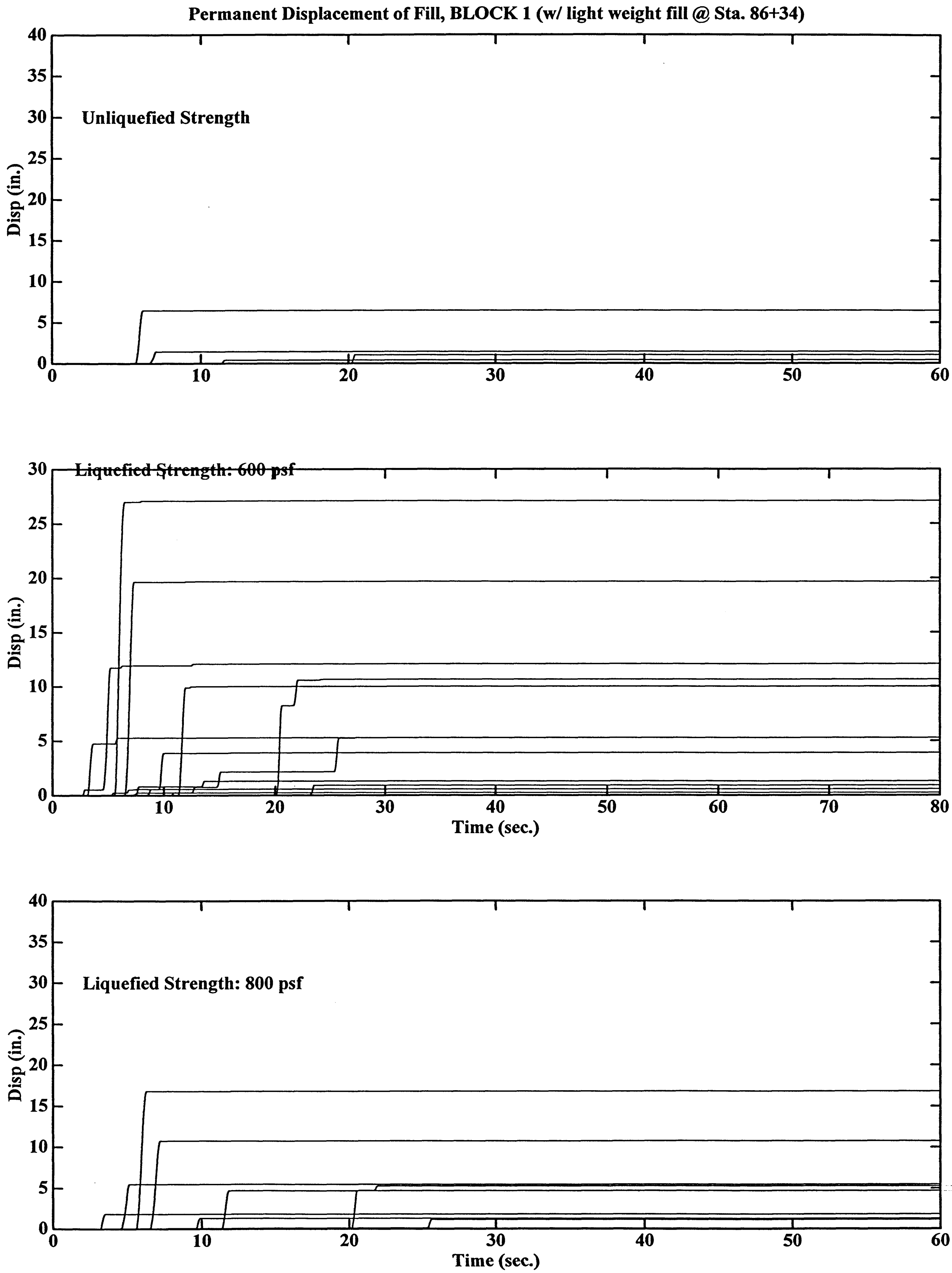
Ground Motion	Permanent Displacement, inches								
	Drained Strength (1)			Residual Strength of 600 psf (2)			Residual Strength of 800 psf (3)		
	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3
SET-1 (+)	0	0	0	5	6	5	2	2	2
SET-1 (-)	0	0	0	12	12	11	5	6	5
SET-2 (+)	1	1	1	11	11	10	5	5	5
SET-2 (-)	0	0	0	1	1	1	0	0	0
SET-3 (+)	0	0	0	10	10	9	5	5	4
SET-3 (-)	0	0	0	5	6	5	1	1	1
SET-4 (+)	6	5	5	27	27	23	17	16	15
SET-4 (-)	0	0	0	1	1	0	0	0	0
SET-5 (+)	0	0	0	1	2	1	0	0	0
SET-5 (-)	0	0	0	4	4	4	1	1	1
SET-6 (+)	0	0	0	0	1	1	0	0	0
SET-6 (-)	1	1	1	20	20	17	11	11	9
minimum	0	0	0	0	1	0	0	0	0
maximum	6	5	5	27	27	23	17	16	15
average	1	1	1	8	8	7	4	4	4

Note:
(1) Using constant drained shear strength of sand (unliquefied strength)
(2) Using constant residual shear strength of sand (liquefied strength of 600 psf)
(3) Using constant residual shear strength of sand (liquefied strength of 800 psf)

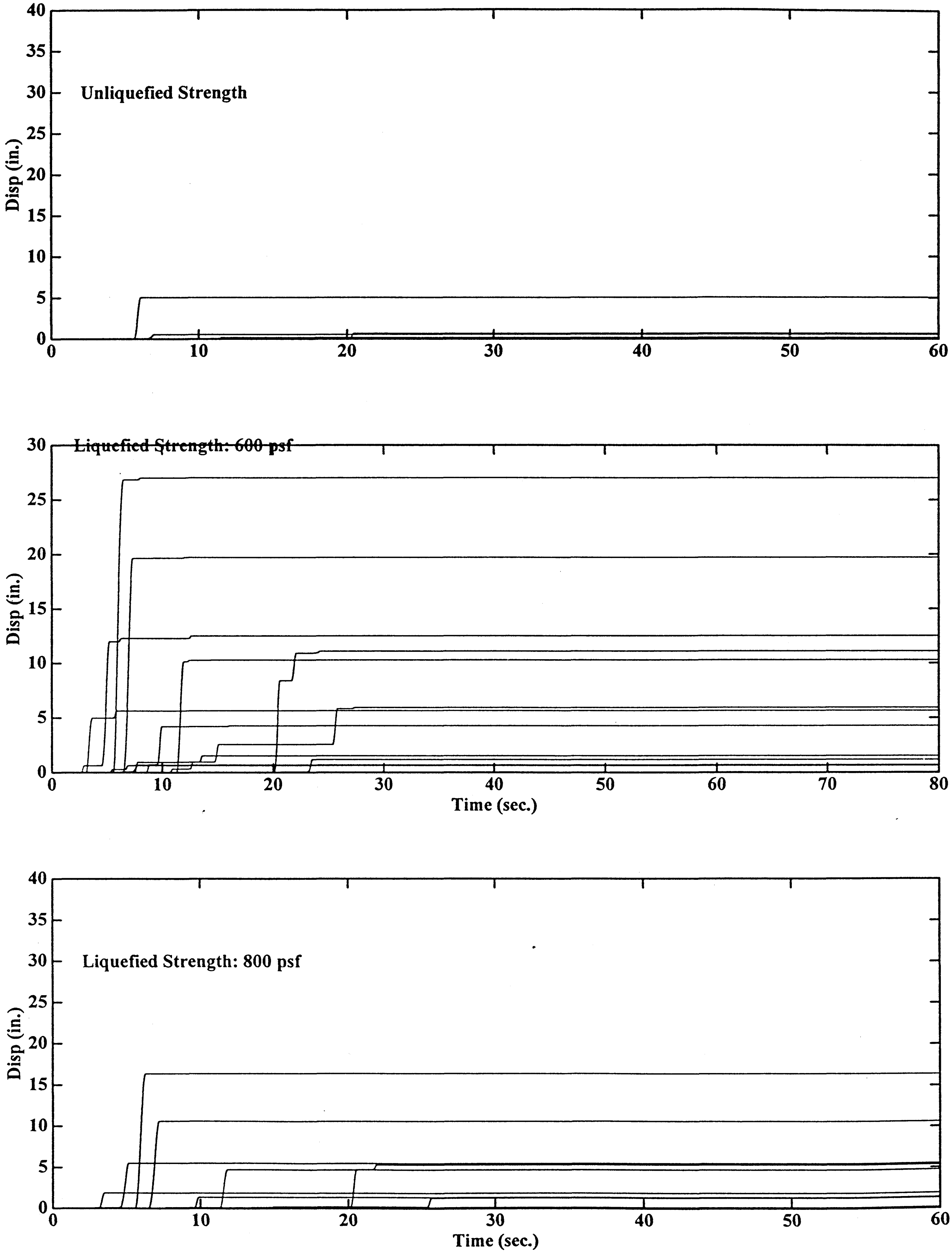
LIGHT WEIGHT FILL @ STA. 86-34

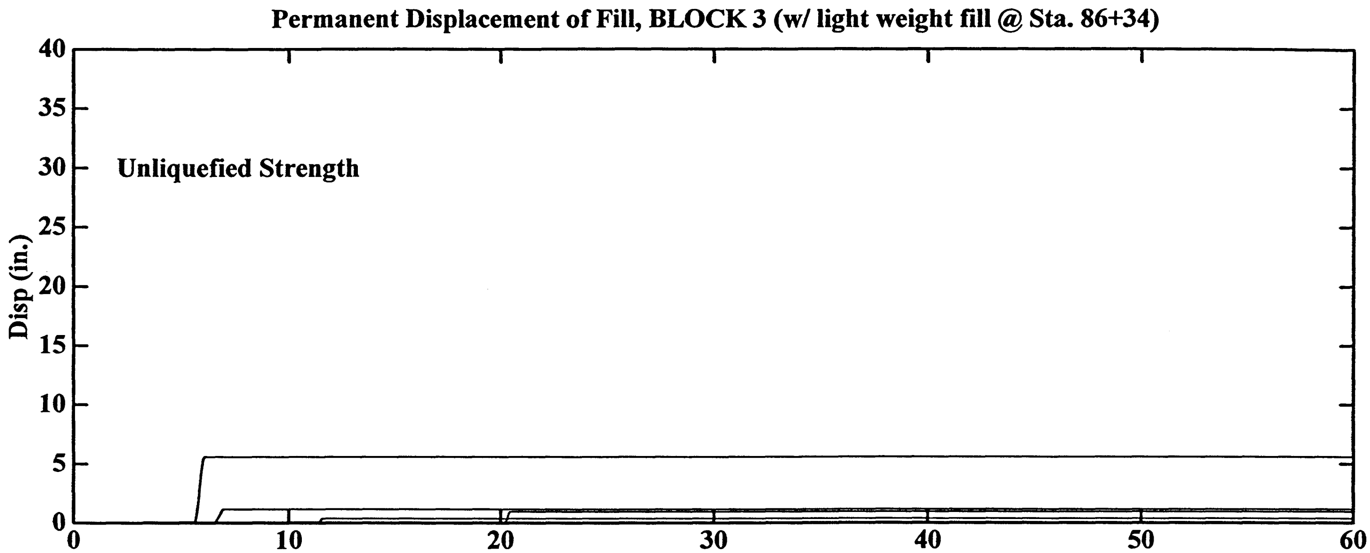
LATERAL SPREADING IN FILL

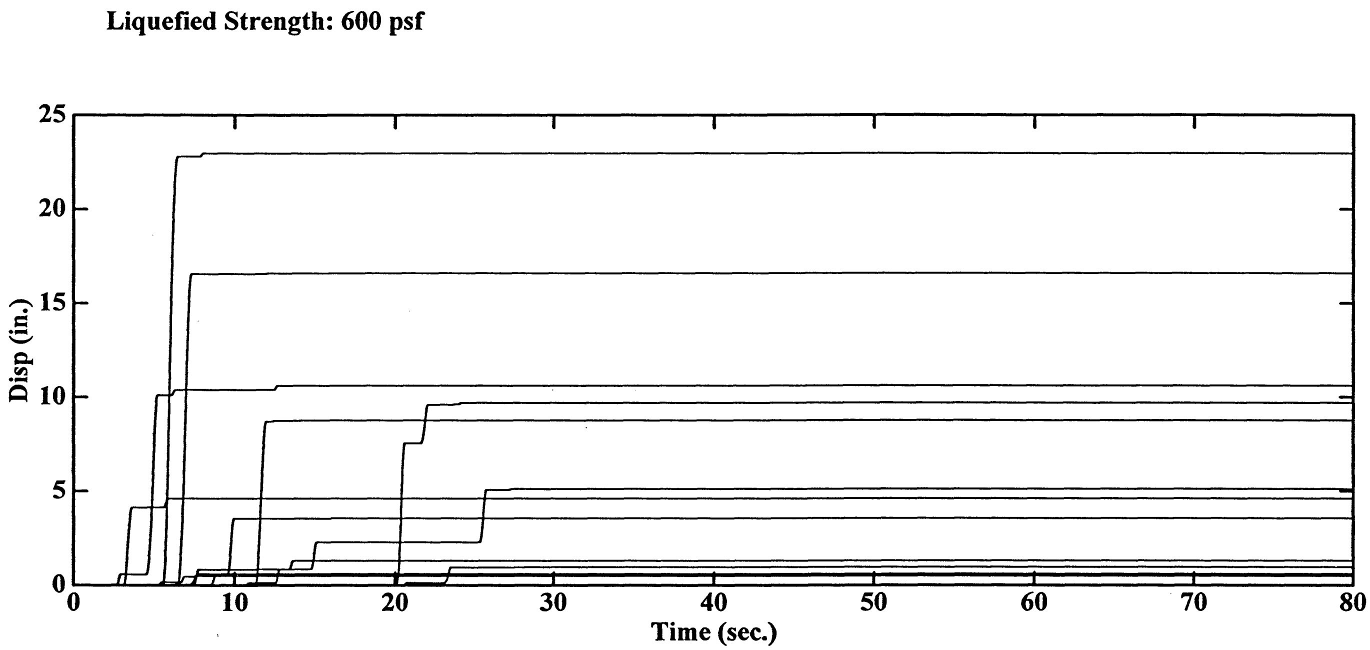


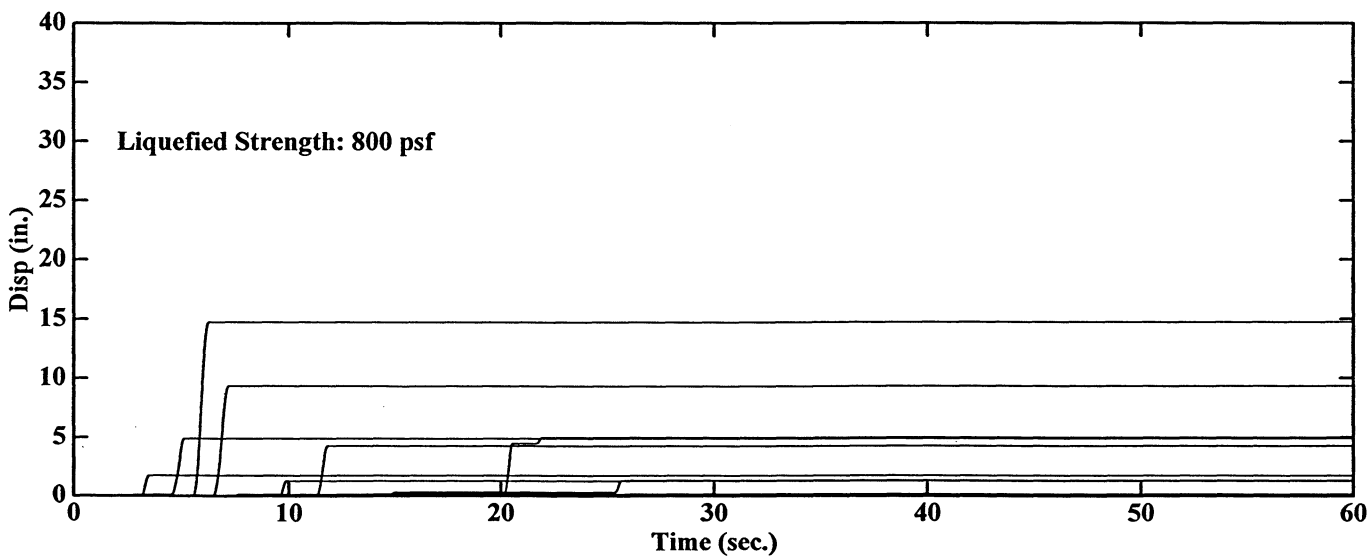

Permanent Displacement of Fill, BLOCK 1 (w/ light weight fill @ Sta. 86+34)
Unliquefied Strength
Disp (in.)
Liquefied Strength: 600 psf
Disp (in.)
Time (sec.)
Liquefied Strength: 800 psf
Disp (in.)
Time (sec.)

Permanent Displacement of Fill, BLOCK 2 (w/ light weight fill @ Sta. 86+34)



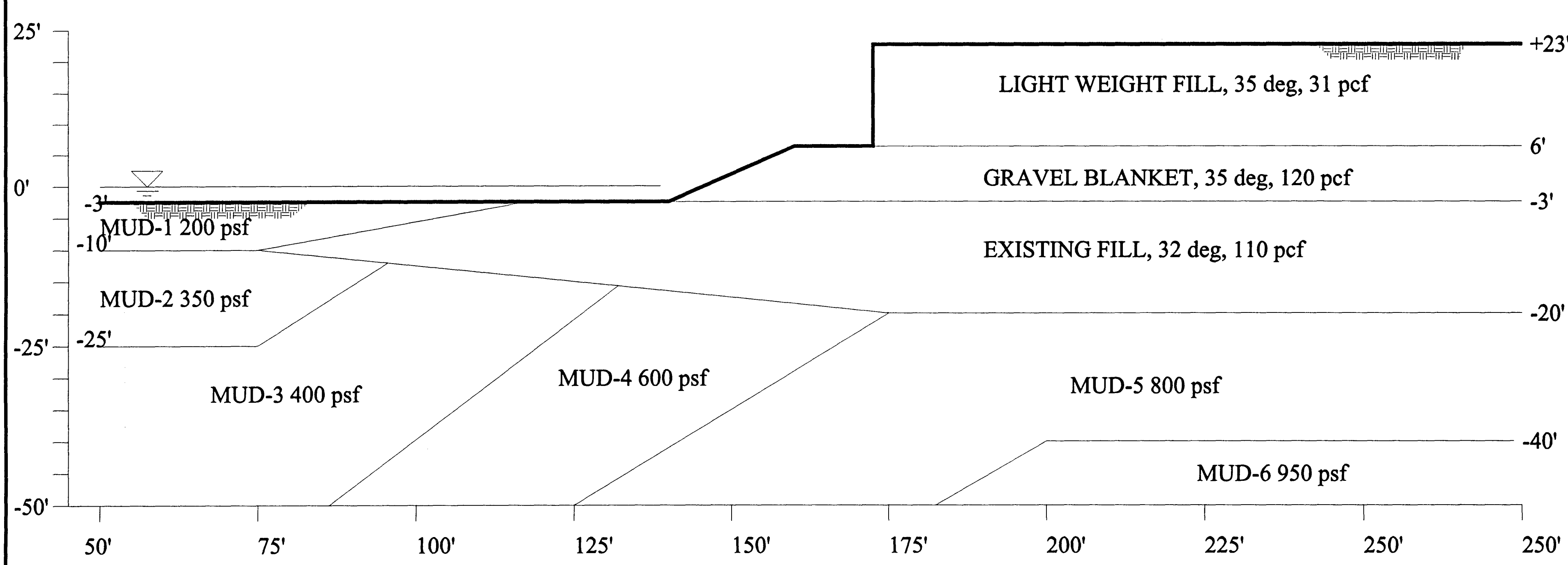

Permanent Displacement of Fill, BLOCK 3 (w/ light weight fill @ Sta. 86+34)
Unliquefied Strength
Disp (in.)
0
5
10
15
20
25
30
35
40
0
10
20
30
40
50
60


Liquefied Strength: 600 psf
Disp (in.)
0
5
10
15
20
25
0
10
20
30
40
50
60
70
80
Time (sec.)


Liquefied Strength: 800 psf
Disp (in.)
0
5
10
15
20
25
30
35
40
0
10
20
30
40
50
60
Time (sec.)

APPENDIX D

Appendix D
Station 86+34
Proposed Light Weight Fill, γ_{fill} = 31 pcf (Grade at El. +23')



Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	NEW S.F.- OAKLAND BAY BRIDGE Project No. 98-107 Date: 6-10-98	IDEALIZED SOIL PROFILE AT OAKLAND SHORE APPROACH STA. 86+34 Figure

TABLE 3 Summary of Pseudo Static Analyses for FILL (w/ light weight fill at Sta. 86+34)

Block	Block Size, Measured from Shore	Drained Strength (1)		600 psf (2)		800 psf (3)		1000 psf (4)	
		Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g	Static F.S.	Yield Acc., g
Block 1	25'	3.54	0.49	2.35	0.26	2.75	0.34	3.15	0.41
Block 2	50'	4.28	0.46	2.75	0.25	3.26	0.32	3.76	0.39
Block 3	100'	5.95	0.45	3.5	0.23	4.29	0.31	5.07	0.38

Note:
(1) Using drained shear strength of sand
(2) Using residual shear strength of sand in fill below watertable (Su=600 psf)
(3) Using residual shear strength of sand in fill below watertable (Su=800 psf)
(4) Using residual shear strength of sand in fill below watertable (Su=1000 psf)

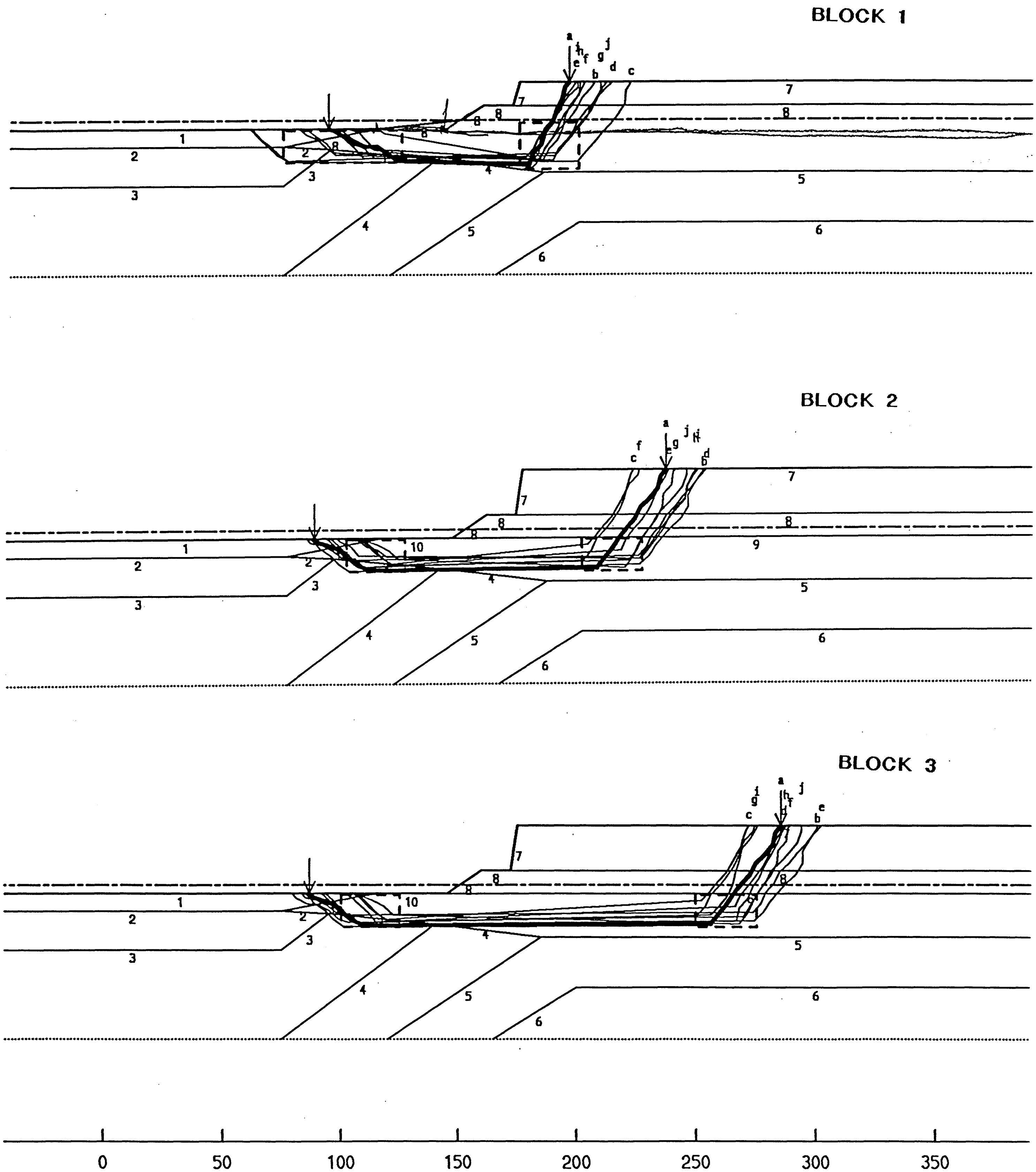
TABLE 4 Summary of Newmark Sliding Analyses for FILL (w/ light weight fill with gravel blanket at Sta. 86+34)

Ground Motion	Permanent Displacement, inches											
	Drained Strength (1)			Residual Strength of 600 psf (2)			Residual Strength of 800 psf (3)			Residual Strength of 1000 psf (4)		
	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3	Block 1	Block 2	Block 3
SET-1 (+)	0	0	0	5	4	4	1	1	1	0	0	0
SET-1 (-)	0	0	0	11	10	9	4	4	3	1	1	0
SET-2 (+)	0	1	0	10	9	9	4	4	3	2	2	1
SET-2 (-)	0	0	0	1	1	1	0	0	0	0	0	0
SET-3 (+)	0	0	0	9	8	8	4	4	3	1	1	1
SET-3 (-)	0	0	0	4	4	4	1	1	1	0	0	0
SET-4 (+)	4	5	3	25	23	21	15	14	12	9	9	7
SET-4 (-)	0	0	0	0	0	0	0	0	0	0	0	0
SET-5 (+)	0	0	0	1	1	1	0	0	0	0	0	0
SET-5 (-)	0	0	0	3	3	3	1	1	1	0	0	0
SET-6 (+)	0	0	0	0	0	0	0	0	0	0	0	0
SET-6 (-)	0	0	0	18	17	15	9	9	7	3	3	2
minimum	0	0	0	0	0	0	0	0	0	0	0	0
maximum	**4**	**5**	**3**	**25**	**23**	**21**	**15**	**14**	**12**	**9**	**9**	**7**
average	0	0	0	7	7	6	3	3	2	1	1	1

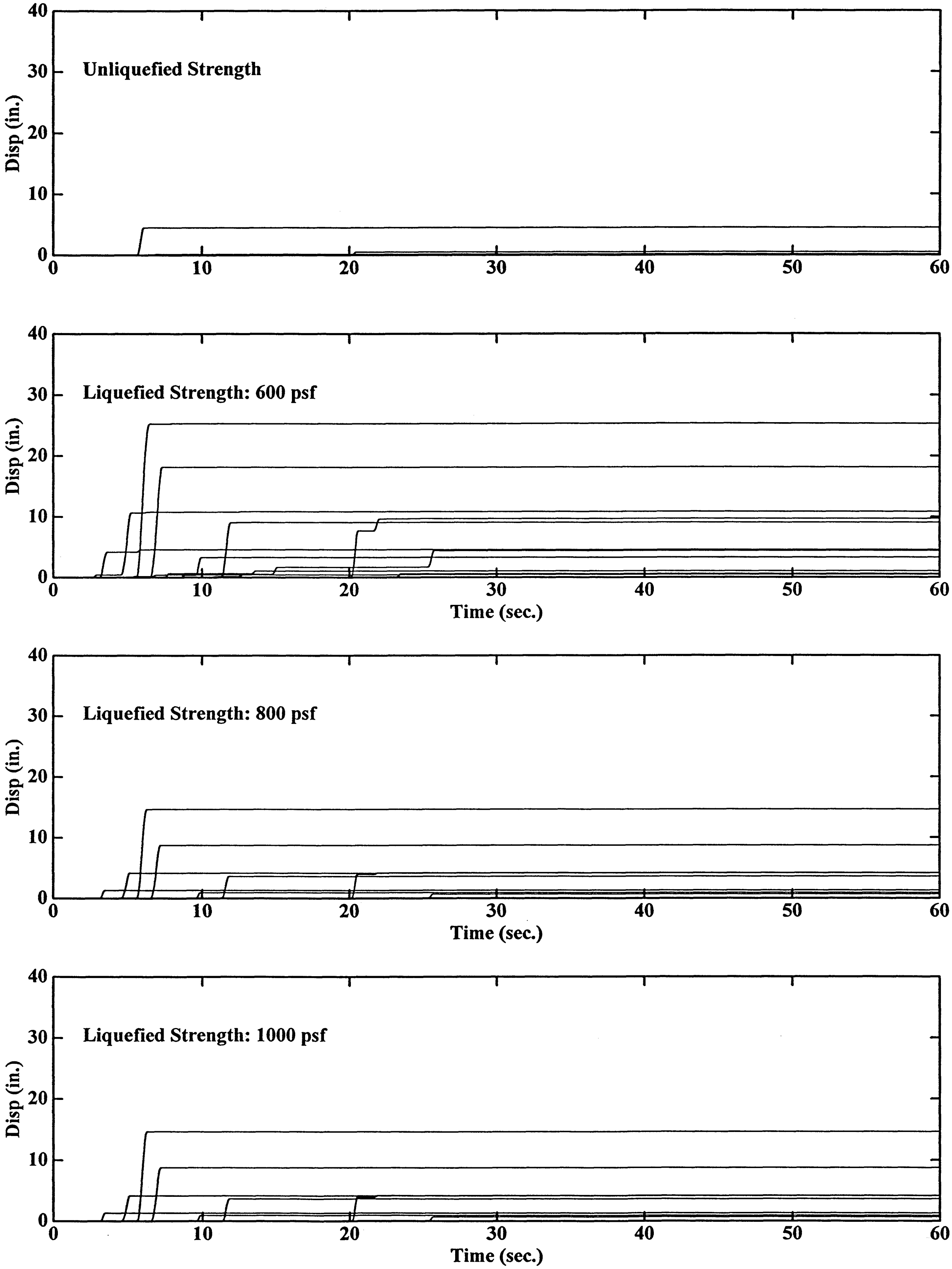
Note:
(1) Using constant drained shear strength of sand (unliquefied strength)
(2) Using constant residual shear strength of sand (liquefied strength of 600 psf)
(3) Using constant residual shear strength of sand (liquefied strength of 800 psf)
(4) Using constant residual shear strength of sand (liquefied strength of 1000 psf)

LATERAL SPREADING IN FILL @ STA 86-34

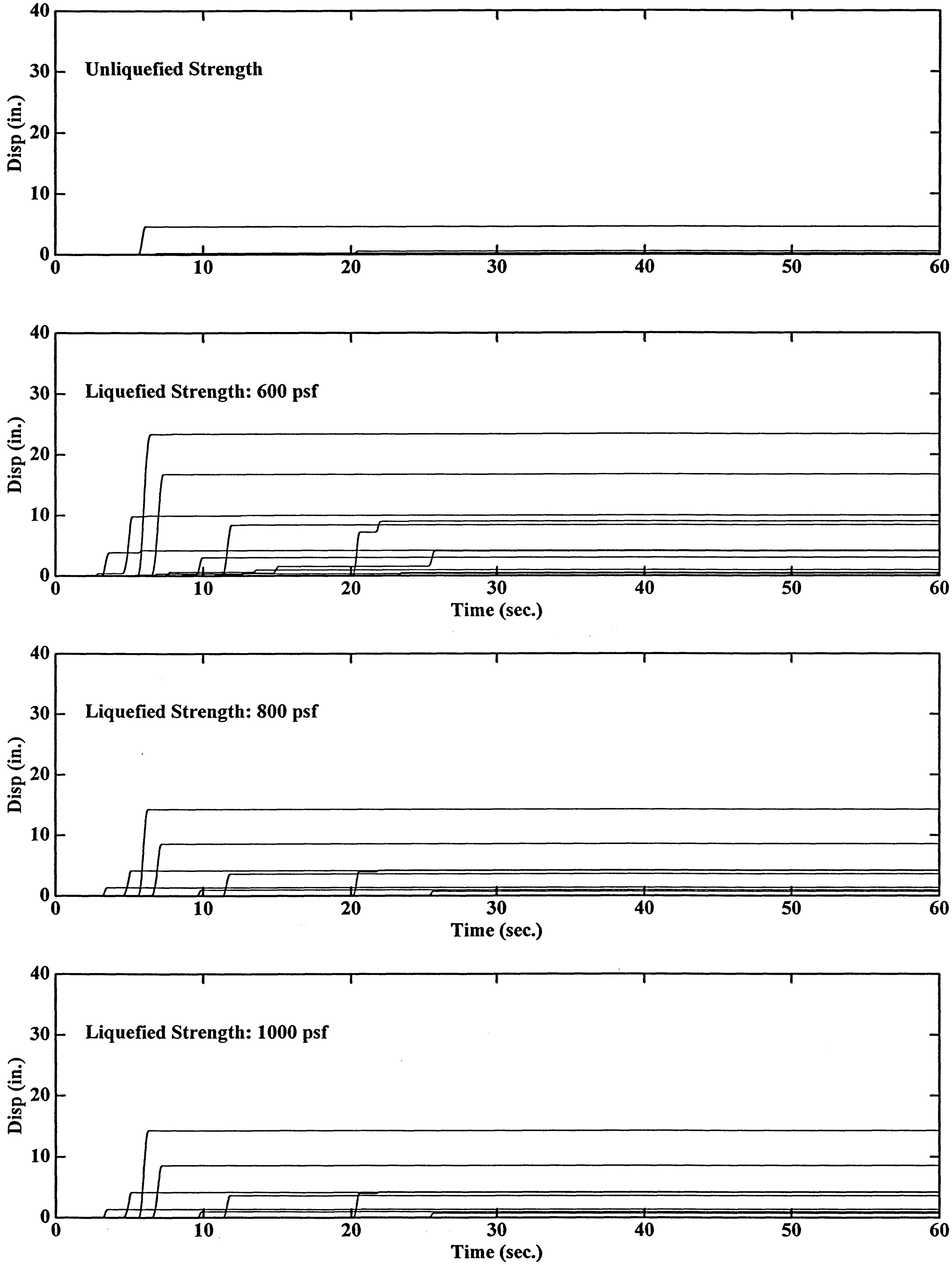
LIGHT WEIGHT FILL WITH GRAVEL BLANKET



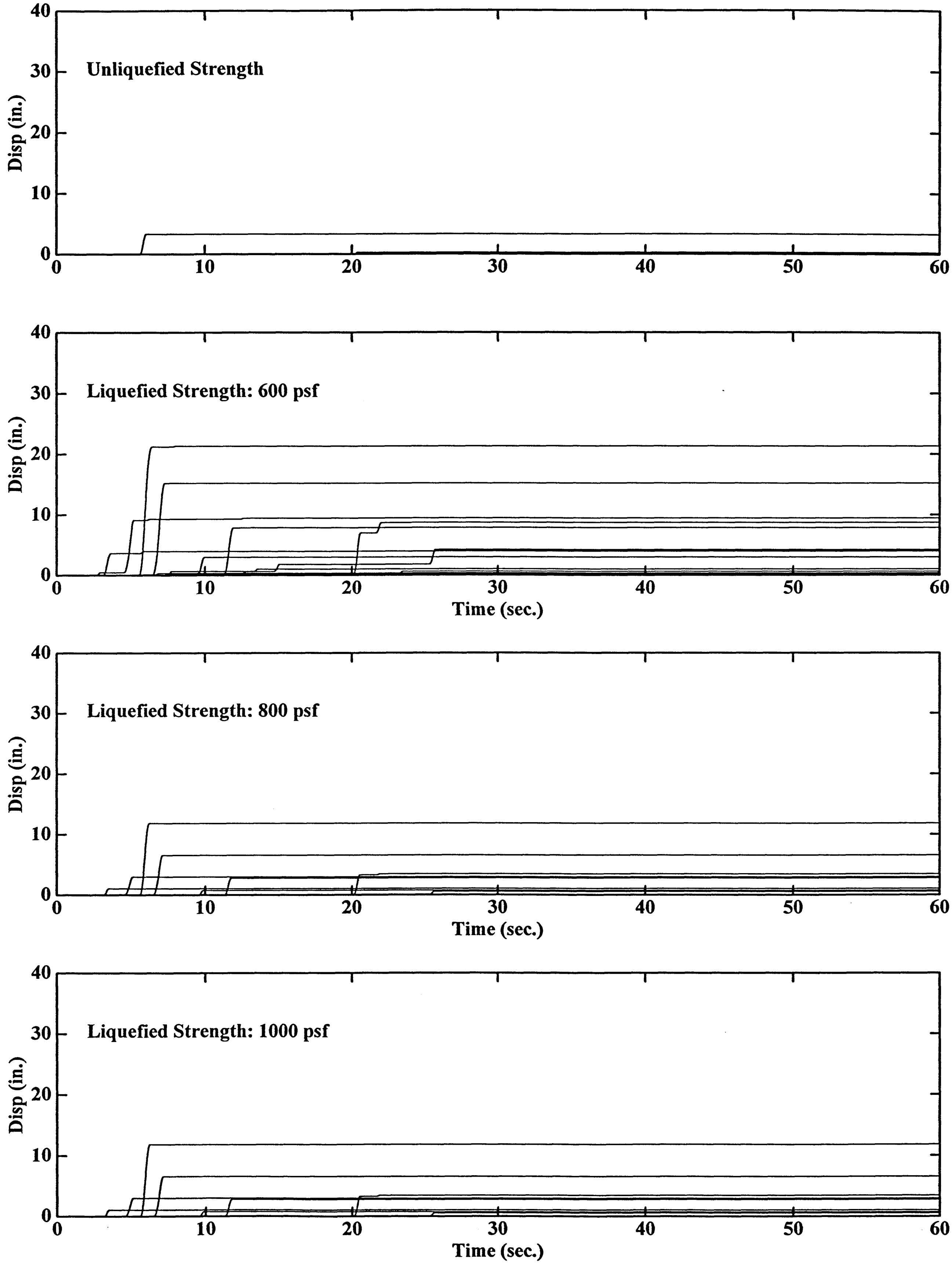
Permanent Displacement of Fill, BLOCK 1 @ Sta. 86+34 (Lt. Wt. Fill & Gravel Blanket)



Permanent Displacement of Fill, BLOCK 2 @ Sta. 86+34 (Lt. Wt. Fill & Gravel Blanket)



Permanent Displacement of Fill, BLOCK 3 @ Sta. 86+34 (Lt. Wt. Fill & Gravel Blanket)



PROJECT MEMORANDUM
FINDINGS FROM TRENCH AND PIT EXCAVATIONS ON OAKLAND MOLE (UPDATED)

April 15, 1999

Caltrans ESC, OSF
5900 Folsom Blvd, P.O. Box 19128
Sacramento, CA 95819-0128

To: Mr. Mark Willian, Contract Manager

From: Mike Kapuskar/EMI; Saverio Siciliano/EMI

Subject: Findings from Trench and Pit Excavations on Oakland Mole (updated)
SFOBB East Span Seismic Safety Project

Dear Mr. Willian:

On April 8 and 9, 1999, EMI performed two trench excavations on the Oakland Mole for the purpose of visual inspection and verification of the location, extent and composition of the pre-1950 rock dike presently buried beneath the existing northern slopes. This investigation was done as an extension of the work already performed for the Oakland Mole for this project. The existence, location and condition of the rock dike affects the design and construction of the proposed new SFOBB east approach structures or fills.

In addition, two pits were excavated per Caltrans' request in the beach areas at the north slopes. The purpose of these pits was to verify maximum unshored excavation depths and to determine whether removal of the (potentially liquefiable) beach sands was a feasible option.

This memorandum describes this investigation and presents our current findings. This information will be included in the Draft Site Characterization Report for the Oakland Mole. If you have any questions, please contact Mike Kapuskar or Saverio Siciliano, Earth Mechanics, Inc. (EMI), at (510) 633-5100 or (714) 751-3826.

1.0 TRENCH EXCAVATIONS

1.1 Description of Excavations

The locations of the trenches are shown on the attached plan. The locations were approximately in line with the visible northern slopes to the west and the east of the jughandles, reflecting the presumed location of a buried rock dike placed during the construction of part of the fill during the 1930's.

Trench 99T-1 was excavated on the western jughandle, at the north edge of pavement of the existing Caltrans maintenance road. Trench 99T-2 was cut inside the area circumscribed by the Caltrans road (the eastern jughandle), the site of presently proposed Abutment E30W. Both trenches were oriented in a north-south trend. The trenches were cut using a track-mounted excavator (Hyundai 290LC) with a 14.2-m (46.7 ft) reach and a 0.6-m (2-ft) wide bucket. This rig was selected to cut pits in the adjacent beach areas between the jughandles (see Section 2.0). The materials excavated were temporarily stockpiled on plastic sheeting to prevent cross-contamination of underlying soils.

The trenches were backfilled with the on-site soil and rock materials in lifts and compacted by tapping using the excavator bucket. Compaction led to some loss in total volume due to the loose consistency of original on-site soils, which was compensated for using soil borrowed from areas adjacent to the trench. At completion of the work, the original ground conditions was restored as well as possible.

To-scale trench cross sections are attached showing the approximate configuration of the cuts and the subsurface materials encountered.

1.2 Observations Trench 99T-1 on Western Jughandle

This trench was excavated in the morning of April 9 under fair weather conditions. The present ground surface at this location is at El. +3.0 m (+10 ft). Surficial material removed from above the rock dike was post-1950's debris material consisting of some moist soil with silt, gravel and cobbles, clay, brick, glass (bottles and light bulbs), wood. Other debris included demolition debris such as broken concrete, concrete reinforcement, metal wire and steel cable, a 0.9-m (3-ft) segment of metal sheet piling (probably from original key construction), a 1.5 cm (5-ft) segment of 460-mm (18") diameter pipe, and rubber floormats.

Rip-rap about 0.6 to 1.2-m (2 to 4-ft) in dimension was encountered at about 1.5 to 1.7 (5 to 5½ ft) depth below the ground surface at the south end of the trench. Individual pieces of rip-rap were removed and are shown on the photos.

The crest of the northern rock slope face was found at about 3 m (10 ft) north of the existing edge of pavement, and the slope had a gradient of approximately 1.4H:1V. Excavation proceeded down the rock face and resulted in substantial extension of the trench towards the north. The rock dike materials encountered consisted of many 300 to 600 mm (12" to 24") and fewer smaller rock pieces (see attached photos) to at least 2.6 m (8.5 ft) depth. Groundwater was encountered at this depth which matched the tide level El. of about +0.27 m (+0.9 ft) at the time of this excavation. Shoring was then installed to this depth. Further attempts to excavate below this depth resulted in substantial progressive caving in the trench segment adjacent to the shoring plates, increasing the trench width from 0.6 to 2.9 m (2 to 9.5 ft), and excavation was discontinued.

The trench was partially backfilled and then cut back to the south along the top of the dike in an attempt to locate the south face of the dike. Several large rock rip-rap boulders were exposed at 1.5 to 1.7 m (5' to 5'-6") depth. A buried utility pipe was encountered at a depth of 0.76 m (30") below ground running 1.1 m (3'-6") parallel to the edge of pavement of the Caltrans road. Manual excavation proceeded carefully below the pipe to expose more of the rip-rap boulders at the same elevation. One large 1.2-m (4-ft) boulder was removed from the top of dike. The underlying rock predominantly consisted of 300 to 600 mm (12" to 24") cobbles with fewer smaller-size cobble fragments and pebbles. The attached photos show the sizes encountered.

1.3 Observations in Trench 99T-2 on Eastern Jughandle

This trench was cut in the morning of April 8 down to a depth of about 3.65 m (12 ft). Weather conditions were light to moderate rain, few minutes of hail, and icy winds, following three prior days of moderate to heavy rain. The present ground surface elevation at this location varies from +3.1 to 3.7 m. The total length of the trench was 12 m, spanning most of the ground inside the road turn-around loop.

The soils excavated above the rock dike consisted of moist silty sand material with small to pebble-size gravel. The top of the buried dike was encountered at the south side of the trench merely 0.7 m (28") depth below existing ground. The northern crest of this dike was found at 9.1 m (30 ft) north of the edge of pavement. The slope had a gradient of approximately 1.4H:1V. The surface of the dike consisted of cobbles 300 to 600 mm (12" to 24") in size and smaller pebble-size gravel pieces.

The north face of the buried dike was exposed and individual cobbles removed. The trench was extended towards the north as the excavation proceeded down the dike's face. At about 12-ft depth below existing grade, groundwater was encountered, which was about 0.5 m (1'-7") depth below the adjacent tide level El. of about +0 m MSL at the time of the excavation. At this depth, sand mixed with some blue-olive green mud was encountered and the excavator's bucket did not find any more rock, indicating the base of buried rock (at least at the north side) was reached at about 3.65 m (12 ft) depth. The trench width increased from 0.6 m to more than 3 m wide due to progressive caving of saturated soils at the bottom of the west and east trench walls and deeper excavation was discontinued.

To locate the back side of the dike, the excavation was then progressively cut back to the south along the top of the dike and was stopped at a buried electrical utility pipe running about 0.9 m (3 ft) parallel to the edge of pavement of the Caltrans road. The surficial dike material was the same as that found along the north face.

2.0 PIT EXCAVATIONS

2.1 Description of Excavations

Per Caltrans request, two pits were excavated in the tidal flat beach area between the two jughandles. The purpose of the pits was to verify how deep the cut would stay open unshored, and whether removal of the (potentially liquefiable) beach sands was a feasible option. The locations are shown on the attached plan. The excavations were made by the long-reach excavator from on-shore per CE permit requirements.

2.2 Observations and Findings in Pit 99P-1 near Western Jughandle

This pit was excavated from the Western jughandle upslope near the site of the all-terrain CPT 99C-3 previously completed by Fugro (see photos). The excavation was performed around noon to 1:15pm on April 9 under fair weather and near-low tide conditions. The soils excavated consisted of gray sands with occasional trash (plastic containers, bottles, metal cable) and interbedded below 0.3 m depth with some Young Bay Mud. Two cohesive lumps of clay and sand approximately 0.5 m (18") in maximum dimension were removed from the bottom of the pit.

After further excavation, water collected at the bottom of the pit and the pit walls caved back in. At this stage, the pit was oval in shape with a larger dimension of about 4.5 m (15 ft). Tension cracks formed about 0.3 to 0.9 m (1 to 3 ft) back from the edge of the pit.

The maximum excavation depth at the center of the pit was measured using a plastic rod (with the tip floating inside the bottom sand-clay unit) to be about 1.7 m (5'-6"). The water collecting in the pit was pumped from the pit using a 150 mm (6") diameter trash pump and discharged back into the bay as allowed by the CE permit. The purpose of the pumping test was to observe the stability of the drained sands of the open cut (simulating the effect of lower tide levels). The excavation depth at the center of the pit, with the tip floating again pushed inside the bottom sand-clay unit, was measured again after pumping to be about 2.2 m (7'-3"). The pit walls flowed back as the water level rose to back to the previous level encountered before pumping to about 1.5 m (5 ft depth).

The pit was backfilled with the soils removed. The location was revisited the next day and there were no evident signs of this location after it had been washed over after one high tide cycle.

2.3 Observations and Findings in Pit 99P-2 near Eastern Jughandle

This pit was excavated from the eastern jughandle upslope and the location is at the site of the all-terrain CPT 99C-5 previously completed by Fugro. The excavation was performed around 9:30 to 10:00am on April 8 under fair weather and near-low tide conditions. The soils removed consisted of gray beach sands interbedded below 4 ft depth with some Young Bay Mud. After further excavation to about 2.1 m (7 ft), the pit walls caved back in. The pit at this stage was circular in shape with a diameter of about 5.5 m (18 ft), and tension cracks formed about 0.3 to 0.6 m (1 to 2 ft) back from the circumference of the pit.

Water collected at the bottom of the pit at a measured depth of 1.3 m (4'-3"). The maximum excavation depth was measured using a plastic rod (with the tip floating inside the bottom sand-clay unit) to be about 1.7 m (5'-6") at the center of the pit.

The pit was backfilled with the soils removed. This location was also revisited the next day and it was found that it had also been washed over after one high tide cycle.

3.0 SUMMARY OF FINDINGS

The former rock dikes placed before the 1950's were left in place in the two jughandle areas. Our observations on the configuration and composition of the buried dikes are schematically summarized in Plate 2 and compared to a "typical" configuration shown on historical records as included with EMI's feasibility report (1998). Based on this comparison, the following findings are made.

3.1 Rock Dike Configuration

The dikes are approximately in line with the visible slopes west and east of the jughandles. The dike configuration at the eastern jughandle is similar to that shown in construction records, with differences in elevation of only about 0.3 m (1 ft). This indicates that the dike materials in this area do not appear to have not settled relative to the Mole sand fills, but rather have settled since construction together with the fills. Based on recent adjacent borehole information, up to about 4 to 5.5 m of sand fills underlie the dike in either of the two areas.

Depth to top of dike. There is considerable variation in the depth of the dike below ground. At the western jughandle, the dike is found about 0.9 m (3 ft) deeper below existing ground, (which is only slightly lower than at the eastern jughandle). Based on the limited investigation, it can be extrapolated that rip-rap and rock dike materials can be expected within the upper 1.8 m (6 ft) of the present Mole fills.

Height. The thickness of the rock dike was measured to be about 3 m (10 ft) at the eastern jughandle (which corresponds with construction records) and in excess of 2.4 m (8 ft) in the western jughandle. Based on these limited measurements and one construction record, it is extrapolated that the bottom of the dike is expected at 3.6 to 4.9 m (12 to 16 ft) depth below existing grade.

Gradients. This dikes are roughly shaped. The north dike face has a gradient of about 1.5:1, somewhat steeper than the north slopes presently visible to the west and east of the jughandles. The south face remains unconfirmed as those zones were blocked in both trenches by an underground high-voltage electrical utility line running parallel to the existing Caltrans maintenance roadway and could not be excavated safely with the bucket. The back face slopes back below the existing Caltrans maintenance road and is likely to have a steeper gradient, such as 1:1 as indicated on the construction record.

3.2 Composition of Dike Material

Face rock. The dike is typically covered at the top with angular and rounded rock rip-rap material about 0.6 to 1.2 m (2 to 4 ft) in size, which may have been removed in places such as the eastern jughandle to allow new fills to be added and the surface to be graded. Less or no large rip-rap was found along the north face.

No rip-rap is expected at the steeper south face where they would be less likely to stay in place and finer-grained rock and sand filter material was used to retain the hydraulically pumped sandy Mole fills behind them. Some larger-size material may have been removed at the top and north face at either jughandle (in the 1950's and more recently during construction of the turn-around fills) to allow the jughandle fills to be added and the surface to be graded. Break-up or removal of the face rock is necessary for proposed pilecap construction or installation of piles.

Core material. The dike core materials exposed and excavated below the rip-rap were composed of quarry-run size rounded and angular rock cobbles predominantly 300 to 600 mm (12" to 24") in size, with smaller broken cobbles and fill soils filling the voids.

3.3 Jughandle Fills

Between 1934 and the late 1940s, the mudflat area to the north and northwest of the current toll plaza was progressively filled.

Western jughandle. By 1957, the northern edge of the Mole near the western tip had been filled out to the north to form a new "pad" of approximately 30 by 60 meters (100 by 200 ft). The western fill jughandle is in an area where early photos from 1936 show a docking pier. By 1963, the portion of the western end of the Mole beneath the SFOBB bridge had been extended to the west by about 90 m (300 ft).

The western jughandle north of the present Caltrans fenceline consists of non-engineered loose fill soils (placed in the 1950's) with varying amounts of organic material, gravel and cobbles, and abundant unconsolidated debris material. The materials on top of the buried dike south of the fenceline consisted of soils with roots and gravel. Large-size debris may be encountered in during excavation of these fills, and break-up and/or removal is required for proposed pilecap construction or installation of deep foundations. Environmental soil testing was not performed. Caving occurred at depths as small as 1.2 m (4 ft) and shoring is required.

Eastern jughandle. The eastern jughandle at proposed "W" Station 88+30 presently serves as a turn-around area for the present Caltrans maintenance road and is believed to have been created in the late 1960's. This area is built on loose to medium dense granular soils with pebble and cobble-size gravel and some debris material. A lens of baymud was encountered at 3.65 m (12 ft) depth in the sand fills. Only one large piece of rip-rap was encountered during excavation. Some break-up or removal of this face rock may be necessary for proposed pilecap construction or installation of piles. Some caving occurred below 1.5-m (5-ft) depth and shoring is required for deeper excavations.

3.4 Pits in Tidal Flats

The materials excavated from the pits consisted of medium to coarse clean to silty sands with some interbeds of Young Bay mud at the bottom. The maximum achievable depth of both pits excavations was about 1.5 m (5 ft). The maximum cut depth is controlled by the tide water level and by the seepage of water from the Mole fill slope through coarser seams of sand exposed during pit excavation.

Attachments:

Plate 1, Plan of trench and pit locations
Plate 2, Comparison of dike configurations
Log for Trench 99T-1
Log for Trench 99T-2

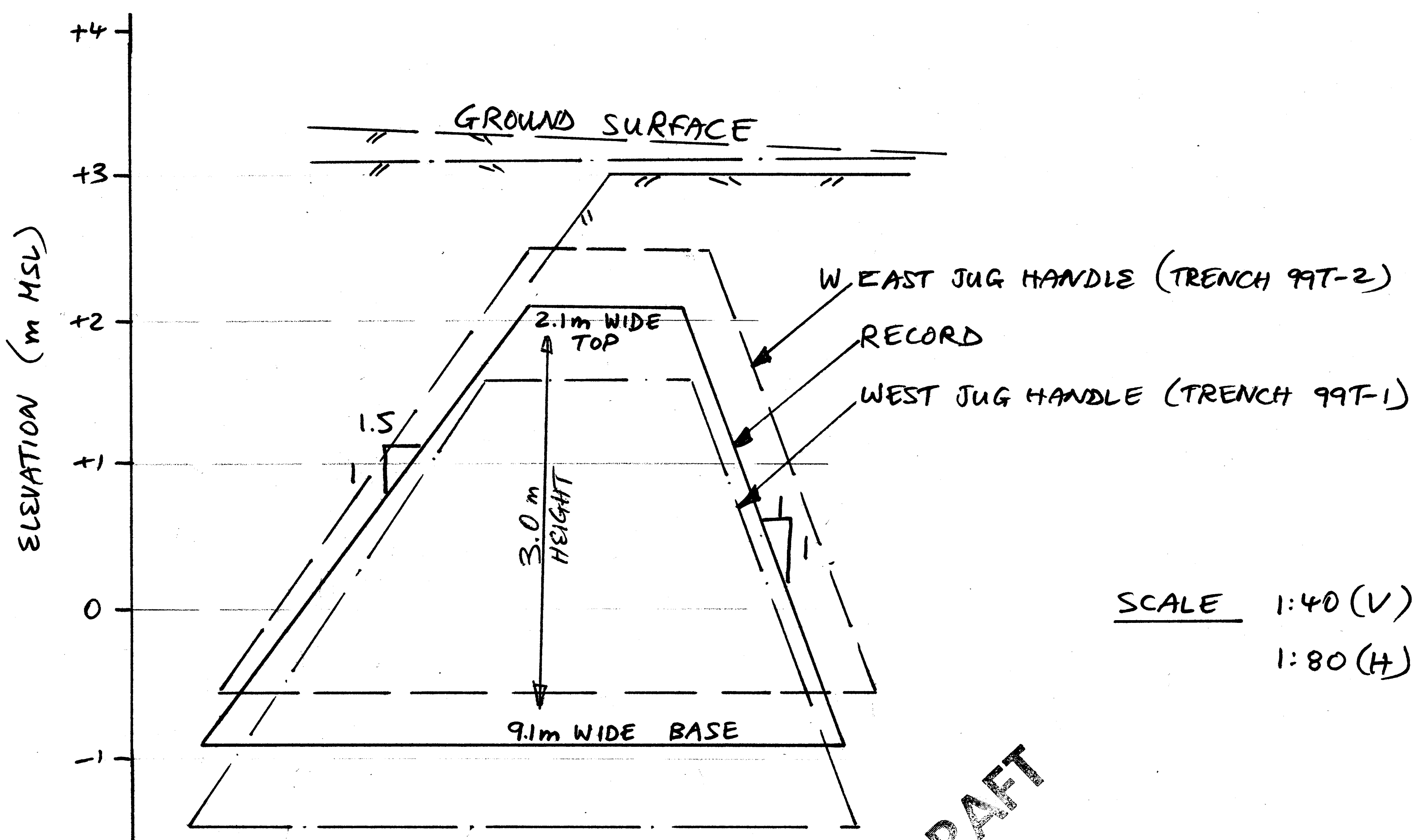
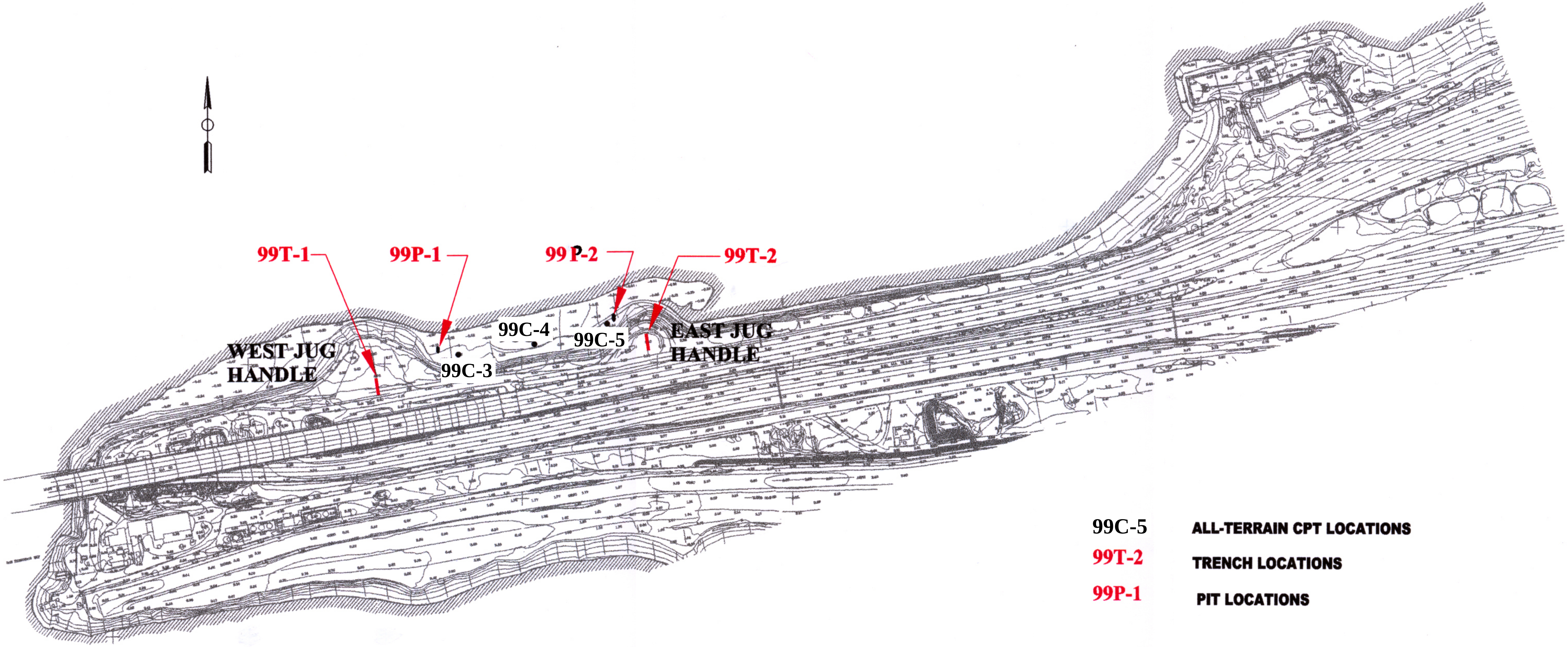


PLATE 2 – COMPARISON OF ROCK DIKE CONFIGURATIONS.



99C-5 ALL-TERRAIN CPT LOCATIONS

99T-2 TRENCH LOCATIONS

99P-1 PIT LOCATIONS

PLAN
1:3000

50 m 25 0 25 50 m

SFOBB EAST SPAN SEISMIC SAFETY PROJECT

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering
7700 Edgewater Drive, Suite 848
Oakland, CA 94621
(510) 633-5100

EMI Project: 98-145 | DATE: 4/12/99

TRENCH LOG

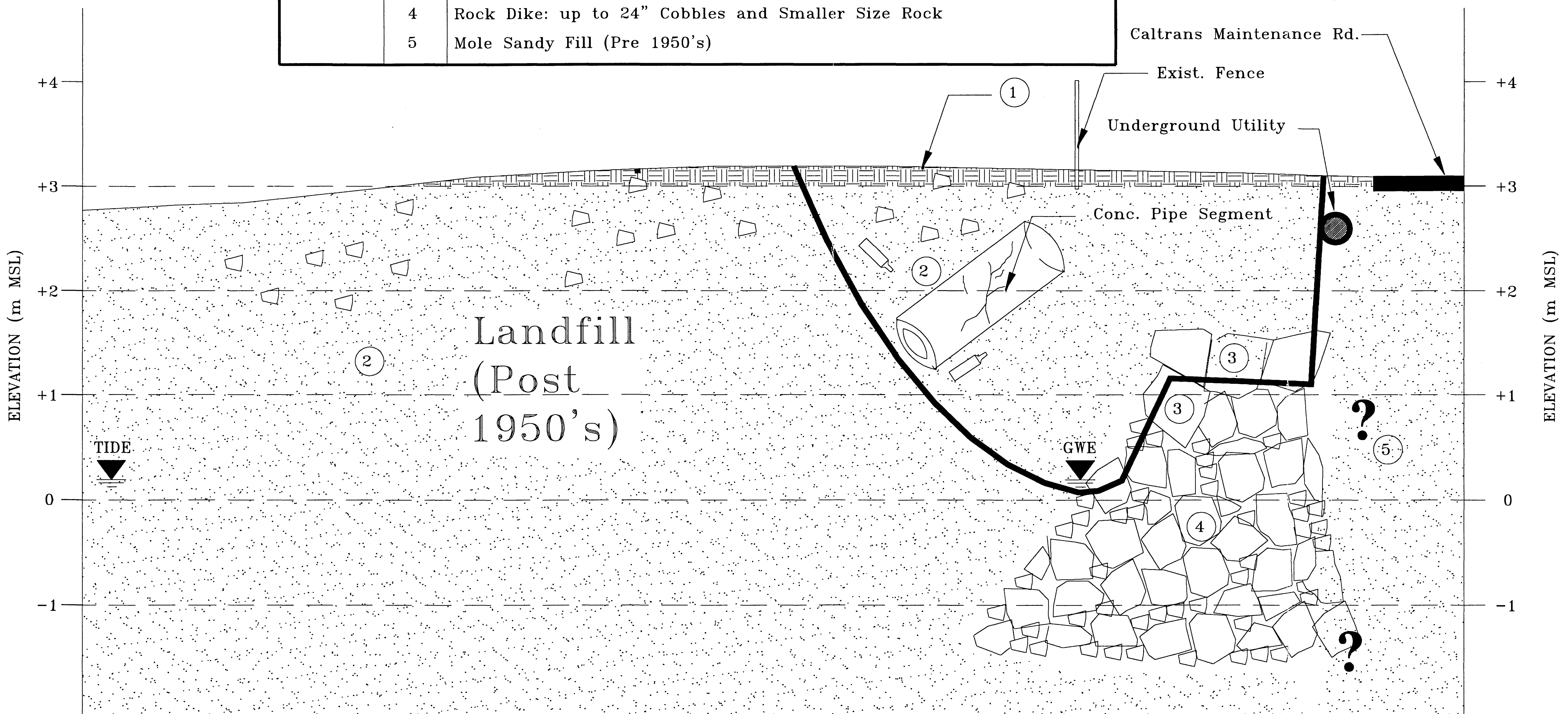
Trench No. 99T-1		Surface Elev.: +3.1m MSL	Slope Level
Location: W Jug-handle (Oakland Mole)		Trench Depth: 3.6 m	Trend N-S
Logged By: SS/MK	Date: 09 APR 1999	Equipment: Hyundai 290LC	Time: 07:30-12:00

GEOLOGIC ATTITUDES	GEOL. UNIT	DESCRIPTION
	1	Tree chips ground cover/vegetation
	2	Sand: Brown, Moist, Dense, Fine Grained with Few Pebbles and Cobbles and Landfill Debris (Post 1950's).
	3	Rock Dike: Large Angular Boulders ±3' in Size.
	4	Rock Dike: up to 24" Cobbles and Smaller Size Rock
	5	Mole Sandy Fill (Pre 1950's)

1m
0 2m

N

Note: GWE Changes with Time According to Tide . Elev. Shown Refers Only to Time of Ecavation.



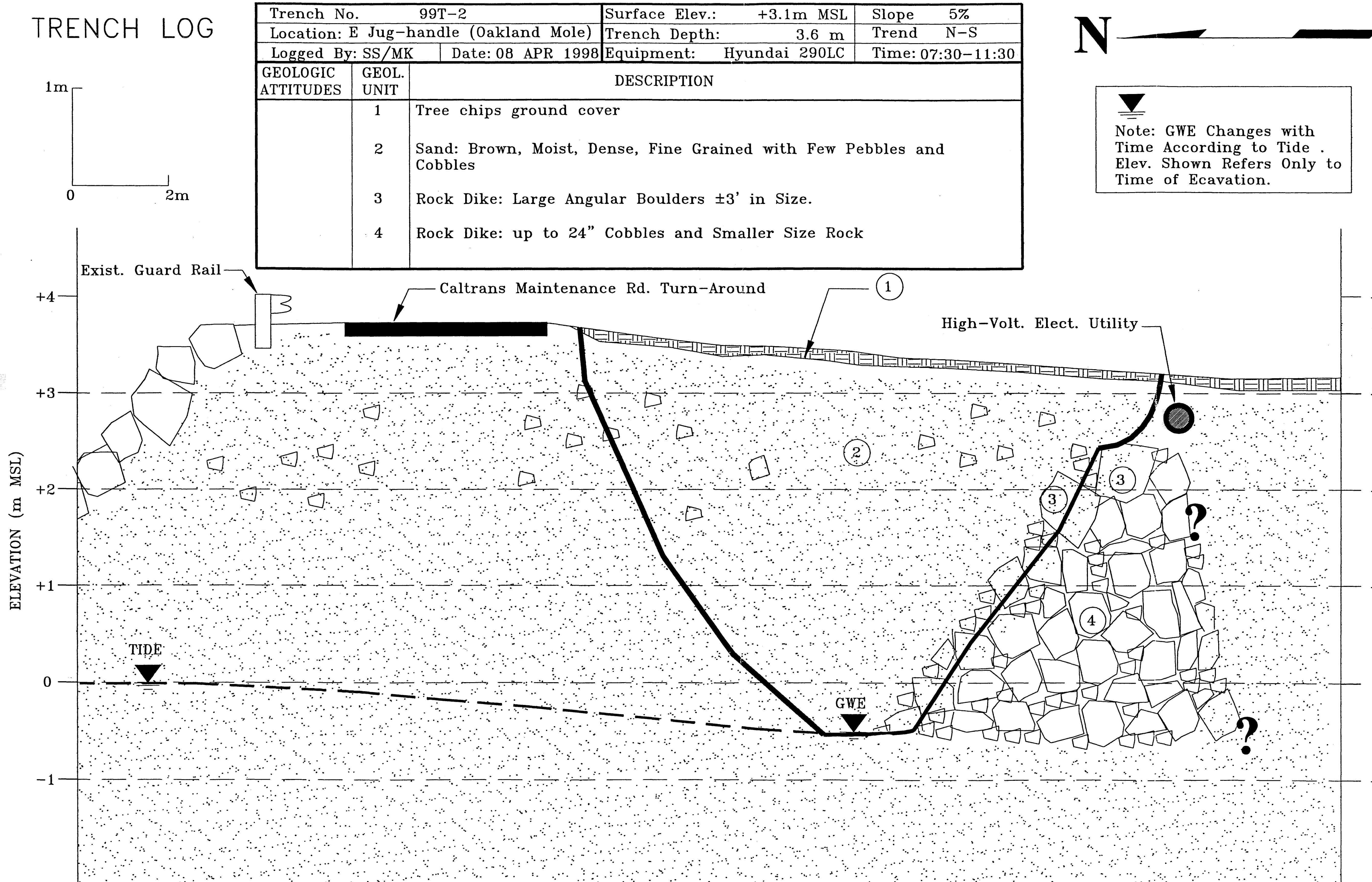
Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	HORIZONTAL SCALE: 1:80 VERTICAL SCALE: 1:40	SFOBB EAST SPAN SEISMIC SAFETY PROJECT Project No. 98-145 Date: 4/12/99	LOG OF EXCAVATION, TRENCH 99T-1, WEST JUG-HANDLE, OAKLAND MOLE (looking East)

TRENCH LOG

Trench No. 99T-2		Surface Elev.: +3.1m MSL	Slope 5%
Location: E Jug-handle (Oakland Mole)		Trench Depth: 3.6 m	Trend N-S
Logged By: SS/MK	Date: 08 APR 1998	Equipment: Hyundai 290LC	Time: 07:30-11:30

GEOLOGIC ATTITUDES	GEOL. UNIT	DESCRIPTION
	1	Tree chips ground cover
	2	Sand: Brown, Moist, Dense, Fine Grained with Few Pebbles and Cobbles
	3	Rock Dike: Large Angular Boulders ±3' in Size.
	4	Rock Dike: up to 24" Cobbles and Smaller Size Rock

Note: GWE Changes with Time According to Tide . Elev. Shown Refers Only to Time of Ecavation.



Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

HORIZONTAL SCALE: 1:80
VERTICAL SCALE: 1:40

SFOBB EAST SPAN SEISMIC SAFETY PROJECT

Project No. 98-145 | Date: 4/12/99

LOG OF EXCAVATION, TRENCH 99T-2, EAST JUG-HANDLE, OAKLAND MOLE (looking East)

View of north slope between two jug handles (Looking E)

Panoramic view of area between two jug handles, 1 of 9 (Looking E)

Panoramic view of area between two jug handles, 2 of 9 (Looking S)

Panoramic view of area between two jug handles, 3 of 9 (Looking S)

Panoramic view of area between two jug handles, 4 of 9 (Looking S)

Panoramic view of area between two jug handles, 5 of 9 (Looking S)

Panoramic view of area between two jug handles, 6 of 9 (Looking S)

Panoramic view of area between two jug handles, 7 of 9 (Looking S)

Panoramic view of area between two jug handles, 8 of 9 (Looking SW)

Panoramic view of area between two jug handles, 9 of 9 (Looking W)

Older (pre-1950's) rip-rap on north slope between jug handles (Looking S)

Newer rip-rap (post-1950's) on west jug handle

Trench 99T-1 Before excavation (Looking N)

Trench 99T-1 During initial excavation (Looking N)

West jug handle (Looking NE)

Trench 99T-1 Location on west jug handle (Looking NE)

Trench 99T-1 Buried rock dike (Looking S)

Trench 99T-1 Dike material from north face of buried rock dike

PHOTO SHEET
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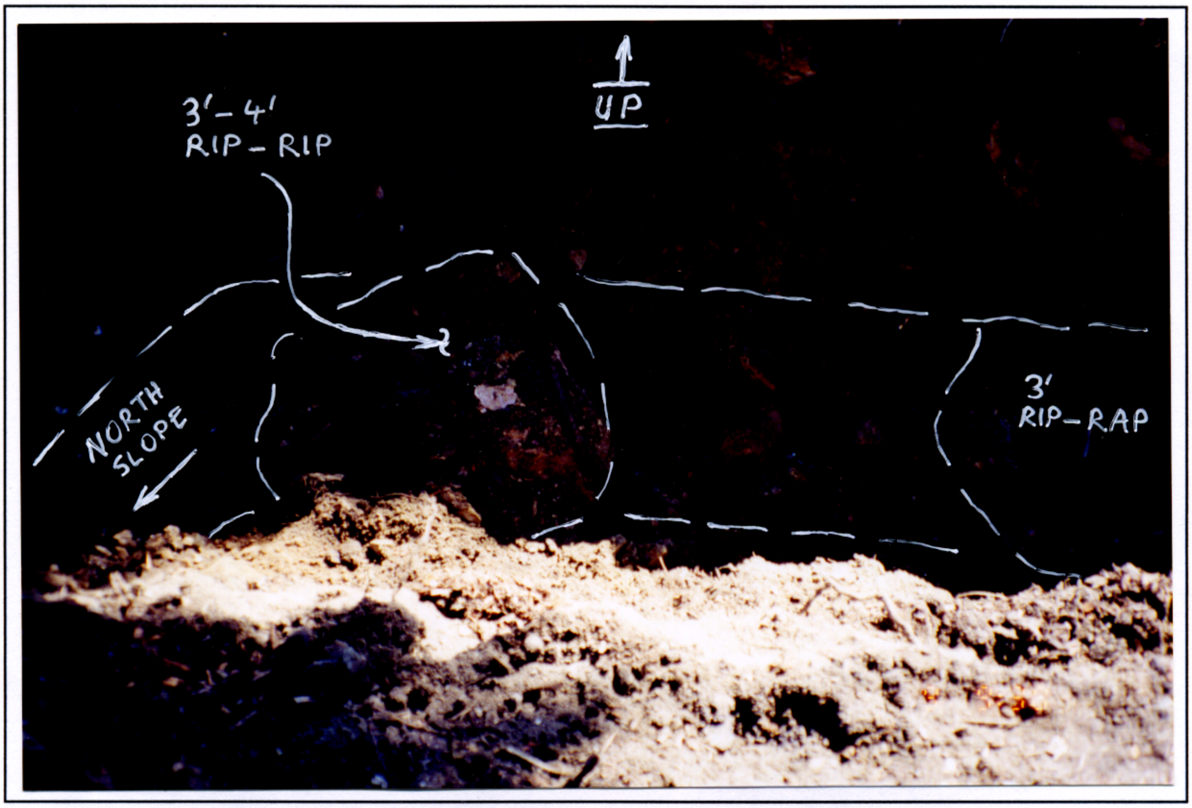
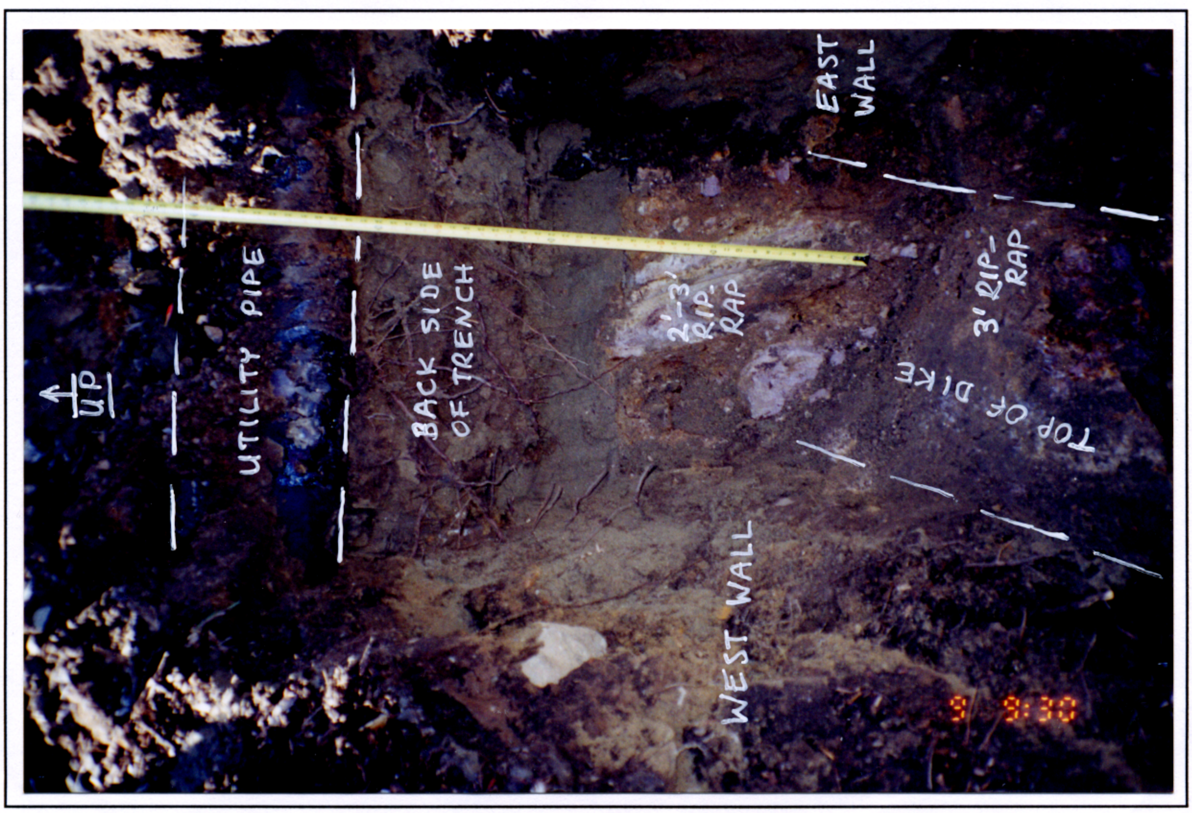
EMI PROJECT NO. 98-145

Trench 99T-1 Plywood shoring installation

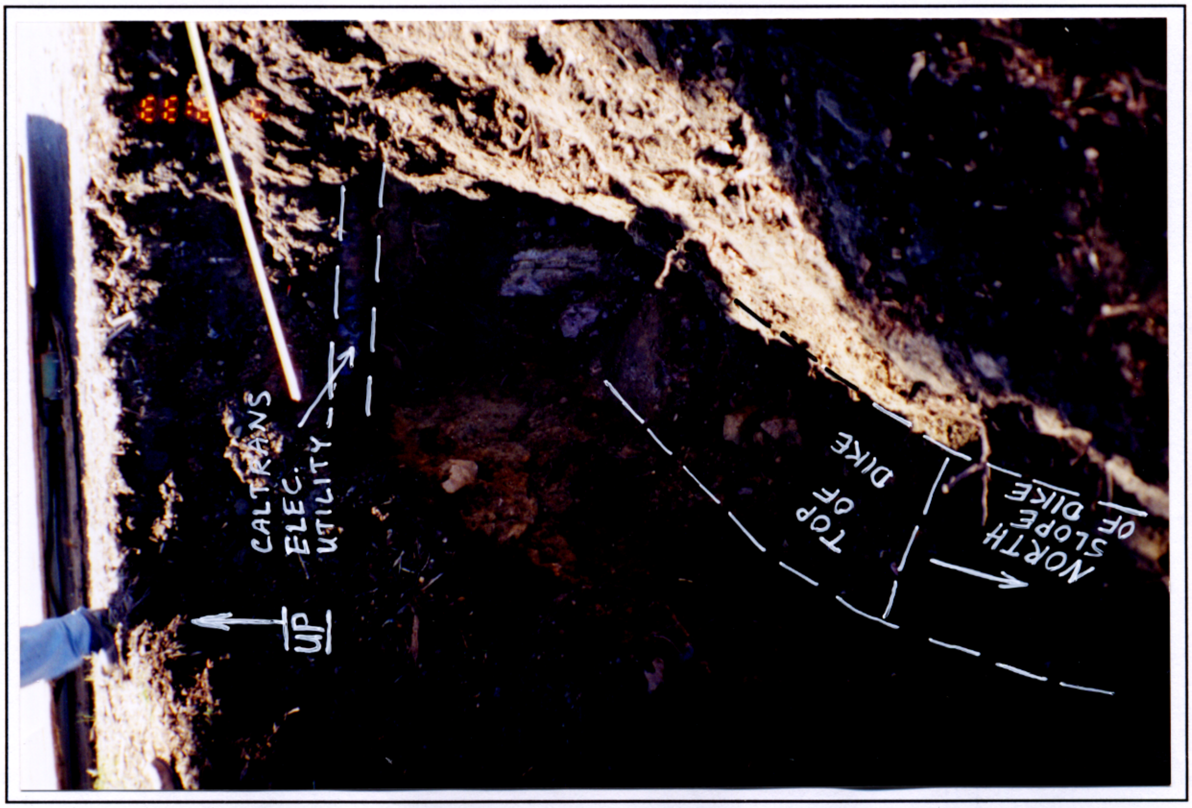
Shoring in-place on trench 99T-1 (Looking N)



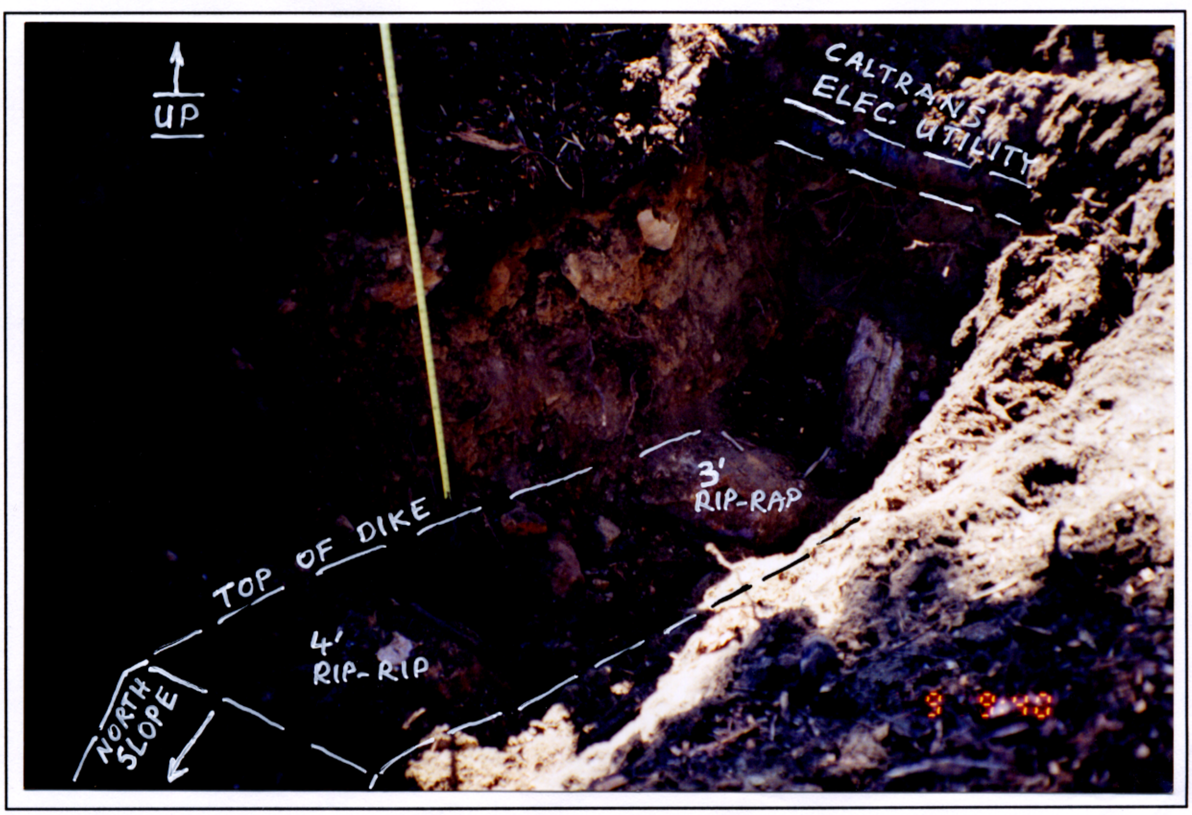
Trench 99T-1 Close up of previous photo showing rip-rap size



Trench 99T-1 Close up of south side of trench



Trench 99T-1 Top and face of buried rock dike (Looking S)



Trench 99T-1 Close up of previous photo showing rip-rap size

Trench 99T-1 Rock dike material

Trench 99T-1 Rock dike material

Trench 99T-1 One large (4') and some smaller boulders of rip-rap from top of buried dike

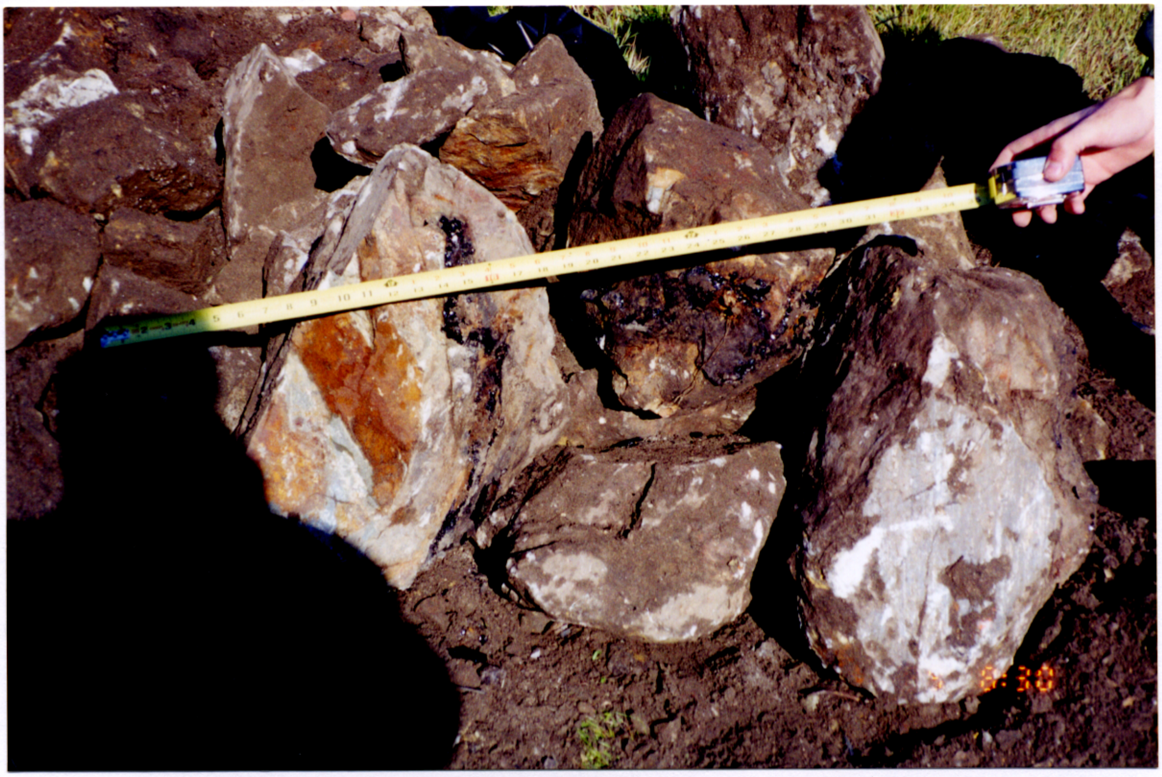
Trench 99T-1 One 3' to 4' boulder from north face of buried rock dike

Trench 99T-1 Excavated materials

Trench 99T-1 Debris material and soil from above buried rock dike

Trench 99T-1 Rip-rap from face and top of buried rock dike

Trench 99T-1 Rip-rap from top of buried rock dike



Trench 99T-1 Rock dike material

Trench 99T-1 Rock dike material

PHOTO SHEET

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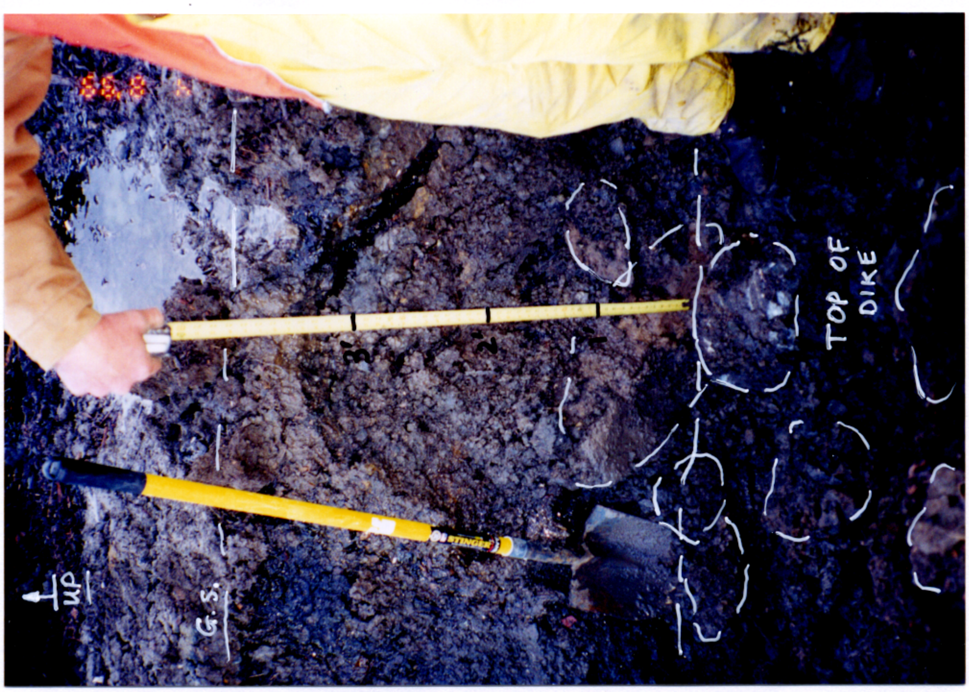
EMI PROJECT NO. 98-145

Trench 99T-2 (Looking SW)

Trench 99T-2 (Looking E)

PHOTO SHEET
SFOBB EAST SPAN SEISMIC SAFETY PROJECT
EMI PROJECT NO. 98-145

Trench 99T-2 (Looking S)

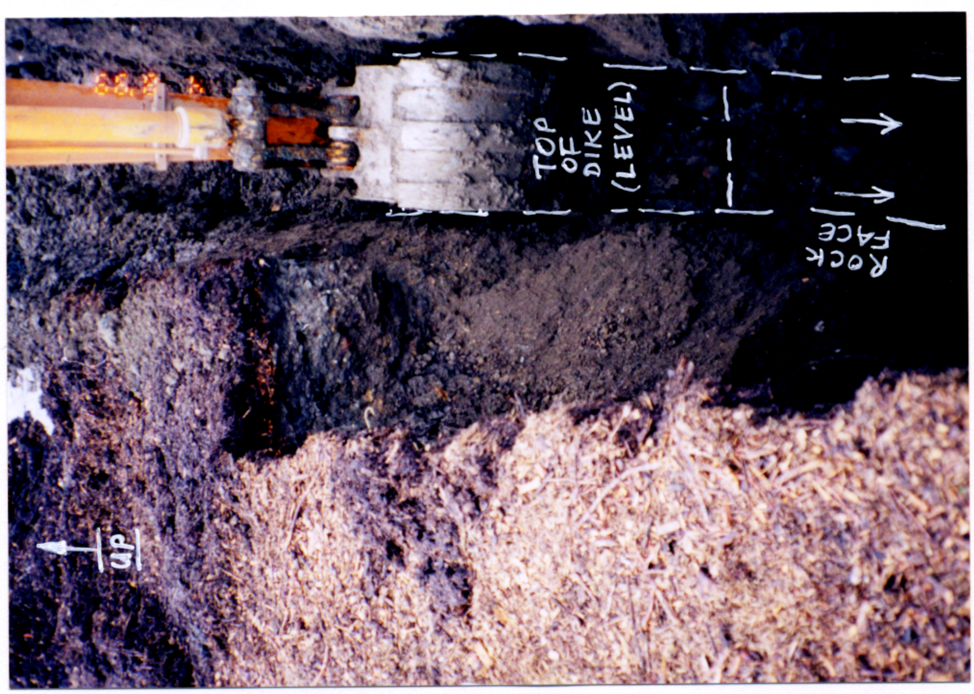


Trench 99T-2 Top of buried dike (Looking E)

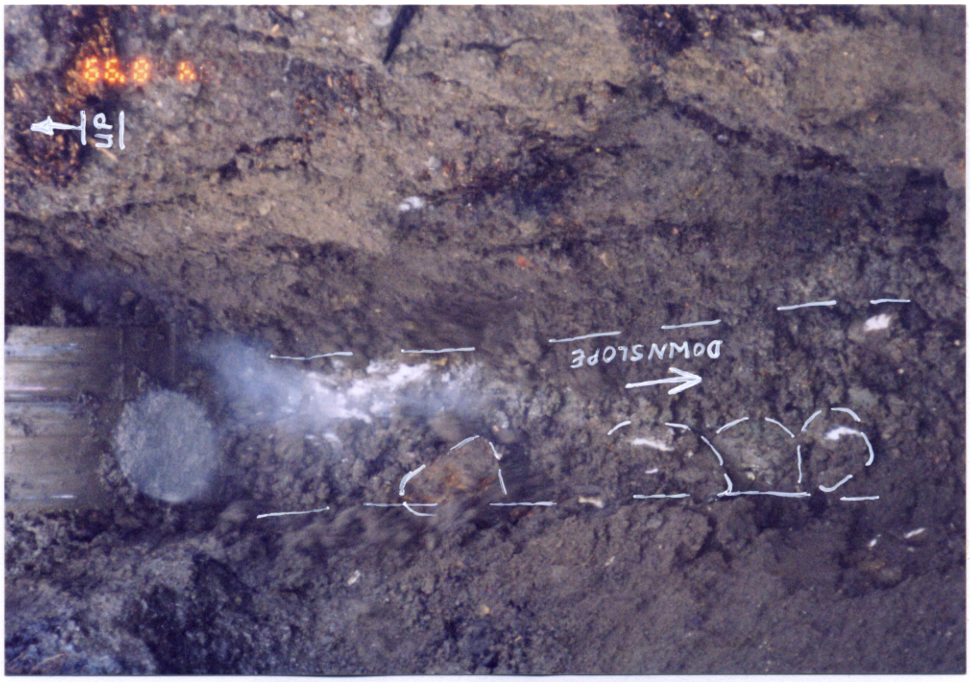
Trench 99T-2 Top of dike (Looking N)

Trench 99T-2 Typical dike material from north face and top of rock dike

Trench 99T-2 Surficial jug handle fill soils



Trench 99T-2 Top and north-facing slope of dike (Looking S)



Trench 99T-2 North-facing slope of dike (Looking S)

Trench 99T-2 North-facing slope of dike (Looking down)

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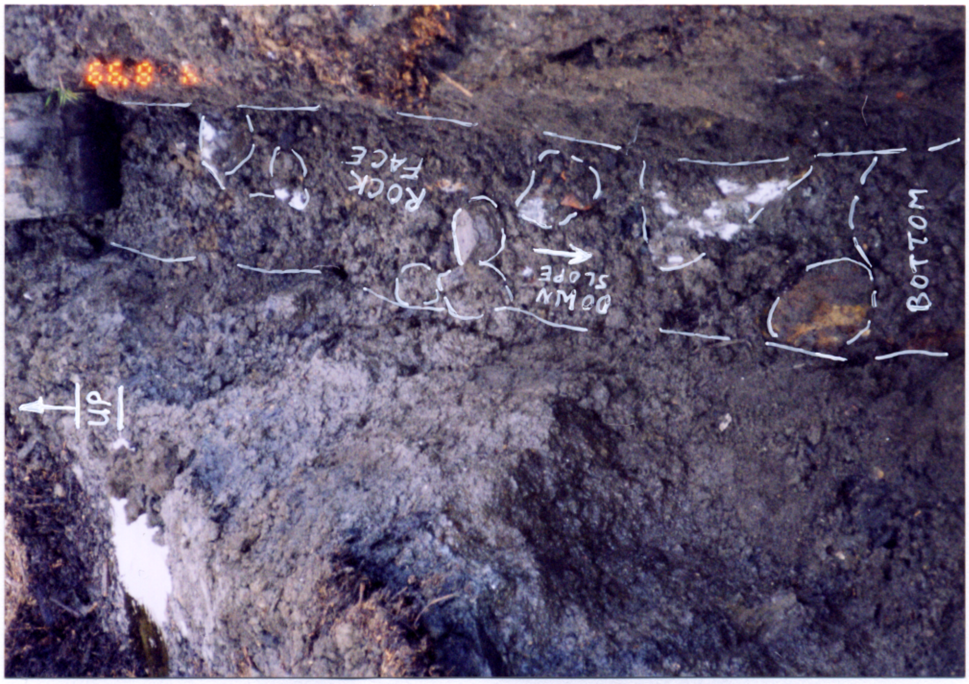
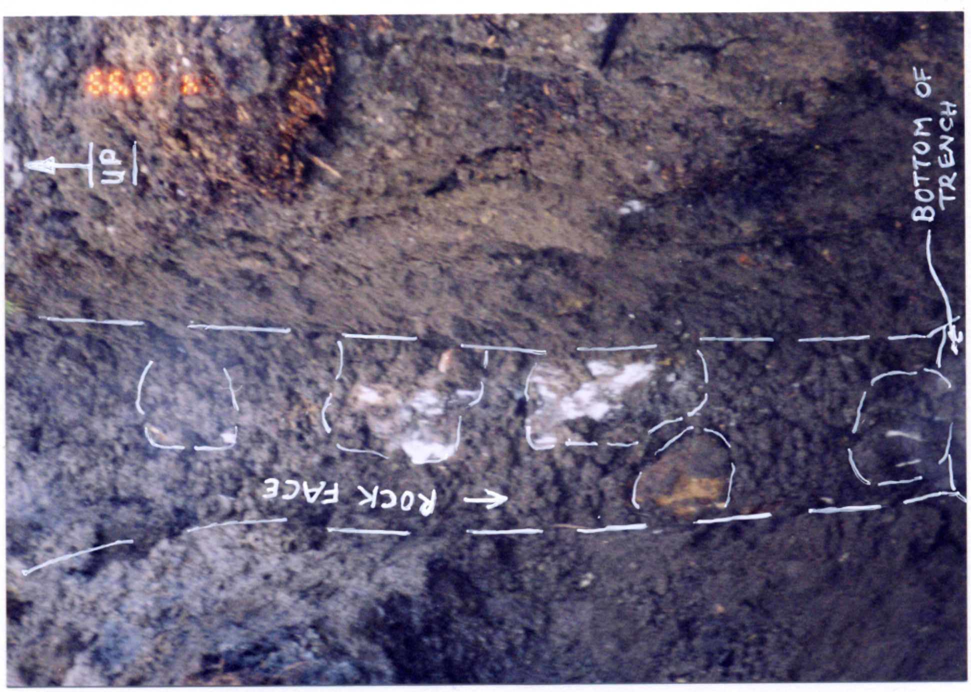
EMI PROJECT NO. 98-145



Trench 99T-2 North-facing slope of dike (Looking S)

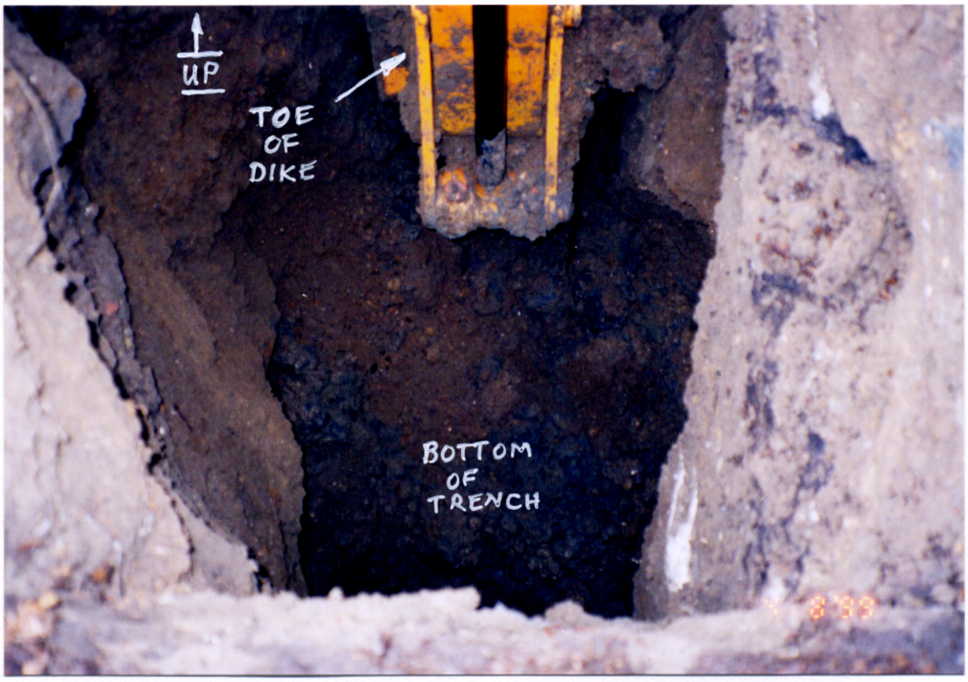


Trench 99T-2 North-facing slope of dike (Looking S)

Trench 99T-2 Largest face rock encountered

Trench 99T-2 Typical dike core material encountered beneath face of dike

PHOTO SHEET
SFOBB EAST SPAN SEISMIC SAFETY PROJECT
EMI PROJECT NO. 98-145



Trench 99T-2 Mixture of sand and blue mud encountered at bottom of trench at 12' depth

PHOTO SHEET
SFOBB EAST SPAN SEISMIC SAFETY PROJECT
EMI PROJECT NO. 98-145

Pit 99P-1 Before excavation started (Looking E)

Pit 99P-1 During excavation (Looking E)

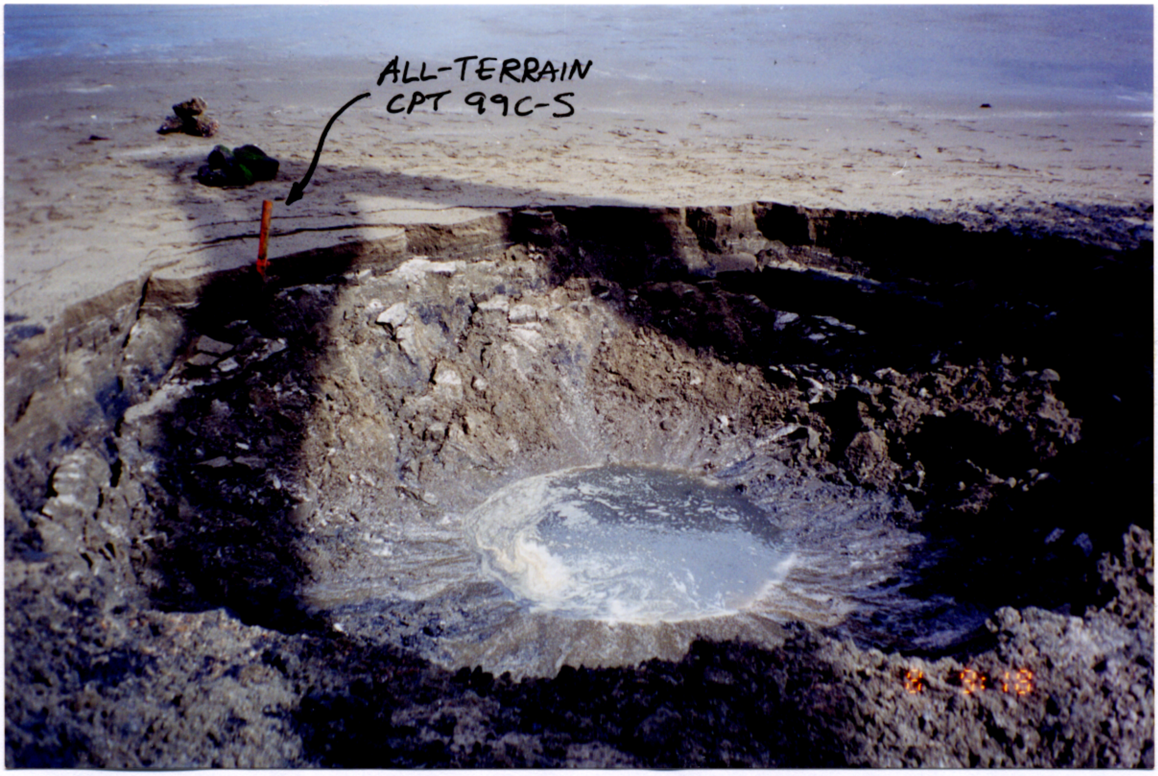
Pit 99P-1 After pumping

Detail of 6" diameter trash pumping equipment

PHOTO SHEET
SFOBB EAST SPAN SEISMIC SAFETY PROJECT
EMI PROJECT NO. 98-145

Pit 99P-2 (looking E)

Pit 99P-2 (Looking N)



Pit 99P-2 (Looking N)

Pit 99P-2 measuring excavation depth

UC BERKELEY REPORT

UNIVERSITY OF CALIFORNIA AT BERKELEY GEOTECHNICAL TESTING FOR THE EAST BAY CROSSING OF THE SAN FRANCISCO-OAKLAND BAY BRIDGE

U. C. Berkeley Geotechnical Testing for the East Bay Crossing of the San Francisco-Oakland Bay Bridge

for
Earth Mechanics, Inc.
Fountain Valley, Ca.

by
Prof. Mike Riemer, Ph.D.
Ann Marie Kammerer
Christopher Hunt
U. C. Berkeley Geotechnical Group

February 1999

GEOTECHNICAL ENGINEERING

DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

UNIVERSITY OF CALIFORNIA • BERKELEY

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UC Berkeley Geotechnical Testing for the East Bay Crossing

1 Testing Program

1.1 Introduction

The geotechnical testing presented in this report was performed for Earth Mechanics, Inc., to assist them in characterizing the likely consolidation and seismic response of the soils beneath the Oakland Mole for the proposed alignment of the new span of the San Francisco-Oakland Bay Bridge. The project was administered through a Service to Industry Agreement with the University's Earthquake Engineering Research Center, and the actual testing was performed by Graduate Student Researchers Ann Marie Kammerer (simple shear testing) and Christopher Hunt (consolidation testing) under the supervision of laboratory director Dr. Michael Riemer. The testing conditions and parameters were selected based on discussions with Hubert Law of Earth Mechanics and Robert Pyke, a consultant for the project.

The purpose of the simple shear testing was to investigate the possible effects of large cyclic straining, such as might be caused by a significant seismic event, on the subsequent stress-strain properties of the soil. To assess this directly, the testing program was designed around testing pairs of specimens from a given sampling tube, which would ideally be identical in composition with each other. One of the specimens would simply be sheared monotonically, while the other would first be cyclically loaded with a specified strain amplitude and number of cycles, then monotonically sheared without allowing the pore pressures from the cyclic loading to dissipate. This approach therefore intends to provide a "Before, and Immediately After an Earthquake" picture of the stress-strain response, provided that the specimens are similar and that the testing conditions adequately model those acting in the field. The procedures and results from this portion of the testing program are presented in Section 2.

The consolidation testing performed as part of this project was focused specifically on attempting to identify the time rate of consolidation under conditions of predominantly lateral drainage. This is a relatively unusual property to measure experimentally, though it is widely agreed that the permeability, and thus also the rate of consolidation, are likely to be substantially different in soils which are deposited in horizontal layers. To make direct comparison with conventional consolidation tests on vertically oriented specimens, it would be ideal to obtain block samples and trim specimens from vertical planes into oedometer rings for testing. As such samples were not available for this testing program, an alternative method was developed. This method utilized the conventional Shelby tube samples available to measure the rate of radial consolidation to applied isotropic stress increments. The procedures, method of interpretation and results of these tests are discussed in Section 3.

1.2 Materials Tested

In the current testing program, soil from seven different sampling locations were chosen for characterization (B2, B5, B6, B7, B9, B10 and P2, based on numbering system employed during sampling and logging). A total of fifteen specimens were successfully tested in simple shear (3 more than the 12 initially planned), while four specimens were tested in radial consolidation. Table 1.1 shows the specimen numbers, the test types, properties and other pertinent information relating to each of them. It should be noted that the "material type" as noted on the table is an estimate based on visual/manual identification only, and that further index testing of the specimens might prove useful for investigating the relationship between material variations and the testing results. From our observations in the lab, however, the material appears somewhat coarser than the 'Bay Mud' from most other sites with which we have testing experience. This observation is supported by the relatively low water contents and high dry densities of the tested specimens.

Nine sample tubes from the Oakland mole were delivered to the UCB soils laboratory by personnel of Earth Mechanics, Inc. All simple shear and consolidation testing was performed on these samples. Since delivery, the samples have been stored in a specially designed "wet room" that maintains a cool, dark, humid environment to prevent moisture changes that could result in either desiccation or loss of soil suction. Preliminary field logs, site maps, and x-rays taken of samples P-2 and P-3 were also provided by Earth Mechanics, Inc.

As testing was performed at the in-situ stress state, appropriate vertical effective stresses had to be calculated prior to testing. The soil profiles, unit weight data and water table locations from the preliminary field logs provided by Earth Mechanics were used for this purpose. Unit weights for each layer were calculated in the following way. Where unit weights were provided and the values showed scatter about a central value throughout the layer, the mean value was used for the entire layer. For layers where the unit weights appeared to trend up or down (as in many of the clay layers) a best-fit line was determined for the layer. Where no unit weight data was provided, the unit weight values provided for layers in adjacent boreholes that had the same description and approximately the same depth were used. Where no unit weight data was provided and similar layers in adjacent borings either did not exist or did not provide unit weights, conservative estimates of the unit weights were made from experience with similar materials. The water table depths provided on the boring logs were assumed to be correct. The calculated effective vertical stress at each depth was plotted to check for reasonableness. Figure 1.1 is an example of the plots developed. Total vertical stress, effective vertical stress, unit weights, and graphical representations of soil type are all shown. The sample and water table positions are also indicated on the figure.

Table 1.1: Samples used in the testing program, and their characteristics

Boring	Sample #	Test Type	γ_t (pcf)	WC (%)	γ_d (pcf)	Material Type
B2	10	MSS	107	54	69	Clay w/shells
		CSS MSS	106	50	71	Clay w/shells
B5	10	MSS	110	45	76	Silty Clay
		CSS MSS	110	48	75	Silty Clay
		MSS	111	76	47	Silty Clay
B6	26	MSS	104	60	65	Clay
		CSS MSS	100	60	62	Clay
B7	12	RC	108	47	73	Clay, w/ silt, shells
B9	8	MSS	116	39	83	Clay w/shells
		CSS MSS	116	40	83	Clay w/shells
		RC	114	40	81	Clay, very silty, large shells
B10	10	MSS	110	49	74	Clay w/silt
		MSS	109	50	73	Clay w/silt
		CSS MSS	110	49	73	Clay w/silt, sand
		RC	113	45	78	Clay, very silty
P2	15	MSS	113	42	80	Sandy Clay
		CSS MSS	111	46	76	Clay w/sand
		MSS	109	51	72	Clay w/shells, sand
		RC	111	47	75	Clay, w/ silt seams and sand

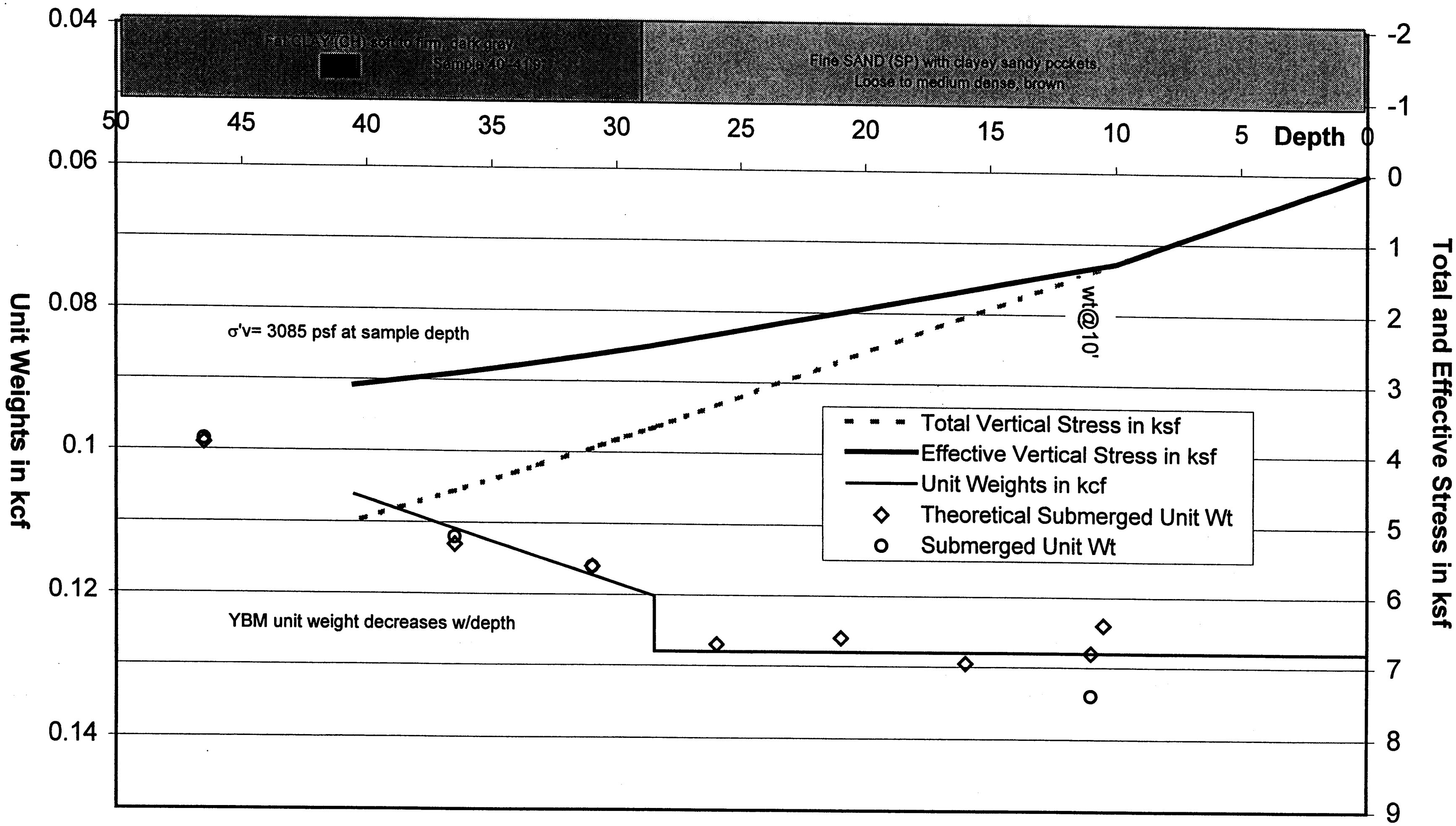


Figure 1.1: Calculation of In-situ Stress State for Boring P-2

2 Simple Shear Testing

2.1 General Considerations for Simple Shear Testing

For a saturated element of soil at some depth below a level ground surface, and below the water table, the initial conditions are usually assumed to include: (1) a major principal stress equal to the vertical effective stress, (2) a minor principal stress equal to the lateral effective stress, and (3) a pore pressure dictated by the hydrostatic conditions. During seismic loading of such an element, if the specimen response is assumed to be undrained, cycles of horizontal shear stress (τ_h) produce shear strains (γ) and may generate excess pore pressures (u). During this loading, there can be no lateral normal strain, as the soil element is surrounded by similarly loaded soil on all sides. There can also be no change in height, since there are no changes in volume or cross-sectional area. At the same time, the total vertical stress also remains unchanged, as the density and depth of overburden are constant.

While simple shear testing does a good job of modeling the application of shear stresses due to seismic loading, it often does not simultaneously achieve the conditions of constant volume and constant total vertical stress. In the conventional type of simple shear test popularized by the Norwegian Geotechnical Institute, the specimen circumference is contained in a wire reinforced membrane. Vertical loads are applied to reproduce the consolidation stresses, while the wire membrane enforces an essentially K_0 condition and the lateral stresses develop accordingly. During cyclic loading of specimens, no effort is made to insure saturation – instead, the testing is performed while keeping the specimen height constant, thereby maintaining constant volume. No pore pressures are measured directly, they are instead inferred from the changes in vertical stresses observed while maintaining the height. In this manner, liquefaction behavior could be investigated in dry sands, by noting the total loss of vertical (and consequently, horizontal) stress. An underlying assumption to this approach, however, is that any generation of pore pressures would be uniformly distributed throughout the soil specimen. Any reduction in the overall vertical load must be a global one, and cannot replicate the softening of a particular seam within the specimen in cases where such localization would be the natural failure mode.

In discussions with Robert Pyke about the current testing program, the possibility of failure due to localized pore pressure generation emerged as a substantial concern. The testing equipment and procedures used in the program were chosen to allow that failure mode to control the soil response if present. The primary feature of the UC Berkeley simple shear testing device (shown in Plate 2.1) that permits such testing is the ability to apply independent chamber pressures and back pressures to the specimen, in a manner directly analogous to conventional triaxial testing systems. This allows back pressure saturation of specimens, which ensures a truly saturated response, and also permits the user to apply lateral effective stresses to augment those carried by the wire reinforcement. Pneumatic actuators on the device, controlled by closed-loop servo valves linked to the computer, allow either load or displacement control in the vertical and shear directions. In the current testing program, the flexibility of this system allowed us to perform the cyclic and monotonic shear tests under a variety of conditions, and in particular allowed the soil to develop pore pressures in whatever parts of the specimen were most vulnerable to seismic loading: this was accomplished by testing the specimens using wire reinforced membranes, back pressure saturating them, and carrying out the tests under constant vertical load. Other tests were performed under more conventional test conditions, both to investigate the effects of these conditions on the results, and to make sure that some critical mode of behavior was not being overlooked.

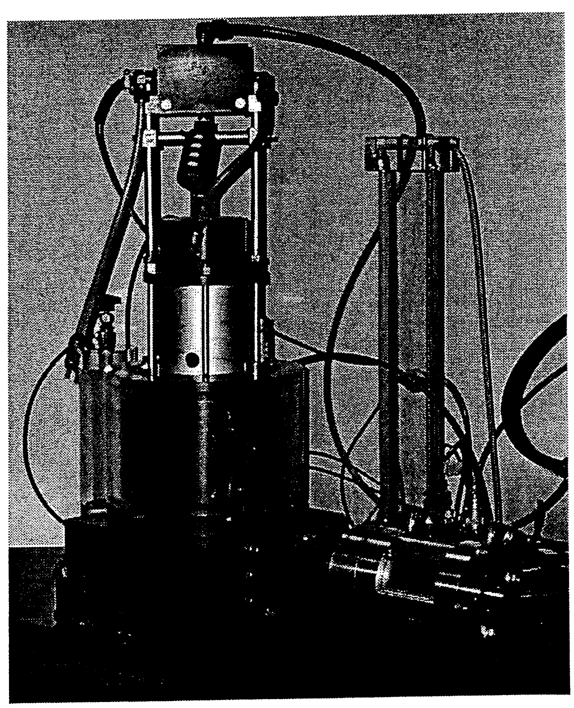


Plate 2.1: Photograph of simple shear device, showing chamber and pneumatic actuators.

2.2 Specimen Preparation and Testing Procedures

Table 2.1 shows the 15 specimens tested, the borings from which they were taken, and summarizes the testing conditions for the simple shear portion of the testing program. All the tests were performed using wire-reinforced membranes, which are used to provide lateral restraint to the specimen, and hence enforce essentially K_0 conditions as the sample is consolidated under axial load. The first eight specimens (numbers 1 through 11) were tested without the addition of any back pressure to improve saturation (i.e. conventional preparation), while the final seven were back pressure saturated using the chamber and back pressure described in 2.1. "B values" of 0.95 were routinely obtained through the reinforced membranes after application of one to two atmospheres of back pressure, suggesting that saturation was obtained, and that truly undrained conditions could be imposed during shearing. In order to allow localizations within the specimens, as noted earlier, most of the tests were performed under constant vertical load conditions, though specimen 5 was tested under constant height conditions for comparison.

Table 2.1: Testing conditions applied during simple shear tests.

Spec #	Boring	Test Types	γ_d (pcf)	BP	Material Type	$\sigma'_{v,con}$ (psf)	Notes on Tests
1		M	76	N	Silty Clay		
4	B5	C (1%) M	75	N	Silty Clay	2650	
5		M	76	N	Silty Clay		constant height
6	B2	M	69	N	Clay w/shells	2700	
8		C (2%) M	-	N	Clay w/shells		
9	B9	M	83	N	Clay w/shells	2600	large shell at edge
10		C (1%) M	83	N	Clay w/shells		
11		M	74	N	Clay w/silt		Unsaturated
12	B10	M	73	Y	Clay w/silt	2700	Saturated
13		C(1.5%) M	73	Y	Clay w/silt, sand		
14	B6	M	65	Y	Clay	3960	
15		C (3%) M	62	Y	Clay		
16		M	80	Y	Sandy Clay		sandier than 17, 18
17	P2	C (3%) M	76	Y	Clay w/sand	3100	
18		M	72	Y	Clay w/shells, sand		to confirm 16

Because of the possible disturbance of the samples due to in-situ vane shear testing, the first six inches of soil from each sampler was removed and inspected. To prepare a specimen, a disk of soil approximately 0.9 inches high is extruded and removed from the remaining sample. The soil is placed between the end caps used in the simple shear device and trimmed to the diameter of the wire-reinforced membrane using a special trimming device. The membrane is applied to the sample and sealed to the caps with rubber o-rings. The weight of the sample can then be determined. Drain lines are attached to the end caps and a vacuum (of lesser value than the calculated horizontal effective stress) is applied to seat the caps and remove air from the specimen. The height of the specimen under the vacuum is measured. While maintaining the vacuum, the specimen is seated in the simple shear device and the initial vertical LVDT reading is correlated with the height of the sample. For the remainder of the testing procedures, the vertical LVDT reading can be used to determine the current height of the sample.

A small seating load is imposed on the top of the sample and the vacuum is slowly reduced. If the test is not back pressure saturated, the vacuum is removed completely. If back pressure saturation is desired, the vacuum is reduced until the effective stress reaches a predetermined value. The value chosen is based on the requirements of the test. An effective stress of 10 kPa was used for the saturated tests. The external chamber that is then placed on the device. The cell pressure is increased to the chosen effective stress while the vacuum is removed. The drain line is then attached to a back pressure source. The cell and back pressures are slowly raised such that the effective stress remains constant at 10 kPa. Once the back pressure reaches approximately 100 kPa, B value checks are performed after each new increment of pressure is added. This process is continued until the B value reaches 0.95 or better.

Consolidation to the calculated vertical effective stress then occurs once the appropriate additional vertical load is added. The applied lateral effective stress of 10 kPa is assumed to simply reduce the stress developed in the wires of the membrane, and does not alter the lateral effective stress under the K_0 consolidation conditions. The specimen is allowed to consolidate until a full log-cycle of secondary compression has occurred (evaluated based on the square root of time method). The B value is again checked to assure saturation. The specimen is then tested as required while keeping the valve on the drainage line closed. For the specimens that were truly saturated, this valve enforced undrained conditions, though of course for specimens without full saturation, air bubbles within the specimen/system would provide partial drainage of any pore pressures developed. Cyclic loading consisted of 25 strain-controlled cycles at a frequency of 0.2 Hz, with single amplitude shear strains ranging from 1% to 3%, as requested. In order to capture the effects of any localization in pore pressure generation, monotonic tests were run immediately after the cyclic tests. Monotonic tests were strain controlled at a rate of 1%/minute, a value commonly associated with Unconsolidated-Undrained (UU) triaxial tests on clay soils to simulate "rapid" shearing.

2.3 Results of Simple Shear Testing

The results of individual simple shear tests are plotted in Appendix A, in terms of the cyclic loading histories and monotonic stress strain behavior. These results are summarized in Figures 2.1-2.6, on which are compiled the monotonic shear stress (τ) vs. shear strain (γ) curves obtained from the specimens for a given sample. For example, Figure 2.1 shows the (τ)-(γ) plots for specimens #1, #4 and #5, all taken from Boring B-5. In spite of the inevitable differences due to variability between specimens, these figures show a reasonably consistent effect of cyclic loading on the subsequent shear response. At small strains, those specimens which had been cyclically loaded showed a much softer/weaker response than those simply sheared monotonically, while at large strains (> 8-12%) there is relatively little effect of the previous cyclic loading. Figure 2.7 shows the cyclic loading from specimen #17 (in the form of the first and last of the 15 cycles) superimposed on the subsequent static loading. From this figure it is clear that the softened initial response of this specimen to static loading is due directly and predictably to its earlier cyclic straining - once the strain exceeds that developed in the cyclic loading, the specimen stiffens and approximates its statically sheared counterpart (for example, specimen #8 in Fig. 2.2)

Figures 2.8 and 2.9 show the effect of the saturation process on the cyclic response of the specimens. For the unsaturated specimens (Fig. 2.8), the cyclic loading produced larger changes in the height of the specimen than for the saturated specimens (Fig. 2.9), while the recorded values of excess pore pressure were substantially larger and developed earlier in the saturated specimen, as would be expected. These same trends were observed in the monotonic tests as well, and suggest that the saturation of the specimens substantially improved the modeling of the assumed field conditions, showing reasonably constant height and constant vertical loads up to shear strains of at least 10% to 12%, though at larger strains the constant height condition degraded somewhat as the specimens were substantially deformed.

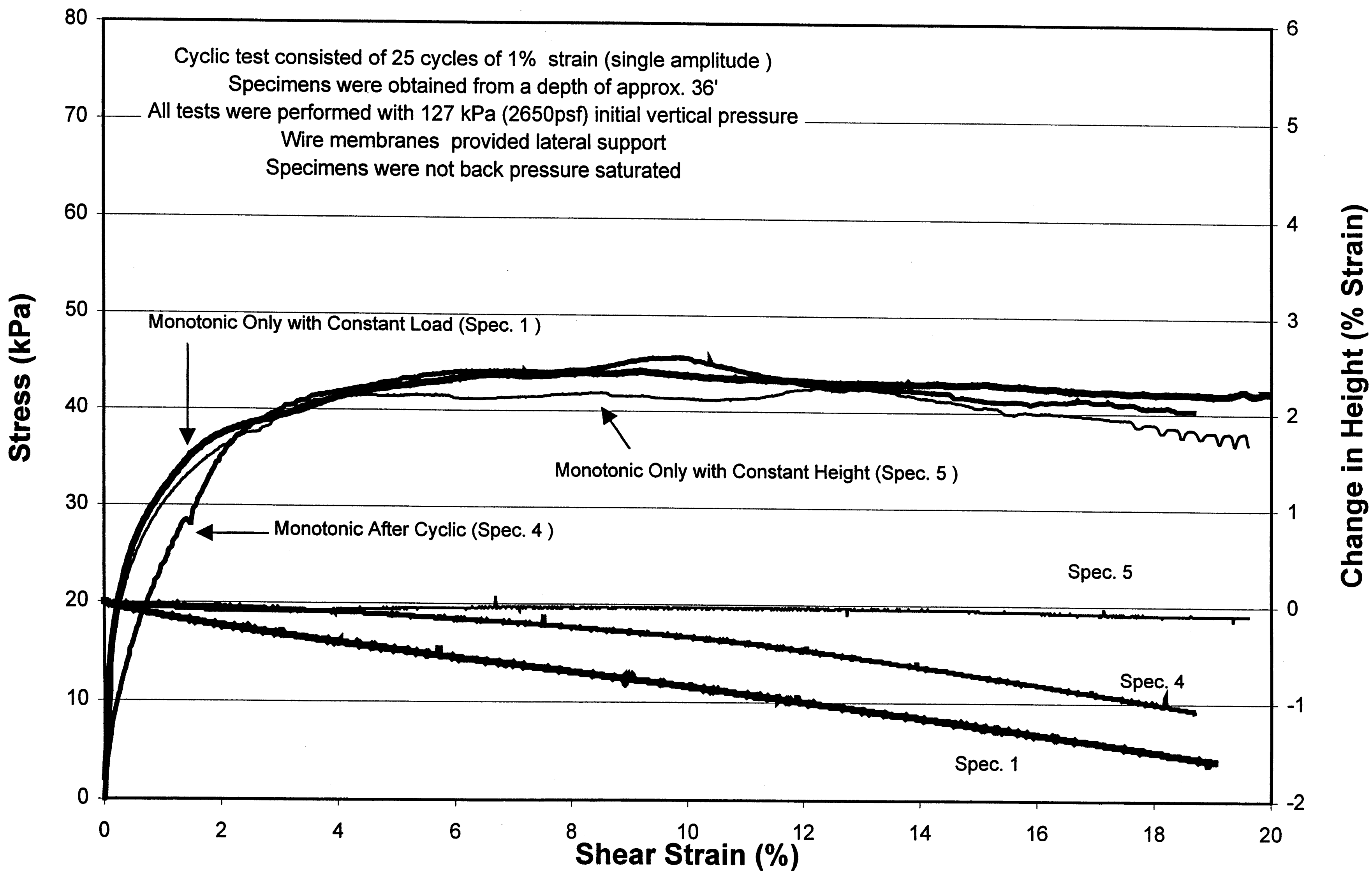


Figure 2.1: Stress-Strain Plots for Specimens from Boring B-5, Sample 10

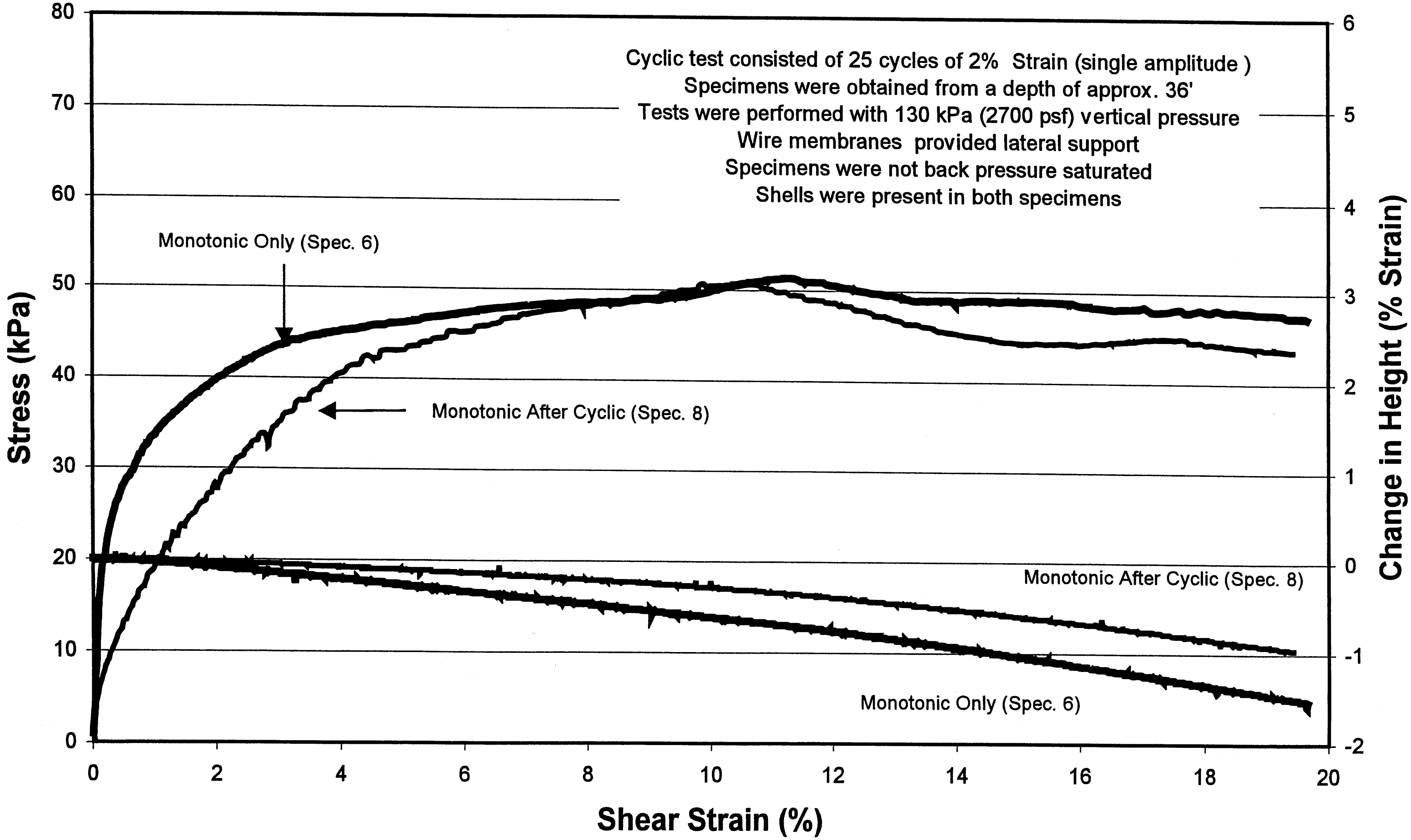


Figure 2.2: Stress-Strain Plots for Specimens from Boring B-2, Sample 10

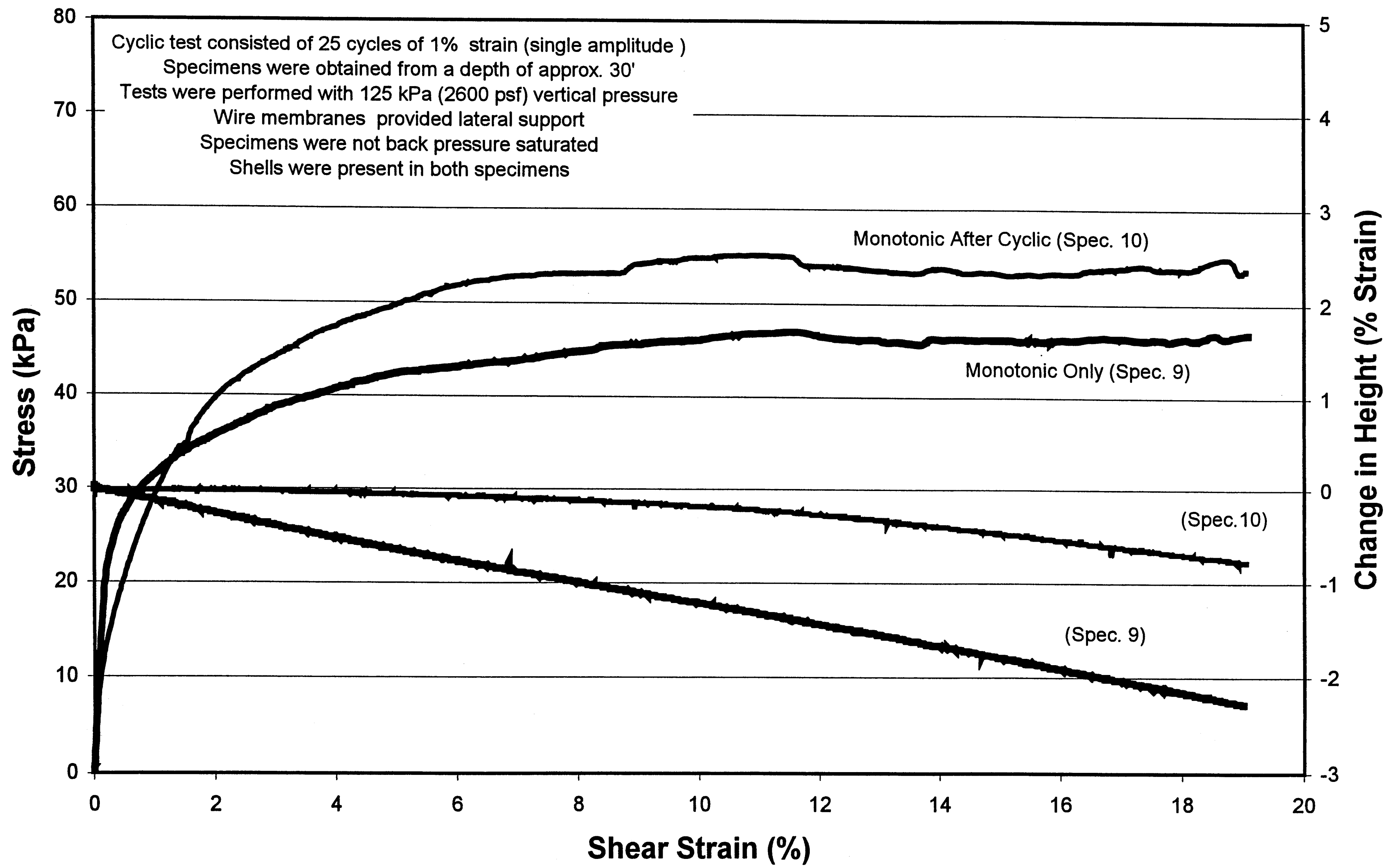


Figure 2.3: Stress-Strain Plots for Specimens from Boring B-9, Sample 8

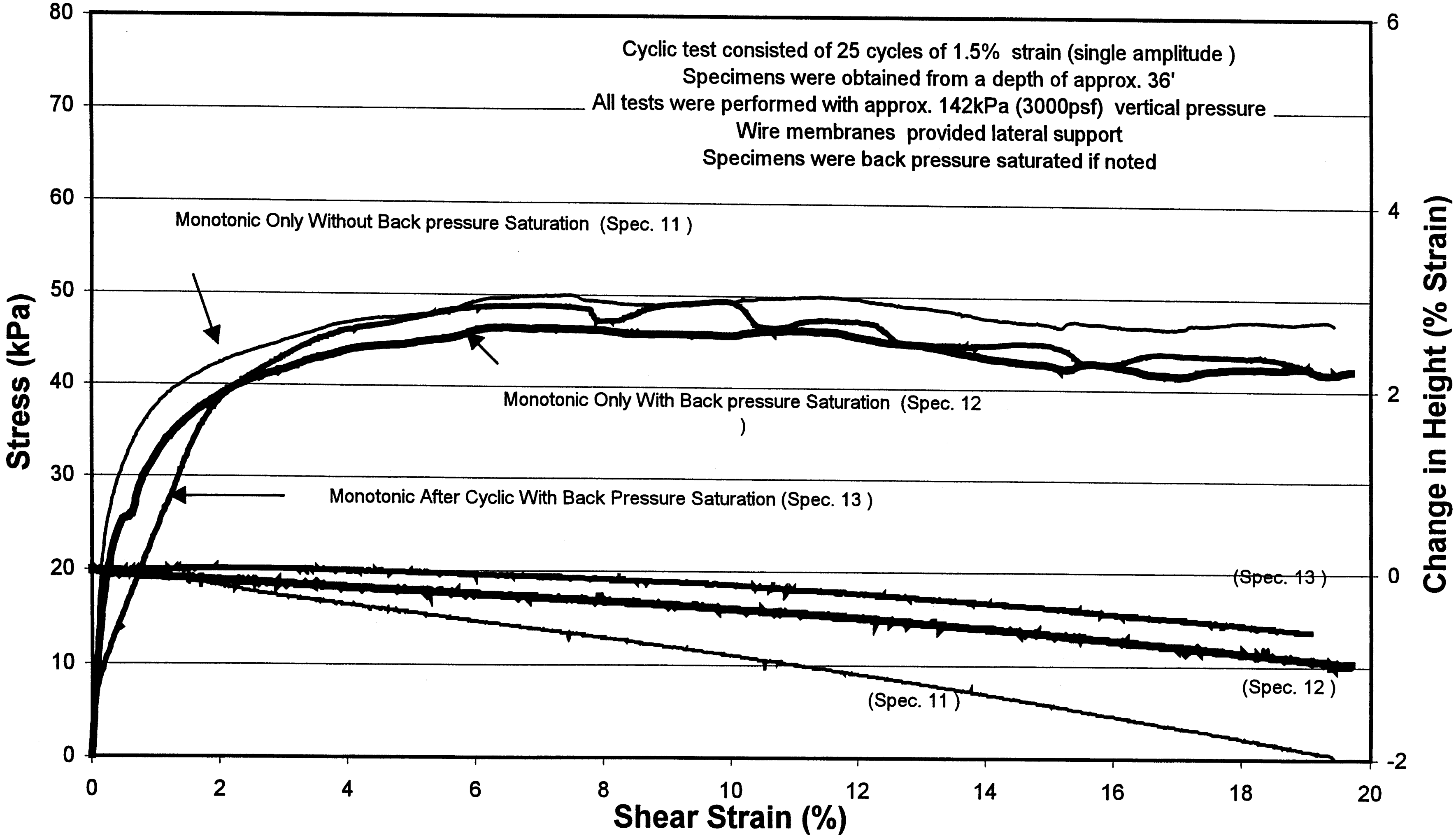


Figure 2.4: Stress-Strain Plots for Specimens from Boring B-10, Sample 10

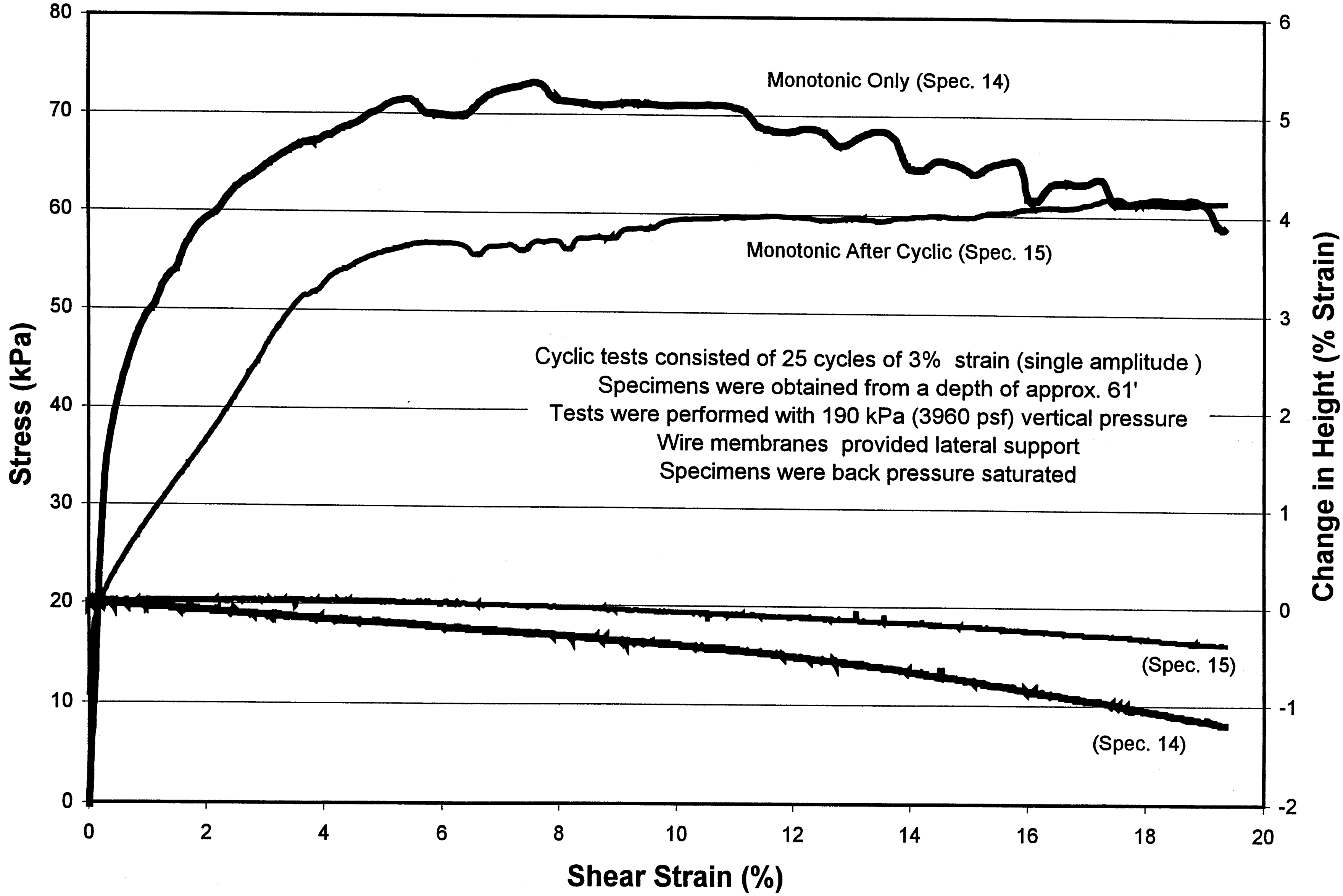


Figure 2.5: Stress-Strain Plot for Specimens from Boring B-6, Sample 26

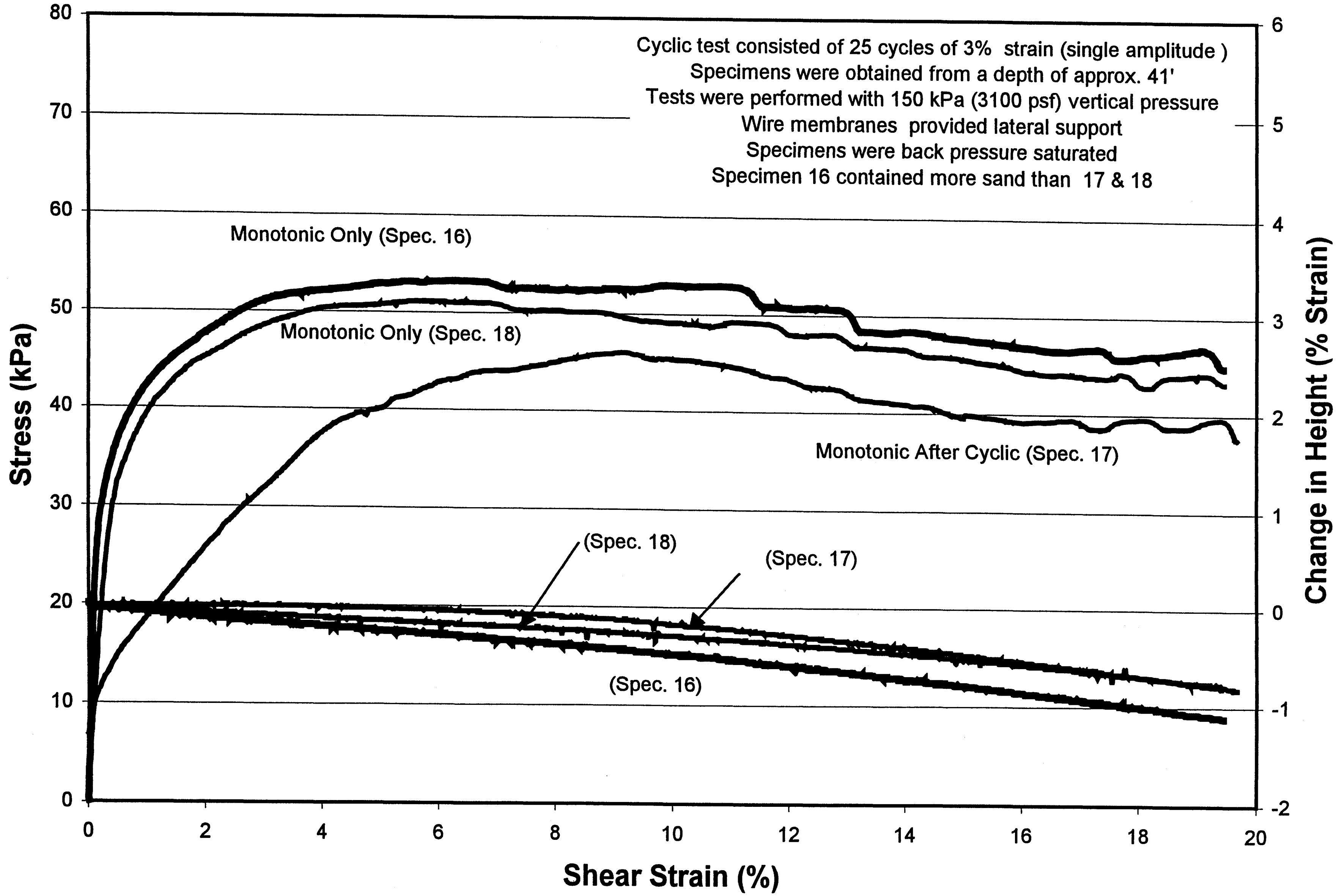


Figure 2.6: Stress-Strain Plots for Specimens from Boring P-2, Sample 15

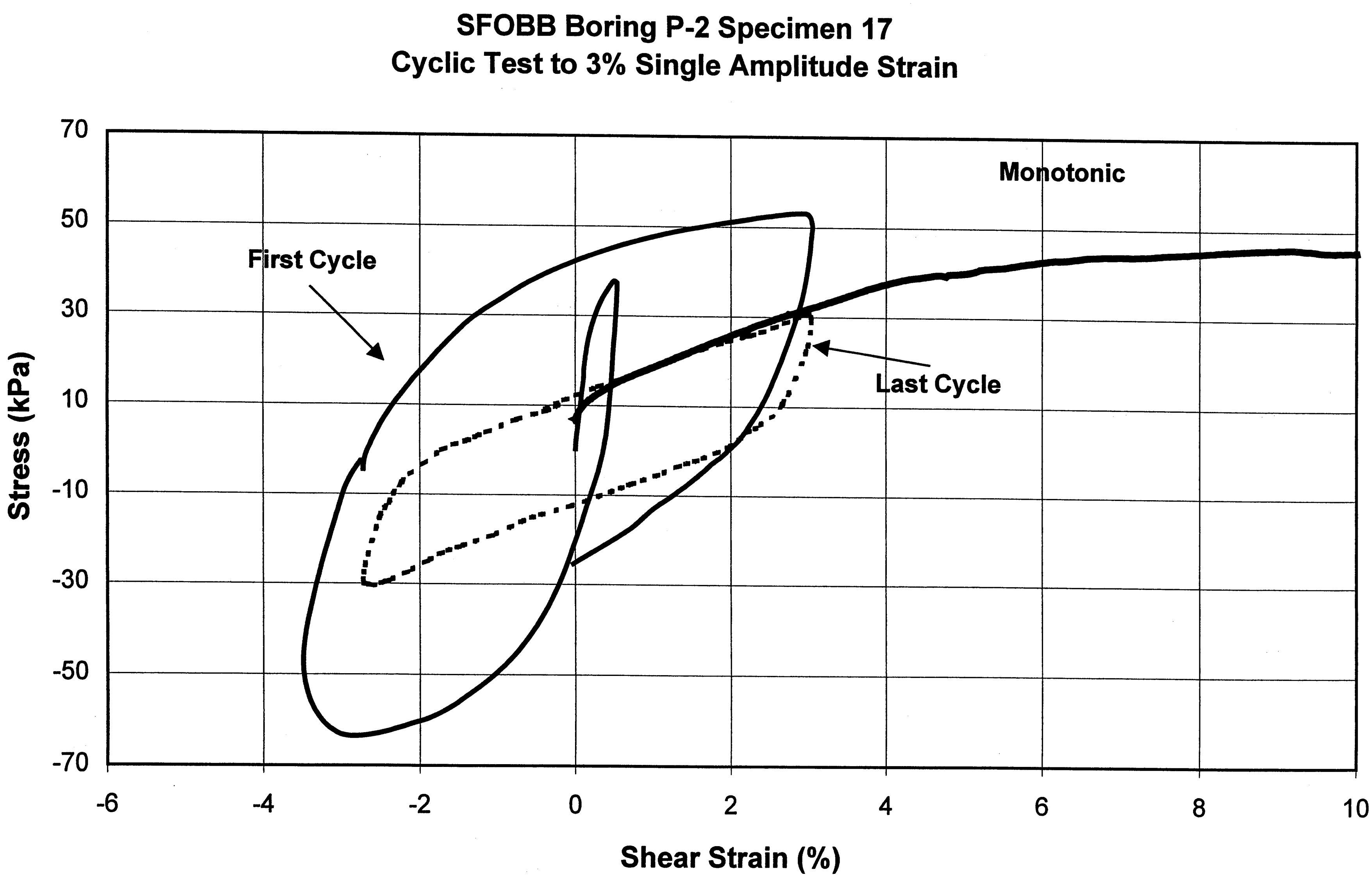


Figure 2.7: Comparison of first and last hysteresis loops with response in subsequent monotonic test

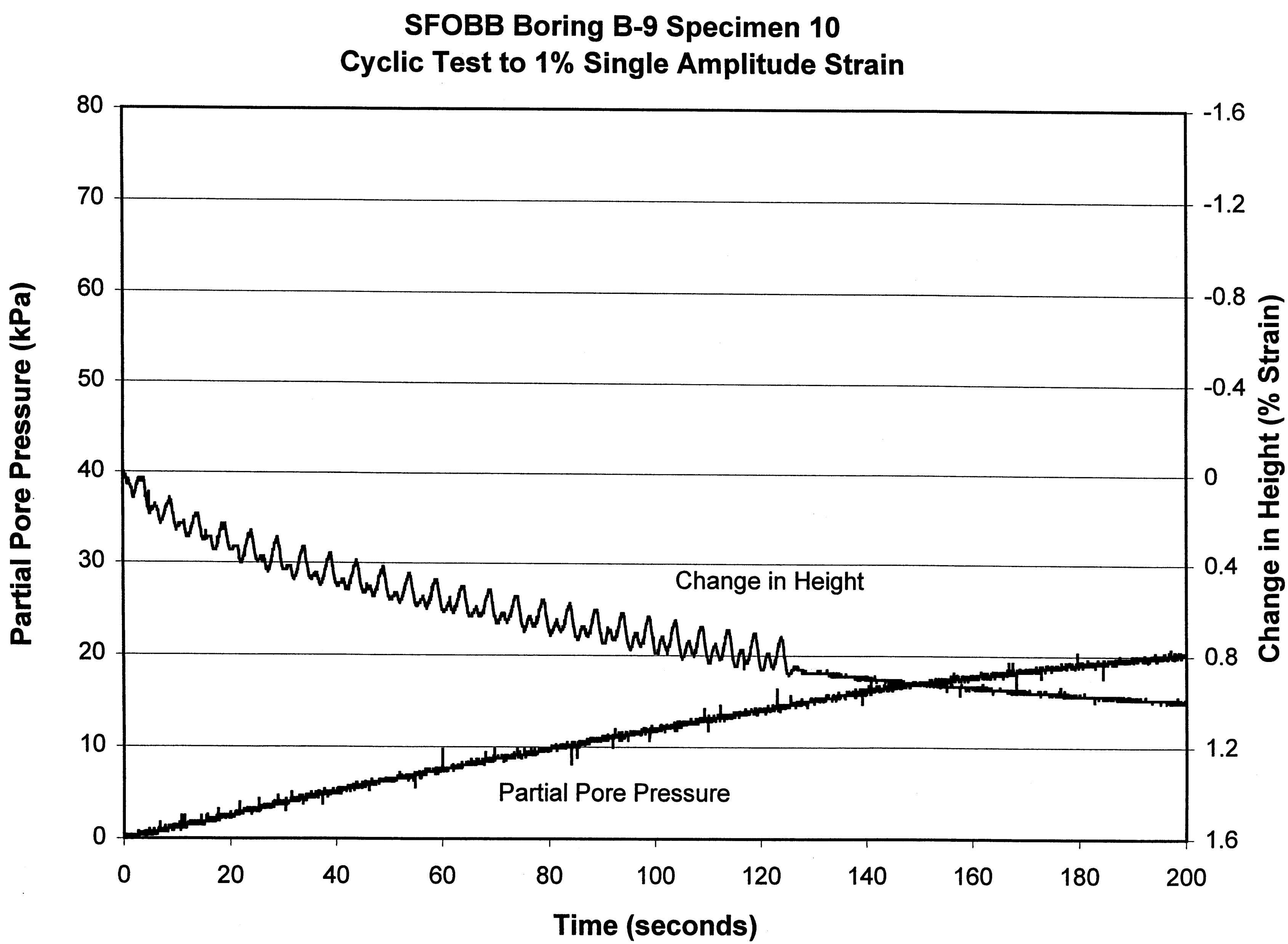


Figure 2.8: Time histories of load, height and measured pore pressure in an unsaturated cyclic test.

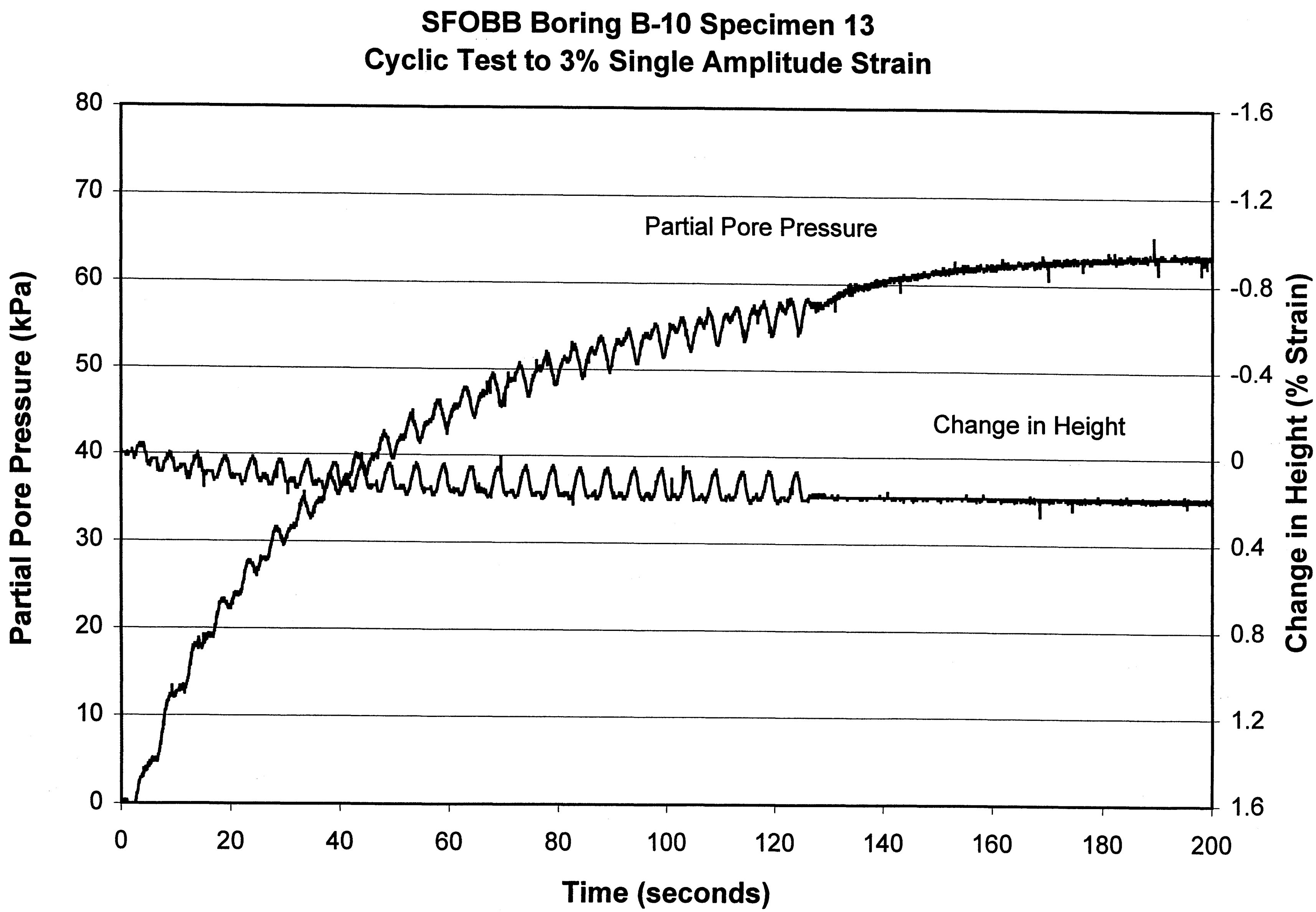


Figure 2.9: Time histories of load, height and measured pore pressure in a saturated cyclic test

3 Consolidation Testing

3.1 General considerations

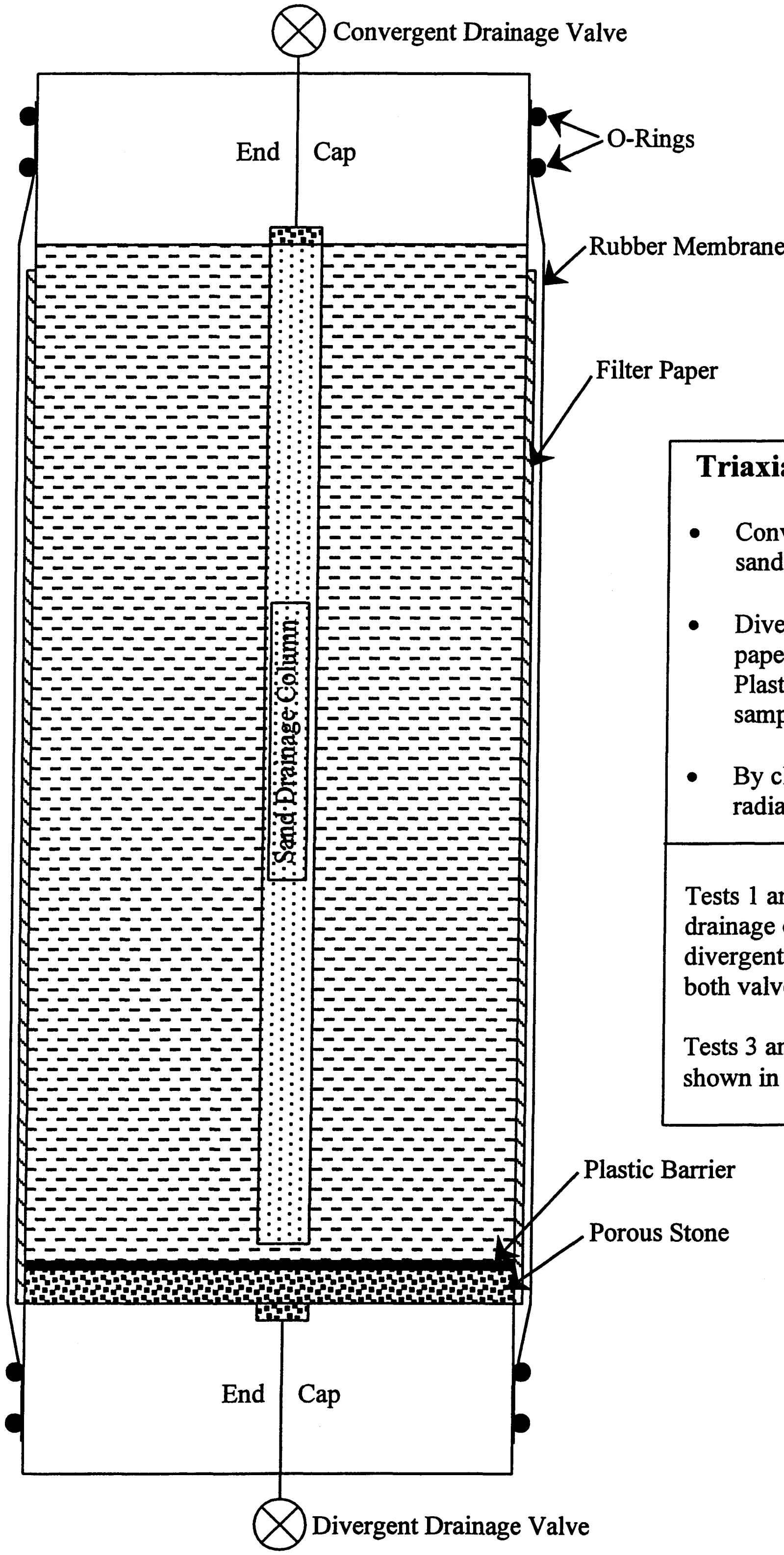
As suggested by Robert Pyke, the focus of the consolidation testing performed at UC Berkeley has been directed toward investigating the ***rate*** of consolidation that could be expected when pore pressures are allowed to dissipate horizontally, as would be the case with the installation of wick drains in the field. Rather than conventional oedometer tests, therefore, which are designed to consolidate horizontal slices of a tube sample using vertical drainage, this testing program employed a modification of triaxial test equipment to measure the consolidation response of large cylindrical specimens to isotropic increases in stress.

Apparent advantages of this approach stem from the substantial height of the specimen, which both improves the likelihood of obtaining a representative sample in soils expected to have horizontal layering, and minimizes whatever end effects do occur due to non-radial flow. One apparent disadvantage is that while the flow is horizontal, it is radially outward, or ***divergent***, rather than toward a central drain, which is the situation in the field. Another disadvantage which becomes apparent very quickly is the substantial drainage path, which is approximately three times that of a conventional 1-D vertical test, and which slows down the testing process substantially. Less obviously, but perhaps more importantly, the time curve is likely to be heavily affected by the soil that is most severely disturbed in the sampling process – the outer surface of the sample, which may be smeared as the sampling tube pushes through it while advancing down into the soil.

3.2 Specimen Preparation and Testing Procedures

The first two specimens, from B-9 and B-10, were prepared by extruding soil from the desired Shelby tube (approximately 7.2 cm in diameter) and trimming the ends for a sample length slightly greater than 15 cm. Each was then placed between two full-sized porous stones, but a sheet of plastic was introduced between the soil and stone at each end, effectively preventing vertical flow of water into the stones. Strips of filter paper (also known as "Bishop's pajamas") were wrapped around the circumference of the specimen, taking care to fully overlap the edge of the porous stone at each end. A conventional triaxial membrane was then placed over the specimen, and saturation procedures performed (the bottom end of the sample represented in figure 3.1 shows this testing setup). Isotropic increments of consolidation pressure were then applied using increments of chamber pressure, and both axial and volumetric strains were recorded over time. The in situ stress states of the specimens were estimated and loading schedules created such that the first consolidation increment would be fully in reconsolidation (less than the lateral stress in the field), the second would be just into normal compression (greater than the vertical stress in the field), and any subsequent increments would be normally consolidated (Table 3.1).

Upon evaluating results from the first two samples and calculating unexpectedly low values of the horizontal coefficient of consolidation, a modified test (Fig. 3.1) was developed to examine consolidation rates on the same sample using both divergent flow (radially outward) as in the first two tests, and convergent flow (radially inward), as would occur with wick drains or stone columns in the field. Sample tubes from P-2 and B-7 were selected and cut to a height of roughly 18 cm. While still in the tube, the soil was cored with a device similar to a miniature sampling tube with an outside diameter of nearly 11 mm. The specimens were then extruded and trimmed to approximately the same height as the previous two samples. The exterior surface of the specimen from B-7 was trimmed, reducing the diameter of the sample by approximately 2 mm, in order to investigate the influence which sampling-induced smear might be having on divergent flow. A small plug of remolded soil was inserted into the core at the base of each sample, and the cores were then filled with fine sand. Each sample was placed on a full-sized porous stone with a sheet of plastic in between to block the vertical flow path. Strips of filter paper were wrapped around the circumference of the specimens. The filter paper was sized so as to overlap the porous stone at the bottom end of the sample, while not quite extending to the full height of the specimen. In this fashion, the drainage valve connected to the lower end cap of the triaxial cell would drain the filter paper (divergent flow), while the valve connected to the upper end cap would drain the sand core (convergent flow). Addition of a conventional membrane completed the setup and testing proceeded as with the first two specimens, except that all but the last consolidation increment were performed with convergent drainage (Table 3.1).



Triaxial Testing of Radial Consolidation

- Convergent drainage valve takes flow from sand drainage column.
- Divergent drainage valve takes flow from filter paper, through porous stone at sample base. Plastic barrier prevents vertical flow from sample to porous stone.
- By closing one or the other drainage valve the radial consolidation path can be specified.

Tests 1 and 2 were performed without the sand drainage column, with both ends prepared as divergent drainage end in the schematic, and with both valves open simultaneously.

Tests 3 and 4 were performed with the setup as shown in the schematic.

Figure 3.1: Radial Consolidation Testing Schematic

Table 3.1: Overview of consolidation testing performed

Boring	Sample #	Test Type*1	WC (%)	Material Type	σ'_{vi} (kPa)	σ'_{hi} *2 (kPa)	Isotropic Loading Increments (kPa)			
							1*3	2	3	4
P-2	15	C/D	47	Clay, silt seams, some sand	148	74	50-70 conv	70-150 conv	150-200 conv	200-250 div
B-7	12	C/D	47	Clay w/ silt, shells	160	80	50-80 conv	80-160 conv	160-210 conv	210-260 div
B-9	8	D	40	Clay, very silty, large shells	120	60	50-60 div	60-125 div	125-200 div	n/a
B-10	10	D	45	Clay, very silty	135	68	50-65 div	65-135 div	135-205 div	n/a

*1- D = Divergent drainage only C/D = Convergent and Divergent drainage capabilities
*2- Assuming $K_0 = 0.5$
*3- All samples were saturated at 50 kPa isotropic effective stress

3.3 Interpretation of Time Curves

Terzaghi's (1943) classic theory gives the well-known equation for vertical consolidation of soils:

$$c_v \cdot \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \tag{3.1}$$

where c_v is the vertical coefficient of consolidation. Standard solutions to this equation show the percent consolidation of a soil specimen or layer as a function of a dimensionless time factor, T, which can be used to back-calculate c_v for the soil in question as:

$$c_v = \frac{T_{90}}{t_{90}} H^2 \tag{3.2}$$

where T_{90} and t_{90} are the time factor and the time, respectively, corresponding to 90% consolidation and H is the thickness of the sample. The choice of 90% consolidation corresponds to the square root of time curve-fitting method by Taylor (1948) for determining t_{90} from laboratory data. In this method the laboratory consolidation data is plotted versus the square root of time, a linear fit is drawn through the early part of the data, the abscissa of this line is multiplied by 1.15 and t_{90} is selected as the point where this multiplied line intersects the plotted data. The corresponding T_{90} for vertical consolidation is 0.848. Alternatively, T_{50} and t_{50} are typically used with the logarithm of time method originally proposed by Casagrande, but this approach is rarely used in relation to radial consolidation.

For radial consolidation, these equations instead become:

$$c_h \cdot \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \cdot \frac{\partial u}{\partial r} \right) = \frac{\partial u}{\partial t} \tag{3.3}$$

$$c_h = \left(\frac{T_{90}}{t_{90}} \right) \cdot r_e^2 \tag{3.4}$$

where c_h is the horizontal coefficient of consolidation and can be determined through solution of Eqs. 3.3 and 3.4 for the particular geometry of the specimen (r_e is the effective radius of the sample) and the boundary conditions specified by the method of testing (divergent or convergent) being used.

Barron (1948) developed radial flow solutions for analyzing consolidation through the use of sand drains in the field. Barron presented solutions for two consolidation scenarios. In free strain the load is assumed uniform across the horizontal plane of influence of the drain and the soils closest to the drain typically consolidate faster than those at greater distance. In the second case, equal vertical strain, the strains on any horizontal plane are assumed equal, and no differential settlement occurs. Berry and Wilkinson (1969) used Barron's solutions for free strain convergent flow to a central sand drain to develop a modified version of Taylor's (1948) square root of time method for determining T_{90} and t_{90}. Sridharan et al. (1996) developed a similar method using Barron's equal vertical strains solution for convergent flow. McKinlay (1961) presented another Taylor-type curve-fitting technique for the case of divergent flow, developed from solutions to heat flow in an infinitely long, homogeneous cylinder, presented by Carslaw and Jaeger (1946, 1986). Several other researchers (Escario and Uriel, 1961; Trautwein et al., 1981) have looked at these and other techniques for determining c_h for various conditions, but most end by recommending some form of Taylor's method. Very few researchers have attempted to use a logarithm of time method for determining c_h (Silveira, 1953).

For divergent drainage under free strain conditions, McKinlay (1961) recommends plotting the laboratory data versus time to a power of 0.465, multiplying the abscissa of the linear fit by 1.22, and using T_{90} equal to 0.335. Fig. 3.2 shows this technique applied to the sample from B-10, producing a c_h value of 2.9 ft^2/yr. However, as this is a simplified curve-fitting technique, it was deemed best to reproduce the actual theoretical curve taken from Carslaw and Jaeger (1986) and attempt to match the laboratory data to this curve. Fig. 3.3 shows this process, in which the

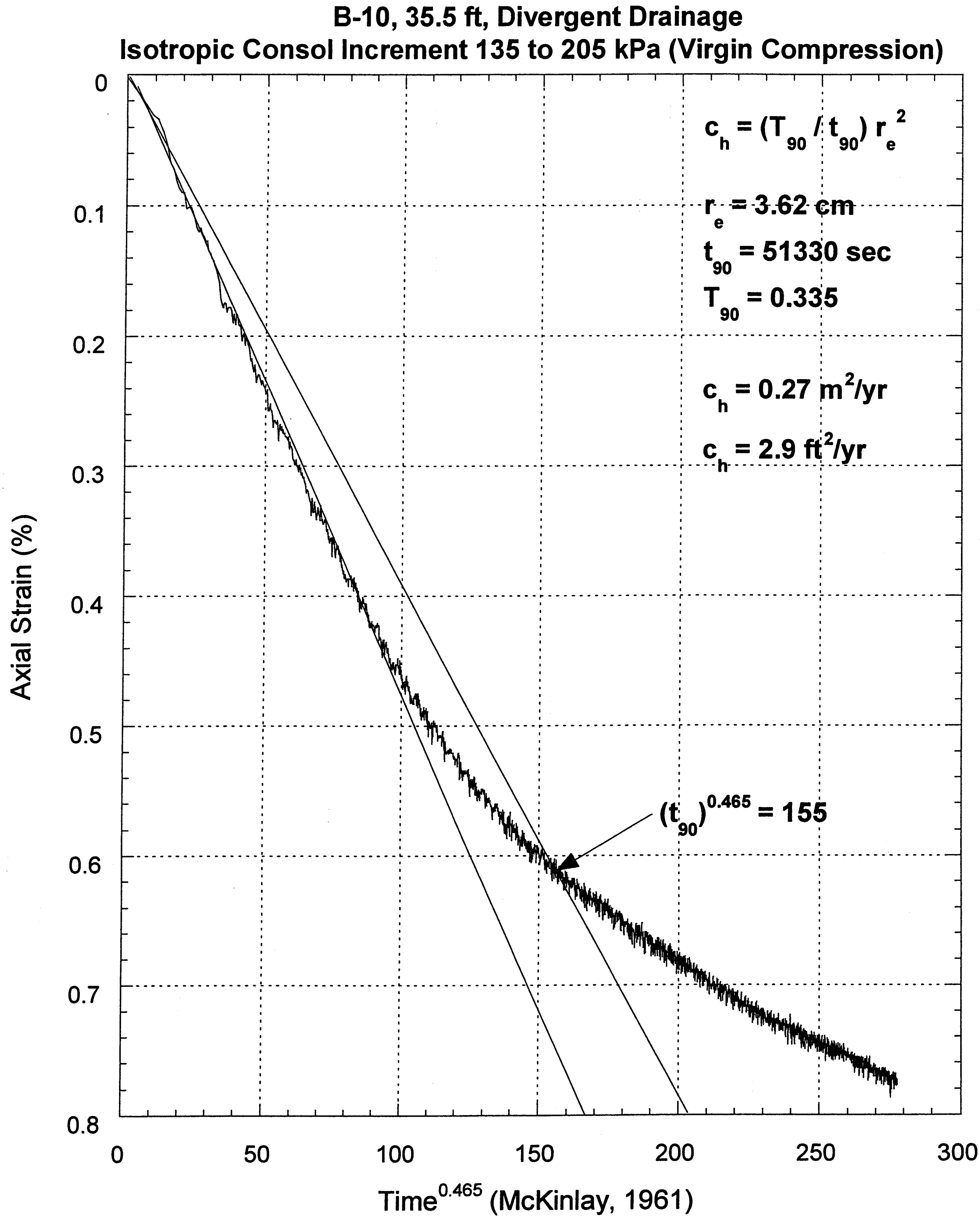


Figure 3.2: Divergent Drainage Simplified Method

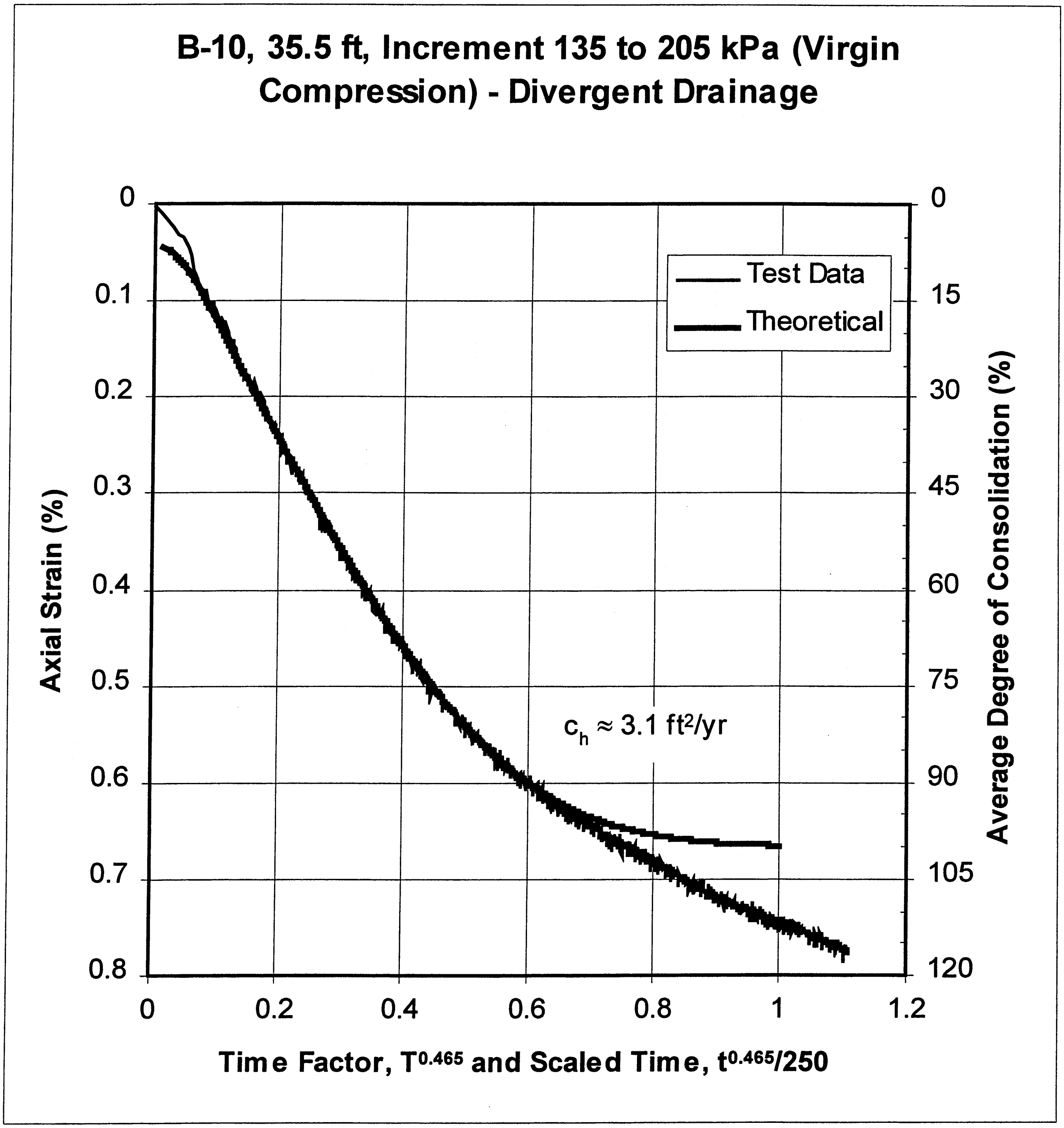


Figure 3.3: Divergent Drainage Curve Fitting Method

laboratory time, t, has been scaled to match the time factor, T, yielding a c_h value of 3.1 ft^2/yr. Secondary compression is quite evident in this figure as the laboratory data falls below the theoretical curve for primary consolidation. In cases where the laboratory data and the theoretical curve diverge prior to reaching 90% consolidation, t_{90} is taken as the equivalent laboratory time on the theoretical curve at 90%, thus minimizing the influence of secondary compression. In general, the divergent drainage tests were found to match theoretical curves well and thus reported values of c_h are determined from this technique rather than McKinlay's method.

For convergent drainage under free strain conditions, Berry and Wilkinson (1969) plot the laboratory data versus the square root of time, use a multiplier of 1.17, and provide a plot of the time factor T versus the average degree of consolidation in the sample for various values of n, where n is the ratio of the diameter of the sample being tested to the diameter of the sand drain. They cannot provide a single value of T_{90} for the convergent drainage case because the ability of the soil to consolidate is dependent on the capacity of the sand drain, which makes T_{90} a function of the ratio n. Sridharan et al. (1996) produce very similar results to Berry and Wilkinson, using square root of time and a multiplier of 1.17, but using time factor T versus degree of consolidation plots based on the equal vertical strain solutions from Barron (1948). It has been shown (Leonards, 1962) that for values of n greater than 5, the free strain and equal vertical strain solutions are approximately equal. The two samples tested with convergent drainage in this study had n values greater than 5 and thus the technique of Sridharan et al. (1996) was used, given that the equations defining the theoretical curves of percent consolidation versus time factor, T, as a function of n are simpler for equal vertical strain versus free strain.

Fig. 3.4 shows the laboratory data from B-7 versus the theoretical curve for convergent drainage with n equal to 6.5. Due largely to the difference in the early portions of the two curves, it was not possible to match the curves well. Most likely these differences are due to layering within the actual sample versus the theoretical curve which assumes a homogeneous material. Thus, in the actual sample silt seams consolidate faster than the clay layers, increasing the initial rate of consolidation, while the latter portion of the data represents more of the interaction between the clay and the silt and is likely dominated by the clay consolidation rate. With this assumption, the simplified method proposed by Sridharan et al. (1996) was deemed suitable as the linear fit to

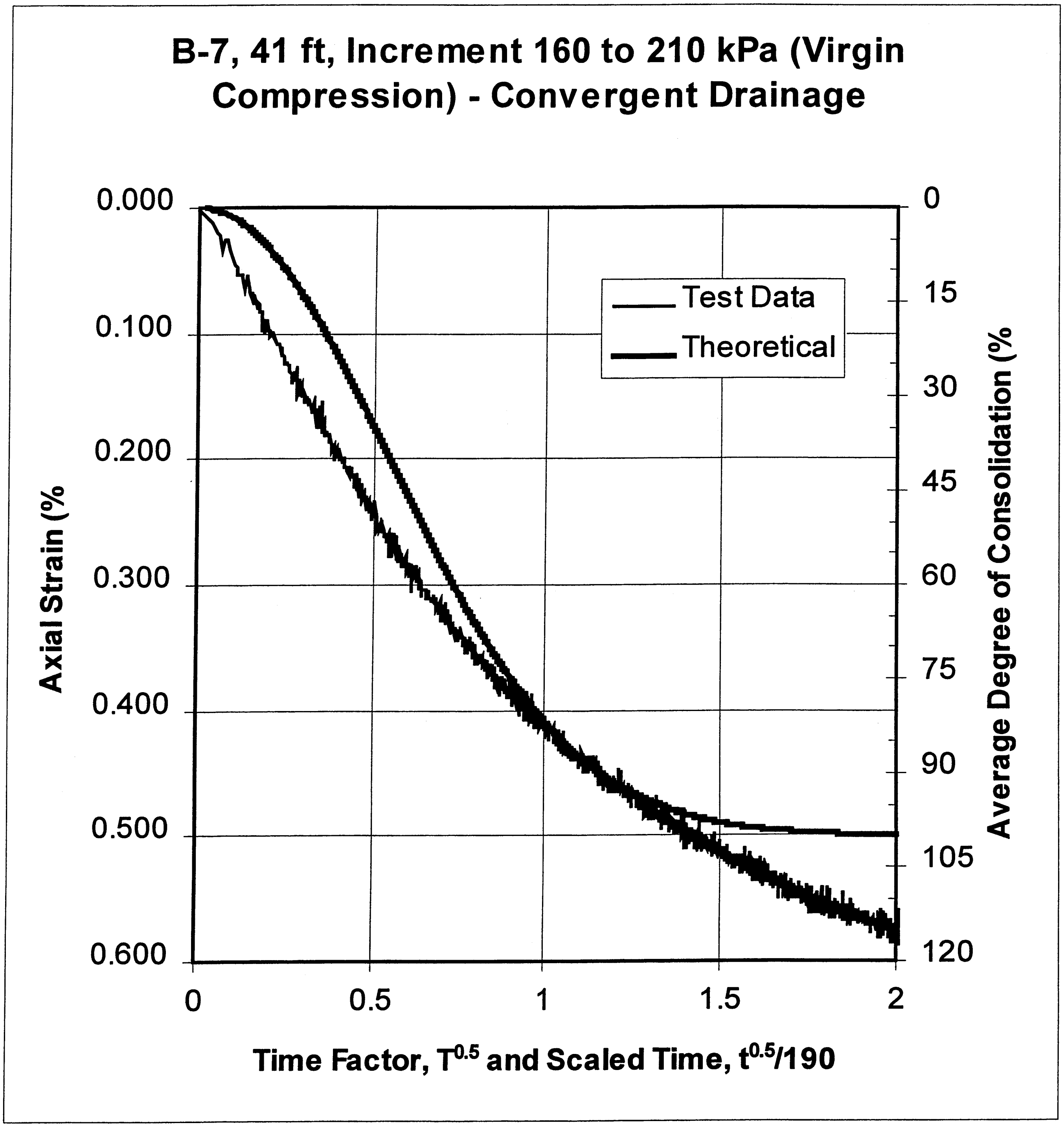


Figure 3.4: Convergent Drainage Curve Fitting Method

the curve is best in the 20% to 60% consolidation range, not the early part of the test. Fig. 3.5 shows use of this technique on the same B-7 laboratory data, resulting in a c_h value of 19 ft^2/yr.

3.4 Results of consolidation testing

The first two specimens, tested with divergent drainage only, yield values for normally consolidated conditions of c_h of 3.1 to 3.2 ft^2/year (from B-10 at 35.5 ft depth) and 10.2 to 10.3 ft^2/year (from B-9 at 30.5 ft depth). These values are somewhat below expectations, considering that we usually assume higher horizontal vs. vertical permeability in such sediments, and that the material seems to be coarser than other San Francisco bay deposits that have values of the vertical coefficient of consolidation, c_v, in the range of 8 to 10 ft^2/yr. Our assumption is that a combination of filter paper and sampling induced smear led to these reduced values of c_h.

The third and fourth consolidation tests were built to investigate the effects of these factors by consolidating the samples with different drainage paths – both divergent and convergent. The samples from B-7 at 41 ft and P-2 at 40 ft yielded divergent c_h values of 0.8 ft^2/yr and 0.5 ft^2/yr respectively. These abnormally low values are assumed due to restricted flow capacity of the filter paper, which produced a more severe effect for these samples than in the first two tests because drainage was limited to one end of the sample only. This phenomenon was in fact observed earlier by Rowe (1959). Convergent drainage values of c_h, while somewhat difficult to interpret, were estimated at 19 ft^2/yr and 39 ft^2/yr for B-7 and P-2 respectively, yielding much more reasonable values.

Table 3.2: Results of consolidation tests

Boring	Sample #	Test Type	Isotropic Loading Increment (kPa)	c_h (ft^2/yr)
P-2	15	Convergent	150-200	39
B-7	12	Convergent	160-210	19
P-2	15	Divergent	200-250	0.5
B-7	12	Divergent	210-260	0.8
B-9	8	Divergent	125-200	11
B-10	10	Divergent	135-205	3

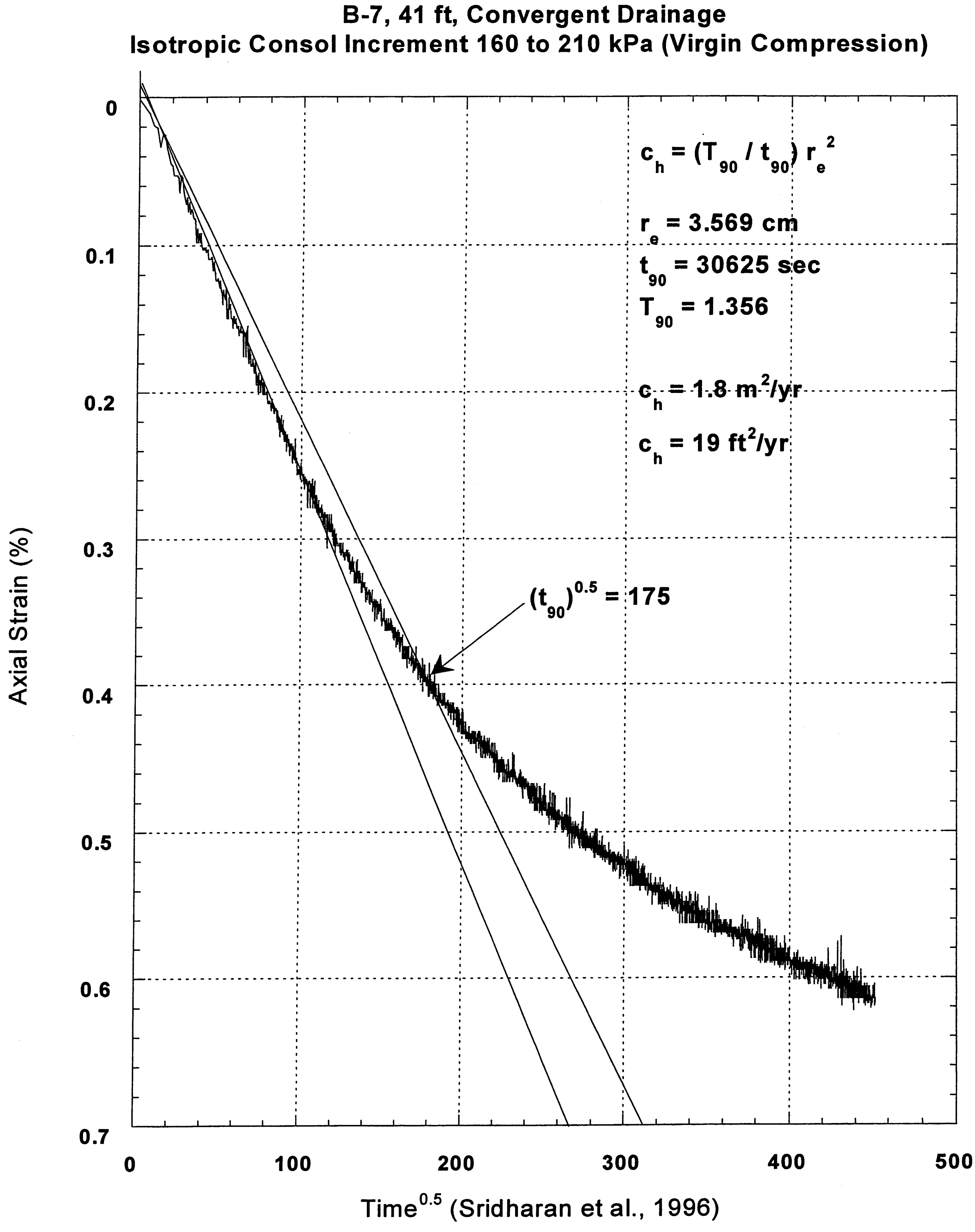


Figure 3.5: Convergent Drainage Simplified Method

In summary, it became apparent during the consolidation testing of the final two specimens (from P-2 and B-7) that the presence of filter paper on the circumference of the soil was probably not sufficient to provide a free draining boundary, and was in fact restricting the consolidation flow from the soil. As such, it seems wise to discount the values of c_h obtained from the divergent drainage conditions, and rely more directly on the higher values obtained from the convergent cases.

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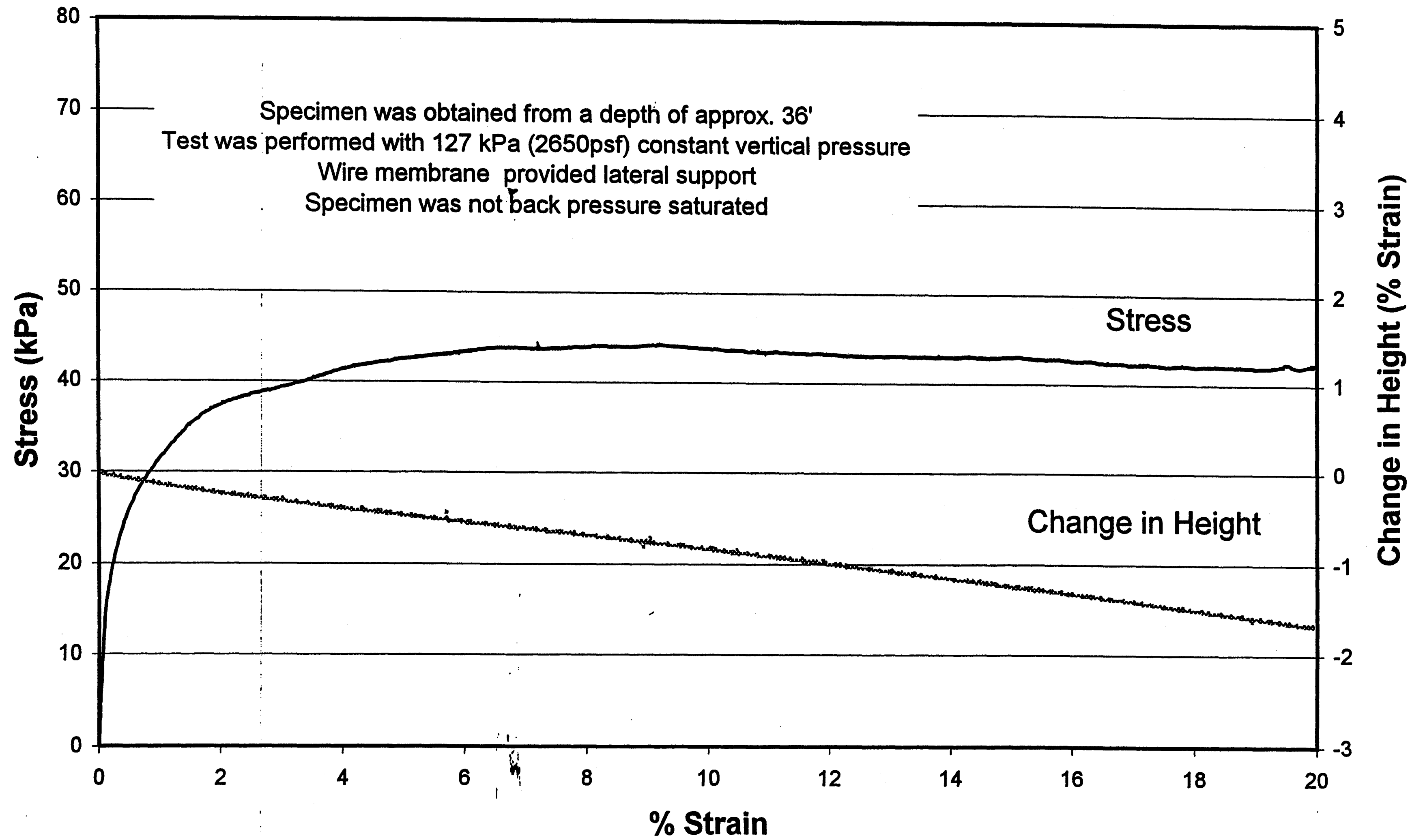
APPENDIX A

Simple Shear Testing Results

SFOBB Boring B-5 Specimen 1
Monotonic Test Only



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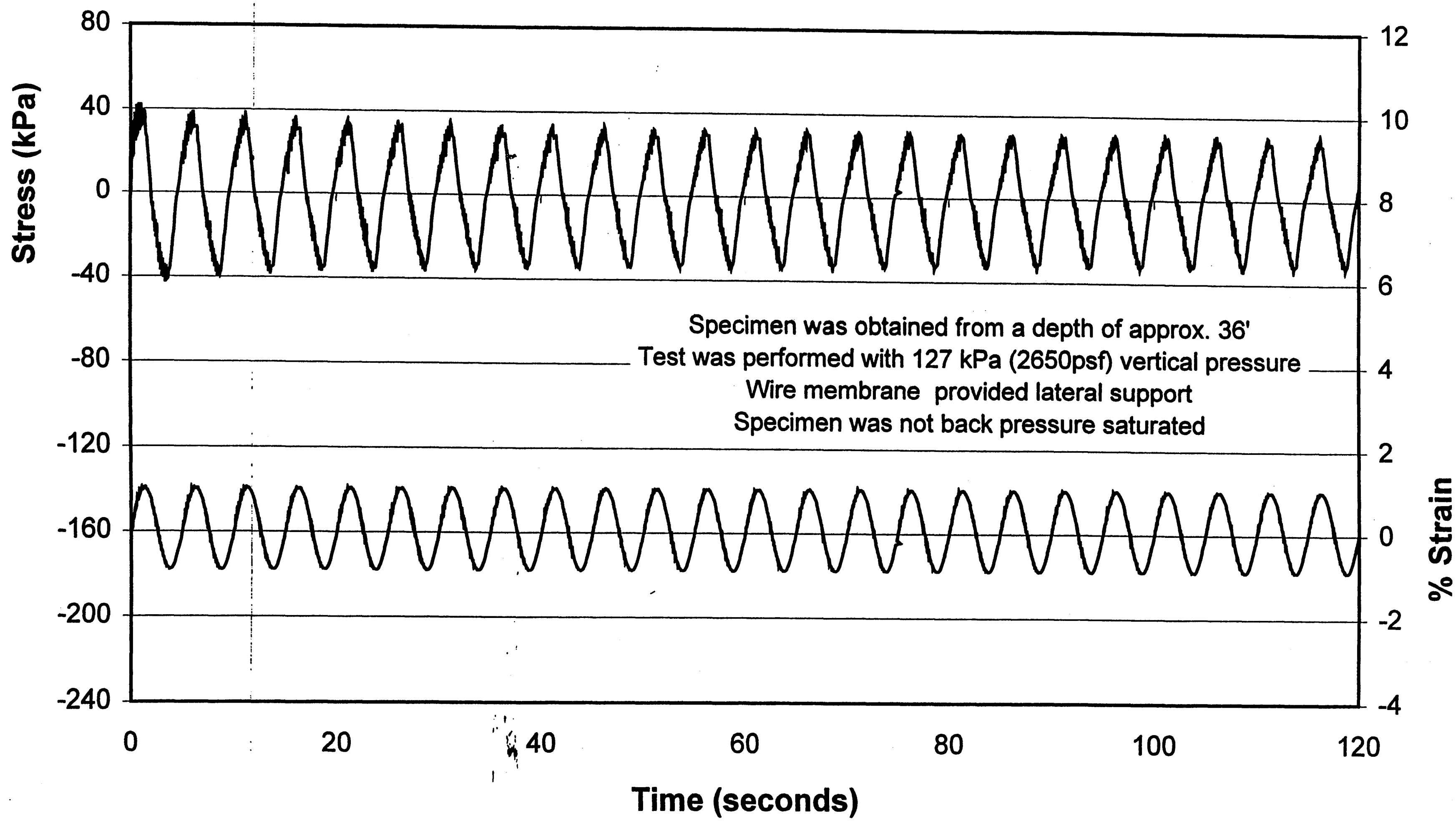
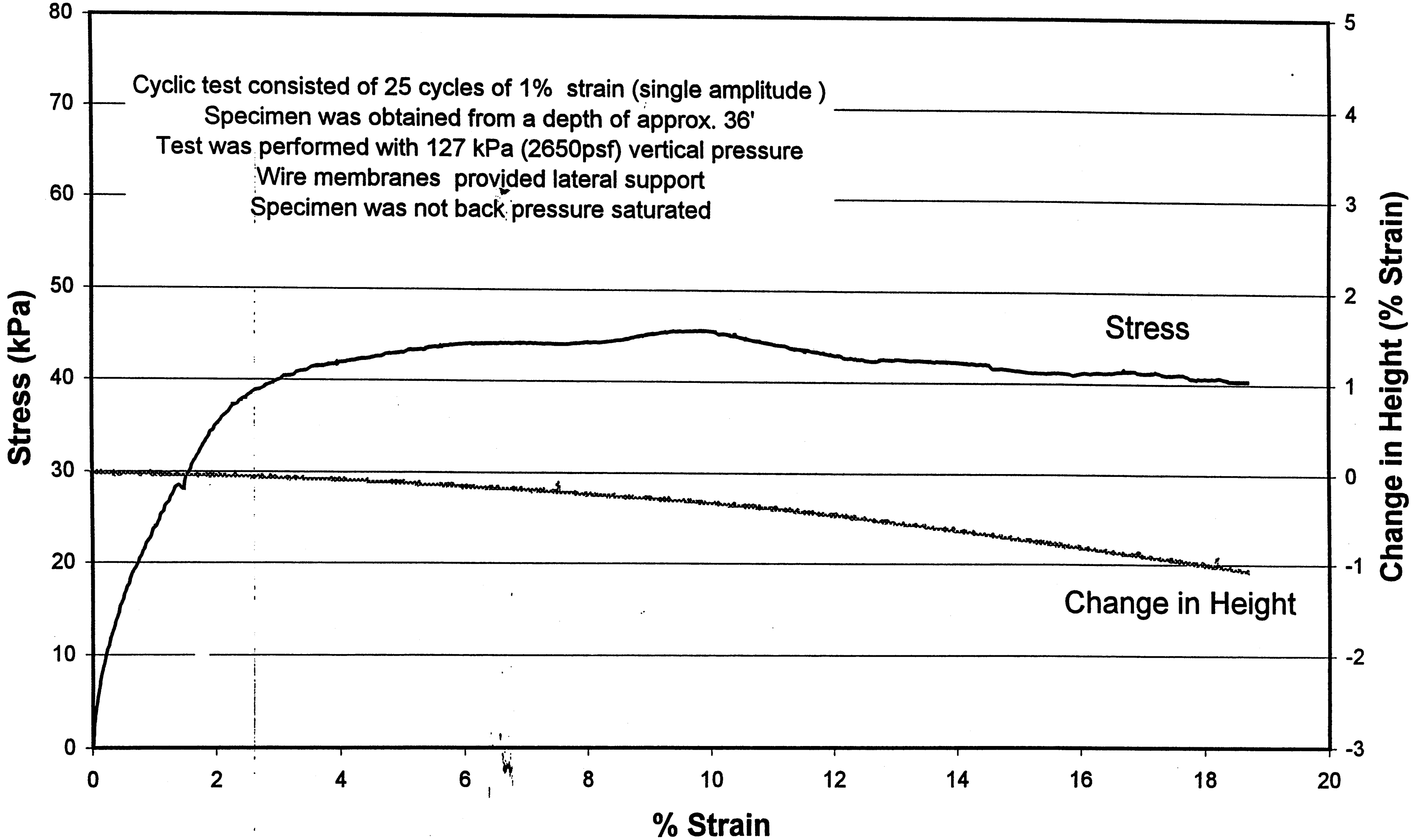
BBMONO1.xls

SFOBB Boring B-5 Specimen 4
Cyclic Test to 1% Strain (Single Amplitude)



SFOBB Boring B-5 Specimen 4
Monotonic Test after Cyclic Test



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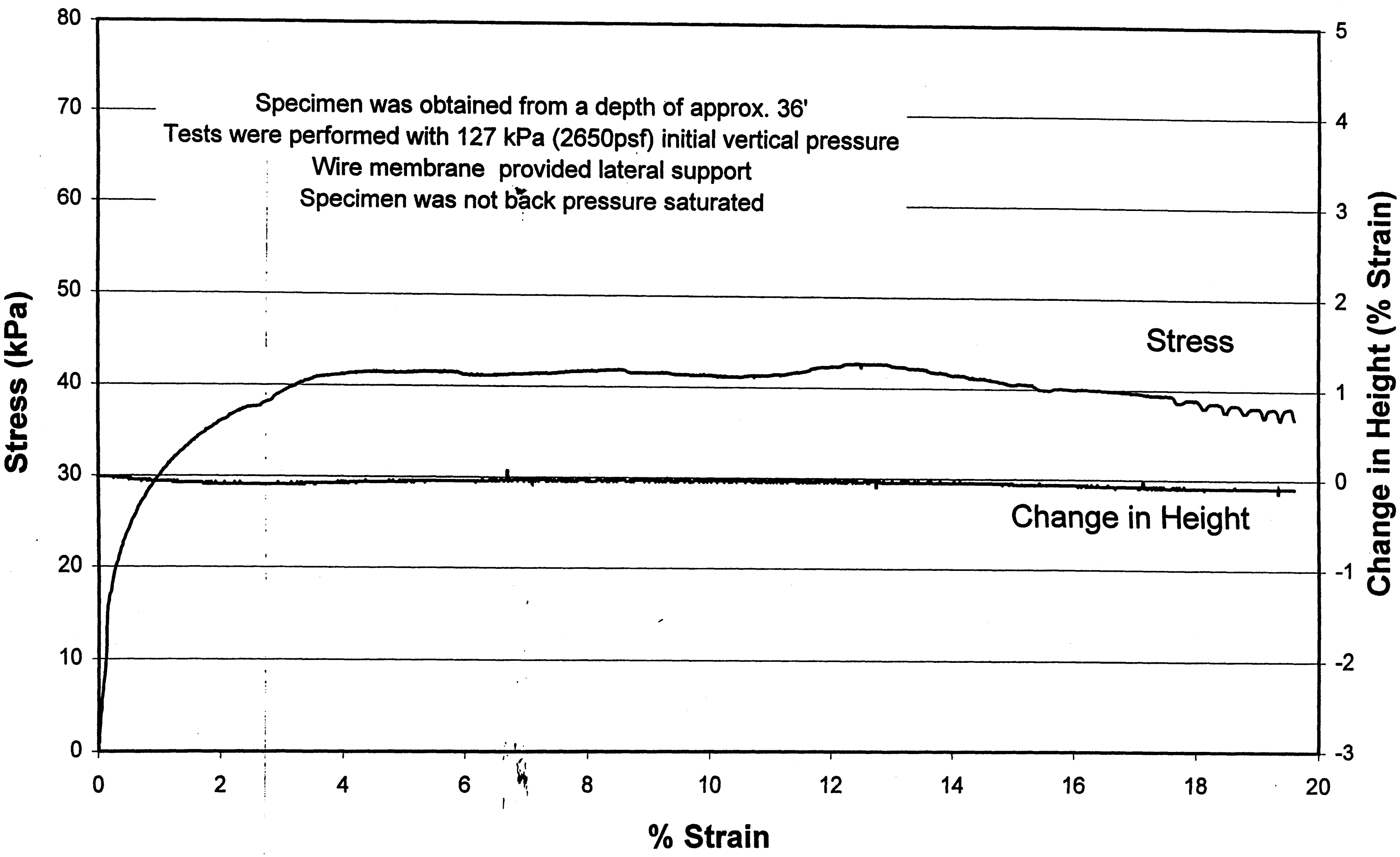
Bbmono4.xls

SFOBB Boring B-5 Specimen 5
Monotonic Test Under Constant Vertical Height



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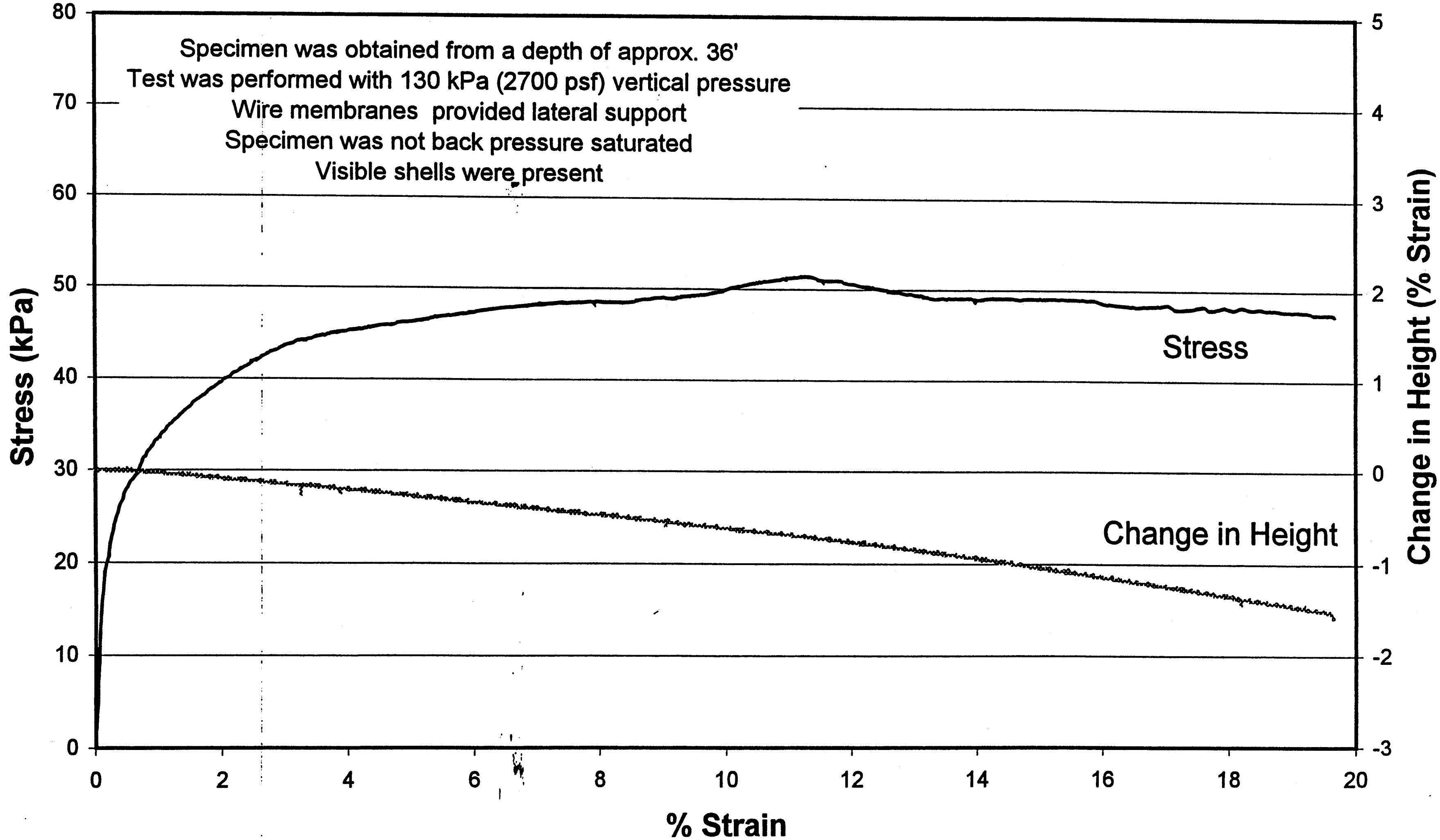
Bbmono5.xls

SFOBB Boring B-2 Specimen 6
Monotonic Test Only



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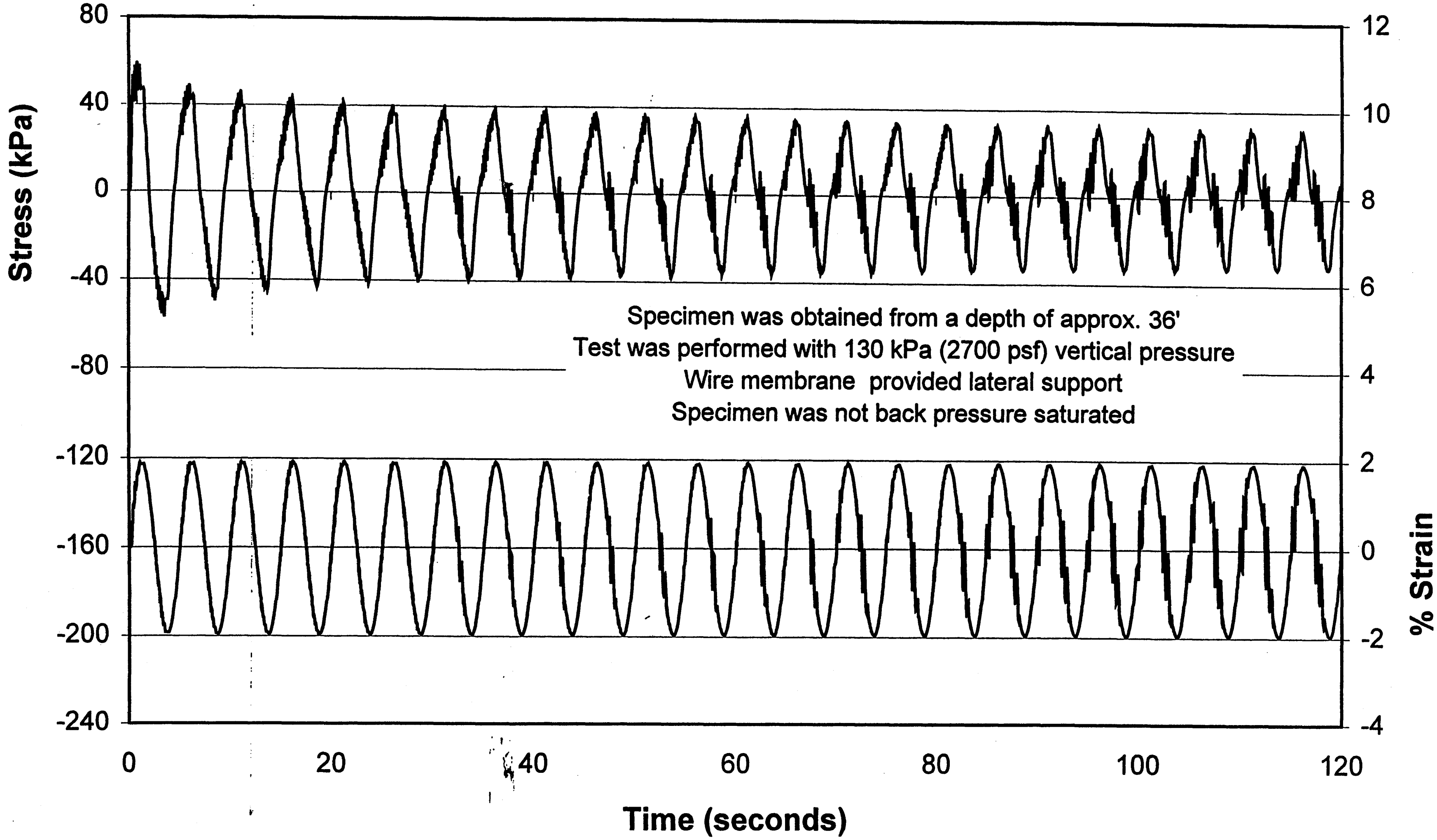
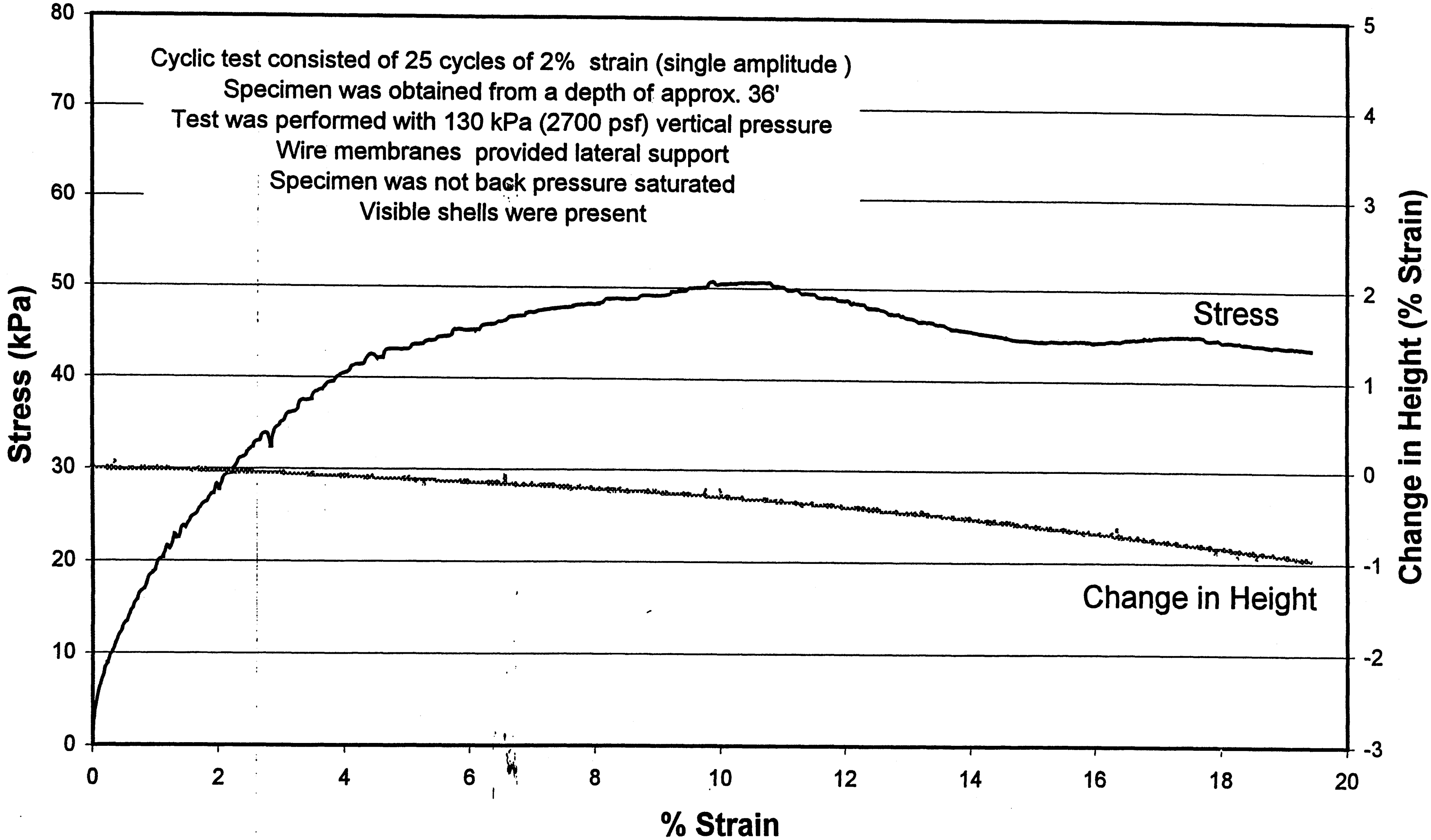
BBMONO6.xls

SFOBB Boring B-2 Specimen 8
Cyclic Test to 2% Single Amplitude Strain



SFOBB Boring B-2 Specimen 8
Monotonic Test After Cyclic



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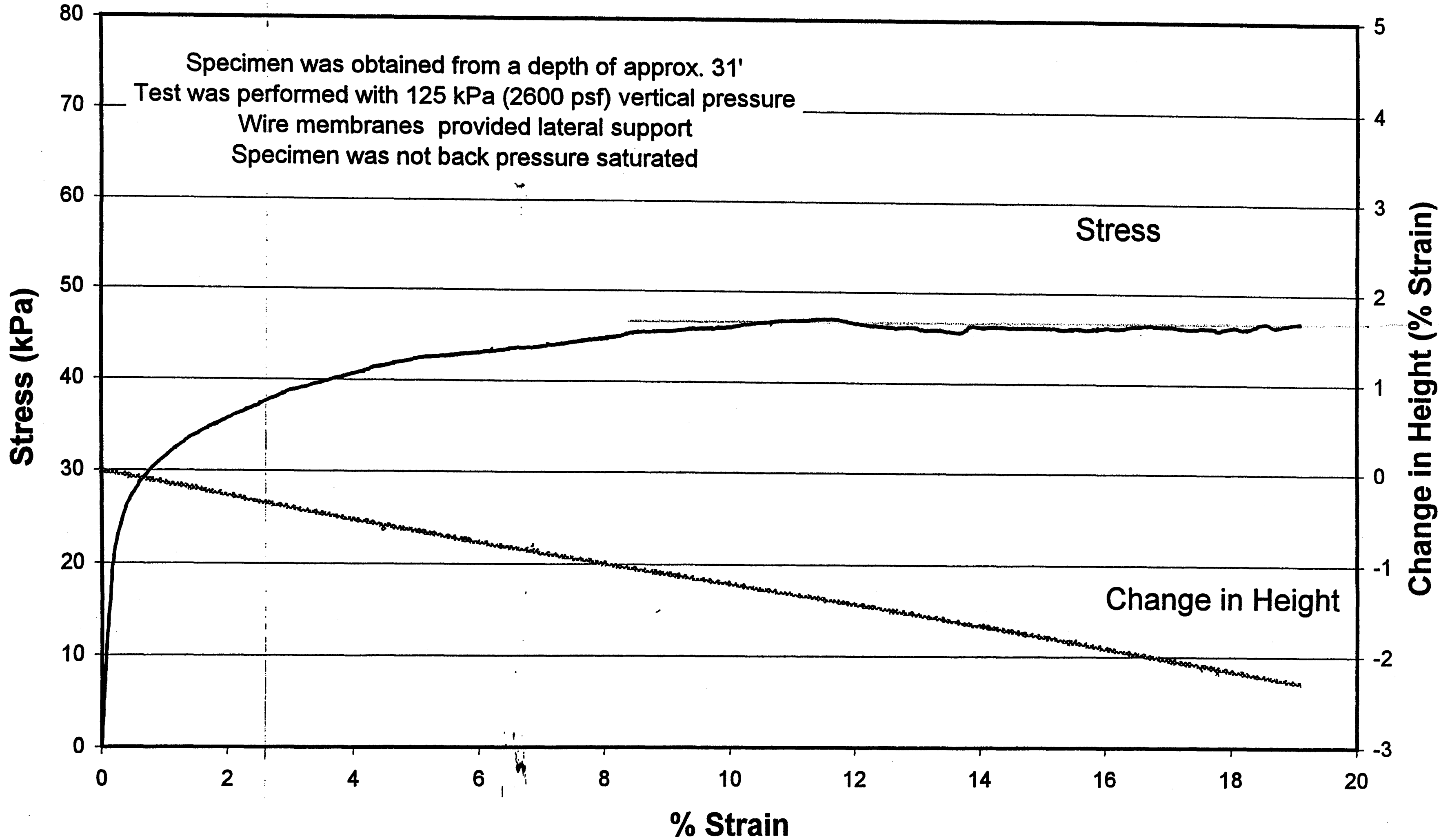
BBMONO8.xls

SFOBB Boring B-9 Specimen 9
Monotonic Test Only



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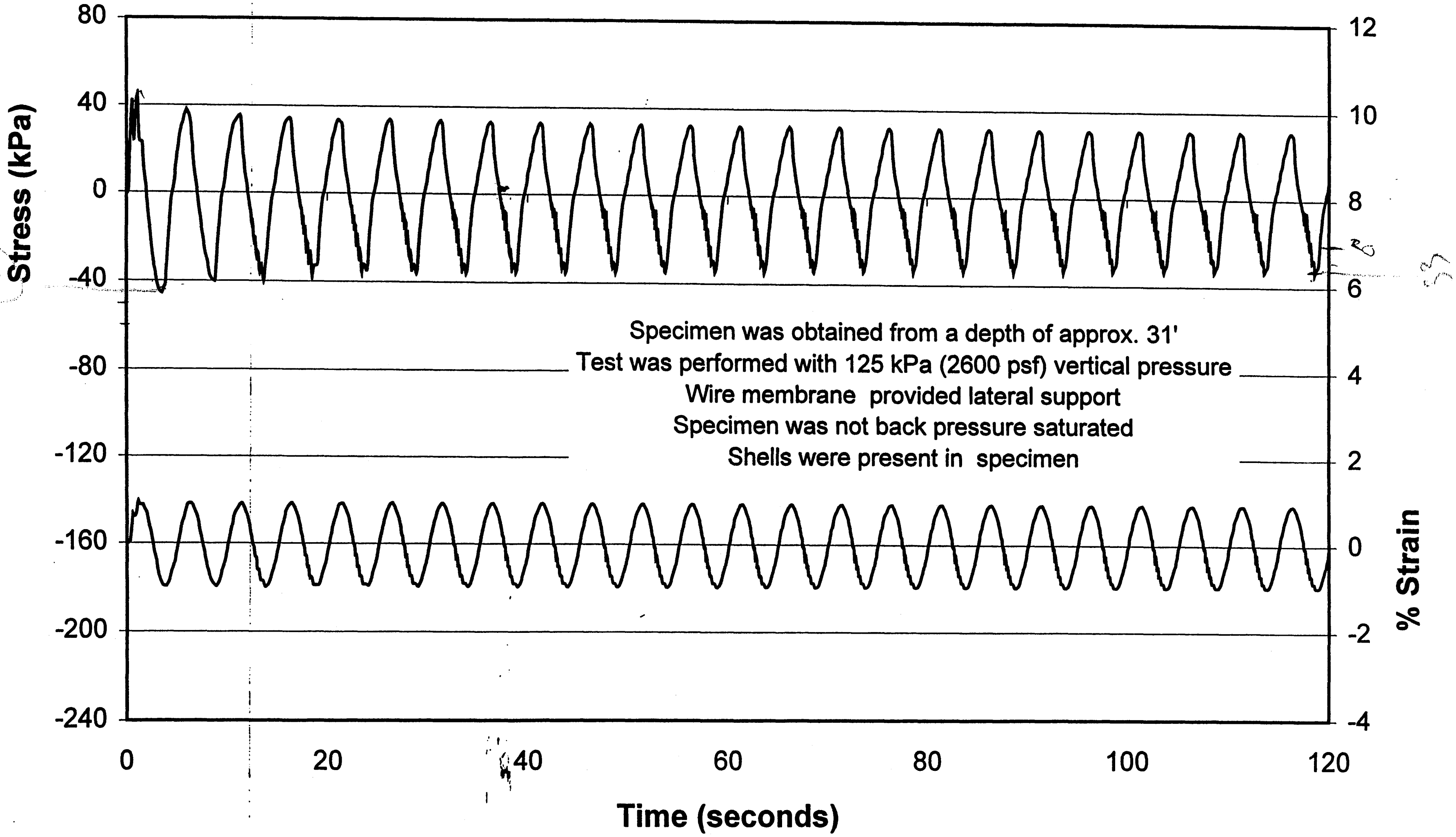
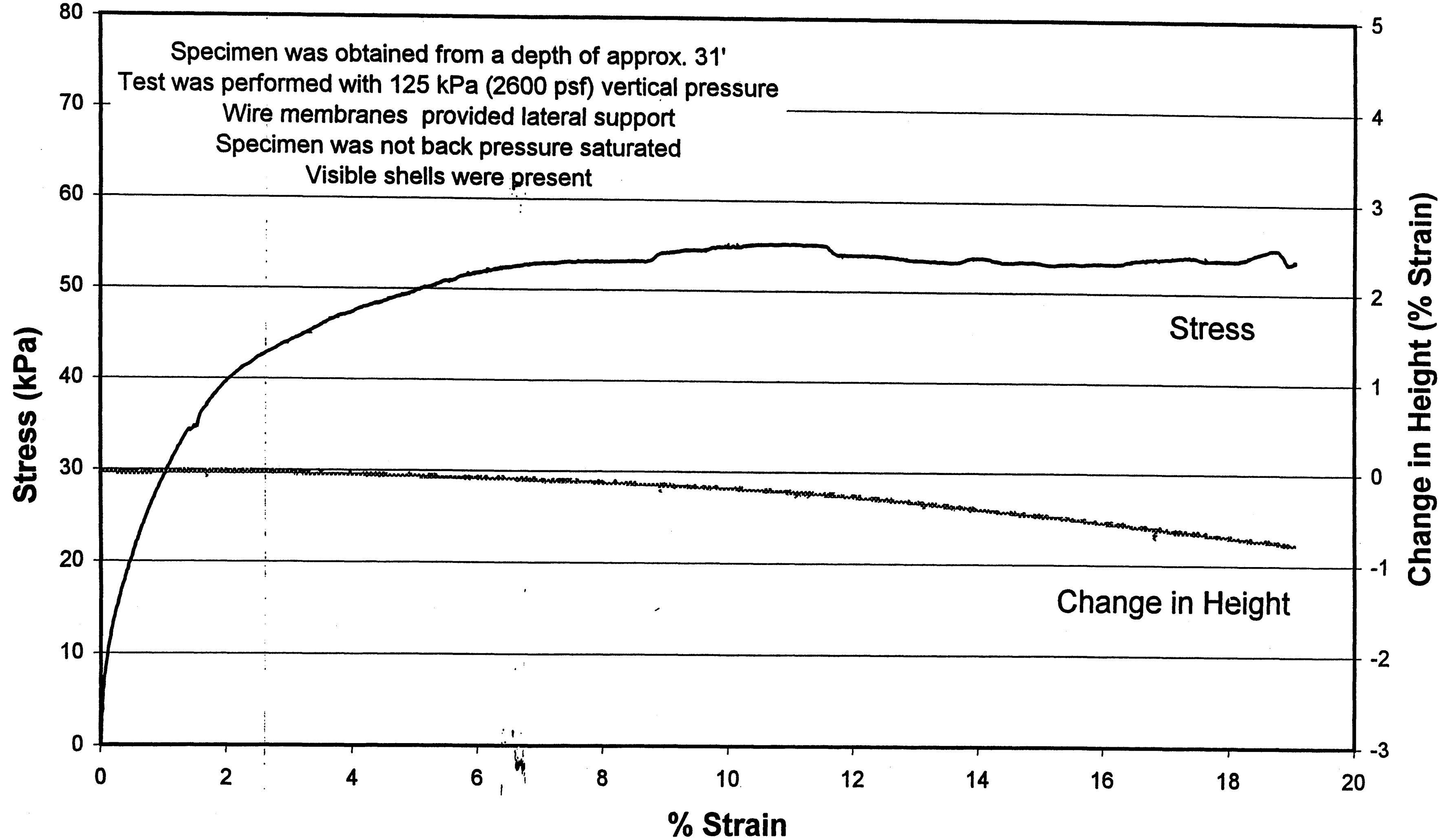
BBMONO9.xls

SFOBB Boring B-9 Specimen 10
Cyclic Test to 1% Single Amplitude Strain



SFOBB Boring B-9 Specimen 10
Monotonic Test After Cyclic



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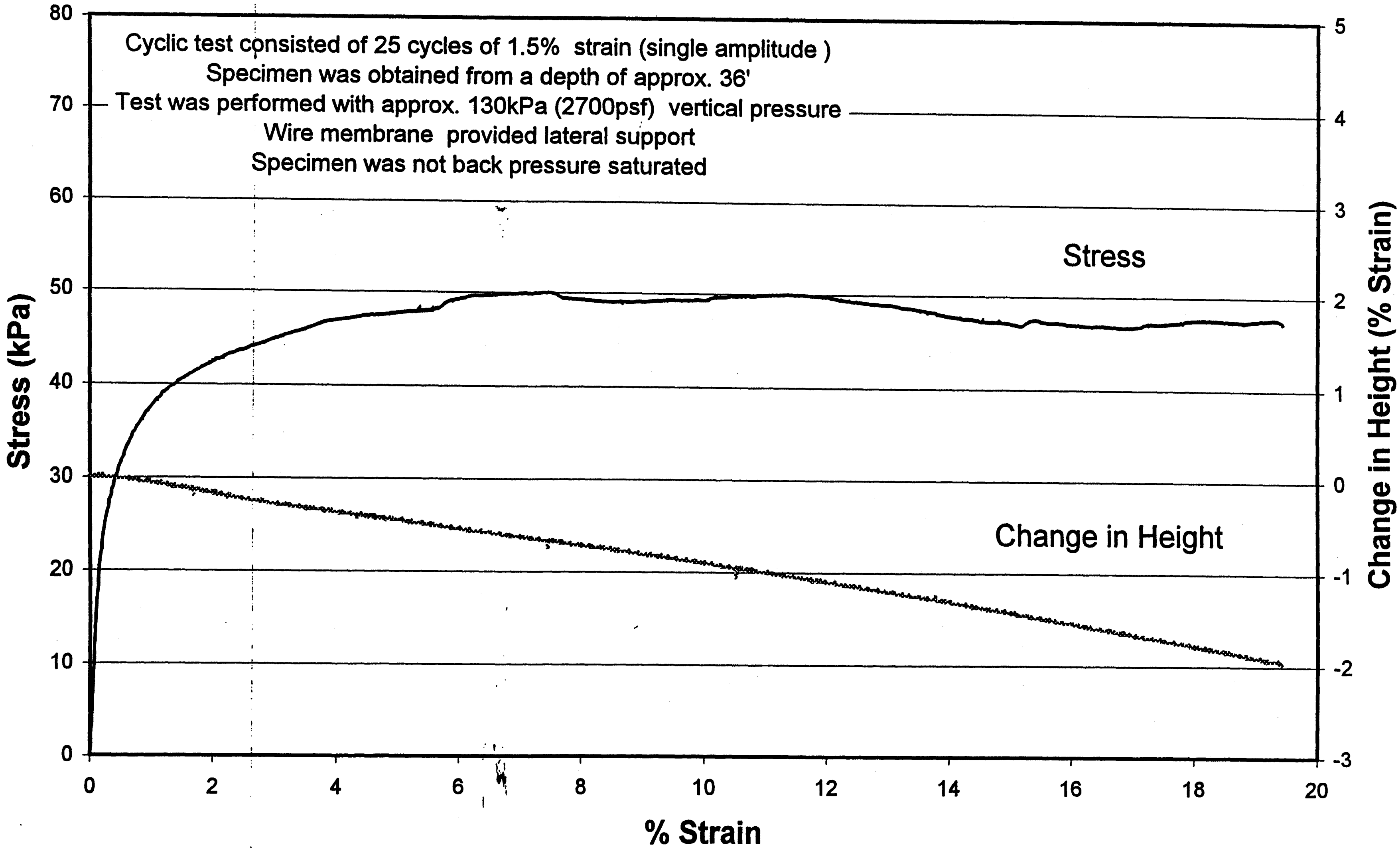
Bbmono10.xls

SFOBB Boring B-10 Specimen 11
Monotonic Test Only



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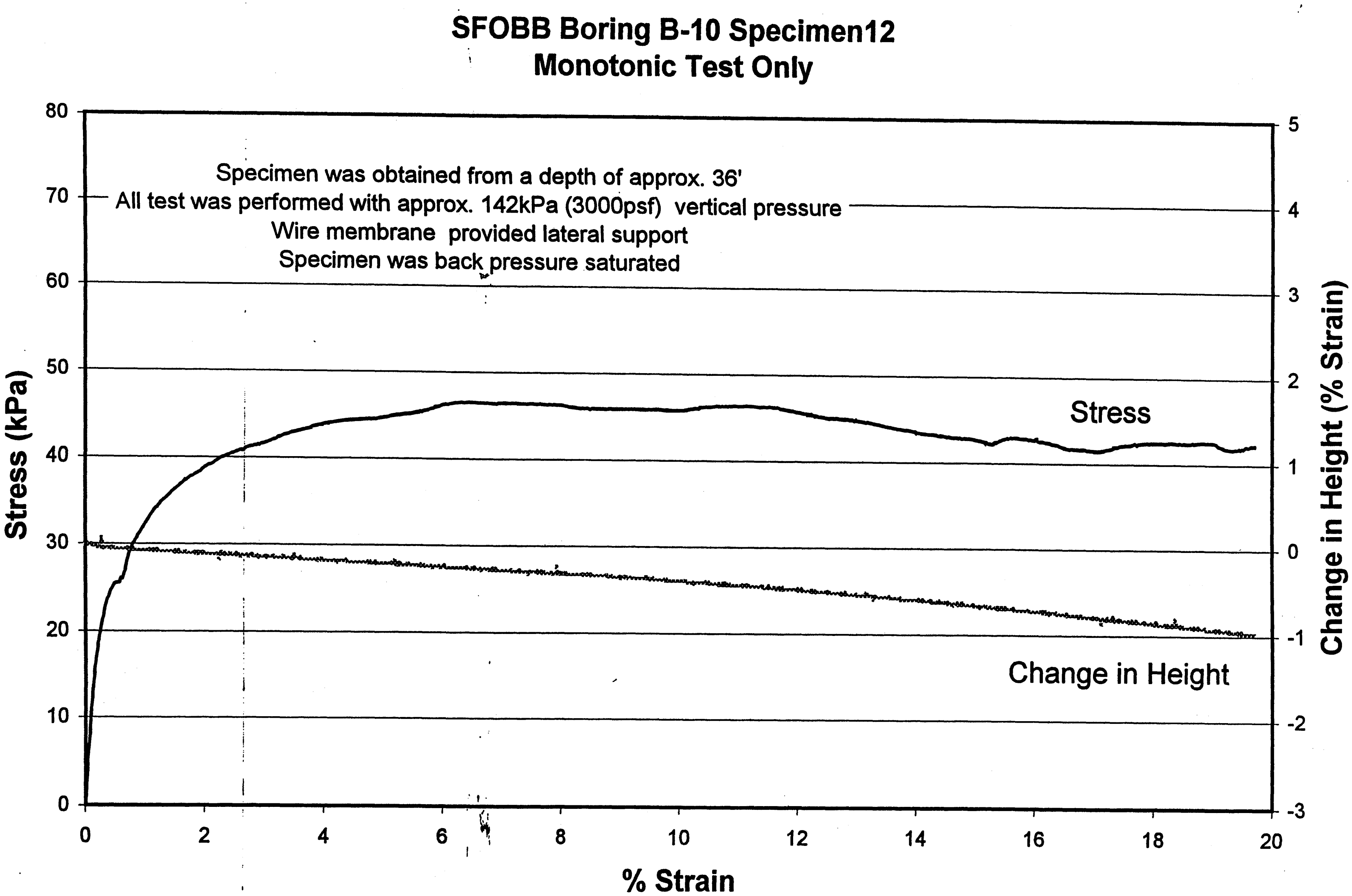
BBMONO11.xls



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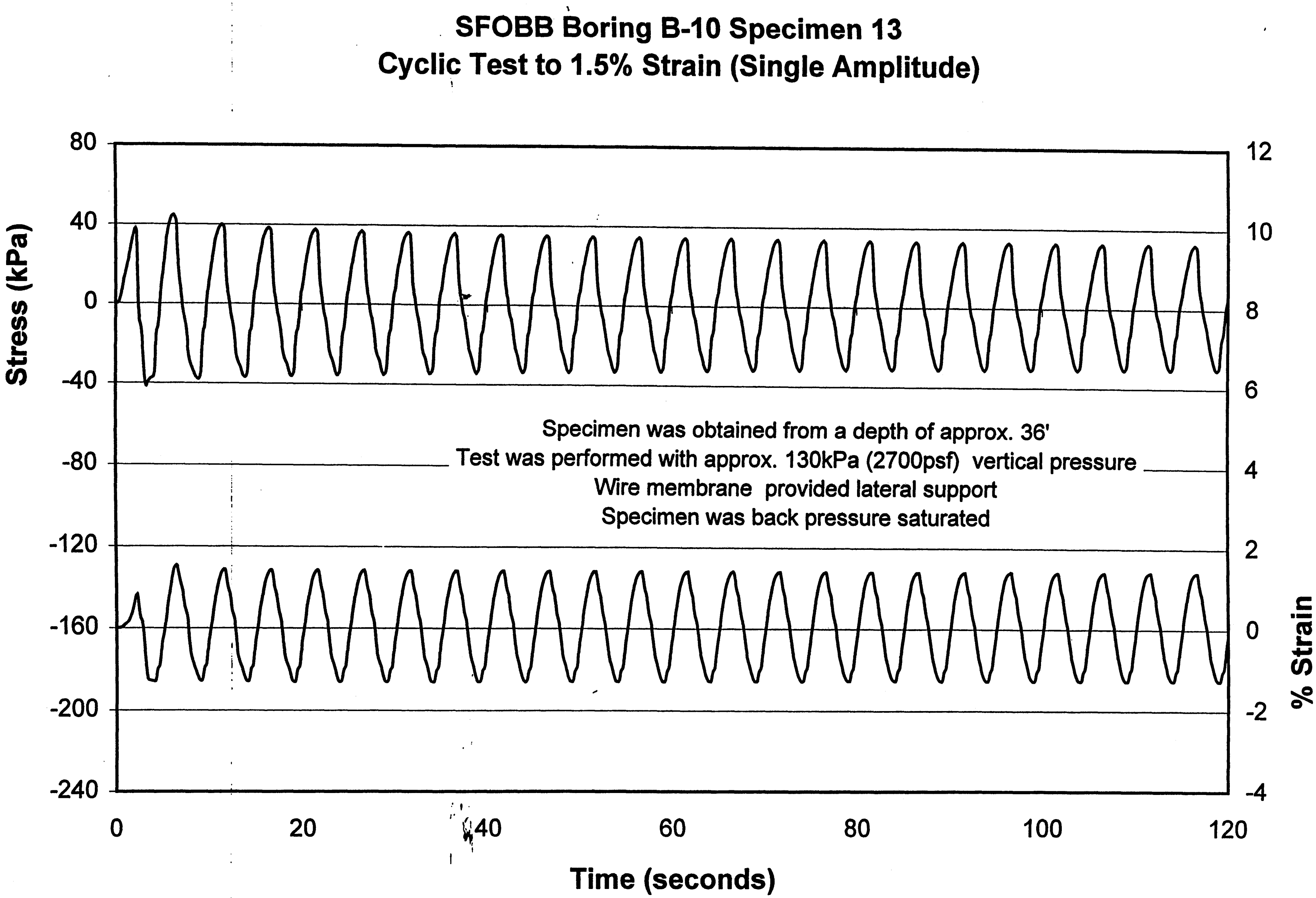
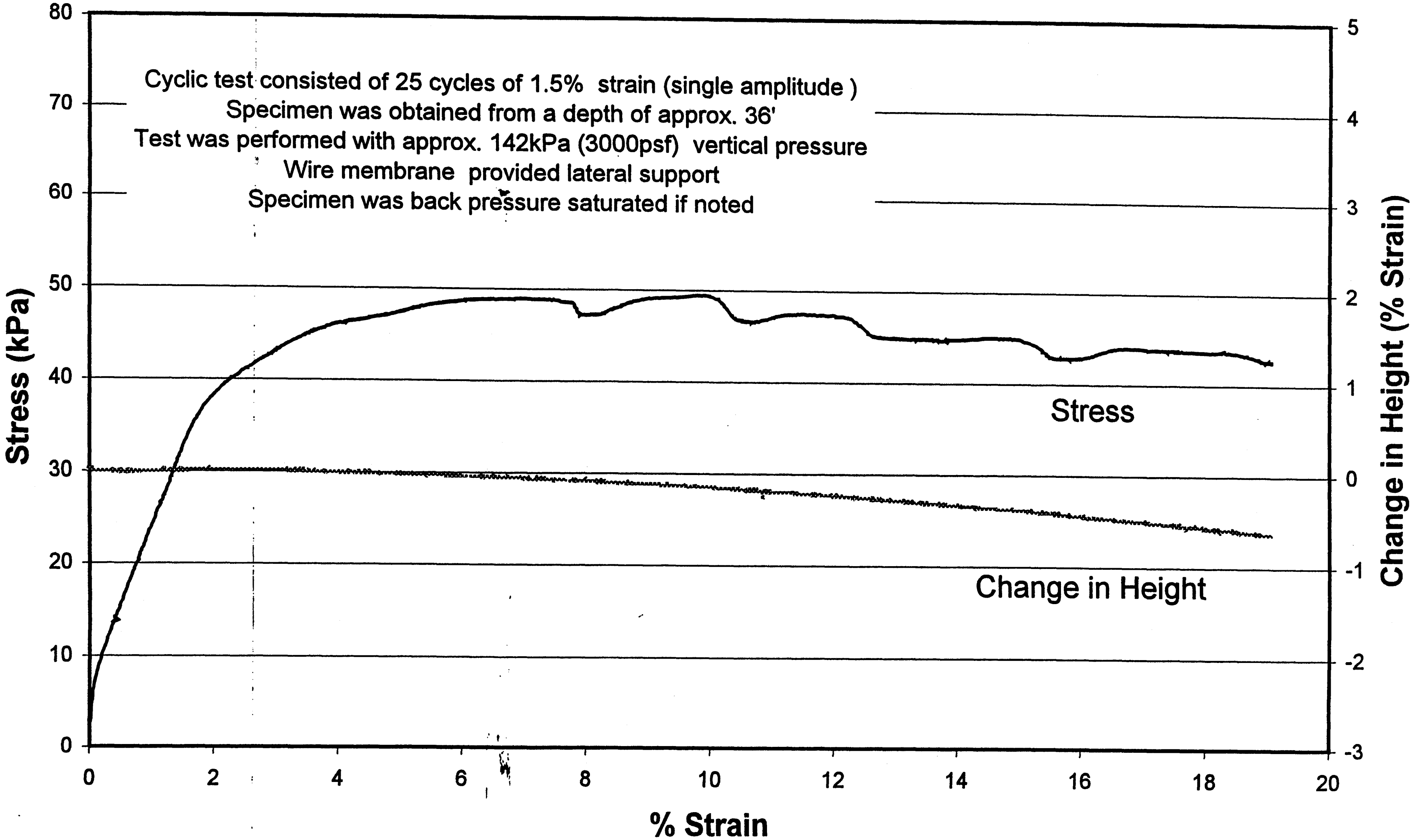
BBMONO12.xls


SFOBB Boring B-10 Specimen 13
Cyclic Test to 1.5% Strain (Single Amplitude)
Stress (kPa)
80
40
0
-40
-80
-120
-160
-200
-240
% Strain
12
10
8
6
4
2
0
-2
-4
0
20
40
60
80
100
120
Time (seconds)
Specimen was obtained from a depth of approx. 36'
Test was performed with approx. 130kPa (2700psf) vertical pressure
Wire membrane provided lateral support
Specimen was back pressure saturated

SFOBB Boring B-10 Specimen 13
Monotonic Test After Cyclic Test



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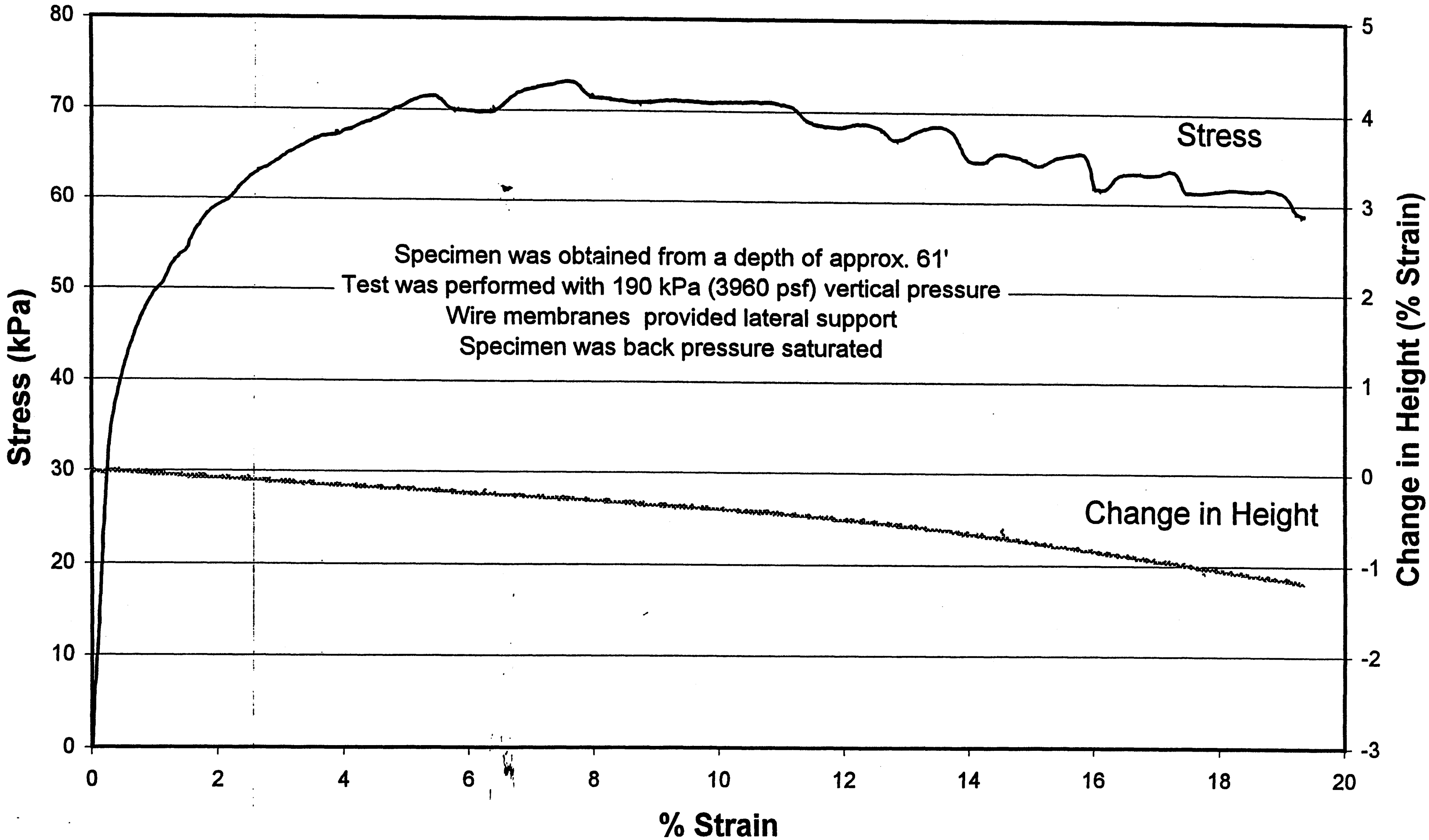
BBMONO13.xls

SFOBB Boring B-6 Specimen 14
Monotonic Test Only



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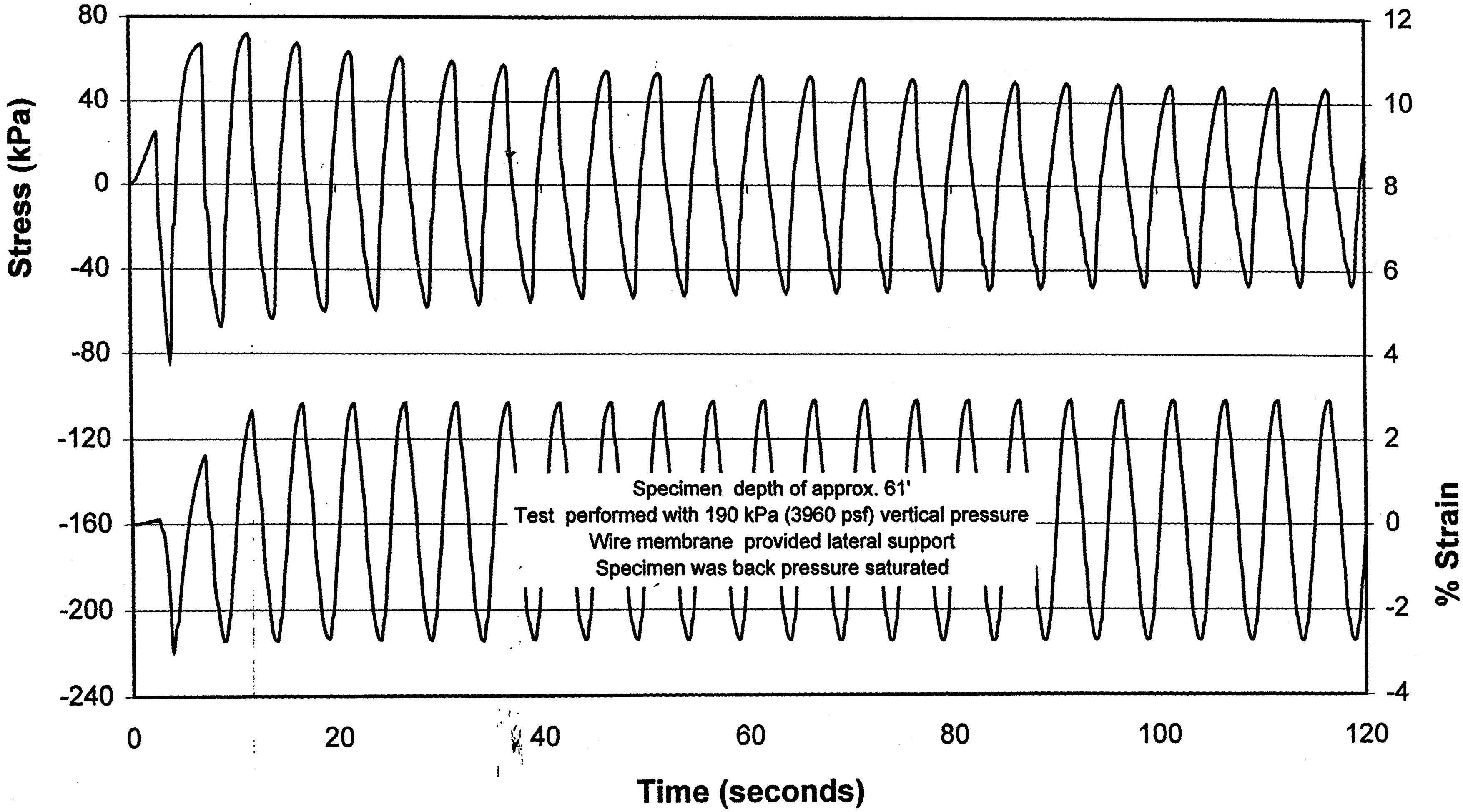
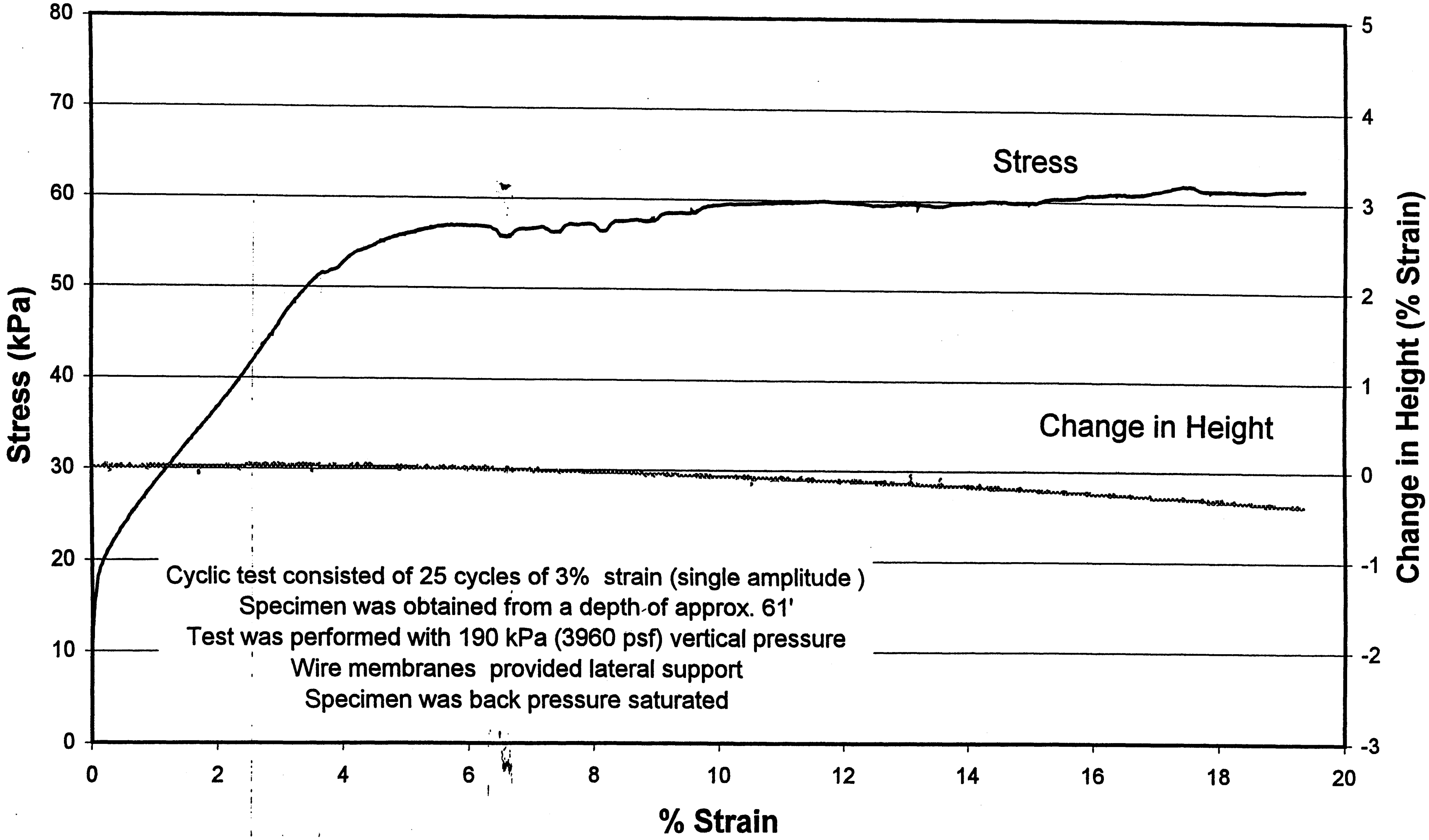
BBMONO14.xls

SFOBB Boring B-6 Specimen 15
Cyclic Test to 3% Single Amplitude Strain



SFOBB Boring B-6 Specimen15
Monotonic Test After Cyclic



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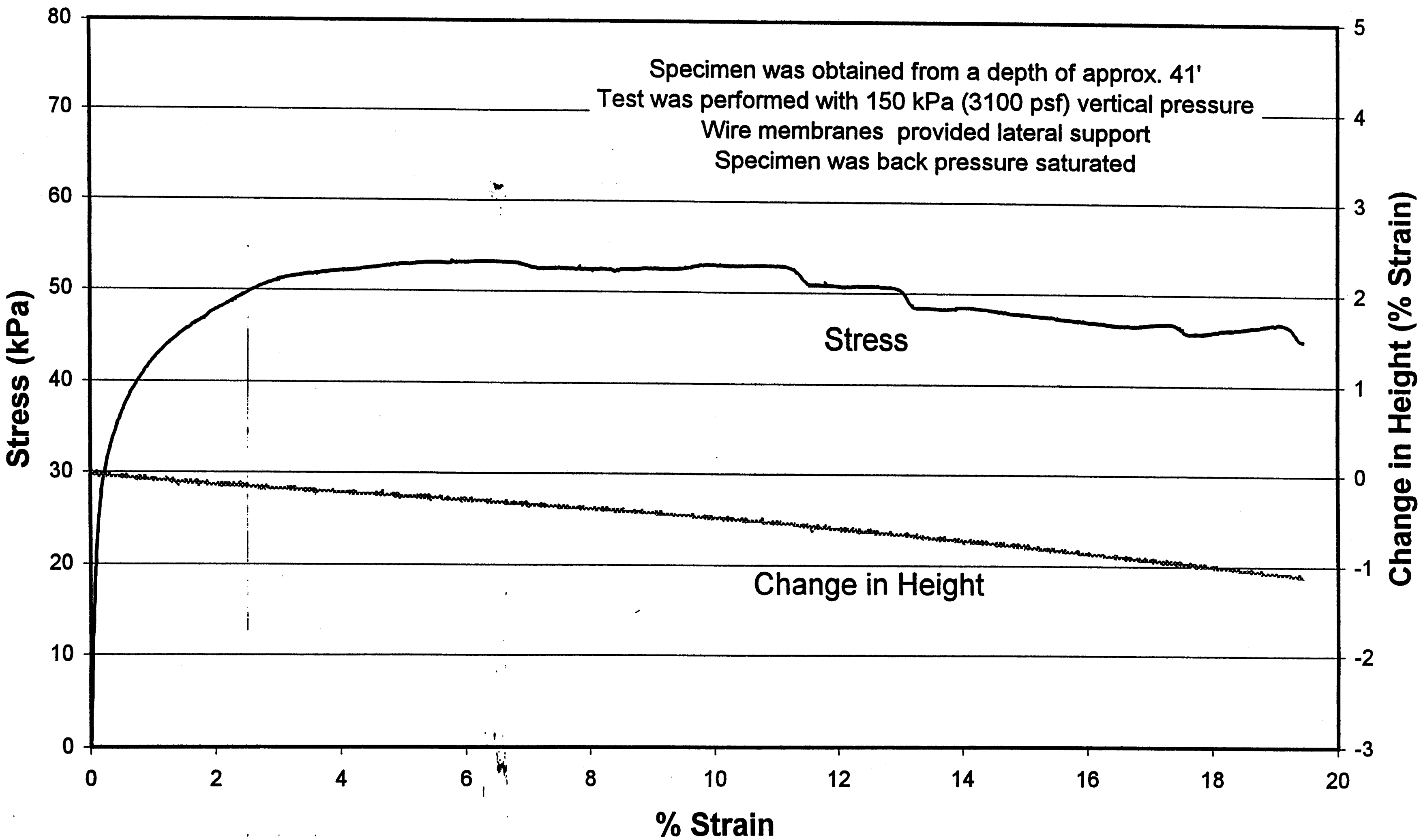
BBMONO15.xls

SFOBB Boring P-2 Specimen 16
Monotonic Test Only



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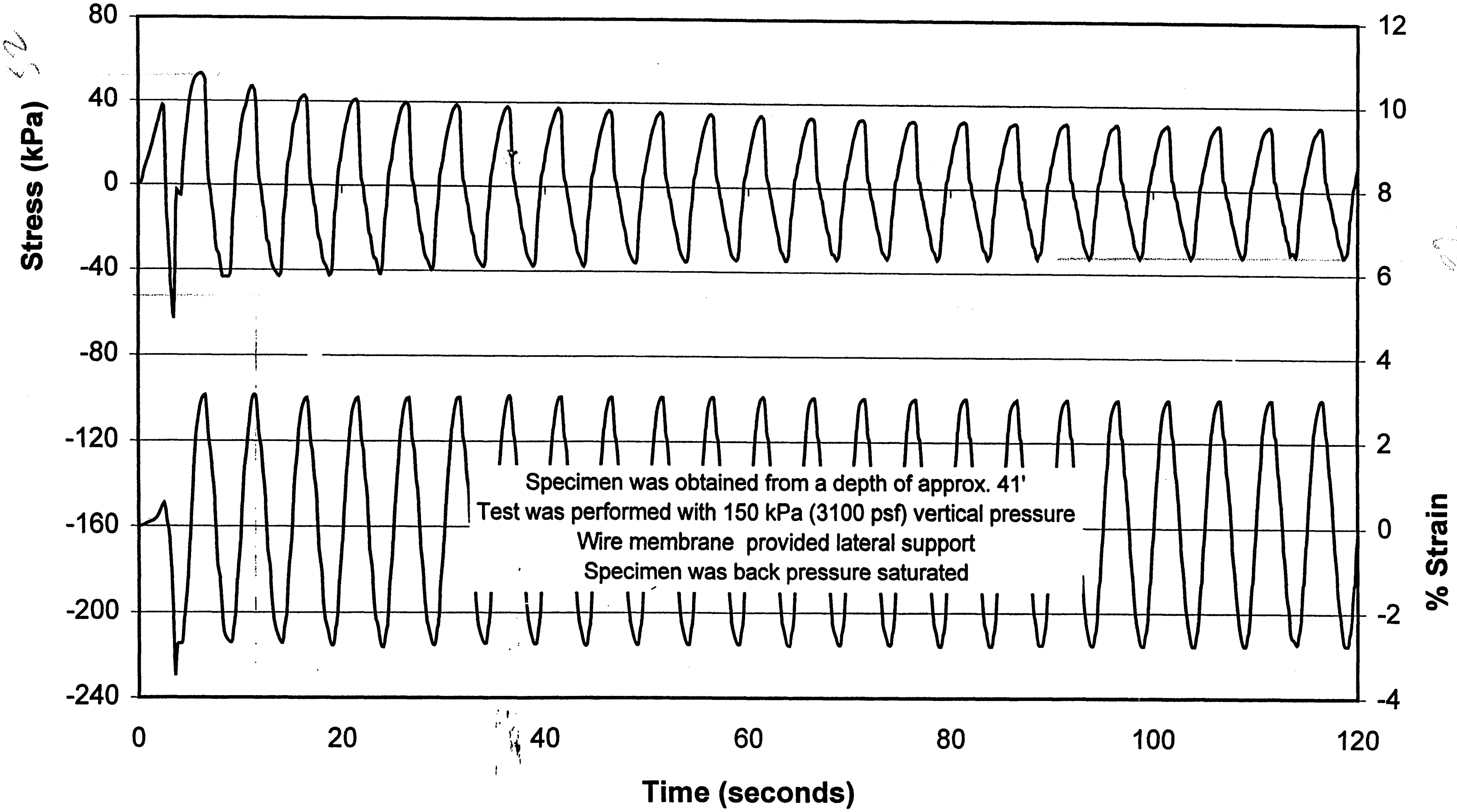
BBMONO16.xls

SFOBB Boring P-2 Specimen 17
Cyclic Test to 3% Single Amplitude Strain



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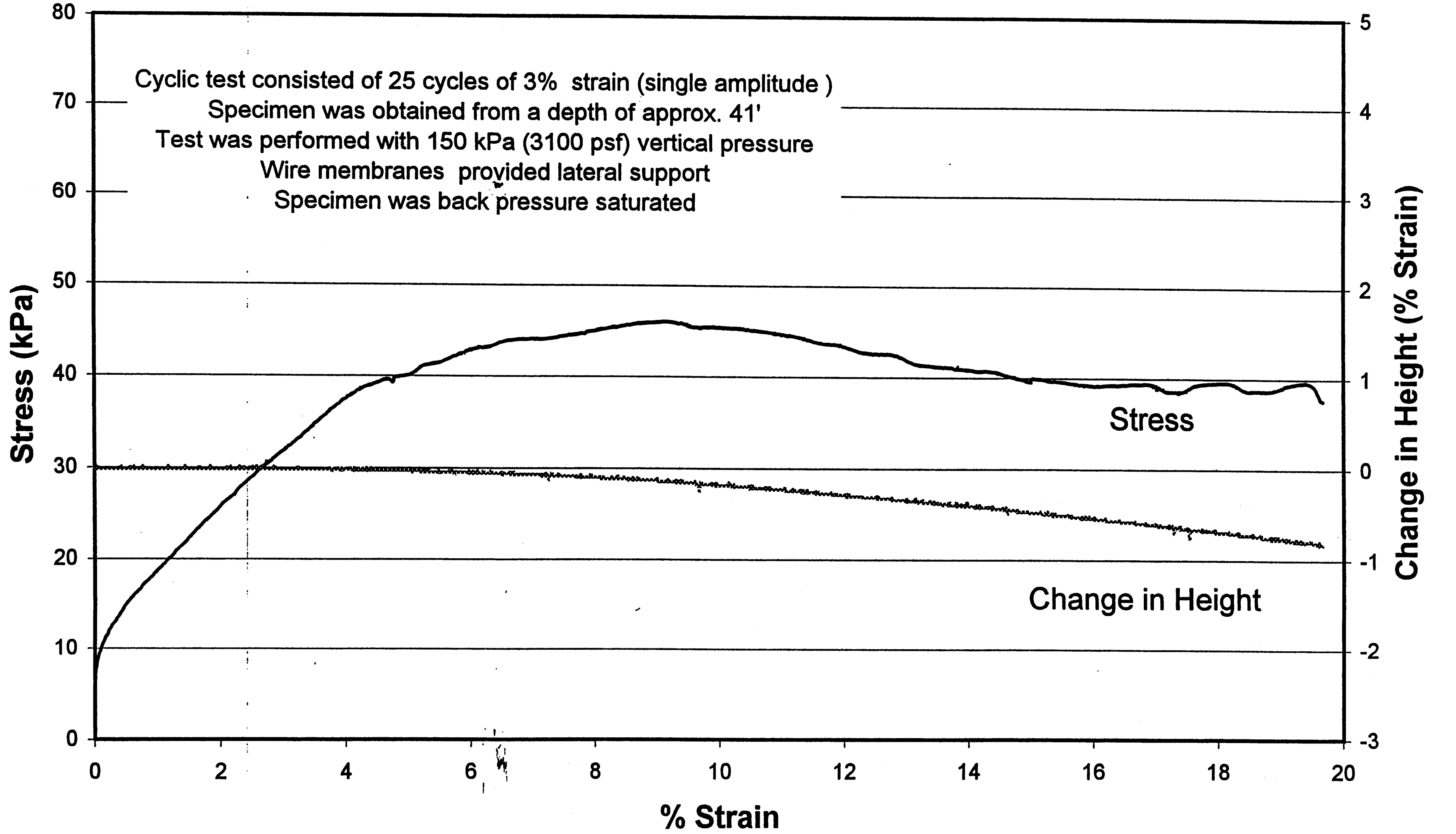
Summary Plots.xls

SFOBB Boring P-2 Specimen 17
Monotonic Test After Cyclic



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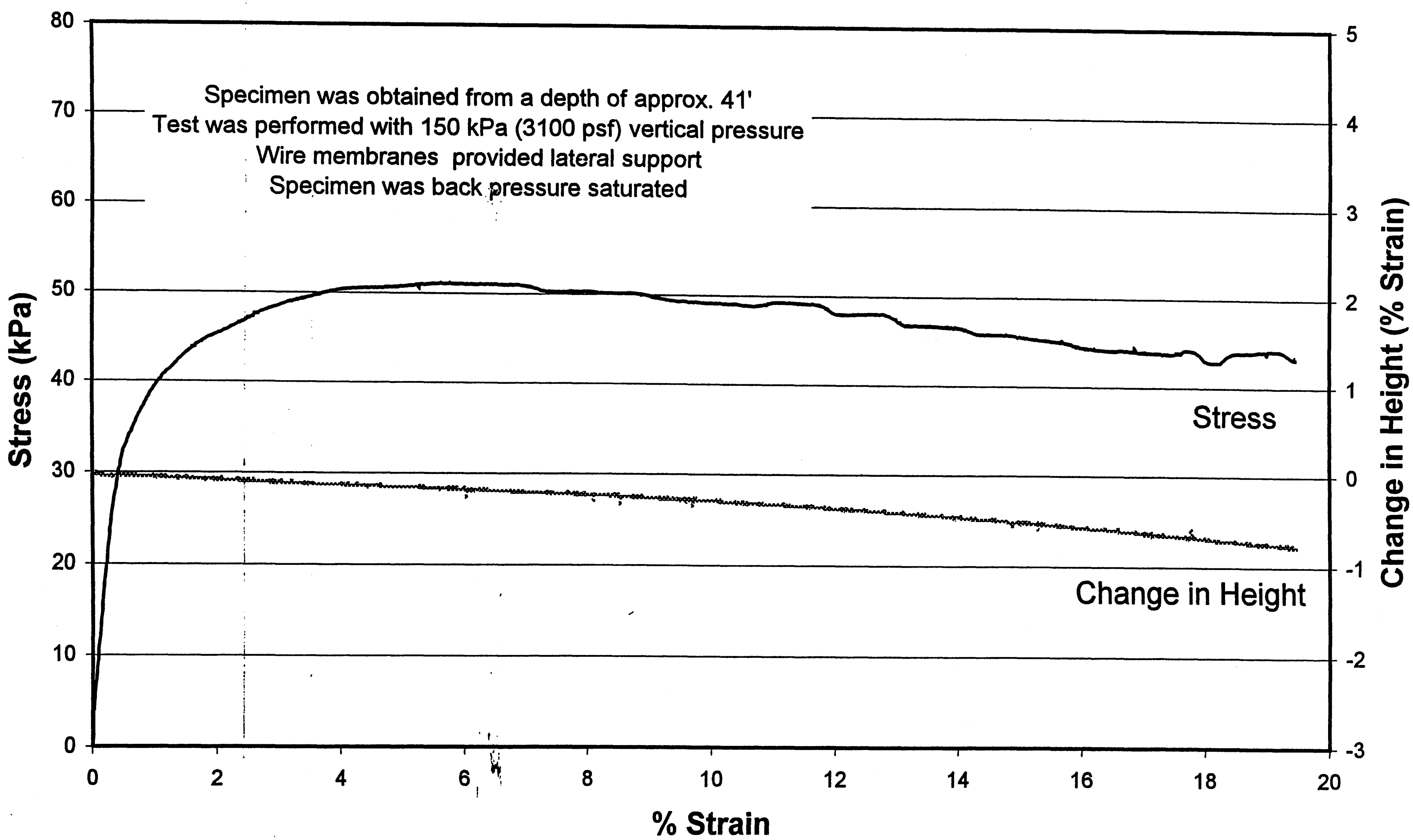
BBMONO17.xls

SFOBB Boring P-2 Specimen 18
Monotonic Test Only



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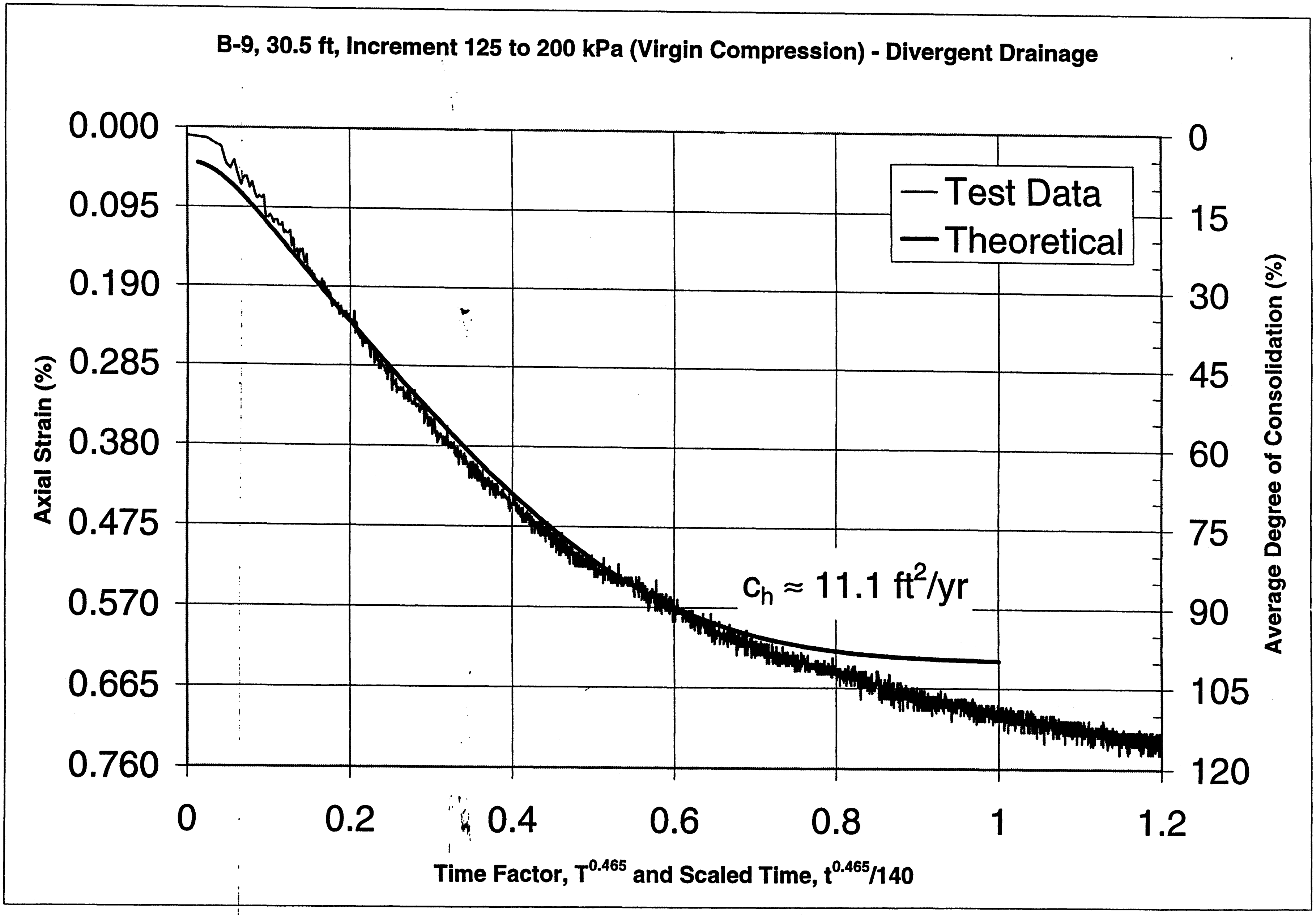
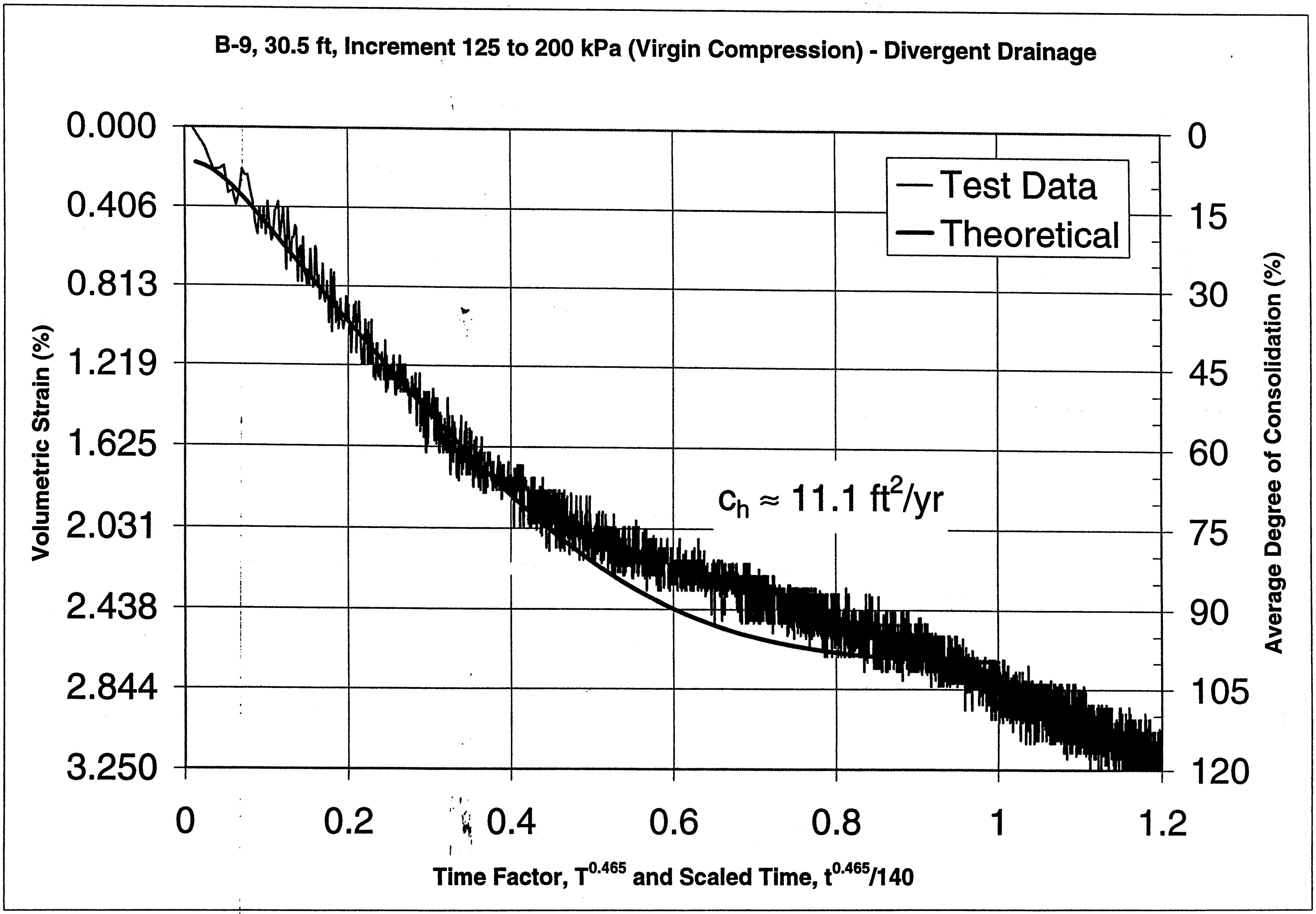
BBMONO18.xls

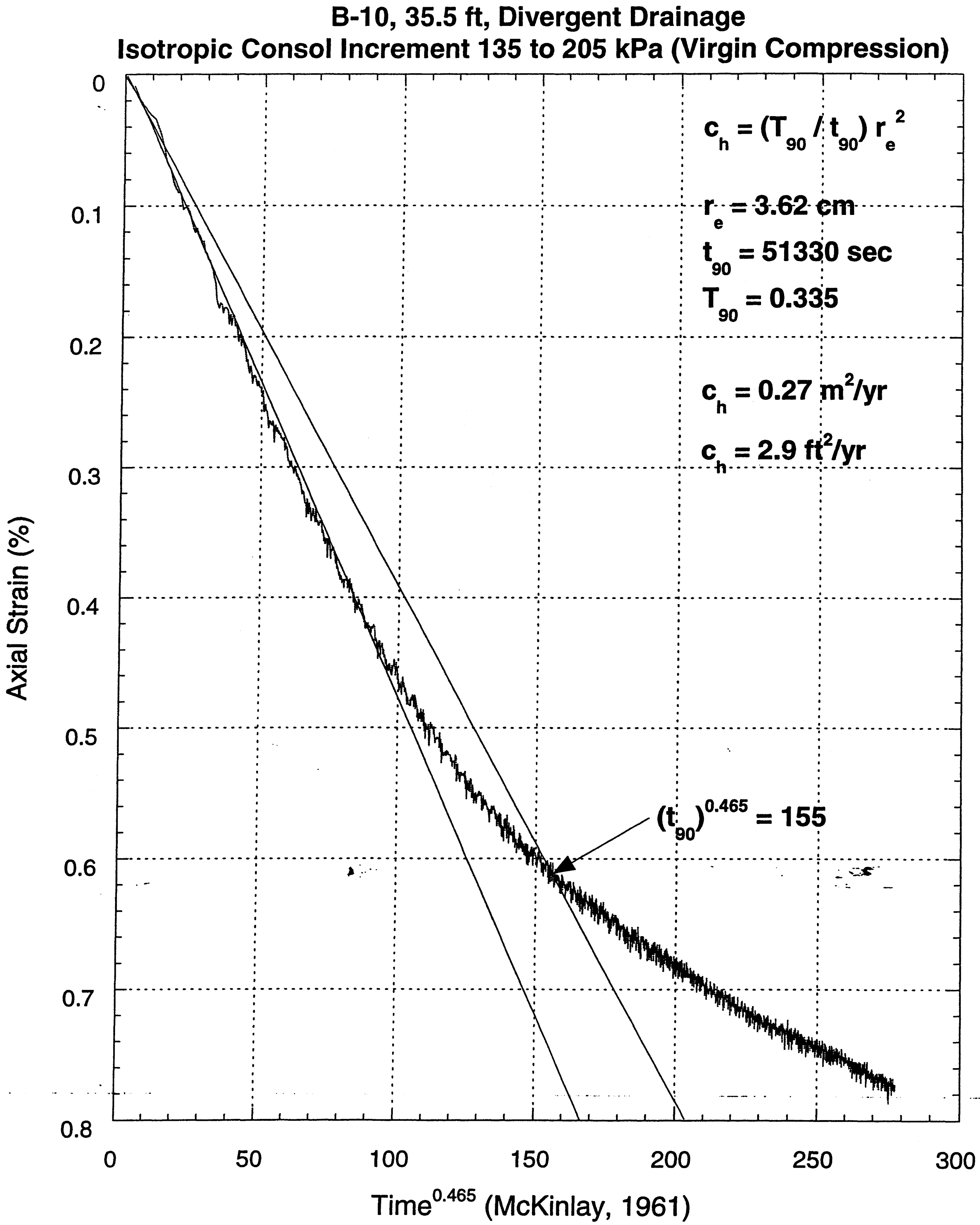
APPENDIX B

Consolidation Testing Results

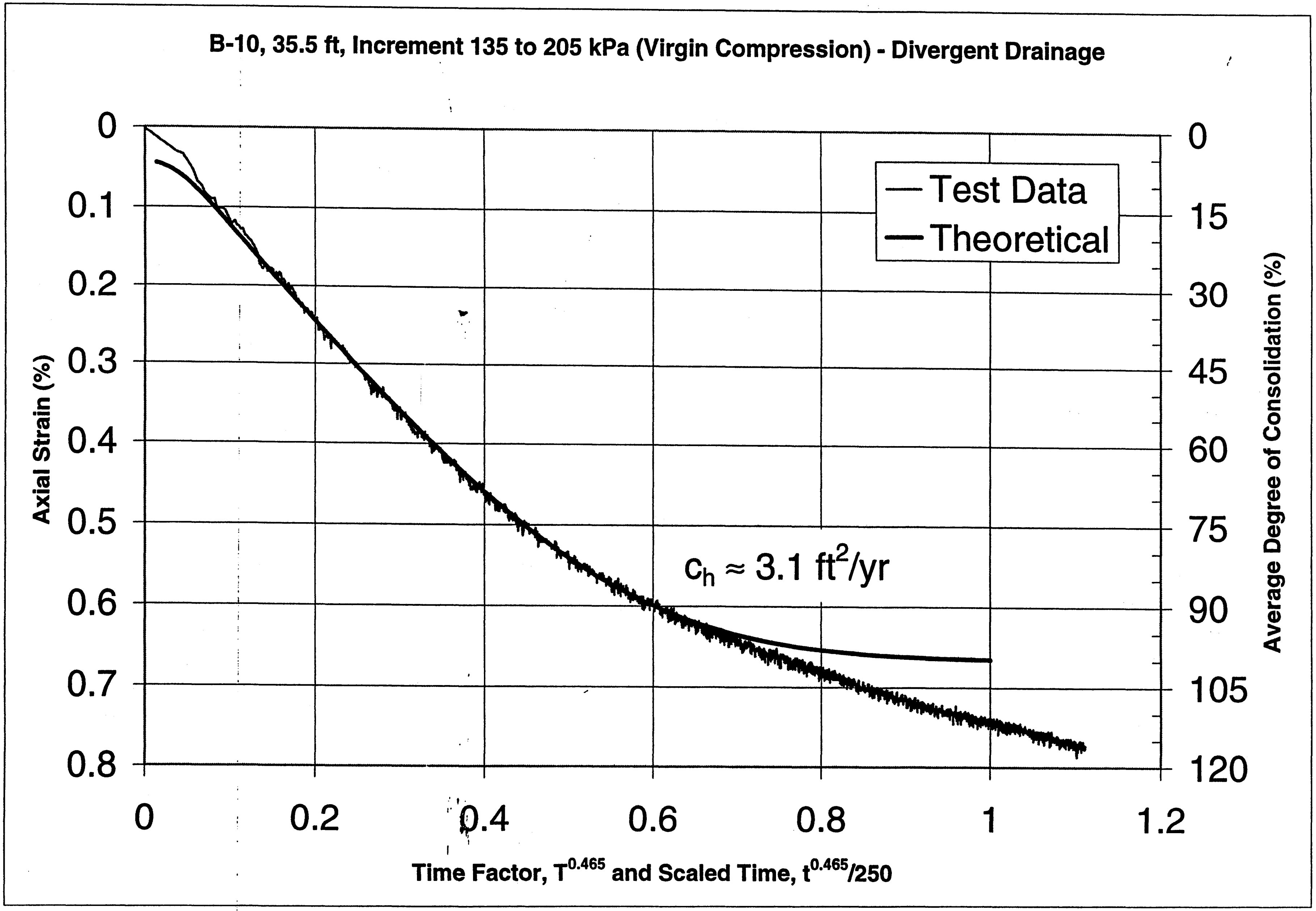

B-9, 30.5 ft, Increment 125 to 200 kPa (Virgin Compression) - Divergent Drainage
Test Data
Theoretical
$c_h \approx 11.1$ ft^2/yr
Axial Strain (%)
0.000
0.095
0.190
0.285
0.380
0.475
0.570
0.665
0.760
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/140$



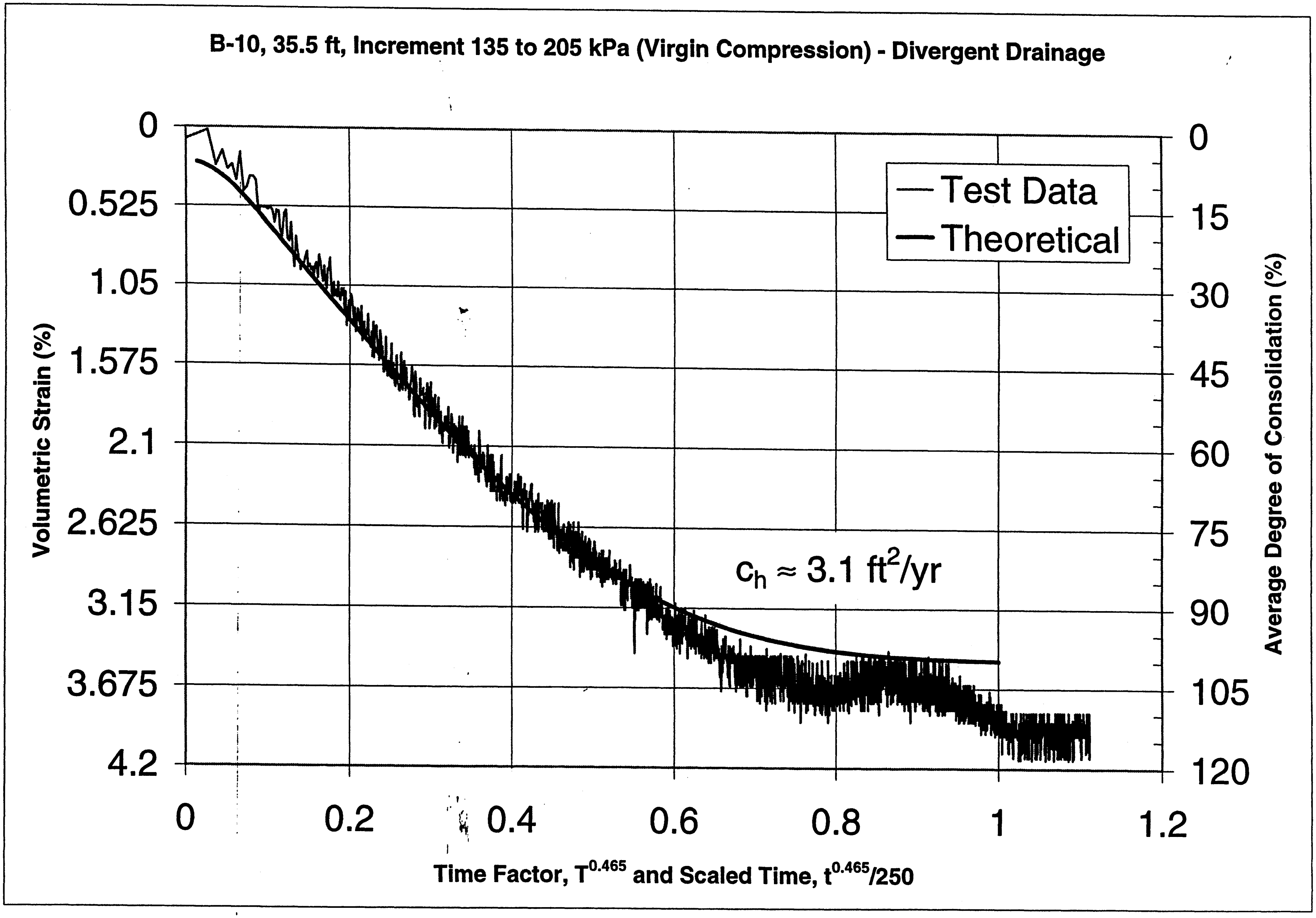
B-9, 30.5 ft, Increment 125 to 200 kPa (Virgin Compression) - Divergent Drainage
Test Data
Theoretical
$c_h \approx 11.1\ ft^2/yr$
Volumetric Strain (%)
0.000
0.406
0.813
1.219
1.625
2.031
2.438
2.844
3.250
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/140$



B-10, 35.5 ft, Divergent Drainage
Isotropic Consol Increment 135 to 205 kPa (Virgin Compression)
$c_h = (T_{90} / t_{90}) r_e^2$
r_e = 3.62 cm
t_{90} = 51330 sec
T_{90} = 0.335
c_h = 0.27 m^2/yr
c_h = 2.9 ft^2/yr
$(t_{90})^{0.465}$ = 155
Axial Strain (%)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
50
100
150
200
250
300
Time$^{0.465}$ (McKinlay, 1961)

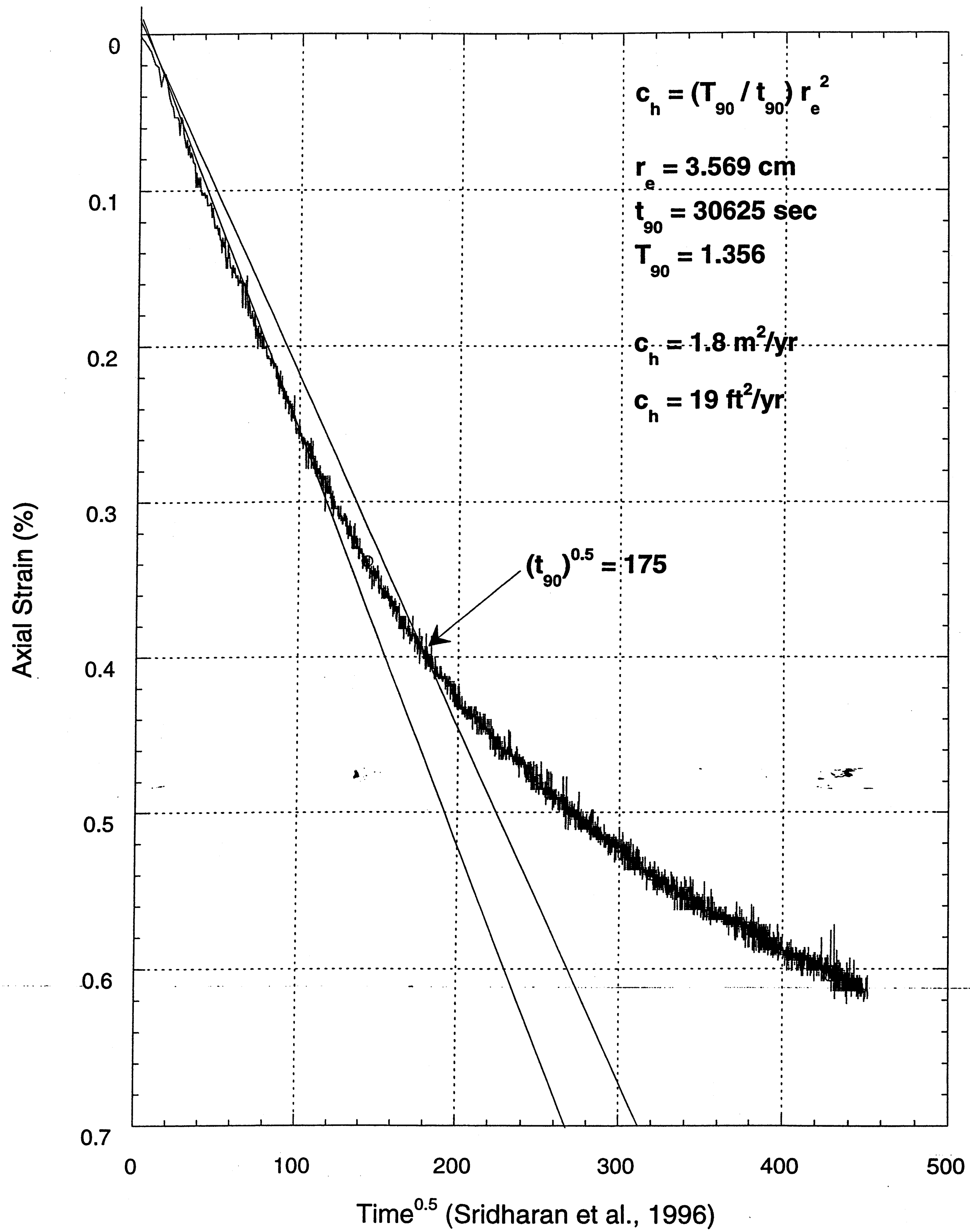


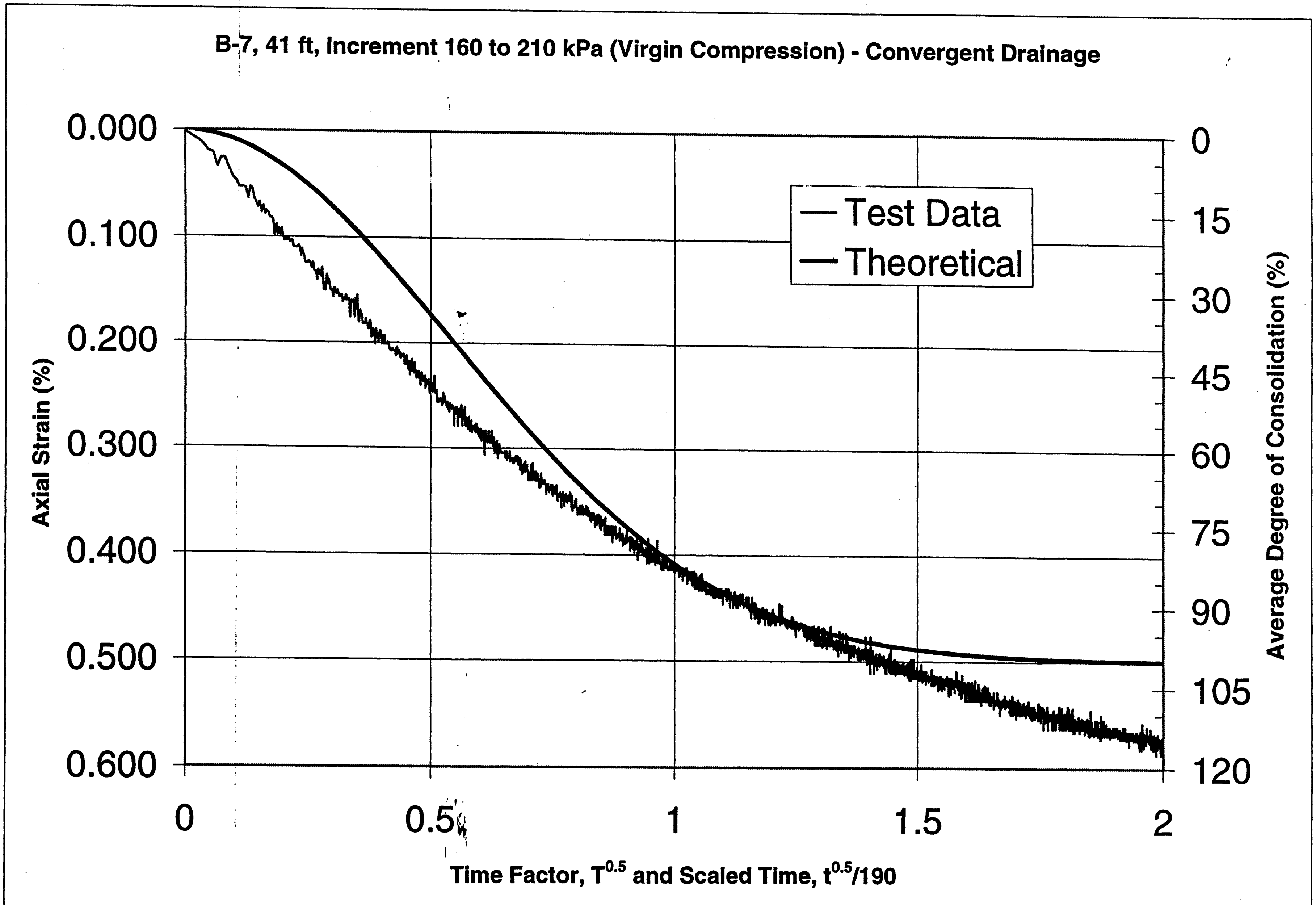
B-10, 35.5 ft, Increment 135 to 205 kPa (Virgin Compression) - Divergent Drainage
Test Data
Theoretical
$c_h \approx 3.1\ ft^2/yr$
Axial Strain (%)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/250$

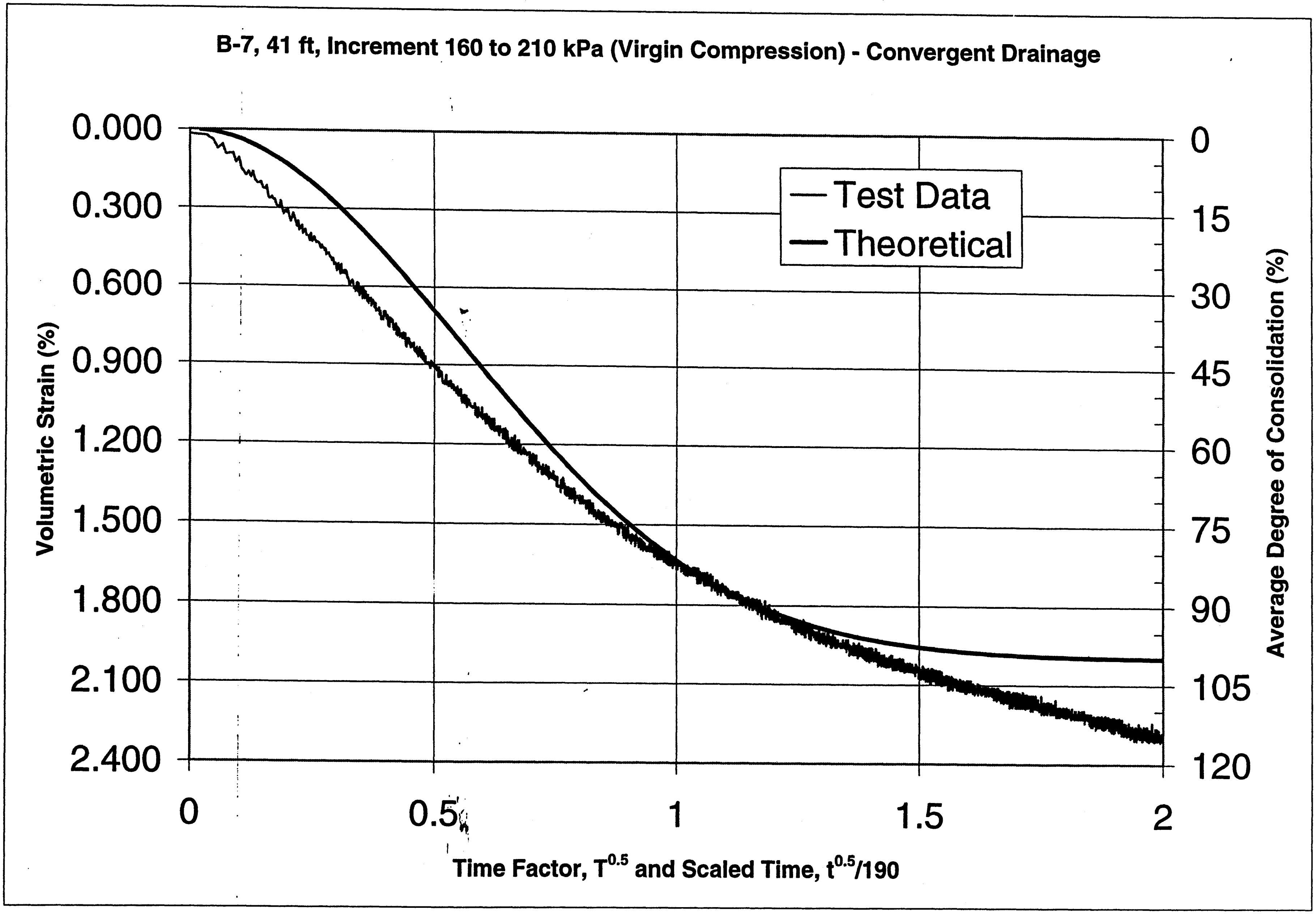


B-10, 35.5 ft, Increment 135 to 205 kPa (Virgin Compression) - Divergent Drainage
Test Data
Theoretical
$c_h \approx 3.1$ ft^2/yr
Volumetric Strain (%)
0
0.525
1.05
1.575
2.1
2.625
3.15
3.675
4.2
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/250$

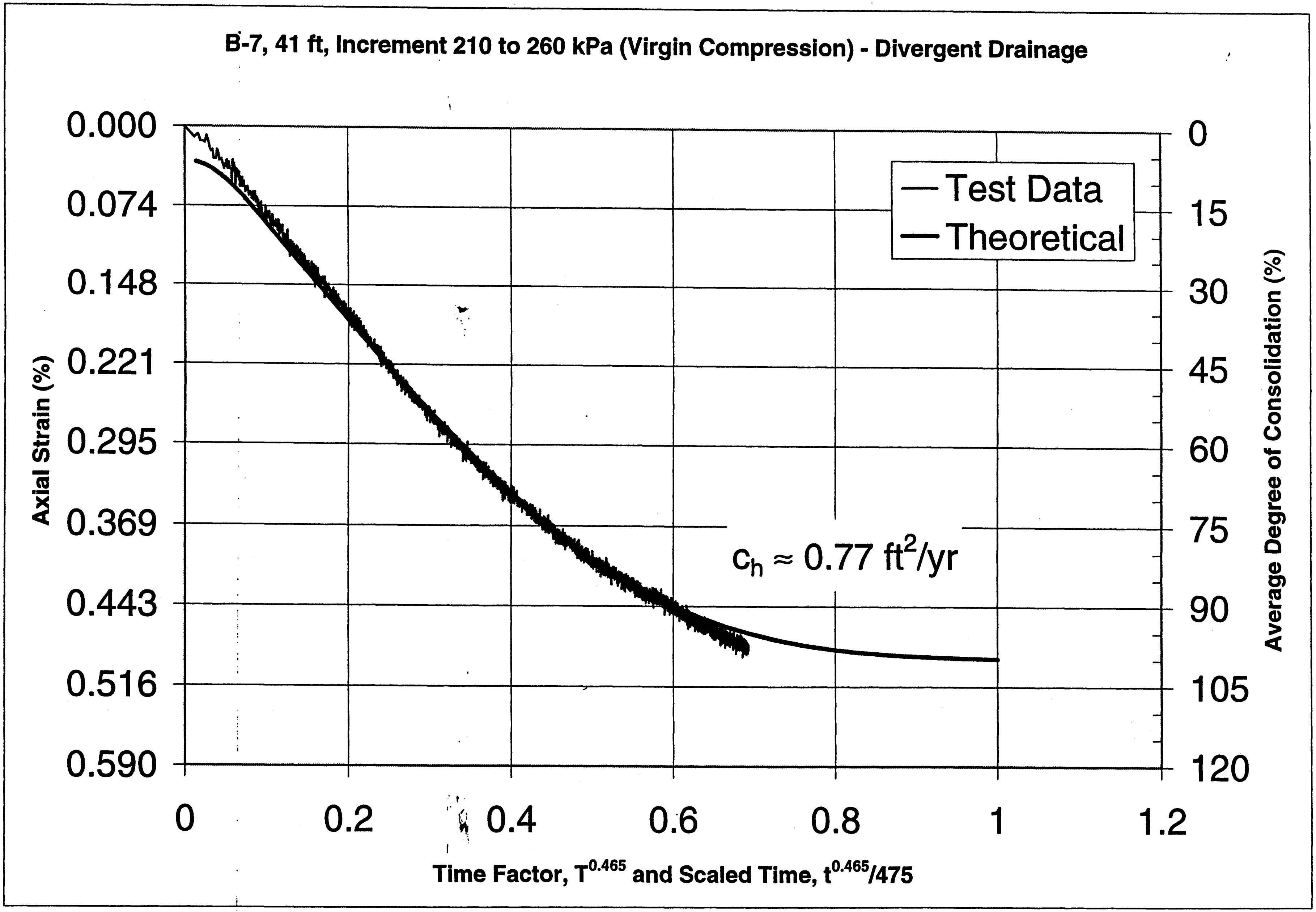
B-7, 41 ft, Convergent Drainage
Isotropic Consol Increment 160 to 210 kPa (Virgin Compression)



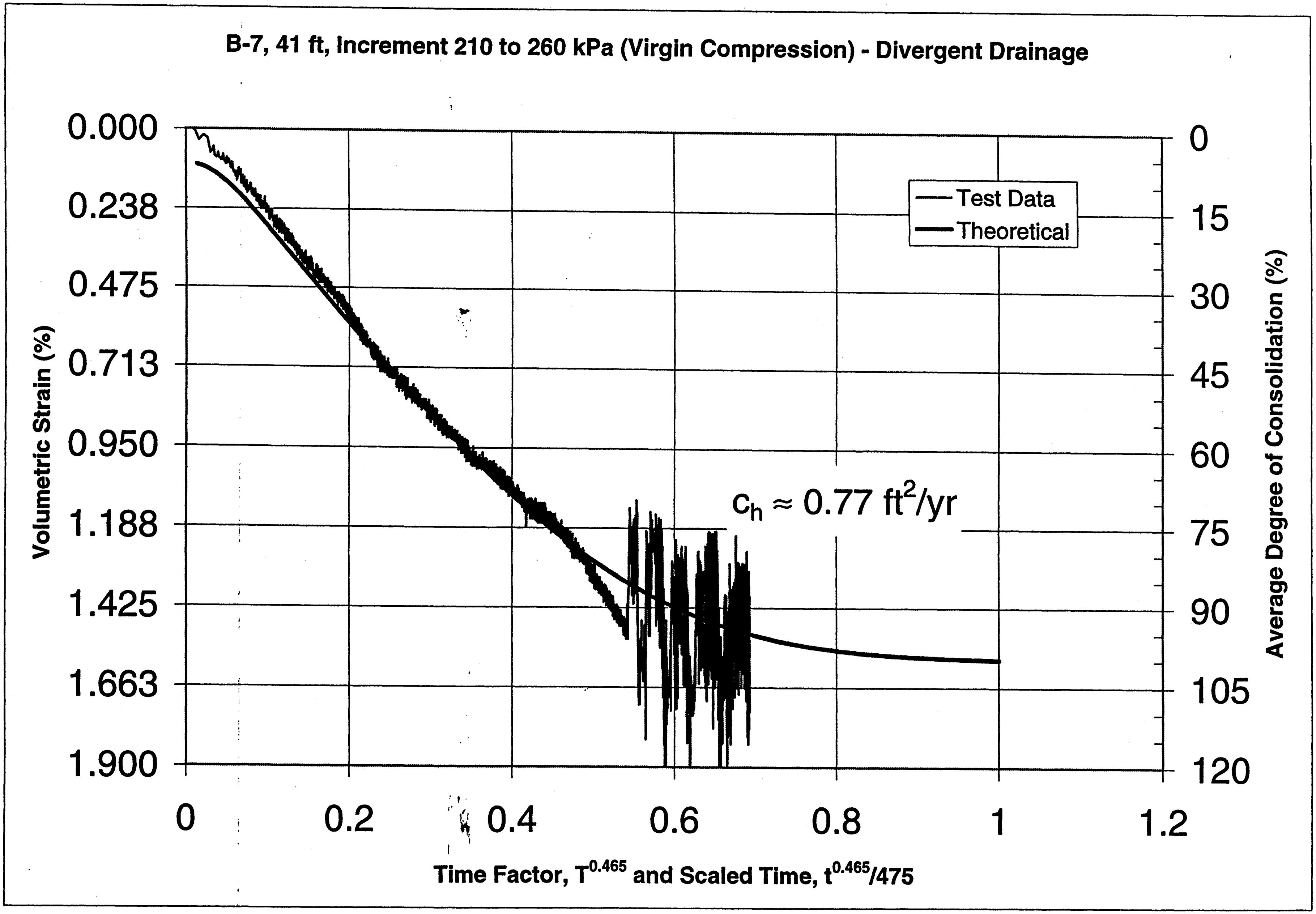

B-7, 41 ft, Increment 160 to 210 kPa (Virgin Compression) - Convergent Drainage
Test Data
Theoretical
Axial Strain (%)
0.000
0.100
0.200
0.300
0.400
0.500
0.600
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.5
1
1.5
2
Time Factor, T^0.5 and Scaled Time, t^0.5/190

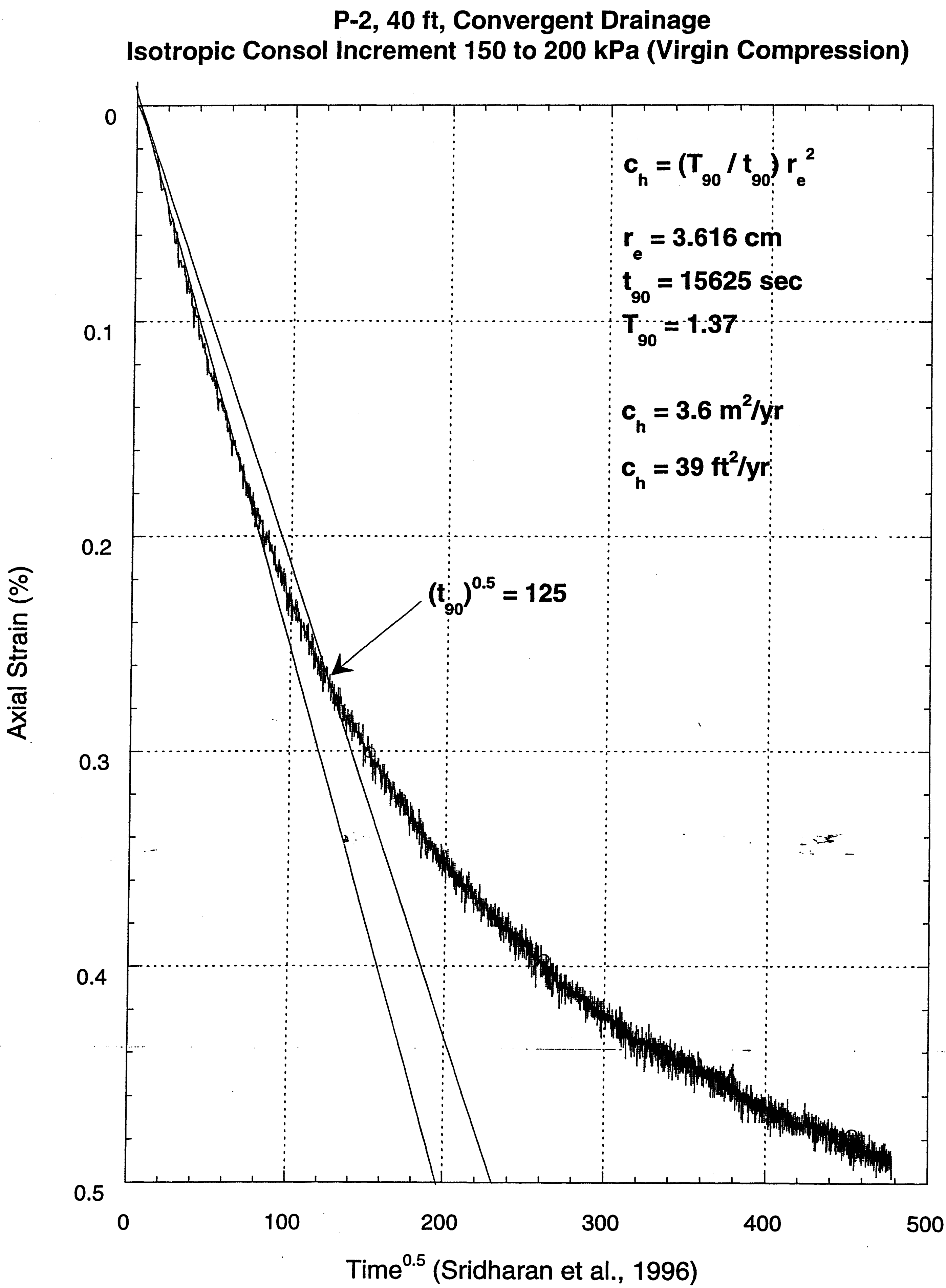


B-7, 41 ft, Increment 160 to 210 kPa (Virgin Compression) - Convergent Drainage
Test Data
Theoretical
Volumetric Strain (%)
0.000
0.300
0.600
0.900
1.200
1.500
1.800
2.100
2.400
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.5
1
1.5
2
Time Factor, T^0.5 and Scaled Time, t^0.5/190

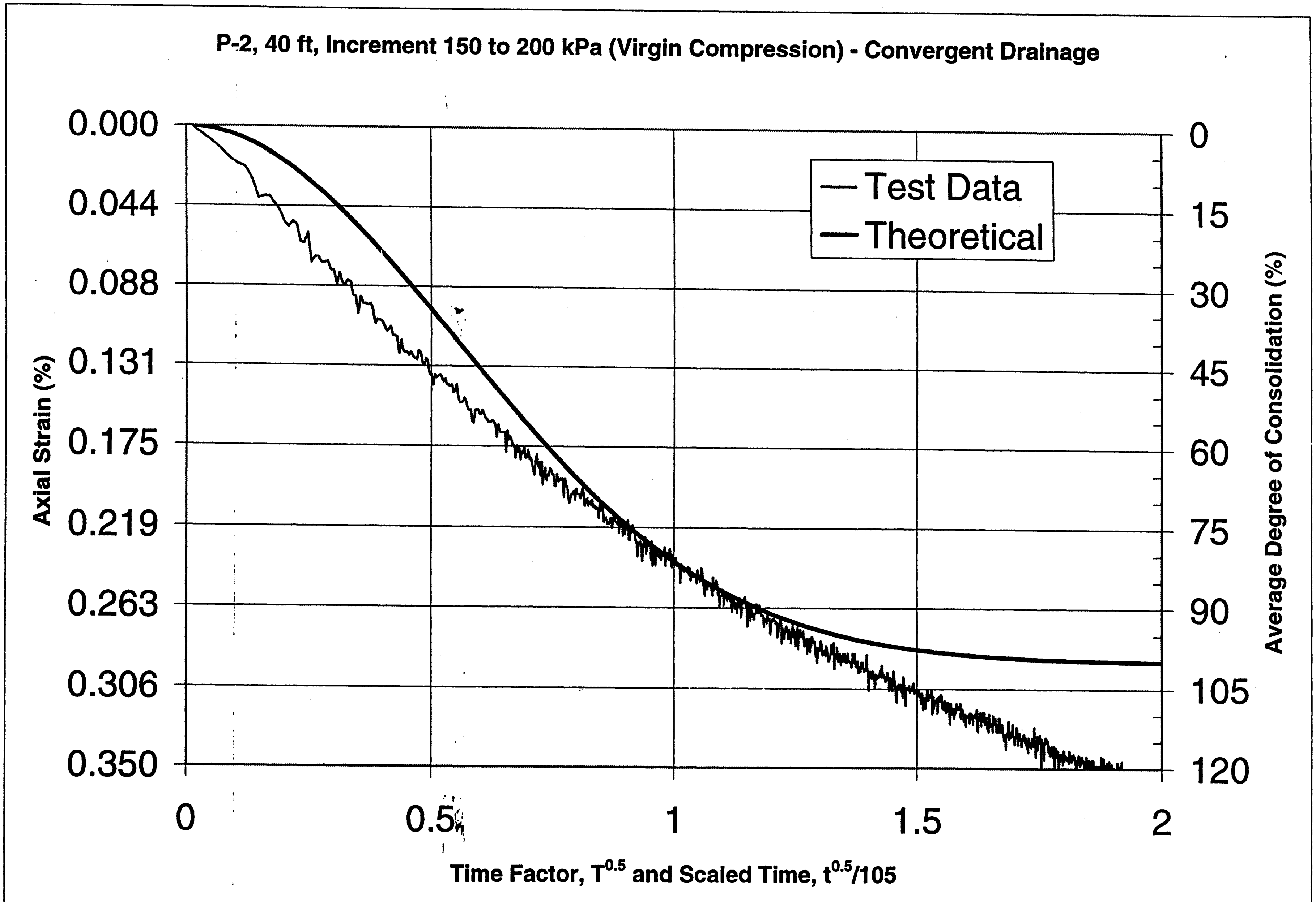


B-7, 41 ft, Increment 210 to 260 kPa (Virgin Compression) - Divergent Drainage
Axial Strain (%)
0.000
0.074
0.148
0.221
0.295
0.369
0.443
0.516
0.590
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/475$
Test Data
Theoretical
$c_h \approx 0.77\ ft^2/yr$

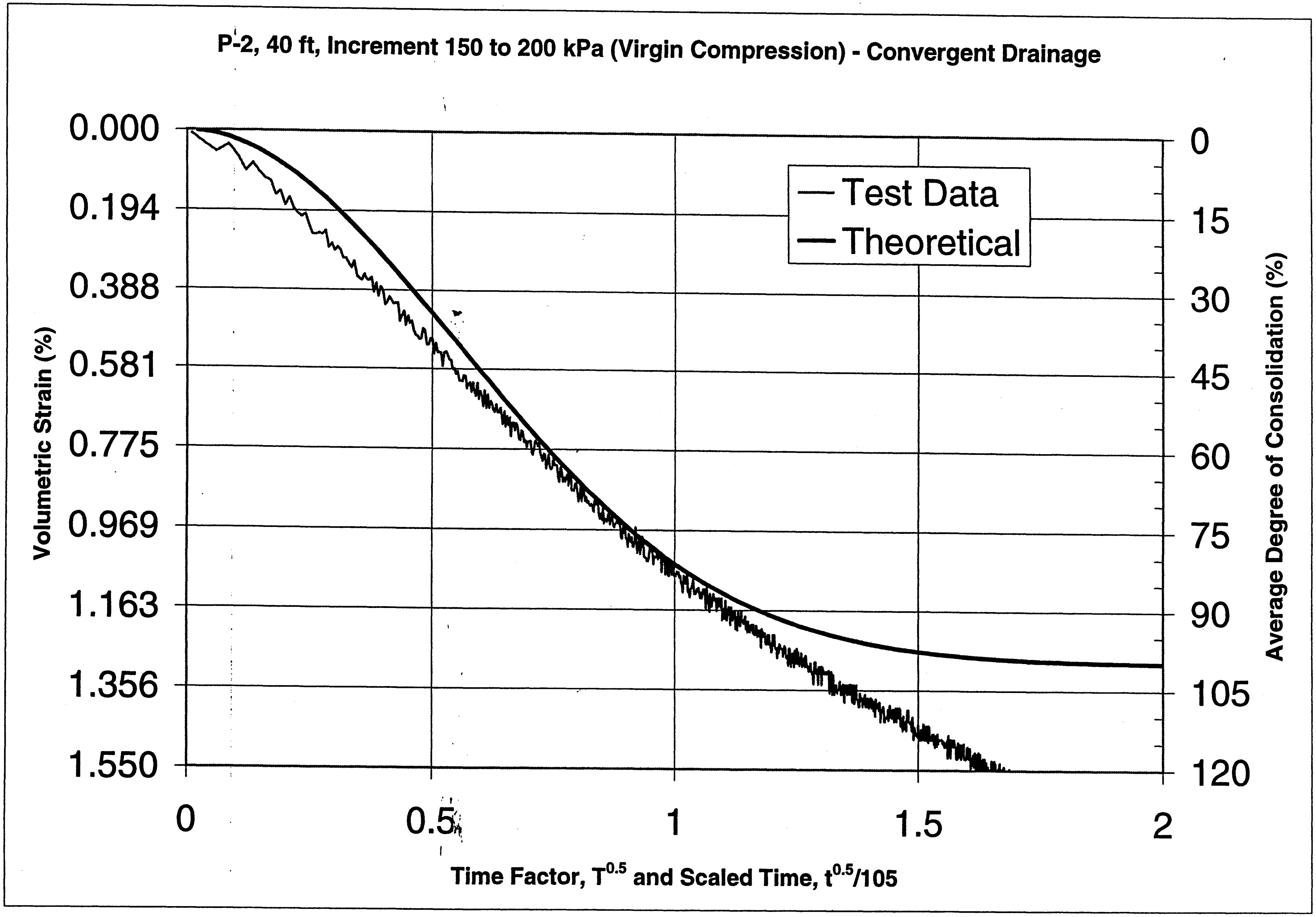

B-7, 41 ft, Increment 210 to 260 kPa (Virgin Compression) - Divergent Drainage
Volumetric Strain (%)
0.000
0.238
0.475
0.713
0.950
1.188
1.425
1.663
1.900
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
Test Data
Theoretical
$c_h \approx 0.77\ ft^2/yr$
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/475$

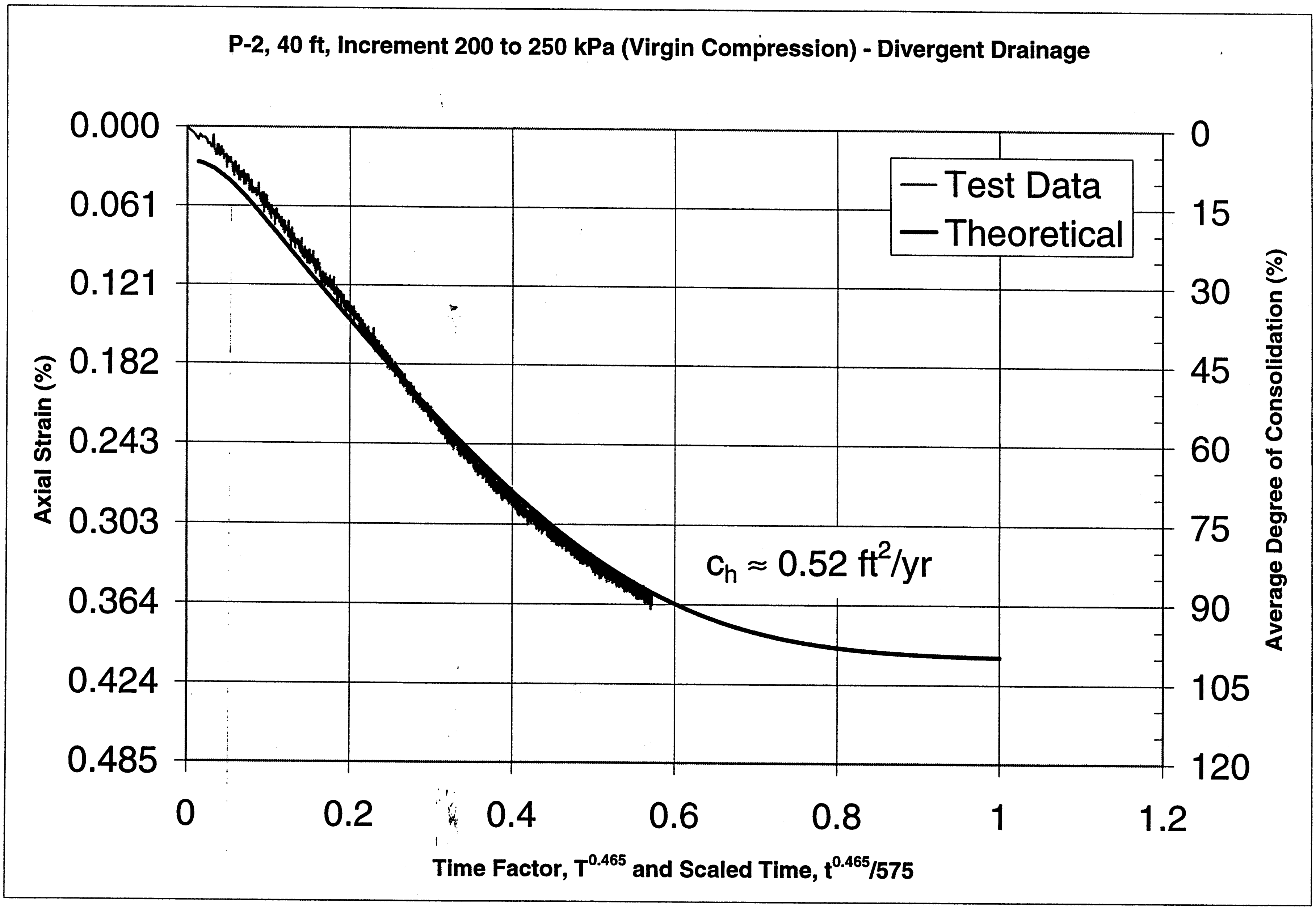


P-2, 40 ft, Convergent Drainage
Isotropic Consol Increment 150 to 200 kPa (Virgin Compression)
$c_h = (T_{90} / t_{90})\, r_e^2$
r_e = 3.616 cm
t_{90} = 15625 sec
T_{90} = 1.37
c_h = 3.6 m^2/yr
c_h = 39 ft^2/yr
$(t_{90})^{0.5}$ = 125
Axial Strain (%)
0
0.1
0.2
0.3
0.4
0.5
0
100
200
300
400
500
$Time^{0.5}$ (Sridharan et al., 1996)

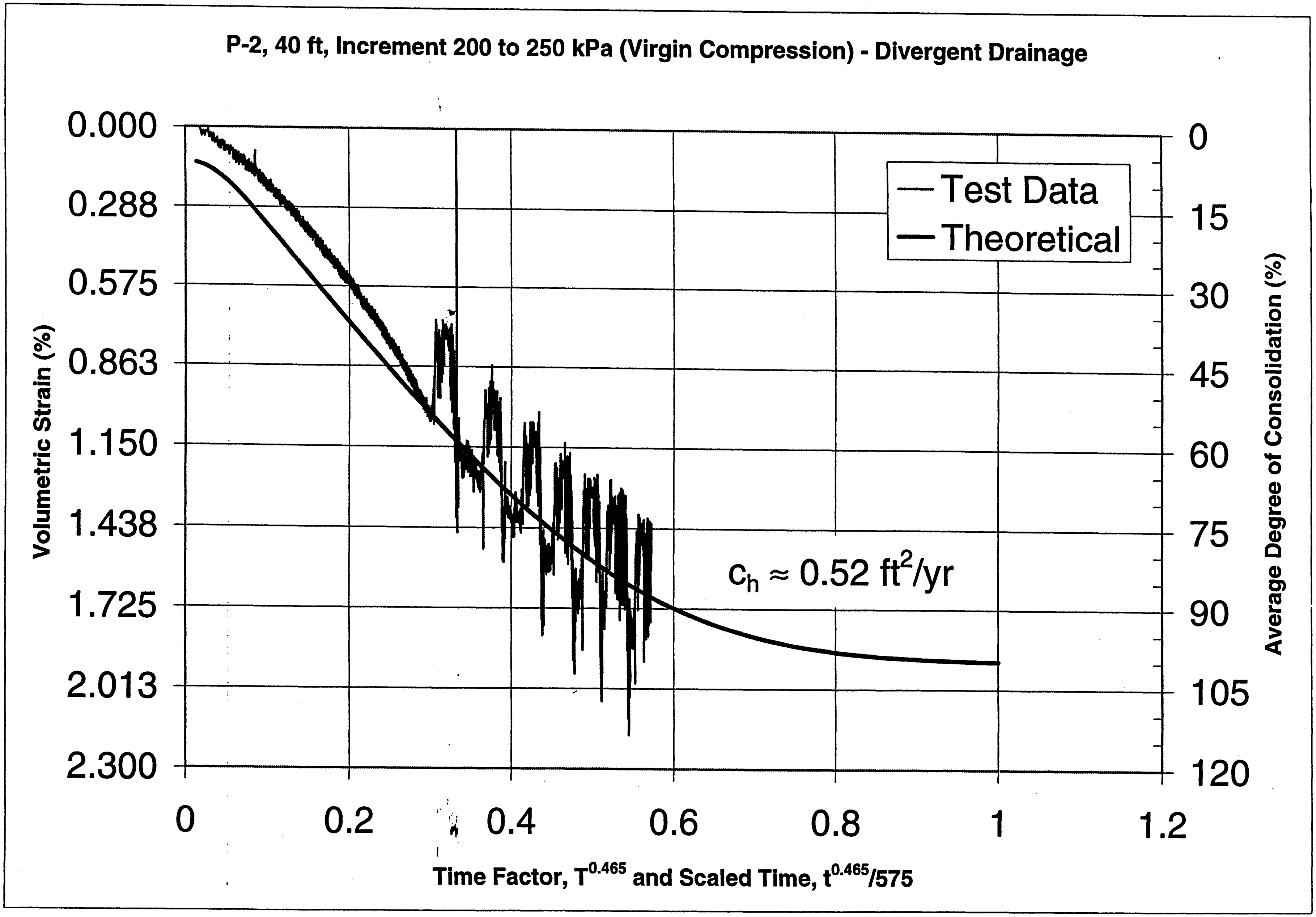


P-2, 40 ft, Increment 150 to 200 kPa (Virgin Compression) - Convergent Drainage
Test Data
Theoretical
Axial Strain (%)
0.000
0.044
0.088
0.131
0.175
0.219
0.263
0.306
0.350
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.5
1
1.5
2
Time Factor, T^0.5 and Scaled Time, t^0.5/105


P-2, 40 ft, Increment 150 to 200 kPa (Virgin Compression) - Convergent Drainage
Volumetric Strain (%)
0.000
0.194
0.388
0.581
0.775
0.969
1.163
1.356
1.550
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
Test Data
Theoretical
0
0.5
1
1.5
2
Time Factor, $T^{0.5}$ and Scaled Time, $t^{0.5}$/105



P-2, 40 ft, Increment 200 to 250 kPa (Virgin Compression) - Divergent Drainage
Test Data
Theoretical
$c_h \approx 0.52\ ft^2/yr$
Axial Strain (%)
0.000
0.061
0.121
0.182
0.243
0.303
0.364
0.424
0.485
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/575$


P-2, 40 ft, Increment 200 to 250 kPa (Virgin Compression) - Divergent Drainage
Test Data
Theoretical
$c_h \approx 0.52\ ft^2/yr$
Volumetric Strain (%)
0.000
0.288
0.575
0.863
1.150
1.438
1.725
2.013
2.300
Average Degree of Consolidation (%)
0
15
30
45
60
75
90
105
120
0
0.2
0.4
0.6
0.8
1
1.2
Time Factor, $T^{0.465}$ and Scaled Time, $t^{0.465}/575$