7- No surcharge ...

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. TOM POLLOCK, Chief
Office of Structures Design

Attention: Mr. Bob Anderson
Design Section 59-234

Date: September 18, 1992

File: 11-SD-56-0.00
11203 030111

Route 56/5 Separation #1
Bridge No. 57-0989F

From: DEPARTMENT OF TRANSPORTATION
Division of New Technology, Materials & Research
Office of Engineering Geology - South

Subject: FOUNDATION RECOMMENDATIONS

INTRODUCTION

This office has completed a subsurface investigation at the proposed 56/5 Separation (Bridge No. 57-0989F) on Route 56, San Diego, CA. The investigation was a joint effort with District 11 Materials and consisted of drilling nineteen rotary borings, fifteen electric cone penetrometer (CPT) soundings, reviewing the site conditions and the available records. Our investigation was based upon conversations with Design Section 10, the Foundation Plan received September 28, 1989 and the General Plan received December 13, 1991.

The proposed bridge will be a multispan, prestressed concrete box girder that will connect westbound traffic on Route 56 to southbound Route 5. Spans lengths range from 138.5 to 198 feet with a total bridge length of 2,617 feet. Bents will be single-column supports with the exception of bents 2-5 which are multi-column supports. Approach fills will be placed at each abutment to a maximum height of approximately 40 feet. The bridge will be designed for 100 ton (compressive load) driven piles with tension capacities of 30 tons.

SUMMARY OF FINDINGS

Our findings are presented within this report and the Log of Test Borings (LTB). The LTB will be transmitted at a later date and is to be included in the contract plans. The layout sheet shows all borings drilled in the area, those borings not shown on the profile will be available through the Office of Geotechnical Engineering.

The 56/5 Separation crosses Route 5 just south of the existing Carmel Valley Road Overcrossing on Route 5. The eastern end of the bridge (Abutment 17) is in an undeveloped field. The bridge parallels Carmel Valley Road crossing the existing El Camino Real alignment and passing over an existing gasoline station and onto the undeveloped shoulder of Route 5. After crossing Route 5, the bridge turns south, crossing Sorrento Valley Road twice and Carmel Valley Creek before terminating at Abutment 1. The ground surface is relatively level with a slight rise associated with Route 5. Carmel Valley Creek, an east-west flowing tributary of the Soledad Valley estuary, is south of the proposed bridge. In this area the creek is a sinuous, perennial stream that shows no incision below the flood plain. The majority of the bridge alignment is covered with native and ornamental plants and grasses with the roads and gasoline station area covered by asphalt paving.

Exploratory borings reveal the subsurface at the bridge site consists of Holocene estuary and alluvial deposits (Power and others, 1982) overlying Eocene bedrock and mantled by artificial fill. The artificial fills at the surface consist of loose, slightly moist to moist, brown, silty to clayey sands. The estuary deposits (Qhe) are very loose to loose dark, fossiliferrous gray silty sands to micaceous silts and clays. These silts and clays are interbedded with gray to light gray to brown, slightly compact to very dense fluvial sands to silty sands (Qhfl). The base of the fluvial deposits is well defined in most areas by dense to very dense, gravelly to cobbly sands to cobbles that overlies the Delmar Formation. The elevation of the top of the Delmar Formation (Td) beneath the bridge varies from elevation -38.1 to -76.4. The Delmar Formation consists of poorly to moderately cemented, green to brown, Eocene mudstones and sandstones.

Ground Water

Ground water was measured between elevations 13.9 and 18.7 feet above sea level. The elevation of the ground water surface is highly dependent upon the seasonal rainfall. In general, from December to late April, ground water is at or near the ground surface.

Corrosivity

The following table lists the results from soil samples taken in borings near the bridge site and tested for corrosivity (California Test 643).

Sample	Sample Type	pH	Min. Resistivity (Ohm-cm)	Soluble Sulfates (ppm)	Soluble Chlorides (ppm)	Years
B-4L @ 30	SM/OL	7.7	958	595	102	25
B-4L @ 65	SM	7.6	1436	-	-	29
B-5L @ 70	Td	7.8	821	144	60	23
B-5L @ 75	Td	7.8	821	144	60	23
B13L @ 120	Td	7.8	958	1255	172	25
R-22 @ 45	SM	7.6	1915	-	-	33
R-22 @ 65	SM	7.4	2257	-	-	35
R-22 @ 75	Td	7.7	2599	-	-	37
R-22 @ 100	Td	8.1	889	129	83	24

The CALTRANS Corrosion Unit classifies sulfates in excess of 2,000 ppm and chlorides in excess of 500 ppm as corrosive. The number of years represents the length of time to perforate an 18 gage galvanized steel culvert. The limited testing indicates that the deposits in the area of the 56/5 Separation are not corrosive to steel and concrete.

Sieve Analysis

A number of samples were selected and submitted for testing for grain-size distribution (California Test 202). The soils were found to be predominantly sandy silts and silty clays and silty sands.

Plasticity

The results of testing for Atterberg's Limits (California Test 204) are shown in the table below. Figure 1 is a plot of the Plasticity Index vs. Liquid Limit from the alluvial soils listed below compared to the limits of liquefiable soils (after Tokimatsu & Yoshimi, 1983).

Sample (Boring # and depth)	Plasticity Index	Plastic Limit	Liquid Limit	Soil Type*
B-4L @ 60(A)	np**	np	np	SM/SC
B-5L @ 25	np	np	np	SM/SC
B-5L @ 80	24	23	47	Td
B-5L @ 85	4	20	24	Td
B-5L @ 120	7	36	43	Td
B-6L @ 20	np	np	np	SM
B-6L @ 90	9	32	41	Td
B-13L @ 25	5	50	55	ML/MH
B-13L @ 45	9	30	39	ML
B-20L @ 25	5	17	22	CL
B-21L @ 10	3	19	22	CL
B-21L @ 35	4	19	23	CL
B-21L @ 50	8	15	23	CL
B-22L @ 30	np**	np	np	SM/SC
B-22L @ 35	7	19	26	ML/OL
B-22L @ 50	7	19	26	ML/OL
B-25L @ 40	np	np	np	SM

*Unified Soil Classification or Formation abbreviation

** np=nonplastic

Seismicity

The Rose Canyon fault is mapped 5 miles west of the site (Reichle and others, 1990). The site is not within the Alquist-Priolo Special Study Zone (Hart, 1990). Mualchin & Jones (1991) proffer the following information for design of structures in the area:

Maximum Credible Earthquake Magnitude	7.0
Peak Horizontal Bedrock Acceleration	0.5 gravity

The depth to "rock-like" material (Vs greater than 2,500 feet per second) varies from 65 to 113 feet below existing ground. The duration of strong-ground motion should be on the order of 15-20 seconds. The bridge site has not experienced ground shaking greater than 0.1 gravity in nearly 200 years (Reichle and others, 1990; Figure 2; Table II).

Secondary Seismic Effects

Power and others (1982) performed a regional evaluation of liquefaction susceptibility in the San Diego Metropolitan area just south of Carmel Valley. Their Table 1-1 indicates that the Holocene fluvial (Qhfl) and Holocene estuarine (Qhe) deposits, similar to those found in our borings, have a moderate to high susceptibility to liquefy during seismic events. They found that the estuarine and fluvial deposits have a mean blow count of 16 and recommended that site specific liquefaction studies be performed in areas where these deposits occur. Reichle and others (1990) hypothesized that Carmel Valley is not an area with high potential of experiencing ground failure due to liquefaction during an earthquake on the Silver Strand fault in Mission Bay.

Figures 2-9 are illustrations of the liquefaction susceptibility of the deposits underlying the Carmel Valley Road Undercrossing and nearby locations. A figure for B-4L was not prepared because it is adjacent to B-5L. The analysis performed for this report utilized the method outlined by the National Research Council (1985), after Seed and Idriss (1982), and supplemented by Ishihara (*in press*). Figures 2-9 show cyclic stress ratio versus normalized blow counts (adjusted for fines content) for various depths below present ground surface within each boring. The points plotted on the figures were determined by the method described below.

Blow counts (abscissa) were determined using the method outlined by the National Research Council [NRC] (1985) and supplemented by Ishihara (*in press*). First, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 incorporating the recommendations contained within NRC (1985; Tables 4-3 & 4-4). Secondly, the measured blow count (N) was normalized to one ton of overburden at 60% energy transfer or $(N_1)_{60}$ using the method outlined in NRC (1985). Third, sieve analysis (California Test 202) was performed to determine the influence of fines content (percentage of materials passing through the #200 sieve) as outlined by Ishihara (*in press*). Samples with less than 20% clay (0.005 mm) were considered liquefiable (Seed and Idriss, 1982) and $(N_1)_{60}$ was then converted to $(N_1)_{60} + \blacktriangle(N_1)_{60}$ using equation (12) from Ishihara (*in press*). $[(N_1)_{60} + \blacktriangle(N_1)_{60}]$ is plotted versus cyclic stress ratio (ordinate) to determine susceptibility to liquefaction for samples with a clay content less than 20 percent during a M=7.0 earthquake.

Cyclic stress ratio was determined by the method presented in NRC (1985). Where a_{max} is the peak horizontal bedrock acceleration as determined from Mualchin & Jones (1987), r_d is the stress reduction factor of 1 at the surface to 0.9 at or below 35 feet. Total overburden and effective overburden were determined using saturated densities of 110 pcf and 130 pcf for estuary and fluvial deposits based upon samples taken near B-23L. These soil densities compare favorably to typical values of soil unit weight determined by Powers and others (1982).

Figures 2-9 show that, with a few exceptions, the sediments beneath the bridge alignment are liquefiable down to the top of the bedrock. The greatest thickness of liquefiable soils are at Bent 7 (B-13L) where liquefaction could occur as deep as 105 feet. Where artificial fills have been placed to support Route 5, the fills form a nonliquefiable layer atop the liquefiable sediments. Figure 10 shows that liquefaction below this layer would induce ground damage (i.e. - lateral spreading) at the two boring locations. The cobbly to bouldery zone overlying the bedrock is not liquefiable.

Liquefaction will create surface manifestations, possibly in the form of lateral spreading, ground oscillations and sand boils, in addition, to ground settlement all along the bridge alignment. To mitigate the effects of liquefaction, the Office of Geotechnical Engineering has recommended that stone columns be placed to a minimum depth of 50 feet at the bents and abutment locations to reduce the potential for lateral spreading.

Settlement

Foundations: Calculations provided by the Office of Geotechnical Engineering indicate that dynamic settlement due to liquefaction can be as great as 1.1 feet along the 56/5 Separation alignment and foundations should be designed against downdrag forces along the pile.

Embankments: The Office of Geotechnical Engineering has recommended that stone columns be placed beneath the approach embankments to support the embankments; however, no construction sequence was provided in the memorandum dated August 17, 1992 or June 22, 1992. Calculations using the Hough Method estimate a ground settlement of 0.8 and 3.0 feet beneath the fills at abutment approachs 1 & 17, respectively.

RECOMMENDATIONS

Additional Studies

The Office of Geotechnical Engineering should review this report and the liquefaction susceptibility at the site and perform additional studies as they deem necessary. The tip elevations of the stone columns should be specified at each location because of the varying depths of liquefiable material.

Seismic Hazards

Ground rupture is not a hazard at the site and, therefore, no special mitigative measures are required. Preliminary design of the bridge should be completed using a peak horizontal bedrock acceleration of 0.5 gravity and a depth to "rock-like" material is 65-113 feet. Final design should be based upon the site specific acceleration response study from the Office of Geotechnical Engineering.

Foundations

Foundations for the proposed bridge should be driven HP14 x 89 steel H-sections or 13 5/8 inch diameter, 1/2-inch thick wall pipe piles (open or closed end). The conical shaped tip is required for the pipe piles; the flat plate end is not an option. For both the open ended pipe and the H-section pile, tip protection is required. Concrete piles are not considered alternatives.

As required by Design Section 10, the allowable compressive capacity of the piles is 100 tons with a tension capacity of 30 tons. Pile capacities were calculated using the SPT method outlined by the FHWA and a minimum factor of safety of 2.0. Piles may be designed using the following table.

SUPPORT LOCATION	BOTTOM OF FOOTING ELEVATION	SPECIFIED TIP ELEVATION*	PILE LENGTH (FEET)	ULTIMATE COMPRESSIVE LOAD (TONS)	ULTIMATE TENSION LOAD FOR SEISMIC DESIGN (TONS)
Abut 1	+27.0	-38.0	65	220	45
Bent 2 LEFT	+7.3	-45.0	52.3	220	45
Bent 2 RIGHT	+8.8	-45.0	53.8	220	45
Bent 3 LEFT	+5.5	-70.0	75.5	220	45
Bent 3 RIGHT	+9.5	-70.0	79.5	220	45
Bent 4 LEFT	+4.5	-85.0	89.5	220	45
Bent 4 RIGHT	+8.0	-85.0	93.5	220	45
Bent 5 LEFT	+4.0	-103.0	107	220	45
Bent 5 RIGHT	+6.0	-103.0	109	220	45
Bent 6	+5.0	-105.0	110	220	45
Bent 7	+20.0	-73.0	93	220	45
Bent 8	+22.0	-68.0	90	220	45
Bent 9	+25.0	-68.0	93	220	45
Bent 10	+10.0	-68.0	78	220	45
Bent 11	+25.0	-68.0	93	220	45
Bent 12	+25.0	-53.0	78	220	45
Bent 13	+25.0	-53.0	78	220	45
Bent 14	+24.0	-53.0	77	220	45
Bent 15	+24.0	-53.0	77	220	45
Bent 16	+25.0	-53.0	78	220	45
Abut 17	+40.0	-33.0	73	220	45

*Probable Tip Elevations are estimated to be within 5 feet of specified tip.

Pile Load Tests

Dynamic and static (compressive and tension) pile load tests are planned for three locations within the interchange under the direction of the Office of Geotechnical Engineering. It is recommended that at least one of the load tests be performed in an area where the ground has been improved with stone columns and another in an area where no ground improvement has been done. Static load tests should be performed on the same day as driving to reduce the effects of soil set up. These tests should be performed prior to the driving of production piles for the bridge so that additional recommendations regarding the pile driving or construction sequence may be made if necessary. The location, specifications and layout for the pile load tests will be provided by the Office of Geotechnical Engineering.

Settlement

Foundations: Static and dynamic settlement of the foundations should be negligible because piles will be founded into the underlying bedrock. Piles founded into the bedrock will resist downdrag (FHWA, 1986).

Embankments: After embankment fills have been placed to full height, an additional ten (10) foot high surcharge is recommended on the 100 feet of embankment closest the bridge. Settlement platforms should be installed and monitored by the Resident Engineer. A minimum settlement period of at least 120 days should be observed to allow for the approximately 0.8 and 3.0 feet of settlement at abutments 1 & 17; however, this settlement period may be accelerated by the installation of the stone columns. The settlement is complete when the rate of settlement is less than 1/4 inch over 10 consecutive days. The actual settlement period shall be determined by the engineer in the field.

Corrosion Protection

The samples tested were all noncorrosive; however, this does not preclude the possibility of corrosive layers unidentified by our testing. The heavy H-section and thick walled pipe pile should mitigate the effects of corrosion during the design life of the foundations.

Approach Slabs

Seismic approach slabs will be required at both abutment locations.

Construction Specifications

The construction sequence should be as follows:

1. **Stone columns installed.**
2. **Embankments placed to full height with settlement platforms installed.** 10' Surcharge?
3. **Settlement period observed.**
4. **Piles driven.**

This sequence is recommended for all support locations, including bents, to increase ground stability and access during pile driving.

Predrilling may be required through the embankments fills to elevations +15 and +20 at Abutments 1 & 17, respectively. Hard driving (in excess of 150 ton ENR bearing) may be anticipated to attain the specified pile tip elevation. The Special Provisions should state that if difficult driving is encountered, this office should be contacted prior to submission of pile driving alternatives (i.e.-jetting or predrilling) to the contractor.

The Special Provisions should state that the conical tip, or equivalent, is the only type of tip allowed for the closed end pipe piles. The Structure Representative should monitor initial pile installation efforts to evaluate the effect of the closed end on the driving. It is the option of the Structure Representative to remove the tip after consulting with this office.

Ground and surface water will effect construction. The contractor may be required to mitigate the effects of surface water in order to work. District 11 Environmental Planning should provide recommendations regarding restrictions on the work area.

REFERENCES CITED

Hart, E. W., 1990, Fault-rupture hazard zones in California: California Division of Mines & Geology Sp. Pub. 42.

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Ishihara, K., *in press*, Evaluation of Liquefaction potential and consequent deformations in sand fills;

Mualchin, L. & Jones, A. L., 1991, Peak acceleration from maximum credible earthquakes in California: California Division of Mines & Geology Open File Report 87 & Map Sheet 45.

National Research Council, 1985, Liquefaction of soils during earthquakes; National Academy Press, Washington, D.C.; Report No. PB86-163110, pp. 240.

Power, M. S., Dawson, A. W., Streiff, D. W., Perman, R. C. & Berger, V., 1982, Evaluation of liquefaction susceptibility in the San Diego, California urban area; v.1, U. S. Geological Survey Technical Report No. 14-08-0001-19110.

Reichle, M. S. and others, 1990, Planning Scenario for a Major Earthquake, San Diego-Tijuana Metropolitan Area; California Division of Mines and Geology Special Publication 100, pp. 181.

Seed, H. B. and Idriss, I. M., 1982, *Ground Motion and Soil Liquefaction during Earthquakes*; Earthquake Engineering Research Institute Monograph, v. 5, pp. 134.

Tokimatsu, K. and Seed, H. R., 1987, Evaluation of settlements in sands due to earthquake shaking; Journal of Geotechnical Engineering, v. 113, no. 8, 861-868.

Tokimatsu, K. and Yoshimi, Y., 1983, Empirical correlation of soil liquefaction based upon SPT N-value and fines content; Soils and Foundations, v. 23, no. 4, 56-74.

If you have any further questions, please do not hesitate to call (213) 620-3780 (ATSS-640-3780).

Report by

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Office of Engineering Geology

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District Materials
District Design
PISouth
R. E. Pending

Enclosures: Figure 1-10
57-0989F
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STATE OF CALIFORNIA

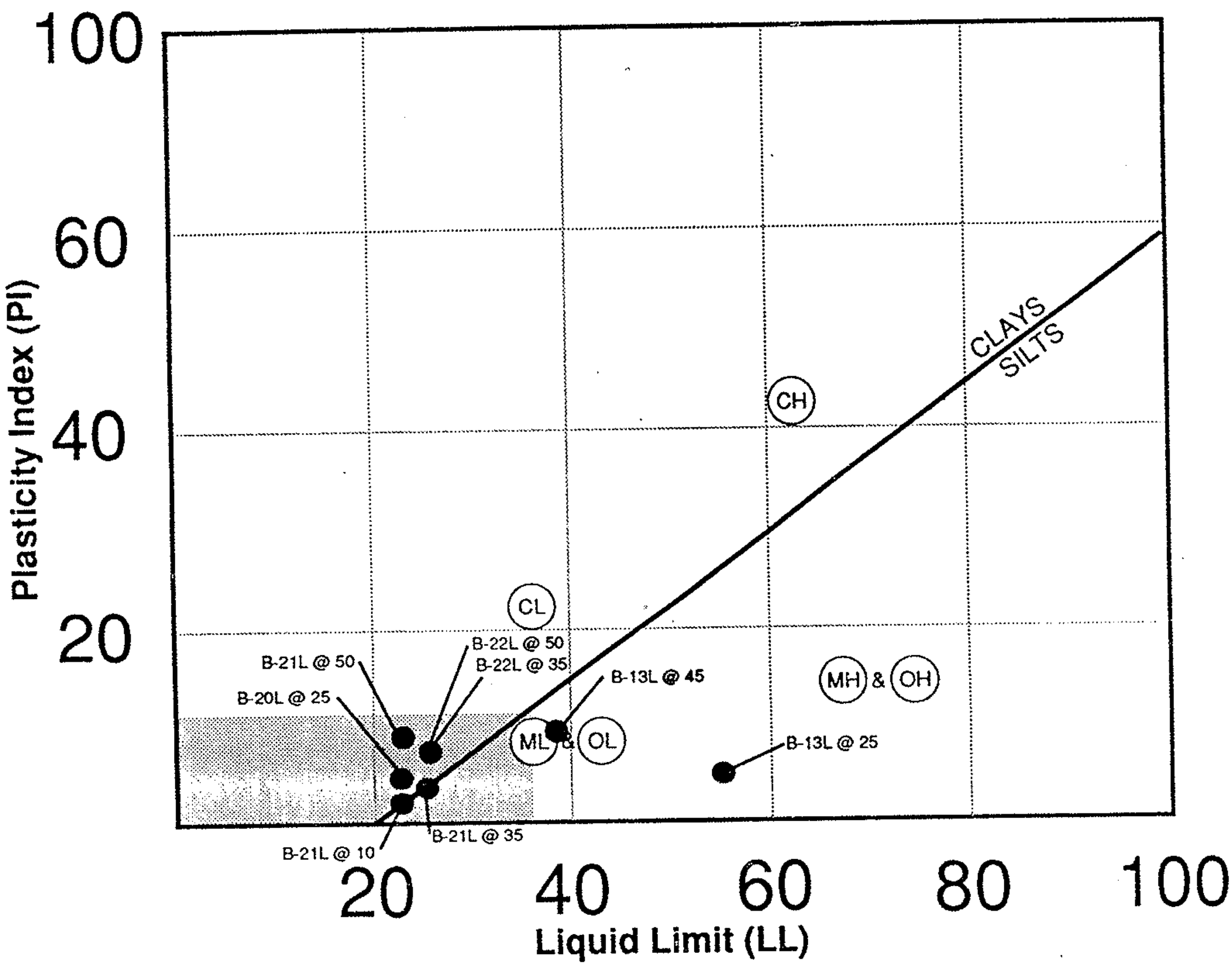
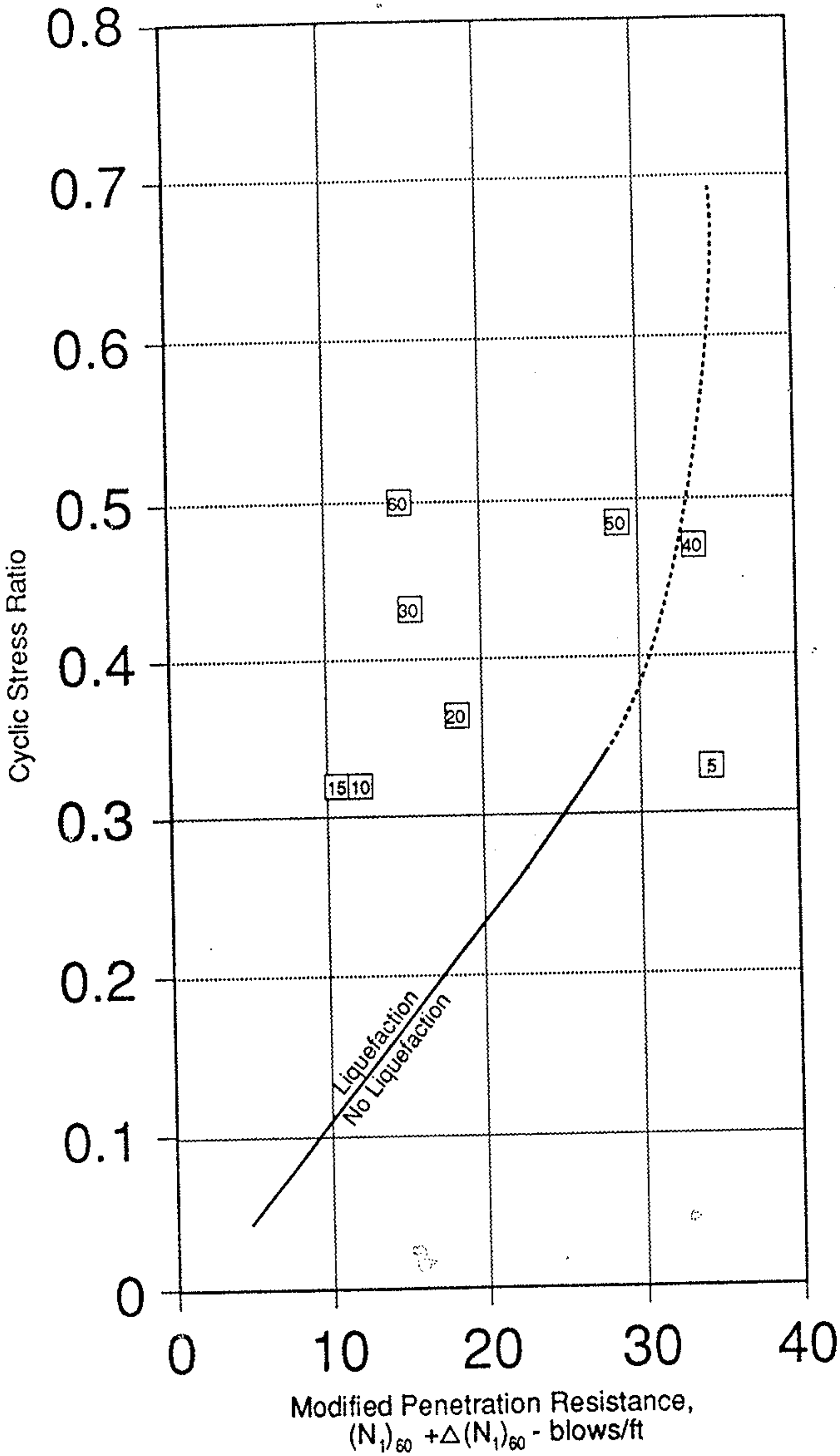


FIGURE 1

56/5 Separation   Br. No. 57-0989F   11-SD-56-0.00   CU: 11203   EA: 030111
Soil Plasticity Chart from selected samples; shaded area represents zone of liquefiable soils (after Tokimatsu & Yoshimi, 1983).

CALTRANS



Bridge Name:

Route 56/5 Separation

Bridge No.:

57-0989F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-5L

Boring Location

5' lt. of 1100+70 of "WS" line

Fault:

ROSE CANYON

Peak Acceleration:

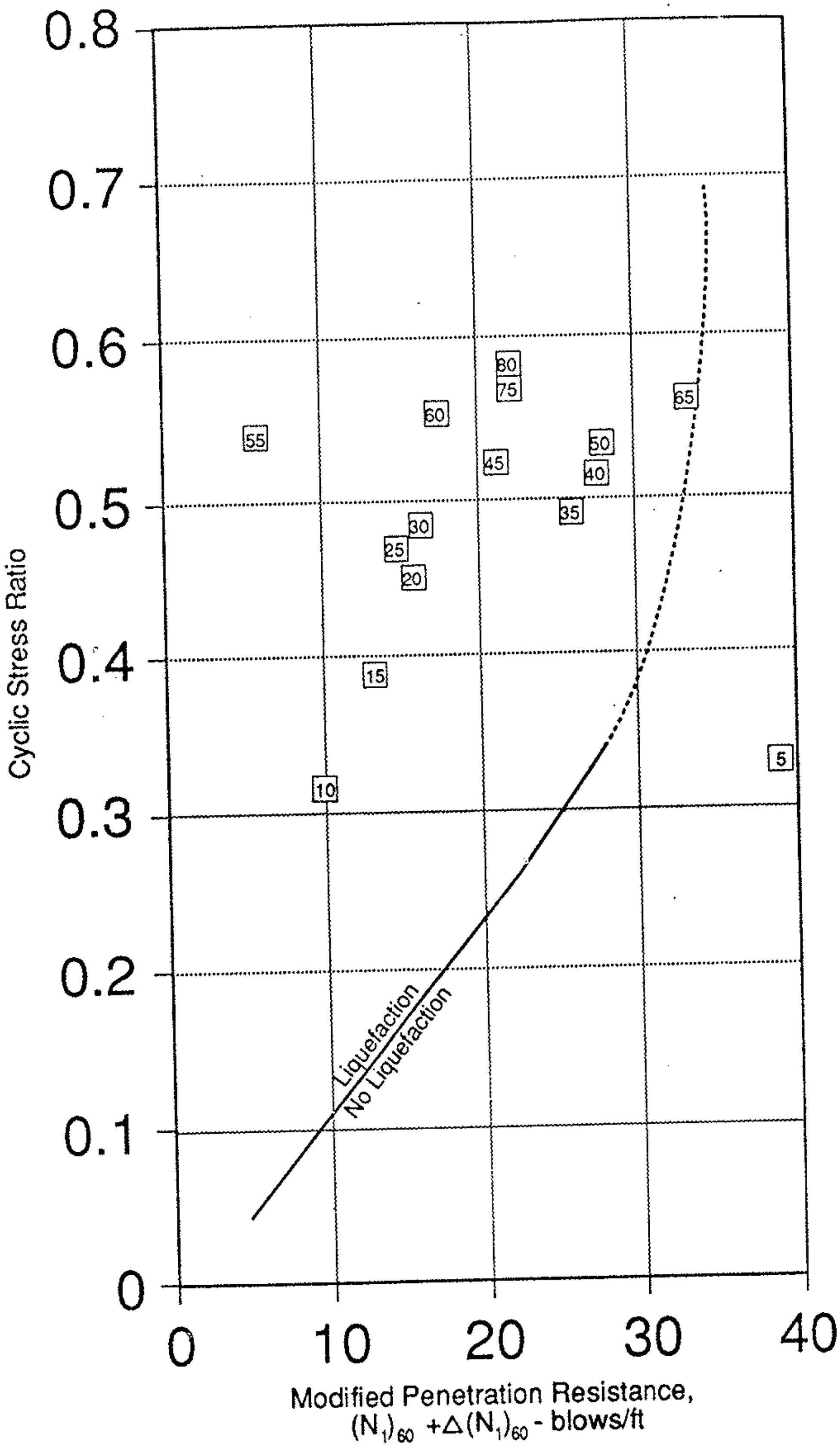
0.5

Density of Soil:

110 to 130 pcf

FIGURE 2:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).

CALTRANS



Bridge Name:

56/5 Separation

Bridge No.:

57-0989F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-6L

Boring Location

53' rt. of 1099+62 of "WS" line

Fault:

ROSE CANYON

Peak Acceleration:

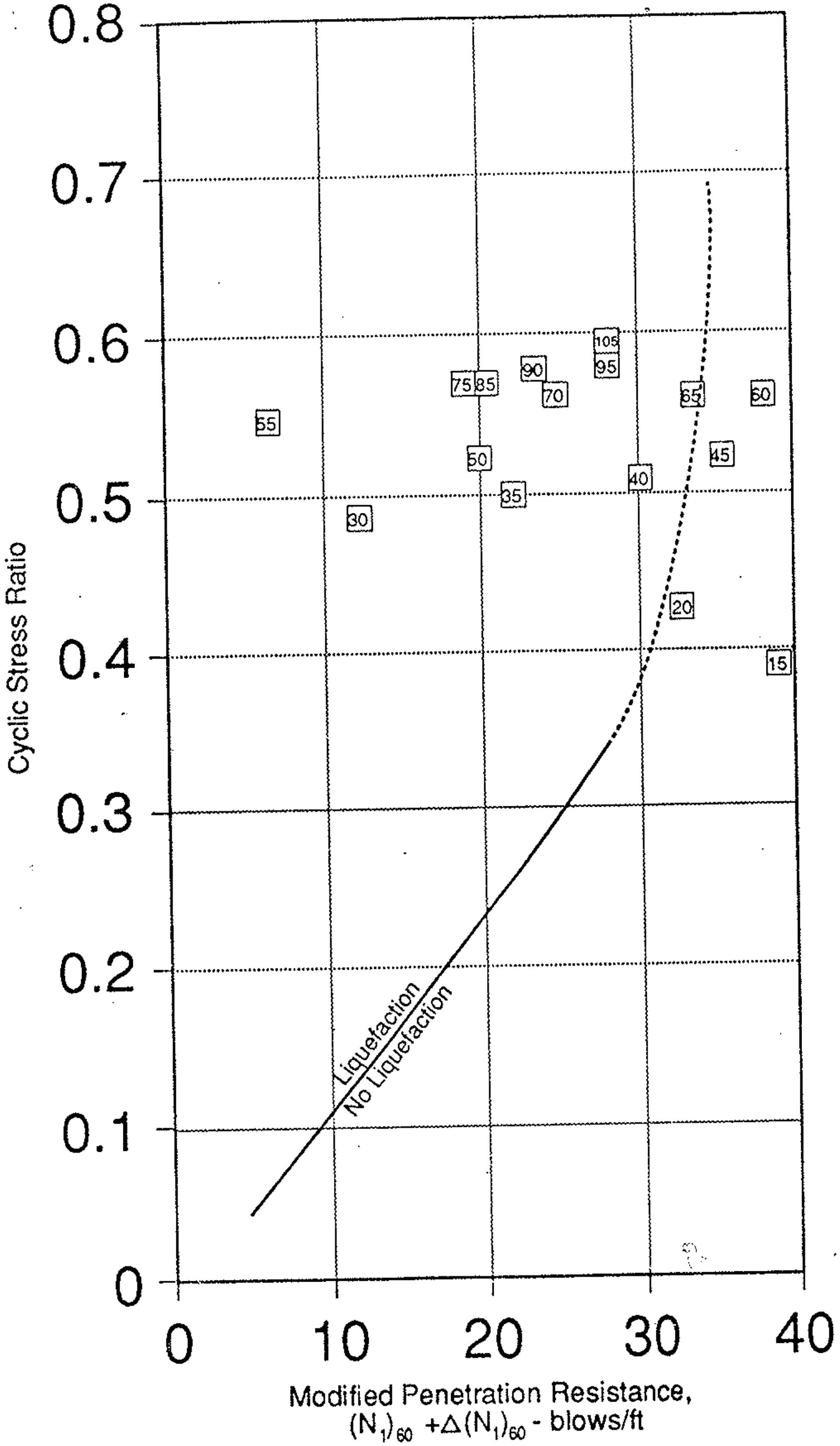
0.5

Density of Soil:

110 to 130 pcf

FIGURE 3:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).

CALTRANS



Bridge Name:

56/5 Separation

Bridge No.:

57-0989F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-13L

Boring Location

19' lt. of 1092+61 of "WS" line

Fault:

ROSE CANYON

Peak Acceleration:

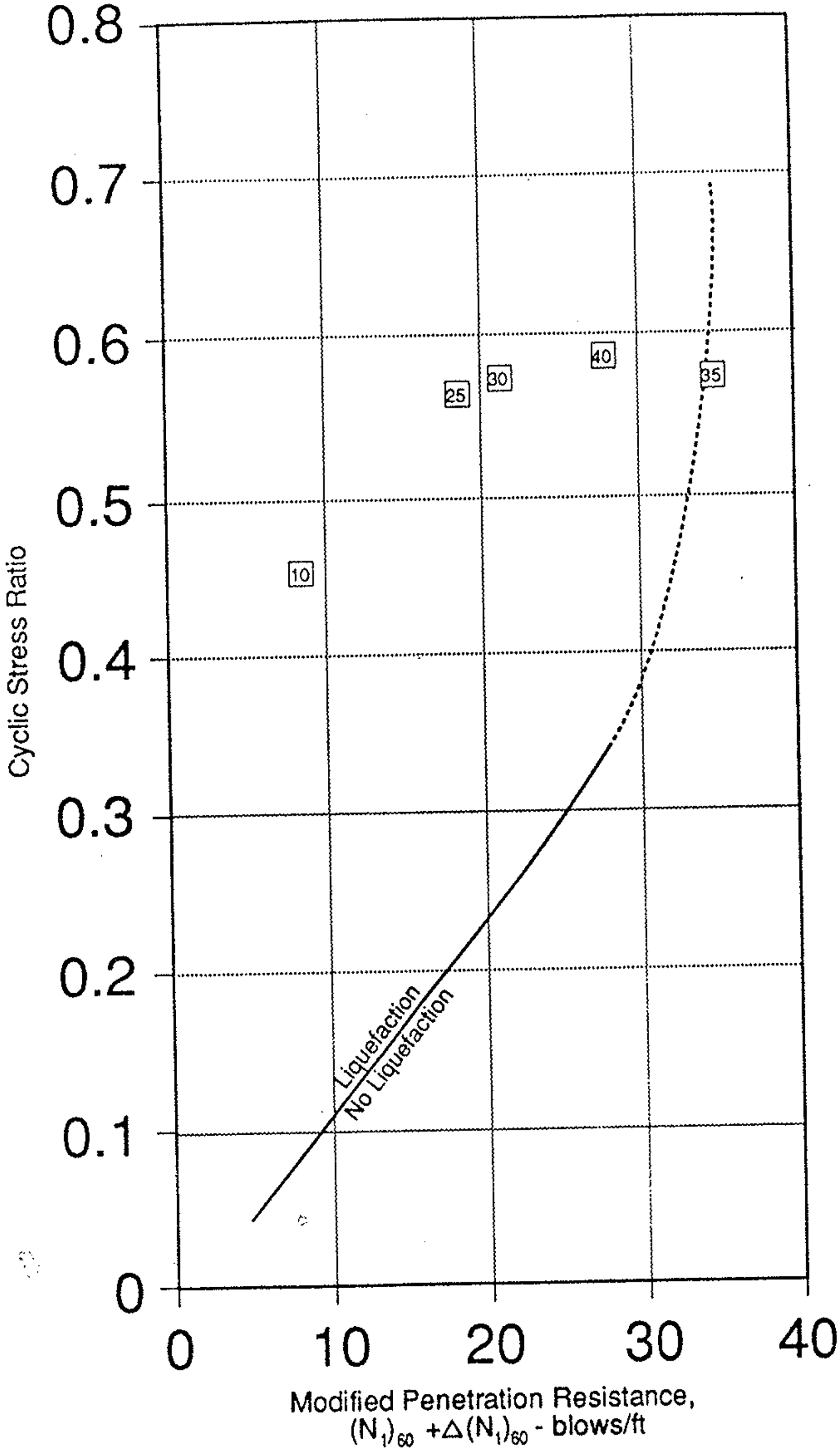
0.5

Density of Soil:

110 to 130 pcf

FIGURE 4:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).

CALTRANS



Bridge Name: Route 56/5 Separation

Bridge No.: 57-0989F

Dist-Cnty-Rte-PM: 11-SD-56-0.00

Boring Number B-16L

Boring Location 16' lt. of 1083+04 of "WS" line

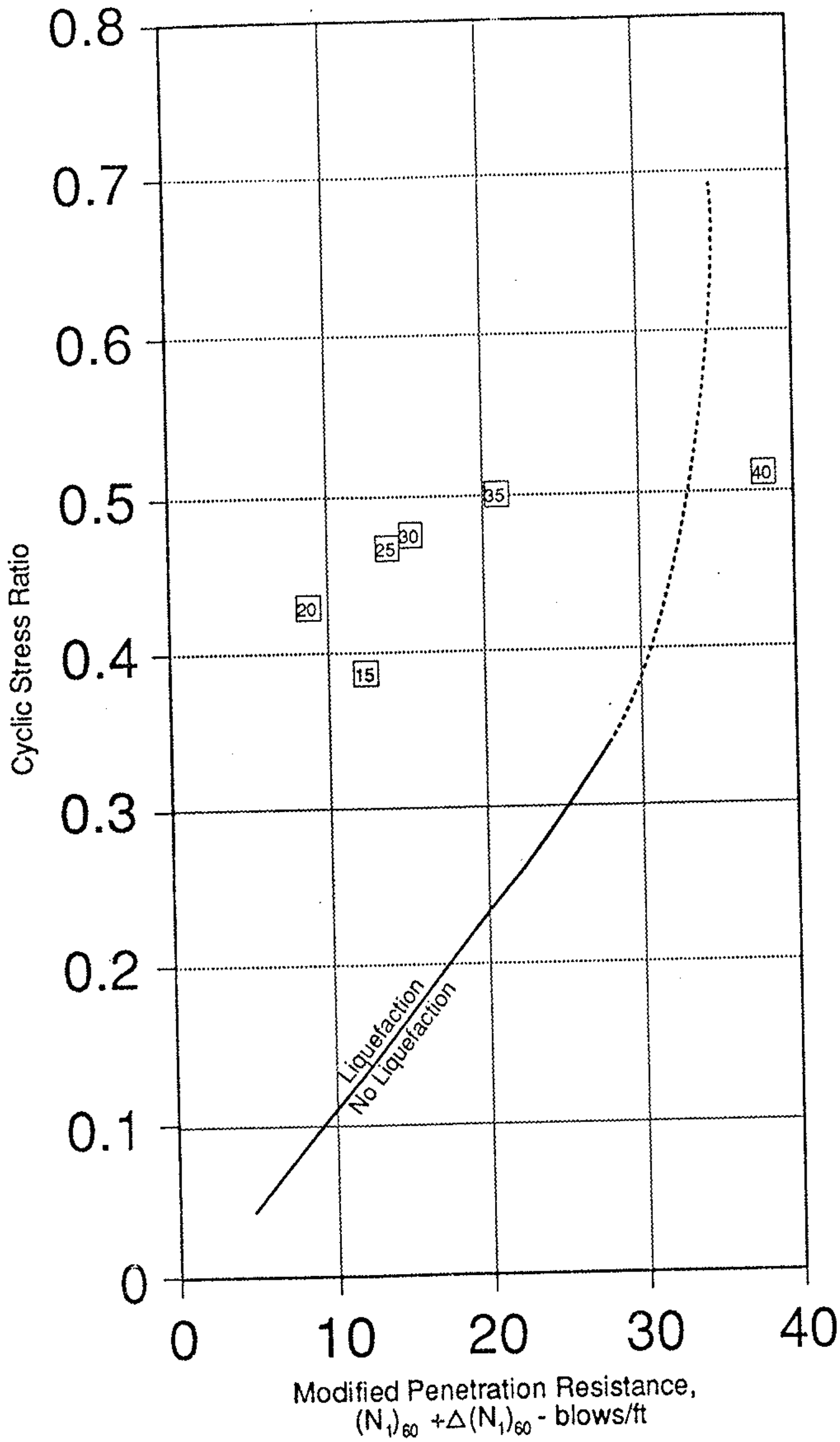
Fault: ROSE CANYON

Peak Acceleration: 0.5

Density of Soil: 110 to 130 pcf

FIGURE 5:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).

CALTRANS



Bridge Name:

Route 56/5 Separation

Bridge No.:

57-0989F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-20L

Boring Location

on 1110+30 of "WS" line

Fault:

ROSE CANYON

Peak Acceleration:

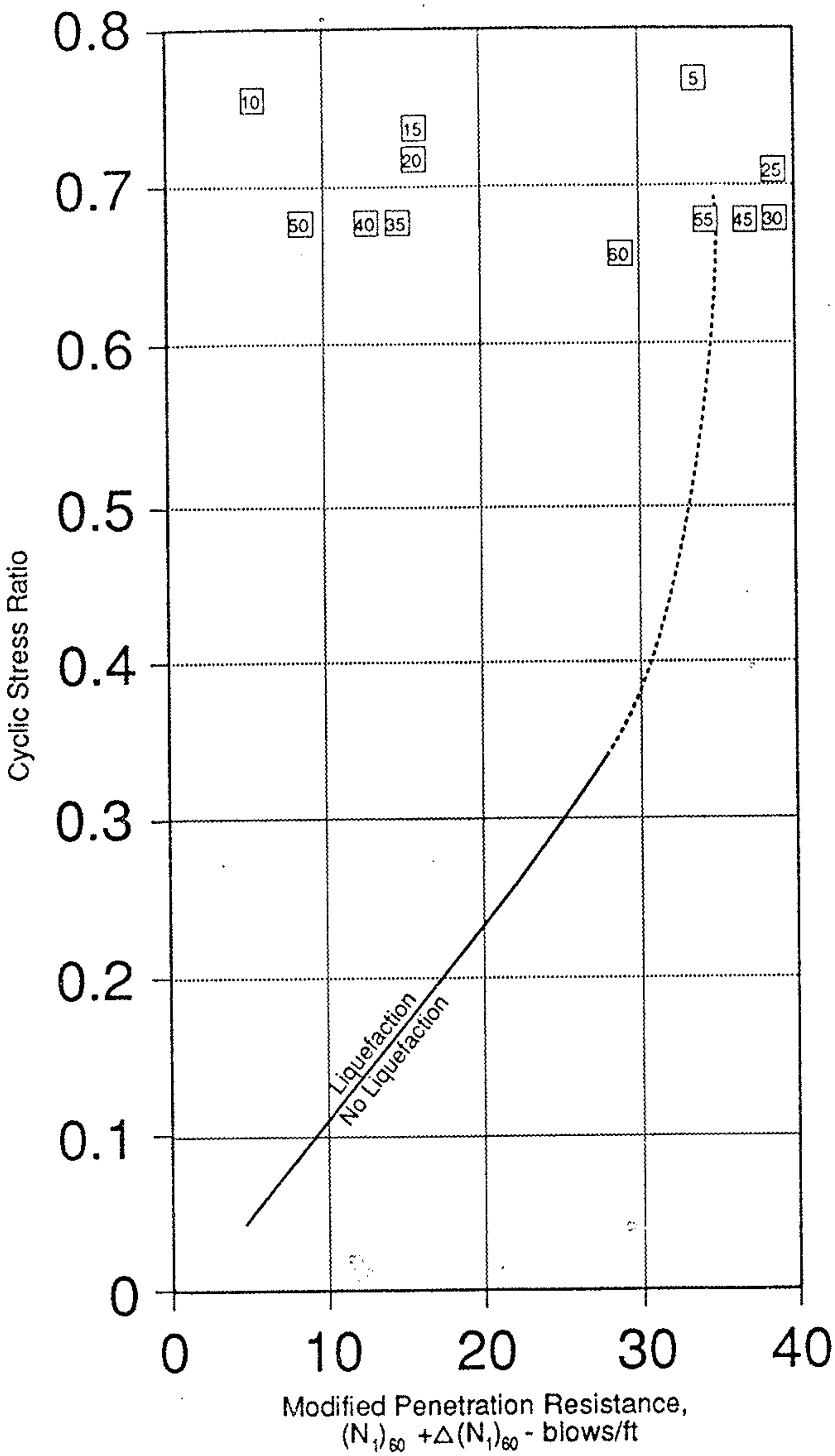
0.5

Density of Soil:

110 to 130 pcf

FIGURE 6:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).

CALTRANS



Bridge Name:

Route 56/5 Separation

Bridge No.:

57-0989F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-21L

Boring Location

on 1108+65 of "WS" line

Fault:

ROSE CANYON

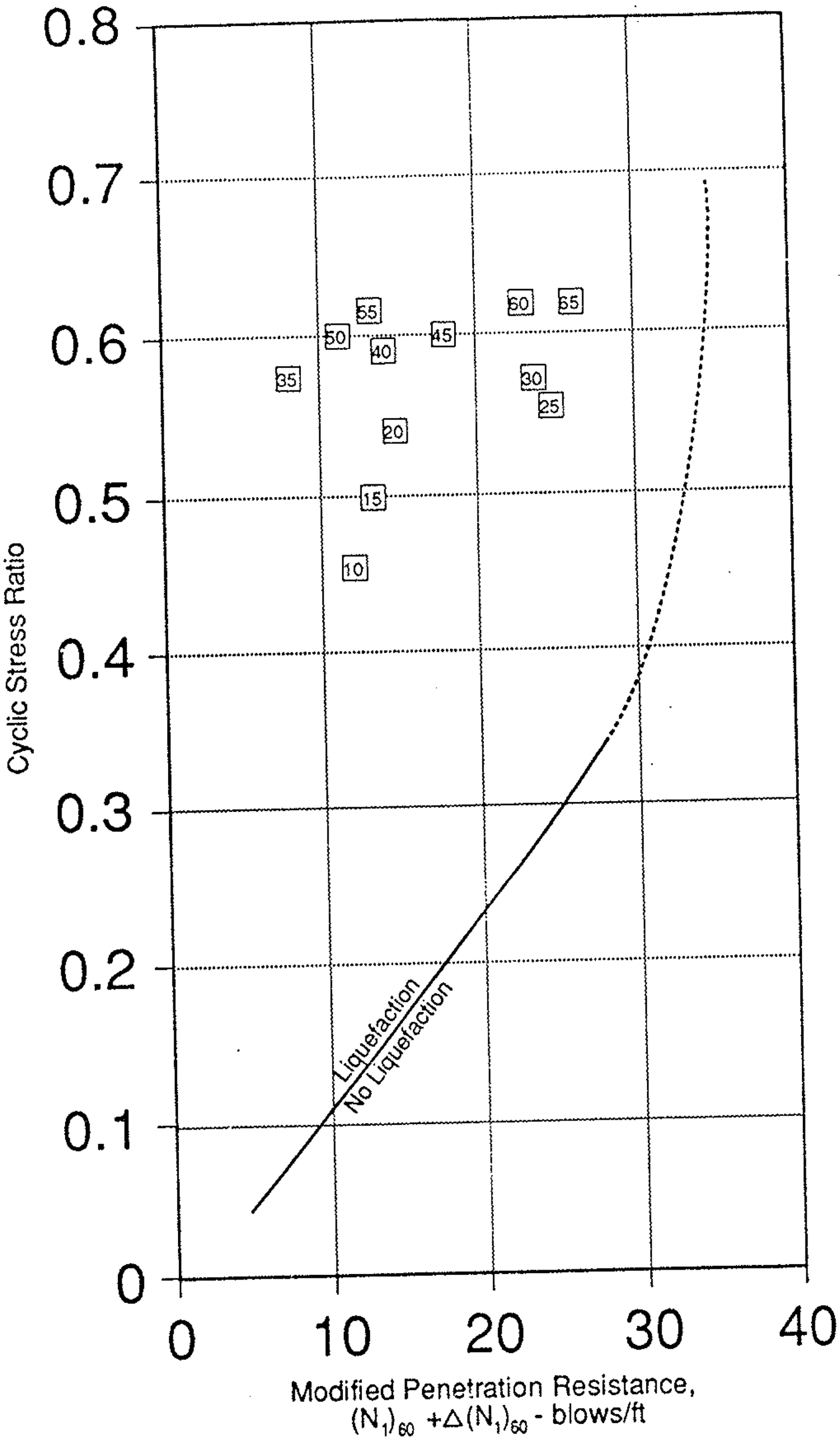
Peak Acceleration:

0.5

Density of Soil:

110 to 130 pcf

FIGURE 7:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).



Bridge Name:

Route 56/5 Separation

Bridge No.:

57-0089F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-22L

Boring Location

4' lt. of 1105+32 of "WS" line

Fault:

ROSE CANYON

Peak Acceleration:

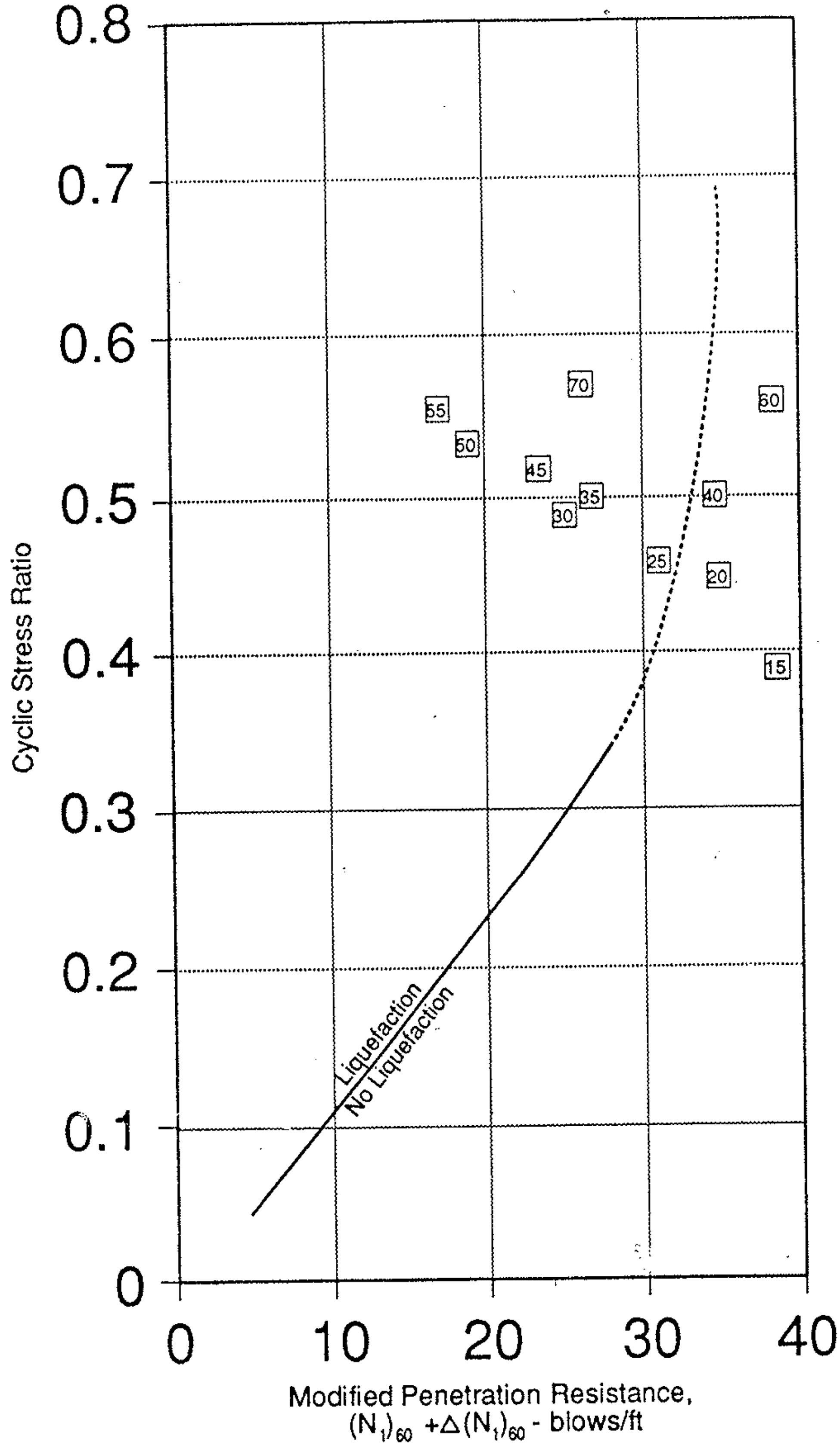
0.5

Density of Soil:

110 to 130 pcf

FIGURE 8:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).

CALTRANS



Bridge Name:

Route 56/5 Separation

Bridge No.:

57-0989F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-25L

Boring Location

21' rt. of 1095+60 of "WS" line

Fault:

ROSE CANYON

Peak Acceleration:

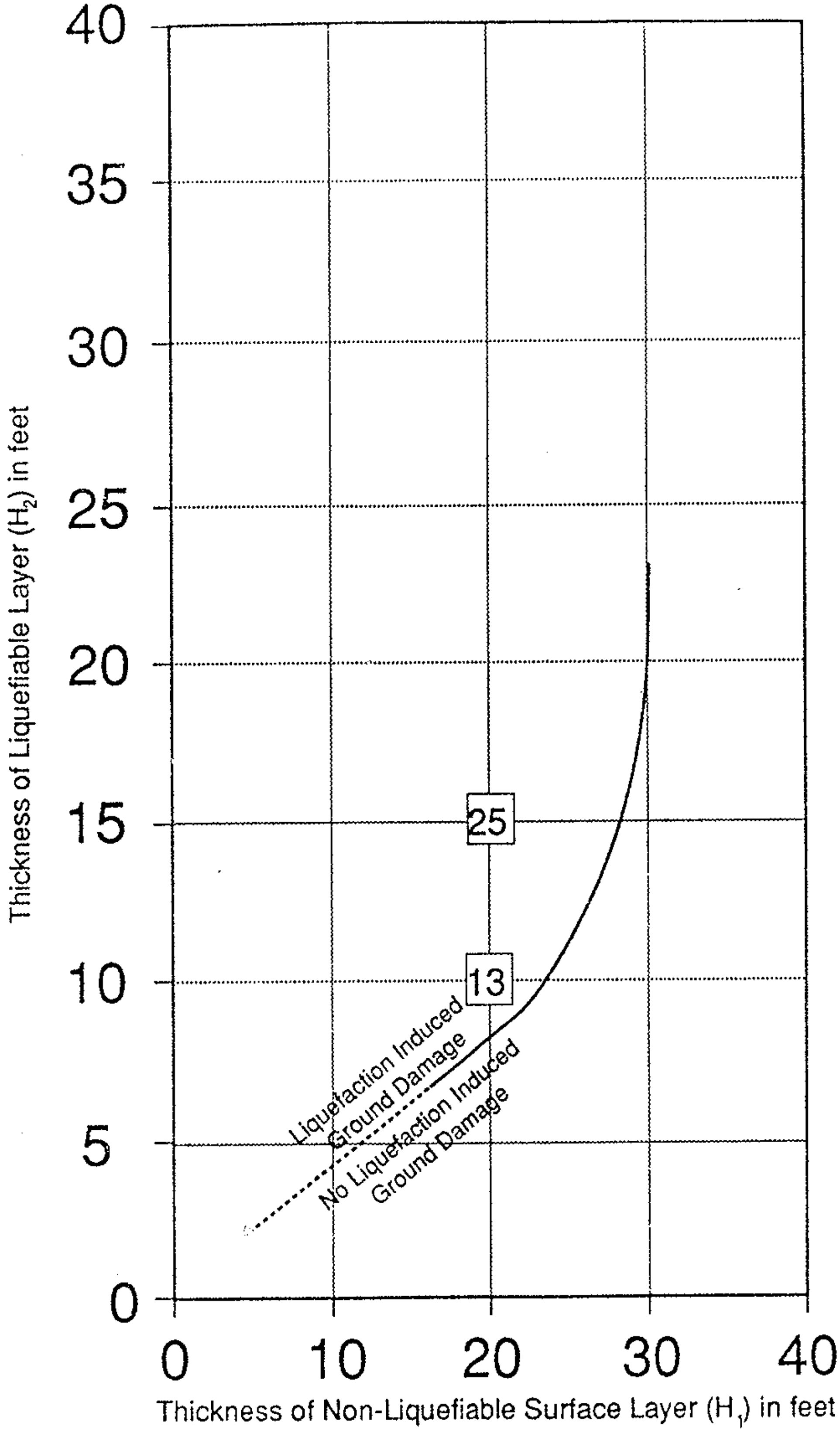
0.5

Density of Soil:

110 to 130 pcf

FIGURE 9:
Diagram showing Cyclic Stress Ratio vs. Modified Penetration Resistance for a M=7.0 earthquake (after National Research Council, 1985). Number in box indicates depth below ground surface. All points are corrected for fine content (Ishihara, in press) and contain <20% clay (Seed & Idriss, 1982).

CALTRANS



Bridge Name:

Route 56/5 Separation

Bridge No.:

57-0989F

Dist-Cnty-Rte-PM:

11-SD-56-0.00

Boring Number

B-13L & B-25L

Boring Location

Fault:

ROSE CANYON

Peak Horizontal Bedrock Acceleration:

0.5 gravity

Density of Soil:

110 to 130 pcf

FIGURE 10:
Diagram for evaluation of surface manifestation of liquefaction for a maximum acceleration of 0.4 to 0.5 gravity (after Ishihara, 1985). Number in box represents boring number with non liquefiable surface layer.

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