MITIGATION OF LIQUEFACTION HAZARDS AT THREE CALIFORNIA BRIDGE SITES

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ABSTRACT

Certain bridge sites in California are requiring ground and substructure modifications to overcome foundation strength loss due to liquefaction as a result of a major seismic event. This paper includes a discussion on two major interchange sites in the San Diego area and a third in the San Francisco Bay Area. The first site is being modified to develop increased lateral pile support via stone column islands. The second site uses underground stone column buttresses for minimizing lateral spreading. A third site, the east approach to the San Francisco-Oakland Bay Bridge overlies liquefiable sands that are susceptible to lateral spreading. Ductile moment resisting piles are recommended at the third site to provide resistance to accommodate the expected soil and inertial loads because overhead restrictions prevent ground modifications.

INTRODUCTION

The California Department of Transportation has been aggressively incorporating stringent geotechnical criteria into the seismic analysis of bridges since the Loma Prieta event of 1989. This event had caused significant damage to certain structures that were both designed before 1972 and founded over soft clay deposits in and around San Francisco. For example, it caused the collapse of approximately 1.1 km (0.7 miles) of the elevated Cypress (I-880) Viaduct, a structure constructed in the late 1950's and also founded on soft clay (Figure 1). Subsequently, the Governor's Board of Inquiry (1) assembled to review and comment on the damage, recommended, among other things, that "... the retrofit program should continue to focus on the structure as a whole, including its foundations and supporting soils. Elements should not be abstractly considered outside the context of their use."

Caltrans engineers immediately implemented a full scale geotechnical investigation program whereby site specific acceleration response spectra (ARS) based on maximum credible events, peak relative soil displacements, and liquefaction assessment would be routinely taken into account. This new program replaces standardized ARS curves and the assumption that peak relative soil displacements would have little or no impact on the sub-structure system. As a consequence, soil-pile-superstructure interaction analyses have since become a norm for many of our structures. However, serious technical and analytical problems arise when sites susceptible to liquefaction are encountered. Seismic motions passing through liquefied layers can be dramatically different than motions that pass through layers with no liquefaction, generally resulting in significant differences in the ARS (Figure 2a) as well as differences in peak relative soil displacements (Figure 2b), even though the deflection of the pile system may or may not coincide with the free field motion. Additionally, liquefaction can pose an even greater threat if there is a potential for large horizontal displacements due to lateral spreading. With these ideas in mind, the following three Caltrans projects are presented, illustrating, in a cursory fashion, the geotechnical design philosophy utilized in evaluating sites where-in liquefaction hazards are present, as well as the ground and sub-structure modifications proposed for design.

Route 5/56 Interchange (Proposed)

The 5/56 Interchange is a new construction project lying approximately 24 km (15 miles) north of San Diego (Site 1 on Figure 3). New construction will begin in 1995 and consist of embankments to 12 m (40 ft) in height, and elevated connectors approximately 24 m (80 ft) high at their apex and 910 m (3000 ft) long. The interchange will be located in a tidal flat area at a distance of about 5.6 km (3.5 miles east) from the active strike-slip Rose Canyon Fault. This fault has a maximum credible event of moment magnitude M_w=7 which generates a peak bedrock acceleration of 0.6g.

More than 100 borings including Standard Penetration Tests (SPT's) together with a number of Cone Penetrometer Tests (CPT's) provided information on the soil conditions. The foundation soils consist of young alluvial and estuarine sediments comprised of clays, silts and sands that vary in depth and consistency throughout the area. Corrected blow counts $(N_1)_{60}$ in the upper 15 m (50-ft) of the sandy deposits vary from 3 to 50 with the predominate portion falling between 10 to 18. Ground water elevation fluctuates seasonally from the ground surface to 3 m (10 ft) below ground surface. Depth to bedrock varies from 6 m (20 ft) to 30 m (100 ft).

Figure 4 illustrates the soil profile for a portion of the site and is fairly typical of other sections. This section also shows a typical slope of less than 1%.

Analysis

The analysis commenced in 1991 with downhole seismic surveys for the development of shear wave velocity profiles for use in computing ground motions. These profiles are used in program SHAKE to provide estimates of peak ground accelerations under total stress conditions as well as estimates of peak spectral accelerations (Figure 5). In figure 6 an example is given of the range of peak cyclic stress ratios as determined by the ground motion study. These ratios are used to estimate liquefaction potential. Blow counts were corrected for percent fines (-#200) and plasticity (PI). Recognition of the liquefaction potential warranted concern in three areas: 1) Loss of lateral support for the pile foundation; 2) Excessive pile curvature due to large relative soil displacements and/or lateral spreading; and 3) Approach fill instability.

Because of the ±60 acre areal extent of the interchange, ground modification of the <u>entire</u> site would not be feasible by any technique. Consequently, ground modifications were made with stone columns <u>only around</u> each of the 35 bents supporting the large connectors and at each approach fill where-in instability by overturning posed a problem.

In a quasi-static analysis method to determine the survivability of the substructure, the first step is to determine the soil spring constants (P-Y curves) for the liquefied state. A literature search (1991) failed to uncover any information on appropriate P-Y values to use. Therefore, the assumption was made to use a soft clay criteria (Matlock (2)) with a residual shear strength based on the liquefied state (Alba et al Figure 7) and a 2% strain for ε_{50}. Information gathered since then (3) indicated that 10% would be a closer approximation for ε_{50}. A lower bound $(N_1)_{60}$ value of 10 was chosen for the liquefied state to be consistent with the residual shear strength.

This information was then used to determine the deflection of a ductile pipe pile system consisting of twenty-four 406 mm dia, x 12.7 mm thick (16 in x 0.5-in) piles per bent. The deflection study was based on a single pile analysis program (BMCOL76 (4)), modified by Caltrans to incorporate non-linear beam elements, a much improved convergence algorithm, and maximum curvature warning statements). Peak inertial loads applied to the pile system were determined based on the total stress ARS. At that time it was questionable whether or not a total stress analysis would be too conservative but it was argued that the proposed ground modification would require leaning more towards a total stress approach.

Assuming that liquefaction would extend to a depth of 12^+ m (40 ft) with a constant residual shear strength of 19.2 KPa (400 psf), it was determined that buckling of the pile system would occur at 60% of the design load. After evaluating various ground modification techniques, 30 m (100 ft) diameter stone column islands (Figure 8) turned out to be the most cost effective alternative for increasing lateral pile support. The principal mechanisms for such support include quick drainage and the densification of the surrounding soil. Using the assumption that a $\emptyset = 30°$ would prevail within the stone column islands reduced the deflections significantly (Figure 9).

Preliminary field testing of 0.9 m (30 in) diameter stone columns placed at 2.4 m (8 ft) C-C spacing to determine the increase in foundation strength indicated a dramatic improvement in the CPT tip resistance between 1.5 to 6.1 m (5 to 20 ft) and 12 to 16.7 m (40 to 55 ft). At that particular test location, only slight increases were noted between the 6.1 to 12 m (20 to 40 ft) depths (Figure 10).

Lateral spreading estimates were attempted, but due to the variability of the non-liquefiable soil deposits at spot locations and the small surface gradient (< 0.7%), any flow whatsoever was assumed to be absorbed by a crush zone in the fringe of the stone columns at any impacted island, thus preventing additional loading on the substructure system.

The installation of stone columns is also planned for areas with high approach fills where-in safety factors less than 1.0 were computed. Residual shear strengths were incorporated as soil resistance factors for the liquefied zone beneath the embankments and from there, a normal static analysis was carried out. Stone column dimensions were chosen to provide minimum factors of safety of at least 1.05 (Figure 11), for all approach fills.

8/805 Interchange (Retrofit)

The 8/805 Interchange was constructed in 1972 in San Diego (Site 2 on Figure 3) and requires extensive seismic retrofit of the substructure and columns. Interstate 805 is the main viaduct at this site with a N-S orientation, a span of some 1500 m (5000 ft), a 36 m (120 ft) height, and a cast-in-place, prestressed box girder design with eight reinforced concrete connector ramps attached (See photo 1). The main viaduct joins the north and south canyon bluffs which are separated by the wide flood plain of the westerly flowing San Diego River, and the east-west I-8 freeway. Foundation soils to depths of 12 m (40 ft) below the original ground (OG) consist of loose to moderately dense deposits of sand and gravel interspersed with clayey silt and silt layers. Beneath these deposits lie very dense sands and gravel mixes which extend to the sandstone/siltstone deposits some 18 m (60 ft) below OG, (see Figure 12). The water table remains near the river elevation throughout the year Beneath the structure, portions of the canyon flood plain have been overlain by well compacted fill to thicknesses of up to 6 m (20 ft) for the support of several multi-story businesses and associated parking facilities. The ground slopes toward the river and varies in grade from 2-7%. Two active strike-slip faults are located nearby, the Rose Canyon (M_w=7.0) and the La Nacion (M_w=6.75), both capable of generating a peak rock acceleration between 0.5 and 0.6g's. Fault distances are 5.4 km east and 3.8 km southwest, respectively.

Analysis

More than 50 site borings including SPT's exist and indicate a range of $(N_1)_{60}$ blow counts between 6 to 59 in the upper native 8 m (26 ft) with a mean lower bound lying between 10 to 20. To model the liquefiable soil, an average value of 12 was used as a basis for design. For these approximations of the liquefiable soil, a peak ground acceleration of 0.6g's was estimated using the total stress analysis program, SHAKE, while the design ARS was based on the average of the ARS from SHAKE and the ARS from an effective stress analysis program SUMDES (5). Since the required ground surface acceleration for initiating liquefaction was less than 0.2g's, inertial loads from the design ARS were imposed on the pile system while using soil resistance values for liquefied conditions as per the previous site study. An s_u of 28.8 KPa (600 psf) and an ε_{50} of 10% were implemented for estimating the liquefied P-Y relationship. From there, pile cap deflections were computed. Initial studies included full inertial loads and indicated significant lateral instability of the existing H-pile system for bents near the San Diego River where liquefaction extended to the ground surface.

Further studies in which peak relative soil displacements (0.5 m (1.7 ft) computed from SUMDES assuming horizontally layered conditions), coupled with 25% of the peak inertial loads, were imposed on the pile using a single pile analysis. The results indicated that many more additional bents would fail by hinging and/or buckling. Furthermore, large overturning forces were exceeding the piles uplift capacity in certain bents due to weak pile-to-pile cap connections and/or insufficient uplift resistance as a result of strength loss in the liquefied zone (strength = zero for all zones where $(N_1)_{60} \leq 25$).

New pile systems were therefore designed consisting of 42-inch diameter steel pipe piles for the viaduct, and 24-inch diameter pipe piles for the ramps. Depending on bent location these piles were designed to accommodate up to 0.9 m (3 ft) of relative deflection over a pile length of 7.6 m (25 ft) before ductile capacities were exceeded. The new piles will be grouped around the old pile system and the existing cap will be extended to accommodate the additional piles.

Because relative displacement, rather than load, was the controlling factor for pile and superstructure design, it became important to evaluate any additional movement due to lateral spreading and then incorporate it into the design. Several factors against lateral spreading such as the fact that blow counts were not uniformly small and the existence of plasticity in portions of the profile where there are low blow counts (PI @ 10-20), were overridden by the following factors supporting the likelihood of lateral spreading: a) an open face to the river, b) impedance of vertical drainage due to the impervious nature of the overlying fill causing high excess pore pressures to persist for some time in the loose to moderately dense native strata resulting in continued straining due to gravity loads, c) large magnitude post mainshock loading very soon after the mainshock is likely, and d) the extremely dense nature of the overlying fill and its proximity to the cap and column, resulting in displacements that coincide with the fill Thus, a number of published papers on lateral spreading were examined (3, 6, 7, 8) and displacements were computed based on the product of limiting strain (γ_{lim}) (9) and the thickness of the liquefiable layer (assumed at 40% and 7.6 m (25 ft), respectively). Potential displacement estimates ranged from 1.5 to 4.5 m (5 to 15 ft), with an average value of 3 m (10 ft).

Ductility calculations indicated that a maximum additional displacement of 0.3 m (1 ft) beyond the peak relative displacement of 0.5 m was tolerable and resulted in the conclusion that ground modification was necessary. Due to the site's large areal extent, only localized ground modifications were considered economical through the use of stone columns acting as an underground buttress. The next best alternative is permanent dewatering. Mobilized shear resistance for this additional displacement was computed under the assumption that it would develop proportionally to the full residual strength (28.9 KPa (600 psf) at the 3 m (10 ft) displacement, Figure 13, Byrne (3)). Thus, a minimum of 1.44 KPa (30 psf) was estimated as the average mobilized strength for the 0.3 m (1 ft) displacement. Incorporating this strength into static stability calculations, stone column dimensions were determined assuming the treated areas would develop a phi of 39°. Figure 14 illustrates the static factor of safety for the 'before' and 'after' stone column addition. A plan view of the stone column area is shown in Figure 15. Since lateral spreading could be skewed to the downhill direction, the buttress was carried parallel to the drip line of the viaduct on the north side of the river. This ground modification location scheme was also dictated by right-of-way restrictions that limited construction activities to the confines of state-owned property.

Oakland Bay Bridge - East Approach

The San Francisco-Oakland Bay Bridge was constructed in the mid 1930's and is considered to be one of the engineering marvels of our time. At that time it was the world's longest steel structure [7 km (4.4 mi)], consuming 18 percent of all the steel fabricated in the United States in 1933. It includes the deepest pier in the world [Pier E3, 71 m (235 ft)] with its tip located on a stratum of stiff clay and sand just short of bedrock at 91 m (300 ft).

Figure 16 shows a plan and aerial view of the West Bay suspension crossing, a portion of the East Bay crossing, and Yerba Buena Island. The east approach cannot be seen in the aerial view, but it is on this section (Piers E23 to E39) that the discussion centers. The east approach superstructure consists of a number of simple-span deck systems that use steel and concrete stringers supported on transverse concrete bents. The pile systems consist of 0.45 m (18 in) tapered timber piles supported primarily by friction in sand and clay sublayers. Two strike-slip faults, the San Andreas (M_w=8.0), and the Hayward (M_w=7.3) are located 22 km (13.8 mi) west, and 7.5 km (4.7 mi) east respectively. Peak site rock accelerations were estimated at 0.5g's.

Figure 17 illustrates the soil profile between Piers E23 - E39, along with a typical cross-section of the bridge. The upper 6.1 to 9.1 m (20 to 30 ft) of fill is hydraulically placed with $(N_1)_{60}$ values between 7 and 30, with an average lower bound between 8 to 10. The fill was placed during the time of construction and displaced up to 5.5 m (18 ft) of the soft bay mud. The undrained shear strength of the soft bay mud ranges from 7 KPa (150 psf) near the mud line to between 22 to 37 KPa (460 to 800 psf) just beneath the fill. Not shown are the sands, silts, and stiff clay mixtures which prevail below this soil to depths of 140± m (460 ft) where bedrock is encountered.

Analysis

Output from SHAKE demonstrated that the ARS varies only slightly from the pile cap level, approximately 1.5 to 3 m (5 to 10 ft) below top-of-fill, to the bay mud-fill interface. Analysis also revealed that the sand fill would liquefy. This liquefaction results in reduced pile resistance and lateral spreading into the bay causing additional pile loading where the head scarp extends into the substructure.

An analysis of the pile foundation using peak inertial loads, liquefied P-Y relationships of the sand based on its estimated residual strength of 14.4 KPa (300 psf), and cyclically degraded strength of the soft bay mud indicated (not surprisingly) complete failure of the timber piles. Due to overhead restrictions of the lower deck to top-of-fill (2.4 to 5.5 m (8 to 18 ft)) and restrictions on lane closures, ground modifications were extremely limited and a decision was made to bypass the liquefied sands with ductile moment resisting piles.

The overhead space limitations prevented the use of a standard pile installation process. A specialty pile by FUNDEX (10) was examined as the best alternative. The recommended FUNDEX pile is a 508 mm (20 in) OD steel pipe pile with a thickness of 25 mm (1 in) at its top and thinner 12 mm (0.5 in) thickness at depth and has a unique 673 mm (26.5 in) fluted tip for helping relieve tip and side friction as the pile is rotated to depth. Piles are segmented into 3 m (10 ft) sections and butt-welded as they are driven. Cement slurry grout, placed under pressure, will fill the annulus between the soil and the pile throughout its length. This reduces gapping and increases P-Y stiffness and corrosion resistance. The new piles will be placed around the existing pile system and new pile caps will be constructed. (As a side note, the excavation required for this operation adds to the project complexity and cost since both the soil and the water are considered hazardous due to high concentrations of lead from repeated painting operations, and must be either cleaned before replacement, or disposed.)

The number of piles to be used at each bent was based on the peak inertial loads determined from the ARS in addition to imposed loads as estimated from soil pressures induced by lateral spreading. Lateral displacement estimates were made using a mechanistic method by Newmark (11) where-in the full residual strength of the liquefied sand was considered. Random block failure surfaces in conjunction with a progressive failure technique proposed by Idriss (12) were used to evaluate the advancement of the scarp head. Strength reductions to 1/3 of the initial undrained shear strength of the soft clay were considered where critical failure planes penetrated into this strata. The 1/3 s_u value was based on the fact that the sensitivity of the bay mud in this vicinity is in the range of 2-4.

The analysis, in general, assumes that for the design earthquake, the peak ground acceleration (PGA) is at a maximum at the beginning and tapers logarithmically to zero. For simplicity, a linear reduction was assumed on this project. The Hayward Fault (M_w=7.3 earthquake) was determined to be the most damaging for this location and was considered to have twelve significant stress pulse cycles associated with its duration Using two cycles per failure surface as a trial, the PGA reduction relationship described above, and the seismically induced displacement relationship proposed by Makdisi and Seed (13) (Figure 18), a progressive failure scheme evolved based on the M_w=7.5 curve after reducing it to a per cycle basis. For each time step, cross-sectional geometry continually flattened to accommodate a likely progressive failure scenario. Figure 19a - 19e summarizes the results of such an analysis for a portion of the east bay approach.

Using the results of this analysis as a guide, an estimate of the possible extent of the head scarp into the substructure region was gleaned. Based on this analysis, lateral pressures equivalent to a value of up to 50 percent of the passive pressure on the pile system for 1/2 the full cap width (maximum extent of scarp head) was imposed as an additional load on the pile system in the lateral direction. Due to a possible skewing of the lateral spreading material, a component of this load was taken longitudinally and added to the longitudinal inertial loads. Figure 20 illustration the results of our quasi-static analysis of the newly designed pile system. Maximum lateral deflections of the subsurface system are estimated to be in the range of 25 to 75 mm (1 to 3 in).

Axial and lateral load tests on the proposed pile system will start sometime in late 1994 with bridge reconstruction to begin shortly thereafter. Further studies incorporating a full dynamic soil-pile interaction analysis will be pursued using the Program PAR (14). It has an extensive support base for P-Y and T-Z cyclic degradation and radiation damping and can include multi-pile systems as well as superstructure components. The program runs on a DOS based PC and preliminary results look promising. Results compare quite favorably with a quasi-static analysis using another project as a test case. Results on the Bay bridge using this approach will be completed by November 1994 and included for review and design.

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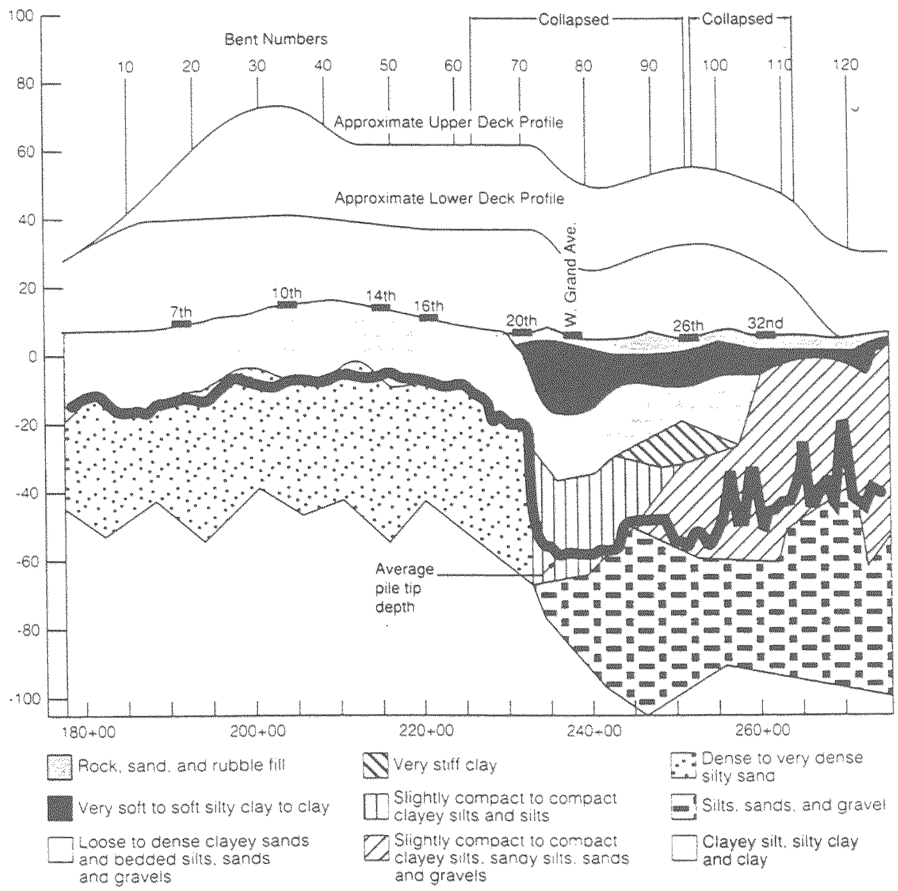

Fig. 1 Soil Profile of Cypress Viaduct Based on Boring Information From 1953-1990

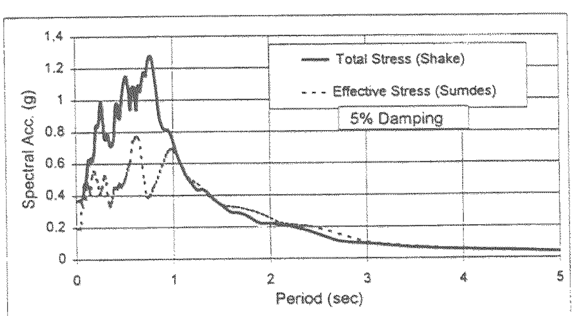

Fig. 2a Comparison of ARS Between Effective And Total Stress Analysis

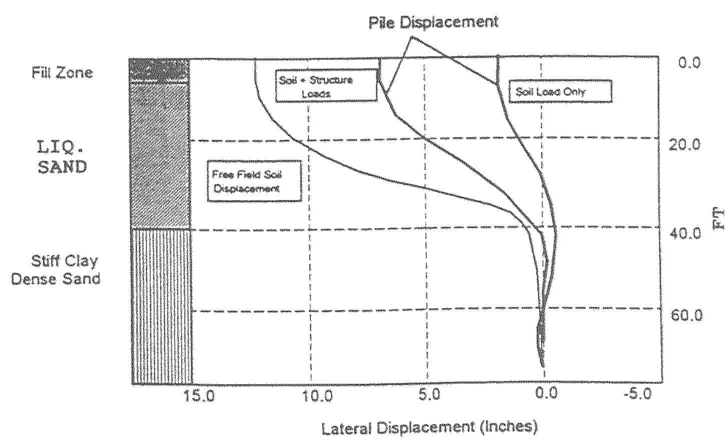

Fig. 2b Loaded Pile and Free Field Soil Displacement Profile

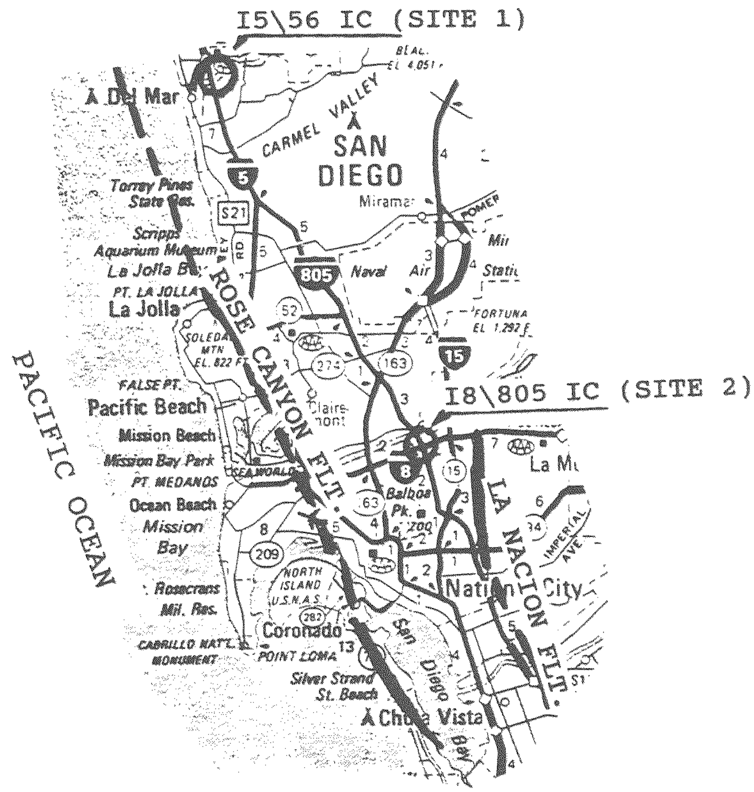

Fig. 3 Site Map of San Diego Vicinity

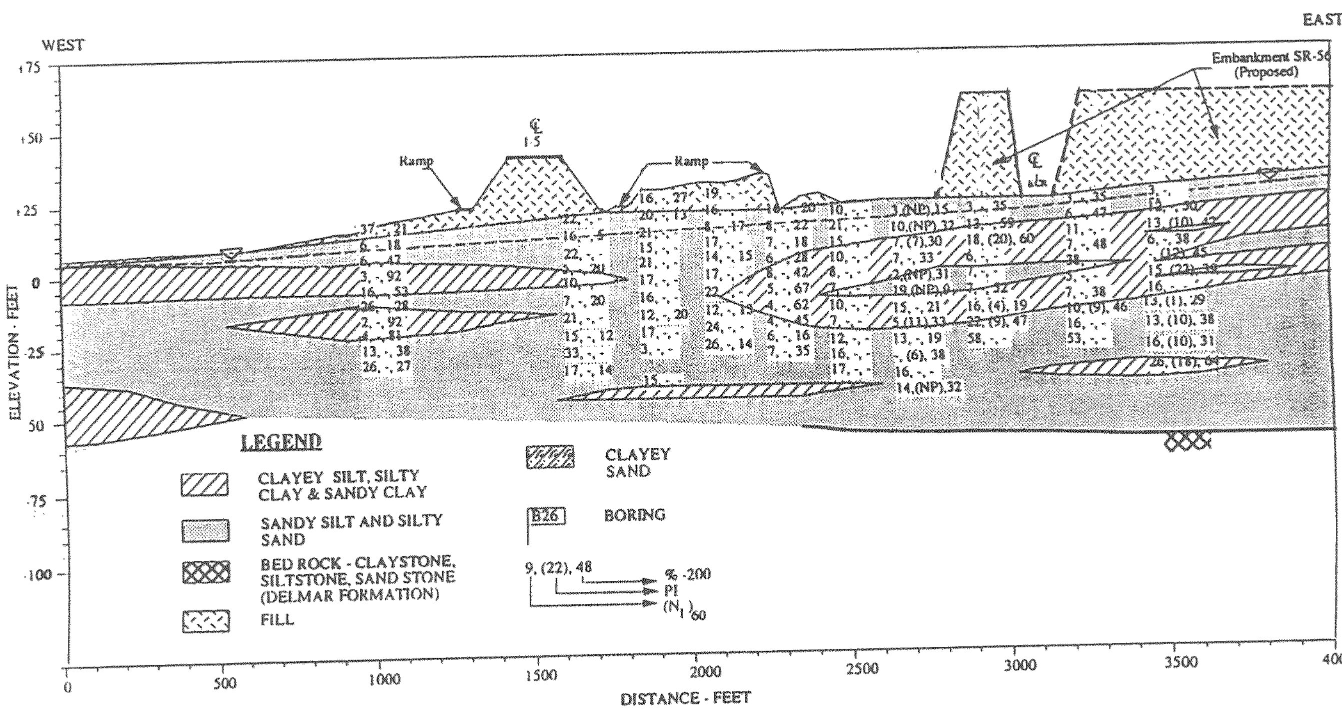

Fig. 4 Soil Profile for Portion of the I-5/56 Interchange

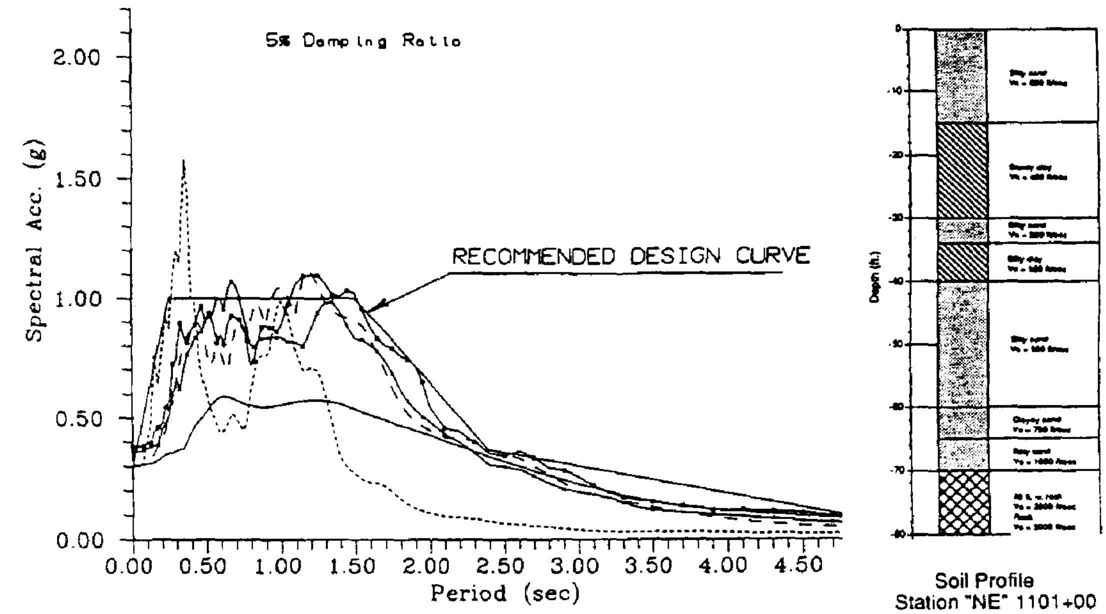

Fig. 5 Acceleration Response Spectra for I-5/56 Interchange

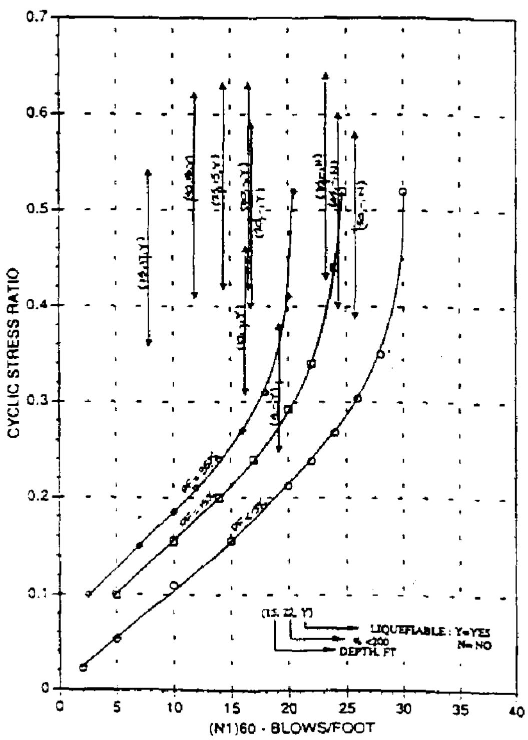

Fig. 6 Liquefaction Potential

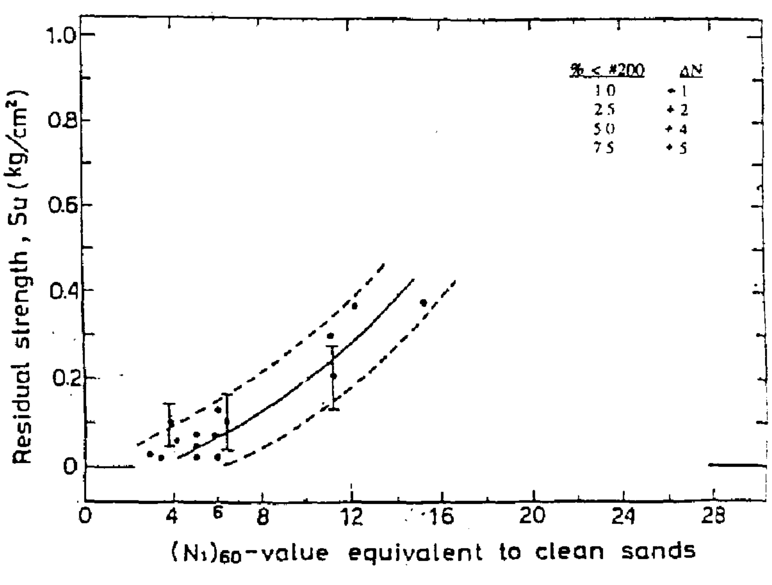

Fig. 7 Residual Strength of Liquefied Sands (Alba et al. 1988

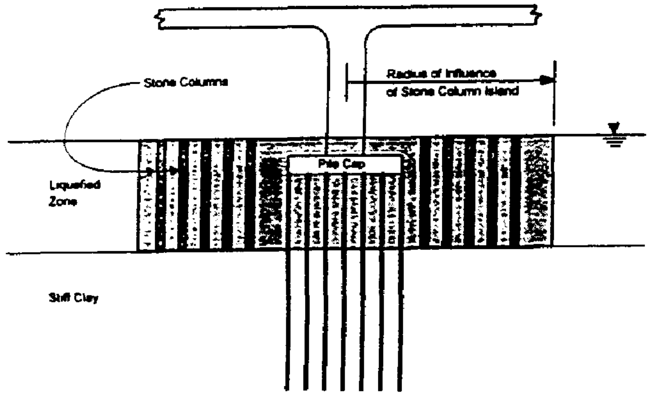

Fig. 8 Schematic Illustrating Stone Column Island Providing Substructure Support in the Liquefied Zone

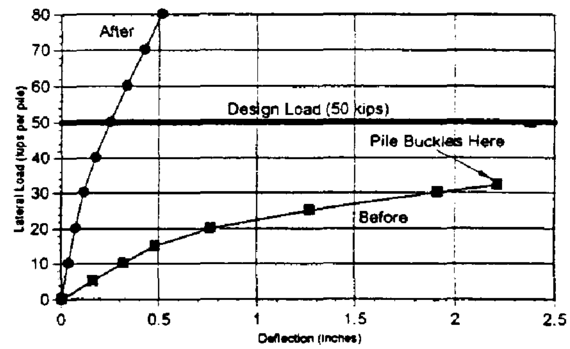

Fig. 9 Load-Deflection Curves for I-5/56 Interchange Before and After Ground Modification

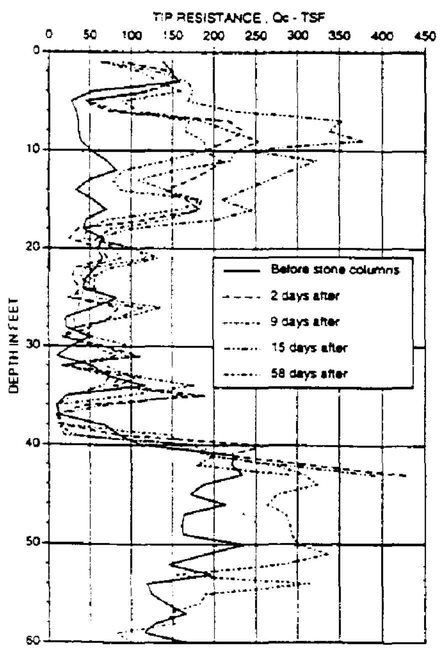

Fig. 10 Cone Tip Resistance Varying With Time After Placement of Stone Column

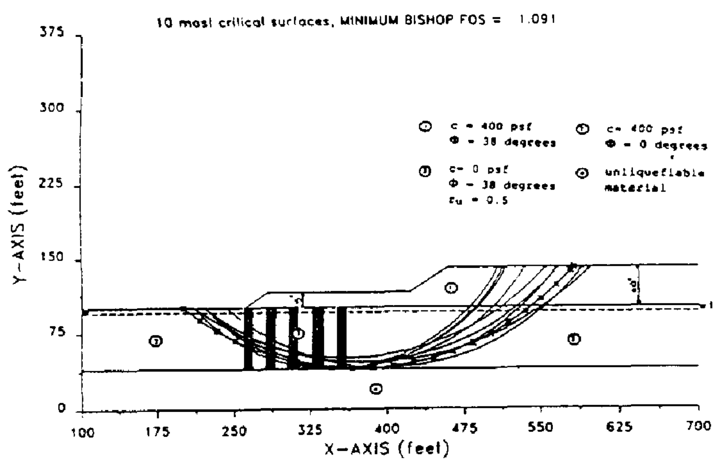

Fig. 11 Stability of 40-ft Embankment With Berm And Stone Column Support