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ABSTRACT

The North Torrey Pines Road Bridge in Del Mar, California was built in 1933 and is eligible for listing in the National Register of Historic Places. As a result of its outdated design and deterioration in a corrosive saline environment, the bridge was classified as structurally and seismically deficient and functionally obsolete. The historic significance of this bridge is important to the surrounding community and thus a seismic retrofit project was initiated with the goal of improving the expected seismic performance of the bridge while preserving its aesthetic and historic character. This paper provides a brief description of the overall retrofit design strategy, and detailed descriptions of the design of compaction grouting ground improvement to mitigate liquefaction and seismic slope instability hazards. Techniques used in the compaction grouting construction are presented, along with some particular construction challenges and solutions. Pre- and post-construction Standard Penetration Test data are compared and the improvements to the soil are discussed. The compaction grouting program was successful in achieving the ground improvement levels required by the design.

INTRODUCTION AND BACKGROUND

The 590 foot (180 meter) long North Torrey Pines (NTP) Bridge is located in Del Mar, California, approximately 19 miles (31 km) north of San Diego. The bridge was constructed in 1933 and was part of the historic Pacific Coast Highway 101, which was the primary north-south route linking southern California's coastal cities prior to the completion of Interstate 5 in the 1960s. The NTP Bridge is on the California Register of Historic Places, is eligible for listing in the U.S.'s National Register of Historic Places, and is valued by the local community for its historical significance and aesthetic appeal. A photograph of the bridge is presented in Figure 1.

The NTP Bridge was determined to be seismically deficient and structurally obsolete. A seismic retrofit project was embarked upon for the bridge with the goals of improving the seismic resistance of the bridge while preserving the aesthetic qualities that are important to its historical significance. This required that the bents, whose structural members were deficient, could not be replaced, increased in size or changed in finish texture. These limitations led to a retrofit strategy that included replacing the bridge deck, seismically isolating it from the bents using sliding bearings, constructing new cast-in-drilled-hole pile foundations at the abutments, and performing compaction grouting to improve the ground around existing pile foundations.

This paper describes the site and subsurface conditions, the compaction grouting ground improvement design and construction, and pre- and post-improvement penetration test results.

Fig. 1. Looking south-southwest at the North Torrey Pines Bridge.

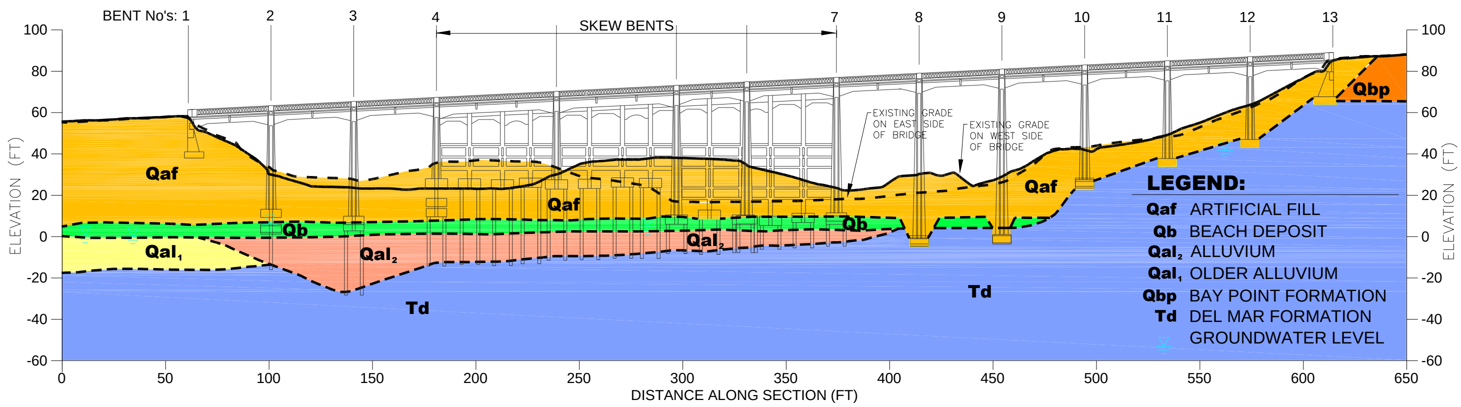

Fig. 2. Cross section showing bridge structure and subsurface stratigraphy.

SITE AND SUBSURFACE CONDITIONS

The bridge is located along the coast of the Pacific Ocean, as close as 200 feet (60 m) from the high tide line. It is situated at the north end of an alluvial valley, with the northern-most quarter of the bridge ascending the valley's sloping boundary. The bridge spans over a state park access road and the San Diego Northern Railway (SDNR) line. It was constructed with three bents skewed at 63 degrees from the longitudinal axis of the bridge that accommodate the railroad and its embankment.

The elevation of the bridge deck ranges from 60 feet (18 m) (all elevations are with reference to mean sea level) at the south abutment to 85 feet (26 m) at the north abutment. Topography at the site is variable due to the presence of an approach embankment at the south abutment, the railroad embankment that runs under the bridge, and the alluvial valley border. The southern approach embankment is about 30 to 45 feet (9 to 14 m) in height (east and west sides, respectively) with side slope inclinations on the west, north and east sides of approximately 1.5:1 (horizontal to vertical).

During the design phase a subsurface field investigation was performed that consisted of sixteen borings ranging in depth from 6 to 160 feet (2 to 49 m), five cone penetrometer test (CPT) soundings ranging in depth from 28 to 56 feet (8.5 to 17 m). Shear wave velocity measurements were made with a combination of borehole P-S suspension logging, spectral analysis of surface waves (SASW), and seismic CPT (SCPT). An array of geotechnical laboratory tests were performed on soil and groundwater samples obtained from the field investigation.

Eight geologic units and artificial fill soils are present at the site. Basement rock was encountered at an elevation of -120 feet (-36.5 m) and consisted of Cretaceous Lusardi Formation boulder conglomerate. Interbedded claystone and sandstone of the Eocene Delmar Formation (T_d) overlay the basement rock. Late Pleistocene clayey sandstone Bay Point Formation (Qbp) caps the Delmar Formation north of the alluvial valley. Within the alluvial valley, late Pleistocene- to Holocene Alluvium ($Qal_1$and Qal_2, respectively) and modern Beach Deposits (Qb) were present. The interpreted stratigraphy longitudinally along the bridge is shown in Figure 2. Groundwater is present at an elevation of approximately 5 feet (1.5 m) above mean sea level.

As-built drawings show abutments 1 and 13 and Bents 8 through 12 supported on spread footings. All of the spread footings bear upon the Delmar Formation claystone/sandstone except for the Abutment 1 footing which bears upon fill. Bents 2 through 7 and the skewed bents are supported on 16 inch (400 mm) square reinforced concrete piles that penetrate into dense alluvium and/or Delmar Formation.

GROUND IMPROVEMENT DESIGN

Details of the overall geotechnical analysis and design effort are presented in Gingery et al. (2009). The following presents details of the liquefaction hazard analyses and its mitigation by compaction grouting ground improvement.

Liquefaction susceptibility was evaluated using the simplified procedure of Youd et al. (2001). The Youd et al. (2001) procedure was selected because at the time it was considered by the designers to be the only "expert consensus" methodology available. The design earthquake parameters used in the liquefaction analyses were a moment magnitude of 7.2 and a peak ground acceleration (PGA) of 0.57g. The liquefaction susceptibility analyses showed potentially liquefiable alluvium and beach deposit soils at the site along the bridge alignment from approximately Abutment 1 to Bent 9. North of Bent 4, the liquefiable layer was 5 feet (1.5 m) or less, but it was laterally consistent throughout the area. From Abutment 1 to Bent 4 the liquefiable soil thickness varied up to about 18 feet (5.5 m). Liquefiable soils were not observed in the area between Bent 10 and Abutment 13. The undrained residual shear strength for the liquefied layers was estimated as per Idriss & Boulanger (2007).

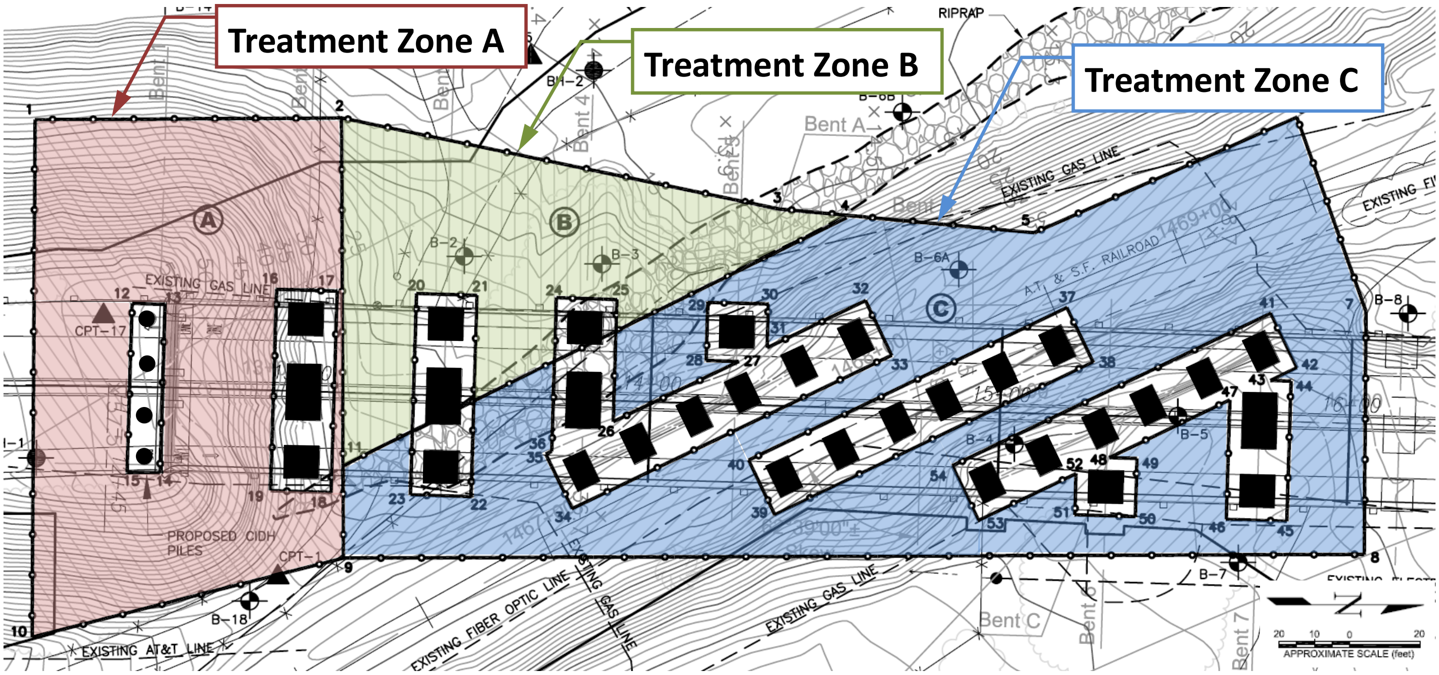

Fig. 3. Compaction grouting ground improvement plan. Rectangular solid black areas are existing pile caps. Circular solid black areas are new CIDH piles. White areas around foundations are treatment exclusion zones.

The liquefied soil strengths were used in the seismic stability analyses to evaluate flow failure and displacement potential of the slopes at the site. (Bray & Travasarou 2007 and Olson & Johnson 2008). The analyses were performed for ten cross sections that were considered representative of the slopes at the site. Analyses were also performed to evaluate the stabilizing effects of the existing pile foundations ("pinning effects") for two cross sections (Boulanger et al. 2007). The slope stability results indicated the west slope of Abutment 1 was prone to a liquefaction flow failure. The north slope of Abutment 1 and the slopes of the existing railroad embankment were prone to lateral spreading displacements ranging from 2 inches to 3 feet (50 to 1000 mm). These displacements were sufficient to cause plastic hinges to develop in the piles and to have unacceptable impacts on the performance of sliding bearings which were planned for the bent-girder connections.

Compaction grouting was selected to mitigate the liquefaction and slope stability hazards. The compaction grouting method was found to provide the most constructible and cost effective solution considering the confined work areas, limited overhead, sloping terrain, and the need to maintain active railroad and vehicular traffic. The mean Standard Penetration Test (SPT) blow counts, $(N_1)_{60}$, within the liquefiable layers was 13 blows per foot (bpf). The percent finer than the #200 sieve range from zero to 18 percent, with most values in the range of 5 to 10 percent. A mean post-treatment $(N_1)_{60}$ value of 25 bpf was established as the design criteria for the compaction grouting because: 1) a value of 25 was sufficient to cause the soil to become dilatant and nearly non-liquefiable and 2) this level of densification was believed to be achievable with compaction grouting based on the trends in pre- and post-treatment penetration testing reported by Boulanger & Hayden (1995). The minimum $(N_1)_{60}$ criterion was established as a mean value (rather than an absolute minimum for any single blow count), since the average shear strength of the slope stability slip surfaces was of concern in the design and since isolated zones of lower blow counts were judged not to significantly impact the seismic performance.

The lateral extent of the compaction grouting ground improvement is shown in Figure 3. Three Treatment Zones (A, B and C) with distinct treatment elevations were established. To avoid damage to the existing pile foundations, treatment exclusion zones were established within 5 feet (1.6 m) of the foundations. The lack of treatment within and immediately around the pile groups was considered acceptable since the original displacement pile driving would have densified these soils.

Treatment depth at Zone A was significantly different from Zone B and C because of existing topography. To achieve the target SPT blow count an Area Replacement Ratio (ARR) of 12.5% was chosen. The ARR is defined as the volume of grout injected within a column divided by the tributary volume of treated soil. An 8-foot (2.4 meter) square center-to-center spacing was used for Zone A, with 8 cubic feet (0.23 cubic meters) of grout injected per 1-foot stage, and 6 foot (1.8 meter) square center-to-center spacing was used for Zone B and C with 4.5 cubic feet (0.13 cubic meters) of grout injected per 1-foot stage. The closer spacing was used in areas where less overburden pressure was present. Test Sections were performed before production work begun and the chosen ARR was determined to be adequate to achieve the performance criteria.

COMPACTION GROUTING TECHNIQUES

Compaction grouting is a ground improvement technique that improves the strength and/or stiffness of the ground by slow and controlled injection of low-mobility grout. The soil is displaced and compacted as the grout mass expands. (ASCE/G-I 2010). The technique was originally developed in the 1950's as a remedial measure for the correction of building settlement, and was used exclusively for that purpose for many years. Currently, compaction grouting is utilized for a variety of geotechnical applications, including liquefaction mitigation.

Rotary drilling techniques were chosen over driving methods to advance the grout injection. Driving methods in sandy materials could lead to a false interpretation of refusal depth because of excessive friction on the casing. Compaction grout was installed in bottom-up 1-foot stages, until the desired volume was injected, a refusal pressure of 700 psi was reached, or ground movement was observed. For most of the locations, volume cut off was reached before pressure refusal or ground movement was observed.

Compaction grout data was uploaded from the field to the engineers for review on a daily basis. This procedure provided higher level of quality assurance on the project, by allowing the engineering staff to review field data quickly. Timely data transfer between the engineering and construction team allowed for greater transparency and helped in determining which areas required secondary or additional ground treatment and which areas did not. Availability of accurate data in timely manner allowed the project team to make appropriate changes.

CONSTRUCTION CHALLENGES AND SOLUTIONS

The engineering and construction of the ground improvement faced multiple challenges and restrictions, namely, densifying target zones with precision, noise limits, inaccessible areas of biological and environmental sensitivity, the presence of high pressure gas lines in the work area, and working around live railroad tracks. Moreover, North Torrey Pines Road is a busy thoroughfare, and the project required that traffic be maintained at all times during construction except for brief shutdowns periods at night.

Ground heave was monitored during grouting operations to assure prevention of excessive ground displacement which could damage existing utilities. The monitoring was also used to monitor compaction grouting effectiveness, since excessive ground heave is typically associated with inadequate confining stress to allow compaction to occur. A portable rotating horizontal laser with multiple receivers was utilized to monitor heave. The laser was located outside the zone of influence. Laser receivers were mounted on stands on grade at random locations within a horizontal distance from the injection point equal to depth of treatment. Periodic survey monitoring of the railroad tracks, existing piles and bridge was also performed throughout the compaction grouting duration.

The spatially-limited working areas were addressed by using low-overhead, limited access drilling and grout injection equipment. In addition, many compaction grout holes were angled to overcome the work area restrictions. Precision in the angled holes was achieved by surveying the injection point location, then calculating the bearing and dip required for the grout probe to reach the treatment zone. The number of grout injection stages for individual locations were modified based on the length of casing within the treatment zone.

Fig. 4. Grout injection in an angled grout hole adjacent to active railroad tracks and bridge.

PRE- AND POST-TREATMENT SPT TESTING

Cone penetrometer tests and SPTs were performed in the design phase prior to compaction grouting treatment, and SPTs were performed after treatment. Figure 5 presents the site-wide pre- and post-treatment $(N_1)_{60}$ data. Note that some of the pre-treatment $(N_1)_{60}$ values were converted from CPT tip resistance using the relative density correlations of Idriss & Boulanger (2008). Post-treatment mean values were calculated from three consecutive $(N_1)_{60}$ values. The post-test mean $(N_1)_{60}$ values exceeded the minimum mean $(N_1)_{60}$ value of 25 required by the specifications.

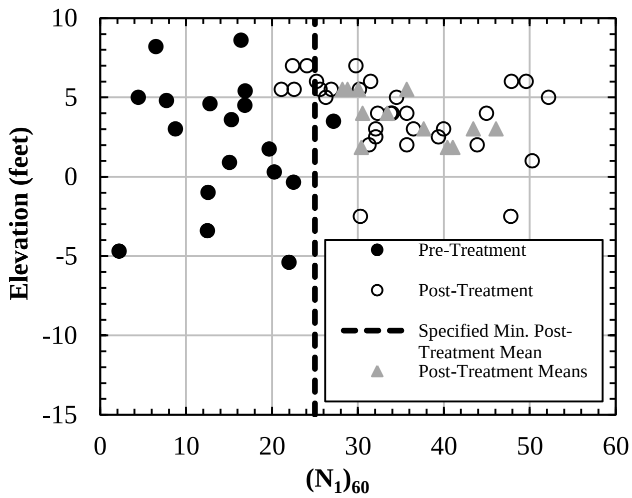

Fig. 5. Pre- and post-treatment SPT data, and the specified minimum mean $(N_1)_{60}$ value.

CONCLUSIONS

The compaction grouting program successfully densified the treated soils to the minimum mean $(N_1)_{60}$ value of 25 required by the specifications. This level of improvement mitigated the liquefaction and lateral spreading hazards at the site, thus allowing the existing pile foundations at the bent to remain in place without expensive and difficult to construct retrofit measures. The use of limited access grouting equipment and angled grout injection columns overcame the restricted working space at the site. Careful surveying during construction assured accurate coverage of the treatment zone and that existing utilities were not damaged by the grout injection.

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