Ground improvement at the Queensway Bay Downtown Harbor, Long Beach, California 7.03

Suji Somasundaram
Gamini Weeratunga
Kris Khilnani

Follow this and additional works at: http://scholarsmine.mst.edu/icchge

Recommended Citation
http://scholarsmine.mst.edu/icchge/4icchge/4icchge-session07/5
GROUND IMPROVEMENT AT THE QUEENSWAY BAY DOWNTOWN HARBOR, LONG BEACH, CALIFORNIA

Suji Somasundaram
Advanced Earth Sciences
Irvine, California-USA-92718

Gamini Weeratunga, Kris Khilnani
Advanced Earth Sciences
Irvine, California-USA-92718

ABSTRACT

Vibro-replacement with stone columns was selected as the optimum ground improvement solution to mitigate liquefaction potential, provide seismic stability, and provide adequate structure foundation support for a proposed harbor/marina development at an oceanfront site. The site is underlain by up to 27 meters of soft/loose hydraulic fills and seafloor sediments. An 18-meter wide ground improvement zone straddling the proposed 600-meter long bulkhead wall alignment was designed to create a non-liquefiable barrier that would prevent flow failures towards the lagoon, limit seismically induced deformations to acceptable levels, and allow the wall to be supported on shallow footings. The construction of a temporary earthfill construction platform served the dual purpose of permitting construction in the dry, and providing a preload that would help accelerate consolidation settlements of the soft cohesive soil layers underlying the bulkhead footing. Stone columns were also used to improve foundation soils below the mud line of a 60-meter long, pile-supported pier, to a level sufficient to provide adequate lateral and uplift capacities during seismic loading. This paper discusses the geotechnical design and field implementation of the ground improvement program.

KEYWORDS

Liquefaction, Liquefaction Mitigation, Seismic Stability, Seismic Deformation, Ground Improvement, Soil Densification, Stone Columns, Vibro-Replacement

INTRODUCTION

The oceanfront site for the proposed downtown harbor and marina in the City of Long Beach is located on reclaimed land within the shoreline aquatic park and the shallow Queensway Bay lagoon. The development (Fig.1), involves approximately 300,000 cubic meters of dredging and placement of 75,000 cubic meters of fill above water to create the harbor, a 600-meter long seawall/bulkhead which encroaches within the limits of the existing lagoon along the northern and eastern boundary, a promenade, a 60-meter long pier, and backland areas for commercial and recreational development. The bulkhead/slope configuration includes final grades to elevation +4.3 meters above mean lower low water level (MLLW) behind the bulkhead, and 3:1 (horizontal to vertical) slopes to elevation -6.7 meters MLLW in front of the bulkhead wall.

Fig. 1 Site Map
SUBSURFACE CONDITIONS

The entire site, including the lagoon bottom, is underlain by fills and hydraulic fills over loose to medium dense / soft to stiff native sea floor sediments extending down to a dense/hard alluvial deposit occurring at elevations ranging from -9 meters to -27 meters, MLLW. The fill above the water table consists of medium dense to dense sands (SP/SM). The hydraulic fills and the sea floor sediments below the water table (and underlying the lagoon bottom) are up to 27 meters thick and consist predominantly of loose to medium dense sands and silty sands (SP/SM), interlayered with soft to medium stiff low plasticity silts and clays (ML/CL). A typical subsurface profile perpendicular to the bulkhead line is shown on Fig. 2.

![Fig. 2 Typical Subsurface Profile](image)

The hydraulic fills and native sea floor deposits occurring between the near-surface dense sand fill layer and the deep alluvial deposit, are the most susceptible to liquefaction and/or significant strength loss. The loose to medium dense, poorly graded sands and silty sands occurring within this stratum exhibit equivalent corrected SPT (N)_{60} values (corrected for overburden and fines content) ranging from 2 to 30 (typically 5 to 20). The fines content (percentage finer than the #200 sieve) ranges from 4 to 45 percent, and is typically less than 20 percent. The cohesive soils interlayered with the sands within this stratum typically consist of silts or low plasticity clays (liquid limits ranging from 25 to 45 and plasticity indices ranging from 3 to 21). Their natural moisture content ranges from 29 to 50 percent and the clay content (percentage of particles finer than 0.005 mm) generally ranges from 10 to 52 percent.

SEISMIC EXPOSURE

Site seismicity is primarily influenced by the Newport Inglewood fault zone and the offshore segments of the Palos Verdes fault, located approximately 4.8 km to the northeast, and 6.4 km to the southwest, respectively. Two levels of earthquake shaking were considered for the design: a lower operating level earthquake (OLE) that has a 50% probability of being exceeded over the 50-year structure lifetime (72-year return period); and a higher, contingency level earthquake (CLE) that has a 10% probability of being exceeded in 50 years (475-year return period). The seismic design criteria required that the structures remain functional with minor repairs under the OLE, and survive the CLE without loss of life but possibly sustain damage that will require significant repairs.

A probabilistic seismic hazard analysis conducted to estimate potential ground shaking (response spectra) at the site treated faults as line sources and incorporated uncertainties associated with recurrence, rupture length and location, and the attenuation relationship. Average peak ground accelerations (PGA) the OLE and CLE were estimated at 0.24g and 0.45g, respectively. Based on a process of de-aggregation, the corresponding magnitudes were estimated at 5.7 and 6.5, respectively. Design spectra were developed for the OLE and CLE conditions, based on the predicted site response spectra.

Representative earthquake acceleration time histories were selected to model the design acceleration scenarios for earthquake induced displacement analyses. The selection was based on the Magnitude of earthquake, local soil conditions (at the recording station), spectral shape (in comparison to the design spectrum) and PGA. Fig. 3 illustrates the acceleration records chosen, along with the design spectrum for the CLE.

![Fig. 3 Selected Earthquake Spectra for the CLE](image)
Liquefaction evaluations (Seed, 1987; Arango, 1996) show that almost the entire thickness of the hydraulic fill and seafloor deposits that occur below the water table and above the native dense/hard deposit could potentially liquefy under the CLE. A significant portion of this material, particularly material within Elevation 0 to -13 meters MLLW onshore, and almost the entire layer offshore, could liquefy or lose strength even under the OLE. Layers of silts and clays which are interlayered with the more granular materials within this interval, may not liquefy but will exhibit significant pore pressure gain and strength loss during earthquake shaking. Due to differences in ground surface elevations, materials behind the bulkhead wall and below higher ground are under higher confining pressures, and consequently have a higher resistance to liquefaction, in comparison to materials in front of the wall. Fig. 4 shows the results of liquefaction analysis at a typical location in the vicinity of the bulkhead wall. Consequences of liquefaction include ground subsidence, lateral spreading or deformation towards the low lying areas (lagoon) and potential damage to structures due to loss of bearing support and/or lateral and vertical movements. With no ground improvement, liquefaction induced ground subsidence in the area of the bulkhead wall was evaluated at 0.05 to 0.35 meters under the OLE, and 0.1 to 0.5 meters under the CLE, using the Tokimatsu and Seed (1987) procedures.

Seismic stability of the 3:1 slopes in front of the bulkhead wall (Figs. 1 and 2) were evaluated using a combination of slope stability and deformation analyses procedures. Slope stability analyses were conducted using post-liquefaction residual strengths of liquefied material (Table 1), and considering potential circular and block failure modes. The residual strength of liquefied sands was estimated based on Seed and Harder (1990) correlations with SPT N-values (N, 60). Typically, the 33-percentile value of the predicted range (Seed & Harder, 1990), corresponding to the average measured SPT N-Value was selected as the design residual strength. Within the silt/clay layers that are prone to strength loss, residual strength was estimated from the actual measured values of sleeve friction (remolded shear strength) from Cone Penetration Test (CPT) soundings. Within non-liquefiable layers, strength reduction due to excess pore pressure generated by earthquake shaking, was estimated using methods proposed by Seed and Harder (1990), and Seed and Booker (1977).

CONSEQUENCES OF LIQUEFACTION

Seismic stability of the 3:1 slopes in front of the bulkhead wall (Magnitude 6.7, PGA 0.34g) were selected, and scaled up by factors of 2.4 and 1.32, respectively, to provide a PGA of 0.45g. For the OLE, two representative time histories were selected. The Magnitude 5.6, 1986 Palm Springs earthquake record (Desert Hot Springs, PGA of 0.3g) representing a near-field moderate earthquake was scaled down to 0.24g. A second earthquake time history record, the Amboy 90-Degree record from the Magnitude 7.4, 1992 Landers earthquake (PGA of 0.14g) was selected to model a far-field San Andreas event of Magnitude 7.5. Although a San Andreas event would only produce a PGA on the order of 0.10g at the site, the long duration of shaking and relatively high amplitudes of long period waves could make such an event significant.
Post-liquefaction static stability analyses of the proposed bulkhead/slope configuration (4.3-meter elevation behind the bulkhead and 3:1 slopes in front of the bulkhead) showed minimum factors of safety below 1.0 for both the CLE and OLE. This indicated that without any mitigation measures, the consequences of liquefaction would include flow failures involving very large lateral displacements on the order of several meters. Empirical predictions (Bartlett and Youd, 1995) indicated liquefaction-induced lateral spreading on the order of 1.5 meters even under the OLE.

LIQUEFACTION MITIGATION AND STRUCTURE SUPPORT SYSTEMS

Bulkhead Wall

For the bulkhead wall, three support options were considered: anchored concrete sheetpile walls; relieving platform supported on vertical and raking piles; and in situ ground improvement with bulkhead wall supported on shallow footings. The first two options were eliminated on the basis of the potential for liquefaction on both sides of the bulkhead wall, large lateral loads induced by lateral deformations, the relatively large depth to non-liquefiable bearing layers located 14 to 27 meters below the ground surface, and the lack of a stable anchor zone in the vicinity of he bulkhead wall. Although a pile supported wall (Option 2) could be designed to withstand the earthquake, the promenade and backland area immediately behind the wall could still experience flow failures and settlement, even under the OLE. Option 3 using in situ ground improvement would mitigate liquefaction potential in the immediate vicinity of the bulkhead wall, by creating a non-liquefiable barrier of limited width, that would minimize potential for flow failures and limit the seismically induced deformations to acceptable levels. This option would permit use of shallow footing foundations, and was selected for design. However, several geotechnical constraints associated with this option had to be considered in design. These constraints included significant but limited earthquake induced movements, the potential for post-construction settlements in fine grained soils underlying the footings, potential undermining of footings due to tidal fluctuations, and the need for dewatering to construct shallow footings.

Pier Structure

Pile foundations were required to support the 60-meter long pier structure. Due to the potential for liquefaction, piles had to be founded in the native dense/hard deposit occurring below elevation -18 to -20 meters MLLW. Seismic analyses of the pier structure indicated that following liquefaction, piles would not have the required lateral and uplift capacities. A limited ground modification program was therefore recommended prior to pile driving, to improve the liquefaction resistance and lateral/uplift restraint capacity of the soils surrounding the piles. In addition, bracing of the superstructure above the mud line was provided to accommodate the design seismic loads.

GROUND IMPROVEMENT PROGRAM

Vibro-replacement with stone columns was selected as the optimum ground improvement solution that was capable of achieving all of the above objectives in a cost-effective manner. The program was designed to:

- densify sands to increase their liquefaction resistance under the design earthquakes
- limit pore pressures and induced cyclic stresses in silt layers and sand layers with high fines content
- reinforce silt and clay layers such that their post-earthquake shear strength is sufficiently high to limit slope deformations
- reduce settlement potential and accelerate consolidation settlements in areas underlain by significant thickness of fine grained soils
- increase liquefaction resistance and lateral/uplift restraint capacity of soils in the pier foundation area but at the same time minimize over-densification that could make pile driving difficult.

Bulkhead Area

The width of the ground improvement zone was designed at 18 meters, extending 12 meters in front of the wall and 6 meters behind the wall. The bottom of ground improvement was either Elevation -18 meters MLLW or refusal in the native dense/hard layer, whichever occurred first. Along the southwest end of the wall (Fig.1), stone columns are not expected to reach the dense native layer. However, improvement to -18 meters MLLW was considered sufficient to limit lateral seismic displacements to the same levels elsewhere along the wall.

The target ground improvement criteria specified included equivalent corrected (corrected for overburden, hammer efficiency and fines content) SPT \( N_{60} \) value of 28 in sands with fines less than 15 percent, and a minimum area replacement ratio of 9 percent within silt, clay, and sand layers with higher fines content. Stone columns with a minimum diameter of 0.9 meters were specified to be installed by dry, bottom-feed method of construction, on a triangular grid spacing of no more than 2.9 meters on center. The 2.9-meter spacing will meet the area replacement ratio criterion. A pilot test program was first performed to evaluate whether a smaller spacing would be required to achieve the densification criterion.

In order to permit the stone column installation to be carried out in the dry, a temporary earthfill construction platform raised to elevation of at least +3 meters MLLW and extending approximately 15 meters into the lagoon in front of the
bulkhead line (and 3 meters beyond the proposed improvement zone) was recommended. Following stone column installation, the temporary fill in front of the bulkhead wall would be excavated to create 3:1 slopes. Within the temporary fill area, the stone column installation was specified to be terminated at an elevation 1.5 meters above the proposed final grade. The interval above this elevation was temporarily filled with onsite soils with no vibratory effort. Fig. 5 illustrates the ground improvement and temporary fill configuration.

![Fig. 5 Schematic of Ground Improvement and Preloading Configuration](image)

Although flow failures are minimized with the proposed ground improvement, some limited but significant lateral deformations could occur under the design seismic events due to liquefaction of the areas surrounding the improved zone. Displacements are anticipated along two likely modes, one a deep seated mode with the potential sliding plane originating behind the bulkhead wall in the backland area and extending below the wall footing before daylighting near the toe of slope, and the other consisting of shallower sliding planes originating at or in front of the wall and extending downslope. Permanent displacements during earthquake shaking were estimated by double integration of the acceleration response of the potential sliding mass, each time the acceleration exceeded the yield acceleration during a given time history of earthquake loading (Newmark procedure). Typical results of the analyses presented in the form of plots of yield acceleration versus deformation are shown in Fig. 6, for the OLE. A similar plot was developed for the CLE. Maximum lateral displacements are estimated to be on the order of 2.5 to 15 cm under the OLE, and 30 to 75 cm under the CLE. Some limited damage following the OLE and extensive damage under the CLE should be anticipated, and repairs would be necessary following such seismic events.

The hydraulic fills and seafloor deposits underlying the bulkhead wall footing contain layers of relatively compressible materials consisting of soft to medium stiff clays and silts. The cumulative thickness of these layers typically range from 3.0 to 7.5 meters, except at the southwest end of the bulkhead wall where their thickness was found to be in excess of 12 meters.

![Fig. 6 Yield Acceleration - Displacement Plot for OLE](image)

Although the presence of the relatively stiff stone columns would tend to reduce the compressive stresses and hence the resulting consolidation settlements in the compressible layers, these layers would still undergo significant settlements under the proposed fill loads. Stone columns typically accommodate these settlements by bulging within the soft cohesive materials (Barksdale and Bachus, 1983). Maximum settlements on the order of 0.6 meters are anticipated under the maximum proposed fill loads, with the bulk of the settlement occurring during fill placement and within the first few weeks following fill placement. The temporary construction fill platform discussed above was also designed to serve as a preload to accelerate consolidation settlements below the bulkhead wall footings. In order to reduce the potential post-construction settlements to levels tolerable by the bulkhead structure, the minimum preloading period was estimated at 2 months under the temporary fill load. Settlement monitoring was recommended to establish actual preload periods and determine when the temporary fill could be removed. At the southwest end of the bulkhead structure (Fig.1), where the thickness of compressible materials was significant, the time for settlements was estimated to be excessive, and an additional 3-meter thick surcharge and increased preloading
period were recommended and implemented during construction.

Pier Foundation Area

Ground improvement within the pier foundation area was designed to cover the footprint of the pier structure and extend 6 meters beyond the perimeter of the pier footprint. Stone columns were designed to extend to the native dense layer at an elevation of -18 to -20 meters MLLW. The stone column configuration in this area had to accommodate the proposed pile configuration (rectangular grid pattern with a 3.0 meter x 2.3-meter spacing). The stone columns were therefore designed to be in a rectangular grid pattern with the stone columns located at the centroids of the pile grid. The target ground improvement criteria was densification of the sand layers to a corrected SPT N-value of 26. Since the grid spacing was fixed, the pilot test program involved varying the stone column diameter only. Stone column diameters of 0.8 and 0.9 meters were specified for the pilot program to establish the appropriate stone column diameter for the production program. To allow stone column installation in the dry, a temporary fill to elevation +1 meter MLLW was recommended.

PILOT TEST PROGRAM

Two locations for stone column trial test pads were selected (Fig.1). Test Pad 1 was located near the central portion of the bulkhead wall alignment, in an area of the original lagoon bottom. The area was raised to elevation +3 meters MLLW prior to stone column installation. Stone columns with a diameter of 0.9 meters, on a triangular grid spacing of 2.3, 2.6 and 2.9 meters on center were installed in a test layout shown on Fig. 7. Densification was monitored by conducting pre- and post-improvement CPTs and SPTs. The area replacement ratio (stone column diameter) was verified by monitoring the volume of stone used for each depth increment.

At Test Pad 1, all three configurations showed similar levels of improvement, with only a slight increase in densification with closer spacing. Typical pre- and post-improvement N-Values and CPT comparisons at Test Pad 1 for a spacing of 2.6 meters on center (baseline case) are illustrated in Figs. 8 and 9, respectively. At a grid spacing of 2.3 meters on center the improvement ratio (ratio of CPT tip resistance before and after improvement) in sands was approximately 10 to 15 percent greater than the baseline case. At a grid spacing of 2.9 meters on center the improvement ratio was about 10 to 15 percent lower than the baseline case. The target densification criteria in sands was achieved with both the 2.6 and 2.3-meter grid spacing, and a grid spacing of 2.6 meters on center was initially chosen for the production program. With this spacing, the equivalent N-Values in the liquefiable sands increased from a pre-improvement range of 2 to 16 to a post-improvement range of 27 to 64. Subsequently during the production program the spacing was further refined to 2.7 meters on center, based on a section of production stone columns installed and tested at that spacing.
Test Pad 2 was located in the area of the pier foundation. Stone column diameters of 0.8 and 0.9 meters were constructed at the pre set rectangular grid spacing. Comparison of typical pre- and post- improvement CPT soundings for 0.8 meter diameter are shown in Fig. 10. The target densification criteria was achieved with both diameters attempted. The smaller diameter of 0.8 meters was selected for the production program.

Fig. 9a Comparison of Pre- and Post- Improvement CPT Tip Resistance, Test Pad 1, 2.6 meter Spacing

Fig. 9b Comparison of Pre- and Post- Improvement CPT Friction Ratios, Test Pad 1, 2.6 meter Spacing

Fig. 10 Comparison of Pre- and Post- Improvement CPT Soundings, Test Pad 2, 0.8-meter Diameter

Results from the pilot test program indicated that sand layers with fines content less than 15 percent exhibited an improvement ratio in the range of 3 to 4. For thin sand layers sandwiched between cohesive layers and sands with fines content in the range of 15 to 30 percent, the improvement ratio was lower, and ranged from 1.5 to 2.5. For a given soil type, and range of initial tip resistance values measured at the site, these improvement ratios appeared to be relatively insensitive to the initial density (initial tip resistance). As expected, penetration resistance within the silt and clay layers did not show any measurable improvement by the installation of stone columns. In these soils, improvement is achieved by reinforcement and increased drainage, and verified by monitoring the area replacement ratio.

Within medium dense to dense sand layers located in the upper 1.5 to 2.5 meters from the surface, and above the water table, stone column installation resulted in some strength/density loss. This could be attributed to lack of confinement near the surface, coupled with the disturbance caused by installation operations.

Amperage readings of the vibratory probe was found to be a useful, though rough, indicator of subsurface soil type and effectiveness of vibratory densification, especially if site-
specific correlations between soil type (or CPT soundings) and amperage readings are pre-established. During initial penetration, clay and silt layers could be identified by the low amperage readings (130 to 150 amperes for the Type S bottom feed vibrator), and the sand layers by the higher amperage readings (140 to 250 amperes, depending on initial density). During vibro-densification (backfilling), the amperages in the sand layers generally increased to values greater than 200 amperes, while amperage in the clay and silt layers showed practically no change. Criteria for refusal in the dense alluvial layer were also established on the basis of measured amperage readings.

Post-improvement pore pressures measurements indicated that excess pore pressures induced in the sand layers by vibro-replacement dissipated relatively quickly, within 3 to 4 days following installation. Excess pore pressures within the finer grained materials were impacted by the preloading, and dewatering in addition to the vibro-replacement, and took a much longer period (more than 8 weeks) to dissipate.

PRODUCTION GROUND IMPROVEMENT PROGRAM

A total of 2174 stone columns (1865 in the bulkhead area and 309 in the pier area) with a combined total length of 38,130 linear meters were installed to depths ranging from 10 to 21 meters below the ground surface. During initial penetration of the vibratory probe motor amperages were monitored to verify if the native dense/hard deposit was encountered before the specified elevation was reached. Along portions of the bulkhead wall alignment and pier structure, existing rock fills that were originally built as rock dikes to retain hydraulic fills, had to be removed prior to stone column installation in order to reach the underlying loose seafloor deposits. These rock fills which extended down to elevation -6.6 meters MLLW, were excavated prior to construction of the temporary construction platform.

Local subsidence on the order of 0.3 to 0.6 meters was observed within some of the improved areas. One relatively large area (approximately 60 meters x 20 meters) of the bulkhead improvement zone subsided by as much as 1.5 meters during stone column installation. Some minor sand boils, and local lateral deformations and slumping of the temporary fill slopes were also induced by the stone column installation. Compressed air used to push gravel through the bottom feed tube was observed to bubble out of the ground and lagoon bottom up to 50 meters away from the point of installation.

Verification testing during the production program consisted of CPT soundings at an approximate frequency of one sounding per 35 stone columns. At least 4 days were allowed for dissipation of excess pore pressures within the sand layers before an improved area was tested.

ACKNOWLEDGEMENTS

The geotechnical work described in this paper was conducted by Advanced Earth Sciences for Ehrenkrantz & Eckstut Architects and Moffat & Nichol Engineers, design consultants to the City of Long Beach. The ground improvement work was completed by Hayward Baker Inc. The support of the City of Long Beach in conducting this work is gratefully appreciated.

REFERENCES


