FOUNDATION SEISMIC RETROFIT OF BOEING FIELD CONTROL TOWER

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ABSTRACT: The existing control tower at King County International Airport in Seattle, Washington, was severely damaged during the Nisqually Earthquake, February 28, 2001. As a result of the damage, the control tower was immediately closed and the existing timetable for a seismic retrofit of the control tower was expedited. The existing tower and associated support structures were founded on timber piles of unknown lengths (probably extending to depths between 25 and 35 feet); subsurface explorations indicated liquefiable soils to a depth of approximately 35 feet. The foundation retrofit included compaction grouting to densify the loose, liquefiable sands beneath the existing structures and the installation of drilled shafts adjacent to the tower to support the new structural steel bracing which was added to increase the resistance of the tower to overturning during the design earthquake. With the air traffic control operations being performed in a temporary facility, the urgency of performing the retrofit led the Federal Aviation Administration to negotiate a contract directly with the general contractor. This paper addresses the method of contract delivery, compaction grouting methods and results for liquefaction mitigation, and the construction of drilled shafts adjacent to an existing structure.

INTRODUCTION

The King County Boeing Field International Airport air traffic control tower in Seattle, Washington was constructed in the early 1960's. From its dedication in 1928, Boeing Field served as the primary airfield in the Seattle area for commercial aircraft

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until the opening of Sea-Tac International Airport in the late 1940s. Since that time, Boeing Field has remained open and currently serves as a hub for several freight carriers, home to Boeing's only paint facility for commercial aircraft constructed in the Puget Sound region, and home to many charter services and private planes.

The FFA operates the air traffic control tower around the clock at Boeing Field. FAA facilities at the site include the air traffic control tower and a single story support building attached to the base of the tower. The control tower and the support building were constructed using reinforced concrete and CMU block construction techniques. Both buildings were supported by driven timber piles although there are no records indicating the actual length of these piles.

As part of a planned FAA upgrade of the facility, a comprehensive seismic evaluation of the facility was performed. In addition to reviewing geotechnical information from the original construction, addition borings and CPT's were performed to assist in the geotechnical analysis. Structural analysis involved developing accurate as-builts and evaluating the response and loading of the structure using the appropriate ground motion spectrum developed using the design event in combination with the site specific soil profile. This evaluation indicated that the site was susceptible to liquefaction and that the control tower exhibited an unacceptable factor of safety against overturning and/or structural failure of the tower in the design seismic event.

NISQUALLY EARTHQUAKE DAMAGE TO CONTROL TOWER

During the evaluation and design phase of the project, a 6.3 magnitude earthquake occurred approximately 70 miles SW of the King County Airport on February 27, 2001. The Nisqually Earthquake produced a peak ground acceleration of 0.15g at the airport.(need reference for this information) Earthquake induced liquefaction produced sand boils and voids beneath portions of the main runway. The ground motion also caused extensive damage to the glass in the cab of the control tower which was immediately taken out of service. Emergency remedial actions included the installation of a temporary control tower and the repair of the main runway using compaction grouting to densify the runway foundation soils in those areas where excessive sand blows and/or pavement displacement indicated the occurrence of liquefaction.

Following several weeks of emergency repairs to the runway, the airport resumed full operations utilizing the temporary air control tower. Subsequently, the pace of the seismic evaluation already underway was quickened so as to expedite repairs to the control tower and return the air traffic controllers to the permanent control tower.

SEISMIC EVALUATION AND RETROFIT DESIGN

Subsurface Conditions

The King County Airport is located in the Duwamish River valley. Historically the river channel meandered across the valley prior to flood control and river bank hardening. Subsequent to those efforts, previously unusable valley property was filled and converted to industrial uses. As a consequence, the site is generally underlain by fill consisting of approximately 15 ft of loose silty-sand which is underlain by approximately 65 to 70 ft of alluvial deposits consisting of loose to dense interbedded layers of clean sand, sitly sand, and sand. Based on the blow counts and CPT tip resistances measured during the site investigation, it was determined that the cleaner sands located from a depth of 15 to 35 ft were susceptible to liquefaction during the design seismic event.

The existing control tower was founded on driven timber piles. Each corner of the tower was supported by a pile cap and a cluster of sixteen piles. Although driving resistances from the original installation were available, there were no accurate records of the actual pile lengths. Based on construction practices and similar structures in the area with better records, it was believed that the timber piles were 25-ft to 35-ft in length. Without the actual pile length records, it was impossible to determine whether the piles were achieving their end bearing in the liquefiable sands or the non-liquefiable stiff silt below.

Even if the existing timber piles were founded below the depth of liquefiable soils, lateral loads in excess of the lateral restraint provided by the timber piles would be generated during the design event. The predicted lateral loads were the result of lateral spread in the liquefiable sands and seismic loading from the structure itself.

Evaluation Results

The results of the seismic analysis can be summarized below:

1) Liquefiable soils from 15 to 35 ft below ground surface.
2) Buildings founded on timber piles which were thought to be 25 to 30 ft in length. Therefore, existing timber piles were obtaining all bearing capacity in liquefiable sands.
3) Even with mitigation of the liquefaction potential in the soils, the existing foundation does not provide sufficient lateral capacity for the tower structure.
4) The existing tower structure has an unacceptable risk of collapse during the design seismic event.

Seismic Retrofit Design

After identifying the risks posed to the facility by the design seismic event, a remediation plan was conceived which consisted of three main goals:

1) Mitigate the potential for liquefaction of the soils between 15 and 35 ft underneath and around the tower and associated support building.
2) Reinforce the structure of the tower to prevent collapse in an earthquake.
3) Increase the lateral capacity of existing foundation.

Mitigation of Liquefaction Potential

In clean sands with less than 10% fines content, liquefaction potential is typically mitigated by densifying the sand with either vibro-compaction methods or compaction grouting methods. Liquefaction potential can also be mitigated by soil stabilization using permeation grouting or soil mixing although these methods are less common for reasons of economics. At the Boeing Field site, the need to densify under the existing timber piles and around the immediate perimeter of the structures dictated that compaction grouting be utilized. The installation of the injection pipes can be performed using small, rotary drilling rigs. The injection pipes can be installed on a batter to actually treat the soils beneath the perimeter of a structure. In addition, the operations building remained occupied and in service during construction so the ground improvement methods could not disturb the building occupants.

Since the site was confined, densification was desired under the tower and the building perimeter, and the operations building attached to the tower was to be occupied during construction, the use of compaction grouting methods to densify the liquefiable sands was selected. The small size of the drilling equipment used to install the casing would limit noise impact on the occupants as would the fact that no percussion or vibratory equipment was required.

By densifying the loose sands, the potential for loss of vertical capacity in the exsiting timber piles would be mitigated. Additionally, the densification would provided greater lateral capacity for the new drilled shafts and prevent the potential of liquefaction and lateral spread loading of the drilled shafts which were founded below the liquefiable soils.

Increasing Capacity of Existing Foundation

While densifying the loose sands below and adjacent to the existing timber piles would improve the seismic response of the site and increase the lateral capacity of the timber piles, the structural response of the tower during the design event would produce loads vertical and horizontal loads which were in excess of both the capacity of the timber piles as well as the capacity of the connection between the structure and the piles. Due to the size of the loads which the tower was capable of producing in the design seismic event, the use of several drilled shafts located outside the existing tower pile cap offered the most reasonable means of providing additional required vertical and lateral capacity to the existing foundation system.

The final design incorporated a total of eight new drilled shafts. Each drilled shaft was 4 ft in diameter with a total depth of 45 ft. A group of four drilled shafts was located on the west and the east side of the tower. Each set of four drilled shafts was connected by a common pile cap which served as the foundation for the new structural steel bracing columns. The layout of the proposed drilled shafts included a drilled shaft adjacent to each corner of the control tower. The clear space between these new shafts and the closest timber pile at each corner was only 3-ft.

Reinforcing Existing Structure

The large vertical and horizontal forces which were generated by the elevated cab of the control tower had to be transferred from the control tower to the new drilled shaft foundations. The existing structure of the control tower itself was insufficient to transfer the lateral loading from the cab of the tower to the foundation. With the decision to utilize drilled shafts to provide additional lateral capacity to the foundation, it was possible to design two pairs of inclined steel columns which would connect to the existing tower just below the cab and carry the horizontal and vertical loading to the new drilled shafts. Due to the proximity of the adjacent support building and fire station, the new drilled shafts could only be installed on the east and west side of the tower. As a result, each pair of inclined steel columns splayed out to provide restraint in both the east-west and north-south directions. The earthquake loads from the control tower were transferred to the new steel columns by means of reinforced concrete collar constructed around the base of the cab.

CONTRACT DELIVERY

Prior to the Nisqually earthquake on February 28, 2001, plans for a seismic retrofit for the Boeing Field Air Traffic Control Tower were in the evaluation and design phase. In addition to evaluating the structural adequacy of the control tower and the existing subsurface soil conditions, the designers had discussions with DBM Contractors, Inc., a local drilling and specialty steel erection contractor, regarding constructability and schedule issues associated with the design options under consideration at the time. When the earthquake struck, causing sufficient structural damage to the control tower to render it uninhabitable, the timetable for completion of the seismic retrofit was accelerated significantly.

The first priority was to return air traffic control capabilities to Boeing Field. Fortunately, a scheme for the erection of a temporary control tower had previously been discussed by the designers and DBM as part of a constructability review. The FAA issued an emergency contract to DBM to erect the temporary control tower on a time and material reimbursable contract. Work began on the temporary tower installation on March 5 and was ready for use by the time the remedial compaction grouting work on the main runway was complete.

The design effort for the permanent control tower retrofit was now required to proceed on a priority basis. As design work progressed over the next several months, DBM provided technical, scheduling and budgeting support. In August 2001, the FAA issued a single-source Request for Offers to DBM. DBM tendered an offer based on the 95% design documents. FAA awarded the contract on September 4 based on the 95% design, and gave a notice to proceed date of September 10. Final design documents were completed on September 7. As the construction phase of the project began to unfold, the events of September 11, 2001 resulted in the need for the FAA to reassess the security requirements at its facilities nationwide. Construction activity was suspended during this time and a revised notice to proceed was issued on October 11. Mobilization of the site began on October 15 and revised pricing for the changes between the 95% and final design documents was completed on October 17.

CONSTRUCTION

After negotiation of the contract, the project was started following a short delay as additional security measures were implemented following the events of September 11, 2001. While the preliminary work of locating utilities and submittal preparation was being performed, a discussion on the sequence of the foundation construction arose after a review of the proposed construction sequence and methods.

The contractor had submitted a work sequence calling for the installation of the drilled shafts before the performing the compaction grouting. The drilled shafts were to be constructed using a surface casing and polymer drilling fluids. The drilled shafts were to be installed before the compaction grouting to achieve the greatest permanent densification from the grouting. If the drilled shafts were installed after the compaction grouting, some loosening of the previously densified soils was expected. However, the owner's representatives became concerned about the potential loss of vertical capacity in the existing piles during installation of the four drilled shafts located directly adjacent to the existing pile groups.

During subsequent discussions, several methods of mitigating the perceived risk associated with drilling next to the existing piles were assessed along with their associated costs. The use of cased hole drilling methods for the shafts was rejected due to cost and increased risk to the quality of the completed shaft. Pretreating the soils from 25 to 35 ft at each of the shaft locations was not selected due to cost and increased schedule duration. Ultimately, it was decided to perform the compaction grouting prior to the drilled shaft installation. By performing the compaction grout first, the loose soils would be densified prior to drilling the shafts which would reduce the potential for over break during shaft construction. The compaction grouting beneath the pile tips would increase the vertical capacity of the existing piles so those piles which are outside of the influence of the shaft construction can carry additional load safely should excessive over break occur. In addition, the compaction grout layout was altered and several locations added to create a partial ring of grout columns between the drilled shafts and the existing piles.

Compaction Grouting

Equipment & Methods

Prior to beginning the work a series of monitoring points were established on the perimeter of the control tower and the support building and baseline elevations recorded. During the performance of the compaction grouting, the elevations of the monitoring points within the treatment zone were measured and recorded using conventional survey equipment at least once per shift. In addition to this daily survey of the monitoring points, a rotating laser system was used at all times during grout injection to provide real time monitoring of the ground surface and adjacent structures within 50 ft of the injection point.

Since the choice of compaction grouting for densification of the subsurface soils was a result of the ability to treat under the perimeter of the structures with battered holes, work on a tight site, and minimize disturbance of the occupied building, the grouting subcontractor utilized a small rotary drilling system mounted on and powered by a skid steer unit. This drilling unit installed the flush joint casing using rotary drilling methods with water flush. Once the casing was installed, a low-slump grout consisting of dirty sand having about 25% fines mixed with 3% to 6% cement was injected in an up-stage manner. The grout was mixed on-site using a continuous mixing system which consists of two conveyors which feed cement and sand from separate hoppers into an inclined mixing auger which discharges directly in the holding hopper on the pump.

Since the compaction grouting was governed by a performance specification which only established the treatment depths and the level of improvement required, the actual pattern and spacing of compaction grout holes was selected by the grouting subcontractor. The 8 ft triangular pattern, with extensive field adjustments and battered installations for existing structures, used for the compaction grouting is shown in Figure 1. The casing was retracted and the grout injected in 1 ft stages. The original specifications also indicated that grout was to be injected in each stage until one of the following criteria was achieved:

1) Header pressure exceeds 700 psi
2) Volume of grout injected at a given stage exceeds 25% of the ground being treated by that stage.
3) Total building movement exceeds 0.25 in.

Construction Issues Related to Compaction Grouting

The compaction grouting was started around the control tower since this offered the only area which could be tested due to the proximity of the remainder of the holes to the perimeter of the buildings. During the initial phase of grouting, one corner of the tower experienced a cumulative upward movement in excess of the 0.25 in total allowed by the contract. After the project team met to discuss this issue, it was decided that the tower could tolerate several inches of vertical movement as long as the relative movement between the corners did not exceed the threshold value of 0.25 in. In order to accommodate this requirement and prevent differential movement in excess of 0.25 in, a pattern of hole injection which alternated between the sides was implemented. Although this worked well on the tower which had a small enough footprint to permit treatment of all of the soil under the tower, the support building experienced some internal cosmetic damage as the compaction grouting around the perimeter moved the outside building wall up to 0.75 in while the interior walls and columns experienced no heave since they were outside the zone of influence of the compaction grout points.

A review of the grouting records also indicates that most building movement occurred when grout was being injected between depths of 25 to 30 ft. This depth corresponds to the likely tip elevation of the timber pile on which the tower and the support building were constructed. It seems that the grouting actually lifted structure and the piles as a unit from the pile tips. While this may raise some questions about the stresses induced in the piling by the grouting, it also allays the concerns about the grouting process actually separating the pile cap from the timber piles.

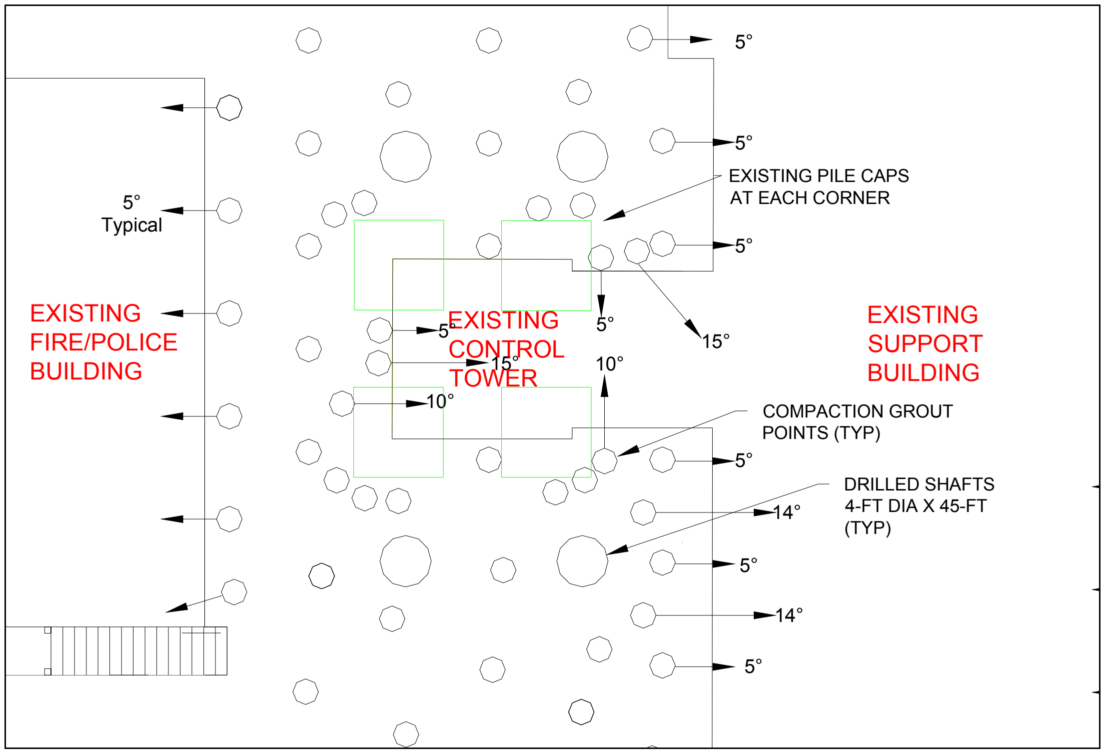

FIG. 1. COMPACTION GROUT POINT LAYOUT

Results of Compaction Grouting

Since the purpose of the compaction grouting was to densify the loose, saturated sands to prevent liquefaction, post-treatment CPT's located between the treatment points were used to verify that a minimum level of densification had been achieved. The specifications required that the post-treatment soils have an average, corrected clean sand equivalent Normalized Cone Penetrometer Resistance $(q_{c1n})_{cs}$ exceeding 125 tsf. Each measured CPT tip resistance was averaged with the resistances measured in the 2 ft above and the 2 ft. There were no performance requirements stipulated for soils with permeabilities less than 1 x 10^{-2} cm/sec. This permeability cutoff served as an indicator of a fine grained soil which is commonly thought not to be a risk of liquefaction and which can only be minimally densified by compaction grouting efforts.

A graph of the raw values for the pre-treatment CPT-01 is shown in Figure 2. The graph of the friction ratio indicates that the material from 15 to 25 ft has a friction ratio less than 0.75% which indicates a clean sand which is very amenable to improvement by compaction grouting. The material from 25 to 30 feet has several spikes greater than one in the friction ratio which indicate a layering of the clean sand with silty material, and the material from 30 to 35 feet has a friction ratio of just under 1% which indicates a silty sand. The silty material has a lower permeability

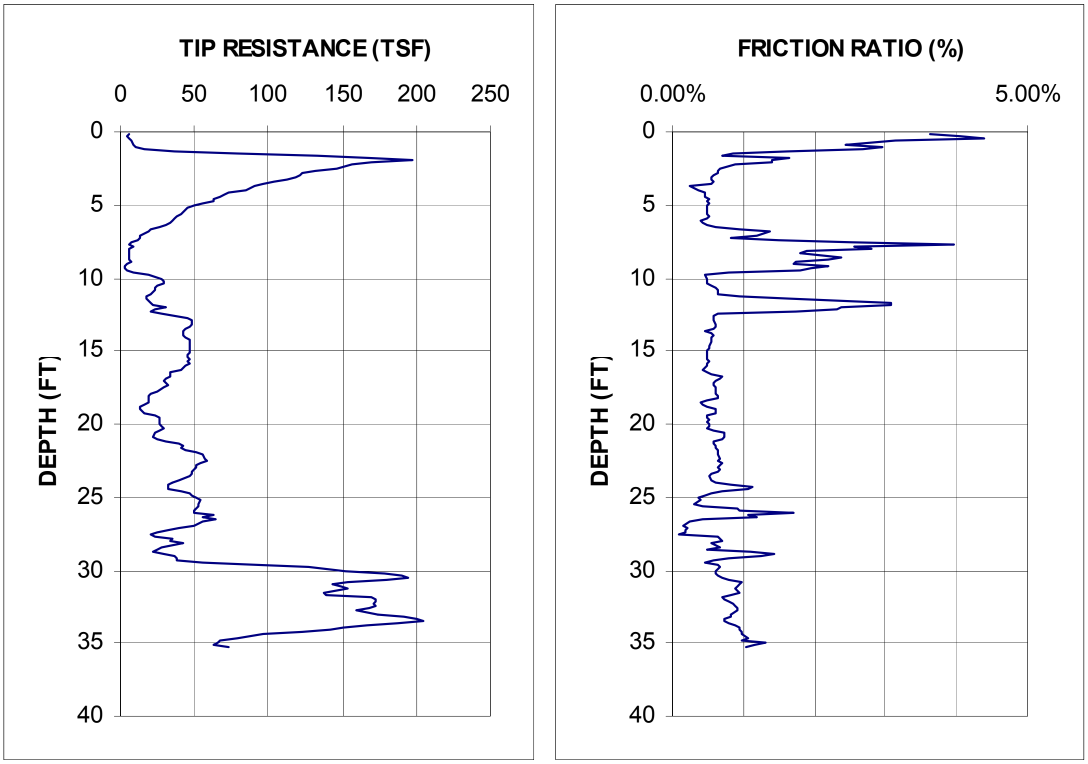

FIG. 2. CPT-01 PRE-TREATMENT RESULTS

which inhibits the effects of densification by reducing the rate at which excess pore pressures dissipate during the injection of the grout bulb.

The corrected tip resistance for the pre-treatment CPT-01 and the post-treatment CPT-04 performed on the east side of the tower are shown in Figure 3 along with the target value of the post-treatment CPT. The post treatment CPTs used to measure the performance of the compaction grouting indicated generally acceptable levels of improvement as compared to the required performance level. There are zones in the treatment zone between 25 and 35 ft where the required densification was not achieved. In fact, the pre and post treatment CPTs show little difference at 28 ft. This zone of no improvement corresponds well with the fine grained soil layers indicated by the higher friction ratios recorded in the pre-treatment CPT. Although the CPT's showed improvement which generally met the performance requirement, the presence of some zones of material which did not achieve the required performance led to some remedial points being added in the critical area of the control tower.

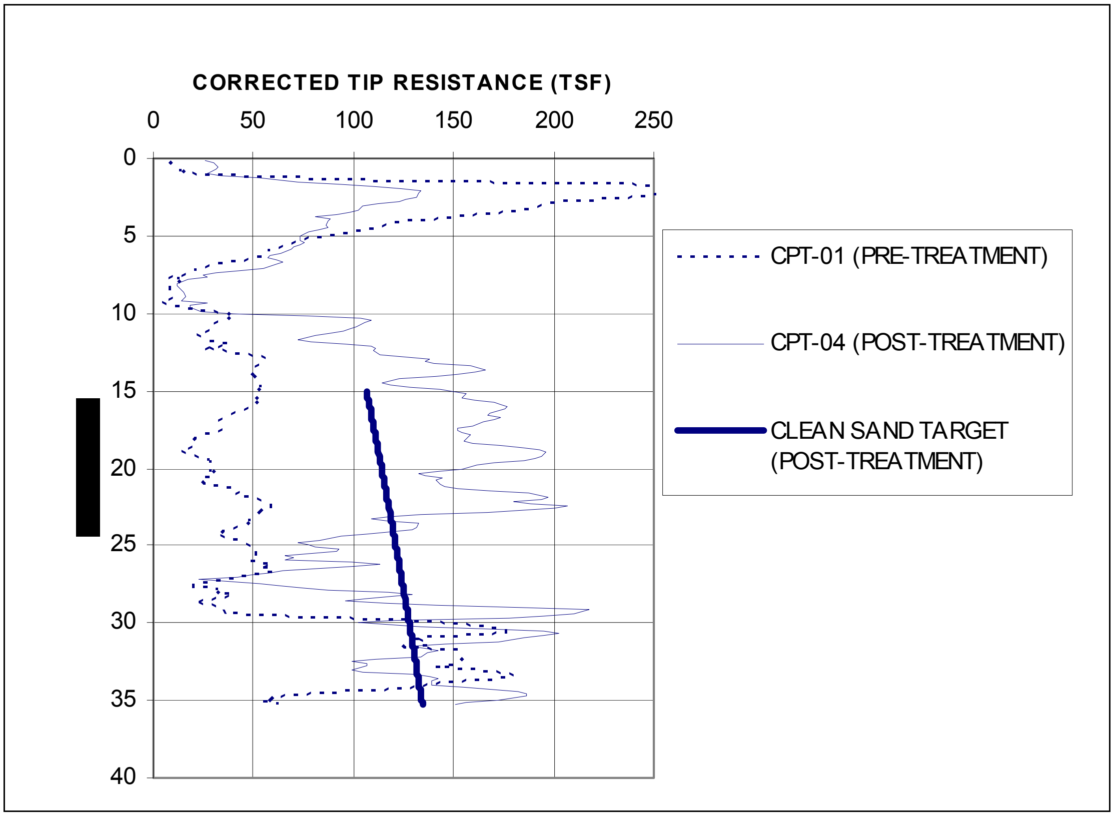

FIG. 3 . COMPARISON OF PRE-TREATMENT AND POST-TREATMENT CPTS

Drilled Shafts

Equipment & Methods

The primary considerations in the selection of the drilling methods and equipment for drilled shaft construction were the anticipated presence of groundwater between 12 and 15 feet below existing grade, and the limited headroom between existing grade and the soffit of the control tower for the four shafts immediately adjacent to the existing structure.

The presence of groundwater dictated the use of either temporary casing or drilling slurry to provide shaft sidewall stability during drilling and concrete placement operations. The use of casing, however, proved impractical for a host of reasons. First, the headroom distance from existing grade to the underside of the cantilevered portion of the tower structure was approximately 46 ft. The planned depth of the shafts was 45 feet. The minimal headroom therefore precluded the use of full-depth temporary casing for the four shafts nearest to the existing structure. The use of telescoping casing was also investigated, but was discounted due to the proximity of the existing timber piles as well as the complications associated with casing removal during concrete placement. Ultimately, it was decided to use polymer slurry for shaft sidewall stability, combined with additional compaction grouting points adjacent to the existing structure to address concerns regarding the induced lateral soil pressure near the bottom of the existing timber piles.

The shafts were drilled using an IMT AF-10 hydraulic drill rig. Approximately two feet of soil was excavated at the shaft locations adjacent to the existing structure to allow enough head room for the boom of the rig to clear the tower overhang. A temporary casing six feet in diameter was installed down to a depth of approximately 12 feet to support the soil around the top of the shaft. The top of the shaft concrete was held down to approximately eight feet below grade to allow for subsequent splicing of the shaft rebar with that required for the pile caps

Upon completion of drilling, rebar cages were placed in the slurry-supported holes. The specifications required Crosshole Sonic Logging (CSL) tests to be performed after concrete placement, so 1 ½" schedule 40 steel pipes were tied inside of the vertical reinforcing bars for subsequent testing. Concrete was placed using a tremie pipe and concrete pump truck. The polymer slurry was pumped out of the shaft and into a holding tank as concrete was placed. CSL test data was collected and analyzed by an independent testing firm. Each shaft was accepted based upon the recommendation of the independent firm without the need for any remedial work.

CONCLUSION

A foundation retrofit system, which combined ground improvement by means of compaction grouting with eight 4 ft diameter drilled shafts, provided a cost effective means of mitigating liquefaction potential and providing increased lateral load capacity to the existing foundation. The foundation contractor was able to negotiate a fixed price contract with a government agency based on past performance with the agency and a proven team of subcontractors. The project was performed on time and on budget, and it resulted in an air traffic control tower that is more likely to survive and maintain operations following the next earthquake in the Pacific Northwest.