A CASE HISTORY ON DESIGN, CONSTRUCTION, AND PERFORMANCE OF STONE COLUMN GROUND IMPROVEMENT BENEATH AN MSE EMBANKMENT

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This paper presents a case study of the deformation-based design, construction, and performance of stone column ground improvement (GI) beneath a mechanically stabilized earth (MSE) wall and bridge abutment with heights up to 50 feet. As part of the widening of Interstate 5 (I-5) to allow High Occupancy Vehicle (HOV) lanes in Tacoma, Washington, a new approach and span over the Puyallup River will be constructed. During the soils investigation and design phase of the project, low plasticity silts (ML) inter- bedded with silty sand layers and organic silt where identified as being potentially liquefiable. Stone column ground improvement was designed using a deformation-based approach to address static settlement and seismic stability of the proposed MSE embankments. The deformation-based design of the stone columns resulted in significant savings over a limit equilibrium-based approach. This paper presents the results of sonic coring taken from the initial stone columns installed and compares these results to the real-time data acquisition reports from the stone column installations. Vibration monitoring results during stone column installation are included. Settlement monitoring at the face of the MSE wall and buried vibrating wire settlement monitoring elements are presented and compared to the original settlement predictions.

Introduction

Stone column ground improvement was implemented as a means of supporting mechanically stabilized wall approaches for a new bridge over the Puyallup River, part of HOV improvements to I-5 in Tacoma, Washington. The work discussed in this case history was part of the Washington State Department of Transportation (WSDOT) I-5/Portland Avenue to Port of Tacoma Road, Stage 1 Northbound HOV contract, developed as an early work element to a follow-on bridge construction project. The area requiring stone column ground improvement is shown in Figure 1. The improved ground supports permanent geosynthetic walls and embankments up to 50 feet high. This wall can be seen in Figure 1. The selected ground improvement primarily provides global stability of the soils beneath the walls during anticipated seismic loading and also reduces the time and magnitude of consolidation settlement under the weight of the planned embankment.

Approximately 74,000 cubic yards of liquefiable alluvial soils were improved by stone columns. Stone column construction occurred in late 2010 and early 2011. This was followed by the embankment and wall construction. Settlement monitoring began with the embankment construction and continued into 2012.

This paper presents a discussion of the deformation-based analysis that allowed significant cost savings over a traditional limit equilibrium-based stability analysis. This paper also presents a summary of the methodology adopted by the Contractor in deciding on layout design, construction methodology, and quality control. Construction observations are also summarized, including:

- Comparison of stone column data acquisition results to preconstruction Cone Penetrometer Test (CPT) results which illustrate the benefits of computer instrumented construction equipment.
- The results of sonic coring of the stone columns installed within the test program.
- Vibrations recorded at varying distances from the vibratory probe.
- Settlement monitoring using vibrating wire settlement indicating devices and surface plates. A Comparison of the observed and predicted settlements.

Subsurface Conditions

The alluvial deposits to be improved were typically soft to firm silt and sandy silt with interbeds of medium dense to dense sand. The plasticity index of the silt typically ranged from 0 to 15 indicating non to low plasticity. Occasional organic materials (PT) (up to about 7% loss upon ignition) and elastic silts (MH) with plasticity above 20 were also present.

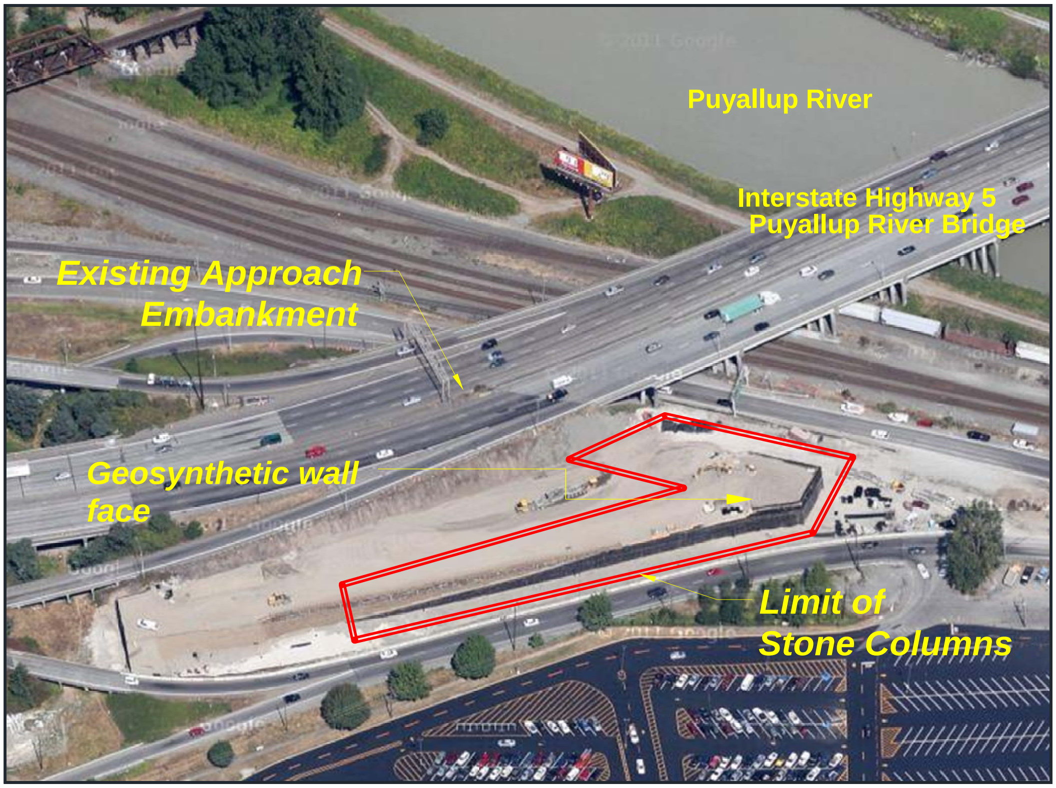


Figure 1. Vicinity Map

The thickness of the alluvial deposits ranged from about 30 to 70 feet. The alluvial deposits were underlain by very dense, glacially consolidated granular soils. A transitional zone was located between the alluvial deposits and the glacial deposits. The transitional zone generally consists of inter-layers of medium dense to dense sand/gravel and soft to firm silt. The groundwater table was typically 5 to 10 feet below the existing ground surface. Piezometers installed in the glacial deposits indicated that a confined aquifer exists in the glacial deposits. The measured pressure head was approximately at the ground surface. A typical CPT log is shown in Figure 2.

Stone Column Ground Improvement Design

Because the alluvial deposits are potentially liquefiable, a site specific seismic ground response analysis was performed. Based on the analysis results, the design Peak Ground Horizontal Acceleration (PGA) was estimated to be 0.27 g.

Geotechnical design challenges at the site were:

- Estimated consolidation settlement of up to 3 feet under the planned embankment height.
- Estimated liquefaction settlement of up to 1 foot during or after a design earthquake event.
- Excessive seismically induced lateral deformation.

The primary need for stone-column ground improvement was to provide global stability for the walls supporting embankments. Global stability analyses considered several cases:

1. Static long-term loading
2. Static construction loading, which included the surcharge loads
3. Start of shaking, with the PGA reduced by one-half as allowed by AASHTO, but full strength soil properties
4. During shaking, in which the seismic load is combined with reduced soil strength.

5. Post shaking, with no seismic acceleration and fully liquefied soil properties

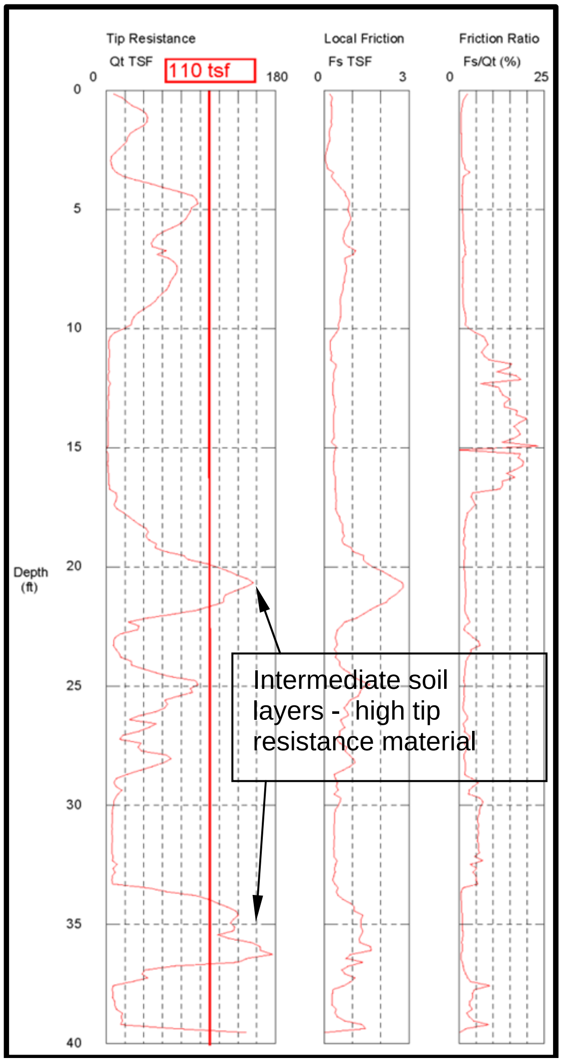


Figure 2. Typical CPT Log

In the past, designers often ignored the during shaking (fifth) condition, but the requirements of AASHTO at the time of the design analysis and the results of a site-specific seismic response analysis indicated that liquefaction-related strength reduction would occur during the time of strong shaking. The during shaking condition also controlled the design of the ground improvement, but would have resulted in unrealistically large replacement ratios and lateral extents of ground improvement if the PGA was used in conjunction with the residual strength values.

Recognizing that the geosynthetic walls and abutments could tolerate several inches of lateral deformation, the post-earthquake allowable permanent lateral deformations of 6 inches for the bridge abutment and 12 inches elsewhere were developed by the geotechnical and structural designers in collaboration. The yield accelerations associated with 6 and 12 inches of deformation were determined from the design chart developed by Bray and Travasarou (2007). A series of slope stability analyses were then performed to determine the degree and intensity of ground improvement to achieve factor of safety 1.0 with the yield accelerations.

Based on the analysis results, the stone-column improvement with an area replacement ratio of 15 percent was selected. Based on other studies conducted at the vicinity of the project site, the final design did not account for any densification of the fine grained alluvial deposits between the stone columns. Composite strength of the improved ground was based on stone column material with a minimum internal friction angle of 40 degrees. Liquefaction potential of the alluvial deposit in the stone column improved ground was evaluated based on the cyclic stress ratio (CSR) reduction factors by Baez and Martin (1993), and Priebe.

Additional benefits from stone column construction were:

- A reduction in total settlement.
- Faster consolidation under the surcharge loading required to meet additional design criteria.

The magnitude of primary consolidation settlement of the improved ground was estimated using the improvement factor suggested by Priebe (1995). A ten-month duration of preloading with a surcharge load equivalent to 20 percent of the final embankment height was designed to limit the long-term secondary compression to 2 inches in 15 years, a WSDOT design criteria for the pavement.

Contract Requirements for Stone Column Construction

The construction contract documents required Contractor selection of stone column diameter and spacing to achieve a 15 percent replacement ratio. The specification also limited the minimum column diameter to 30 inches. The friction angle of the aggregate was to be quantified based on an engineering judgment of the aggregate material. The stone volume over any 10-foot deep column length had to equal or exceed 80 percent of the design volume and the volume over the entire length had to equal or exceed the design volume per column. Provisions were made to allow lower than design volumes of stone if equipment response indicated penetration through material that exceeded a target density equivalent to a CPT tip resistance, q_t, of 110 tons per square foot (tsf) or a standard penetration test (SPT) N_{60} value of 24 blows per foot. Comparison of target CPT tip resistance with the limiting value of 110 tons per square foot (tsf) can be seen in Figure 2.

Test sections were required at two locations prior to the start of production stone column installation to verify that the Contractor's means and methods could produce columns of the diameter that were selected, and that the completed columns contained a sufficient volume of gravel to meet the design replacement assumptions. Each test section contained five stone columns centered on two CPT test locations. The test sections were also to be used to determine correlations between equipment response factors (e.g., amperage) and the presence of the final bearing layer or intermediate layers with pre-existing density sufficient for project requirements (i.e. indicated by CPT cone tip resistance greater than 110 tsf).

Verification of the depth to the bearing layer by CPT or SPT testing was required at a frequency of 1 test per 7,000 square feet of gross improved area if the equipment response in the test sections showed a clear indication of the bearing layer or 1 test per 2,500 square feet if the equipment response was unclear.

Bearing Layer Profile

Prior to commencing installation of the stone column test program, the specifications required refinement of the bearing layer profile across the site. Initially, 14 CPTs were carried out throughout the site. This included 2 CPTs within each of the two test sections plus an additional 10 CPTs across the rest of the site. As the CPTs could not penetrate the required 10-ft to verify bearing layer thickness, further investigations in the form of 6 boreholes with standard penetration tests were also carried out. The resulting aerial density of explorations, counting the original design explorations, was 1 per 1600 square feet.

The refined profile of the bearing layer provided additional assurance that the equipment response was indicating that the bearing layer had been reached.

Pre-bid profiling indicated that the total quantity of stone column improved ground would be approximately 90,000 cubic yards. Contract documents allowed the use of either CPTs or SPTs to verify the bearing layer profile. While the Contractor initially chose the use of CPTs due to cost and schedule implications, it was found that CPTs were not able to penetrate the bearing layers to a depth of 10 feet (at tip resistance greater than 110 tsf) due to the nature of the material. Since the CPTs could not penetrate 10 feet through the layers with tip resistance greater than 110 tsf, SPTs with boreholes were also used to verify the bearing layer profile consistently. The additional SPT investigation was carried out to ensure that the bearing layer identified was the true bearing layer as per contract specification and not an intermediate dense layer. Such bearing layer profiling conducted by the Contractor determined that actual ground volumes requiring stone column treatment were significantly less (approximately 15% less). Additionally, such accurate profiling enabled a more precise evaluation of project quantities for costing and material/equipment requirements for the Contractor.

Construction Methods

Layout Design

The design required a minimum 15 percent replacement ratio of stone column area to gross treatment area. To obtain this ratio, the stone columns were installed on an 8-ft equilateral triangular pattern with the target minimum column diameter of 3.25-ft. (39-inch). On the steep embankment area adjacent to I-5 NB, 3.5-ft (42-inch) diameter stone columns were installed. General considerations in selecting the layout spacing and size of column include anticipated daily production rates, gradation and density (loose and as placed) of aggregate material chosen to be installed, anticipated treatment depth, equipment capability and the physical constraints/limitations of the site. Figure 3 shows the stone column layout selected.

Installation Equipment

Due to the quantity of stone column ground improvement that needed to be carried out, coupled with an aggressive construction schedule, two installation units were mobilized. The depths of the stone columns were anticipated to range from 30 feet to as much as 70 feet in the existing embankment area. This required that the length of the probe from probe tip to top of skip bucket needed to be adjusted accordingly and suitable base cranes were selected. Manitowoc 4000 and 4100 crawler cranes were chosen as the base carriers. Bottom feed electric probes (V-23) were utilized to place the stone directly at the bottom of the individual columns. Figure 4 shows the equipment arrangement used for carrying out the work.

The dry bottom feed method was selected by the contractor to avoid environmental issues associated with wet top feed and to provide a more reliable means of constructing a continuous stone column than top feed methods. The dry bottom feed method also provides positive support of the surrounding ground since the probe is in direct contact with the ground. The use of an electric powered probe enables the operator to monitor the electric current (amperage). High amperage signifies high resistance to penetration as well as high density of material surrounding the probe. During penetration, this could be an indication that the bearing layer has been reached provided it corresponds to the identified bearing layer profile. During backfill/stone placement, high amperage typically signifies that the stone and surrounding soil have been densified. Although, in the ground conditions at this site, densification of the surrounding soils was not the intent of the design and was not possible due to the fine grained nature of the soils.

For this work, the stone columns were to provide replacement and thus strengthen the sub-surface without necessarily providing densification. The probe penetrated to the bearing layer using air flush in combination with pre-drilling. Although water jetting during penetration was an option, the environmental ramifications of water jetting along with the volume of spoils produced and the deleterious impact to the working pad provided sufficient reasons to use dry bottom feed methods.

For penetration of the probe through the soil layers having high cone penetration test tip resistance, a back-up plan of pre-drilling was considered. Pre-drilling through high-resistance layers is a viable option especially in cases where densification is not required and replacement is the stated goal of stone column installation.

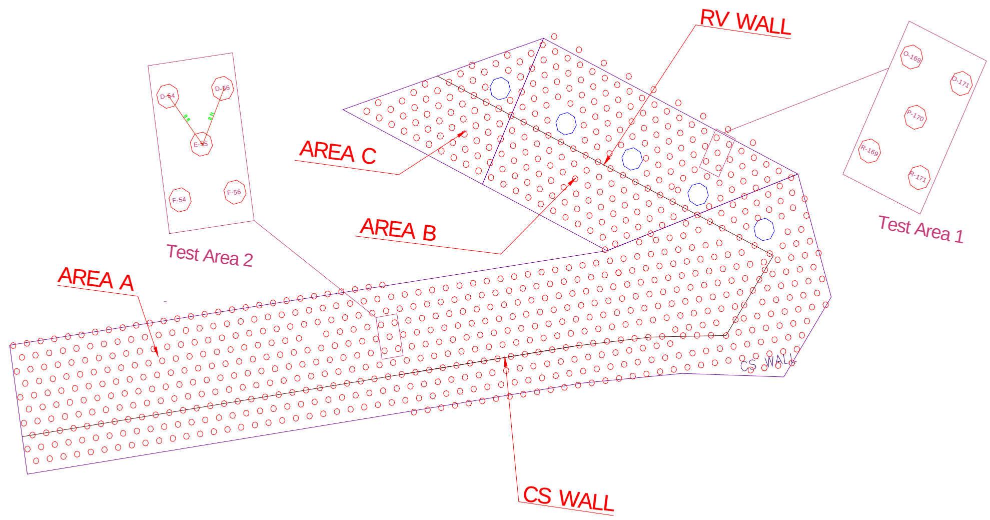


Figure 3. Layout of stone columns

Test Section Installation

Initially, the two groups of five stone columns each were installed as test sections. These two test section locations are identified in Figure 3. Profiling CPTs carried out prior to the test section installation indicated the presence of an intermediate layer of dense material (identified as a non-bearing layer) that would prove time consuming to penetrate with the vibro-probe. This dense layer was predominantly found to be in the area surrounding Test Section 1 (Areas designated B and C - observe Figure 3) and as seen as the high-resistance material per the typical CPT plot shown in Figure 2 where it was confirmed to be difficult to penetrate with the probe. During the test section, limited water-jetting was attempted but abandoned due to ineffectiveness. Therefore, it was decided to penetrate through this layer with the aid of pre-drilling.

Figure 4. Equipment set-up showing crane mounted vibro-probe

Data Acquisition

Given the project specific requirements with regard to the stone column volumes, it was necessary that the installation personnel (operator and supervisor) be continuously aware of the size of the column forming in-situ. This enabled immediate correction of improper construction instead of having to re-penetrate and place stone at a later time. To achieve this, the Contractor used continuous real-time monitoring devices to measure the weight of stone placed per lift plus the total weight of stone, together with depth of probe, amperage in the vibro-probe motor and air-pressure. Figure 7 shows a view of the monitoring display.

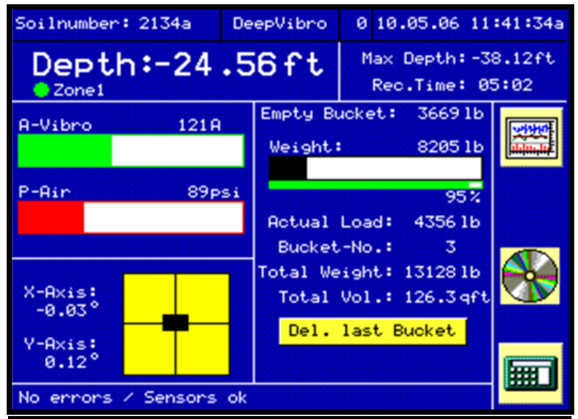


Figure 5. Real-time data acquisition available to Operator

Construction Observations

Test Sections

The test sections showed good correlations between probe amperage and the tip resistance from adjacent CPT tests. Figure 6 shows the in-cab monitoring data and a comparison with CPT tip resistance.

Figure 5 illustrates a typical cab log from a test section side-by-side with the adjacent CPT log. It can be observed from the probe amperage (in green) reaching above 200 amperes at close to termination depth of approximately 34 feet (elevation -15 feet) that the bearing layer was reached. Additionally, during the backfill operation, high amperage was reached at the latter stages. This corresponds to high tip resistance from the CPT data. This can be understood by considering the fact that as CPT tip resistance shows high values due to dense in-situ material, placement of stone within this material will also require high amperage in the probe.

During production, the probe amperage in combination with broadly spaced pre-construction and additional construction CPT and borings with SPT tests provided reliable indications of the bottom of the low strength layer slated for improvement.

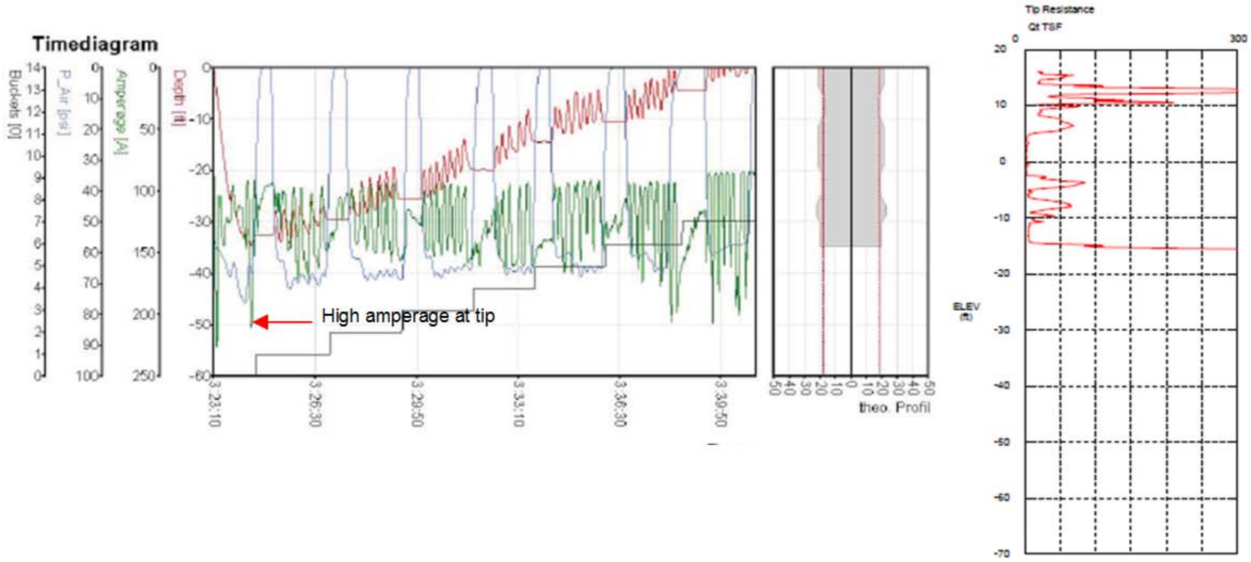


Figure 6. Typical log of Stone Column Installation from real-time data acquisition with proximate CPT data

After installation of the stone columns, the test areas were excavated of all overlying material to enable visual examine of the tops of the stone columns and also to properly identify column locations. Figure 7 shows one exposed column. Physical measurements of these columns showed that the observed diameters generally correlated well with the theoretical profile shown in the output log in Figure 6.

Rotary sonic coring of the columns was also performed. The sonic cores were somewhat difficult to interpret. Much of the cores contained obviously clean, high strength gravel with very little intrusion of native soil, but some of the cores included significant native material (Figure 8).

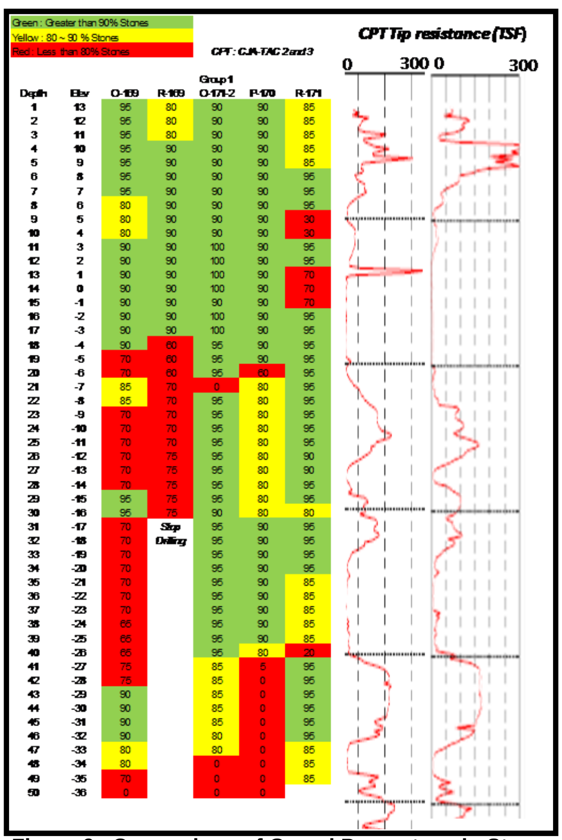
Comparing plots of percentage gravel (through visual examination) with depth to the adjacent CPT logs, for Test Section 1 (Figure 9), did not yield any obvious subsurface-related reason for the inconsistency and unexpected composition of the core. Examination of construction records suggested that the appearance of native intrusion was due to misalignment of the sonic core within the columns. Additionally, due to plugging of the probe at various times during the test column installation, the probe had to be withdrawn from the hole and unplugged before reinsertion. The probe would occasionally become plugged with quarry spalls which were placed to provide a firm working platform and had contaminated the stone column backfill material. Withdrawal and reinsertion was also deemed as a reason for intrusion of native material.

Figure 7. Exposing the top of test section columns

During production, shorter pulls and reinsertion of the probe into the already built columns were undertaken by the Contractor to build full diameter columns. Reinsertion of probe well into the already built column was also undertaken whenever the probe became plugged to avoid excessive intrusion of native material. After initial startup, operators became better at avoiding these plugs.

Mobilization of a sonic coring subcontractor as well as excavating 2 to 3 feet below the working surface to expose the column top and to ensure that the cores were drilled within the columns provided additional complications in a relatively busy and congested work area.

Figure 8. Comparison of Sonic Cores



Depth	Elev	O-169	R-169	Group 1 O-171-2	P-170	R-171
1	13	95	80	90	90	85
2	12	95	80	90	90	85
3	11	95	80	90	90	85
4	10	95	90	90	90	85
5	9	95	90	90	90	85
6	8	95	90	90	90	95
7	7	95	90	90	90	95
8	6	80	90	90	90	95
9	5	80	90	90	90	30
10	4	80	90	90	90	30
11	3	90	90	100	90	95
12	2	90	90	100	90	95
13	1	90	90	100	90	70
14	0	90	90	100	90	70
15	-1	90	90	90	90	70
16	-2	90	90	100	90	95
17	-3	90	90	100	90	95
18	-4	90	60	95	90	95
19	-5	70	60	95	90	95
20	-6	70	60	95	60	95
21	-7	85	70	0	80	95
22	-8	85	70	95	80	95
23	-9	70	70	95	80	95
24	-10	70	70	95	80	95
25	-11	70	70	95	80	95
26	-12	70	75	95	80	90
27	-13	70	75	95	80	90
28	-14	70	75	95	80	95
29	-15	95	75	95	80	95
30	-16	95	75	90	80	80
31	-17	70	Stop	95	90	95
32	-18	70	Drilling	95	90	95
33	-19	70		95	90	95
34	-20	70		95	90	95
35	-21	70		95	90	85
36	-22	70		95	90	85
37	-23	70		95	90	85
38	-24	65		95	90	85
39	-25	65		95	90	85
40	-26	65		95	80	20
41	-27	75		85	5	95
42	-28	75		85	0	95
43	-29	90		85	0	95
44	-30	90		85	0	95
45	-31	90		85	0	95
46	-32	90		80	0	95
47	-33	80		80	0	85
48	-34	80		0	0	85
49	-35	70		0	0	85
50	-36	0		0	0	

Figure 9 Comparison of Gravel Percentage in Stone Colums with CPT Tip Resistance

Vibration Monitoring

The stone columns were constructed close to an existing high pressure gas line. Vibrations were monitored continuously when equipment was operating within 100 feet of the line. A 3-channel seismograph recorded vibrations in the longitudinal, vertical, and transverse directions. Typically, vibrations were less than 0.5 inches per second. The highest recorded vibrations were 1.1 inches per second in the transverse direction when the probe was working 6 to 8 feet from the monitoring point which was directly above the gas line. Figure 10 shows the velocity record from the closest work to the gas line.

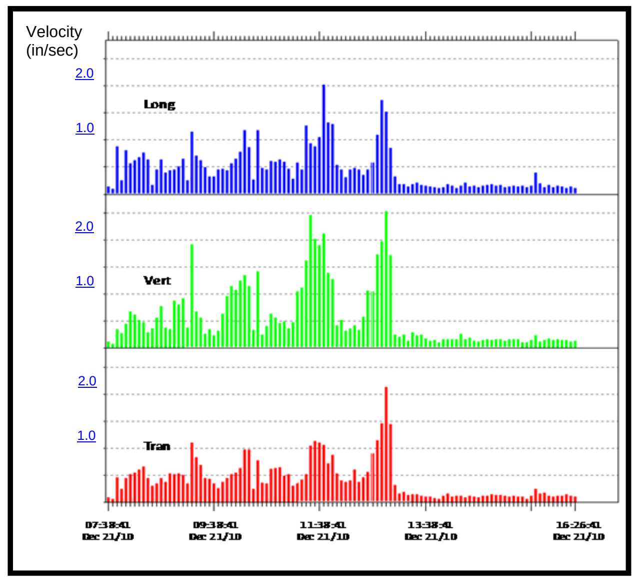


Figure 10 Vibrations Monitored at Gas Line Located 6 to 8 feet from probe

Settlement Monitoring

Following construction of the geosynthetically-reinforced backfill, surcharge loads were placed in front of and on top of the walls to increase the rate of consolidation settlement and reduce waiting time prior to construction of the cast-in-place concrete wall face; and to also mitigate settlement due to most of the anticipated secondary compression from what would eventually become the high speed lanes of I-5. Vibrating wire settlement monitoring devices were placed at the base of the embankment both near the wall face and near the end of the geosynthetic reinforcement. The vibrating wire settlement monitoring devices were placed on the native soil mid-way between stone columns.

One foot square steel settlement plates with extendable risers were also installed just in front of the wall face. Example plots of settlement vs. time are shown in Figure 11.

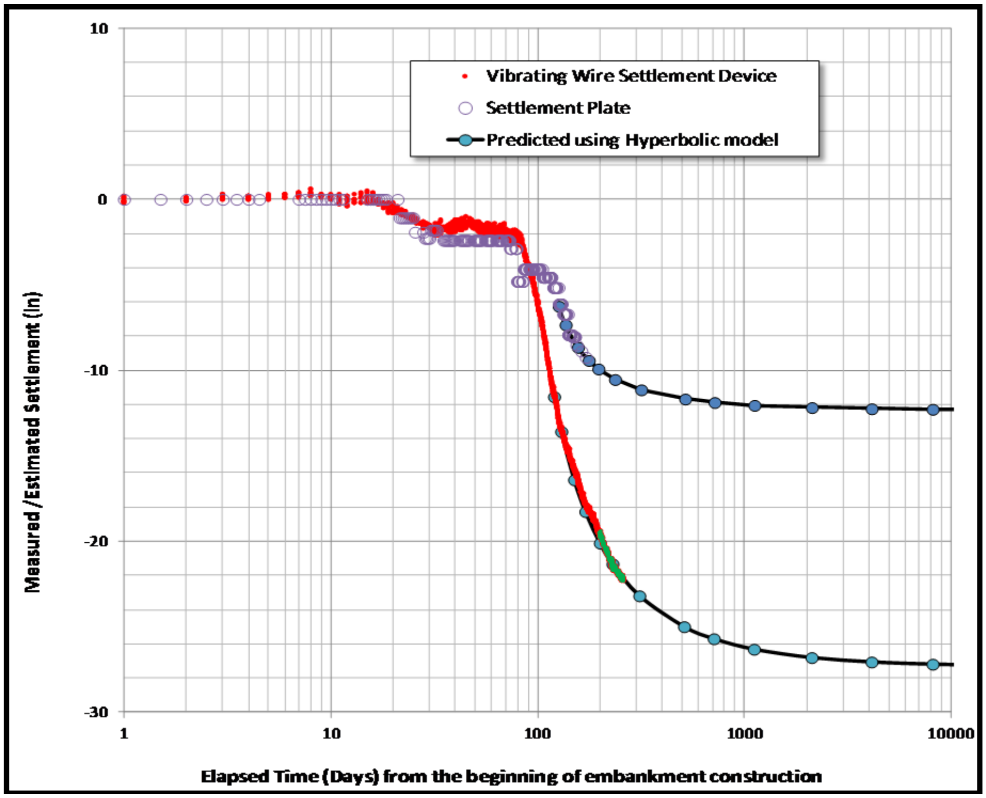


Figure 11 Typical Measured Settlement Data

The estimated magnitudes of the primary consolidation settlement were approximately 20 and 11 inches for unimproved and improved ground respectively, without considering the 20 percent surcharge. As shown in Figure 11, the measured settlement using settlement plates was quite comparable with the estimate. However, the vibrating wire settlement devices typically indicated greater settlement than the settlement plates. The vibrating wire settlement devices were placed midway between stone columns, at least 8 inches below the top of the columns, on 12-inch x 12-inch x 1/8-inch plates, and directly under the face of the wall. The settlement plates were constructed on top of the sand drainage blanket above the stone columns, with 24-inch x 24-inch x ¼ inch base plates, and located at least 2.5 feet in front of the wrapped face. It is postulated that these slight differences in details combined to make the measurements from the vibrating wires represent more of the unimproved ground settlement while the measurements from the settlement plates represent overall average settlement of the improved ground.

Discussion

Development of a replacement-based performance specification for stone column construction in silty materials is a complex task especially where densification of silty soils between stone columns is not possible. The design depends upon both the in-situ strength and stiffness of the stone columns and the area replaced by stone columns relative to the area of unimproved native soil.

The in-situ strength and stiffness of the stone within columns is difficult to determine directly. Stiffness is commonly estimated based on strength. CPT or SPT testing, both of which are readily available, will not penetrate gravel or do not provide realistic correlations to in gravel. A Becker Hammer test might provide a more reliable estimate of in-situ strength of the stone within the columns, but the Becker Hammer is not a readily available tool in many if not most U.S. locations. The Becker Hammer test would also introduce considerable construction delay and additional cost.

In the case of the Tacoma HOV project, the strength of the in-situ stone was not directly measured. Instead, the acceptance of installed stone columns was based on the visual inspections of cored samples. The degree of the intrusion of native soils into stone column was the main factor to determine the acceptance.

The sonic core allowed the visual inspection of a nearly continuous core and provided valuable information for the Contractor to adjust installation techniques to achieve better quality in stone column construction. Although sonic coring does not provide a direct correlation to the strength of the installed column, it does provide a visual confirmation of the continuity of the stone columns installed. Once installation procedures have been finalized, and production stone column installation commenced, a further round of sonic coring would have provided good confirmation of the methods.

Verification of the installed cross sectional area of stone columns can be much easier than strength verification, particularly with use of an automated data acquisition system. The in-cab system used in the Tacoma case study, especially in combination with the high frequency of exploratory CPT and SPT tests, provided confidence that specified volumes of stone were being placed within the liquefiable materials.

Conclusions

Accurately verifying the bearing layer profile needs to be done pre-bid to properly estimate quantities. This would ensure that there is reduced risk to both the client and contractor during the construction phase due to quantity overruns/under runs. Reduction in risk at the bid stage would be beneficial to the Client in the form of reduced overall costs.

The automated data acquisition system is an excellent tool for real-time control of stone placement and for documentation. Requirement of this type of system should be considered on large projects or in conditions with layered loose and dense materials. Additional tools for easily verifying strength of in-situ stone on a production basis are still lacking in the industry. Sonic coring can be used as a tool to assess the effectiveness of various construction techniques.

The results of settlement monitoring reinforced the importance of redundancy in both the numbers and types of monitoring devices. Although both the vibrating wire settlement indicating devices and the settlement plates were useful in identifying the end of primary consolidation, isolated settlement indicating devices placed between columns should not be relied upon to predict the overall settlement of embankment material over stone columns.

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