Protecting a Navigable Waterway Using a Deep Soil Mixing Wall

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ABSTRACT

This paper will discuss a case history that utilized a deep soil mixing (DSM) wall to protect a navigable recreation waterway from future landslides, as well as existing infrastructure crossing the site. The project that is the subject of this case history is located in Burnaby, British Columbia, Canada. The site is located in a former marsh area along a protected active creek named Still Creek. The site has been filled several times beginning around 1974 and continuing through about 1987. Several landslides occurred between 1979 and 1987 because of the filling activity. In addition, small slope failures have resulted in localized depressions. During additional filling, a large slide occurred in 1978 resulting in the blockage of Still Creek. A second large slide occurred in 1978, again blocking Still Creek. Although the site is currently statically stable, analysis indicates that during a major seismic event, the ground could move as much as 3 to 4 feet towards Still Creek.

Geotechnical issues as this site include:

- unstable banks;
- 30 feet of peat soils;
- an active block landslide resulting from un-dissipated pore water pressures that remain even after 20 years;
- on-going settlement under existing fill;
- significant predicted settlement of feet with placement of new fill; and
- surface soils that cannot support equipment.

To mitigate the potential of future landslides into Still Creek, and to protect the existing infrastructure, the above geotechnical issues needed to be addressed. Various options were analyzed for both static and seismic stability. A DSM wall was chosen for mitigating the potential lateral movement of on-site soils statically and during a seismic event. The suitability of the proposed DSM site stabilization measures was evaluated based on the results of numerical modeling using FLAC and global and internal stability modeling. The DSM wall is intended to provide a factor of safety of 1.5 against soil yield and failure of the site under static conditions.
conditions and to limit lateral displacements of the ground to no more than 6 to 12 inches (50 to 100 mm) under 1:475 year design earthquake ground motions.

To create an optimum design using the soil parameters and constructability, a cellular DSM wall was used that consisted of a cellular structure with two east-west walls connected by perpendicular webs spaced at about 15-foot centers. The DSM zone was completed into the underlying dense till-like soils.

This paper will also discuss the conditions at the site, the analyses, construction considerations, construction of the wall, and the quality control testing program used.

INTRODUCTION

This paper discusses a case history that utilized a deep soil mixing (DSM) wall to protect a navigable recreation waterway from future landslides, as well as existing infrastructure crossing the site. The project is located in Burnaby, British Columbia, Canada. The location of the site is shown in Figure 1. The site is located in a former marsh area along a protected active creek named Still Creek. The site has been filled several times beginning around 1974 and continuing through about 1987. Several landslides occurred between 1979 and 1987 because of the filling activity. In addition, small slope failures have resulted in localized depressions. During additional filling, a large slide occurred in 1978 resulting in the blockage of Still Creek. A second large slide occurred in 1978, again blocking Still Creek. Although the site is currently statically stable, analysis indicates that during a major seismic event, the ground could move as much as 3 to 4 feet towards Still Creek.

![Figure 1 – Vicinity Map](image-url)
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SITE CONDITIONS

Surface

The general site topography is flat, with a slight gradient towards Still Creek, which is located on the south side of the site as shown in Figure 2. Still Creek runs from west to east. The existing ground surface varies from 45 to 55 feet, geodetic datum.
The current flood control elevation for the site is 48 feet for the 25 year event and 50 feet for the 200 year event.

The Trans Canada Highway is located approximately 300 feet south of the site.

**Subsurface**

**Geologic Setting**

The site is located within a central valley depression of the Fraser Lowland region. Hoy et.al. (1967) infer that the valley is an erosional feature formed in the highly consolidated silty to sandy glacial Vashon till deposits which overlie Tertiary sedimentary bedrock. Following erosion, the valley was partially infilled with post-glacial, fine-grained sediments deposited as glaciomarine meltwater outwash from retreating glaciers. With isostatic rebound of the ground as the ice sheets melted, the valley was slowly cut off from the surrounding waters, resulting in the eventual cessation of glaciomarine sediment deposition and the gradual accumulation of soft and compressible peat along the margins of Still Creek and its tributaries, which drain the central valley and surrounding areas to Burnaby Lake.

**Subsurface Conditions**

The site is underlain by five soil units consisting of fill, organic silt, peat, soft clay and dense till as described below. The organic silt, peat and soft clay are...
considered to be extremely weak. The maximum depth of these weak soils is typically encountered slightly north of the current Still Creek channel. The elevation of the dense soils gradually rises from the south to the north.

**Fill:** The fill thickness generally ranges from about 10 to 17 feet with no identified trend over the site. The fill was generally loose to compact with SPT (Standard Penetration Test) values ranging from about 5 to 20 blows per foot (b/ft) with an average of about 9 b/ft. This information implies that the fill was placed in an uncontrolled manner and was not well compacted.

**Organic Silt/Peat:** The organic silt thickness is about 1 to 4 feet with an underlying fibrous to amorphous peat with a thickness of about 6 feet. The combined thickness of the organic layers ranged from about 6 to 10 feet with no identified trend across the site. Both units were very soft-to-soft with SPT values in the range of 0 to 5 b/ft, averaging about 3 b/ft. The natural moisture contents of the peat generally ranged from about 200 to 400 percent. While the organic silt water contents generally ranged from 80 to 100 percent.

**Soft Clay - Silt:** The soft clay unit ranged in thickness from about 10 feet in the northwest corner of the site to about 25 feet on the south side adjacent to Still Creek. Within the site the clay thickness appears to be about 20 feet. The plastic index values range from 20 to 50 percent and plot just below the A-line. The unit consistently had very low SPT values in the range of 0 to 1 b/ft. Field vane tests indicate that the undrained shear strengths generally ranged from about 200 to 600 pounds per square foot (psf). These tests also indicated that the soil is sensitive with the sensitivity ranging from 10 to as high as 40 based on the ratio of the undisturbed to remolded strengths. The natural moisture contents were well above the liquid limits as would be expected for a sensitive soil.

**Till:** The dense till was generally encountered at a depth of about 35 to 50 feet below the current ground surface with the top of the till varying from an elevation of about 25 feet in the northwest corner of the site to about elevation 0 on the south side adjacent to Still Creek. The upper 2 to 5 feet of the till unit was dense with SPT values in the 25 to 40 b/ft range. Below this upper zone, the till was very dense with SPT values generally in excess of 50 b/ft.

A generalized cross section of the subsurface conditions is shown in Figure 3.
A plot of depth versus undrained shear strength (Su) of the weak soils are presented in Figure 4. These values are based on field vane results. The results indicate that the undrained shear strength is less than 200 pounds psf for the peat and organic silt. The field vane tests indicated the clayey silt and silt that underlies the organic silt to be between about 200 and 800 psf.
Normalizing the results of the peak undrained shear strength with depth for the peat and organic silt, yields the following:

- $S_u/\text{Vertical Effective Stress} = 0.25$ to $0.35$

Based on laboratory triaxial testing, the normalized peak undrained shear strength with depth for the peat and organic silt is:

- $S_u (\text{Residual})/\text{Vertical Effective Stress} = 0.05$ to $0.10$
- $S_u (\text{Remolded})/\text{Vertical Effective Stress} = 0.01$ to $0.03$

For a comparison, the values for the $S_u (\text{Remolded})$ are close to the friction between two sheets of teflon material. These values also indicate a high degree of reduction of the strength of the material due to strain softening on the order of about 80 percent for residual, and about 95 percent for the remolded. These values are in general agreement with very soft, highly plastic soils (Sun, Golesorkhi, Seed – 1988)

**Groundwater**

The depth to the groundwater generally ranged from about 4 to 12 feet with some borings identifying inflow as shallow as about 2 feet. These measurements can be affected by shallow perched groundwater and the lack of time allowed for the groundwater to stabilize. The piezometers sealed at shallow depths indicated that shallow groundwater likely occurs at a depth of about 4 to 8 feet below the ground surface. The deeper piezometers installed in the fills and upper organic soils indicate a slight downward gradient. The deeper piezometers installed in the clay and underlying till by others located on the south side of the site indicate a slight upward gradient and the possibility that excess consolidation pore pressure still exists in the clay. The piezometer data is presented in Figure 5 below. This plot shows the normal hydrostatic pressure line and an inferred static piezometric line based on an upward gradient from the pressure measured in the top of the till. Four of the ten piezometer points in the clay indicated no excessive pore water pressure and plot near the upward gradient line. However, six of the piezometer points show some excess pore pressure in the clay.
The site is located in a former marsh area, which was filled in beginning around 1974 and continuing through about 1987. Several landslides occurred between 1979 and 1987 due to the filling activity. Small failures that occurred during the early filling stages resulted in a depression on a portion of the site. During additional filling, a large slide occurred in 1978 resulting in the blockage of Still Creek. A second large slide occurred in 1978, again blocking Still Creek. There was some additional minor filling in 1986 and 1987.

Initial site filling was carried out from 1975 through about 1978 and comprised placement of mineral fill directly over the natural peat. The initial layer of mineral fill was limited in thickness to not more than 3 feet and was placed directly over the undisturbed peat after the cutting of trees and flattening of underbrush. Subsequent lifts of mineral fill were placed only when excess pore pressures and settlements decreased to less than 1 foot.
Some 5 to 8 feet of mineral fill was placed in a series of lifts within the north portion of the site. However, because of continuing high pore pressure and rapid settlements under thin fill lifts, it was not possible to achieve comparable fill placement within the south portion of the site near Still Creek. Fill thickness was generally limited to 3 to 5 feet over a period of 2 years or more.

In 1978, two stability failures caused rapid settlement of the existing fills and movement of these fills towards Still Creek. The result of the failures was blockage of Still Creek on at least two occasions. The landslides were likely the result of stockpiling of mineral fills because of site access difficulty.

Some additional filling was attempted over a period of several years into the early 1980s.

**COMPRESSIBILITY**

In order to evaluate future settlement of the site for the proposed structures, placement of fills, and paving, an understanding of the past compressibility characteristics of the soils at the site was needed. Our analyses included evaluation of previous settlement monuments, visual observation, evaluation of piezometer data, laboratory testing, and back calculations on the previous settlement at the site.

**Previous Settlement and Conditions**

Beginning in July 2002, settlement monuments placed at the ground surface were monitored. Two lines of surface survey points located approximately 100 and 200 feet north of the Still Creek were monitored over an 80-day period. The results indicated that the settlements ranged from about 0.2 inches to over an inch, with the larger settlement measured at the southeastern side of the site. The amount of measured settlement implies a rate on the order of 1 to 4.5 inches per year, which is still on-going after 15 years since the fills at the site were placed.

Visual observations were also made in 2002. There is an existing former restaurant building in the north central portion of the property which is supported on timber piles. Based on measurements taken at the pile caps, the ground surface appears to have settled between about 8 and 26 inches since the piles were installed.

Based on the piezometer data shown in Figure 5, there still remains excess pore pressure in the clay unit. The excess pressure is attributed to the placement of the fill at the site, and has not completely dissipated even after 18 years since
placement of the fill. The data indicates that excess pore pressure near the middle of the clay unit is on the order of 350 psf. With a calculated total fill load of about 1,600 psf since the filling began, the 350 psf excess pore pressure implies that the center of the clay unit was about 80 percent consolidated in 2002 (the date of the survey readings).

Using laboratory data, the soil parameters for the organic soils and the soft clay are as follows:

- **Organic Soils**
  - Total Unit Weight = 90 pounds per cubic foot (pcf)
  - Compression ratio \((Cc/e+1)\) = 0.45
  - Coefficient of Consolidation \((Cv)\) = 0.5 to 1 square feet per day \((\text{ft}^2/\text{day})\)
  - Secondary Compression Coefficient \((C_{a})\) = 0.02 to 0.04

- **Soft Clays**
  - Total Unit Weight = 120 pcf
  - Compression ratio \((Cc/e+1)\) = 0.25
  - Coefficient of Consolidation \((Cv)\) = 0.04 to 0.08 \(\text{ft}^2/\text{day}\)
  - Secondary Compression Coefficient \((C_{a})\) = 0.01 to 0.015

To further assess the compressible model of the existing soils, consideration was given to the drainage path. A major uncertainty relates to the effective drainage path of the excess pore pressure, particularly in the clay, as to whether the water is draining both downward and upward. If we assume that drainage occurs in both directions, then to reach 90 percent of primary consolidation due to the fills placed at the site should have occurred over a period of about 3 years. If single drainage is assumed, then 90 percent of primary consolidation would increase to about 23 years. Since we anticipate that about 88 percent of the primary settlement may have occurred over a period of 18 years, single drainage appears to be occurring at the site. Other aspects that needed to be included in the evaluation were the impact of clay remolding because of the landslide activity and settlements of soft soils that generally increase if the soils are subject to high shear stresses. Based on sensitivity analyses, the impact of the clay remolding and settlement due to the high shear stress were found to be minimal as compared to the overall settlement of the soils at the site, and therefore were not included.

Using the results of the model described above, we estimate that the total settlement since the filling was started is likely in the range of 4 to 8 feet.
Future Settlements

The estimated additional settlement over the next 25 years, assuming no change in loading, is likely in the range of 3 to 8 inches, with a potential of up to approximately 12 inches. The low range reflects the case that the majority of the primary settlement is complete. The higher range reflects some additional primarily settlement because of remaining excess porewater pressure in the clays.

The additional settlement induced by additional filling was estimated to range from about 2 to 4 inches per foot of new fill. Since significant secondary settlement has already occurred, the effect of load increases less than 1 to 1.5 feet of new fill will be primarily to increase the rate of creep, but not necessarily to initiated additional primarily consolidation.

Methods were evaluated to lower the amount and impact of the existing and new fills to be placed at the site. In addition, consideration was given to reducing the current conditions by excavating the existing fills and replacing them with lightweight fills. After further evaluation, it was found that the existing condition and the planned grading at the site would significantly impact the stability of the site. Therefore, that impact was needed before addressing the settlement issue.

SLOPE STABILITY

The proposed project required the need to raise the site from about one to three feet. In order to evaluate the impact of the placement of fill on the stability of the site, a slope stability analysis was performed. The analysis consisted of the following:

- Static stability with the new loading conditions
- Static stability using a ground improvement wall
- Seismic stability analysis with the wall
- Additional stability analysis with the wall for different fill loading conditions under both static and seismic conditions

Model Development

A finite element model was developed for this analysis using the soil stratigraphy from several geotechnical subsurface investigations. Material properties were developed from field and laboratory data, and a review of available information. Input seismic ground motions were defined by matching the Capitola Station record from the Loma Prieta Earthquake to the Class C 475-year acceleration

The site was modeled using FLAC (Cundall, 2000). The FLAC model was calibrated with a 1D SHAKE model developed at the south end of the building. The computed peak ground surface accelerations derived from the SHAKE analysis were on the order of 0.11g, which is about 50 percent of the input outcropping acceleration that corresponds to the 475-year site-specific ground motions. In the 2D FLAC models, the computed peak ground surface accelerations vary from about 0.06g to 0.12g.

**Results**

Based on the analysis, the current conditions resulted in a predicted horizontal displacement of 1 to 2 inches under static loading conditions and 16 to 33 inches of movement under seismic conditions. The movement extended to roughly 600 feet from Still Creek. Placement of new fill would only significantly increase the movement both statically and seismically.

As a result, consideration was given to different methods to address the stability of the site. A Deep Soil Mixed (DSM) wall was selected and modeled. The wall was to be placed near the top of the bank of Still Creek. The criteria desired by installing the DSM wall were establish by the then property owner and were to provide a factor of safety of 1.5 against soil yield under static conditions. Lateral movement due to the 475-year seismic event was to be limited to 6 to 12 inches. This allowable movement was established by rule-of-thumb for the Vancouver area. Because of the soft soils at the site, the building was to be supported on driven piles. The criteria of 6 to 12 inches were found to be at the upper end of tolerance for the piles and was not desired to occur. As such, various analyses were performed with different fill conditions to further reduce the horizontal movement. Some of these included placing light-weight material as fill to temporarily support the construction of the building slab, and the grades in the surrounding parking areas were lowered with light-weight fill properties used to model the raising of site grades. A printout of one of the stability analyses is presented in Figure 6.
Figure 6 – FLAC Stability Analyses (Golder, 2007)

Based on the stability analyses, lateral displacement across the building site that was used to design the DSM wall is presented in Figure 7 below.
Figure 7 – Lateral Displacement Used in FLAC (Golder, 2007)
The results indicate lateral movement during the seismic event of about 1 to 2 inches at the ground surface beneath the planned building as shown in the above figure; the model represents the lowest grade for the area on the creek side of the proposed building. The soil between the DSM wall and Still Creek was estimated to move about 8 inches under seismic conditions. It should be noted that the DSM wall was modeled with the top of the wall located about 10 feet below the ground surface in order to optimize the wall design and to reduce the length of the cantilever portion of the wall. By lowering the top of the wall, significant movement would occur along the Still Creek slope, but would not propagate to the building. The slope stability analysis indicated a lateral movement of about ½ inch at the top of the wall (at a depth of 10 feet). The type of movement indicated by the analysis is similar to a block type failure. Such a failure would experience compression for a significant distance from the creek, as indicated in the analysis. Based on review of the various alternative treatments, the DSM method was selected.

**DSM Wall Background**

DSM involves the addition of binders (most commonly combinations of cement, lime, gypsum, and slag) to the in situ soils to create a chemical reaction product which bonds the soil particles together and results in improved soil mass consistency, strength, and deformation characteristics. Because DSM methods are fast, clean, flexible, and have minimal impact to surrounding facilities, such methods are being used in a growing number of civil applications, including excavation, tunneling, or foundation support, in situ retaining wall construction, and land development. The improved soil strength and dynamic stiffness within the DSM zone can limit horizontal deformation resulting from both static and seismic loading.

DSM methods rely on the physical mixing of in place soils with binders using mixing shafts with attached paddles or other means to form in situ columns or panels of strengthened soil. Traditional column-type deep mixing machines, such as those developed in Japan and Scandinavian countries, are constructed with mixing shafts consisting of auger cutting heads, discontinuous auger flights and mixing paddles and can vary from single to 8-shaft configurations depending on the purpose of deep mixing. The machine lowers augers from the surface into the existing ground, the augers are turned and binders are added to the soil in situ, forming a strengthened column of ground. Other technologies allow for the installation of rectangular-shaped panels using cutter-wheels to mix the in situ soil with binders.

**Implemented Scheme**

For this project, a DSM retaining wall using a cellular box pattern treatment involving the construction of a continuous line of columns connected with
perpendicular “rib” or “web” segments comparable to the webs on a steel beam was implemented. The width of the wall is dependent on the bending and base sliding resistance requirements while the replacement percentage is governed by the required composite strength of the stabilized zone.

The composite strength of the DSM zone is a function of the obtainable field soil-cement strengths and the layout of columns or panels within the improvement zone. The constructed DSM wall comprised a cellular structure with two east-west walls connected by perpendicular webs spaced at 15-foot centers. The upslope wall of the DSM had a nominal thickness of 6.5 feet through the peat, organic silt, and soft clay zones. The replacement ratio in the peat and organic silt and the silty clay and till was 46 and 38 percent, respectively. The DSM wall configuration is presented in Figure 8.

![Figure 8 – DSM Wall Configuration (Golder, 2007)](image)

**DSM WALL CONSTRUCTION**

**Wall Construction**

Construction of the DSM wall was undertaken by GAIA Inc (GAIA). Test panels were conducted within similar subsoil conditions. A variety of soil-cement mixes were developed and installed in sacrificial test panels which were subsequently cored and tested for compression and tensile capacity. The data collected from the insitu sampling of the test panels and associated laboratory testing were used to select an appropriate cement-soil-water mix design.
The DSM soil improvement technique utilized to construct the stabilization wall comprised a Bauer cutter/mixer head with a footprint measuring approximately 9 feet by 3 feet as shown in Figure 9. The cutter head was connected to a Kelly bar secured to a guide mast, which was custom fitted to a piling rig. The cutter head was inserted into the ground while mixing and injecting cement slurry into the in situ soils to create an essentially vertical column or panel of stabilized soil. The overall DSM structure comprised two rows of (longitudinal) walls that were continuous and overlapped the individual panels. Similar overlaps were provided with the transverse walls, such that the final product comprised a series of enclosed cells over the designated length of the DSM structure.

The panels extended from the bottom of the overlying mineral fill (approximately 10 feet below ground surface) through the soft peat, organic silt, and non-organic silty clay into the underlying dense till. The wall also included zones of partial depth panels which extended from the bottom of the overlying mineral fill (approximately 10 feet below ground surface) through the soft peat and organic silt and were keyed into the non-organic silty clay. The partial panels were required to provide additional stiffness within the peat and organic deposits. The DSM wall cross section is shown in Figure 10. In total, 333 DSM panels were installed, of which 289 were full-depth panels and 44 were partial-depth panels.
To provide continuity of the overall DSM wall, the plan layout of each individual full-depth panel was overlapped a nominal 8 inches with each adjacent panel along the long axis. The traverse walls were also provided with an 8-inch overlay, including overlaps at the intersections with the longitudinal walls. Similarly, the partial-depth panels were overlapped a nominal 8 inches along the long axis and a nominal 4 inches along the short axis.

As the cutter head was advanced into the ground, deviations of the cutter head from vertical were automatically recorded on a continuous basis for the full-depth of the panel. Torque, cutter wheel rotation and inclination of the Kelly bar were manually adjusted so that each panel was constructed within 1 percent of verticality. The verticality of each panel was confirmed by reviewing the recorded deviations from the output data generated by the DSM system.

The base of the full-depth panels were designed considering that they were keyed a nominal 2 feet into the dense till. The depth to till was evaluated on a panel-by-panel basis based on an increase in penetration resistance of the cutter head observed during installation, an increase in the wheel torque and comparison with the available geotechnical bore holes. The panels were subsequently continued a nominal 2 feet beyond the inferred contact with the till.

The bottom of all partial-depth panels were keyed a nominal 2 feet into the non-organic clay horizon. The bottoms of the partial panels were taken to be fixed at an elevation of about 20 to 21 feet. These fixed depths were established based on inferences made from available subsurface exploration data. During installation, the partial panel depths were determined by measuring the depth of
the cutter head below the known ground surface elevation. These depths were confirmed by reviewing the recorded total panel depth from the output data generated by the DSM system.

**Soil-Cement Strength and Stiffness**

Minimum requirements were specified for the resulting soil-cement material with respect to unconfined compressive strength, tensile strength, and compression modulus. In order to verify that the minimum requirements were being met, samples of the soil-cement mix material were recovered on a daily basis from selected panels and subsequently tested in the laboratory. The test samples included both wet (slurry) samples collected immediately following panel installation, as well as core samples obtained by diamond drilling after some strength gain of the soil-cement had been achieved.

Wet samples were generally obtained at a minimum frequency of one sample per day. Samples were generally recovered from three specific depths: 16, 29, and 39 feet below grade for full-depth panels and approximately 12 feet below grade for partial-panels. The samples were cast into cylinders, stored on site for initial setup, and then transported to the laboratory for wet curing and testing.

Diamond coring of the insitu panels was carried out on three separate occasions to obtain continuous samples of insitu soil-cement material for laboratory testing. A single panel was cored shortly after commencing operations. Eight additional panels were subsequently cored. The coring was carried out using HQ-3 equipment. The test holes generally extended from the ground surface into the unaltered till soils a nominal 1.5 feet beyond the bottom of the soil-cement material. The core was placed into core boxes, appropriately labeled, and transported to the laboratory for curing and laboratory testing. Following completion of drilling, the holes were grouted with a lean cement mix.

**Unconfined Compressive Strength**

Representative samples were cut from the continuous core and tested for unconfined compressive strength and tensile strength. The samples were generally tested at 7, 14, and 28+ days, although further tests were often suspended if passing tests were achieved at 7 days. The test results indicated that the soil-cement material exceeded the minimum design criteria.

**Tensile Strength**

The tensile capacity of the soil-cement material was evaluated using the Brazilian point load test. Tensile tests were performed on the wet samples only. The test
results indicated that the tensile capacity of the soil-cement material exceeded the minimum design criteria set forth in the specifications for both the organic horizon and non-organic soil horizons.

**Unconfined Compression Modulus**

The unconfined compression modulus of the soil-cement material was approximated by recording the axial displacement of the compression boom during the unconfined strength testing of the wet samples. Strain was recorded on several samples collected from the horizons. The test results indicated that the soil-cement samples recovered from the organic horizon met the unconfined compression modulus requirements. The test results also indicated that the tests performed on samples recovered from the non-organic horizon generally met the modulus requirements.

**CONCLUSIONS**

Although the site was statically stable, analysis indicates that during a major event, the ground could move as much as 3 to 4 feet towards Still Creek, possibly blocking the creek. Several slope stabilization options were considered, including the use of sheet piles, lime columns in conjunction with sheet piles, and deep soil mixing (DSM). A DSM wall was chosen for mitigating the potential lateral movement of on-site soils statically and during a seismic event.

The suitability of the proposed DSM site stabilization measures was evaluated based on the results of numerical modeling using FLAC and global and internal stability modeling. The DSM wall is intended to provide a factor of safety of 1.5 against soil yield and failure of the site under static conditions and to limit lateral displacements of the ground to 6 to 12 inches under a 1:475 year design earthquake ground motions.

To create an optimum design using the soil parameters and constructability, a cellular DSM wall was used that consisted of a cellular structure with two east-west walls connected by perpendicular webs spaced at about 15-foot centers. The DSM zone was fixed within the underlying dense till-like soils.

DSM involves the addition of binders (most commonly combinations of cement, lime, gypsum, and slag) to the insitu soils to create a chemical reaction product which bonds the soil particles together and results in improved soil mass consistency, strength, and deformation characteristics. Because DSM methods are fast, clean, flexible, and have minimal impact to surrounding facilities, such methods are being used in a growing number of civil applications, including excavation, tunneling, or foundation support, insitu retaining wall construction, and land development. The improved soil strength and dynamic stiffness within
the DSM zone can limit horizontal deformation resulting from both static and seismic loading.

REFERENCES

1. AMEC Report dated January 30, 2001 entitled Geotechnical Reports and Correspondences, Eastbrook Executive Park, Burnaby, BC.


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