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Ground Improvement

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ABSTRACTS

Selection, Design and Specification of Ground Modification for Mitigation of Earthquake-Induced Liquefaction & A Briefing of Observed Earthquake Performance
Keynote Presentation by Dr. Juan Baez of Advanced Geosolutions Inc., Lake Forest, CA

The evaluation of earthquake-induced liquefaction has become a routine part of geotechnical engineering design. For a given project, if an analysis identifies a potential for liquefaction and the consequences of liquefaction are deemed unacceptable, then some form of hazard mitigation is required. Mitigation efforts may consist of removing the liquefiable soils, bypassing the liquefiable soils with deep foundations, structurally accommodating the deformations or strength loss caused by liquefaction, or preventing the onset of liquefaction through ground improvement. The fundamental ground improvement mechanisms for liquefaction mitigation include densification, drainage, and reinforcement. When evaluating, recommending and specifying various ground improvement methods for liquefaction mitigation, practitioners should understand the fundamental mechanics involved and applicability limitations of the various methods. This presentation offers a review of the above topics and highlights observations and lessons learned from a number of strong earthquakes (Niigata ’64, Loma Prieta ’89, Kobe ’95, Kocaeli ’99, Christchurch ’11, and Tohoku ’11) case histories.

Geotechnical Challenges for a Mixed-Used Development at Gibsons, B.C.
Karim Karimzadegan, Nima Tafazzoli, Takahiro Shozen, Horizon Engineering

The subject site is located on the waterfront in the town of Gibsons, BC. The proposed development comprises a multi-level hotel, conference centre, and residential development, to be partially supported over two to three parkade levels, and offshore amenities.

The Town of Gibsons and the subject site are underlain by the Gibsons Aquifer, which is a confined aquifer comprising sand and gravel that provides drinking water for the town. The confining Gibsons Aquitard is inferred to comprise variable thicknesses of sand, peat, silty sand to sandy silt to silt, and localized till-like materials within the subject site. Artesian groundwater pressures have been observed within the Gibsons Aquifer. Hydraulic connections have been observed between the Gibsons Aquifer and the ocean at the central portion of the site and between the Gibsons Aquifer and the town well. It was required that the Gibsons Aquifer not be negatively impacted by the proposed development as it was the source of drinking water for the town.

Several phases of subsurface investigations were carried out at the subject site comprising drilling onshore and offshore auger and sonic holes, advancing Dynamic Cone Penetration Tests (DCPTs) and WildCat Cone Penetration tests, excavation of test pits, and installation of piezometers. The purpose of the proposed subsurface investigations was characterizing the aquitard and aquifer, depth to the top of the aquitard, the aquitard thickness, artesian groundwater pressures within the underlying aquifer at the test hole locations.

Potential geotechnical challenges for this project were recognized as artesian groundwater pressures, high non-artesian groundwater levels, sea level rise, loose and compressible soil, liquefaction, impact of dredging of foreshore area, methane gas buildup, soil contamination, slope stability under static and seismic loading conditions.

Seepage and deformation analyses were carried out to assess the site and subsurface conditions during and after construction of the proposed development. The foundations for the entire building footprint were proposed to be conventional strip and pad foundations or a raft foundation. Due to the presence of loose and compressible subgrade materials, ground improvement was recommended beneath the proposed foundations. Different ground improvement techniques were studied, and Deep Mixing was selected as the practical solution, that would maintain the aquitard’s properties that resist the underlying artesian pressures of the Gibsons Aquifer throughout the installation and curing process.

In general, the intent of the ground improvement construction methodology is to treat the soil in situ without need for mass excavation above the aquifer and greatly reduces the amount of soil import and export which will minimize construction time and traffic. Risk management measures are added to minimize the potential for breach in the aquifer. Preventative measures and remedial actions are proposed to provide a reasonable assurance that the construction process consider protection of aquifer during the course of ground preparation and improvements.

Use of Earthquake Drains for Liquefaction Mitigation at an Anchored Sheet Pile Wall System: Seismic Soil-Structure Interaction Analysis

Viet Tran, Stantec Consulting Ltd.

Soil anchors and earthquake drains were used to reduce the soil liquefaction and improve the seismic performance of the sheet pile wall system. The design of earthquake drains was carried out using the computer program FEQDrain and the seismic soil-structure interaction analysis was carried out using computer program FLAC implementing UBCSAND and PM4SAND soil constitutive models. The analyses were carried out for crustal, inslab and subduction earthquake motions corresponding to a return period of 475 years consistent with the 2015 National Building Code of Canada (NBCC). The performance of the sheet pile wall using the two soil models UBCSAND and PM4SAND were compared.

Ground Improvement Design and Construction for Seattle’s Elliot Bay Seawall Replacement and Retrofit

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Located on Seattle’s downtown waterfront and up to 500 feet from the original shoreline, the Elliott Bay Seawall was constructed in the early 1900’s over soft/loose non-engineered fill, estuary and beach deposits. The fill includes wood (sawdust, mill ends) from historic waterfront sawmills and debris from the 1889 Great Seattle Fire. Before the seawall, access to waterfront piers was via an extensive timber trestle network. The seawall was built to replace the trestles with reclaimed land and consisted primarily of timber-pile-supported walls (concrete gravity walls and concrete-and-sheet-pile-faced timber relieving platforms) with hydraulically placed backfill. The fills, estuary and beach deposits are liquefaction susceptible.

Because of the effects of the 2001 Nisqually earthquake on the waterfront, the City embarked on an evaluation of the existing seawall, including a condition and seismic vulnerability assessment. The evaluation led to design and construction of nearly 3,200 feet of replacement seawall and retrofit of 500 feet.

Numerous seawall replacement/retrofit designs were considered with the final vetted design using an Improved Soil Mass (ISM). The ISM supports a seawall superstructure and provides all seismic lateral restraint. The ISM consists of cellular arrangement of jet-grouted soilcrete columns.

Jet grouting was selected for ISM construction because of the variable soils, debris, obstruction posed by the 8,300 original seawall piles, other undocumented trestle piles, and constraints of the dense, urban waterfront environs. With approximately 6,000 soilcrete columns up to 93 feet deep with a total volume of approximately 175,000 cubic yards, the Elliott Bay Seawall Project is one of the largest jet-grout projects ever constructed.

This paper describes the seawall assessment, ISM design, and ISM construction. It presents the ISM performance-based criteria and non-linear time-history analyses used for design. It discusses construction aspects including installation, QA/QC, and spoils management. Lastly, it presents design and construction challenges and lessons learned.

Design of Ground Improvement to Withstand Seismic Hazard from Crustal and Subduction Earthquake Sources

Guoxi Wu, BC Hydro Engineering

The fourth generation seismic hazard maps of Canada developed by Geological Survey of Canada (GSC) included hazard values for a probability of 2%/50 years that were adopted in the seismic provisions in the 2005 and 2010 National Building Code of Canada (NBCC). However, these hazard values were derived from only the crustal earthquake sources (magnitude in the order of 7), while seismic hazards from the Cascadia subduction earthquake source (magnitude in the order of 9) were evaluated separately using a deterministic approach for hazard assessment based on the distances to the site. The hybrid method mixing probabilistic and deterministic approaches makes it impossible to design a certain structure to withstand seismic risk at a given overall probability level including all earthquake sources.

The 2015 GSC fifth generation seismic hazard model addressed the above issue by providing seismic hazard maps (e.g., 2%/50 years) with seismic hazards from all earthquake sources including the contribution from the Cascadia subduction earthquake. However, the total Uniform Hazard Spectra (UHS) possesses challenges to civil engineers in how to apply the UHS in engineering design as the two earthquake sources have dramatically different magnitudes (M7 for crustal and M9 for subduction interface) and thus they would result in ground and structural response (such as ground displacement, soil liquefaction potential, or bending moment in building columns) in an order of magnitude difference. Using the same UHS for crustal and subduction sources will not work in most cases for engineering performance assessment or in design of new buildings.

This paper will provide an overview on how to make use of crustal, in-slab, and interface subduction hazard values from the 2015 GSC Model for the 13148 grid points (10 km by 10 km) in southwestern Canada (southern BC and western Alberta). The UHS for crustal/in-slab earthquakes and UHS for Cascadia subduction interface earthquake can be derived at a couple of probability levels. Structure performance assessment can then be determined separately for the two main earthquake sources at a couple of probability levels. The overall probability at a given performance level (displacement, liquefaction or others) can then be determined by adding the probabilities from each of the two individual performance hazard curves. The overall performance (displacement, liquefaction or others) at a target probability level (e.g., 2%/50 years) is then determined from the overall combined performance hazard curve.

The paper will then present hands-on examples, for a project site involving ground improvement as a counter measure to soil liquefaction, for determining seismic ground displacements from empirical equations (Bray and Travasarou 2007, Macedo et al 2017) and also from finite element time history analyses to illustrate the proposed procedures.

Ground Improvement Design and Construction for Vancouver's South Fraser Transmission Relocation
Claude Berard, Keller

Located along the southern arm of the Fraser River between Richmond and Delta, the George Massey Tunnel was constructed in the late 1950's. The tunnel is approximately 630 metres long and made up of six precast concrete sections that had been sunk into the loose sand and silt of the river bed. This area is one of the most seismically active regions in Canada and the deltaic sediments are typically highly susceptible to earthquake induced liquefaction within the upper 20 to 30 metres.

With increasing standards for seismic design of structures, the Province of British Columbia embarked on an evaluation of the existing tunnel, including a condition and seismic vulnerability assessment. The evaluation led to the proposal to replace the George Massey Tunnel. In order to facilitate this replacement, the existing 230 kV transmission line running through the tunnel needed to be relocated. A new overhead transmission line spanning across the Fraser River is planned. This plan includes five separate locations for the new transmission structures that require ground improvement to depths of 30 metres to mitigate liquefaction and seismically induced lateral spread.

Several of the new structure locations are adjacent to or within watercourses connected to the salmon bearing Fraser River, which lead to a high level of environmental sensitivity.

Numerous ground improvement designs were considered with the final design using vibro-replacement (stone columns). The dry bottom-feed was the selected methodology to ensure construction of a continuous stone column to the required improvement depth of 30 metres. Approximately 1,000 stone columns of 900mm nominal diameter are being constructed across the five separate structure locations to improve a total plan area of 6,700 m^2.

This paper describes the ground improvement design and construction. It presents the densification performance-based criteria and justification of utilizing the dry bottom-feed technique. It discusses construction aspects including infill material, installation, and QA/QC. Lastly, it presents design and construction challenges as well as lessons learned.

Annacis Is. and Northwest Langley Wastewater Treatment Plant Ground Improvement
Andrew Port and Steven Ahlfield, Klohn Crippen Berger

The Annacis Island Wastewater Treatment Plant (AIWWTP) and Northwest Langley Wastewater Treatment Plant (NLWWTP) are owned and operated by Metro Vancouver. To meet increasing demand requirements, NLWWTP Phase 1 Upgrade began in 2012 and was completed in 2016. The AIWWTP Stage 5 Phase 1 Expansion began in 2014 and construction is currently ongoing.

The 2007 and 2012 editions of the British Columbia Building Code require that wastewater treatment facilities provide post-disaster immediate operations for an earthquake with a 2% probability of exceedance in 50 years. Seismic hazard analyses indicated that the peak firm ground acceleration was 0.49 g at the NLWWTP site and 0.51 g at the AIWWTP. Both sites are underlain by a thin surficial layer of fine-grained soils and Fraser River fine sand deposits that extend to 30 m and 50 m maximum depths. Most of the sand deposits are liquefiable under the design earthquake and effective stress seismic displacement analyses indicated large lateral and vertical displacements would occur. To reduce seismic induced displacements and to meet post-disaster immediate operation requirements for all new structures, densification of the fine sand to a maximum depth of 34 m was required beneath and around new structures. Vibro-replacement stone columns were selected as the densification method. The ground improvement contractor elected to conduct vibro-replacement at both sites using the top feed wet method with a combination of air and water injection.

At AIWWTP ground improvement consisted of densifying an area of approximately 43,000 square meters to a maximum depth of 34 m through the installation of 5,500 stone columns. A total of 138 cone penetration test (CPT) were conducted during densification trials and production work to confirm post-ground improvement CPT penetration resistance.

At NLWWTP ground improvement consisted of densifying an area of approximately 12,300 square meters to a maximum depth of 30 m through the installation of 2,150 stone columns. A total of 70 CPT were conducted during densification trials and production work to confirm post-ground improvement penetration resistance. The expansion area was preloaded for 6 months following completion of vibro-replacement and CPTs were conducted after preload removal.

The large areas and durations of the projects, in conjunction with the number of post-densification CPT tests allowed for collection of a significant amount of CPT data. This paper presents observations on the CPT post-densification data. Key observations include the ageing effect, or lack thereof, the effects of preloading, variability in ground improvement results measured by CPT data scatter and repeatability of CPT testing.

George Massey Tunnel Pile Load Test Program
David Siddle, Golder Associates

As part of the George Massey Tunnel Replacement Project, Golder was retained by The Ministry of Transportation and Infrastructure to perform a geotechnical investigation of a full scale, instrumented static pile load test adjacent to the George Massey Tunnel's southern portal. The aim was to obtain site specific measurement data on the effects of pile driving on sensitive adjacent structures and site specific measurement data on pile load-deflection response, displacement and pile load transfer during execution of the static pile load test which would potentially reduce uncertainties and risk for a future design-build contractor for the proposed replacement bridge.

The work included design and implementation of automated noise, displacement, settlement, vibration and robotic survey systems to monitor the effect on the adjacent structures and existing tunnel to pile driving in real time. Five large diameter (2.0 m) steel pipe piles were installed to a depth of approximately 67m, followed by the construction of a load test reaction frame weighing over 100 metric tonnes, which was used to conduct a static load test capable of exerting over 60MN of axial load by way of 12 synchronized large capacity hydraulic rams. The final deliverable was a factual report associated with the static load test, pile driving dynamic response, construction monitoring, and adjacent structure monitoring. This paper provides an overview of this technically challenging project, and discusses what was learned along the way.

Hybrid Steel Sheet Pile Cofferdam with Soil-Cement Improved Ground for the Westridge Marine Terminal Bulkhead

Hollie Ellis, William Perkins, and Gerard Buechel, Shannon & Wilson Inc.
Henrik Kristiansen, Kiewit Infrastructure Engineers

The Westridge Marine Terminal was developed in the 1950s and is currently undergoing an expansion as part of the Trans Mountain Expansion Project. Initial Terminal development included filling along the Burrard Inlet shoreline to create space for oil handling and loading facilities. As part of the Terminal construction, the foreshore will be expanded with a new bulkhead. Existing soils underlying the expansion are liquefaction-susceptible fills and marine deposits overlying competent glacially-overridden soils.
New bulkhead concepts considered included: (a) tied-back sheet piles; (b) ground improvement (GI) with sheet pile fascia; and (c) free-standing cellular cofferdams. In a collaborative process between designers and contractors, a hybrid system utilizing cellular cofferdams and GI was ultimately selected and designed.

The steel sheet pile cellular cofferdam will consist of 21.1-m-diameter circular cells connected via arc cells. An approximately 45-m-wide concrete slab will be constructed atop the cofferdam, extending landward to support new facilities. Initial cofferdam concepts included excavating the loose fill and marine deposits and replacing these with select granular fill within the cell to maintain stability of the cofferdam. In addition, excavation and replacement of existing loose soils with select granular fill within 3 m landward of the cells was needed to reduce the lateral pressures acting on the cells.

The cellular cofferdam concept was initially developed using U.S. Army Corps of Engineers methodologies and supplemented by three-dimensional dynamic soil-structure interaction analyses using 475- and 2,475-year return-period design earthquake ground motions. These analyses indicated that excavation and replacement of the loose soil with select granular fill alone would not provide acceptable lateral deformations under the design earthquake ground motions. These analyses also indicated that the existing soils landward of the cofferdam would be subject to liquefaction and excessive settlement under both the 475-year and the 2,475-year return period design earthquake motions.

To provide the required bulkhead seismic performance and support the on-land processing and loading infrastructure, the final bulkhead and foreshore design utilizes a hybrid system of cellular cofferdams and soil-cement GI. The final design is based on: installing the cofferdam sheet piles; leaving existing soils within the cells and behind in the foreshore area in place and backfilling to grade as needed; and installing jet grout and deep soil mixing GI to form continuous, interconnected soil-cement panels and struts within the cofferdam cells and the foreshore area behind the cells. The GI within the cofferdam cells increases the strength and stiffness of the cells, reduces the flexibility of the cofferdam structure, and increases the resistance of the structure to earthquake forces and reduces permanent ground displacements. GI in the upland area reduces the seismic lateral loads acting on the cofferdam cells, further reducing the bulkhead and foreshore permanent ground displacements, and mitigates liquefaction of the existing soils between the interconnected soil-cement panels and struts. To provide acceptable total and differential vertical and lateral permanent ground displacements for the design earthquake ground motions, new facilities are supported directly on the soil-cement struts and panels or on concrete slabs supported on these. Three-dimensional non-linear time history analyses were used for the final performance-based design.

An Evaluation of Installation Effects of Stone Columns Using a Full Displacement Method in a Post-Glacial Deposit

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John Howie, University of British Columbia

The construction of highway embankments on soft/loose soil deposits often requires the use of ground treatment to enhance the properties of the foundation soils. One approach is the use of stone columns for settlement control, stabilization, densification and/or reinforcement of the foundations soils. The design of structures/embankments supported on stone columns is conventionally done using analytical approaches based on the area replacement ratio (a_s) which is a function of the stone column diameter, layout and spacing. These solutions also idealize the stone columns and surrounding soil as a uniform block with composite properties. Selection of appropriate design parameters requires consideration of stone column installation effects on the soil being treated.

This paper describes the results of a field trial developed to obtain insight into the effect of stone column installation using a full displacement method in a post-glacial soil deposit comprising fine and coarse-grained soil layers with varying levels of over-consolidation and sensitivity. Field measurements were obtained using push-in pressure cells installed in the test section prior to stone column construction. The analysis of instrumentation readings and in-situ testing data collected in the field trial also provided a valuable opportunity to improve the understanding of soil behaviour during and after stone column installation.

The instrumentation data indicated that the installation of stone columns caused breakdown of soil structure in fine-grained soils resulting in very low effective stresses during ground treatment and a permanent reduction in undrained strength (s_u). The instrumentation data collected in coarse-grained soil also indicated a temporary condition of very low effective stress during stone column installation but an apparent increase in strength and stiffness.