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BC Ministry of Education Seismic Retrofit Guidelines, 2020 Edition Liquefaction Guidelines – Geotechnical and Structural Considerations

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ABSTRACT

Liquefaction guidelines in the BC Ministry of Education's Seismic Retrofit Guidelines, 2020 edition provide geotechnical engineers with specific requirements regarding site investigation, analysis, and determination of liquefaction induced soil deformations that are essential for performing seismic assessments. Additionally, the guidelines provide structural engineers with procedures to evaluate the building and its foundations and offer various mitigation approaches if remediation is necessary. Close collaboration between geotechnical and structural engineers is essential in assessment of liquefaction demands and the influence on structural response and to ensure retrofit design is carried out only if required.

Geotechnical: The geotechnical approach varies according to whether the site is located on an alluvial fan, a near-shore floodplain, or an inland floodplain. The site is first characterized using location, geometry, and properties of the non-liquefiable crust. Liquefaction triggering assessments are completed using CPT or SPT data from site investigation and the risk of liquefaction-induced flow slides is assessed. The appropriate method according to site type is used to estimate differential post-earthquake vertical movement and differential post-earthquake horizontal movement. The bearing, sliding, and punching shear resistances of the foundations are also determined.

Structural: The structural approach includes determining the buildings' residual drift after strong shaking to which horizontal and vertical soil deformation-induced drifts due to liquefaction are added. The soil deformations for the site are converted to drift demands on the buildings' Lateral Deformation Resisting System (LDRS) and the Vertical Load-bearing System (VLS) separately. Consequently, the total calculated drift is compared with Life Safety/Collapse Prevention Drift Limits (CDL) for the LDRS and VLS. Furthermore, the capacity of all structural elements (above and below grade) and their connections are assessed for liquefaction induced deformations and the resulting forces. This paper will provide examples regarding assessment of different LDRS/VLS/foundation systems and illustrate a structural upgrade scheme where mitigation is required.

INTRODUCTION

Liquefaction occurs when a build-up of pore-water pressure during an earthquake causes soil to rapidly lose strength and stiffness. This soil response can induce significant loads and deformations on structures when they occur in soils beneath and adjacent to foundations. This paper is based on Volume 11 – Liquefaction Guidelines of the BC Ministry of Education's Seismic Retrofit Guidelines 2020 Edition (SRG 2020) which provides guidance on how to evaluate liquefaction potential, estimate the consequences of liquefaction, assess the performance of the structure, and mitigate demands caused by liquefaction, if required.

EVALUATION OF LIQUEFACTION POTENTIAL

Site Investigation

The primary information required to complete a liquefaction assessment includes soil data collected from in-situ testing, site topography and a foundation layout showing footing depths, sizes and loads.

In-situ testing comprises penetration tests, such as the Standard Penetration Test (SPT), Cone Penetration Test (CPT) or Becker Penetration Test (BPT). Shear wave velocity should also be measured in the top 30 m (Vs30) for consideration of site amplification effects, which affect both the inertial loading on the structure and soil liquefaction. The extent of site investigations needed to establish the likely performance of school buildings in potentially liquefiable ground may vary considerably from site to site. In general, the Guidelines recommends a minimum of four investigation locations spaced strategically at 50 m or closer around and as close as possible to the building, with additional holes as required to characterize slopes adjacent to the school. Measurement of the undrained strength of cohesive soil layers is also required and is particularly important where the cohesive soil forms a non-liquefiable crust over the liquefiable layers.

Detailed site topography is critical for proper geotechnical evaluation of liquefaction displacements. For liquefaction assessment, topography several hundred meters away from a structure must be clearly understood, which typically extends outside the limits of a school property. High quality data, such as that obtained from LiDAR survey should be used to characterize the site. An example contour plan showing processed LiDAR data is shown in Figure 1. LiDAR is an excellent tool as it permits collection of topographic information from adjacent properties. For larger scale areas containing multiple concentrated school sites, the cost for an aerial LiDAR survey can be as low as about $1,000 CAD per site. An example contour plan showing processed LiDAR data is shown in Figure 1.

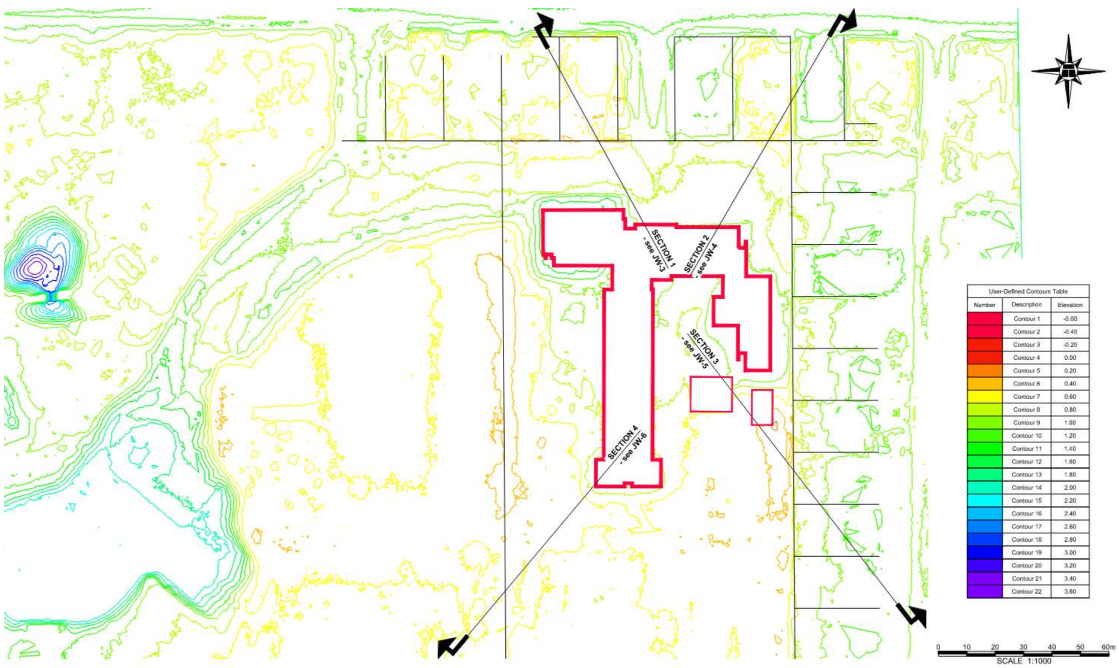

Figure 1. Example of Processed LiDAR Survey Showing Local Topography Surrounding School on Inland Floodplain Site

Cyclic Stress Demand vs. Cyclic Resistance

Liquefaction triggering is evaluated by comparing the estimated cyclic stress demand induced by the design earthquake to the estimated cyclic resistance of the soil. The demand and resistance are expressed as stress ratios: cyclic stress ratio (CSR) and cyclic resistance ratio (CRR). The factor of safety against liquefaction is defined as the ratio of CSR to CRR. Triggering of liquefaction is expected to occur at a factor of safety less than one.

Calculation of CRR and CSR in Volume 11 of the guidelines is consistent with Boulanger and Idriss (2014) [1]. CRR is determined using empirical equations developed for in-situ testing data, principally SPT blow counts and CPT tip resistances. CSR may be determined from the simplified method, adopted from Seed and Idriss's (1971) procedure [2], or from the results of a site-specific response analysis (SSRA). The equation for the simplified method of determining CSR is shown in Eq. 1 below.

$$CSR = 0.65 \frac{a_{max}}{g} \cdot \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot \frac{r_d}{MSF} \tag{1}$$

where a_{max} = peak ground surface acceleration, g = acceleration of gravity (in same units as a_{max}), σ_{vo} and σ'_{vo} = total and effective vertical stresses at the depth of interest, and r_d = depth reduction factor applicable to a depth of 20 m, and MSF is a magnitude scaling factor which weights the contribution of each magnitude to liquefaction potential relative to the reference magnitude M7.5. For M7.5, MSF=1.0.

The peak ground acceleration and earthquake magnitude values to be used in Equation 1 is under active consideration with the recently released 6th generation Canadian seismic hazard values. The current version of the Guidelines recommends continuing the standard practice of using the probabilistic acceleration and mean magnitude from the 6th generation seismic hazard model for assessments. However, further study is needed to confirm the appropriateness of this approach with the 6th generation hazard model, which now considers site amplification probabilistically.

SITE LOCATION AND TYPE

If evaluation yields factors of safety against liquefaction triggering of less than one under the design earthquake then the school site is deemed to be a liquefiable site.

Liquefaction can induce multiple surface manifestations, including lateral spreading, settlement, and bearing failures. The magnitude of lateral displacements is highly dependent on the location of the site, topography and thickness of liquefiable material present. Where thick liquefiable soils are present close to large slopes, such as deep ditches, creeks and rivers, large lateral displacements can occur. The deep ditch, creek or riverbank is a free face towards which liquefiable soils tend to move, commonly referred to as lateral spreading.

Challenges with Inland Floodplain Conditions

Richmond, BC, is located on a nominally 9 km wide by 14 km long island within the Fraser River Delta and contains around 50 school sites which are mostly located away from the Fraser River. Inland Richmond school sites are typically characterized by relatively flat topography and thick non-liquefiable cohesive silt crusts underlain by thick liquefiable sand deposits. The predominant topography is typically associated with minor slopes of 1% to 2% to facilitate surface drainage on fields and playgrounds. Given the number of school sites in Richmond on liquefiable soil, it was necessary to carefully evaluate how the geotechnical conditions and topography at these sites would be considered under the updated 2020 Guidelines.

Geotechnical assessments completed for Richmond schools using previous versions of the Guidelines produced lateral displacement estimates on the order of 300 mm to 600 mm. These estimates were prepared using widely-used empirical approaches such as Youd et al. (2002) [3] and Zhang et al. (2004) [4]. For Richmond, the thickness of the liquefiable deposits (often >20 m) dominates the other inputs to these methods and large horizontal displacements are predicted regardless of the ground geometry input. Large horizontal displacement estimates typically result in expensive, disruptive structural retrofits or full school replacements. Therefore, the authors examined the applicability of the empirical approaches to the inland Richmond school sites.

The Youd et al. (2002) database of case histories, which is a refinement of the database presented in Bartlett and Youd (1995) [5], includes observations of lateral displacement towards a free face such as a river bank and on constant sloping ground of alluvial fans.

Lateral spreading towards a free face was no longer observed at setbacks more than 300 m from the free face. Beyond the 300 m setback, displacements were randomly oriented and influenced by local topography. The observed displacements at various setback distances to the free face, as presented in Bartlett and Youd (1995) is shown in Figure 2a. The Zhang et al. (2004) database uses some of the same case histories used in Youd et al. (2002) but evaluates the data as ratio of setback distance (L) divided by slope height (H) as shown in Figure 2b. The dataset stops at an L/H of less than 40, due to a lack of case histories beyond this limit.

Based on data presented in Youd et al. (2002) and Zhang et al. (2004), a 300 m setback from a creek or river bank can be used as a screening tool to determine if liquefaction-induced lateral displacement towards a free face must be considered for a school.

The applicability data presented in Youd et al. (2002) and Zhang et al. (2004) for constant sloping ground was also reviewed, since site grades surrounding Richmond schools are relatively flat. A review of the case histories in the Youd et al. (2002) and Zhang et al. (2004) dataset shows that the length of constant ground slope are longer in the case history database than typical Richmond conditions and the non-liquefiable crust thicknesses are also thinner in the case history database than typical conditions at Richmond schools. Slope lengths in the case history database are typically 200 to 300 m whereas typical constant slope lengths in Richmond are 10 to 50 m. The non-liquefiable crust thicknesses in the case history database are typically less than 2 m, whereas non-liquefiable crust thicknesses at Richmond school sites are typically 4 to 6 m.

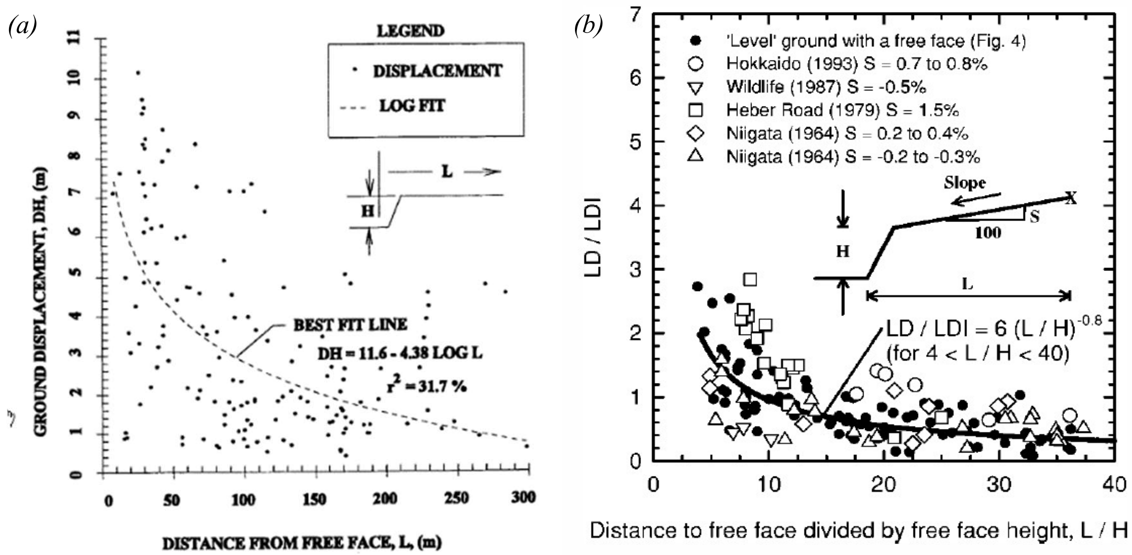

Figure 2. Observed Lateral Ground Displacement as a Function of Set Back from Free Face Feature from (a) Bartlett and Youd (1995) and (b) Zhang et al. (2004)

Site Type

Based on the review of empirical methods and conditions at inland Richmond school sites discussed above, the 2020 Guidelines provide an updated framework to consider the consequences and mitigation measures. The Guidelines provide three basic liquefiable site types to facilitate discussions between geotechnical and structural engineering team members: "alluvial fan" (site on large-scale constant sloping ground), "near shore floodplain" (sites on floodplains where the location is less than 300 m from river/creek banks or ocean high tide lines); and "inland floodplain" (sites on floodplains where the location is more than 300 m from river/creek banks or ocean high tide line).

ESTIMATING CONSEQUENCES OF LIQUEFACTION

There are several consequences of liquefaction that must be considered for seismic school retrofit projects. Lateral and vertical ground displacements due to liquefaction will induce permanent displacements in the structure. Shallow foundations could experience punching failures into the underlying liquefiable soil. Pile foundations will have reduced axial and lateral resistance due to liquefaction. The guidelines outline varying approaches to estimating consequences according to site type.

Lateral Displacement

For alluvial fan sites and near shore floodplain sites, it is recommended that the lateral displacements estimated using either Youd et al. (2002) or Zhang et al. (2004) be taken as the differential horizontal movement (ΔH) to be provided to the structural engineer. The primary direction, based on slope geometry, should be provided to the structural engineer. The lateral displacement perpendicular to the primary direction of movement should be taken as 50% of the ΔH in the primary direction in absence of detailed analysis.

For inland floodplain sites, four sections that represent the slope geometry surrounding the school should be generated and evaluated using two parallel methods: limit equilibrium analysis and Youd et al. (2002).

Limit equilibrium analysis should be completed for each section to estimate the yield acceleration of the governing static slip surface with liquefied soil properties. An example of this procedure is shown in Figure 3. From the yield acceleration, horizontal displacement estimates should be calculated using the simplified Newmark method outlined in Macedo et al. (2017) [6].

Horizontal displacements are also estimates for each section using the Youd et al. (2002) method with a depth cut-off applied equal to four times the slope height per Youd (2018) [7], to limit the thickness of liquefiable material considered. To calculate ΔH using the Youd et al. (2002) method, the geometry input into the calculation is determined using the near and far edges of the school along each section.

The ΔH for each section should be taken as the maximum of the two parallel displacement estimates. ΔH estimates should be presented to the structural engineer using zonal plans, with zone extents interpreted from site topography and school shape. An example ΔH plan for an inland floodplain school site is provided in Figure 9.

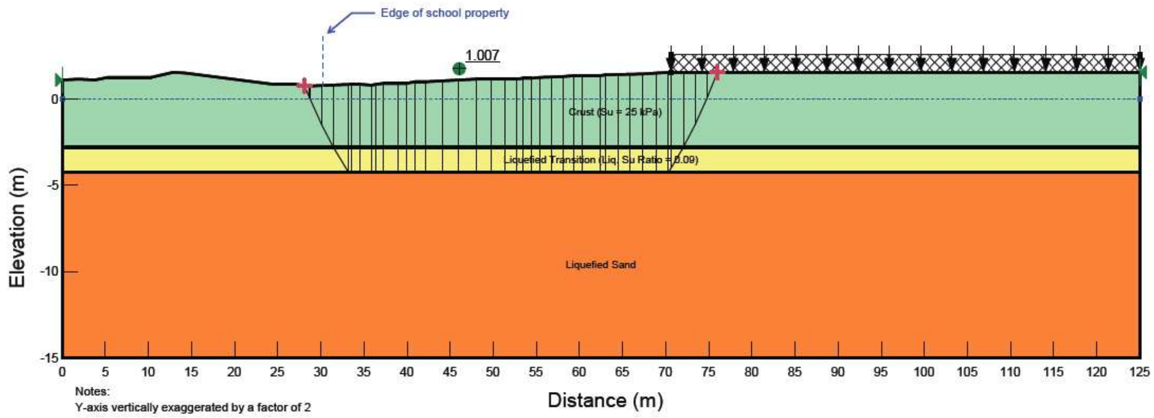

Figure 3. Example Limit Equilibrium Analysis to Determine Yield Acceleration of the Governing Slip Surface

Vertical Settlement

The Guidelines follow the approach of Idriss and Boulanger (2008) [8] for estimation of post-earthquake vertical settlements. The method involves dividing the soil column into discrete layers, calculating the maximum volumetric strain due to reconsolidation for each layer, and summing up the volumetric strains for layers where liquefaction has been triggered. The Guidelines also recommend the use of the procedures in Cetin et al. (2009) [9] that include a depth-weighting factor that limits contributions to surface settlement from layers located below 18 m depth.

For alluvial fan sites and near shore floodplain sites, the differential vertical movement (ΔV) should be determined based on engineering judgement. ΔV should not be less than 50% of the estimated total post-liquefaction settlement. For inland floodplain sites, crust attenuation is more likely, and a slope may be provided in place of a ΔV estimate. For a typical inland Richmond site, a slope of 60H:1V may be used. This post-liquefaction differential settlement estimate for inland Richmond sites was developed considering the attenuation provided by a 3 m thick silt crust with an undrained strength of 30 kPa, which is the typical minimum thickness / strength of the non-liquefiable crust based on site investigations completed at 36 Richmond schools.

Other Consequences

Volume 11 of the Guidelines also provides guidance for assessing punching shear and liquefaction effects on pile foundation. For the punching shear check, loading provided by the structural engineer is compared with the strength of the crust, assuming the loading gets divided across two vertical shear planes in the case of a strip footing. The punching shear assessment is visualized in Figure 4. Contributions of piles to resistance are ignored and a gap of 50 mm is assumed to be present between the pile and pile cap, unless otherwise confirmed through direct visual observations. Closure of this gap is assumed during the earthquake and is added to the ΔV.

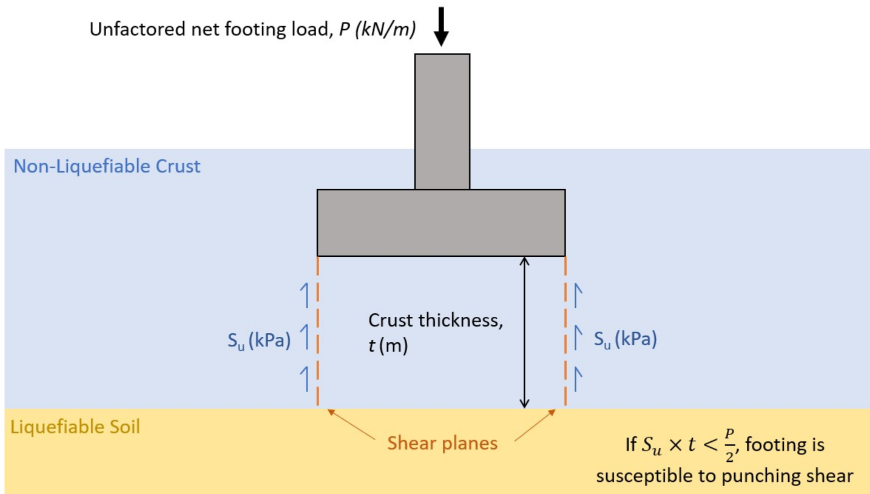

Figure 4. Illustration of Punching Shear Assessment

STRUCTURAL CONSIDERATIONS

General

The evaluation of the effects of liquefaction on a structure is based on the following considerations:

- During strong shaking, the soil has not liquefied, and the demand for a site with no degradation of soil strength or stiffness is considered. In this phase, the structure must be assessed or retrofitted (as necessary) as if liquefaction is not an issue.
- After strong shaking, on a liquefiable site, the soil liquefies and induces new and different demands on the structure. In this phase, the structure must be assessed or retrofitted (as necessary) to accommodate the liquefaction effects, starting in a partially damaged state with residual drift.

In these guidelines the above two effects are considered to not be concurrent, thus the structure must be assessed and retrofitted as necessary to accommodate both effects.

Required Information

The following key structural and geotechnical information is required for the liquefaction assessment:

<u>*Structural:*</u>

- Reasonable investigation to confirm the extent, depth, and size of the existing foundations
- Locations of Lateral Deformation Resisting System (LDRS) and Vertical Load-bearing Support (VLS) and their drift limits
- Capacity of the connections for floor and roof framings

<u>*Geotechnical:*</u>

- Lateral spreading effects (ΔH), and differential vertical settlement (ΔV)
- Fundamental soil parameters such as: punching shear capacity of non-liquefiable soil layer, friction coefficient between soil and footings and between soil layers, passive pressures.

Structural Assessment

Two structural assessment procedures for liquefaction are outlined in the guidelines: Simplified Assessment procedure and Detailed Assessment procedure. The focus in this paper is the Simplified Assessment.

In the Simplified Assessment procedure, the following two conditions are considered:

<u>*Condition 1:*</u> Near shore sites where large lateral spread (ΔH) and differential vertical settlement (ΔV) are expected, and the geotechnical engineer will provide a single value for each of ΔH and ΔV.

<u>*Condition 2:*</u> Inland sites where the non-liquefiable crust is greater than 3m below base of footing and where it is expected that lateral spread and differential vertical settlement to be relatively small. For the lateral soil spreading effect the geotechnical engineer will provide a plan of the school, separated into zones, with each zone showing ΔH in each orthogonal direction. The differential vertical settlement, ΔV, is to be calculated assuming a 1:60 slope between the base of disconnected footings unless the geotechnical engineer determines otherwise.

Typical Foundations

Most foundations are located in a non-liquefiable crust located over a liquefiable layer, as illustrated in (a) and (b) in Figure 5 or located over a zone of a combination of liquefiable and non-liquefiable layers.

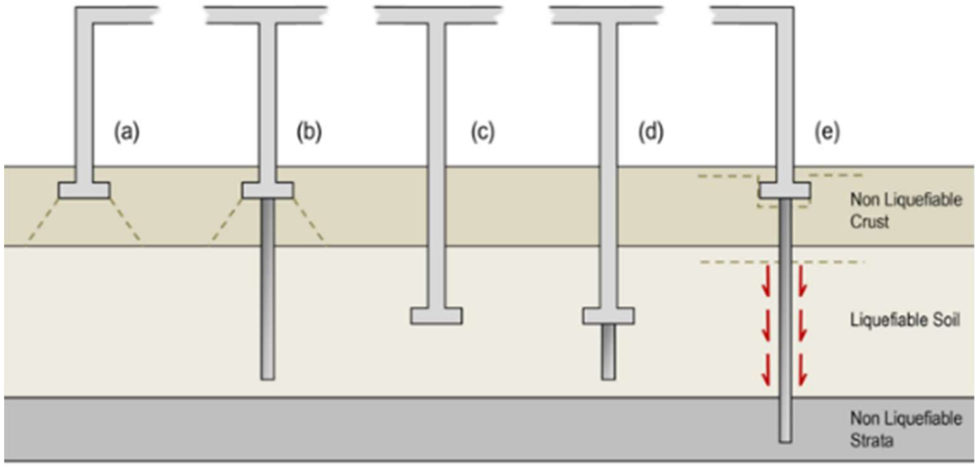

Figure 5. Foundations Located in Crust or in Liquefiable Soil

During liquefaction, it is to be assumed that the liquefiable layer offers near-zero vertical resistance (for structural elements in that layer). The vertical resistance is provided by the foundation (or pile caps and grade beams) in the non-liquefiable crust or by piles extending below the liquefiable layer.

The potential for punching shear failure of the foundations through the crust must be assessed. If the punching shear capacity is inadequate, the structure shall be rated High risk.

If the footings are founded in the liquefiable layer as illustrated in conditions (c) and (d) in Figure 5, the structure shall be rated as High risk. If the piles of a piled foundation extend through the liquefiable layer(s) to a firm non liquefiable strata, per (e) in Figure 5, the pile capacity shall be checked including down drag effects. If the pile capacity is inadequate for the noted loading condition, the structure shall be rated High risk.

Allowable Structure Drifts Due to Liquefaction Effects

Due to strong shaking prior to liquefaction, the residual drift (RD) in the structure (due to inelastic response and some damage) shall be assumed to be 20% of the design Collapse Prevention Drift Limit (CDL) of the LDRS in each principal direction.

Liquefaction Drift Limit (LDL) shall accommodate the following three components:

- Residual Drift (RD)
- Effective Drift demand due to lateral soil (horizontal) spreading effects (EDH)
- Effective Drift demand due to differential vertical soil settlement effects (EDV)

Thus $RD + EDH + EDV < LDL$ for the structure to be Low risk, otherwise it is High risk.

The above drift components should be calculated for the LDRS and VLS separately and checked against their design CDL limits. The guidelines provide directions on how to calculate EDH and EDV for various conditions of continuous footings and/or individual non-connected footings, and various LDRS and VLS.

Liquefaction Assessment for Condition 1

Lateral Soil Spreading Effects

Lateral soil spreading effect, ΔH, due to liquefaction shall be determined on all sides of a building, as shown in Figure 6.

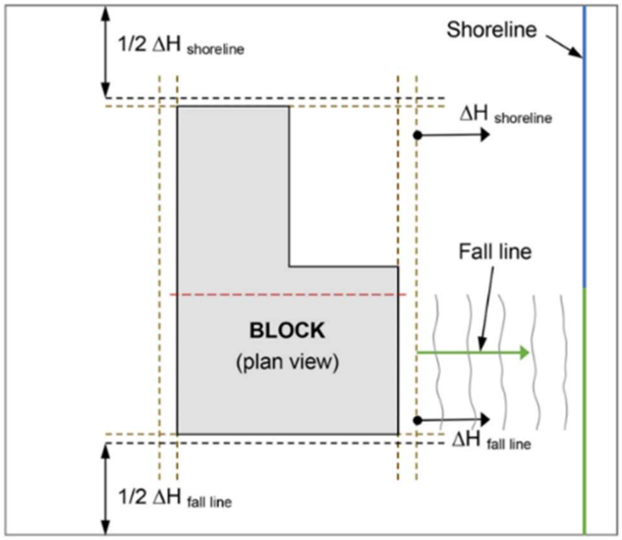

Figure 6. Lateral Soil Spreading Effects for Shoreline and Fall Line

For the purposes of the assessment, it is assumed that the crust can 'rupture' and can cause a movement of ΔH between any non-interconnected foundations. The ΔH movement is in the direction perpendicular to and towards the shoreline or parallel to the fall line. The movement in the direction parallel to the shoreline or perpendicular to the fall line shall be assumed to be 50% of the perpendicular direction. This "rupture" is illustrated in Figure 7.

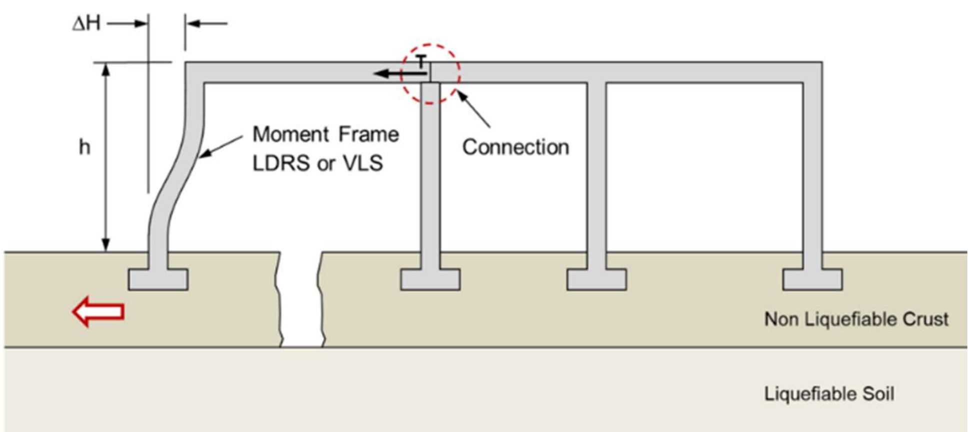

Figure 7. Lateral Soil Spreading Effects for Condition 1: Moment Frame LDRS and VLS

The resulting drift demand EDH is ΔH/h (where h is the floor height) in percent. This is one of the three drift items discussed previously and shall be calculated for each principal direction.

<u>*Differential Vertical Settlement Effects*</u>

The differential vertical settlement due to liquefaction, ΔV, within the perimeter of a given building shall be determined by the geotechnical engineer. This shall be one value used for the entire structure. This is illustrated in Figure 8 below.

For the purposes of assessment, it is assumed that the crust can 'shear' to accommodate the uneven settlements in the liquefiable soil layer and cause a movement of ΔV between any foundations non-interconnected or connected by an element without adequate capacity to accommodate the differential settlement, or bay lines, or VLS in the structure.

The resulting effective drift demand EDV is ΔV/L (where L is the span) in percent. This is one of the three drift terms discussed previously and shall be calculated for each principal direction.

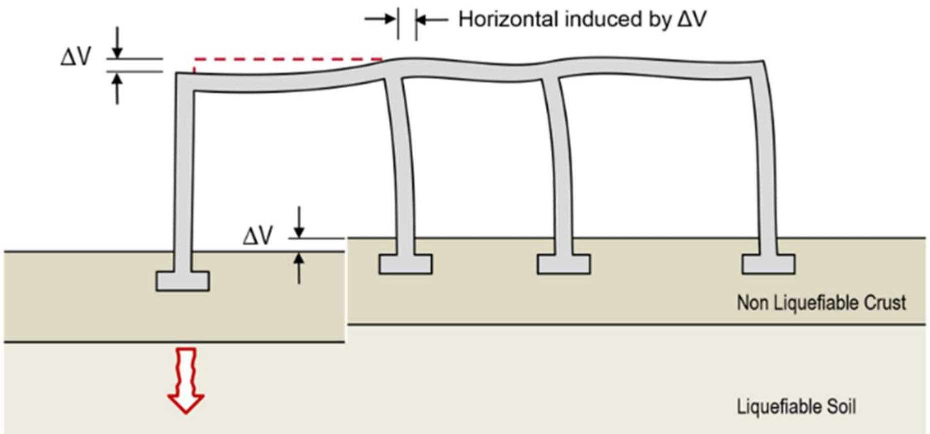

Figure 8. Differential Vertical Settlement Effects for Condition 1: Moment Frame LDRS and VLS

Liquefaction Assessment for Condition 2

Lateral Soil Spreading Effects

The geotechnical engineer will provide a plan of the building separated into zones (black boxes in Figure 9) and each of these zones will have ΔH provided in each orthogonal direction. At boundaries between the zones, the maximum ΔH of each adjacent zone in each direction is to be used. Also, at areas of discontinuous footings between adjacent buildings, the structural engineer should request refined ΔH estimate from the geotechnical engineer.

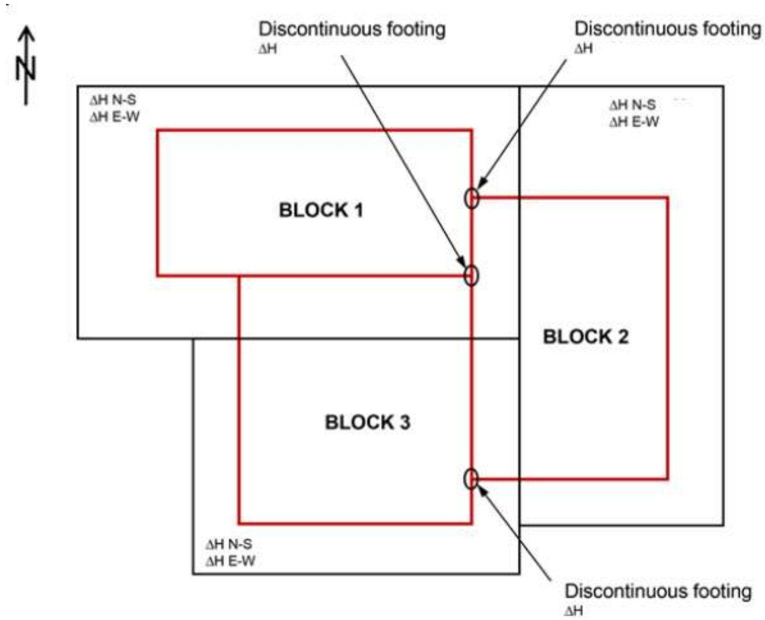

Figure 9. Lateral Soil Spreading Information for Condition 2

For the purposes of assessment, it is assumed that the crust does not ‘rupture’ but strains, however, the effects of ΔH are similar to those illustrated in Figure 7.

Differential Vertical Settlement Effects

For Condition 2, it is recommended that differential vertical settlement be calculated assuming a 1:60 slope between the base of disconnected footings unless the geotechnical engineer determines otherwise. Thus, EDV is 1.7%, except for certain conditions explained in the guidelines (such as ‘pinned’ connections of VLS, where EDV is zero).

Examples

Several numerical examples are provided below for Condition 1 and 2, with various LDRS and VLS.

Condition 1:

The structure consists of a concrete moment frame on non-connected footings, with steel columns supporting the gravity load elsewhere, also on non-connected footings.

LDRS: Concrete Moment Frame on spread footings; CDL: 3.0%
VLS: Steel Columns on spread footings; CDL: 6.0%
Differential Horizontal Movement (ΔH): 200 mm
Differential Vertical Movement (ΔV): 150 mm
Floor Height (h): 4000 mm
Bay Width (L1): 8000 mm

CDL of LDRS (thus the LDL)	3.0%
Residual Drift @ 20% of CDL of LDRS	0.6%
EDH LDRS: 200 mm / 4000 mm	5.0%
EDV LDRS: 150 mm / 8000 mm	1.9%
Total Drift LDRS: 0.6+5.0+1.9	7.5% > 3.0% as such building is High risk, and remediation is required

CDL of VLS (Thus the LDL)	6.0%
Residual Drift @ 20% of CDL of LDRS	0.6%
EDH VLS: 200 mm / 4000 mm	5.0%
EDV VLS (pinned)	0%
Total Drift VLS: 0.6+5.0+0	5.6% < 6.0%

Even though the drift for the VLS did not exceed the LDL, the building is considered High risk due the drift of the LDRS, and remediation is required.

Condition 2:

The structure is a single storey wood frame school building with 3 blocks. ΔH in the north-south direction is 30mm.

LDRS: Wood Shear Wall; CDL: 4.0%
VLS: Wood Columns on spread footings; CDL: 6%
Differential Horizontal Movement (ΔH): 30 mm
Bay Width (L1) and Foundation Length (Lf): 4000 mm
Differential Vertical Movement (ΔV): 66 mm (based on 1:60 ratio)
Floor Height (h): 4200 mm

CDL of LDRS (thus the LDL)	4.0%
Residual Drift @ 20% of CDL of LDRS	0.8%
EDH LDRS: 30 mm / 4200 mm	0.7%
EDV LDRS: 66 mm / 4000 mm	1.7%
Total Drift LDRS: 0.8+0.7+1.7	3.2% < 4.0% as such no remediation is required

CDL of VLS (thus the LDL)	6.0%
Residual Drift @ 20% of CDL of LDRS	0.8%
EDH VLS: 30 mm / 4200 mm	0.7%
EDV VLS (pinned)	0%
Total Drift VLS: 0.8+0.7+0	1.5% < 6.0% as such no remediation is required

Since the drifts for the LDRS and VLS did not exceed their LDL limit, no remediation is required.

Friction and Soil Pressure Loading on Foundation Elements

Lateral spreading may induce lateral loading in existing tension ties between footings or along strip footings. Figure 10 illustrates a scenario where the crust may ‘rupture’ and ‘slide’ under the footing inducing a friction load on the underside of the footing that can then be transferred into the tension tie (existing or new retrofit item) or other structural components in the foundation. This load can be supplemented by passive pressure of the soil being compressed against the footing integral with the sliding crust, adding further tension loading into the tie. Such scenarios shall be assessed for the crust rupturing at any location within the boundary of a structure, transferring lateral load to various foundation components. The passive pressure loading to be included only if ΔH is greater than 2% of the depth to the underside of the footing.

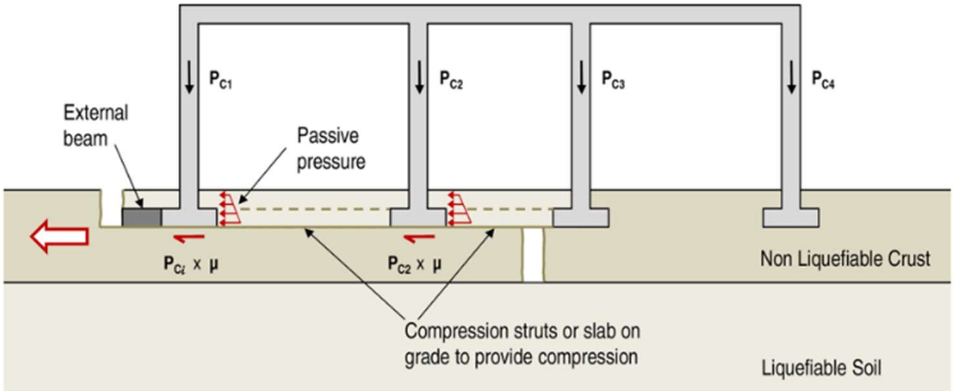

Figure 10. Possible Friction and Soil Pressure Loads

Additional Checks for Pile Foundations

For piles located in the liquefiable layer with piles caps in the crust, the following assumptions are made:

- The piles do not contribute axial, lateral, or punching shear resistance; thus, the pile caps and grade beams are the only elements providing vertical support
- There is a vertical displacement due to closure of the gap below the pile caps that unless field verification is completed, is assumed to be 50 mm. This gap thickness is to be added to the differential vertical movement estimate provided by the geotechnical engineer for Condition 1 or 1:60 slope for Condition 2.

Detailed Assessment

Structures classified as High risk based on the Simplified assessment method shall be remediated or may be re-evaluated employing a more detailed assessment that consists of modelling the interaction between the structure and the liquefied soil. Such a non-linear analysis will be beneficial if the geotechnical/structural engineer believes that it may eliminate the need for the retrofit or contribute to optimize the extent of the retrofit work involved.

Certain Timber Framed Structures, Requiring No Remediation

The guidelines outline conditions for certain one or two storey, primarily timber framed buildings, with only wood LDRS's and with VLS with a CDL of 6% for which no remediation is required. Basic conditions of no punching shear failure of footings and LDRS's have adequate capability to accommodate ΔH and ΔV must be met.

REMEDIAL OPTIONS

General

Any remedial work required is based on providing life safety and prevention of collapse, so that an affected facility can be safely evacuated in the event of a major earthquake. Structures may or may not be re-usable after the event. The options for mitigation of liquefaction and its consequences for an existing building will generally fall into two categories: soil remediation, or structural modifications.

Geotechnical Based Remediation

There are a wide variety of techniques for ground improvement that can be used to mitigate the effects of liquefaction. However, such techniques are disruptive for an existing facility and usually quite expensive, so that the preferred methods are structural.

Structural Based Remediation

There are a wide variety of techniques for structural remediation that can be used to mitigate the effects of liquefaction. For inadequate punching shear capacity these include: enlarging footings; adding mini piles between existing footings with a new grade beam; and creating a complete raft foundation interconnecting all existing footings. For excessive lateral spread effects these include: interconnecting all footings with new tension ties; creating an external ring beam to minimize differential movement between footings; creating a raft foundation interconnecting all footings; designing an LDRS with larger CDL; and providing new supplementary VLS with larger CDL.

Figure 11 includes photos from the structural upgrade of a school located on a liquefiable site. The structural mitigation included a combination of reinforced concrete external ring beams, beams within the building connecting to the external beams, and ties to footings where appropriate. This combination of new elements has proven to be very cost effective and was able to be constructed without disruption to the teaching program at the school.

(a) *(b)*

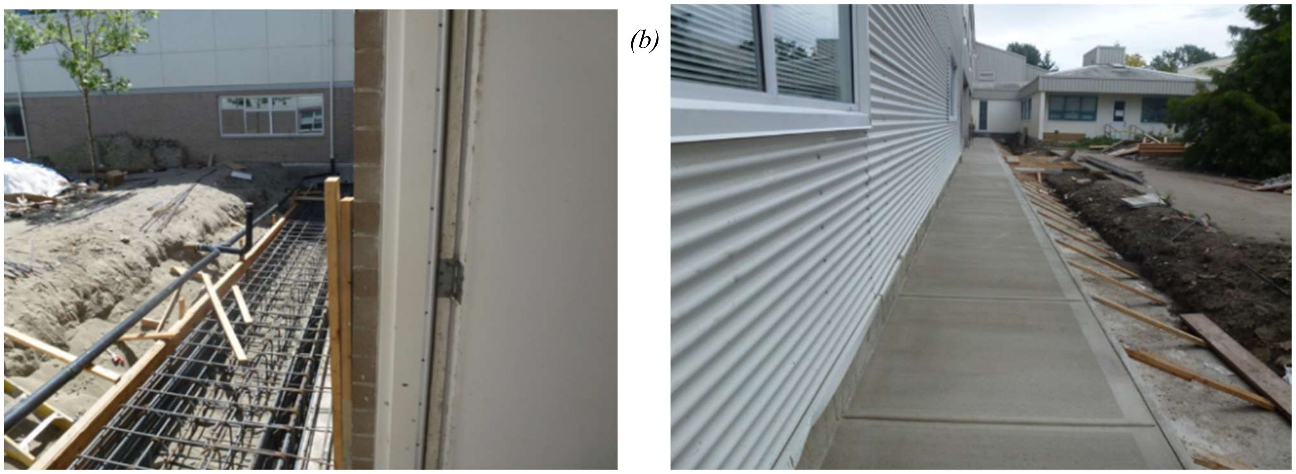

Figure 11. External Ring Beam (a) During Construction, (b) as Sidewalk When Completed

CONCLUSIONS

The liquefaction guidelines in SRG 2020 are a significant advancement over the previous version of SRG as the current guidelines now differentiates between significant liquefaction effects near shorelines versus minor liquefaction effects for most inland sites. This has resulted in a significant reduction in the number of school buildings requiring mitigation for liquefaction effects, and a reduction in extent of mitigation for those buildings deemed High risk for damage due to liquefaction induced soil deformations. The guidelines continue to provide a simple and consistent approach to address liquefaction effects.

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