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CHALLENGES FACED IN IMPLEMENTING THE NEW SEISMIC DESIGN GUIDELINES FOR DIKES IN SOUTH WESTERN B.C., CANADA

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Abstract

This paper presents an overview of the basis and methodologies proposed for seismic design of High Consequence Dikes in Southwestern British Columbia, Canada and challenges faced during implementing the Guidelines for two local dikes constructed over a thick deposit of liquefiable sands. The Guidelines adopt a combination of traditional and performance-based design criteria for the seismic design of dikes. Dike seismic performance is specified in terms of measureable indicators such as crest displacement and settlement of the dike structure. The methodologies and criteria were established following a review of practices currently followed in other regions of the world that are also prone to high seismic hazards.

Keywords: Seismic Design Guidelines; Performance-Based Design; Liquefaction; High Consequences Dikes.

1. Introduction

Densely populated urban communities and regional infrastructure in British Columbia, Canada, are protected from flooding by close to 500 km of river and sea dikes. These dikes comprise earthen embankments of varying heights and dimensions, constructed over varying foundation soils using different construction practices and materials. The stability of the dike systems and their performance under the loads imposed by natural hazards such as floods, storms, and earthquakes is of paramount importance in protecting both rural and urban communities and regional infrastructure. The protection offered by the dike system is dependent on the performance of the weakest areas of the dike system under consideration.

British Columbia has experienced three historical devastating floods that occurred in 1894, 1948, and 1972 resulting from the annual spring snowmelt freshets of the Fraser River system. Spring snowmelt freshets pose the main flood hazard in the floodplain areas. Autumn or winter rainfall flooding can also occur as well as some inundation resulting from channel obstructions due to ice jam formations. These latter processes are considered to be less destructive than the former.

It is reported that the 1948 Fraser River Flood caused an estimated $20 M (in 1998 dollars) of damage and ten deaths. The damage from the 1972 flood is estimated at $37 M with the predominantly affected areas located primarily in the upstream communities of Prince George and the Oak Hills area of Kamloops, and the Surrey area of the lower Fraser Valley. The extent of damage to the existing urban communities and regional infrastructure due to large scale flooding resulting from breaches in the different diking systems has been estimated to reach upwards of C$50 Billon (in 2013 dollars).

The oceanic water levels inclusive of effects of storms, tides, and tsunamis caused by local submarine slumps and distant subduction earthquakes pose the main flood hazard. Moderate size earthquakes have not occurred in the region in the recent past, following construction of the dike systems.

As part of the Flood Hazard Management program, The Flood Safety Section of the BC Ministry of Forests Lands and Natural Resources Operations (MFLNRO) is committed to providing comprehensive and up to date policies and guidelines regarding dike design and flood construction levels with the goal of reducing or preventing injury, loss of life and property damage from flood events [1]. MFLNRO provides guidance with respect to dike safety and flood hazard area land use management, and supports other provincial and local government agencies in flood emergency management through the Dike Maintenance Act (DMA) and Environmental Management Act.

Dike design in British Columbia has historically been based on hydraulic criteria to prevent failure by static instability, overtopping and/or piping. Dike failures resulting from seismic activity has not been considered in detail, generally due to economic drivers and limited knowledge of seismic performance and appreciation. Significant portions of British Columbia are situated in a seismically active zone where there is significant potential for extensive damage to dike systems from seismic events. To address this risk, the Flood Safety Section undertook the development of seismic design guidelines for dikes, which are a condition of the DMA approval process. The Guidelines are applicable for High Consequence dikes where the consequences of dike failure are high, and include design and construction of new dikes or upgrading of existing dikes. The different dike systems under consideration are shown on Fig. 1.

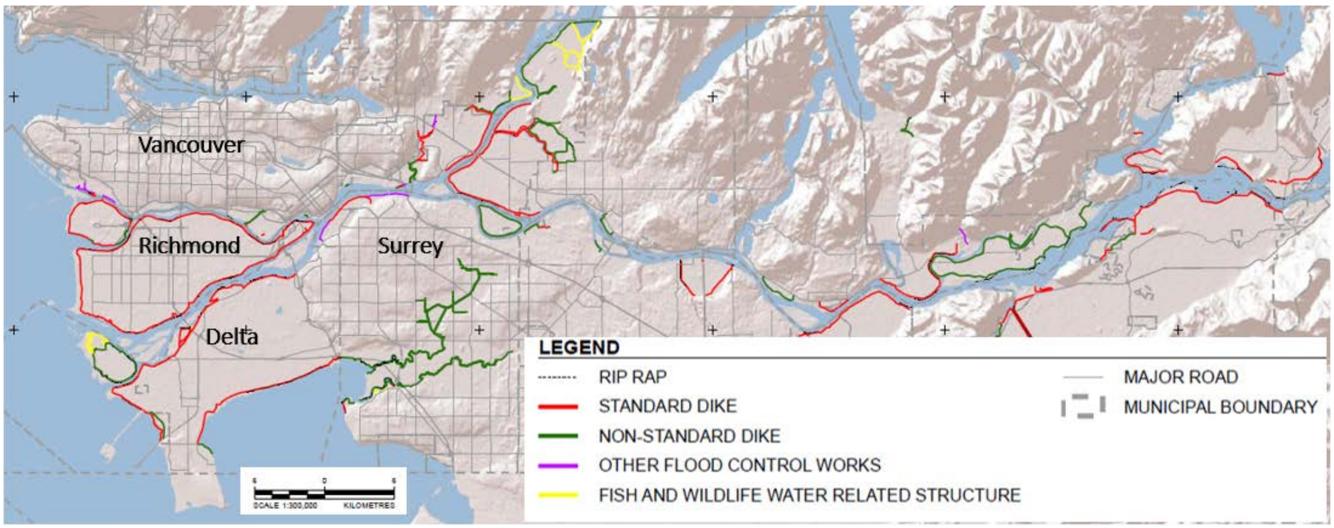


Fig. 1 – The System of Dikes in Southwestern British Columbia

2. Guidelines Synopsis

The performance-based seismic design guidelines developed for High Consequences Dikes address the following:

- Ground motions to be considered in the analysis and design of dikes along with corresponding performance expectations;
- Suitable geotechnical investigation methods to characterize and obtain engineering properties of site soils;
- Commonly used methods for seismic analysis considered appropriate for dikes;
- Threshold seismic events that trigger a post-event evaluation of the integrity of the dike system;
- Seismic rehabilitation and strengthening measures; and
- Post-earthquake temporary emergency repair and permanent remediation measures.

For details, the reader should refer to the Guidelines document that can be downloaded from http://www.env.gov.bc.ca/wsd/public_safety/flood/fhm-2012/draw_report.html.

As part of the work carried out for developing the Guidelines, the available published seismic design requirements/guidelines established for dikes and levees in other jurisdictions were reviewed in order to provide insight on seismic design guidelines "currently" adopted by the profession. The following three broad trends in the overall analysis procedures were apparent:

1. In general, dikes are designed using low hazard probabilities when considering high consequence circumstances. Historically, this has been achieved implicitly through traditional design criteria by

prescribing factors of safety against failure and considering conservatively estimated loads and capacities. These traditional deterministic criteria have evolved over time to achieve dike designs with acceptable risks.

2. More recently, seismic design of dikes has evolved to include performance-based design criteria considering more than one level of ground shaking and by specifying the acceptable performance for each level of shaking.
3. A more comprehensive approach that has evolved, but is at the initial stages, is the design of earth structures such as dams through risk assessment and management by specifying the probability of failure or reliability of particular components with respect to various functions. This latter approach involves an assessment of the societal risk and considers many other factors such as loss of life, impact to the environment and cultural values, and impact to infrastructure and economics.

The Guidelines developed adopted a combination of traditional deterministic and performance-based design criteria for the seismic design of dikes. The required performance of dikes is specified in terms of measureable criteria such as displacements of the dike crest as a result of seismic loading. Satisfactory dike performance is implicitly taken into consideration by specifying multiple levels of earthquake shaking and corresponding performance expectations that can be varied to achieve a high or low degree of safety/reliability.

The Guidelines are not intended to explicitly consider probability of dike failure and/or level of post-earthquake flood protection. Consideration of combined probabilities and level of post-earthquake flood hazard protection must be developed on a regional dike network level basis, which is outside the scope of the current work. A regional dike-network-level risk framework is under consideration for the Fraser River in the Lower Mainland and Fraser Valley by the Fraser Basin Council.

It is expected that from time to time the Guidelines will be expanded and/or modified in response to feedback from planners and practitioners. Designers are to use their own judgment in interpreting and applying the Guidelines contained in the document.

3. Application of Guidelines to Highly Vulnerable Sites

Some of the dikes are located in sites that are highly vulnerable to damage from seismic liquefaction during and/or following an earthquake. Seismic strengthening and remediation of these dikes using ground improvement techniques are costly and may only be practical for short sections of dikes and appurtenant structures. For dike segments where the performance criteria cannot be met, provision can be made to:

- Re-aligning the dike;
- Overbuilding the dike to the extent possible and practical to satisfy post-earthquake vertical displacement requirements provided that displacement analyses confirm that the dike core will retain its hydraulic integrity and the landside face geometry remains intact;
- Incorporating the “dike” into massive fills required for adjacent land development (i.e. the “superdike” concept) again with sufficient analyses to confirm that the flood protection system would retain its hydraulic integrity; and
- Documenting the expected damage, developing a remediation plan, restricting land use and regulating floodplain development in the protected area to justify removal of the High Consequence Dike classification.

As seismic design requirements may impact dike alignment and land acquisition requirements, it is prudent that pre-feasibility geotechnical studies, including the seismic assessment, be completed prior to detailed civil design of the dike.

4. Seismic Design Objectives for Dikes

The seismic design objectives for dikes are as follows:

- Dikes subjected to seismic ground motions with a short return period (or a high annual exceedance probability) event during the design life should perform with insignificant damage to the dike body, without compromising the post-earthquake flood protection ability;
- Dikes subjected to seismic ground motions with an intermediate return period (or an intermediate annual exceedance probability) event during the design life may experience some repairable damage to the dike body, without compromising the post-earthquake flood protection ability; and
- Dikes subjected to seismic ground motions with a long return period (or a low annual exceedance probability) event during the design life may undergo significant damage to the dike body potentially requiring more complex subsurface repairs, with the short-term post-earthquake flood protection ability possibly compromised.

Typical return periods considered are summarized in Table 1:

Table 1 – Typical Return Periods and Event Classifications

Return Period Classification	Return Period (Years)	Event Classification
Short	100 to 200	Frequent
Intermediate	475 to 975	Intermediate
Long	2,475 to 10,000	Rare

5. Seismic Hazards Considered

Potential seismic hazards affecting the dikes include the following:

- Ground shaking;
- Slope movements caused by ground shaking;
- Bearing capacity and sliding failure;
- Soil liquefaction and flow slide failure;
- Vertical and horizontal total and differential ground displacements;
- Loss of free board due to ground subsidence and slope failure; and
- Piping failure through fissures induced by ground movements.

The seismic hazard to Southwestern British Columbia results from the offshore subducting of the Juan de Fuca Plate beneath the Continental Plate. This tectonic environment gives rise to three different types of earthquakes, each with its own specific characteristics; i.e.: shallow crustal earthquakes (up to M_w7.5, with epicenters as close as a few km), deep inslab earthquakes (up to M_w7.5, with epicenters as close as 40 km), and interface or subduction earthquakes (up to M_w9, with the epicenters as close as about 110 km).

In order to avoid unrealistically low combined probabilities, the “mean annual river water” and “mean annual sea water” levels are considered in the seismic assessment of dikes. However, in some instances (e.g. for sea dikes), the sensitivity to varying water levels should be considered. In addition, future dike upgrades will need to consider the projected future sea level changes.

The designers are recommended to also refer to the Ministry of Environment, Water Management Branch report entitled, Climate Change Adaptation Guidelines of Sea Dikes and Coastal Flood Hazard Land Use (3 Volumes), dated January 2011.

6. Geotechnical Investigations for Dike Design and Analysis

Flood protection dikes are almost always located along river banks and shorelines. Historically, river banks and shorelines have experienced considerable damage following earthquakes due to soil liquefaction, slope failure, settlement, and permanent lateral movement. As a result, dikes have a high geo-hazard exposure and need to be investigated in detail to allow identification and assessment of soil conditions and strata that are vulnerable to liquefaction, loss of shear strength, and movement.

The main objective of a geotechnical investigation is to identify soil strata that are susceptible to liquefaction and/or cyclic softening as a result of strong ground shaking, to determine their in-situ state and engineering properties. A suitable investigation should include, but may not be limited to, the following:

- Continuous or near-continuous profiles of the soil strata;
- Measurement of depth to ground water levels on either side of and within the dike;
- In-situ testing of soil strata susceptible to liquefaction or cyclic mobility in the form of penetration resistance, strength, and shear wave velocity;
- Sampling of soil strata susceptible to liquefaction or cyclic mobility;
- Gradation of soils susceptible to liquefaction or cyclic mobility;
- Index testing of soils susceptible to liquefaction or cyclic mobility; and
- Cyclic simple shear testing of fine-grained soils to investigate liquefaction susceptibility or cyclic mobility.

Dikes comprise hundreds of kilometers of earth fill embankments constructed over varying ground conditions that may include reclaimed areas, buried channels, previous failures, river meanders, bar deposits, and marshy/swampy areas. The flood protection offered by the dike system is dependent on the performance of the weakest areas of the specific dike system, and this aspect should be taken into consideration when planning the field investigations.

Several different field investigation methods are commonly used by the practitioners to obtain engineering properties of soils. These include the Standard Penetration Testing (SPT), the Cone Penetration Testing (CPT), Becker Penetration Testing (BPT) and Shear Wave Velocity Testing (SWVT) methods.

Other field exploration and in-situ testing methods for assessment of soil liquefaction may be used provided that site-specific correlations have been developed with one of the methods described above. The soil liquefaction susceptibility map shown in Fig. 2 was prepared [3] to assist the practitioners and planners with initial screening level evaluations.

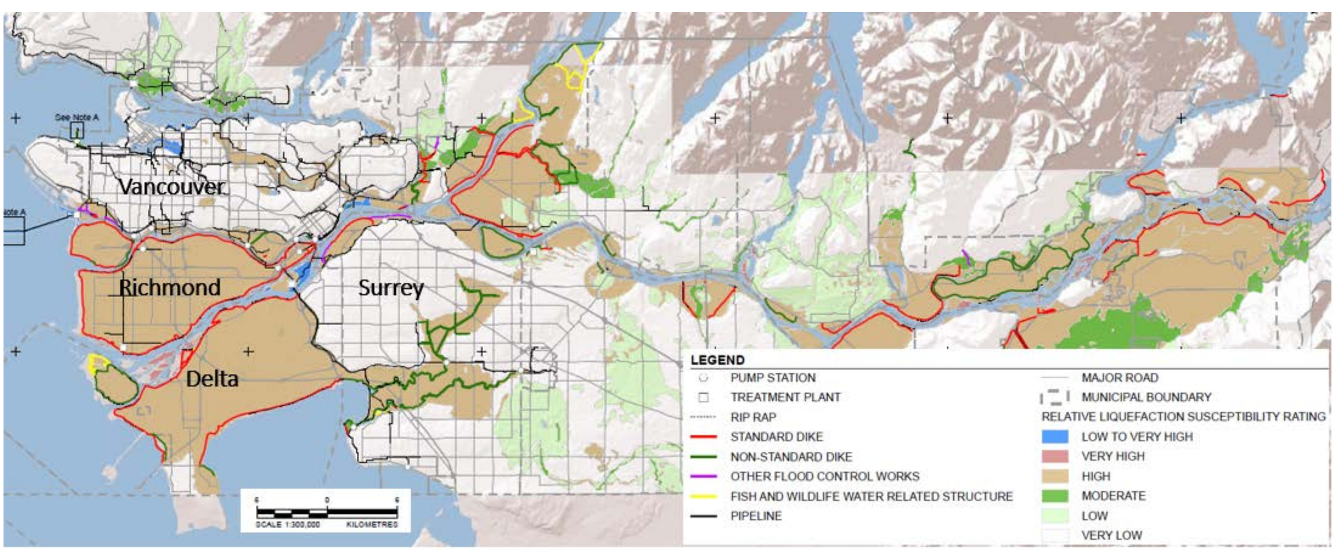


Fig. 2 – Soil Liquefaction Susceptibility Map

The Guidelines require that a minimum of three borings drilled for each section of the dike; one on the water side of the dike, one through the center of the dike, and one on the land side of the dike, with the horizontal spacing of data sections along the dike not exceeding 300 m. Closer spacing of data sections may be required where significant variations in subsurface conditions are anticipated. Drilling boreholes over water is costly, but establishing reliable soil stratigraphy and parameters is important in the dike stability analyses.

For dike segments where the initially available subsurface data is limited, the analyses and investigations may be carried out in stages, starting with screening level analyses/investigations. However, the final design and analysis of the dike segment need to incorporate subsurface investigations as identified above.

7. Performance-Based Design Criteria

A performance-based seismic design is accomplished by defining appropriate levels of design earthquake shaking corresponding acceptable levels of damage. The design earthquake motions include those from frequent events that are likely to occur within the life of the dike as well as infrequent or rare events that typically involve very strong ground shaking.

The acceptable levels of damage are specified in terms of displacements to be experienced by the dike system. Damage is categorized in terms of “Performance Categories”, which are related to the effort required to restore the full functionality of the dike system.

The performance of the dike system should be checked for all three Design Earthquake Ground Motion Levels defined below:

1. Design Earthquake Ground Motions

Ground motions that correspond to three different return periods are considered in seismic design:

- Earthquake Shaking Level 1 (EQL-1) equivalent to ground motions with a 100-year return period
- Earthquake Shaking Level 2 (EQL-2) equivalent to ground motions with a 475-year return period
- Earthquake Shaking Level 3 (EQL-3) equivalent to ground motions with a 2,475-year return period

2. Performance Categories and Permissible Displacements

Performance Category A: No significant damage to the dike body, post-seismic flood protection ability is not compromised.
Performance Category B: Some repairable damage to the dike body, post-seismic flood protection ability is not compromised.
Performance Category C: Significant damage to the dike body, post-seismic flood protection ability is possibly compromised.

The maximum allowable dike displacements to achieve the desired performance are provided in Table-2.

Table 2 – Summary of Maximum Dike Crest Displacements Corresponding to Performance Categories

Performance Category	**EQ Shaking Level**	**Maximum Vertical Crest Displacement**	**Maximum Horizontal Crest Displacement**
A	EQL-1	< 0.03 m	< 0.03 m
B	EQL-2	0.15 m	0.3 m
C	EQL-3	0.5 m	0.9 m

The maximum allowable displacements given in Table 2 have been established with the intent of preserving the structural integrity of the dike body. They represent total displacements. It is implied that for earthen dikes, satisfying the maximum allowable dike crest displacements at sections that are located a maximum horizontal distance of 300 m along the dike would reduce the hazards associated with a dike breach as a result of differential or relative displacements.

The design of structural elements such as floodwalls may need to satisfy alternate (less tolerant) displacement criteria in order to achieve the performance expectations described herein.

The designer has to independently confirm that the displaced configuration of the diking system would provide at least 0.3 m of post-earthquake freeboard above 1:10-yr return period water level to meet performance expectations. Individual communities that are assessed as having high economic loss and damage to environment as a result of flooding may impose more stringent minimum post-earthquake freeboard than specified herein.

8. Analysis Methods

The assessment of seismic hazards on dikes involves several steps:

Step-1: Evaluate applicable ground surface acceleration, crest acceleration, and accelerations at selected locations of the dike;

Step-2: Evaluate liquefaction potential of soil;

Step-3: Evaluate stability of slopes under seismic loads, including post-earthquake flow-slide failure;

Step-4: Evaluate seismic displacements; and

Step-5: Evaluate post-event piping failure potential.

Steps 1 through 4 may be carried out using either simplified (i.e. Newmark [2]) or finite difference/finite element methods of analyses, as the situation may warrant, with the realization that simplified methods provide limited information in comparison to rigorous methods. Step-5 involves an assessment of post-event field inspection observations and does not require specific analyses.

In design, soil-structure interaction analysis may need to be carried out to address relative displacements (and performance) of appurtenant structures such as flood boxes and pump stations and the adjacent earthen dike structure. Appurtenant structures may also include residential/commercial developments built into the dike body where consideration of seismic earth pressures affecting the seismic performance of any below grade walls must be explicitly considered, particularly the potential for cracking of concrete.

During earthquake shaking, the earthen dike mass may or may not move relative to the adjacent more rigid appurtenant structure(s) depending on the ground conditions and foundation elements that support appurtenant structures. In situations where relative displacements are expected to occur between the appurtenant structure(s) and the adjacent earthen dike mass, appropriate design elements (e.g., flexible wing walls, “water stops” or similar technology) should be incorporated to prevent leakage/soil loss at these interfaces.

Guidelines on the appropriate methods of analyses for Steps-1 through 4 are provided below and summarized in Table-3:

EQL-1 (100-yr Return Period):

Slope stability based on pseudo-static analysis method. Displacements based on Newmark analysis method.

EQL-2 & EQL-3 (475-yr Return Period and 1:2,475-yr Return Period):

The types of analyses required are dependent on the Liquefaction Index, *Li*, established based on Seed’s Simplified Method of Analysis (SSMA).

Li is defined as follows:

L0: No liquefaction, no significant excess pore water pressures ($R_u \leq 20\%$);
L1: Complete liquefaction not expected (i.e., $FOS_{liq} > 1.2$), limited excess pore water pressures ($R_u \leq 50\%$);
L2: Liquefaction occurs in zones of limited thickness; and

L3: Complete liquefaction of soils.

Table 3 – Analysis Methodologies for Varying Liquefaction Indices

Liquefaction Index	Slope Stability	Displacements
Insignificant (*L0*)	Pseudo-Static	Newmark[1]
Mild (*L1*)	Pseudo-Static (Reduced Shear Strength)	Newmark[1] (Reduced Shear Strength)
Moderate (*L2*)	Pseudo-Static (Residual/Liquefied Shear Strength)	Newmark[1] Finite Difference/Finite Element Numerical Models (Suitable Soil Models to Account for Non-Linear Strength Reduction Under Cyclic Loading)
High (*L3*)	Pseudo-Static (Residual/Liquefied Shear Strength) Pseudo-Static (Remediated Case)	Newmark (Unremediated Case Residual/Liquefied Shear Strength)[1] Newmark (Remediated Case, Without Optimization)[1] Finite Difference/Finite Element Numerical Models

[1] *Accurately estimating seismic movements experienced by the dike is difficult. The Newmark sliding block method of analysis, when used with appropriate soil properties, should provide reasonable estimates of displacements. It has been considered as the preferred method of estimating displacements with the intent of the practitioners utilizing uniform procedures in design. Other methods of displacement calculations may be more suited and should be used, at the discretion of the practitioner, and when predicting more accurate magnitude and pattern of displacements is required.*

Reduced strengths for *L0* and *L1* can be determined based on the anticipated excess pore water pressures developed in the different dike zones due to ground shaking and should be a maximum of 80% of the drained strength of each respective zone.

The critical slip surface that corresponds to static stability is determined first. The magnitude of seismic displacements is estimated from the Newmark method for the critical slip surface(s) computed with appropriate soil properties.

A pseudo-static seismic coefficient (k_h) equal to ½ of the site-adjusted PGA, residual/liquefied soil strengths, and a minimum FOS of 1.2 are to be considered when the Liquefaction Index is assessed as *L2* or *L3*.

9. Challenges Faced in Implementing the Guidelines for Local Dike Systems

9.1 Background

Remediating and/or reconstructing dikes to satisfy the seismic performance expectations are often constrained by site access, loss of real estate, site soil conditions, and environmental concerns. This section of the paper presents a summary of challenges faced in two recent projects when exploring practical and economically feasible dike remedial measures to achieve the specified performance requirements.

Economically feasible ways of controlling the seismic loading-induced lateral displacements of an existing dike include; a) flattening the waterside slope; and/or b) buttressing the waterside slope by constructing a berm or a secondary dike system. Even though these alternatives can improve the performance of the dike, they often have environmental implications such as habitat destruction and changing river flow regimes etc. It is common practice in British Columbia for the environmental agencies to impose habitat offsetting measures using a ratio of 2:1 (two aerial units of offsetting for every one aerial unit of destruction) and these measures are costly. An assessment of environmental impacts also involves detailed and lengthy studies and consultations with First Nations groups. An attractive alternative would be to flatten the waterside slope to the extent practicable and relocating the dike crest further landwards. This alternative, however, often comes at the cost of loss of valuable real estate.

Dikes that meet the lateral displacement criteria, but are predicted to undergo large post-liquefaction settlements and do not meet the free board criteria, could be mitigated by raising the dike crest by the amount of the predicted settlement, provided that the dike core integrity can be maintained and confirmed. Placing additional fills to raise the dike elevation will increase the driving forces of the slope and affect the dike stability. There is also some risk of the post-liquefaction settlements occurring differentially and affecting the dike core integrity.

If none of the above techniques are viable, improving the seismic performance of the dikes can be achieved by implementing ground improvement measures to mitigate effects of soil liquefaction. It is the authors' assessment that densifying ground below the waterside slope of the dike is the most efficient way to control the effects of soil liquefaction (i.e. lateral and vertical displacements).

Details of two dike upgrade projects are presented herein along with the technical challenges faced in implementing the proposed designs. For confidentiality reasons, the projects are referenced as Project-1 and Project-2.

9.2 Project-1

This project involves upgrading approximately 400 m length of dikes along Fraser River. The dike system was originally constructed for protection against flooding of an industrial island. The referenced 400 m dike segment protects two separate industrial parcels of land. The municipality having jurisdiction of the sites dictated that the dike segments be raised in two stages – by about 0.6 m in the first stage and by an additional 1.4 m in the second stage. The dike configuration implemented during Stage-1 construction needed to be valid for the future dike raising by an additional 1.4 m.

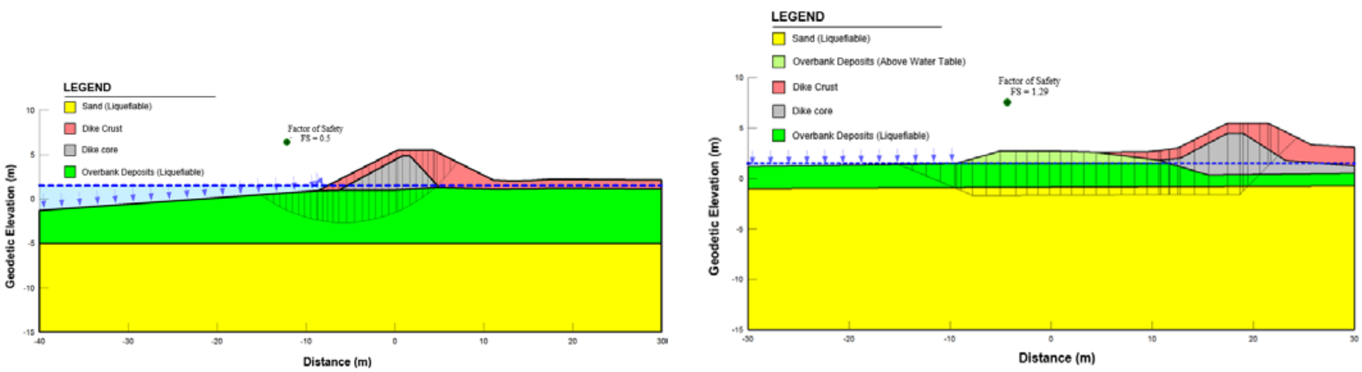
The original dikes were constructed in the late 1970s when seismic design standards were not in use. The dikes consist of a silty core with sand fill slopes constructed at slopes varying from 3H:1V to 2.5H:1V. The dike is underlain by several meters of overbank sediments comprising a mixture of clayey silt, low to non-plastic silts and silty sands followed by a 25 to 30 m thick deposit of Fraser River sand, in turn, underlain by marine silts and clays extending to depths in excess of 150 m.

The riverbed slope at the toe of about half of the subject dike (referred to as Segment-1) has a slope comparable to the waterside slope of the dike. The toe area of the remaining half of the subject dike (Segment-2) consists of a sandbar of some 2 m in height that has been built over time. The site that belongs to this portion of the dike was previously occupied by a saw mill and the operations required equipment access from land to the shoreline for transportation of logs.

Challenge#1: The seismic performance of the dike Segment-2 was assessed to be better than Segment-1, resulting primarily from the 2 m high overbank sediments forming a bar on the waterside acting as a buttress dike. Both the overbank sediments and the sands underlying the dike were assessed as having a high liquefaction potential for EQL-2 and EQL-3 shaking. The liquefaction extends over the full depth of the sand deposit, and results in a flow slide failure in dike Segment-1. For dike Segment-2, a flow slide failure was not predicted and the computed lateral displacements were less than the maximum prescribed in the Guidelines. In summary, for seismic stability of the dike segments, remedial measures were only required for dike Segment-1, although both Segments 1 and 2 form flood protection for the same industrial facility. The comparison of post-liquefaction stability of the dike segments are presented in Fig. 3.

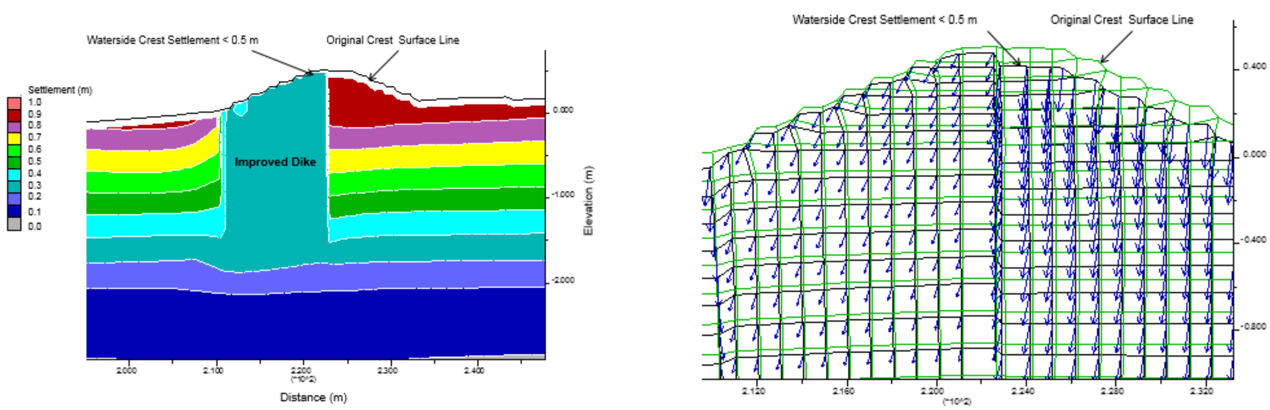
Challenge#2: Although dike Segment-2 was assessed as stable and meeting the lateral displacement criteria, the estimated post-liquefaction settlements were close to 1 m, about twice the settlement permitted in the

Guidelines. Even with 1 m of settlement, the post-earthquake configuration of the dike meets the free-board criterion specified in the guideline (i.e., 0.3 m above 1:10-yr flood elevation). The settlements were calculated using the Tokimatsu-Seed (1986) [4] empirical method by assigning a volumetric strain to each liquefied layer and summing the settlement of each layer. Of the 1 m settlement, about 0.7 m is estimated to occur as deep-seated settlements throughout the site and in the neighboring properties. It was the authors' assessment that such widespread and deep-seated settlements should not have an impact on the integrity of the dike body. Analysis of case history data was required to support this type of dike behavior, but the authors were unable to locate any applicable case histories.



(a) (b)

Fig. 3: Post-Liquefaction Stability of Dike Segments (a) Segment-1 (Sloping Ground towards Waterside – $FOS_{flowslide} < 1.0$), (b) Segment-2 (Buttressed towards Waterside – $FOS_{flowslide} > 1.0$)



(a) (b)

Fig. 4: Computed Post-Earthquake Dike Settlement Pattern With Ground Improvement (a) displacement contours after settlement (b) distorted mesh around the crest after settlement

Challenge#3: From cost and effectiveness of improvement considerations, the authors explored ground improvement using vibro-replacement stone columns, with all work carried out from the land side. One disadvantage of this technique is that installation of stone columns makes the improved portion of the dike more pervious than the existing soils. This required additional measures such as installation of a seepage cut-off wall to a depth of about 10 to 12 m involving interlocking and water tight Fiber Reinforced Polymer (FRP) sheet-piles.

Different options for width, depth and position of the improved zone i.e., landside toe, central, and waterside toe were investigated. The waterside toe alternative was found more effective for the dike stability through a series of slope stability analyses using Slope-W computer program and this option was pursued.

Therefore, improving soils underlying the waterside slope of the dike Segments-1 and -2 was considered over a width of about 10 m and to a depth of about 20 m to provide the necessary protection against seismic loading conditions. Such an improvement program would result in dike crest movements that meet the criteria in the Guidelines.

The cost of the sheet-piles was comparable to the cost of vibro-replacement stone columns, doubling the cost of dike improvements. A computer 2D model of the dike-foundation system using FLAC 2D program was developed and analyzed to assess the likely settlement pattern of the dike with ground improved resulting from the interaction response of the dike crest and the surrounding liquefied soils. The result indicates that the crest settlements at the waterside would be less than 0.5 m. The settlement pattern obtained from the FLAC 2D analysis is presented in Fig. 4. As maybe seen the dike crest settlement at the waterside meets the vertical displacement criterion (see Table 2). It was suggested to switch the improved zone by 1 m towards landside to provide a minimum crest width with enough freeboard after settlements.

9.3 Project-2

The dike segment for Project-2 is located some 50 m landward from the shoreline of a slough. The dike is underlain by approximately 4 m of fine-grained over bank deposits followed by a layer of loose to compact Fraser River sand deposit extending to a depth of 33 m followed by marine deposits extending to a depth in excess of 150 m.

The seismic hazard assessment on dikes at the Project-2 site was carried out in accordance with seismic design guidelines for dikes [1]. Site-specific one dimensional ground response analyses were carried out to evaluate the liquefaction potential of the Fraser River sand and it was identified that the loose to compact Fraser River sand is potentially liquefiable to a depth of 30 m for EQL-2 and EQL-3. There is very low potential for liquefaction under EQL-1. The earthquake demands for all three level of shaking and resistance are shown in Fig. 5.

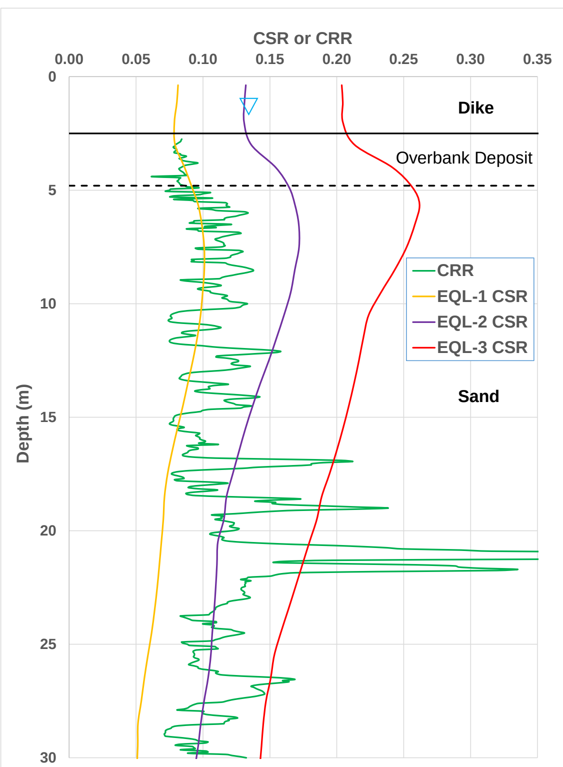


Fig. 5 – Comparison of Earthquake Demand and Cyclic Resistance of Soils for EQL-1, EQL-2 and EQL-3

Under the scenario where the dike is to be raised to a geodetic elevation of +5.6 m from its original +3.1 m, the estimated post-liquefaction lateral deformations of 0.5 m and 1.0 m under EQL-2 and EQL-3 respectively towards shoreline indicate that the dike body integrity will be affected by exceeding the guideline criteria (see Table-2). The estimated lateral deformations towards the land side met the performance criteria.

Further analyses were carried out to evaluate the option of buttressing the dike with 1.5 m thick sand fill on the water side extending approximately 20 m past the toe of the dike, similar to buttress in Project-1. This option was considered feasible as considering the availability of land and proposed future developments on this land. The analyses with extended buttress indicated that the integrity of the dike body can be maintained.

Challenge#1: The estimated settlement at the dike crest from post-liquefaction reconsolidation exceeded the tolerance criteria provided in the Guidelines. The settlements occur primarily in the Fraser River sands that are predicted to liquefy to about 30 m depth when subjected to liquefaction were in the order of 0.7 m while the Guideline criteria for maintaining integrity of the dike body are 0.15 m and 0.5 m for EQL-2 and EQL-3, respectively. The reconsolidation settlement was calculated for the entire depth of liquefaction.

Of the 0.7 m of settlement, about 0.4 m is expected to occur as deep-seated settlements on a regional basis. Thus, in a strict sense, only 0.3 m of settlement would be critical for the integrity of the dike core. The post-earthquake configuration of the dike met the required freeboard of 0.3 m for the 1:10yr flood.

Challenge#2: A recent study by Cetin et al. (2009) [5] indicates that based on past case-histories, the surface manifestation of vertical settlements resulting from soil liquefaction at depth and the associated volumetric strains is limited to the upper 18 m of soil deposits. Using this methodology, the estimated vertical reconsolidation settlement is within the tolerance criteria in the Guidelines. However, the authority having jurisdiction for the subject dike segment has been reluctant to approve the results of the latter study, for purposes of consistency with dike improvements being carried out throughout the province. Further compilation of available case history data is required to support this type of dike behavior. The authors were unable to locate case histories that are specifically applicable for dikes.

10. Conclusions

1. The performance-based Guidelines developed for High Consequence Dikes in Southwestern British Columbia, Canada, require that the dike displacement criteria be satisfied for all three levels of ground shaking; i.e., EQL-1, EQL-2, and EQL-3. It is difficult to meet the seismic performance criteria for dikes built on deltaic soils comprising potentially liquefiable soils extending to depths in the order of 25 to 30 m. Flow slide failure of the dike slopes leading to unacceptably large lateral displacements and large post-earthquake settlements exceeding 0.5 m were computed for the dikes referenced in Projects-1 and -2.
2. The flow slide failure and/or large lateral displacement hazards can be minimized or reduced by constructing buttress dikes on the waterside, subject to environmental approvals and availability of land. Improvements in dike performance can also be achieved by implementing costly ground improvement measures such as proposed for Segment-1 of Project-1 discussed in this paper.
3. It is difficult to reduce post-earthquake settlements occurring in sites with 25 to 30 m of liquefiable sands using cost-effective methods. Settlements calculated using simplified methods such as Tokimatsu & Seed [4] for deep soil deposits comprising liquefiable soils may be conservative for use in design. Although over building the dikes is an option, confirming the dike core integrity using analytical methods alone is difficult without case-history evidence.
4. Non dike-specific case-histories analyzed by Cetin et al. [5] indicate that soil liquefaction below about 18 m may not result in post-earthquake settlements that will lead to surface manifestations in the form of subsidence and fissures. While these findings have a positive impact on estimating realistic post-earthquake settlements in dikes, it was difficult to convince the authorities having jurisdiction over the dike segments referenced in Projects-1 and -2 without dike-specific case histories to support such behavior.

10. References

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