Seismic Design Guidelines for Dikes
2nd Edition

Dike Embankment Failure at Capitol Lake

Nisqually Earthquake, February 2001

Hanshin Earthquake, 1995

Ministry of Forests, Lands and Natural Resource Operations
Flood Safety Section

June 2014

BRITISH
COLUMBIA

SEISMIC DESIGN GUIDELINES FOR DIKES
2nd Edition

Prepared For

Ministry of Forests, Lands and Natural Resource Operations
Flood Safety Section

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June 2014

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1. Preamble   The purpose of this document is to provide guidelines for consideration of seismic stability and integrity of the *High Consequence Dikes* in Southwestern British Columbia and Vancouver Island with the intent to provide flood protection for the densely populated urban communities and regional infrastructure. Guidelines are provided on:

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

The west coast of British Columbia, including Vancouver Island, is a region of high seismic hazard in Canada. The seismic hazard is caused by the Juan de Fuca Plate subducting under the North American Plate resulting in large earthquakes of magnitude varying from M_w6 up to M_w9.

Densely populated urban communities and regional infrastructure in British Columbia are protected from flooding by some 300 km of river and sea dikes. The original dikes constructed in the early years were levees built with local fills and to very rudimentary standards. These dikes were upgraded in the 1970s and 1980s to design standards that existed at that time. During these upgrading works, the potential for earthquake-induced soil liquefaction and associated potential for damage was reviewed, but the cost to design for seismic loading was not judged to be commensurate with the consequences to the community. Since that time, municipalities throughout the Lower Mainland have seen extensive population growth and the consequences to the communities and infrastructure has increased. The extent of damage resulting from large scale flooding resulting from breaches to the different diking systems has been estimated to reach upwards of $50 Bn (2013 dollars).

Flood protection dikes are almost always located along river banks and shorelines that have historically experienced considerable damage following earthquakes. Therefore, dikes have a high geo-hazard exposure and need to be investigated in detail to allow identification and assessment of soil strata that are vulnerable to liquefaction, loss of shear strength, and displacement during seismic design.

This guideline has adopted a combination of traditional and performance-based design criteria for the seismic design of dikes. Dike performance is specified in terms of measureable criteria such as crest displacements of the dike structure. The methodologies and criteria provided in the document were established following a review of practices currently followed in other regions of the world that are also prone to high seismic hazards.

The maximum allowable displacements have been established with the intent of preserving the structural integrity of the dike body. It is implied that by conducting field investigations and displacement analyses at sections that are separated by a horizontal distance less than 300 m and satisfying the maximum allowable dike crest displacements prescribed in this guideline would reduce the potential of a dike breach as a result of differential or relative displacements.

It is acknowledged that achieving the dike performance criteria specified in this document may be difficult and costly in some instances considering that a majority of the dikes have originally been built to rudimentary standards. It is also recognized that the flood protection offered by the diking system is dependent on the performance of the weakest areas of the specific diking system. After the seismic assessment has been completed it may be necessary for the Diking Authority to consider, with backup analyses, possible alternatives such as dike realignment in areas of high seismic vulnerability, overbuilding the dike, incorporating the dike into adjacent land development areas, and restricting land use and regulating floodplain development to arrive at practical and cost-effective remedial measures.

The guidelines have been developed with the intent of achieving a consistently uniform assessment of the dike integrity and performance under seismic loading conditions by the different practitioners. Prescriptive seismic loading criteria and analysis methods including reporting requirements are identified, where feasible, to achieve this objective.

"Mean annual sea water levels" and "mean annual river water levels" are judged as appropriate to be used in the dike seismic integrity assessment with regards to seismic stability and displacement calculations, rather than water levels established based on probabilistic methods. For some urban communities, varying water levels including the projected rise in sea levels and higher levels of free-board are to be considered to provide an appropriate level of protection.

There are only a few countries in the world where extensive population is protected by flood control dikes located in areas of high seismicity, and in some ways British Columbia is unique in this respect. In the event of dike failure under flood conditions, extensive infrastructure and environmental damage as well as social disruption to communities should be expected.

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

The locations of most of the dike systems under consideration of these guidelines are shown on Figure 1.

3. Application   These guidelines apply to the design and construction of new and major upgrades to High Consequence Dikes. Seismic assessments and designs must generally be consistent with these guidelines to obtain Dike Maintenance Act approval from the Ministry prior to construction (also see "Discussion of Application of Guidelines for Highly Vulnerable Sites" below).

4. Background   This is the second edition of the "Seismic Design Guidelines for Dikes" which were first published in August, 2011. The following section briefly reviews the history and rationale for seismic guideline development for dikes.

The west coast of British Columbia including Vancouver Island is an earthquake prone region because of the tectonic setting and movements at plate boundaries. The Lower Mainland is a high seismic hazard region where there is potential for extensive earthquake damage to the Fraser River and sea diking systems that protect dense urban areas and critical regional infrastructure. Seismic hazard is generally lower in the British Columbia interior due to the reduced intensity of shaking and because there are fewer *High Consequence Dikes*.

The 1968 to 1994 Fraser River Flood Control Program (FRFCP) reconstructed approximately 300 km of river and sea dikes to a hydraulic standard of approximately 1:200 annual exceedance probability. Except for the major Barrowtown Pump Station in Abbotsford, the FRFCP design criteria (developed over 40 years ago) did not address seismic design issues largely because of cost constraints. This approach was rationalized on the basis of the rare chance of occurrence of a major flood simultaneously with a large earthquake. In the 1970's it was also anticipated that development of upstream storage reservoirs in the Fraser River watershed would significantly reduce the Fraser River flood threat and justify a relatively modest standard of dike protection.

Extensive urban development in the floodplain over the past several decades has increased reliance on the diking systems and significantly increased the consequences associated with poor performance of the diking system. Comprehensive hydraulic modeling and better understanding of sea level rise has resulted in higher design flood levels. In consideration of increasing sea level rise related flood hazard and urban development related consequences, the Ministry initiated development of design guidelines to explicitly address the seismic hazard aspects of dike design.

In November 2010, interim guidelines were issued that required the design of dikes to consider the effects of seismic activity on the integrity of the dike structure and required the owner to demonstrate that it would be possible to re-construct the dike within 6 months of the earthquake to retain a 1:10 yr annual exceedance probability flood.

This approach had the benefit of assessing potential damage, however, the feasibility of repair and re-construction was difficult to demonstrate. While rapid re-construction may be feasible for discrete, short sections of dike, re-construction to address widespread damage throughout the diking system may be difficult when dike work would be competing for resources for re-construction of other critical infrastructure such as water supply, sewer, roads and bridges. If the dikes cannot be repaired promptly, large sections of communities in low lying areas would be vulnerable to flooding, even from low return period events.

To address these issues, the Ministry published new guidelines in August 2011 (the first edition of these guidelines) that specified a level of required dike performance in terms of vertical and lateral dike deformation in response to three different levels of earthquake shaking or ground motions.

A review of the published seismic design requirements established for dikes and levees in other jurisdictions has been undertaken to provide insight on seismic design guidelines "currently" adopted by the profession. The reader is referred to Appendix A for details. The following summarizes the broad trends in the overall analysis procedures:

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 design criteria have evolved over time to achieve 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. Another 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 types 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.

This guideline has adopted a combination of traditional 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 within the dike structure as a result of design seismic loading. Satisfactory dike performance is implicitly taken into consideration by specifying multiple levels of earthquake shaking or ground motions and corresponding performance expectations that can be varied to achieve a high or low degree of safety/reliability.

These 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 guidelines. 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.

5. Discussion and Application of Guidelines to Highly Vulnerable Sites

Seismic strengthening and dike remediation methods are covered later in these guidelines (Section 16.) and in Appendix C. Ground improvement methods are costly and may only be practical for short sections of dike and appurtenant structures. After the seismic assessment has been completed it may be necessary for the Diking Authority to consider the following possible alternatives in situations where the displacement criteria cannot be met:

- Re-aligning the dike to avoid the high cost of ground improvement where the dike is located in seismically vulnerable areas adjacent to steeply sloping river banks.

- Overbuilding the dike to satisfy post-earthquake vertical displacement requirements provided that displacement analyses confirm that the dike

core will retain hydraulic integrity and the landside face geometry remains intact (*i.e.* "modifying the dike profile" as discussed on Section 16.).

- 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.

- Documenting expected damage, putting together a remediation plan, restricting land use and regulating floodplain development in the protected area (*e.g.,* flood proofing bylaws and other regulatory tools) to justify removal of the *High Consequence Dike* classification.

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

6. Disclaimer   This document provides a set of guidelines (requirements) for the seismic design of *High Consequence Dikes*. Golder is the sole author of this document with input and direction from the Ministry of Forests Lands and Natural Resource Operations and review and input by Thurber Engineering Ltd. It is expected that the guidelines may be further expanded and/or modified in response to feedback from the Ministry and other stakeholders.

Designers must use their own judgment in interpreting and applying the guidelines contained in this document and take full responsibility for the designs proposed. Golder Associates Ltd. and the Ministry of Forests Lands and Natural Resource Operations do not guarantee and are not responsible for the content, accuracy and completeness of the guidelines and accept no responsibility for the use of these guidelines. Designers' use of the guidelines is at their sole risk and liability.

7. Definitions   *Dike:* In the *Dike Maintenance Act,* a "dike" is defined as an embankment, wall, fill, piling, pump, gate, floodbox, pipe, sluice, culvert, canal, ditch, drain or any other thing that is constructed, assembled or installed to prevent the flooding of land. For purposes of this guideline, the definition has been simplified to represent *a* manmade barrier constructed of soil and/or structural elements (such as floodwalls) along a water course for the primary purpose of providing flood protection.

High Consequence Dikes: flood protection dikes where the economic and/or life safety consequences of failure during a major flood are very high. These dikes typically protect urban or urbanizing areas, and failure could result in large economic losses and/or significant loss of life. The majority of the dikes reconstructed under the 1968 to 1994 Fraser River Flood Control Program would be considered High Consequence Dikes.

Floodwall: a manmade barrier constructed of material other than soil along a water course for the primary purpose of providing flood protection.

Liquefaction: describes a phenomenon whereby a saturated soil loses significant portion of strength and stiffness in response to a sudden change in stress and/or pore pressure condition which causes the soil to behave like a viscous fluid.

Liquefaction Susceptibility: is a reference to the ability of the soil deposit to liquefy when subjected to an applied stress. For example, a dry or unsaturated granular deposit does not have the ability to liquefy due to the absence of saturated conditions. Cohesive deposits are also not susceptible due to the particle structure of the deposits and the interstitial forces that hold the clay molecules together (cohesion).

Liquefaction Potential: is a reference to the resistance of a deposit to liquefaction as a result of a sudden change in stress level and/or pore pressure condition due to cyclic loading.

Cyclic Mobility: refers to cumulative displacements that occur during each cycle of shear stress due to softening (without liquefaction) and the displacements are generally significantly less than liquefaction-induced displacements.

Flow Slide: significant translational type displacement of a land mass when static shear stresses exceed the undrained residual shear strength of liquefied soil (*i.e.*, a factor of safety against slope failure equal to or less than 1.0 when using liquefied soil strengths and no inertial loads).

Bearing Capacity: the ability of a deposit to support an external load without developing a soil shear failure (*i.e.*, plastic flow and/or a lateral expulsion of a soil from beneath the applied load).

Moment Magnitude: is a measure of the amount of energy released during an earthquake which is not dependent on ground shaking levels or level of damage (*i.e.,* Wood & Neumann (1931)), but reflects factors that are characteristic to the rupture of the fault that produces the earthquake.

Return Period: a reference to the frequency or return time period for a specific seismic hazard (*i.e.*, 1 in 2,475 yr). The seismic hazard is normally expressed as a percent chance of exceedance over a specified period of time (*i.e.*, a 2% chance of exceedance in 50 years). Another form of expressing the seismic hazard is in terms of probability of exceedance which considers the probability that an earthquake will generate a level of ground motion that exceeds a specified reference level during a given exposure time.

Piping Failure: piping occurs when hydraulic gradients at or near the ground surface or free face exceeds unity (*i.e.*, the change in total head pressure head over a unit path length exceeds unity). Over a prolonged period of time, internal erosion can generate an "effective" pipe condition leading to increased flow and eventual collapse of the soil deposits.

8. Design Objectives

The design objectives for dikes are established 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 rare 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.

Refer to Section 13(2) for performance categories with respect to displacements and anticipated damage.

Typical return periods or annual exceedance probabilities considered are summarized in Table 1:

Table 1: Return Periods and Annual Exceedance Probabilities

Return Period Classification	Event Classification	Return Period (Years)	Annual Exceedance Probability	Percent Chance of Being Exceeded in 100 Years
Short	Frequent	100 to 200	0.01 to 0.005	65% to 40%
Intermediate	Intermediate	475 to 975	0.0021 to 0.001	20% to 10%
Long	Rare	2,475 to 10,000	0.0004 to 0.0001	4% to 1%

9. Seismic Hazards Considered

Potential seismic hazards affecting the dikes located in Southwestern British Columbia include the following:

- Ground shaking;
- Slope displacements caused by ground shaking;
- Bearing capacity and sliding failure;
- Soil liquefaction;
- 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 displacements.

The seismic hazard to Southwestern British Columbia results from the offshore subducting of the Juan de Fuca Plate beneath the Continental Plate. The 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 epicenter as close as a few km from the site of interest), deep intra-plate earthquakes (up to $M_w7.5$, with epicenter as close as 40 km from the site of interest), and inter-plate or subduction earthquakes (up to M_w9, with epicenter as close as about 140 km from the site of interest).

10. Water Levels and Post-Earthquake Freeboard

Spring snowmelt freshets pose the main flood hazard for the Lower Fraser River flood plain areas. Flooding that results from heavy rainfall in the fall and winter is also a significant concern for the diked reaches of coastal rivers. High tides can impact a sea dike at any time of the year, and storm surge events are most frequent during the fall and winter period. The flood hazards resulting from channel obstructions due to ice jam formations are considered to be possible, but less likely, particularly in Southwestern BC. Combining the peak water levels that occur annually over a short period with rare earthquake ground shaking for use in a dike stability/displacement analysis, is generally considered to result in overly conservative designs.

In order to avoid unrealistically low combined probabilities, "*mean annual river water level*" and "*mean annual sea water levels*" should generally be used in the stability and displacement calculations completed as part of the seismic assessment of dikes. However, in some instances (*e.g.* for sea dikes exposed daily to both high and low tides), the sensitivity of the stability and displacement calculations to varying water levels should be considered. Future dike upgrades may need to consider projected rise in sea levels.

The designers should 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.

Vertical displacement will reduce the available post-earthquake free board relative to the design flood level. However, the loss of free board is not as critical as the integrity of the diking system to protect against the occurrence of smaller floods during a possibly extended period (*i.e.*, several months to a few years) before the damaged sections can be fully restored to their design condition.

The Performance Categories and Permissible Displacements provided later in these guidelines are intended to limit the damage and retain post-earthquake flood protection capability for at least the 1:100-yr and 1:475-yr return period ground motions with some compromised flood protection for the 1:2,475-yr return period ground motions (See Table 2).

11. Information to be Accessed and Reviewed

Dikes may be constructed, rehabilitated, or altered in segments, however, the seismic stability of the entire dike should be addressed. As a minimum, the following information should be collected, compiled, and reviewed prior to and during construction, rehabilitation, and alteration of dikes:

1. Available drawings showing the dike cross section, materials used for construction, construction details such as level of compaction for the dike segments of concern and adjoining segments, drawings and design details for appurtenant structures such as flood boxes and pump stations;
2. Dike inspection reports summarizing past performance of the dike segment;
3. Available geotechnical borings, reports, and as-built (record) drawings prepared and completed by engineers and contractors during initial dike construction and subsequent modifications/alterations (if applicable);
4. Data on mean river water and mean ocean water levels;
4. Ground shaking levels for return periods of 100-yr, 475-yr, and 2,475-yr by establishing the UTM coordinates for the site and from the Natural Resources Canada website http://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index_2010-eng.php
5. Topographic and Bathymetric data for the area adjacent to the dikes. The geometric data should extend a sufficient distance beyond the dike (upstream and downstream) to support static, hydrostatic and dynamic analyses of the dike structure, particularly where performance criteria are considered for establishing a seismic design basis for the dikes. Bathymetry data collected within the last 2 years is preferable. A detailed discussion is required to demonstrate the appropriateness of the topographic and bathymetric data used in the analysis; and

6. Surficial geology maps and aerial photographs.

It should be recognized that the ground surface accelerations experienced by the dike structure during a seismic event at any given site will be influenced by the underlying ground conditions. Amplification or de-amplification of firm ground accelerations may occur but will depend on the site-specific ground conditions and should be considered in design.

12.
Geotechnical Investigations for Seismic Design

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 displacement. 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 displacement.

The main objective of the 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 and/or cyclic mobility in the form of penetration resistance, strength, and shear wave velocity;

- Sampling of soil strata susceptible to liquefaction and/or cyclic mobility;

- Gradation of soils susceptible to liquefaction and/or cyclic mobility;

- Index testing of soils susceptible to liquefaction and/or cyclic mobility; and

- Cyclic simple shear testing of fine-grained soils to investigate liquefaction susceptibility and/or cyclic mobility.

Dikes comprise hundreds of kilometres of earth fill embankments constructed over varying ground conditions that may include reclaimed areas, buried channels, previous failures, river meander and 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 field investigations.

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

- **SPT Method:** This is the most common field investigation method used to identify soils that are susceptible to liquefaction. It has the advantage that a disturbed sample is recovered after each test at set intervals of generally 1 to 2 m. This procedure has strict requirements for hammer energy, sampler size, and drilling method. Performing SPTs in gravelly soils require special consideration due to the coarse size of soil particles

and the resulting misleadingly high blow count data. The method is operator dependent and should be carried out in accordance with current ASTM standards. The results should be carefully interpreted and corrected according to established procedures. SPT sampling is not considered appropriate for use in coarser granular soils containing significant gravel and larger particle sizes.

- **CPT Method:** For many sites, the CPT is the preferred method to identify soils that are susceptible to liquefaction. The method provides a near-continuous indication of soil consistency and type with depth, and is less susceptible to operator-related differences in measurements; however, testing should be carried out in accordance with current ASTM standards. The method is capable of detecting relatively thin layers of soil. The cone results should be correlated with data from nearby borings, where local correlations do not exist. The method does not provide soil samples and is not suitable for use in soil strata containing significant gravel and larger particle sizes or highly consolidated deposits (*i.e.,* glacial till).

- **BPT Method:** This is the preferred method of field investigation for sites underlain by coarse-grained soils (*i.e.*, gravel and cobbles) that are susceptible to liquefaction. Reliable data can be obtained provided that accurate bounce chamber pressure or hammer energy measurements are carried out and industry established and accepted procedures for data reduction and interpretation are followed.

- **SWVT Method:** Shear wave velocity is the primary parameter used for Site Classification as per 2010 National Building Code of Canada (NBCC) and other current standards and codes. The data is used for the determination of soil shear moduli for ground response analysis. Experienced individuals should perform these tests, as the collection of results requires considerable skill and expertise.

Other field exploration and in-situ testing methods for assessment of soil liquefaction may be used with site-specific correlations with one of the methods described above.

In addition, soil sampling using sonic drilling methods may provide important information on the soil layering in some projects. Sonic drilling is able to provide continuous sampling to assist with identifying transition zones between strata as well as provide the opportunity for bulk sampling. Generally, the method does not provide a means to develop engineering properties of the deposits.

All field investigations should be undertaken in accordance with published standards, where applicable.

The field investigations should include test holes that extend to depths between about 10 m and 30 m below existing ground surface or to practical refusal to further penetration, whichever occurs first.

A minimum of three borings should be considered 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. The horizontal spacing of data sections along the dike should not be greater than 300 m. Closer spacing of data sections may be required where significant variations in subsurface conditions are anticipated.

For dike segments where the available subsurface data is limited, the initial 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 should incorporate subsurface investigations as identified above.

13. Performance-Based Seismic Design Criteria

A performance-based seismic design is accomplished by defining appropriate levels of design earthquake ground motions and corresponding acceptable levels of damage. The design earthquake motions include those from frequent events that are likely to occur within the life of the structure as well as infrequent or rare events that typically involve very strong ground shaking. The acceptable level of damage is specified in terms of displacements to be experienced by the structure. Damage is categorized in terms of "Performance Categories", which are related to the effort required to restore the full functionality of the structure. **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 described below shall be considered in seismic design.

   - Earthquake Shaking Level 1 (EQL-1)

     1:100-yr return period ground motions that are equivalent to having a 40% chance of exceedance in 50 years or 63% chance of exceedance in 100 years.

     For dikes located in the Lower Mainland, this level of shaking is associated with an M_w6 earthquake.

   - Earthquake Shaking Level 2 (EQL-2)

     1:475-yr return period ground motions that are equivalent to having a 10% chance of exceedance in 50 years or 19% chance of exceedance in 100 years.

     For dikes located in the Lower Mainland, this level of shaking is associated with an M_w7 earthquake.

   - Earthquake Shaking Level 3 (EQL-3)

     1:2475-yr return period ground motions that are equivalent to having a 2% chance of exceedance in 50 years or 4% chance of exceedance in 100 years.

     For dikes located in the Lower Mainland, this level of shaking is associated with an M_w7 earthquake.

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.

Table 2: Summary of Maximum Allowable Dike Crest Displacement Corresponding to Performance Categories

Performance Category	Earthquake Shaking Level	Maximum Allowable Vertical Displacement	Maximum Allowable Horizontal Displacement
A	EQL-1	small (<0.03 m)	small (< 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 with 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.

It should be recognized that 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 shall 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.

14. Selection of Time-Histories

- Time-histories selected for Newmark or dynamic analysis of the dike system should be from a seismo-tectonic & geologic settings comparable to those relevant to the site of interest.

- The time-histories selected for Newmark or dynamic analysis should be consistent with the magnitude/distance/duration scenario developed for the analyses.

- Linear scaling or time domain spectral matching are the preferred approaches to matching a time-history to a target response spectrum over the period range of interest.

- Multiple time-histories should always be used in dynamic analyses. Typical practice considers the maximum response of three (3) time-histories or an average response of seven (7) or more time-histories. When selecting three (3) time-histories, they should correspond to three different historical earthquakes.

Unless otherwise approved by the Diking Authority, the seed ground motions obtained from the following two websites shall be used with uniform and linear scaling to match the site-specific peak firm-ground horizontal acceleration in dynamic analysis of dikes:

a) http://peer.berkeley.edu/peer_ground_motion_database/
b) http://strongmotioncenter.org/

15. 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 and associated consequences;

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) 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.

It should be recognized that 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. A description of the proposed details and anticipated performance of appurtenant structures should be provided for review by the Deputy Inspector of Dikes under the Dike Maintenance Act and by the Diking Authority.

Guidelines on the appropriate methods of analyses for Steps-1 through 4 are provided below:

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 1:2475-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) and the empirical liquefaction resistance charts shown in Figures 2 and 3.

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$_u$ ≤ 50%);

L2:   Liquefaction occurs in zones of limited thickness; and

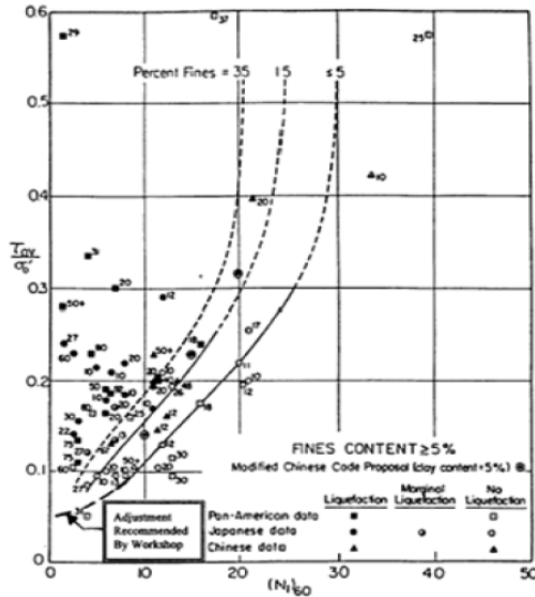*L3:*   Complete liquefaction of soils.

Figure 2: Empirical Liquefaction Resistance Charts When Using SPT (M$_w$ = 7.5) (Youd et al, 2001).
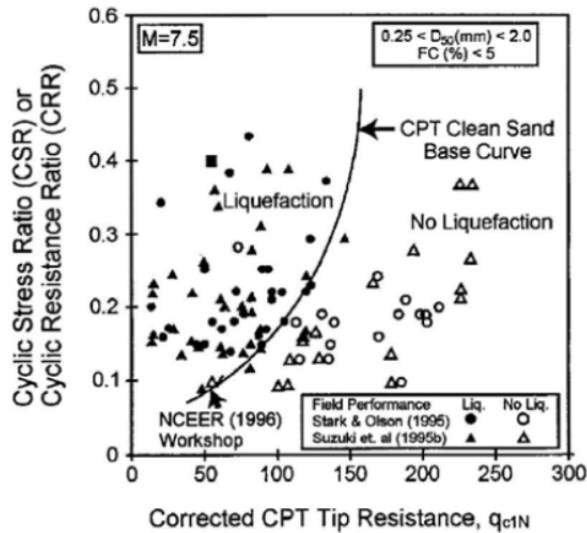

Figure 3: Empirical Liquefaction Resistance Charts When Using CPT (M$_w$ = 7.5) (Youd et al, 2001).

Fine-grained silty soils that exhibit plasticity indices [PI] less or equal to 7, should be treated as granular soils with appropriate corrections to penetration resistance based on fines content.

Fine-grained soils with PI values in excess of 7 are unlikely to undergo liquefaction and substantial softening during cyclic loading and should be treated as clayey soils with undrained shear strength consistent with field and laboratory measurements.

The ground motion amplification factors given in Table 3 as a function of the Site Class and Table 4 may be used in the SSMA to compute the peak ground surface acceleration, as per Table 4.5.1.9 of NBCC 2010. Alternatively, they may be derived or inferred from site-specific ground response analyses carried out using wave propagation methods. The peak horizontal ground surface accelerations used in the SSMA shall be the larger of the values computed from site-specific wave propagation analyses or,

- 80% of the PGA derived from the amplification values given in Tables 3 and 4 when combined with Class C PGA for EQL-1

- 60% of the PGA derived from the amplification values given in Tables 3 and 4 when combined with Class C PGA for EQL-2

- 50% of the PGA derived from the amplification values given in Tables 3 and 4 when combined with Class C PGA for EQL-3

The amplification factors given in Table 3 are applicable for level ground conditions. For convex surfaces such as dikes, the crest amplifications may be established as the product of the firm-ground acceleration, amplification at level ground ($Ampl_{level-ground}$) and a modification factor $Ampl_{apex}$ to account for the apex angle of the dike (see Figure 4 for definition of apex angle and suggested amplification factors. Topographical amplification factors established by other published data sources are also acceptable:

[1] $PGA_{crest} = PGA_{firm-ground} \times Ampl_{level-ground} \times Ampl_{apex}$

Table 3: Amplification of Ground Motions – Level Ground

Site Classification	Amplification - Level Ground Conditions ($Ampl_{level-ground}$)				
	$Sa (0.2) \leq 0.25$	$Sa (0.2) = 0.50$	$Sa (0.2) = 0.75$	$Sa (0.2) = 1.00$	$Sa (0.2) \geq 1.25$
Soil Class E	2.1	1.4	1.1	0.9	0.9
Soil Class D	1.3	1.2	1.1	1.1	1.0
Soil Class C	1.0	1.0	1.0	1.0	1.0
Soil Class B	0.8	0.8	0.9	1.0	1.0

Table 4: Modification Factors to Account for Topographic Effects

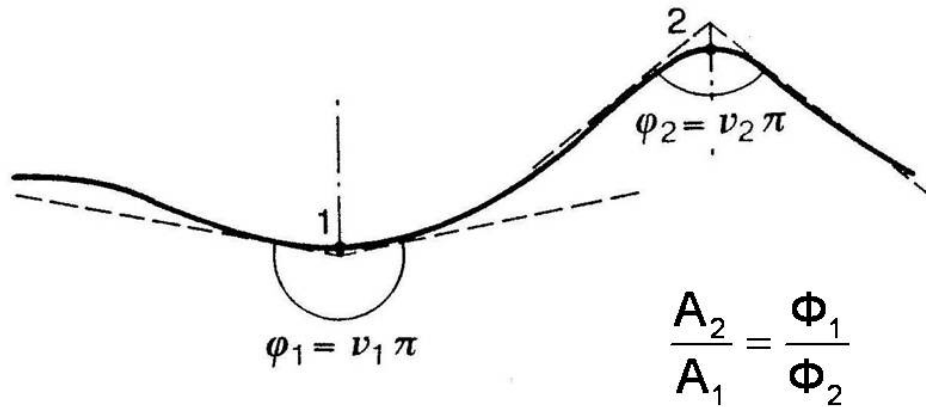| Apex Angle (φ_2)	Modification Factor ($Ampl_{apex} = A_2/A_1$)
180^0	1.0
165^0	1.1
150^0	1.2
140^0	1.3

Figure 4: Topographic Amplification (Faccioli, 1991).

Table 5: 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] *The well-established Newmark sliding block method of analysis, when used with appropriate soil properties, is considered as the preferred method of estimating seismic displacements compared to other empirical methods. For consistency and uniformity in assessment of displacements by Practioners, only this method of analysis will be accepted by the Provincial Flood Safety Section for analysis scenarios outlined in Table 5. Other (rigorous) methods of displacement calculations may be more suited and should be used, at the discretion of the practitioner, when predicting more accurate magnitude and pattern of displacements are required.*

Reduced Strength for the *Liquefaction Indices* of *L0* and *L1* should 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.

Dike slope stability and displacement analyses shall consider both circular and non-circular slip surfaces as appropriate for the site-specific soil stratigraphy. The critical slip surface that corresponds to static stability shall be determined first. The magnitude of seismic displacements estimated from the Newmark method shall correspond to the critical slip surface(s) established, and with soil properties appropriate for seismic analysis.

Insight into the anticipated Liquefaction Index may be obtained from the attached Figure 5 that shows the liquefaction susceptibility of soils in Southwestern British Columbia.

Residual shear strength for liquefied zones should be estimated based on the charts shown on Figure 6a and 6b. The lower bound strength curves correspond to sites with relatively thick layers of liquefiable soils that are overlain by lower-permeability soils that would impede dissipation of excess pore water pressures.
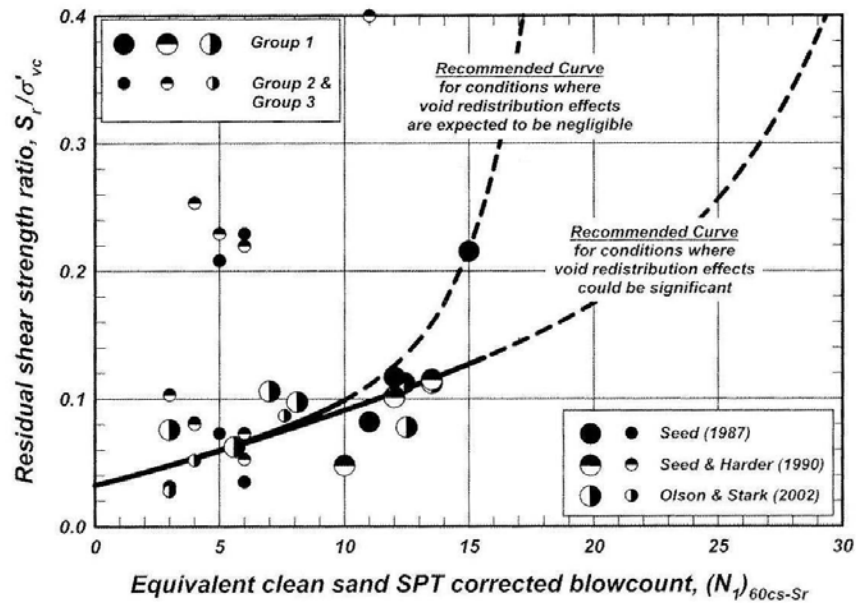

Figure 6a: Residual Shear Strength Ratio of Soils vs SPT Resistance (Idriss & Boulanger, 2008).
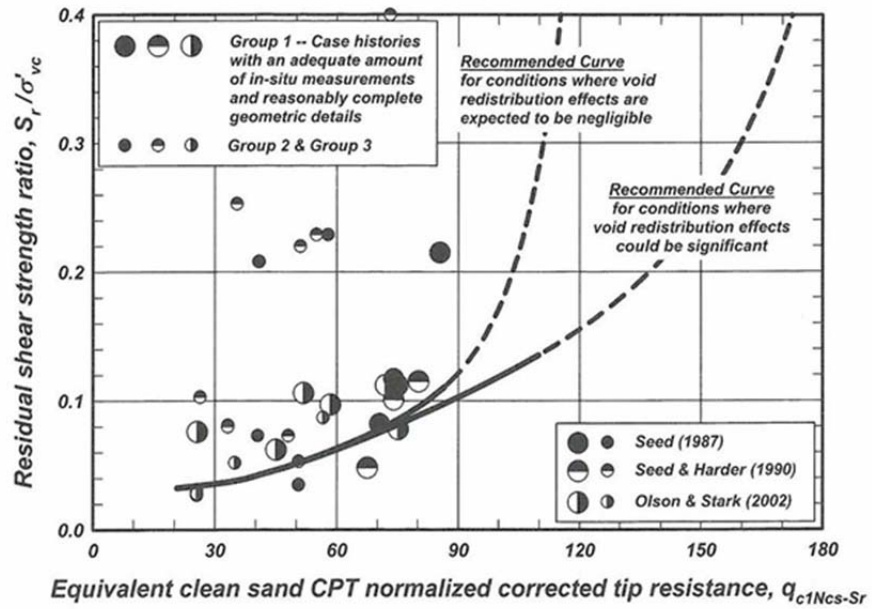
Figure 6b: Residual Shear Strength Ratio of Soils vs CPT Resistance (Idriss & Boulanger, 2008)

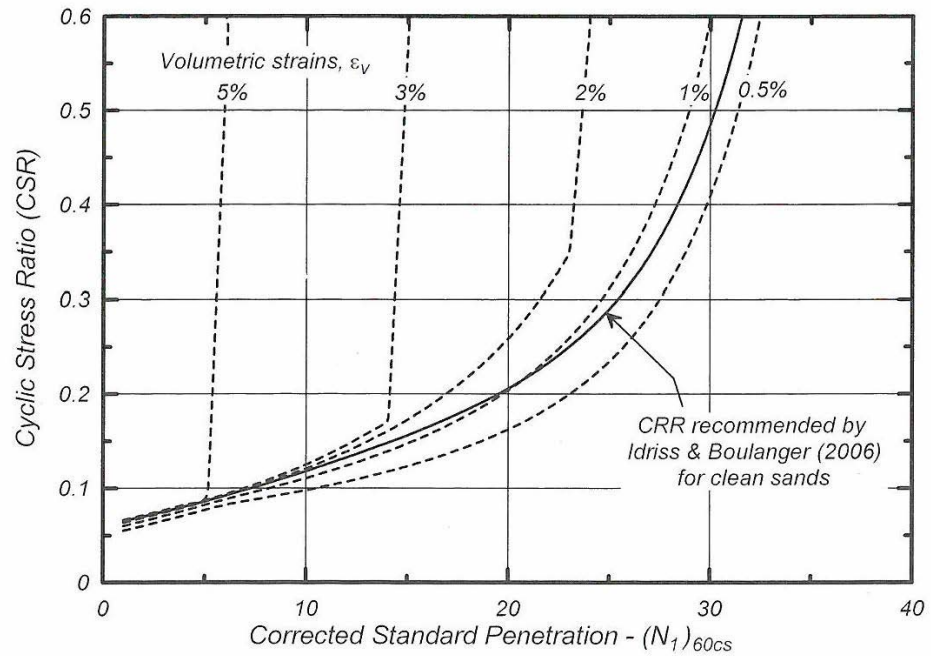Post-liquefaction vertical settlements should be estimated using the charts given in Figures 7a and 7b.

Figure 7a: Volumetric Strain of Soils vs SPT Resistance ($M_{w=}$ 7.5)
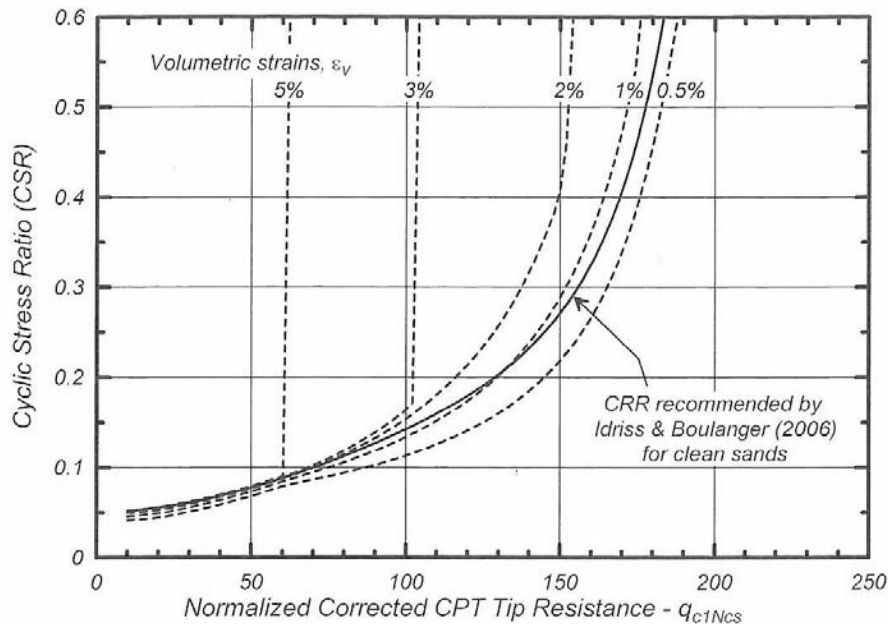*(Idriss & Boulanger, 2008)*

Figure 7b: Volumetric Strain of Soils vs CPT Resistance ($M_{w} = 7.5$)
(Idriss & Boulanger, 2008)

A pseudo-static seismic coefficient (k_h) equal to ½ PGA, residual/liquefied soil strengths, and minimum FOS of 1.2 shall be considered when the Liquefaction Index is assessed as L2 or L3 [Ref. Flow Chart shown in Figure 8].

A pseudo-static seismic coefficient (k_h) equal to PGA, reduced soil shear strengths, and a minimum FOS of 1.0 shall be considered when the Liquefaction Index is L0 or L1 [Ref. Flow Chart shown in Figure 8].

The proportion of the PGA to be considered, the factor of safety against slope failure, and the anticipated seismic displacements in the above analyses are inter-related. Typically, ½ PGA and a factor of safety ranging from 1.1 to 1.3 are considered to result in acceptable seismic displacements [Anderson *et al*, 2008, NCHRP-611]. These analyses do not explicitly consider effects of residual excess pore water pressures.

A typical flow chart(s) showing the analysis steps is shown on Figures 8 (a) through 8 (d). A dike system that is underlain by potentially liquefiable soils will require several design iterations to satisfy dike integrity and flood protection requirements.
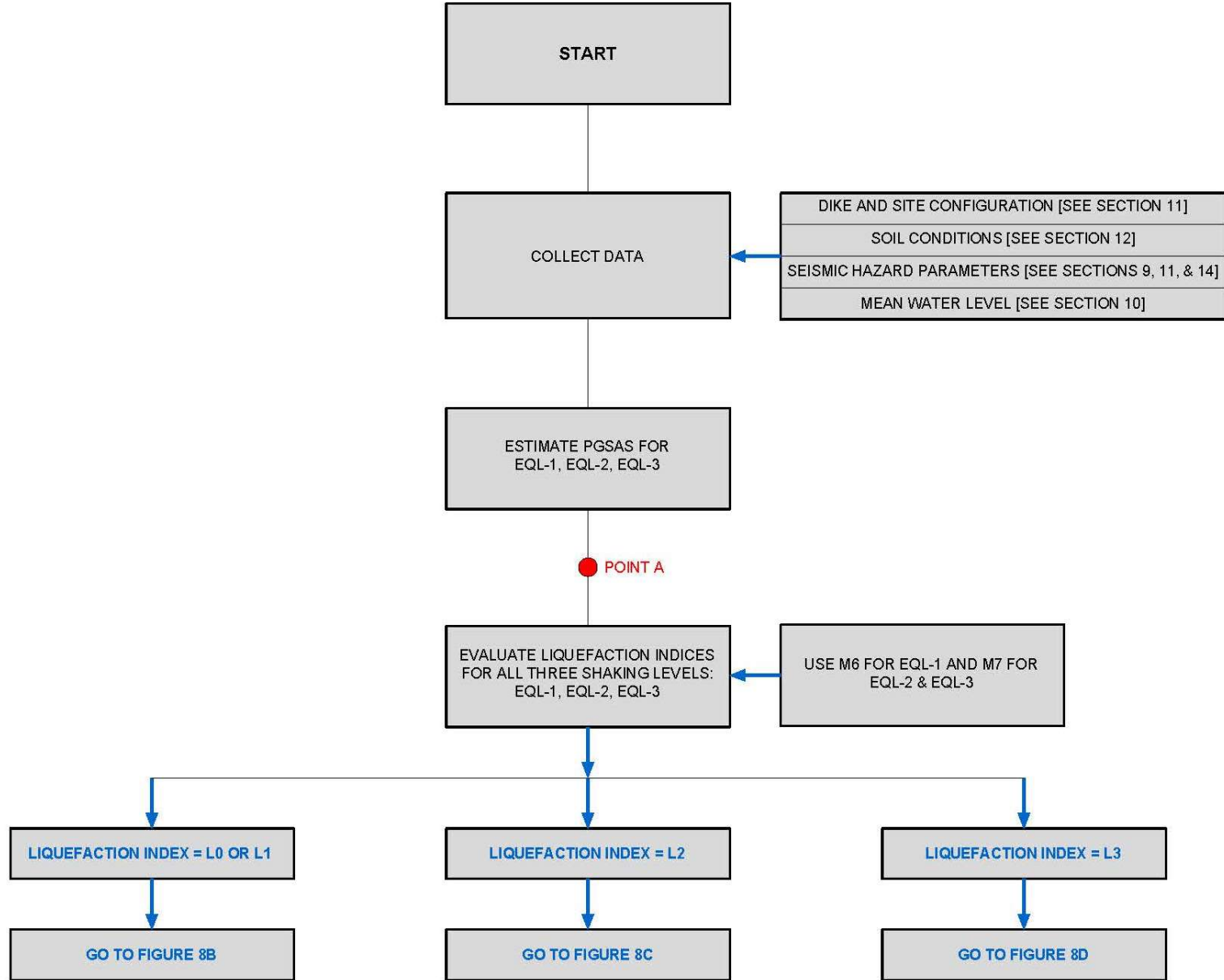
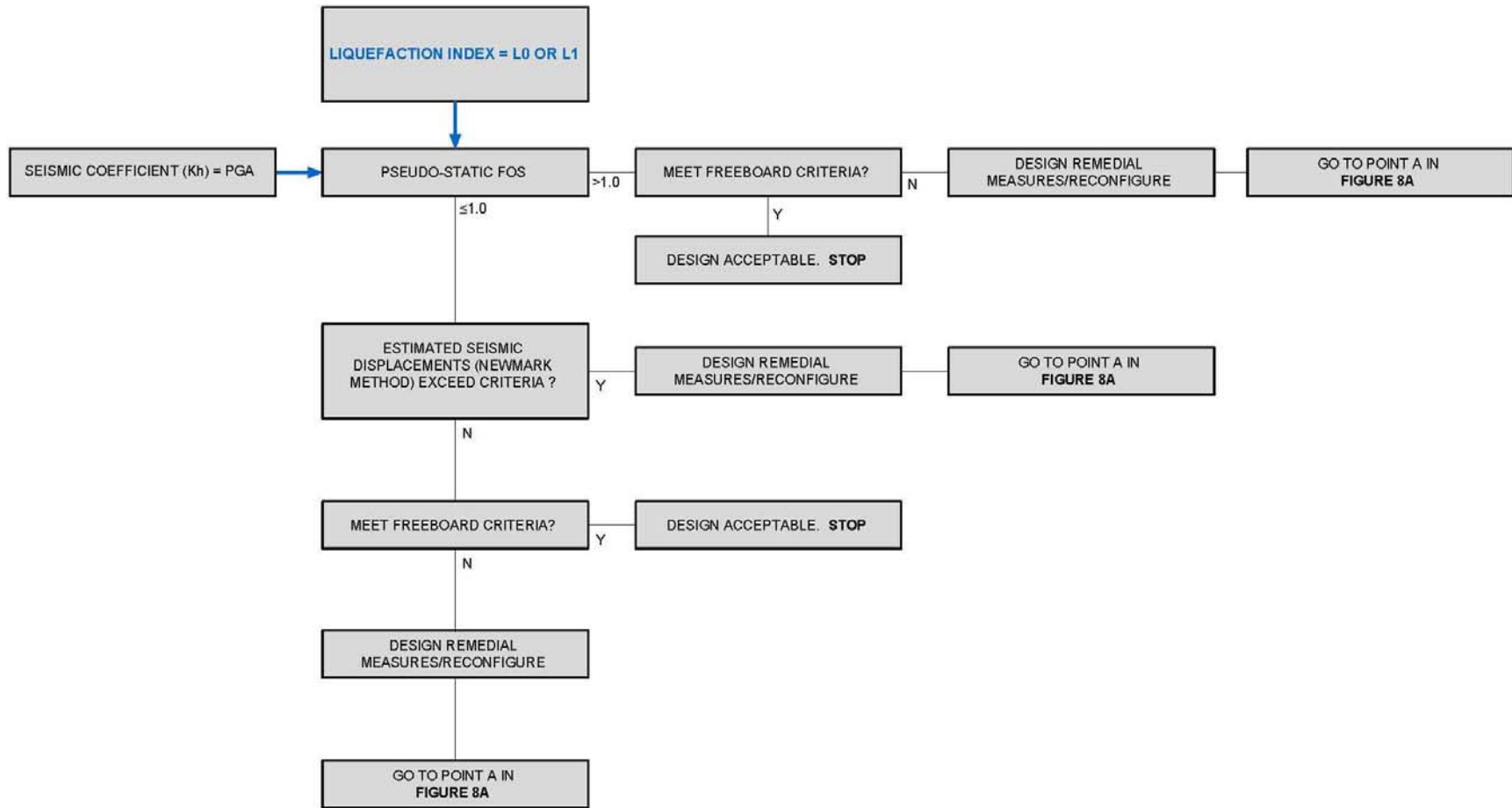*Figure 8a: Data Collection and Evaluation of Liquefaction Index*

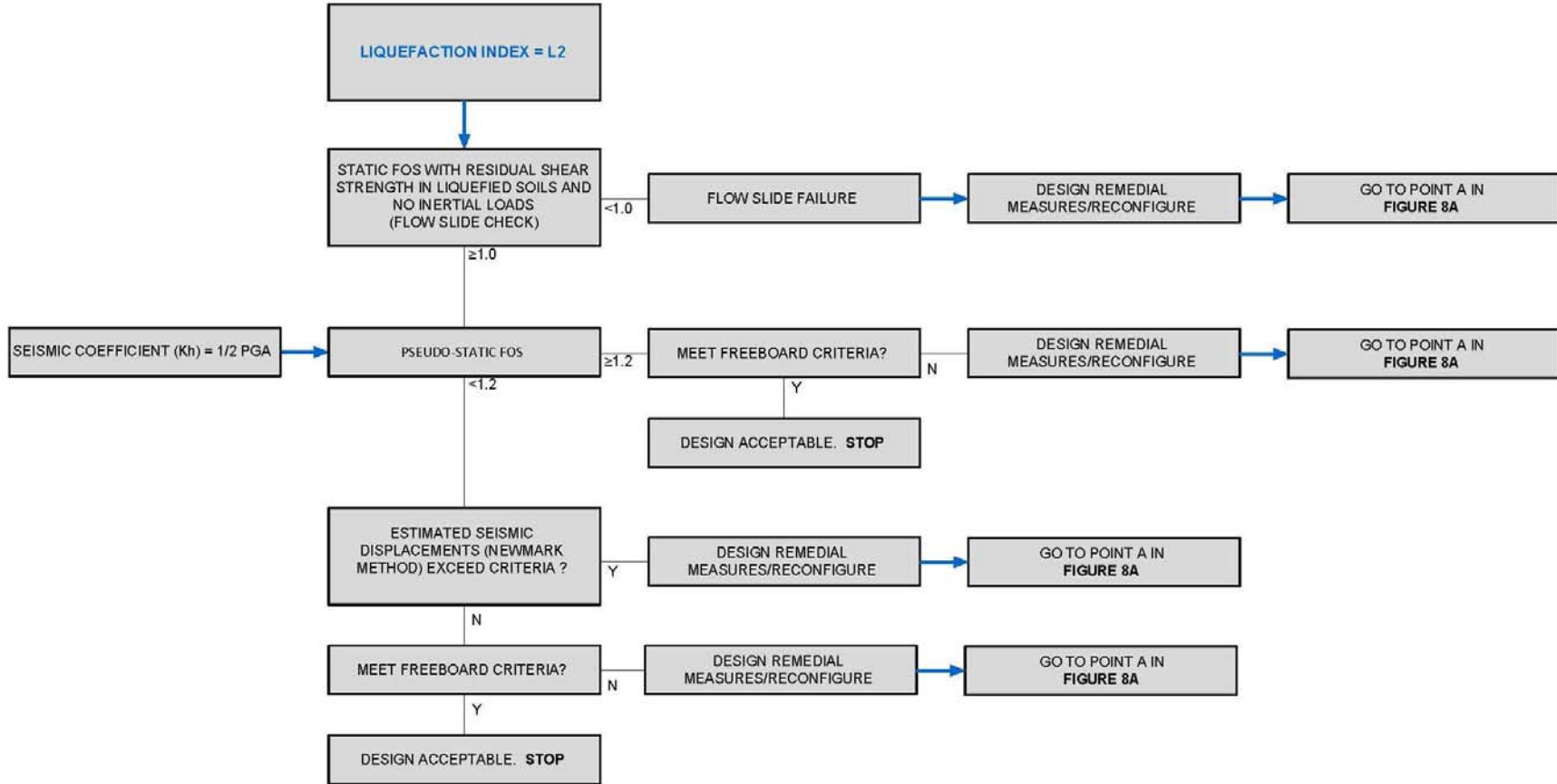*Figure 8b: Typical Analysis Flow Chart for Liquefaction Indices L0/L1*

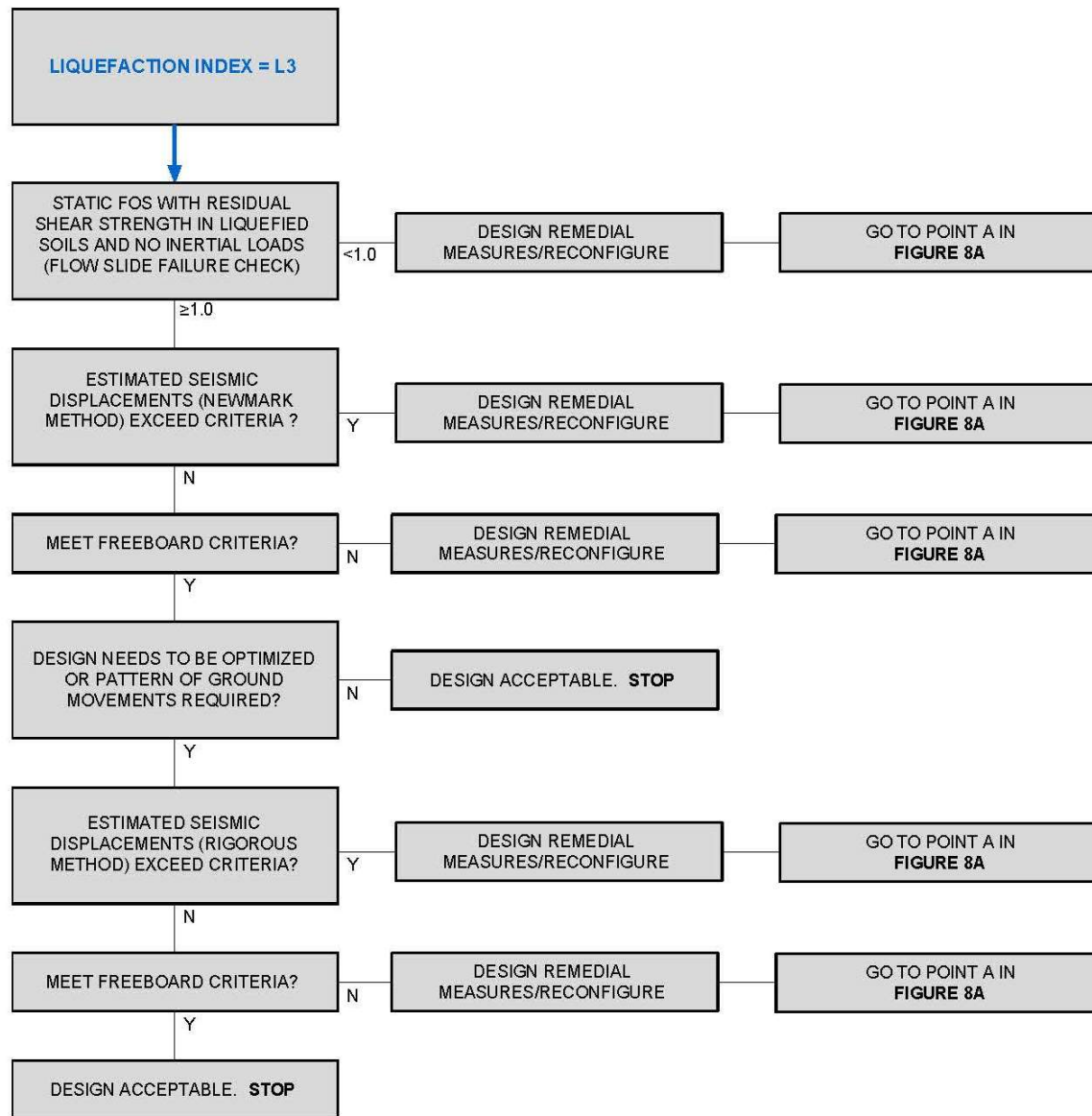*Figure 8c: Typical Analysis Flow Chart for Liquefaction Index L2*

Figure 8d: Typical Analysis Flowchart for Liquefaction Index L3

16. Dike Remediation Methods

Methods of seismic remediation or strengthening of dikes vary depending on the modes of failure and/or assessed post-earthquake performance of the dike-foundation system under consideration. One, or a combination of several, remediation methods may be utilized to achieve the required post-seismic performance. These measures are to be implemented following design and evaluation of dike performance. They may not be suitable for emergency repair of dikes immediately following a significant seismic event.

The type of failure mechanism that may be anticipated during and/or following a seismic event will be dependent on several factors including, but not limited to, the following:

- Nature/distribution of the soil deposits contained within the dike, beneath and beyond the dike;

- Strength, stiffness and sensitivity of the materials within the dike as well as within the foundation deposits;

- Geometric profile of the dike as well as the topographic and bathymetric surface profiles both upstream and downstream of the dike; and

- Level of water retained by the dike as well as groundwater levels (phreatic surface) within the dike and the adjacent land.

The designers should consider the factors noted above in the seismic evaluation of dikes.

The following methods of improving seismic stability of dikes can be considered in order to meet the performance expectations:

- Modify the existing dike profile – Modifying the existing dike profile may include a combination of raising the dike, widening the dike and flattening the landside and waterside slopes to reduce the potential for slope instability during and/or following seismic shaking, or at least maintain sufficient free board to retain the water contained by the dike following a major earthquake; and

- Implement ground improvement measures – Ground improvement measures may include densification and strengthening of site soils, improving drainage to reduce the potential for soil liquefaction and associated softening, and/or incorporating inclusions into the dike in the form of sheet piles, secant piles, soil-cement mix panels or jet-grout columns.

The designers should consider one or several methods in combination, for dike remediation and strengthening, as considered appropriate.

Details of dike remediation and strengthening techniques including the limitations of each method and equipment are provided in Appendix C.

The data and information used in the dike analysis and findings of the analyses should be summarized in a report consistent with the Table of Contents outlined in Appendix E.

Ground improvement methods are costly and may be practical for sections of dike and appurtenant structures. After the seismic assessment has been completed, it may be necessary for the Diking Authority to consider

the following possible alternatives in situations where the displacement criteria for dike structure integrity cannot be met:

Re-aligning the dike to avoid the high cost of ground improvement where the dike is located in seismically vulnerable areas adjacent to steeply sloping river banks;

Overbuilding the dike provided that the detailed analyses confirm that the dike core will retain hydraulic integrity (*i.e.,*. modifying the dike profile as discussed above); and

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.

17. Threshold Seismic Event for Post-Event Dike Integrity Inspection

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

Based on past experience, earthquakes of magnitude $M_w \leq 5.0$ are unlikely to cause liquefaction and significant ground displacements in earth structures of relatively good workmanship. Figures 9 and 10 show documented evidence of damage as a function of the earthquake magnitude versus distance from the source.
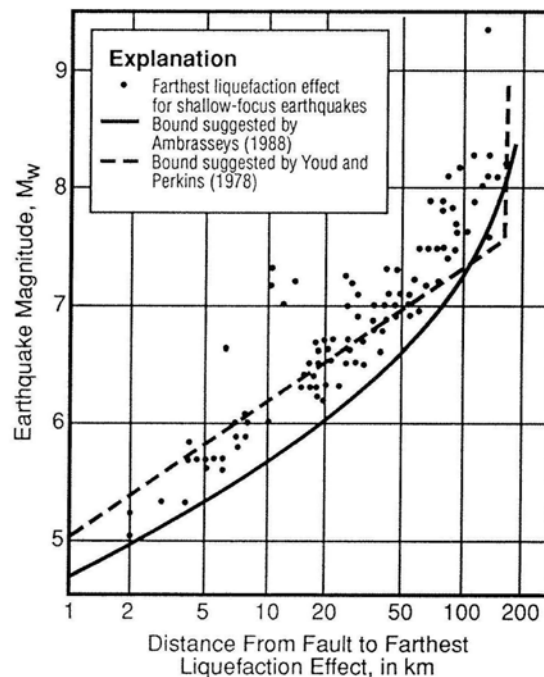

Figure 9: Distance from the Fault to Farthest Liquefaction Observed vs Earthquake Magnitude (Youd, 1991)
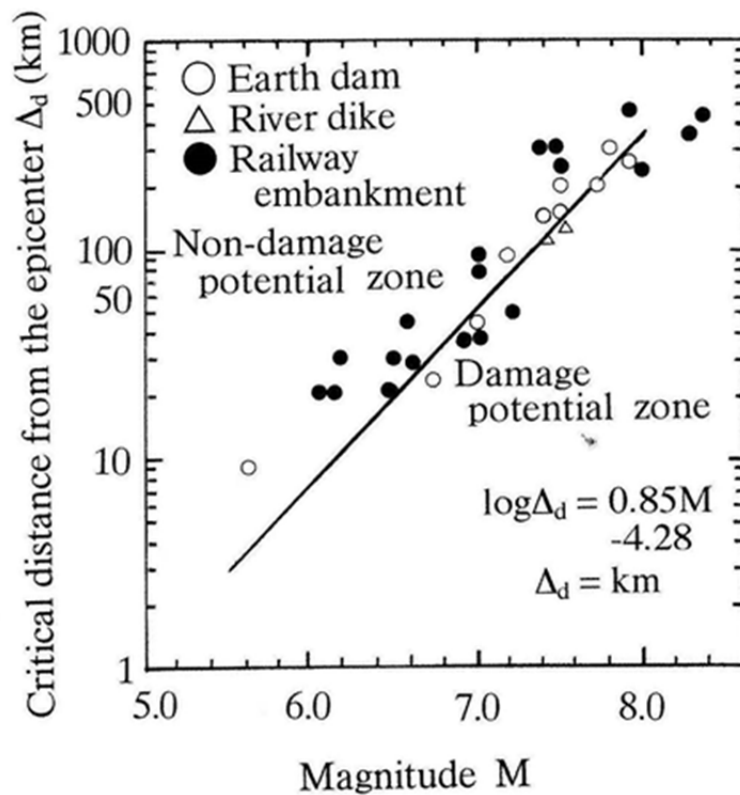
Figure 10: Distance from Earthquake Epicenter vs Earthquake Magnitude Causing Damage to Earth Structures (Tani, 1996)

The threshold level of ground surface shaking to cause soil liquefaction and induce ground displacements of significance is generally estimated to be about 0.1 g for most locations in Southwestern British Columbia. This threshold level of shaking can be caused by a number of different pairs of "Earthquake Magnitude - Epicenter Distance":

Table 6: Earthquake Magnitude- Epicenter Distance Pairs for Dike Inspection

M_w 5	occurring within 1 km of the dike
M_w 6	occurring within 8 km of the dike
M_w 7	occurring within 50 km of the dike
M_w 8.2	occurring within 180 km of the dike

Often, the US Geological Survey and Geological Survey of Canada use the Moment Magnitude (denoted by M_w) to quantify the energy released by an earthquake. There are other earthquake magnitude scales that have been and are being used to describe the energy released by an earthquake; Richter Magnitude (M_L), Body Wave Magnitude (m_b), Surface Wave Magnitude (M_s), Japanese Meteorological Agency Magnitude (M_{JMA}), *etc.* Most of these magnitude scales correlate well for earthquakes that vary between magnitude M5 and M7. A typical chart correlating the different earthquake magnitudes to the Moment Magnitude is shown in Figure 11.
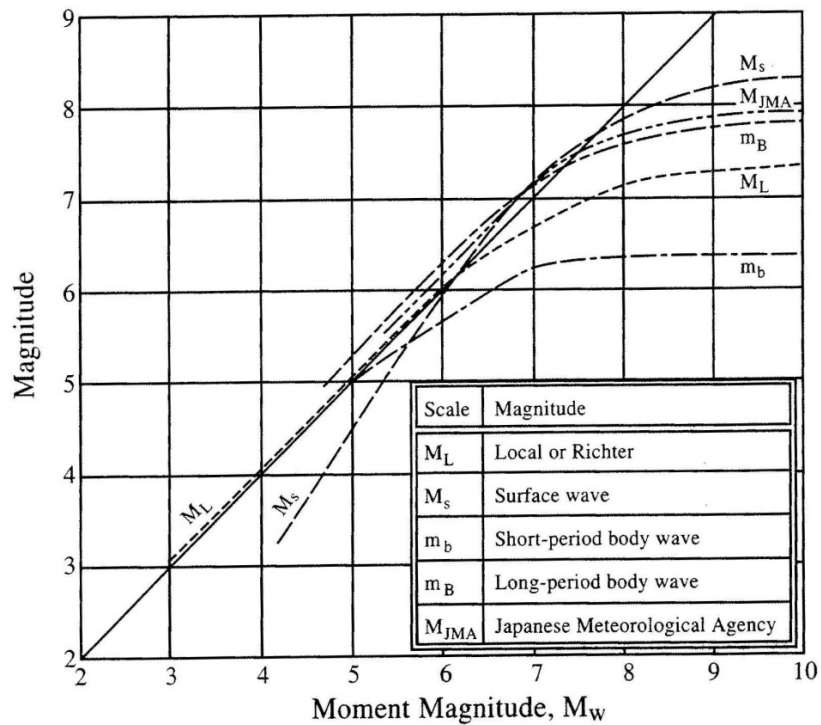
Figure 11: Correlation of Moment Magnitude with other Magnitude Scales (Youd & Idriss, 1997)

The Modified Mercalli Intensity (MMI) Scale is one of the first scales that were developed to assess the perceived intensity of earthquake shaking (as felt by people) and damage. Peak ground accelerations are approximately correlated to the MMI scale in the following manner (Bolt, 1988).

Table 7: Modified Mercalli Intensity Scale Correlated to Damage Potential and Peak Ground Acceleration

Scale	Shaking Intensity	Damage	PGA
MMI-V	Moderate	Very Light	0.03–0.04 g
MMI-VI	Strong	Light	0.06–0.07 g
MMI-VII	Very Strong	Moderate	0.10–0.15 g
MMI-VIII	Severe	Moderate to Heavy	0.25–0.30 g
MMI-IX	Violent	Heavy	0.50-0.55 g
MMI-X	Extreme	Very Heavy	> 0.60 g

The MMI scale can be used to initiate evaluation of the post-seismic dike integrity for magnitudes equal to or higher than MMI-VII.

Early information of the occurrence of an earthquake will likely be available in terms of the earthquake magnitude and epicentral location. The use of the "Earthquake Magnitude – Epicenter Distance" pairs described previously are more useful than the MMI scale, to trigger dike integrity inspections. A typical event notification from the US Geological

Survey is reproduced below for information:

== PRELIMINARY EARTHQUAKE REPORT ==

***This event supersedes event AT00992705.

Region: VANCOUVER ISLAND, CANADA REGION

Geographic coordinates: 50.218N, 129.535W

Magnitude: 6.1 Mw

Depth: 10 km

Universal Time (UTC): 28 Aug 2008 12:37:35

Time near the Epicenter: 28 Aug 2008 05:37:35

Local standard time in your area: 28 Aug 2008 12:37:35

Location with respect to nearby cities:

153 km (95 miles) WSW (251 degrees) of Port Hardy, BC, Canada

308 km (191 miles) W (276 degrees) of Campbell River, British Columbia, Canada

412 km (256 miles) WNW (302 degrees) of Neah Bay, WA

472 km (293 miles) WNW (285 degrees) of Vancouver, British Columbia, Canada

Post-seismic dike integrity evaluation work should be triggered for any of the earthquake magnitude – epicenter distance pairs identified in Table 6.

Details on dike inspection and subsequent engineering evaluations are presented in Appendix B.

18. Post-Earthquake Emergency Repair of Dikes

The post-earthquake breach of a dike can occur due to overtopping, piping and seepage, and sliding and foundation failure.
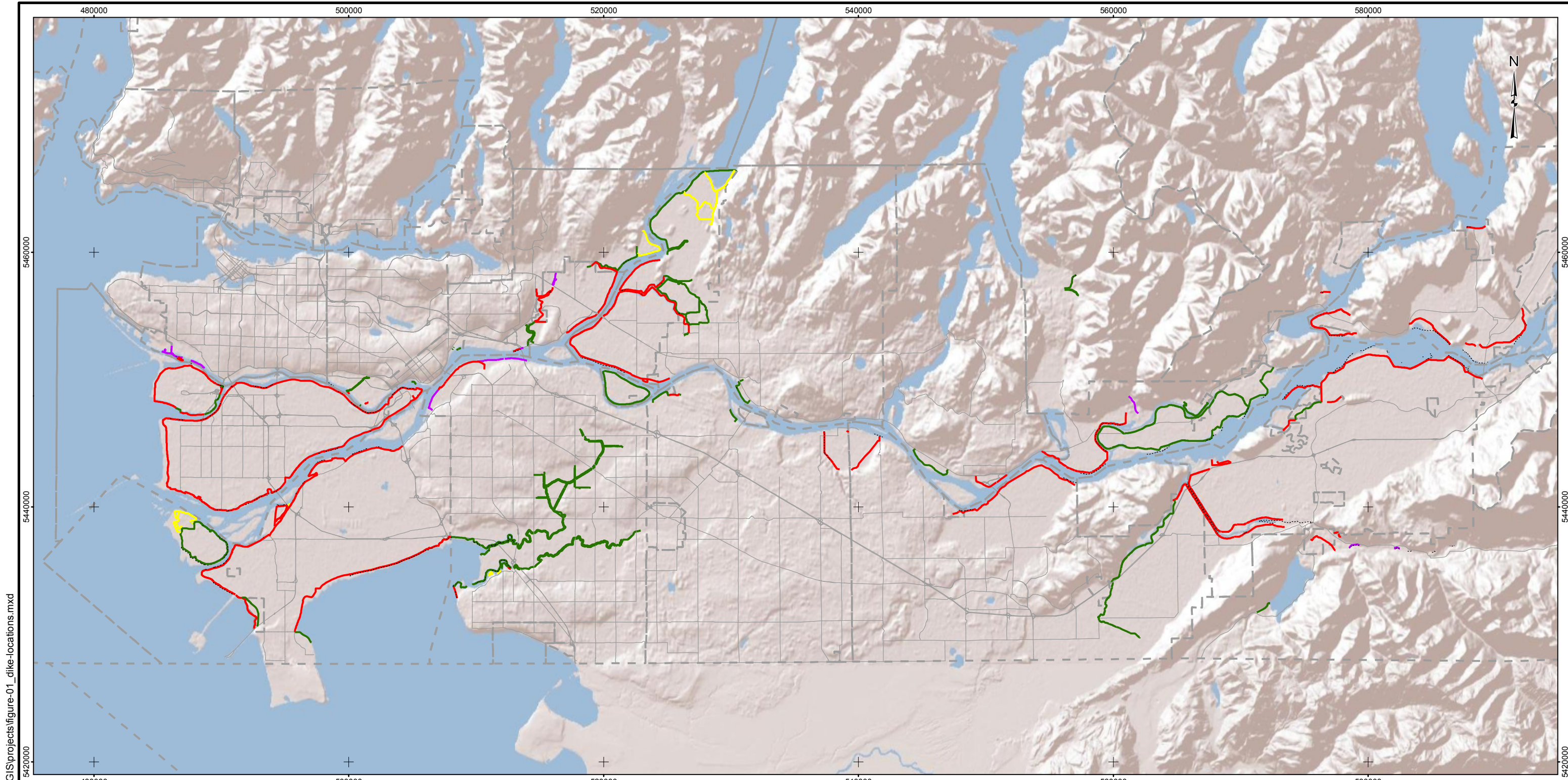
Infilling by placing and compacting of suitable fills into the zone of disturbance/failure caused by the breach, placing sand bags, and the use of water-filled fabric tubes are considered to be practical remedial measures for emergency dike repair. Details are provided in Appendix D.

There is a low probability of a seismic event with a long return period (*i.e.*,. a rare event) and large peak ground accelerations occurring concurrently with the high water levels in the Fraser River during the spring snowmelt freshet.   The post-earthquake emergency repair measures are intended to provide flood relief over the period in between the seismic event and freshet time when the available time to implement permanent dike rehabilitation and strength is inadequate.

19. References

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4. Youd, T.L. and Idriss, I.M. eds. 1997. Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York, 276 p.

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10. Seed, H. B., and Idriss, I. M., 1971, "Simplified procedure for evaluating soil liquefaction potential." J. Soil Mechanics and Foundations Div., ASCE 97(SM9), 1249–273.

11. Idriss, I.M. and Boulanger, W. 2004, "Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes." Proceedings of the 11th international Conference on Soil Dynamics and Earthquake Engineering and the 3rd International Conference on Earthquake Geotechnical Engineering, Univ. of California, Berkley, CA, 2004.

12. Bolt, Bruce A (1988), "Earthquakes", W. H. Freeman and Company.

13. Wood H. O. and Neumann F. (1931), "Modified Mercalli Intensity Scale of 1931", Seismological Society of America Bulletin, Vol. 53, No. 5, pp. 979-987.

LEGEND

········· RIP RAP

───── STANDARD DIKE

───── NON-STANDARD DIKE

───── OTHER FLOOD CONTROL WORKS

───── FISH AND WILDLIFE WATER RELATED STRUCTURE

───── MAJOR ROAD

⌐ ¬
└ ⌐ MUNICIPAL BOUNDARY

6   0   6

SCALE 1:300,000   KILOMETRES

BRITISH COLUMBIA

MINISTRY OF FORESTS, LANDS AND NATURAL RESOURCE OPERATIONS FLOOD SAFETY SECTION

PROJECT
SEISMIC DESIGN GUIDELINES FOR DIKES

TITLE
DIKE LOCATIONS IN THE FRASER VALLEY

Golder Associates
Greater Vancouver Office, B.C.

PROJECT	10-1447-0333	FILE No.		
DESIGN	BM	13JUN11	SCALE AS SHOWN	REV. 0
GIS	CDB	13JUN11		
CHECK	BM	09AUG11		
REVIEW	UA	09AUG11	**FIGURE 1**	

LEGEND

PUMP STATION

TREATMENT PLANT

RIP RAP

STANDARD DIKE

NON-STANDARD DIKE

OTHER FLOOD CONTROL WORKS

FISH AND WILDLIFE WATER RELATED STRUCTURE

PIPELINE

MAJOR ROAD

MUNICIPAL BOUNDARY

RELATIVE LIQUEFACTION SUSCEPTIBILITY RATING

LOW TO VERY HIGH

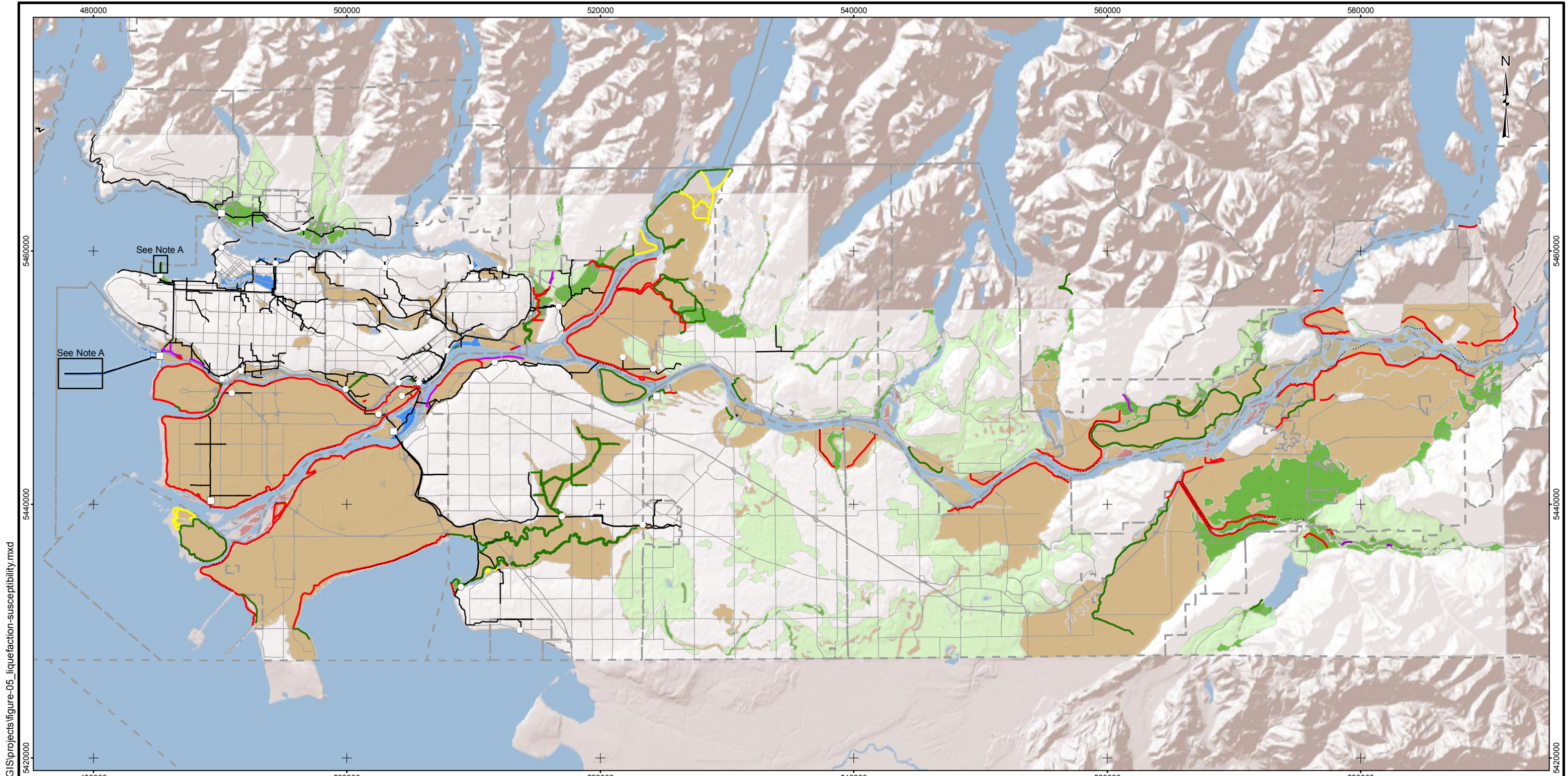VERY HIGH

HIGH

MODERATE

LOW

VERY LOW

REFERENCE

LOWER MAINLAND DIKE INVENTORY DATA OBTAINED FROM THE MINISTRY OF ENVIRONMENT (MOE), WATER STEWARDSHIP DIVISION, FLOOD HAZARD MANAGEMENT, COASTAL REGION. SURFICIAL GEOLOGY DATA OBTAINED FROM THE GEOLOGICAL SURVEY OF CANADA. ROAD AND MUNICIPAL BOUNDARY DATA OBTAINED FROM DMTI SPATIAL INC. PUMP STATION, TREATMENT PLANT, AND PIPELINE DATA OBTAINED FROM GREATER VANCOUVER SEWERAGE AND DRAINAGE DISTRICT. SHADED RELIEF OBTAINED FROM ESRI. DATUM: NAD83 PROJECTION: UTM ZONE 10

NOTE

A. INFERRED SOIL CONDITIONS BASED ON GSC SURFICIAL GEOLOGY MAP.
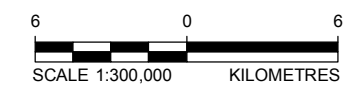
SCALE 1:300,000   KILOMETRES

BRITISH COLUMBIA

MINISTRY OF FORESTS, LANDS AND NATURAL RESOURCE OPERATIONS FLOOD SAFETY SECTION

PROJECT

SEISMIC DESIGN GUIDELINES FOR DIKES

TITLE

LIQUEFACTION SUSCEPTIBILITY

Golder Associates
Greater Vancouver Office, B.C.

PROJECT	10-1447-0333	FILE No.		
DESIGN	BM	13JUN11	SCALE AS SHOWN	REV. 0
GIS	CDB	13JUN11		
CHECK	BM	09AUG11	FIGURE 5	
REVIEW	UA	09AUG11		

APPENDIX A

Literature Review of Seismic Design Guidelines for Flood Protection Dikes

A1.0 INTRODUCTION

Using dikes as a form of flood protection is common in many areas of the world. However, the art of dike design and construction has been developed and initiated predominantly in three countries, namely the USA, Japan, and the Netherlands. In particular, the design issues relating to earthquake performance of dikes is limited to mainly the west coast and the central Mississippi area of the USA and Japan. A review of information relating to flood protection dikes indicates that a plethora of information exists in the USA and to some extent in both the Netherlands and Japan.

A2.0 EVOLUTION OF DESIGN PHILOSOPHY FOR FLOOD PROTECTION DIKES

Many dikes were originally constructed without major engineering design input and really developed as grass roots construction projects to prevent flooding. In the USA, the age of the dikes throughout the country can vary from recent upgrading of about 50 years ago to in excess of 100 years where very little additional work has been done over that period. In the USA, where lesser standards have been historically applied, it is evident that better safety standards are required and more formalized approaches are necessary. The Netherlands, USA and Japan have recently started to formalize their approach to design and upgrading of dikes.

Historically, dikes have been designed using traditional design methods such as slope stability analysis and factor of safety criteria considering higher than average loading applied to the dike and lower resistances (soil strengths) within the dike structure and underlying foundation soils. These factors of safety have inherently built in acceptable values of societal risk although not specifically defined. More recently, however, societal risk is being assessed by following the approaches described below:

■ Using a performance-based approach by adopting a classification system for the dikes based on importance; establishing multiple levels of ground shaking varying from frequently occurring to rarely occurring ground motions; and thereafter establishing performance expectations for each level of ground shaking and for each importance category of dikes (see Figure A-1, Note: 1000 gal = 1 g); and

■ Using risk assessment and risk management by specifying probability of particular types of failure or, alternatively, the reliability of particular components with respect to various functions.

Figure A-1: Evolution of Design Concepts and Methods of Analysis (Ishihara, 2009).

A2.1 Performance-Based Design Philosophy

In Japan, the Ministry of Land Infrastructure and Transport developed a policy for "Performance–Based Seismic Design Criteria for River Facilities" (*ref. Sugita and Tamura, 2007*). This policy has formulated an approach where the dike system is classified into simple categories on whether the structure is important for flood control and whether or not alternative measures are available if the dike was no longer functioning. The required performance level of the dike is then defined and the relevant return period for design established based on the importance of the structure. Calculations must then be carried out for stability and settlement to assure that the dike will continue to function during and/or following the design level earthquake accelerations as per the consequence classification. They identify two different levels of ground shaking (earthquake accelerations) and three levels of performance expectations:

Earthquake Levels

Level 1 Earthquake: Ground motions that occur with a high probability during the service life of the structure; and

Level 2 Earthquake: Maximum credible ground motions at the site

Performance Levels

Performance Level 1:   The structure or component does not lose soundness as a river facility;

Performance Level 2:   The structure retains function as a river facility against the water level defined for assessing seismic performance; and

Performance Level 3:   The structure experiences limited damage and repair to damages can be made within a short time.

Typical criteria for seismic displacements and remaining free-board along with the post-event flood protection capability of the dike (or performance), proposed by the US Army Corps of Engineers for urban and urbanizing areas, are summarized below *(Koester, 2010A)*:

Amount of Deformation	Significant Damage to Internal Structures	Remaining Freeboard for Post-Seismic Evaluation	Post-Seismic Protection Ability
< 0.3 m	No	>0.3 m	Probably Uncompromised
0.3 to 0.9 m	Possibly	>0.3 m	Possibly Uncompromised
0.9 to 3 m	Likely if existing	None	Likely Compromised
Unlimited	Yes	None	Compromised

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A2.2 Risk-Based Design Philosophy

Modern engineering practice such as probabilistic, modeling, and failure mode analysis along with risk considerations, described in the latter approach, are now the normal approach to dam design and construction. These techniques indicate that better safety standards and practices are generally needed. The risk of flooding in different areas has resulted in different standards being adopted depending on the risk that the community is prepared to accept. For example, loss of dike protection in the Netherlands can be catastrophic for the country, the amount of risk that the country is prepared to assume has to be low and dictates that the design flood levels accepted for dike design are much higher than in other areas.

Under current practice in the USA, the approach to dike design considers a combined probabilistic and deterministic approach that utilizes conditional risk and uncertainty based on water levels for carrying out deterministic dike analyses. There are efforts currently underway to develop new risk-based approaches to dike design. The approach to design is contained in the US Corps Engineering Design Manual EM 1110-2-1913 "Design and Construction of Levees" (US Army Corps of Engineers, 2000). This approach is currently under development and not as yet formalized. This design manual does not specifically require consideration of earthquakes "because of the low probability of earthquake coinciding with periods of high water" but makes a statement that "depending on the severity of the earthquake and the importance of the levee, seismic analysis to determine liquefaction susceptibility may be required. In high earthquake zones this analysis might therefore be required".

Recommendations from the National Committee on Levee Safety has indicated that a classification system for dike design be adopted and further states that the current classification system is interim with the intent that there will be a future need to classify dikes by potential hazard and risk in order to set appropriate design criteria and requirements. An example of a hazard potential classification proposed by the National Committee on Levee Safety is as follows:

Table A-1: Example Hazard Potential Classification (Ref. Report to US Congress on Recommendations for National Levee Safety program, 2009)

Hazard Potential Classification	Number of People Potentially Inundated	Number of People Potentially Inundated to Depths $\geq$ 3 feet	Additional Considerations
High	$\geq$ 10,000	$\geq$ 10,000	Includes areas of consequence where critical life safety infrastructure is at risk (e.g. major hospitals, regional water treatment plants, and major power plants)
Significant	> 1,000	< 10,000	Includes areas of consequence where the number of people potentially included is low, but there may be significant potential for large economic impacts or losses
Low	< 1,000	0	-

A3.0 RISK-BASED APPROACH

A risk-based approach is considered a better method to both assess the requirements of flood control dikes and prioritize available upgrading and maintenance funds. This approach will allow funds to be allocated in a manner consistent with the potential risk of damage and loss of life. Future risks to dikes come from the following major components:

- The likelihood of floods;

- The likelihood that the dikes will not perform as designed or intended; and

- The consequence as a result of failure on people property and environment.

Tolerable societal risk is inherent in all we do as individuals. We do not in general attempt to quantify these risks and passively accept them in going about our daily lives. However, mathematically they can be quantified as is done for insurance actuary purposes. The process that puts all components of risk as discussed above into a societal context enables better decision making in comparison with other published risk guidelines; e.g. the airline industry, dam safety programs, transportation, and medicine.

In seismic considerations, the general risk assessment procedure consists of the following:

- Probability of ground motions at the dike site;

- Probability of dike rupture due to failure; and

- The expected loss (in $$) due to failure.

The probability of dike failure is a function of dike slope deformations as shown below on Figure A-2 (*Rosidi, 2007*)
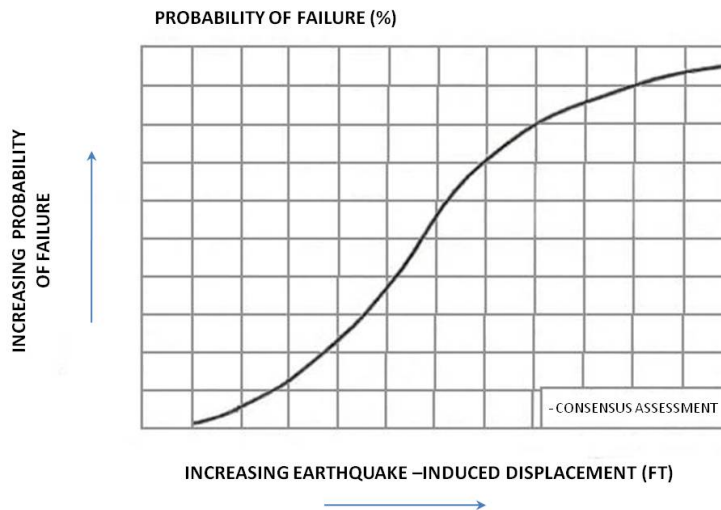

PROBABILITY OF FAILURE (%)

INCREASING PROBABILITY OF FAILURE

- CONSENSUS ASSESSMENT

INCREASING EARTHQUAKE –INDUCED DISPLACEMENT (FT)

Figure A-2: Example of a Levee Fragility Curve (Rosidi 2007).

The conditions that define failure include, piping (internal erosion) overtopping, cracking, slope, slope sliding and settlement. The suspected damage to a dike under earthquake motions is estimated by integrating the probabilities of failure considering all the various conditions that contribute to a failure such as site conditions, dike construction materials, water levels, and earthquake ground motions. A logic tree approach is frequently used as shown on Figure A-3.

SEISMIC VULNERABILITY EVALUATION

Section	PGA	Operating Scenario	Liquefaction Controls Response	Field Acceleration	Max Ave. Acceleration	Ave. Acceleration Time History	Displacement	Failure Probability

Section n^{th}

0.14 g
43 -yr RP

0.33 g
475 –yr RP

0.52 g
2,500 –yr RP

Low tide high reservoir
weight = 0.34

High tide low reservoir
weight = 0.66

Yes — $K_y(liq)$ — K_{max}

No — $K_{y(non-liq)}$ — K_{max}

$K_{ave.}^{-1}$ Disp-1   FP-1
weight = 0.5

$K_{ave.}^{-2}$ Disp-2   FP-2
weight = 0.5

Mean Failure Probability for segment n^{th}

$$= \sum_i \quad \sum_j (PGA)_i (Oper)_j \sum_k (P_{liq})[Fp(dis_{ijk})] + (1 - P_{liq})[Fp(dis_{ijk})]$$
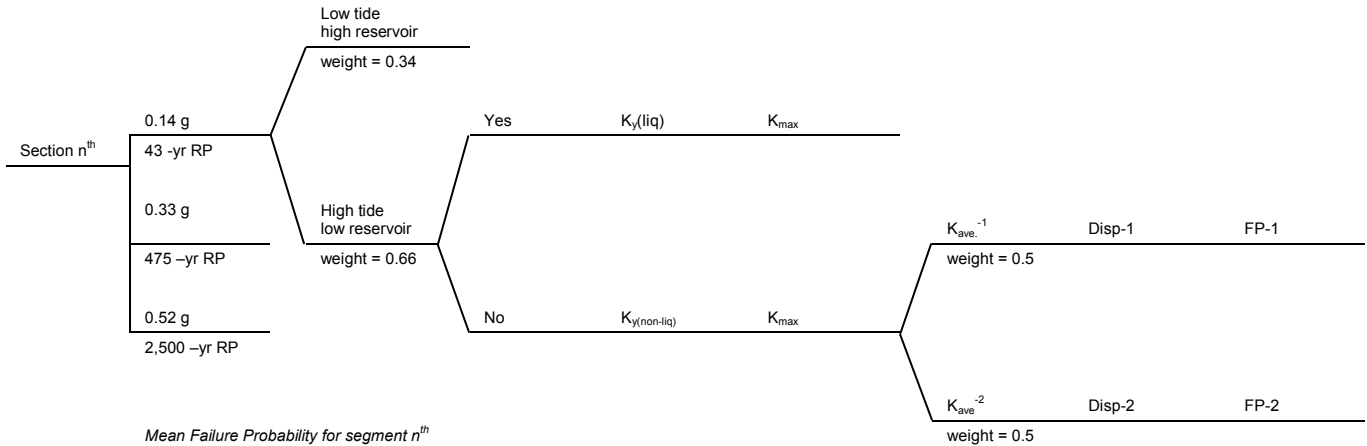
Figure A-3: Logic Tree Approach for Integrating the Results of Various Assumptions (Rosidi, 2007).

The intent of tolerable risk assessments adopts the concepts of Equity and Efficiency as shown on Figure A-4.

Bureau of Reclamation f-N chart for Displaying Probability of Failure, Life Loss, and Risk Estimates-Portrayal of Risk

- **Equity** – The principle, which holds that the interests of all are to be treated with fairness and that individuals and society have the right to be protected (ICOLD);

- **Efficiency** – In relation to society's use of resources, that principle, which seeks to gain greatest benefit from the available resources; and

- This leads to the notion that tolerable risk should consider both **societal** and **individual** risks as an integral part of the framework for managing risks.

Figure A-4: Concept of Equity and Efficiency (NCLS, 2009).

A4.0 FLOOD LEVEL AND FREEBOARD

Design criteria for flood levels are variable throughout the world. In particular the design flood levels in the USA (currently FEMA considers a 1:200-year design flood level as a minimum, Halpin et al. 2009) are much lower than in the Netherlands where a minimum 1:1,250-year design flood level for river dikes has been adopted (Ten Brinke, W.B.M, et al, 2010). The dike performance is assessed using the traditional design criteria in terms of factors of safety.

Draft guidelines prepared by the California Department of Water Resources (August, 2008) for Urban and Urbanizing Area State-Federal Project Levees (FEMA, 2008), consider levees in the deltas differently, where there is a high likelihood of having coincidental high water and earthquake loading. Levees which infrequently experience loading from high water are proposed to be evaluated using typical water surface elevations.

In addition to the flood level, freeboard requirements also vary throughout the various countries. Recent concern about sea level rise from global warning has resulted in significant upgrading of the freeboard of the Dutch dikes. In the USA, FEMA has adopted the National Flood Insurance Program (NFIP) regulations that require no dike freeboard to be less than 0.6 m (2 feet) and in general 1 m (3 feet).

The Japanese performance-based design criteria consider the "highest daily water level under usual conditions" in the seismic performance of river facilities. This water level is established considering that the earthquake and flood would not occur simultaneously. Around river mouth or delta areas, the water level considered would include the combination of the highest tidal water level, the wind waves and a tsunami wave.

A5.0 SEISMIC SHAKING

Seismic performance of dikes has generally been ignored in most countries in the past, except in specific cases.

The USA procedures have in general neglected to consider seismicity except in California and in the Mississippi Delta (FEMA, 2008). In Japan there is a much greater appreciation for seismic design of dikes and consequently there has been more focus on seismic considerations in dike design (Koester 2010B). In the Netherlands, the country is considered to be very benign with respect to earthquakes and because of low probability, seismic effects on dikes are not considered in their design criteria.

In the USA, current thinking for dikes that infrequently experience loading from high water is to accommodate the earthquake risk by providing efficient and prompt post-earthquake dike repair and flood response. Ground remediation for liquefaction prevention might be considered in some extreme cases (FEMA, 2008). For repair and improvement work associated with urban and urbanizing area levees, seismic ground motions having a return period of 200-years are being proposed. Furthermore, repairs for improvements primarily for the purpose of seismic strengthening are generally not considered to be justifiable for levees that are subjected to only seasonal high water loading.

Draft engineering and design guidelines developed by the US Army Corps of Engineers for certification of levee systems for the national flood insurance program (NFIP) consider that levee systems located in regions which experience strong ground motions from earthquake activity should consider seismic effects on the stability of dikes. They have adopted ground motions with a return period of 475-years for seismic design with the provision that if the PGA is not greater than 0.15 g, no evaluation is required. For PGAs greater than 0.15 g, the levee and its foundation are to be checked for liquefaction using a simplified procedure such as the Simplified Seed Method (Seed and Idriss, 1971).

In Japan, the requirements are that dikes shall be assessed considering the appropriate earthquake design motions consistent with the performance level deemed appropriate for the dike under consideration. River facilities such as levees should consider seismic ground motions that have a "high probability of occurrence during the service life" as well as "maximum credible ground motions" with expected performance descriptors attached to each level of shaking.

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APPENDIX B
Earthquake Inspection Checklist

B1.0 INTRODUCTION

An earthquake may cause failure or severe damage to a dike and its appurtenant structures. In the event of damage, immediate action may be necessary to prevent further weakening or catastrophic failure of the structure. Accordingly, all diking districts personnel should be carefully instructed in the procedures to be followed if an earthquake should occur that produces motions of intensity sufficient to possibly cause damage.

There are two phases of the inspection procedure:

1)   An immediate inspection; and

2)   Follow-up inspections.

The general procedures described herein may be used as guidelines.

B2.0 INSPECTION IMMEDIATELY FOLLOWING AN EARTHQUAKE

If an earthquake is felt on/or near the dike, or has been reported to have occurred, with a moment magnitude of, M_w 5 or greater within 1 km, M_w 6 or greater within 8 km, M_w 7 or greater within a 50 km, M_w 8+ or greater within 180 km radius from the site, follow these procedures:

■   Immediately make an overall inspection of the dikes. Check the dike for sloughs, slides, cracks, displacements, settlements, sinkholes, springs or seeps, and other signs of distress.

■   If the dike appears to be damaged to the extent that there is increased or new flow passing downstream, report immediately.

If visible damage has occurred but, in the best judgment of the inspector, is clearly not serious enough to cause failure of the dike, the nature, location, and extent of damage should be observed and the rate of any change recorded.

Thoroughly inspect the damage using the following checklist:

■   Check the crest and both faces of the dike for cracks, settlement, displacement or seepage;

■   Check the land side of the dike for visible landslides, new springs and sand boils;

■   Check drains and seeps for increased flow or stoppage of flow; and

■   Check structures and pumping equipment for misalignment and distress.

If no apparent damage has occurred to the dike embankment, or appurtenant structures, make a "No Damage" report. Continue to inspect and monitor the facilities for at least 48 hours. Some damage to structures may not be readily apparent during an inspection immediately following an earthquake. It is possible that settlement of structures, reactivation of old slides, or development of new slides or springs/seeps may not have occurred during ground shaking, but could appear after the initial inspection. A secondary inspection should be carried out two (2) weeks to a month after the initial inspection.

B3.0   ENGINEERING FOLLOW-UP INSPECTION

In the event that an inspector reports that damage has occurred or a dike has been severely shaken, qualified engineering and geologic personnel should be dispatched as rapidly as possible to the site to make a technical evaluation of the extent of damage and the degree of hazard it presents. The members of such an inspection team should be familiar with the possible modes and causes of failures, and should also be familiar with the main features of the project.

B3.1   Possible Modes and Causes of Dike Failures

The members of an inspection team must be aware of the modes of dike failures, both static and dynamic. Team members should have had some training in dike failures, both static and dynamic to reinforce their engineering understanding of why failures occur.

Some of the more common causes of static and dynamic failures of dikes and examples of adverse conditions are discussed in this section. Adverse conditions that can lead to failure are categorized as follows:

Failure Category	Causes
Foundation Instability	Liquefaction
	Slides
	Removal of solid and/or soluble materials by water subsidence differential settlement
	Joint openings and/or grout curtain rupturing movement on faults under or adjacent to dikes
Defective Flood Control Structures	Obstructions
	Damaged structure walls, slabs
	Crushed or deteriorated concrete
Embankment Dike Defects	Liquefaction
	Slope instability
	Excessive seepage
	Removal of solid and/or soluble material surface soil erosion caused by overtopping embankment settlement producing inadequate freeboard for required spillway capacity cracks or sinkholes

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B3.2   Foundation Deficiencies

These deficiencies are associated with the geotechnical condition of the foundation materials or with the foundation design of the dikes. Differential settlement, slides, excessive pressures, weak layers or zones, and inadequate control of seepage are all potential failure mechanisms within a foundation. Foundation conditions, which have low shear strength or seams of weak material such as shales, bentonitic materials, or faults, can result in sliding of the foundation and embankment. Also, thin highly permeable layers in the foundation, which have no provisions for pressure relief allow transmission of excessive uplift pressures and cause sliding.

Seepage through the dike foundation can cause piping or internal erosion of soil. Such removal of foundation material forms voids, which can increase until a portion of the remaining unsupported material collapses and failure of a section of the foundation occurs. Water can also cause a breakdown of some foundation materials such as shales, or reduce the shear strength of bedrock.

Some of these weaknesses can be identified by visual examination of the foundation conditions. Visible cracks in a dike embankment can be indicative of foundation movement. Visual evidence of piping is sediment in the seepage water.

B3.3   Flood Control Outlet Works

Many adverse conditions such as obstructions to the flow, structural weaknesses, or faulty drains can be identified by visual examination. Structural failure in a conduit or, other structure could obstruct the flow in the system. Loss of the power source to operate facilities such as pumps may also present operational conditions, which compromise the safety of the dike.

Slides from the slopes above flood boxes *etc.* can block approach channels and also damage intake structures and associated mechanical equipment.

Cracking and movement of concrete structures may indicate distress.

B3.4   Seepage

Uncontrolled seepage through the dike embankment or foundation can result in excess pore pressures which weaken the soil mass and may cause springs, boils, or slope failures and/or the movement of soil particles to unfiltered exits, creating voids which can lead to a piping failure. The "pipes" or "tunnels" under a dike also can cause the collapse of surrounding materials. This can then lead to the formation of settlement cracks or, ultimately, to breaching of the embankment.

B3.5   Detective or Inferior Materials

Low-density, saturated, cohensionless soils in an embankment or foundation can experience liquefaction due to an increase in pore pressure and loss in shear strength when subjected to earthquake-induced shear stresses. Depending on a variety of factors, including material properties and in-place conditions, pre-earthquake stress conditions, and magnitude and duration or seismically-induced stresses, the dike or its foundation can exhibit instability, settlement, and subsequent loss of freeboard.

B3.6   Appurtenant Structures

All appurtenant structures that could affect the safe operation of the dike should be examined.

Approach channels are usually submerged and may require special underwater investigation. Channel erosion protection such as rip rap should be checked to determine if the protection is still functional.

All channels should be examined for evidence of sinkholes, boils or piping and should provide satisfactory clearance around flood control and pumping structures so the structures can operate as designed.

Fill adjacent to flood control structures should be examined for subsidence caused by soil movement, and contacts between the fill and the structures should be examined for evidence of piping or voids. Cut or fill slopes adjacent to the structures should be examined for unstable conditions such as sloughs, slides, debris and the like.

B4.0   REFERENCES

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APPENDIX C

Failure Mechanics and Ground Treatment Methods

C1.0 INTRODUCTION

In addition to the traditional static and hydraulic modes of failure routinely reviewed for new dike construction and/or dike upgrades, a review of the potential modes of dike failure that may be generated by earthquake induced ground motions should be considered as part of the seismic evaluation of the dike stability. The type of failure mechanism that may be anticipated during and/or following a seismic event will be dependent on several factors including, but not limited to:

- The nature/distribution of the soil deposits contained within the dike, beneath and beyond the dike;

- The strength, stiffness and sensitivity of the materials within the dike as well as within the foundation deposits;

- The geometric profile of the dike as well as the topographic and bathymetric surface profiles both upstream and downstream of the dike; and

- The level of water retained by the dike as well as groundwater levels (phreatic surface) within the dike and the adjacent land.

The above information coupled with the required performance criteria stipulated for specified or select seismic hazards would be relied upon for assessing potential failure mechanisms that may develop in response to seismic induced motions along the dike. Some examples of failure mechanisms that may be caused by earthquake induced ground motions are:

- A disruption or stepwise shearing of the dike caused by slip or thrust fault movement within the foundation materials;

- Significant loss of strength of soils within, beneath and/or beyond the dike structure as a result of liquefaction, cyclic softening and/or cyclic mobility;

- Bearing capacity failure of the dike due to significant weakening of the foundation deposits;

- Slope failures induced by ground motions and/or weakening of the underlying deposits;

- Translational sliding of the dike on weak foundation soils;

- Piping failures through cracks induced by ground movements;

- Overtopping of dikes due to seiches and/or tsunamis;

- Overtopping of dikes due to water course obstructions such as a slide; and

- Loss of free board due to settlement or differential ground movements resulting from bearing or slope failure, or other ground induced differential movements.

A brief discussion of various earthquake induced failure mechanisms that may affect the integrity of dike systems are presented below.

C1.1   Fault Movement

The energy released from movement along a fault is the cause of most earthquakes. There are several different types of faults within the earth's crust that include:

1) Dip-slip fault – where relative movement along the discontinuity is approximately vertical;

2) Strike-slip fault – where relative movement along the discontinuity is generally horizontal; and

3) Oblique – slip fault has components of both the strike and dip slip fault.

The potential damage to a dike system generated by any of the three types of fault movements described above would likely manifest as a "stepwise" or shear displacement of the structure which can be in the order of several metres long.

C1.2   Liquefaction/Cyclic Softening/Cyclic Mobility

C1.2.1   Liquefaction

Liquefaction represents a phase change within a material from a solid or gas to a liquid resulting from change in external pressures. In soil deposits, liquefaction occurs within saturated, predominantly granular soil materials when water contained within the soil pores is pressurized to a level consistent with the total overburden stress within the soil deposits. Such conditions can prevail under static conditions when hydraulic gradients within the deposits are high. An example of static liquefaction is "quick sand" or "sand boils" that are frequently associated with excessive hydraulic gradients at, or near the downstream toe of the dikes. Under dynamic conditions, excess pore water pressures are created by a volumetric change occurring within the soil structure resulting from contractive behaviour of the deposits during cyclic shaking. Depending on the extent of liquefaction and geometric profiles of the surface conditions, liquefaction can lead to local (bearing) failure and/or global (flow) failure of the deposits, particularly where imposed static shear stresses are greater than the post liquefied (undrained residual) strength of the deposits.

Similar to contractive granular deposits, highly sensitive deposits containing a mixture of clays and silts may also experience "liquefaction type" failures. Typical characteristics of highly sensitive fine-grained deposits include a high to very high moisture content and a significant strength variation between small strain (peak) strength and large strain (remoulded) strength. Examples of sensitive fine-grained deposits are "quick" clays, such as the Leda Clays and Champlain Clays found in Eastern Canada and the Still Creek Clay deposits in Burnaby, BC. Similar to liquefaction, a sudden, abrupt loss of strength resulting from cyclic shaking can lead to bearing and/or flow type failures.

C1.2.2   Cyclic Softening

Cyclic softening is generally associated with temporary (transient) strength loss of deposits during cyclic shaking. Deposits which generally exhibit this type of strength reduction are fine-grain, predominantly silt deposits. Unlike highly sensitive clays and contractive (loose) granular deposits, cyclic straining of silts generally do not result in flow slides associated with liquefaction, but may experience large deformations during pore pressure build-up. The main difference between liquefaction and cyclic softening is that deformations do not continue after cyclic loading ceases. Deformations predominantly occur when the combined static and inertial (earthquake induced) stresses exceed the transient strength loss of the silt deposits. As deformation occurs, dilative (opposite of contractive) conditions develop resulting in a reduction in pore pressure allowing for strength gain to at, or near pre-earthquake conditions. Consequently, deformations generally manifest as a progressive "ratcheting" type of movement.

C1.2.3   Cyclic Mobility

Cyclic mobility is generally associated with loose to dense unsaturated granular deposits, firm to hard, insensitive fine-grained (silty clay, clayey silt) deposits and/or dense, saturated deposits. Cyclic mobility occurs when continued cyclic shear strain results in a dilative response caused by limited pore pressure build-up in unsaturated soil conditions and/or following a brief contraction of the soil particles, such as the case with dense to very dense saturated sands. Deformations accumulate in each cycle of shear stress; however, the magnitude of the accumulated deformations are typically quite small relative to displacements generated by liquefaction and/or cyclic softening.

C1.3   Bearing Capacity Failure

Bearing capacity failure is a sudden or an abrupt loss of local support of the foundation soils as a result of static shear stress within the foundation materials exceeding the shear strength of the deposits. Under seismic conditions, bearing capacity failure may occur when liquefaction develops within the foundation materials resulting in a substantial loss of shear strength. Bearing capacity failure may be manifested as a local rotational slip of the foundation or a "punching" or plunging type failure. Both failure mechanisms can result in a significant reduction in the dike crest elevation. The predominant displacements generally occur within the vertical axis compared to the horizontal axis. Illustrative examples of bearing capacity are presented below.
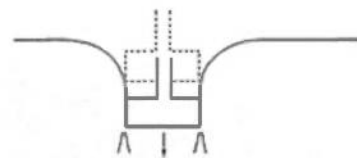

Figure C1-a: Local Bearing Failure

Figure C1b: Punching Shear Failure

C1.4   Slope Stability Failure

Similar to bearing capacity failure, slope instability occurs when the static and/or combined static and dynamic shear stresses within a sloping soil mass exceeds the net shear resistance of the deposits along a select slip plane.   Unlike bearing capacity failure, a slope failure movement generally occurs in a discreet direction potentially resulting in significant vertical and horizontal displacements, with the horizontal displacement vector generally representing the predominant movement axis.   Slope stability also generally relates to massive or global failure mechanism compared with local conditions with bearing capacity.   Consequently, the condition of the deposits at significant depth can affect slope stability.   The most common slip surfaces associated with slope stability are circular or arc shaped; however, depending on the distribution of soil types and strengths below the slope surfaces, failure slip surfaces can be also be planar or multi-planar.   The following figure presents a typical circular slip failure.
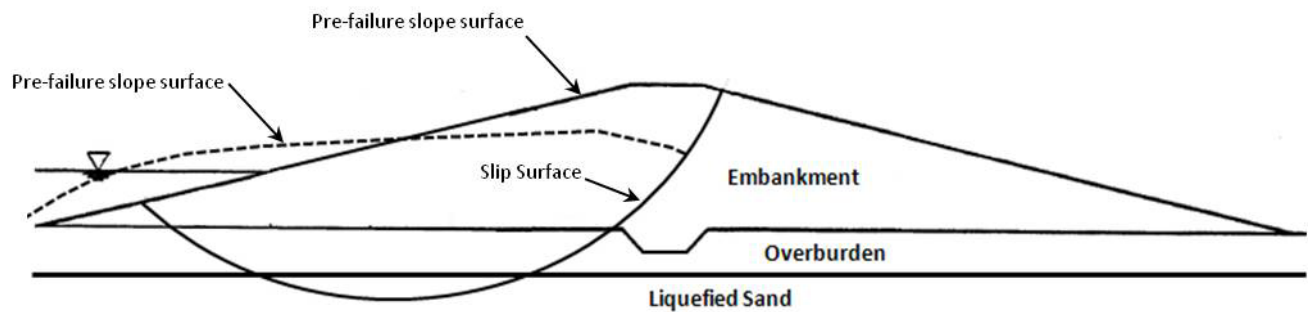

Figure C-2: Global Stability Failure, Circular Slip- Surface.

C1.5   Translational Sliding

Translational sliding is a form of slope instability that represents predominantly horizontal (planar) movement with limited vertical displacement.   Such conditions can develop within horizontally stratified deposits of variable strengths and thicknesses or zones of liquefied deposits in conjunction with topographic conditions which expose or daylight the underlying weak soil deposits.
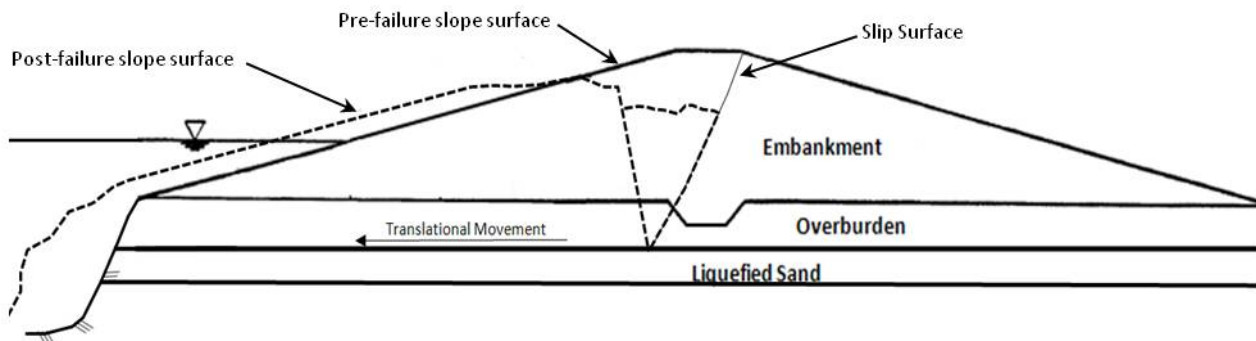

Figure C-3: Translational Failure

C1.6 Piping Failure

Failures by piping can be developed from two different processes which can be categorized as subsurface erosion induced or heave induced. Of the two types of piping conditions, subsurface erosion induced piping failures are the principal failure mechanism with respect to dikes and/or dams. Under static conditions, piping occurs when hydraulic gradients at or near the toe of a water compound exceeds unity (*i.e.* the change in total head pressure head over a unit path length exceeds unity). Piping manifest as a local spring or discharge of water containing suspended sediments. Over a prolonged period of time, internal erosion will progress towards the water compound. When the internal erosion reaches the water compound, an "effective" pipe condition is generated which can substantially increase the flow and erosion process, quickly leading to eventual collapse of the foundation deposits and the dike/dam supported by the foundation deposits.

During seismic shaking, tension cracks can develop within the dike due to differential distortions along the facility. In the instances where the tension cracks extend through the full cross section of the dike, an effective "pipe" condition is developed which allows unobstructed flows to pass through the dike where retained water levels are intercepted by the tension cracks. Similar to the static piping, flows through the dike can result is significant or substantial erosion in a relatively short period of time, resulting in failure of the dike and loss of water containment.

C1.7 Overtopping

Overtopping of the dike under seismic conditions can occur when there is a significant crest elevation decrease as a result of liquefaction induced settlement and/ or slope failure occurring within the dike. Overtopping of the dike can lead to a breach in the structure as a result of rapid erosion developing along the downstream face of the dike. Overtopping of the dike can also occur as a result of a large wave passing over the dike (such as a tsunami for sea dikes), or an effective dam created across a river course by a large landslide.

C2.0 MITIGATION METHODS FOR EARTHQUAKE INDUCED DIKE FAILURES

Where the design and analyses of new or upgraded dikes indicate that conventional dike construction methods/practices currently outlined by MFLNRO for static dike design will not meet with the seismic performance criteria established for high consequence dikes, additional design measures will need to be considered to improve the dike system to meet with the updated seismic design objectives. The extent of additional dike improvements or mitigation methods required to meet with the seismic design objectives will be dependent on the severity of the affects or impacts on the dikes by the selected seismic events. For example, where the selected seismic ground motions results in predominantly vertical subsidence of the dike, a simple raising of the dike to accommodate the estimated earthquake induced subsidence to maintain the desired free board may be sufficient. However, where there is a potential for significant or substantial earthquake induced combined horizontal and vertical deformations, such as a global slope failure within the dike, a combination of several stability improvement approaches such as widening and raising the dike, flattening the slopes of the dike, implementation of ground stabilization measures, *etc.* may be required. The feasibility of the various dike improvement approaches to meet with seismic design requirements will be site specific and consequently, cannot be generalized with a stipulated approach (*i.e.* the dike improvement solutions will be unique to meet with the site specific conditions).

It is recognized that some potential failure mechanisms can only be speculated (at best) and will be difficult to confidently evaluate with respect to developing mitigation measures. Such mechanisms would include the potential for a strike or thrust fault located below the dike, or determining the potential size and magnitude of a seiche or tsunami wave that may impact a dike. In such instances, sound engineering judgment may provide the only basis for reviewing these conditions.

The following presents various methods for improving dike stability to meet with the seismic performance criteria as established by the MOE.

C2.1   Modifying the Existing Dike Profile

Where space permits, modification of the existing dike profile may provide sufficient stability to meet with the objectives of the seismic performance criteria. Modification of the existing dike profile may include a combination of the following:

■   Raising the dike;

■   Widening the dike; and/or,

■   Flattening the waterside and landside slopes.

The objective of modifying the dike profile is to reduce the potential for slope instability during and/or following seismic shaking, or at least maintain sufficient free board to retain the water following a major earthquake. As a minimum, the performance expectation of the post earthquake dike condition is that the dike can be reclaimed within a relatively short period of time to accommodate a potential 10 year flood event.

In the event that spatial constraints and/or the extent of post earthquake deformations are excessive, other forms of ground treatment to meet the seismic performance criteria should be considered, which may be implemented in conjunction with the measures described above. The following discussion presents a review of ground treatment alternatives that may be considered to improve seismic stability of a dike.

C2.2   Ground Improvement

The predominant failure mechanisms are deformations and/or displacements generated by liquefaction and/or cyclic softening of the deposits within and/or beneath the dike. By mitigating the risk of liquefaction or cyclic softening within and/or beneath the dike, substantial damage or failure of the dikes can be significantly reduced, or at least sufficient stability will be provided to the dike that meets with expected performance criteria for the design seismic hazard. Depending on the site-specific ground conditions and project-specific design requirements, it may be feasible to limit the extent of ground improvement to specific areas and still reduce deformations to within the performance criteria (as opposed to extensive densification of all potentially liquefiable zones beneath and beyond the dike profile). There are several commonly used approaches for mitigating liquefaction and they can be categorized according to their function as presented below:

- <u>Densification or Strengthening</u> – densification is a process used to describe the effective "tightening" of the soil particles to increase the cyclic shear resistance of the deposits. Some examples of typical ground densification methods are; vibro-compaction, displacement piling such as driven piles or compacted gravel piles, vibro-replacement, explosive compaction, compaction grouting, rapid impact compaction and dynamic compaction.

- <u>Drainage</u> – Vertical drainage provides for dissipation of pore pressure during seismic shaking and consequently, allows for volumetric decreases within loss of soil strengths. Examples of vertical drains include wick drains, sand compaction piles and stone columns (depending on the surrounding deposits).

- <u>Inclusions</u> – Depending on the type of failure mechanism under consideration, inclusions can be used to reduce the risk of significant movements and/or the effect of piping failures caused by differential distortions within the dike alignment. Examples of such inclusions are; sheet piles, secant piles, soil-cement mix panels and jet-grout columns. Inclusions may also be used for reinforcement and containment of the liquefied deposits by reducing the cyclic loading on the deposits and consequently, reducing the potential for liquefaction.

The following presents a brief discussion of the various ground improvement techniques as described and adopted from the 2007 Task Force Report for the Greater Vancouver Region, a public domain document. Cited references are provided in the Task Force document.

C2.3   Densification or Strengthening

a)   <u>Vibro-compaction and vibro-replacement</u> are generally performed with electric or hydraulic powered vibrators that are jetted into the ground with water or water/air mixture. In vibro-compaction, the natural sand self-feeds into the void created by the vibro-flot, whereas in vibro-replacement crushed stone is used as backfill. Top-feed and bottom-feed vibro-replacement methods are used. In the top-feed method, the stone is introduced into the jetted hole from the ground surface, whereas in bottom feed the stone or gravel is transmitted down the hole via a pipe and introduced at the bottom of the hole. Bottom feed is often carried out using air jetting. The latter method allows the use of finer backfill material and potentially less mixing of the native soil and stone occurs. Vibro-replacement is generally effective in soils containing less than 15 to 20% fines (Martin et al, 1999). Luehring et al, 1998 showed that non-plastic sandy silts can be densified by a combination of vibro-replacement and vertical wick drains.

The equipment should be capable of delivering sufficient centrifugal force to cause the required densification. Stone backfill materials should generally be clean and hard. Crushed stone should be used when the stone backfill is to provide reinforcement for vertical or horizontal deformations. The depths of the vibrator, stone usage, and amperage or power developed are often monitored during the work. Achievable improvement depths are typically in the 25 to 35 m range although greater depths have been achieved with special equipment. Vibro-replacement is the most widely used liquefaction countermeasure in Vancouver and elsewhere in North America (Hayden and Baez 1994).   Design information and equipment characteristics can be found in many publications including Barksdale and Bachus (1983), Mitchell and Huber (1985), Dobson (1987), Baez (1995 and 1997).

b) <u>Dynamic compaction</u> uses impact on the ground surface to densify subsurface soils. Weights typically ranging from 10 tons to 30 tons are repeatedly lifted by a specially modified crane and dropped from about 15 m to 40 m heights. The amount of compaction and densification depth is a function of the weight, drop height, number of drops per point and the spacing of the grid. Empirical relationships are available to design deep dynamic compaction programs (Lukas 1986). Typically, treatment depths of up to approximately 11 meters are achievable in granular soils. A granular pad is often placed if surficial saturated cohesive soils are present or if the groundwater table is within 1 to 1.5 m of the surface. The major limitations of the method are limitations on the depth of compaction that can be achieved and, if used in an urban environment, vibrations, flying matter, noise, and perception of damage. For these latter reasons, work often requires 30 to 60 m or more clearance from adjacent occupied buildings or other sensitive structures.

Energy delivered to the ground, sequence spacing and timing of drops, as well as ground response in the form of crater depth and heave of the surrounding ground are important quality control parameters. The location of the water table and presence of surface "hard pans" can greatly affect the quality and outcome of the densification process. Pore water pressure within recently treated areas should be allowed to dissipate before secondary treatments are implemented.

c) <u>Rapid impact compaction</u> (RIC) is analogous to dynamic compaction in that the ground surface is impacted with a weight. The difference is that the weight and drop height are smaller and more frequent. In RIC, a weight of approximately 7.5 tonne is dropped about 1.2 m to impact a 1.5 m diameter footing. A pile-driving-like hydraulic hammer is used to lift and drop the weight at 40 to 60 blows per minute. Like dynamic compaction the penetration depths are limited, however improvement depths greater than 6 m have been achieved in ideal granular soils. The efficiency of RIC will diminish with finer materials. The ability of RIC developing the required densification for significant ground motions associated with seismic hazards with low return periods may also be limited or unachievable. The drop height, number of blows, penetration per blow, and total energy per point are monitored by a data acquisition system. Vibrations from the work will be similar to those from pile driving and must be considered if working near structures, especially if they are occupied.

d) <u>Compaction grouting</u> involves pumping low slump, mortar-type grout under pressure to density loose soils by compaction. Effective treatment requires that the grout push the soil aside rather than fracture or permeate it. Compaction grouting pipes are typically drilled-in or driven steel pipes of 50 mm internal diameter or greater. Pressures ranging from 700 to 2100 kPa (100 to 300 psi) are used to inject a stiff, 75 mm or less slump, cement grout. In granular soil susceptible to liquefaction refusal pressures of 2800 to 3500 kPa (400 to 500 psi) are common (Martin et al. 1999). Grout pipes are typically installed in a grid pattern of 1.5 to 3 m spacing. Often, primary spacing patterns with secondary or tertiary intermediate patterns are used. Spacing and sequence of the grout points affects the quality of densification and ground movements achieved.

Grouting volumes in granular soils typically range from 3 to 12 percent of the treated soil. However, volumes up to 20 percent have been reported for extremely loose sands or silty soils. The procedure is not effective when vertical confinement is less than 2.4 to 3 m of overburden (Martin et al. 1999). Information on this technique can be found in Graf (1992), Baez and Henry (1993), Boulanger and Hayden (1995), and Warner (2004).

Slump, consistency of the mix, grout volumes, injection pressures, pore water pressures, and ground movement at the surface, next to sensitive structures, and at depth are often monitored during the work. Grout is typically injected in stages from the bottom up. At each stage a stopping criteria of grout volume, pressure, or heave is followed before proceeding with the next stage. Grout casing should be at least 50 mm internal diameter to avoid high back pressures before sufficient grout is injected. Over-injection of grout in a primary phase may result in ground heave and diminish densification effectiveness (Martin et al, 1999).

e) Compaction Piles- The driving of piles on close centers densifies the soil by pressure and vibration. Pile shafts also have a reinforcing effect by acting as dowels between the soil layers. Timber and sand or gravel compaction piles are commonly used. If the timber piles are permanently below the water table the timber need not be treated. Piles are typically placed on 1.2 to 2 m centres and splices are sometimes used to increase depth. Sand and gravel compaction piles are often made by vibrating in a pipe with an expendable bottom plate. Upon extraction sand or gravel is introduced into the displaced void. Sand and gravel compaction piles can also be constructed using expanded base pile procedures. The spacing of sand and gravel compaction piles depends on pile diameter but is typically in the 1.2 to 2 m range. Densification performance is often tested by carrying out penetration tests between the piles.

f) Explosive compaction (EC) is carried out by setting off explosive charges in the ground. The principal advantages of EC relative to other vibratory densification techniques is that it can be carried out to great depth provided the soils are largely saturated. The method requires only drilling equipment and a supply of explosives to implement. However, careful engineering design of the EC process is required to assure reasonably uniform densification (through selection of the sequencing of the blast hole patterns) and minimize offsite vibration effects (if required). The method is particularly cost effective where relatively large volumes of soil are required to be densified at depths in excess of 6 m. The density of loose deposits can typically be increased to relative densities in the range of 70 to 80% ($(N1)_{60}$ of 20 to 25 and cone penetration resistances (Qc1) of 100 to 130 bars).

Once an area of ground has been shot and pore pressures have largely dissipated, repeated applications ("passes") of shaking caused by controlled blast sequences causes additional settlement depending on soil density and stiffness. The degree of densification obtainable will also depend on the fines content of the sand as is the case for other methods of densification. The range of particle size for which blasting is practical is the same as for vibro-compaction.

It has been observed that where blasting is used, there is a considerable time effect on the values of penetration resistance. For the above reasons, initial evaluation of the effectiveness of the EC process is based on direct measurement of soil volume change using in-situ settlement gauges.

C2.4   Drainage Measures

Passive drainage can be beneficial in both limiting the triggering of liquefaction and reducing the related deformations. Seismic drains were proposed by the late Prof. Seed as a means of mitigation in the 1970's (Seed & Booker 1977). Design procedures and the public domain program FEQDRAIN (Pestana et al. 1997) are available. The intent of Seed's procedures was to prevent triggering of liquefaction.

Recent research has shown that pore water migration and redistribution during earthquakes is important and trapping of water under low permeability layers can lead to very weak interlayers or even a water film (Kokusho 2003; Byrne et al. 2006). It is believed that this is the reason for the low residual strengths observed in liquefaction case histories. For sites with low permeability barriers over liquefiable sand, drains can be used to mitigate this effect and reduce ground deformations and potential for flow slide failure. Drains should also be considered for reducing the pore pressure buildup within densified ground that is surrounded by liquefied soil, especially where spatial constraints limit access for ground densification.

The performance of the drains is dependent on the following parameters:

- Soil permeability;

- Drain spacing;

- Vertical flow capacity of the drain;

- Soil density (capacity for volumetric compression);

- Filter compatibility between the drain and native soil;

- Elevation of point of discharge relative to water table; and

- Rate of loading provided by the earthquake (*i.e.* if the earthquake liquefies the soil in one pulse the drains will not have time to respond and temporary liquefaction may still occur, whereas if many pulses are required to liquefy the soil then the drains may work well as the excess pore water is dissipated as it is generated).

The use of drains without soil densification may not necessarily provide adequate pore pressure relief during the period of strong earthquake shaking (unless the drains are on very close centers). However, they can have beneficial effect in preventing flow slides and reducing lateral deformations

Drains can be constructed in several ways:

- Pre-fabricated perforated pipe within a filter cloth sock -these can be installed in a drill hole or vibrated into the ground using a mandrill;

- Slotted pipe with surrounding filter sand or fine gravel installed in drill holes;

- Traditional water wells with screen and filter;

- Gravel or sand compaction piles;

- Vibro-replacement columns constructed using filter sand and bottom-feed methods; and

- Prefabricated wick drains.

Golder
Associates

Shake table tests (Sasaki and Taniguchi, 1982) indicate that gravel drains can accelerate the dissipation of excess pore water pressures, thereby limiting the loss of shear strength within the saturated deposits. Following the 1993 Kushiro-Oki, Japan, earthquake, Iai et al. (1994a, 1994b) observed that quay walls having back fill treated by the gravel drain pile and sand compaction pile techniques suffered no damage, while quay walls having untreated backfill were severely damaged due to liquefaction. Seismic drains were tested at Massey Tunnel by liquefying the soil with blasting.

The tests showed that both prefabricated drains with filter cloth sock and slotted pipe with surrounding filter sand drains performed well. The tests also showed that large settlements may still occur in the vicinity of the drains, and illustrated the importance of vertical flow capacity of the drain (large volumes of water have to flow over a short period of time). However, the tests showed that liquefaction may still be triggered in the loose sand soils located between the drains if they are subjected to high intensity short duration shocks (*i.e.* if pore pressures build up in the soil significantly faster than it is dissipated by the drains).

C2.5 Inclusions

Inclusions within the dike may be considered where there is a significant risk of piping of the structure, particularly if analyses indicate that the dike is at risk of producing continuous transverse cracks through the dike (below the water level) as a result of significant differential horizontal or vertical displacements. Such conditions may prevail where there is an abrupt change in the foundation deposits along the dike. The risk of piping failure may be reduced by providing a relatively ductile, low permeability structure through the dike. Depending on the magnitude of differential displacements the inclusion may comprise sheet piles, which is the most common method used to establish a hydraulic cut-off. However, in some instances, the predicted strain or relative displacement of the dike may be sufficient to yield or cause separation of the sheet piling. Other approaches for mitigating the potential for post-earthquake hydraulic failure is to install a cut-off barrier comprising plastic concrete that is capable of accommodating significant differential displacements while maintaining hydraulic integrity of the cut-off barrier as well as maintaining a stiffness consistent or greater than the encapsulating dike materials.

The inclusions may also be used to provide stability and reinforcement to the dike, similar to the methods described for ground improvement and strengthening. In such cases, a mix of soil and cement are employed to develop the stiffness and strength required to accommodate shear and bending stresses imposed by cyclic shaking. Areas targeted for liquefaction mitigation may be stabilized by creating coherent cells which encapsulate and confine the deposits and reduce the cyclic loading/pore pressure development within the cells. The size and distribution of the cells are generally determined through rigorous analyses to review the effects of distortions coupled with pore pressure generation within the cells.

Reinforcement cells may be created by jet grouting or soil-cement mixing methods. A brief discussion of each method is presented below.

Jet Grouting - The most common technique used in Jet Grouting involves the insertion of the Jet Grout Pipe to design depth for the bottom of the soil-crete column. The pipe comprises a high strength drill rod with a hollow center and special Jet Grouting Nozzles at the tip. The jetting pipe is advanced to the target depth using water jets attached to the initial rod. Once the target depth has been reached, the rods are rotated slowly and pressurized with grout slurry made typically of Portland cement and water.

High pressures (4000 - 6000 psi) are maintained to force the grout out laterally through special ports located in the sides of the pipe, near the bottom. The slurry exits the jet port at very high velocity, impinges on the soil, penetrating it several inches to feet away from the jets. The rotating jets destroy soft soil formations, and uniformly mix the native soil with cement. Finally the rotating pipe is drawn slowly upward at a carefully controlled rate so that the jets create an approximately cylindrical column of treated soil. The actual diameter of the jet grout column is dependent specific items; soil conditions, grout mix, nozzle diameter, rotation speed, withdrawal rate and grout pressure. Jet grout mixes are tailored to each project considering the soil type and the application. Containment cell walls can be created by interconnecting (overlapping) two or more rows of jet grout columns to achieve the desired wall thickness. The following figure presents a photograph of typical jet grouting rig.

Figure C-4: Typical Jet Grouting Rig

Soil-Cement Mixing - Soil-mixing involves mixing cementitious materials with the native soil using a mechanical mixing unit. Traditionally, soil mixing has been carried out using multiple hollow stem auger arrangement. However, recent advances in the technology has produced a cutter soil mixer (CSM) specifically design to generate homogenously mixed columns of soil-cement mixtures. The CSM differs from the traditional auger technique where soil-cement mixtures are created with rotating cutter heads about a vertical axis. Cement, fly ash, quicklime, and other additives are typically used in the grout to blend with the indigenous soils. Figures 5 and 6 present photographs of the two equipment types.

Soil mix column depths of 30 m or more can be achieve with the equipment described above. Typically, the liquefiable soil is contained within soil-mixed walls created by overlapping the soil mix inclusions. The procedure reduces shear strain within the soil to reduce the potential for liquefaction and confines soils that do liquefy to limit displacement. The walls also add shear strength to the overall soil-wall system. Column shear strengths of 1 MPa or more can be achieved, even in silty soils. The method has been used at Jackson Lake dam in Wyoming (Ryan and Jasperse, 1989), which comprised honeycomb shaped cells placed at the heel and toe of the dam. Other applications have included construction of underground barriers to circumvent significant lateral displacements in liquefied or cyclically softened deposits. Columns are usually tested using wet sampling, coring, CPT, pressuremeter, or seismic methods. Some variation in uniformity and strength should be expected.

Figure C-5: Typical Soil-Mix Auger Rig

Figure C-6: Typical Soil Cutter Mix Rig

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APPENDIX D
Remediation of Seismic Damage to Dikes

D1.0 INTRODUCTION

Dike geometry and construction vary considerably throughout the Fraser floodplain. The dikes were originally constructed many decades ago using less sophisticated construction techniques and with less quality control than present day earthworks construction. Dike upgrade works over the last 40 years or so used engineering principles and better material and construction control to develop robust and safe structures. However, economics sometimes dictated the degree of safety and inherent design criteria adopted. Regardless, the geometric standard for dike construction throughout the Fraser floodplain has generally been implemented and most dike slopes conform to the Dike Design and Construction Guide, Best Management Practices for British Columbia published in 2003 by the Ministry of Environment (formerly, Ministry of Water, Land and Air Protection). This therefore, somewhat simplifies the repair procedures as geometric conformity exists.

D1.1 Dike Breach

A breach in a dike can be initiated as a result of applied dynamic forces from earthquakes, which are associated with the forces of the water being retained, forces on the body of the dike, and weaknesses in the body of the dike and the underlying foundation soils. The common dike breach mechanisms are as follows:

■ **Overtopping**: Riverine flooding, tsunami, and wave run-up results in overtopping of the crest of the dike whereby the overtopping flow finds a low depression and weak spot in the dike which initiates localized erosion and down-cutting of the dike fill. The erosion continues and accelerates due to concentration of flow which can then lead to a breach in the dike. Overtopping could also occur from complete loss of freeboard due to lateral spreading of the dike structure by earthquake-induced liquefaction of the foundation soils beneath and adjacent to the dike.

■ **Piping (Internal Erosion) / Seepage:** Piping is a result of uncontrolled seepage either through the structure of the dike or through the underlying foundation soils. The seepage can occur post-earthquake due to cracking of the dike fill materials or liquefaction-induced movements. These discontinuities (cracks and localized openings) can concentrate seepage and result in piping (internal erosion or loss of soil) and the formation of "sand boils" which eject water and sediments. Uncontrolled piping can propagate into preferred seepage flow paths that develop within and beneath the dike which results in hydraulic connection to the water side and progressive failure and breach of the dike.

■ **Sliding/Foundation Soil Failure:** Movement, cracking or failure of the dike, or portions of the dike, that occur as a result of the lateral force(s) transmitted to the dike structure that exceeds the reduced shear resistance of the dike fills and underlying foundation soils (caused by soil liquefaction during earthquake shaking).

D1.2   Dike Remediation

D1.2.1   Temporary Emergency Repairs

The inspection guidelines for flood control dikes after an earthquake event are outlined in Appendix B. In the event that there is evidence of a breach or a developing breach from any of the above mechanisms, emergency repairs will likely need to be initiated. The degree to which such work will be required depends on the urgency dictated by the water level behind the dike at the time of the breach. Dike emergency (temporary) repair techniques include:

- Infilling by placing and compacting suitable fill into the breach to fill the hole in the system. This is somewhat time consuming and will depend on the availability of fill and truck and equipment access to the breach location. The infilling can temporarily plug the hole in the dike but further review, analysis and design will be required to remediate the dike once the flood threat has passed;

- Sand-bagging is a commonly used method for raising dikes crests or filling voids when emergency conditions dictate. Sand bags can also be used to prevent piping from boils by constructing a sand bag ring around the boils. This technique restricts flow and velocity which prevents soil loss; and

- Water Filled Fabric Tubes (Aqua dam, rubber inflatable dam). The use of inflatable fabric /rubber tubes to fill breaches in dikes has been investigated. The photographs shown on Figures D-1 and D-2 are reproduced from *Resio et al (2008)* and show the effectiveness of this repair in filling a breach and stopping uncontrolled flow. The tubes can be transported quickly and easily to the breach location and filled with water using a small pump system. The use of such tubes/bags also allows the freeboard of any dike system to be quickly raised.

Figure D-1: Sample deployment of deep-breach closure system, with a small residual flow still persisting along the upper edges of the breach opening (Resio et al, 2008).
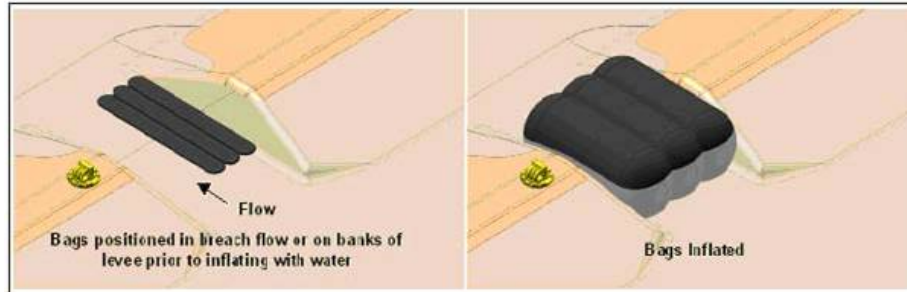
Figure D-2: Parallel flow concept Resio et al, 2008).

D1.2.2 Permanent Remediation of Damaged or Failed Dike Sections

After an earthquake event (and after the flood threat has passed, where both of these events occurred concurrently), a detailed inspection of the damaged section(s) of dike will need to be carried out to assess the extent of damage and the likely mechanism(s) of future failure. The type of remediation required will be a function of the level of damage that has occurred to the dike. Some likely damage scenarios and options for remediation of damage or failed dike sections are outlined below.

D1.2.2.1 Remediation of Localized Small Scale Damage

Localized sloughing and slumping, where the overall integrity of the dike has not been compromised, can in most instances be repaired by means of regrading and addition of some fill. It may be possible to seal localized tension cracks using bentonitic grout.

D1.2.2.2 Remediation of Dike Failures

In areas where the dike has failed (*i.e.* loss of freeboard, side slope failure, lateral movement of the dike and the like) where the remaining dike structure in the affected area is no longer capable of serving its intended purpose, complete re-construction should be carried out. If the dike failure(s) has occurred as a result of seismic liquefaction, then consideration should be given to densification of the ground adjacent to the toe(s) of the dike and possibly beneath the dike in severe circumstances. Details on densification techniques commonly used are outlined in Appendix C. Detailed analysis should be carried out to determine:

■ The extent of densification (*i.e.* width of zone to be densified beyond the toe of the dike) that is required. Property considerations (*i.e.* proximity of neighbouring structures, limited property width or Right-of-Way *etc.*) may in some instances dictate whether or not densification is feasible;

■ fill requirements for reconstruction in areas where dike failure has occurred from piping; and

■ Filter compatibility between the various materials used to re-construct the dike to minimize the potential for future piping.

D1.2.2.3   Sand Boils

Where sand boils and/or piping failures are identified beyond the land side toe of the dike, drains may be installed to promote controlled release of hydraulic pressures during future seismic and/or flood events. Alternatively, a filtered berm can also be installed to reduce the potential for future boils and/or piping failures. Filter compatibility will be key to the successful operation of these remedial measures

D1.2.2.4   Appurtenant Structures

In areas where relative deformation has occurred between a more rigid structure (*i.e.,* a pumphouse or floodbox) and the adjacent more flexible earthen dike structure, site-specific investigations (Geophysics or possibly probing) should be carried out to try to identify if voids or zone of loosened soil have developed in the surrounding areas adjacent to the structure. If voids are detected, then injection grouting (using bentonitic grout) may be used to fill these voids or alternatively, localized reconstruction may be required to repair the dike. Damage that has occurred to pumphouses, floodboxes and the like should be addressed by a qualified structural engineer.

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APPENDIX E

Report Format/Table of Contents

Seismic Design and Analysis Requirements for Dikes

REPORT FORMAT/TABLE OF CONTENTS
DESIGN AND ANALYSIS REQUIREMENTS FOR HIGH CONSEQUENCE DIKES

The following outline summarizes the reporting requirements for seismic design and evaluation of High Consequence Dikes for site-specific seismic studies. For consistency of review by the Flood Safety Section, the following is suggested.

A.   Summary of Dike, Soil and Water Level Data
1.   Plan Location of Dike and UTM Coordinates
2.   Dike Segment Identification and Type of Dike (River, Sea Dike, etc.)
3.   Dike Configuration
4.   Applicable Appurtenant Structures
5.   Dike and Foundation Soil Conditions
6.   Mean annual water level and 1:10 yr water level
7.   Surrounding Dike Identification(s) and Specific Site Features
8.   Impacted Community (Farm, Urban, etc.)

B.   Site-Specific Seismic Hazard
1.   Plate Tectonic Environment
2.   Site-Specific PGAs and Uniform Hazard Spectra (from GSC Interactive Website) for Site Class C for EQL1, EQL2, and EQL3
3.   Site Class as per 2010 NBCC or 2012 BCBC
4.   Fa Value as per 2010 NBCC or 2012 BCBC
5.   Time-Histories of Accelerations for EQL-1, EQL-2 and EQL-3 Used in the Design Where Optimization of Deformations is Undertaken
        [Mean response values to be considered when using 7 or more time-histories.
        Maximum response values to be considered when using 3 time histories].

C.   Results of Field and Laboratory Investigations
1.   Field Investigation Program and Methods Used
2.   Laboratory Testing Program and Standards Used
3.   Geotechnical/Geological Model Developed for the Site

D.   Dike and Foundation Soil Characterization
1.   Material Zones, Strength, and Stiffness Characteristics
2.   Summary of Parameters – Static Stability Analysis
        [Fines Corrected SPT N60 or Qt Profiles or Values, angle of internal friction, cohesion, total and submerged unit weights]
3.   Summary of Parameters – Seismic Stability Analysis
        [Fines Corrected SPT N60 or Qt Profiles or Values, Maximum Shear Moduli and Damping Ratio Profiles, Total unit weight of geological material zones, Peak Undrained Shear Strength and Post-Liquefaction Residual Shear Strength of Soils, Hydraulic Conductivity Values, Estimated Overconsolidation Stress]

E.   Analysis Methodology
1.   Summary of Analysis Procedures for Static and Seismic Stability
2.   Summary of Analysis Procedures for Seismic Deformations
3.   Summary of Analysis Procedures for Seepage
4.   Computer Software Programs
5.   Analysis Assumptions

F.   Results of Dike Analyses
1.   Slope stability – Static and Seismic
2.   Liquefaction Potential
3.   Settlement Potential
4.   Seepage
5.   Defensive Design Measures
6.   Deformation Analyses

G.   Results of Foundation Analyses – Appurtenant Structures
1.   Liquefaction Potential
2.   Bearing Capacity
3.   Settlement and Deformation Analyses
4.   Defensive Design Measures

APPENDIX F

Example Problem – Dike Seismic Stability

1.0   PREAMBLE

An example problem for dike seismic stability analysis has been formulated to **illustrate** the concepts, methods of analyses, and procedures described in this guideline. The example problem does not include seepage and site-specific wave propagation analyses.

If the river water levels required for the analyses are not provided by or available from the local government or the Flood Safety Section, consultants are to undertaken analyses to establish the required data.

2.0   DESCRIPTION OF THE DIKE

The dike considered in this example problem is located near a river bank. Figure 1 shows the dike location and Figure 2 shows a cross section of the dike. Material properties considered in the slope stability analyses are summarized in Figure 2. The existing dike extends to Elevation +4.7 m, has a gravel road that is approximately 4.5 m wide, and has landside and waterside slopes of 3H: 1V.

3.0   AVAILABLE INFORMATION (SECTION 11)

Item No.	Details of Information	Yes/No	Reference
1	Dike Location	Yes	Figure 1
2	Dike cross section	Yes	Figure 2
3	Dike Classification	Yes	High Consequence
4	Type of Dike	Yes	River
5	Appurtenant structures	No	-
6	Surficial geology data	Yes	GSC Map 1484A
7	Bathymetry Data	No	From Available River Bathymetry Survey
8	Geotechnical Report	Yes	Golder (2001)
9	Dike inspection report(s)	No	-
10	Mean river water level/sea level	Yes	+0.5 m (Geodetic El.) See MOE Map
11	1:10 yr Water Level	Yes	+0.4 m (Geodetic El.)
12	Conventional Dike/Super Dike/Other	-	N/A

4.0   GROUND MOTION PARAMETERS (SECTION 11)

Site coordinates: Latitude (N), Longitude (W)

Site Class C Ground Motion Parameters (cf. NBCC 2010 Interactive Website):

Return Period	PGA (g)	Sa(0.2s)	Sa(1.0s)
1:100-yr	0.12 g	0.24 g	0.08 g
1:475-yr	0.27 g	0.53 g	0.17 g
1:2,475-yr	0.50 g	1.01 g	0.34 g

Earthquake Magnitude for Liquefaction Analysis (Section 13 of the Guidelines Document):

- Mw 6 for EQL-1

- Mw 7 for EQL-2 and EQL-3

Site Classification: Class D (NBCC 2010, Table 4.1.8.4A)

Dike Apex Angle: 163 degrees (Refer to Figure 3 for illustration)

Apex angle amplification factor   = (180/163)   = 1.1

EQL-1:   Sa(0.2 s) = 0.24 g

Fa = 1.3 (NBCC 2010, Table 4.1.8.4B, also Table 3 of the Guidelines Document)

PGA_Max   = 0.12 x 1.3   = 0.16 g

PGA_Min   = 0.12 x 1.3 x 0.80   = 0.13 g

Design PGA   = 0.13 x 1.1   = 0.14 g: EQL-1 Includes topographic effects

[*Note: If site-specific 1D ground response analyses indicate higher PGA than PGA_Min, use the computed PGA as the Design PGA*]

EQL-2:   Sa(0.2 s) = 0.53 g

Fa = 1.2 (NBCC 2010, Table 4.1.8.4B, Also Table 3 of the Guidelines Document)

PGA_Max   = 0.27 x 1.2   = 0.32 g

PGA_Min   = 0.27 x 1.2 x 0.60   = 0.19 g

Design PGA   = 0.19 x 1.1 g   = 0.21 g: EQL-2 Includes topographic effects

[*Note: If site-specific 1D ground response analyses indicate higher PGA than PGA_Min, use the computed PGA as the Design PGA*]

EQL-3:   Sa(0.2 s) = 1.01 g

             Fa = 1.1 (NBCC 2010, Table 4.1.8.4B)

             PGA_Max   = 0.50 x 1.1   = 0.55 g

             PGA_Min   = 0.50 x 1.1 x 0.50   = 0.28 g

             Design PGA   = 0.28 x 1.1 g   = 0.31 g: EQL-3, Includes topographic effects

             [*Note: If site-specific 1D ground response analyses indicate higher PGA than PGA_Min, use the computed PGA as the Design PGA*]

5.0   LIQUEFACTION INDEX CALCULATION

Results based on Seed's Simplified Analysis, C = 0.65 , Mean Shear Stress Reduction Coefficient; r_d. (Refer to Figure 4 for liquefaction assessment)

EQL-1:   Liquefaction Index   = *L0*

EQL-2:   Liquefaction Index   = *L2*

EQL-3:   Liquefaction Index   = *L3*

6.0   PSEUDO-STATIC ANALYSIS

EQL-1:   Liquefaction index   = *L0*
             Seismic Coefficient   = PGA   = 0.14; no liquefaction
             FOS_Slip Surface 1   = 1.5
             FOS_Slip Surface 2   = 1.8
             FOS_Slip Surface 3   = 1.8
             (Refer to Figure 5 for typical slip surfaces that can affect the integrity of the dike body)

             FOS > 1
             Displacement calculations are NOT required.
             Settlement calculations are NOT required.
             Meets freeboard requirement (i.e. ≥ 0.3 m, > 1:10 yr flood level)
             Design satisfies EQL-1.

EQL-2:   Liquefaction Index   = *L2*
             Seismic coefficient for flow slide failure   = 0.0
             Seismic coefficient for displacement calculations   = ½ PGA   = 0.11

Stability

FOS_Flow Slide Failure = 1.02 > 1.0 (See Figure 6 for stability analysis on flow slide)

(q_{c1N-CS} = 120, Residual shear strength ratio = 0.16 with void redistribution curve is used for assessment of residual shear strength ratio)

Displacements

FOS_Slip Surface   = 1.0 (Ky = N = 0.02 g, Simplified Newmark Displacement = 6 V^2/2gN = 0.7 m for N/A < 0.2, and normalized acceleration to velocity ratio A/V ≈1, for Fraser Lower Mainland Only)

(See Figure 7 for yield acceleration assessment)

Maximum Horizontal Displacement=   0.7 m > 0.3 m

Performance criteria are not satisfied.

Modification of configuration or ground improvement is necessary.

7.0   MODIFIED CONFIGURATION – PSEUDO STATIC ANALYSIS

Flatten Waterside Slope (Waterside slope is flattened to 5H:1V – See Figure 8 for previous and new configuration)

Revised dike apex angle: 169 degrees

Apex angle amplification factor   = (180/169)   = 1.1

Calculate new design PGAs:

EQL-1:   Design PGA   = 0.13 x 1.07   = 0.14 g: EQL-1 Includes topographic effect,
EQL-2:   Design PGA   = 0.19 x 1.07   = 0.20 g: EQL-2 includes topographic effects
EQL-3:   Design PGA   = 0.28 x 1.07   = 0.30 g: EQL-3, Includes topographic effects

EQL-1:   Liquefaction index   = L0

(CSR profiles are changed with the alteration in PGA. Refer Figure 9 for new liquefaction assessment)

Seismic Coefficient   = PGA   = 0.14 ; no liquefaction
FOS_Slip Surface 1   = 1.8
FOS_Slip Surface 2   = 1.8
FOS_Slip Surface 3   = 2.1

FOS > 1

Displacement calculations are NOT required.

Settlement calculations are NOT required.

Meets freeboard (i.e. ≥ 0.3 m, > 1:10 yr flood level)

Design satisfies EQL-1.

EQL-2:   Liquefaction Index   = L2
                  Seismic coefficient for flow slide failure   = 0.0
                  Seismic coefficient for displacement calculations   = ½ PGA = 0.10
                  <u>Stability</u>
                  FOS_Flow Slide Failure = 1.12   > 1.0
                  (q_{c1N-CS} = 120, Residual shear strength ratio = 0.16 with void redistribution curve is used for assessment of residual shear strength ratio)
                  <u>Displacements</u>
                  FOS_Slip Surface 1   = 1.000 (Ky = N = 0.05 and Simplified Newmark Displacement = 6 $V^2/2gN$ = 0.2 m for N/A < 0.2, and normalized acceleration to velocity ratio A/V≈1, for Fraser Lower Mainland Only)

                  Maximum Horizontal Displacement   =   0.2m < 0.3 m
                  Vertical Settlement (Ref. Figure 7b and CPT Data)   =   0.1 m < 0.15 m
                  (Vertical settlement calculated from CSR = 0.16 and volumetric strain = 0.75%)
                  Meets freeboard (i.e. ≥ 0.3 m, > 1:10 yr flood level)
                  Design satisfies EQL-2.

EQL-3:   Liquefaction Index   = L3
                  Seismic coefficient for flow slide failure   = 0.0
                  Seismic coefficient for displacement calculations   = ½ PGA = 0.15
                  <u>Stability</u>
                  FOS_Flow Slide Failure = 1.1   > 1.0
                  <u>Displacements</u>

                  FOS_Slip Surface 3   = 1.0 (Ky = N = 0.05 and Displacement = 6 $V^2/2gN$ = 0.6 m for N/A < 0.2, and normalized acceleration to velocity ratio A/V≈1, for Fraser Lower Mainland Only)

                  Maximum Horizontal Displacement   =   0.6 m < 0.9 m
                  Vertical Settlement Vertical Settlement (Ref. Figure 7b and CPT Data) = 0.4 m < 0.5 m
                  (Vertical settlement calculated from CSR = 0.24 and volumetric strain = 2.35%)
                  Design satisfies freeboard requirement (i.e. ≥ 0.3 m, > 1:10 yr. flood level).
                  Design satisfies EQL-3.

                  NOTE: Design is satisfied with the improved slope on waterside of the dike. However designer may have elected to go with ground improvement as an alternative, if dike setback on landside is not possible. In such a case, start with the original configuration of the dike.

                  Design Complete. No more iterations are required.

8.0   SIGNOFF

ABC, P. Eng.   XYZ, P. Eng.

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Designer   **Reviewer**

Date: 12 November 2013

Dike

Name: Dense SAND and SILT (FILL)   Unit Weight: 19 kN/m³   Cohesion: 0 kPa   Phi: 35 °
Name: Compact to Dense gravelly CLAYEY SAND   Unit Weight: 19 kN/m³   Cohesion: 0 kPa   Phi: 35 °
Name: Stiff to Very Stiff SILTY CLAY to CLAYEY SILT   Unit Weight: 18 kN/m³   Cohesion: 30 kPa   Phi: 0 °
Name: Firm to Stiff SILTY CLAY to CLAYEY SILT   Unit Weight: 18 kN/m³   Cohesion: 25 kPa   Phi: 0 °
Name: Compact SAND   Unit Weight: 19.5 kN/m³   Cohesion: 0 kPa   Phi: 31 °
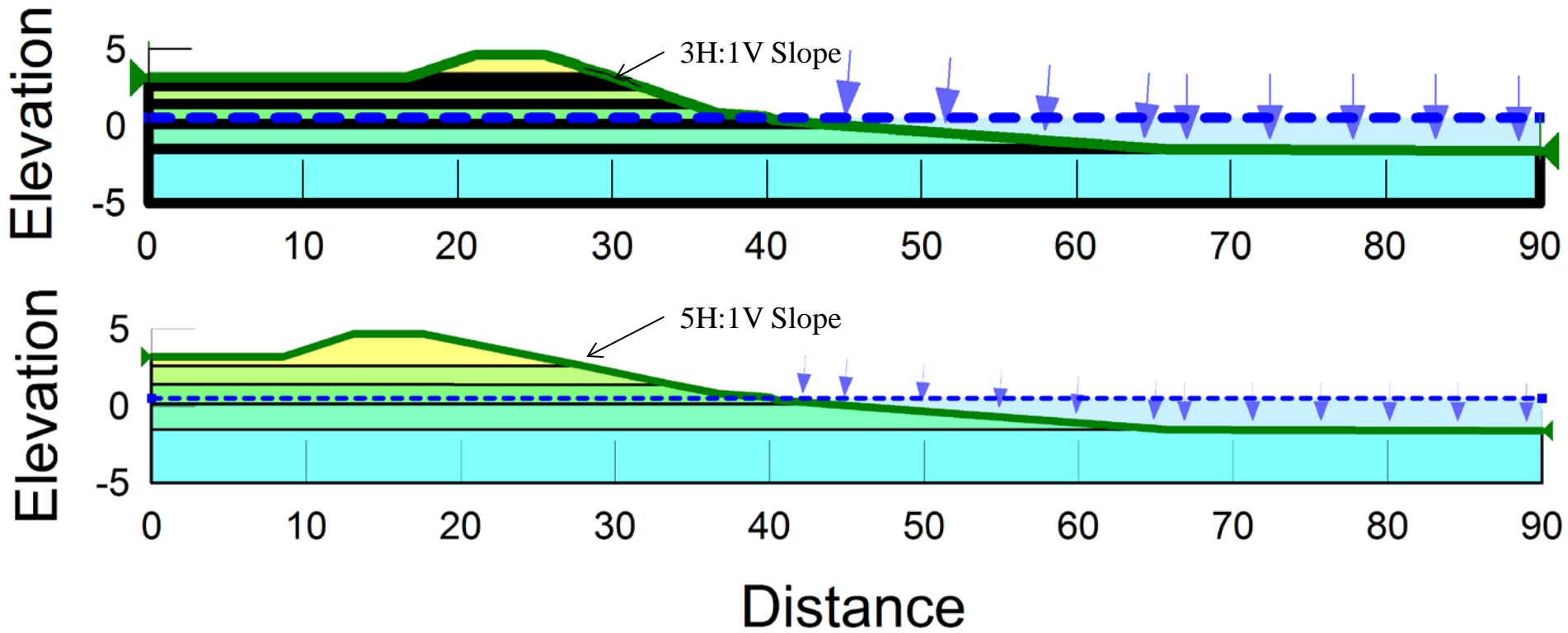
PROJECT **SEISMIC DESIGN GUIDELINESS FOR DIKES MINISTRY OF FORESTS, LANDS AND NATURAL RESOURCE OPERATIONS - FLOOD SAFETY SECTION**	
TITLE **DIKE SEISMIC STABILITY EXAMPLE – DIKE CROSS SECTION**	

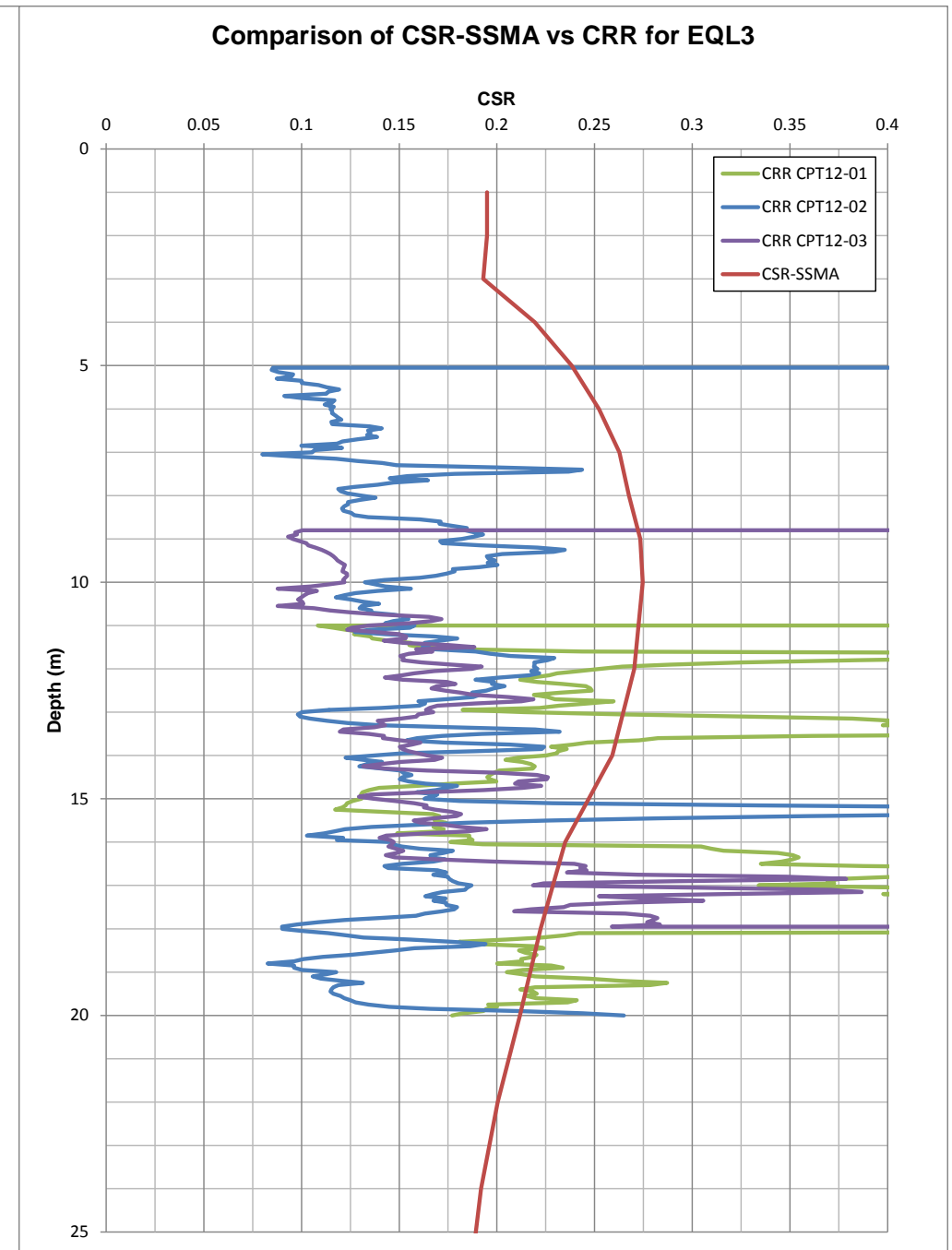|		
PROJECT No. 12-1447-0119	PHASE No. 1000	
DESIGN AT 06NOV13	SCALE NTS	REV.
CADD ---- 06NOV13		
CHECK UDA 06NOV13	**FIGURE 2**	
REVIEW UDA 06NOV13		

Apex Angle (= 163°)

3H

1V

Comparison of CSR-SSMA vs CRR for EQL1

Comparison of CSR-SSMA vs CRR for EQL2

Comparison of CSR-SSMA vs CRR for EQL3
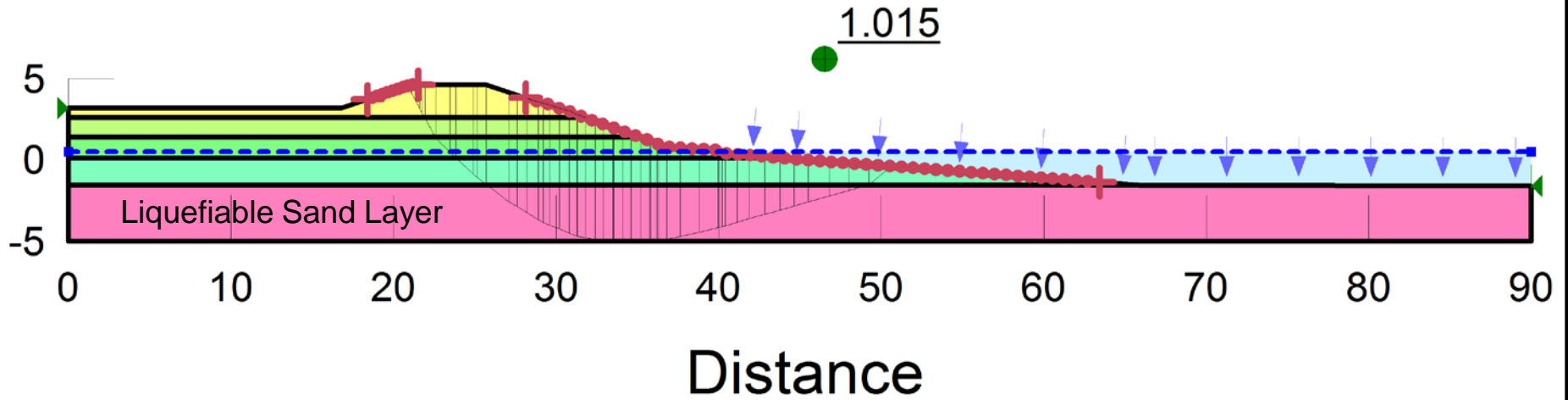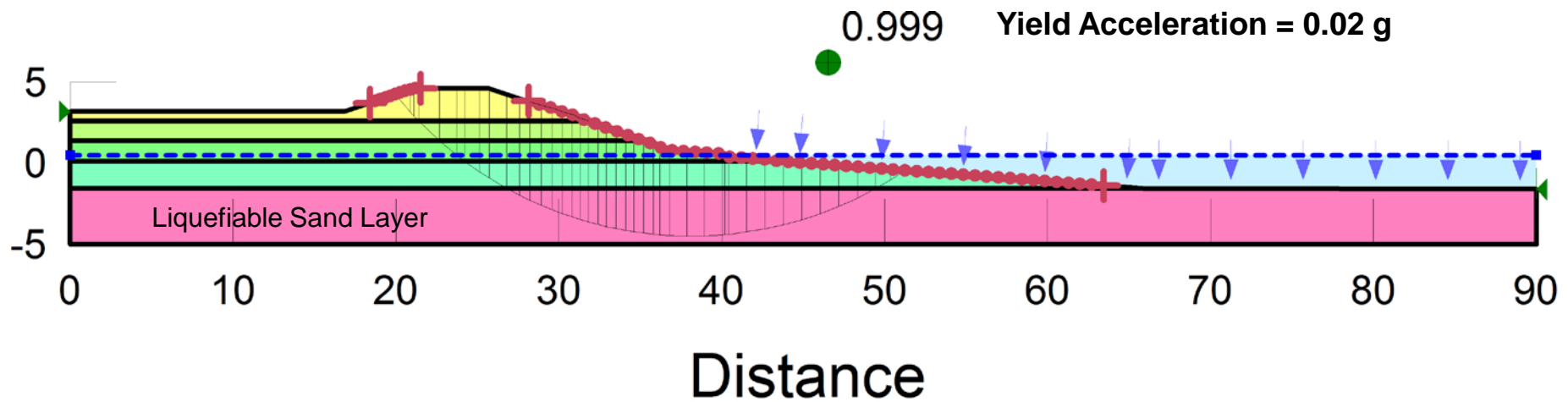
NOTE:

Normalized residual shear strength ratio for liquefied sand layer, $S_r/\sigma'_v = 0.16$

PROJECT	SEISMIC DESIGN GUIDELINESS FOR DIKES MINISTRY OF FORESTS, LANDS AND NATURAL RESOURCE OPERATIONS - FLOOD SAFETY SECTION				
TITLE	DIKE SEISMIC STABILITY EXAMPLE – NO FLOW SLIDE FAILURE				

0.999   **Yield Acceleration = 0.02 g**

Liquefiable Sand Layer

NOTE:

Normalized residual shear strength ratio for liquefied sand layer, $\frac{S_r}{\sigma'_v} = 0.16$
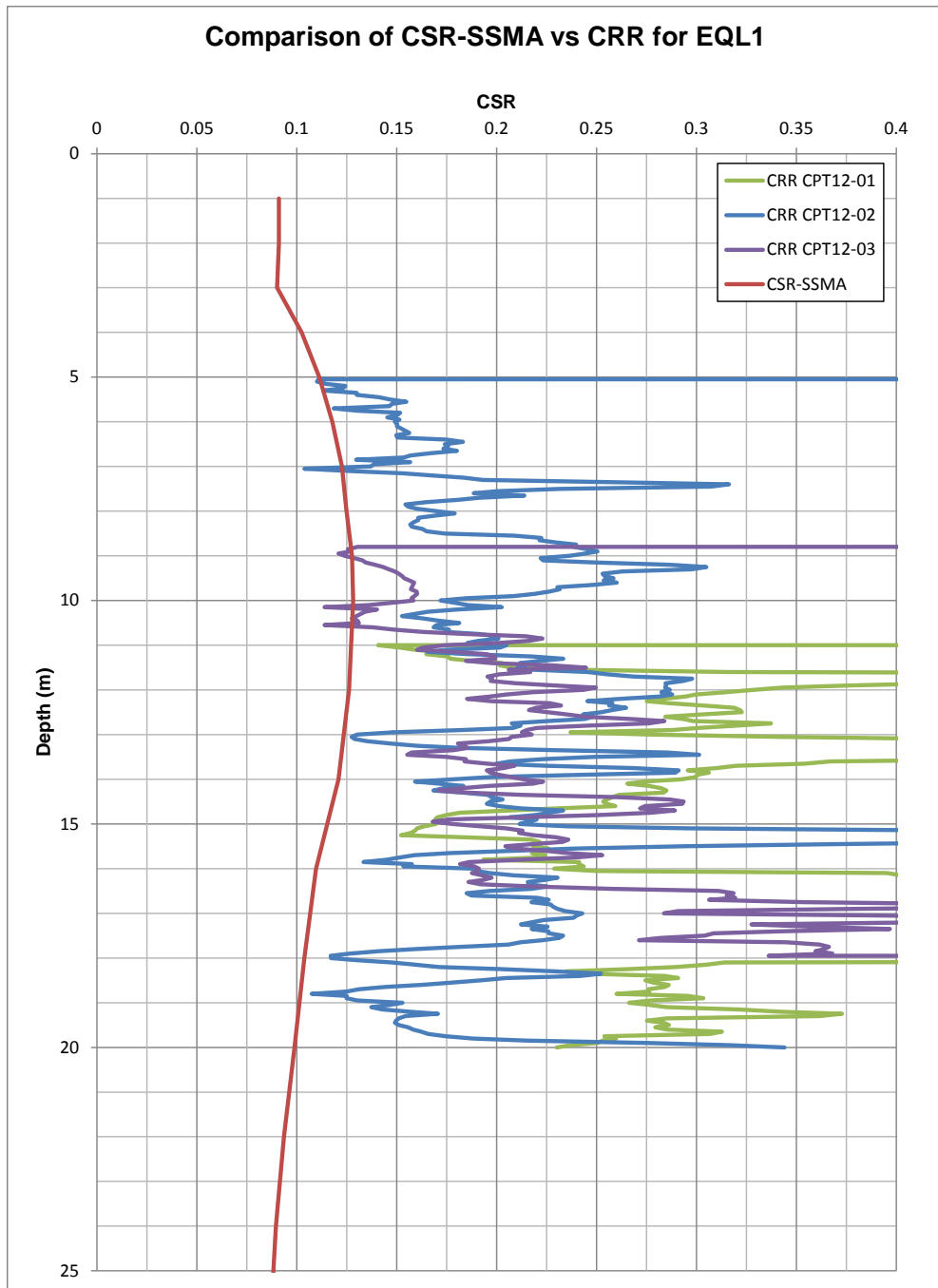
Name: Dense SAND and SILT (FILL)   Unit Weight: 19 kN/m³   Cohesion: 0 kPa   Phi: 35 °
Name: Compact to Dense gravelly CLAYEY SAND   Unit Weight: 19 kN/m³   Cohesion: 0 kPa   Phi: 35 °
Name: Stiff to Very Stiff SILTY CLAY to CLAYEY SILT   Unit Weight: 18 kN/m³   Cohesion: 30 kPa   Phi: 0 °
Name: Firm to Stiff SILTY CLAY to CLAYEY SILT   Unit Weight: 18 kN/m³   Cohesion: 25 kPa   Phi: 0 °
Name: Compact SAND   Unit Weight: 19.5 kN/m³   Cohesion: 0 kPa   Phi: 31 °

3H:1V Slope

5H:1V Slope

Distance

Comparison of CSR-SSMA vs CRR for EQL1

Comparison of CSR-SSMA vs CRR for EQL2

Comparison of CSR-SSMA vs CRR for EQL3

Legend (each plot):
- CRR CPT12-01
- CRR CPT12-02
- CRR CPT12-03
- CSR-SSMA

Axes: CSR (0, 0.05, 0.1, 0.15, 0.2, 0.25, 0.3, 0.35, 0.4); Depth (m) (0, 5, 10, 15, 20, 25)