6.1 Scope

Add the following:

Section 6 - Foundations and geotechnical systems, addresses some construction practice and requirements. The designer shall consider all design-related construction requirements and provide the necessary design information and recommendations in the geotechnical reports and the Plans.

6.2 Definitions

Replace the following:

Double corrosion protection – a proven system of double covering of the tendon to protect against corrosion consisting of encapsulation of the tendon inside a plastic sheath pre-filled under factory conditions with grout or corrosion protection compound (grease or wax) designed to minimize crack width in the pre-grouted assembly. The whole assembly is then grouted into the anchor hole.

Add the following:

Embankment – earth or rock slope, with or without a foundation unit, that has been altered by cuts or fills, structurally stabilized, subject to ground improvement, or drainage modification and is adjacent to, or supporting, a highway or bridge.

Two-Stage MSE Wall – MSE walls where the in the first stage, the retention structure is built with a flexible facing, allowing foundation settlement to occur, and the second stage adds a cast-in-place or precast wall facing.

Geosynthetic reinforced soil (GRS) – Geosynthetic reinforced soil (GRS) consists of alternating layers of compacted granular fill and closely spaced (≤ 300 mm) geosynthetic reinforcement.

Note: Geosynthetic reinforced soil (GRS) is internally supported and has distinct behavior from Mechanically stabilized earth (MSE). Reinforcement spacing is the key differentiation between GRS and MSE.

Replace the following:

Geotechnical System – a group of interrelated elements designed to transmit loads to the ground or to retain the ground.

Note: examples of such systems include deep foundations, shallow foundations, retaining structures, ground anchors, embankments, and their components.

Add the following:

Natural slope – the unaltered earth or rock slope adjacent to a highway or structure, where the highway performance depends on the natural slope’s performance.

6.4 Design Requirements

6.4.1 Limit states

6.4.1.1 General

Add the following:

Serviceability Limit State (SLS) Combination 1, given in Table 3.1, shall be used for global (overall) stability of embankments, geotechnical systems, and natural slopes which affect the performance of the highway or structure.

6.7 Geotechnical report

6.7.3 Design information

Delete the last sentence and replace it with the following:

Signing and sealing of the Geotechnical report shall be in accordance with the Association of Professional Engineers and Geoscientists of the Province of British Columbia requirements.

Commentary: *Requirements for signing and sealing are governed by the Association of Professional Engineers and Geoscientists of the Province of British Columbia.*

6.9 Geotechnical resistance

6.9.1 General

Add the following:

The following benchmarks in Table 6.2a provide guidance for determining the Degree of Understanding for use of Table 6.2 for deep foundations:

Table 6.2a
Benchmarks for Degree of Understanding for Deep Foundations

Test Method/ Model	**Degree of Understanding**		
Compression	**Low**	**Typical**	**High**
Static Analysis	• Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at project site.	• Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at each bridge pier and abutment.	• Design based on CPT or BPT data representative of conditions at each bridge pier and abutment. **<u>OR</u>** • Design based on BPT data representative of conditions at each bridge pier and abutment, and • Measure bounce chamber pressure and consider BPT friction.
Static Test	• Design based on a single test pile for bridge pier as per ASTM D1143, and • Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data, and • Test pile size and length shall be similar to the production piles.	• Design based on a single test pile for bridge pier as per ASTM D1143, and • Test pile instrumented with at least a tell-tale. Force applied at pile head above ground, and • Test pile size and length shall be similar to the production pile, but toe condition and installation method shall be the same as production piles, and • Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. **<u>OR</u>** • Design based on a single pile test with single level high capacity, sacrificial loading unit embedded in the foundation unit instrumented with force measurement, and • Test pile size and length shall be similar to the production pile, but installation method shall be the same as production piles, and • Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.	• Design based on a single test pile for bridge pier as per ASTM D1143, if bridge piers are separated less than 500 m, and • Design based on two test piles for bridge pier as per ASTM D1143, if bridge piers are separated more than 500 m, and • Test pile instrumented with at least toe tell-tale and strain gauges attached to pile at appropriate elevations. Force applied at pile head above ground, and • Test pile length shall be similar to the production pile, but size, installation method, and toe condition shall be the same as production piles, and • Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. **<u>OR</u>** • Design based on one test pile with two levels of high-capacity, sacrificial loading units embedded in the foundation unit if bridge piers are separated less than 500 m, and • Design based on two test piles with two levels of high-capacity, sacrificial loading units embedded in the foundation unit if bridge piers are separated more than 500 m, and • Test pile size and length shall be similar to the production pile,

			but installation method shall be the same, and • Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.
Dynamic Analysis	• Wave equation analysis (WEAP unless otherwise Consented to by the Ministry) performed before construction for multiple driving systems **<u>OR</u>** • Wave equation analysis (WEAP unless otherwise Consented to by the Ministry) performed using pile driving blow count data from previous installations at the site.	• Wave equation analysis (WEAP unless Consented to by the Ministry) performed with pile driving blow count data on production piles for the full depth and known driving system.	• Wave equation analysis (WEAP unless Consented to by the Ministry) performed using pile driving blow count data on production piles for full depth, damage observations and measured blow rate data for diesel hammer or using known efficiency for a hydraulic hammer.
Dynamic Test	• Pile dynamic testing (PDA unless otherwise Consented to by the Ministry) and dynamic analysis (CAPWAP unless Consented to by the Ministry) conducted on an adjacent bridge pier or abutment used with pile driving blow count data obtained for the pile. **<u>OR</u>** • Design based on a single rapid load test on a pile for bridge pier or abutment as per ASTM D7383, and • Results extrapolated to other bridge piers and abutments by consideration of borehole, CPT or BPT data, and • Test pile size and toe condition may not be the same as production piles.	• Pile dynamic testing (PDA unless otherwise Consented to by the Ministry) and dynamic analysis (CAPWAP unless otherwise Consented to by the Ministry) conducted at each bridge pier and each abutment, and • blow count data for other piles at the same piers or abutments collected with a hammer having consistent driving energy.	• Pile dynamic testing (PDA unless otherwise Consented to by the Ministry) and dynamic analysis (CAPWAP unless otherwise Consented to by the Ministry) conducted at each bridge pier and each abutment, and • Have borehole, CPT or BPT data to define the ground conditions, and • Have consistent driving energy delivered from the driving system with measured blow rate data for diesel hammers or known efficiency for a hydraulic hammer.

Table 6.2a (continued)
Benchmarks for Degree of Understanding for Deep Foundations

<table>
<tr><th rowspan="2">Test Method/ Model</th><th colspan="3">Degree of Understanding</th></tr>
<tr><th></th><th></th><th></th></tr>
<tr><th>Tension</th><th>Low</th><th>Typical</th><th>High</th></tr>
<tr><td>Static Analysis</td><td>• Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at project site.</td><td>• Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at each bridge pier and abutment.</td><td>• Design based on CPT or iBPT data representative of conditions at each bridge pier and abutment.
<u>OR</u>
• Design based on BPT data representative of conditions at each bridge pier and abutment, and
• Measure bounce chamber pressure and consider BPT friction.</td></tr>
<tr><td>Static Testing</td><td>• Design based on a single test pile for bridge pier as per ASTM D3689, and
• Results extrapolated to other bridge piers and abutments by consideration of borehole or CPT data, and
• Test pile size and length shall be similar to the production piles.</td><td>• Design based on a single test pile for a bridge pier as per ASTM D3689, and
• Test pile size and length shall be similar to the production piles, but the installation method shall be the same as production piles, and
• Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.
<u>OR</u>
• Design based on a single pile test with single level high-capacity, sacrificial loading unit embedded in the foundation unit instrumented with force measurement, and
• Test pile size and length shall be similar, but installation method shall be the same as production pile, and
• Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.</td><td>• Design based on a single test pile for bridge pier as per ASTM D3689, if bridge piers are separated less than 500 m, and
• Design based on two test piles for bridge pier as per ASTM D3689, if bridge piers are separated more than 500 m, and
• Test pile size and installation method shall be the same as production piles, and
• Test pile length shall be similar to production piles, and
• Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.
<u>OR</u>
• Design based on one test pile with two levels of high capacity, sacrificial loading units embedded in the foundation unit instrumented with force measurements, if bridge piers are separated less than 500 m, and
• Design based on two test piles with two levels of high capacity, sacrificial loading units embedded in the foundation unit instrumented with force measurements, if bridge piers are separated more than 500 m, and
• Test pile size and length shall be similar to production piles, but installation method shall be the same, and
• Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.</td></tr>
</table>

Note: Pile relaxation must be considered when using pile driving blow count or PDA data, for example, in some very stiff soils or some weak rocks. Restrike data should be used if these conditions may be present.

Designs shall be based on information available at the time of design; and higher resistance factors shall not be used based on the intent to do load testing or dynamic monitoring during construction. Higher resistance factors may be used based on data from load testing or dynamic monitoring that has been done to confirm resistance during construction.

Design and acceptance procedures for deep foundations shall be based upon the use of blow counts established from dynamic analysis or dynamic testing unless otherwise Consented to by the Ministry.

In Table 6.2 under the column entitled “Application”, replace “Embankments (fill)” with “Embankments”

The geotechnical resistance factors given in Table 6.2 for Global Stability of Embankments shall be used for geotechnical systems, and for natural slopes that affect the performance of the highway or bridge. The resistance factors given in Table 6.2 have been developed with the intent of achieving the following Factors of Safety (FOS) against global failure:

Table 6.2b
Resistance Factors, Consequence Factors and Factors of Safety for Global Stability of Embankments, Geotechnical systems and Natural slopes that affect the performance of the highway or bridge

(to be used in conjunction with Table 6.2)

Degree of Understanding	**Low**			**Typical**			**High**		
Resistance Factors for Global Stability – Permanent from S6-19	0.60			0.65			0.70		
Resistance Factors for Global Stability – Temporary from S6-19	0.70			0.75			0.80		
Consequence Factor from S6-19	High	Typical	Low	High	Typical	Low	High	Typical	Low
	0.90	1.00	1.15	0.90	1.00	1.15	0.90	1.00	1.15
FOS for Global Stability – Permanent	1.85	1.67	1.45	1.71	1.54	1.34	1.59	1.43	1.24
FOS for Global Stability – Temporary	1.59	1.43	1.24	1.48	1.33	1.16	1.39	1.25	1.09

The resistance and consequence factors (and the corresponding FOS values) in Table 6.2b shall be used with the load factors specified for the SLS Combination 1 in Table 3.1 of Chapter 3. This use is consistent with the methodology followed when computing the FOS values on global stability of embankments using the currently available computer software.

The FOS values for the Temporary condition apply to short term loading conditions lasting up to two years. Temporary condition does not apply to seismic or other transient loading conditions.

The following benchmarks in Table 6.2c provide guidance for determining the Degree of Understanding for use of Table 6.2 for global stability of embankments:

Commentary: *Assessment of existing embankments and/or natural slopes not altered by the project may be required on a case-by-case basis as requested by the Ministry.*

Table 6.2c
Benchmarks for Degree of Understanding for Embankments

Degree of Understanding	Low Understanding	Typical Understanding	High Understanding
Global Stability	• Shear strength parameters established based on subsurface data from nearby sites and published correlations with the consistency/density of site soils supplemented with geological evidence, and • Stability of embankment evaluated using accepted computer software that incorporates the method of slices and limit equilibrium method of analysis, and • Embankment fill density and strength based on Ministry standard specifications and published parameters.	• Shear strength parameters established based on a minimum of one borehole and published correlations with the consistency/density of site soils supplemented with geological evidence, and • Stability of embankment evaluated using accepted computer software that incorporates the method of slices and limit equilibrium method of analysis, and • FOS computed for an inferred groundwater profile, and • Embankment fill density and strength based on Ministry standard specifications and published parameters.	• Site-specific soil stratigraphy and consistency/density of soils established based on a minimum of two boreholes or 2 CPTs along the slope profile with laboratory testing to determine shear strength parameters, and • Groundwater profile established based on in-situ measurements, and • Low spatial variability of the subsurface soil conditions, and • Stability of embankment evaluated using accepted computer software that incorporates the method of slices and limit equilibrium method of analysis. Both force and moment equilibrium of slices shall be satisfied, and • Sensitivity of the computed FOS evaluated for differing groundwater profiles and anticipated variations in shear strength parameters, and • Embankment fill density and strength based on Ministry standard specifications and laboratory or in-situ testing. Fills placed with engineering supervision.

Note: For low-volume road bridges, modifications to the resistance factors may be considered when Consented to by the Ministry.

6.13 Integral and semi-integral abutments

6.13.3 Design requirements

6.13.3.1 General

Delete the third paragraph and replace with:

Sufficient lateral pile restraint shall be provided for integral abutments. Integral abutments shall not be used where the soil is susceptible to liquefaction, slope instability, sloughing, or boiling unless Consented to by the Ministry.

6.14 Seismic design

6.14.2 Seismic design and performance requirements

6.14.2.1 Performance requirements for foundations and geotechnical systems

Delete the third paragraph and replace with:

Unless specified otherwise by the Ministry, the following seismic performance criteria shall be met for geotechnical systems within the bridge approach embankment interface zone:

a) Lifeline geotechnical systems shall have
   i. 100% of the travelled lanes available for use following ground motions with a return period of at least 975 years. Any repair work shall not cause service disruption; and
   ii. 50% of the travelled lanes available for use following ground motions with a return period of at least 2475 years. If damaged, normal service shall be restorable within one month.
b) Major-route geotechnical systems shall have 100% of the travelled lanes available for use following ground motions with a return period of at least 475 years. Any repair work shall not cause service disruption.
c) Other geotechnical systems shall have 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years. If damaged, normal service shall be restorable within one month.
d) Lifeline, major route, and other geotechnical systems shall meet the life safety requirement of no collapse following ground motions with a return period of at least 2475 years and it shall be possible to evacuate the bridge safely.

***Commentary**: The potential impact of adjacent structures or geotechnical systems to the embankment bridge interface zone on the seismic performance of the embankment bridge interface zone should be considered.*

6.14.2.3 Seismic performance criteria

Delete the clause and replace with:

Unless otherwise specified by the Ministry, the following seismic performance criteria shall be met for geotechnical systems outside the bridge approach embankment interface zone if the geotechnical system falls within Seismic Performance Category 2 or 3, as defined in Table 4.10:

a) Lifeline geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 975 years. If damaged, normal service shall be restorable within one month.
b) Major-route geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years. If damaged, normal service shall be restorable within one month.

c) Other geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, restorable for use within one month following ground motions with a return period of at least 475 years.
d) Large permanent foundation deformations of geotechnical systems may be acceptable provided the specified post-seismic travelled lane functionality can be achieved.
e) Lifeline, major route, and other retaining structures taller than 6 m and in Seismic Performance Category 3 shall meet the life safety requirement of no collapse following ground motions with a return period of 2475 years.

***Commentary**: Retaining structures in item (e) includes, but is not limited to, such types as retaining walls, light weight fill, MSE systems, reinforced soil systems, and soil-nail systems.*

The Ministry may approve alternate requirements for the seismic design of geotechnical systems on a case-by-case basis by using a risk-based approach that considers the economic and societal consequences of the performance and the seismic hazard levels. For instance, the Ministry may consider alternative requirements for seismic performance of a retaining wall that is dependent on the performance of an existing large slope where the wall has little destabilizing effect on the slope and where stabilization of the slope for seismic loading is impractical.

Seismic design should be considered on a case-by-case basis for temporary geotechnical systems exceeding two years in service. For temporary lifeline and major route geotechnical systems, a return period of at least 100 years should be considered.

Seismic design should be considered for all partially constructed geotechnical systems with construction exceeding two years. For lifeline and major route geotechnical systems, a return period of at least 100 years should be considered.

6.14.3 Geotechnical investigation

Replace item a) with the following:

a) site designation in accordance with Clause 4.4.3.2;

6.14.4 Geotechnical resistance factors and analysis

6.14.4.1 Geotechnical resistance factors

Replace the contents of Table 6.3 with:

Design scenario	Seismic resistance factor*
Capacity-protected elements	ϕ_{gu} static values + 0.2
Forced-based design	ϕ_{gu} static values + 0.2
Performance-based design	ϕ_{gu} static values + 0.2

*ϕ_{gu} static values from Table 6.2; in no case shall the seismic resistance factor be greater than 1.0.

Commentary: *For analysis purposes, a nominal resistance factor of 1.0 may be used if a sensitivity analysis using appropriate bounds on soil parameters is completed.*

6.14.4.2 Analysis methods

Replace the second paragraph with:

The geotechnical analysis and design methods shall be subjected to independent peer review when specified herein or as directed by the Ministry.

Delete the third and fourth paragraph and replace with:

Where the potential for liquefaction is present, the requirements of Clause 6.14.8 for liquefaction evaluation and effects shall be met.

For retaining structures, the minimum analysis requirement shall be as specified in Table 6.14.4.2-1. For embankments, the minimum analysis requirement shall be as specified in Table 6.14.4.2-2.

Table 6.14.4.2-1
Minimum analysis requirement of retaining structures (all zones)

SPC	Lifeline		Major-route		Other	
	$H_s \leq 6$m	$H_s > 6$m	$H_s \leq 6$m	$H_s > 6$m	$H_s \leq 6$m	$H_s > 6$m
1	No seismic analysis is required.					
2	SDBM	RDAM	SDBM/FBM*		FBM	
3	SDBM	RDAM	SDBM	RDAM	SDBM	RDAM

*SDBM shall be used within the bridge interface zone where performance-based design is used for a bridge.

Table 6.14.4.2-2
Minimum analysis requirement for embankments

SPC	Lifeline			Major-route			Other		
	Within IZ		Outside IZ	Within IZ		Outside IZ	Within IZ		Outside IZ
	$H_s \leq 18$m	$H_s > 18$m		$H_s \leq 18$m	$H_s > 18$m		$H_s \leq 18$m	$H_s > 18$m	
1	No seismic analysis is required.								
2	SDBM	RDAM	SDBM	SDBM/PSA*		PSA	PSA		
3	SDBM	RDAM	SDBM	SDBM	RDAM	SDBM	SDBM	RDAM	PSA

*SDBM shall be used within the bridge interface zone where performance-based design is used for a bridge.

Legend:

Outside IZ = Outside the embankment bridge interface zone

Within IZ = Within the embankment bridge interface zone

SDBM = Simplified displacement-based method including Newmark-based type methods.

FBM = Force-based method including Mononobe-Okabe or Generalized Limit Equilibrium (GLE). Global stability shall be assessed by PSA.

RDAM = Rigorous dynamic analysis method including complex 2D or 3D finite element or finite difference dynamic analysis.

PSA =Pseudo static analysis. In this type of analysis, the seismic loading is modeled as a statically applied inertial force, the magnitude of which is a product of a seismic coefficient and the weight of the potential sliding mass.

$\boldsymbol{H_s}$ = Exposed height of retaining walls, top of embankment to bottom of embankment, or in the case of a geotechnical system, height of retaining wall plus embankment height

When FBM is used, and a retaining structure is able to move horizontally a minimum of 50 mm, without compromising the performance of the retaining structure or adjacent structures, the horizontal and vertical seismic coefficients shall not be less than one-half of the corresponding peak ground accelerations at the ground surface as estimated using Clause 4.4.3.3.

When PSA is used for stability analysis of embankments or retaining structures, the horizontal and vertical seismic coefficients shall not be less than one-half of the corresponding peak ground accelerations at ground surface as estimated using Clause 4.4.3.3. SDBM or RDA shall be used when the pseudo-static limit equilibrium analysis indicates a factor of safety less than 1.3.

When RDAM is used, the following requirement shall be met:

1. SDBM shall also be conducted for comparison.
2. The ground motions shall be established according to criteria 4.4.3.6.
3. A high degree of site understanding as defined in Clause 6.5.3 is required.

Commentary: *NCHRP Report 611 – Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments provides methods for the seismic analysis and design of embankments and slopes using the displacement-based approach.*

Geotechnical systems such as abutments, retaining walls, and embankments are often designed using an equivalent seismic coefficient generally varying between 0.5 and 1.0 of the peak ground accelerations and assuming rigid behavior of the soil mass. Permanent deformations may be ignored when the pseudo static equilibrium analysis indicates a factor of safety greater than 1.3.

6.14.6 Deep foundations

6.14.6.1 Analysis

Delete the third paragraph and replace with:

Piles shall be explicitly incorporated into the structural model when piles are subjected to liquefaction induced kinematic effects.

Add the following:

Sensitivity studies using appropriate bounds on strength and stiffness used in soil-pile interaction modelling shall be carried out during design to exclude unacceptable failure modes, or unacceptable performance and/or unrealistic performance of the foundations or structure.

Reduction in lateral soil resistance developed at the soil-pile interface due to soil liquefaction shall be incorporated in the design. The effect of liquefaction on both the shape and the magnitude of the p-y curves shall be considered.

Commentary: *Consider the soil layers with excess porewater pressure ratio (r_u) of 0.7 or higher as liquefied. The p-y curves for liquefiable soil layers may be developed using soft-clay p-y models with the residual strength of the liquefied soil. For soil layers with r_u less than 0.7, a reduction factor of (1- r_u) should be used on the static (pre-liquefaction) p-y curves to model the softened conditions.*

An alternative method with SPT N data may be considered as follows:

When an excess pore pressure ratio (r_u) close to 100% is predicted in a given foundation soil layer, the soil reaction computed from the non-liquefied "p-y" curve should be reduced by multiplying by the p-multiplier (m_p) shown below:

$m_p = 0.0031N + 0.00034N^2$

where, N = clean sand equivalent corrected blow count SPT$(N_1)_{60cs}$

Where the predicted excess pore pressure ratio in a given soil layer is less than 100%, the p-multiplier should be proportionally scaled by the ratio of $100/r_u$ for that layer.

Modification to the p-y curves to account for the weakening effect the liquefied soil has on overlaying and underlaying non-liquefied strata should be considered [Ref. CALTRANS Memo to Designers 20-15 Attachment 1, dated May 2017].

The sensitivity of the strength and stiffness of the p-y curves on the predicted performance of piled foundations should be assessed during design by factoring the values by ½ to 2 as recommended by "Recommended Design Practice for Pile Foundations in Laterally Spreading Ground" Ashford (2011).

6.14.6.3 Axial resistance

Delete and replace with:

The axial resistance of deep foundations in liquefiable soil shall be evaluated for the following conditions:

1. The factored axial resistance in the liquefied and non-liquefied conditions shall be greater than the combination of dead load, seismic demand and the accompanying live load specified in Table 4.16.
2. The factored axial resistance in the post-liquefaction condition shall be greater than the combination of down drag, unfactored dead load and the accompanying live load specified in Table 4.16.

Commentary: *The Ministry uses performance-based design for deep foundations in liquefiable soil.*

Geotechnical resistance factors are identified in Table 6.3 of the BC Supplement to CHBDC S6:19.

The dead load used for the liquefied and non-liquefied conditions includes vertical acceleration which may be accounted for by a factor or be explicitly modelled in the analysis.

6.14.7 Abutments and retaining walls

6.14.7.1 Abutment and approach fill interaction

Delete the second paragraph and replace with:

The seismic design of abutments shall consider:

a) forces arising from seismically induced lateral earth pressures in accordance with Clause 6.14.7.2;
b) additional forces arising from wall inertia effects, including the weight of soil that is immediately above the heel of the wall; and
c) the transfer of seismic forces from the superstructure through bearing supports.

Add the following:

Unless Consented to by the Ministry, design of abutments shall include the following two cases:

1. Combine 100 percent of forces obtained from (a) with 50 percent of forces obtained from (b) and (c)
2. Combine 50 percent of forces obtained from (a), but not less than the static active pressure, with 100 percent of forces obtained from (b) and (c)

If the inertial load of the bridge pushes the abutment into the backfill, the abutment shall be designed for the forces arising from the passive pressure condition and forces obtained from (b) and (c).

Delete the first sentence of the last paragraph and replace with:

Approach slabs providing a structural transition from approach fills to abutments shall be in accordance with Clause 1.7.2 of the BC Supplement to CHBDC S6:19.

Commentary: *Research carried out using centrifuge tests of reduced scale walls by Atik & Sitar (2010) has indicated that the wall inertial forces and lateral earth pressures can be out of phase. The load combinations above reflect an approximation to capture this behaviour and are taken from AASHTO LRFD Bridge Design Specification, 9th Edition.*

The effective abutment stiffness and ultimate passive resistance can be determined based on CALTRANS SDC Ver.2 Clause 6.3.1.2. The backfill passive pressure force varies nonlinearly with the abutment displacement. The bilinear model in CALTRANS SDC Ver. 2 Clause 6.3.1.2 is based on experimental studies using engineered structural backfill to a relative compaction of at least 95%.

6.14.7.2 Seismic forces on retaining walls

Add the following:

The point of application of the dynamic portion of the earth pressure is 0.6 H above the base of the wall.

6.14.8 Liquefaction

6.14.8.1 Evaluation of liquefaction potential

6.14.8.1.2 Liquefaction assessment

Delete and replace with:

A liquefaction assessment shall be conducted for all foundation soils and adjacent soils that impact the behaviour of structures, bridges and geotechnical systems.

Future ground water levels due to climate change effects over the full life of the structure shall be considered in the liquefaction assessment.

Commentary: *Liquefaction should be considered for all structures where failure could have safety implications for highways users. This would include sign bridges, large cantilever sign structures and high-mast lighting but would not typically include standard signal and luminaire poles.*

6.14.8.1.3 Liquefaction potential of foundation soils

Delete the last two paragraphs and add the following:

When evaluating liquefaction potential with various methods, each method shall be followed independently.

The evaluation shall incorporate variability of the in-situ penetration resistance and earthquake ground motions.

Commentary: *Saturated low-plastic silts exhibiting sand-like behaviour, sands, sand-silt mixtures, gravels confined by low permeability soil layers, and gravel-sand mixtures, all have a high potential for liquefaction. Silts and sand-silt mixtures with PI < 7 are classified as soils exhibiting sand-like behaviour.*

There are two commonly accepted methods of evaluation for liquefaction potential referred to as CPT-based and SPT-based methods. Other evaluation methods, including Vs-based liquefaction evaluation, may only be used for high level initial screening.

The CPT-based method of assessment for liquefaction potential is preferred because of the repeatability of the test, production of near continuous penetration resistance and pore pressure profiles, accurate identification of soil stratigraphy, and the availability of a data base correlating CPT resistance to liquefaction triggering from past earthquakes. The CPTs should be paired with boreholes to correlate/confirm soil types and fines content.

Use the SPT-based method of assessment for liquefaction potential, when SPT blow counts or equivalent SPT blow counts are available from BPTs, when CPT data is not available or CPTs are not feasible in the soils investigated. Energy measurements are required when SPT data is used for assessment for liquefaction potential. Site-specific correlations of energy corrected BPT blow counts with equivalent SPT blow counts are required when using BPT data in assessment for liquefaction potential.

When both SPT and CPT results are available for a given site, CPT data may be correlated to SPT data or vice versa and used along with the applicable method of assessment for liquefaction potential.

Alternatively, the assessment for liquefaction potential results may be weighted using engineering judgement to account for epistemic uncertainty associated with calculating the representative cyclic resistance ratio profile. CPT data should not be converted to equivalent SPT values for later use with the SPT-based liquefaction triggering method.

The assessment for liquefaction potential using the simplified stress-based method should be carried out in a manner consistent with how the method was developed. Using techniques and adjustment factors from one variant of a method with other variants is not appropriate and shall not be performed.

The evaluation for liquefaction potential shall be conducted in accordance with the following table:

Table: 6.14.8.1.3
Minimum methods for evaluation of liquefaction potential *

SPC	Lifeline	Major route	Other
1	Routine	Simplified	Simplified
2	Routine	Simplified	Simplified
3	Rigorous	Routine	Simplified

* More complex evaluation methods may be used when agreed to or directed by the Ministry.

Simplified: Simplified analysis shall be based on "Method 1: Simplified stress-based method of analysis" as per the CHBDC commentary C.6.14.8.1 using following criteria:

1. Peak ground acceleration shall be adjusted for the site using non-liquefied soil properties. In SPC=1, PGA corresponds to the ground motions with a return period of 2475 years.
2. Magnitude shall be the mean earthquake magnitude obtained from the de-aggregation of PGA.
3. At least a typical degree of site understanding as defined in Clause 6.5.3 is required.

Routine: Routine analysis shall be based on the 1D dynamic site response analysis with equivalent linear models using non-liquefied soil parameters satisfying the following criteria:

1. Simplified method shall be conducted as well for comparison.
2. The ground motions for liquefaction analysis shall be established according to the criteria in Clause 4.4.3.6.
3. A typical degree of site understanding as defined in Clause 6.5.3 is required and downhole shear wave velocity measurement shall be collected.

Rigorous: Rigorous analysis shall be based on 2D or 3D non-linear effective stress analysis using the following criteria:

1. Address pre-triggering, triggering, and post-triggering aspects of liquefaction.
2. Routine 1D analysis shall also be conducted for comparison.
3. The ground motions shall be established according to the criteria in Clause 4.4.3.6.
4. A high degree of site understanding as defined in Clause 6.5.3 is required and downhole shear wave velocity measurement shall be collected.
5. Requires an independent peer review as per Clause 4.4.6.1.

For Routine and Rigorous analysis, sensitivity studies shall be completed to mitigate the potential for unacceptable failure modes and unrealistic performance of the foundations, structures, and geotechnical systems. The sensitivity studies shall consider variations of the soil, foundation, and structure stiffness and strength parameters including the assumed depth to firm ground.

Commentary: *For Routine analysis, the equivalent-linear model provides reasonable results for strains less than about 1%. Equivalent-linear analysis should be used with caution where large strains are likely to occur.*

When Routine or Rigorous analysis is required as per Table 6.14.8.1.3, such analysis can be omitted if a liquefaction potential assessment, conducted using Simplified analysis methods, indicates no liquefaction potential with a high factor of safety.

Computer programs with non-linear effective-stress models are available to assess liquefaction triggering and the consequences of liquefaction. Computer programs capable of modelling pre-triggering, triggering, and post-triggering aspects of soil liquefaction responses are considered suitable for Rigorous analyses. Also, the effects of soil-structural interaction and ground improvement can be included.

Detailed geological and geotechnical site characterization, selection and calibration of constitutive models used in analysis, determination of material parameters and their spatial variability, general limitations of numerical modeling, development of input ground motions that appropriately reflect the seismic hazard of the bridge site, and detailed documentation of methodology, assumptions and findings are important factors affecting the quality of the analysis results. Practitioners should refer to Boulanger and Beaty (2017) for more details on checks and balances required when conducting Rigorous analysis.

For lifeline structures and structures in Seismic Performance Category 3, when uncertainty exists with regards to cyclic resistance of fine-grained soils, the liquefaction susceptibility should be evaluated using laboratory cyclic shear testing of representative undisturbed soil samples.

6.14.8.2 Effects on bridge foundations, culverts, and geotechnical systems

Revise this clause as follows:

Change “culverts” to “culverts and buried structures” at all occurrences.

6.14.8.2.1 General

Add to the existing clause:

If liquefiable soils are identified, then the methods of analysis for estimating liquefaction-induced ground movements shall be based on Table 6.14.8.2.1.

Table 6.14.8.2.1
Minimum methods of estimating liquefaction-induced ground movements*

SPC	Lifeline	Major route	Other
1	Simplified	Simplified	Simplified
2	Simplified	Simplified	Simplified
3	Rigorous	Simplified	Simplified

* More complex evaluation methods may be used when agreed to or directed by the Ministry.

Simplified: Simplified method includes empirical-based approaches, semi-empirical approaches, and Newmark-based analysis using the following criteria:

- Peak ground acceleration shall be based on non-liquefied soil properties. In SPC=1, PGA corresponds to the ground motions with a return period of 2475 years.
- Ground acceleration shall be based on the method used in Clause 6.14.8.1.3.
- Magnitude shall be the same as used in Clause 6.14.8.1.3.
- A typical degree of site understanding or better as defined in Clause 6.5.3 is required.

Commentary: *Excessive load or displacement demands caused by lateral spreading and settlements are commonly mitigated using ground improvement techniques or structural enhancement. Both options should be considered to develop the most appropriate solution. CALTRANS- "Memo To Designer 20-15, Lateral Spreading Analysis for New and Existing Bridges" provides guidance on how to calculate the foundation restraining action.*

In addition to Youd et al. (2002) and Newmark-based analysis using residual strength in the liquefied condition, semi-empirical approaches, such as Faris et al. 2006, Zhang et al. 2004, Idriss and Boulanger 2008, can be considered to estimate liquefaction induced lateral movement and settlements.

For estimates of liquefaction induced lateral displacement using the simplified method, at least two approaches from the above methods should be selected to evaluate a likely range of potential lateral displacements. Engineering judgement should be used to determine lateral displacement values to be used in the assessment of the structure or geotechnical system performance. The assumptions, limitations, and applicability of the chosen methodologies should be assessed.

Rigorous: Rigorous analysis shall be based on 2D or 3D non-linear effective stress analysis or non-linear total-stress analysis using following criteria:

1. The type of analysis shall be Consented to by the Ministry.
2. An independent peer review shall be carried out.
3. The analysis shall address pre-triggering, triggering, and post-triggering aspects of liquefaction.
4. The Simplified method of analysis shall be conducted for comparison.
5. The ground motions shall be the same as those used for the rigorous analysis for liquefaction potential assessment from Clause 6.14.8.1.3.
6. A high degree of site understanding as defined in Clause 6.5.3 is required.
7. Sensitivity analysis shall be undertaken of the effects of liquefaction using a deformation range from one-half to double the deformation amount obtained from the rigorous analysis.

Commentary: *Simplified and Rigorous analysis include the evaluation for liquefaction induced flow failure using post-liquefaction soil parameters. The minimum factor of safety against flow slide failure should be 1.0.*

The state-of-practice constitutive models used in evaluating the consequences of soil liquefaction such as lateral displacements predict different results due to epistemic uncertainty. To address the epistemic uncertainty, rigorous analysis may be carried out using more than one state-of-practice constitutive model and the results may be weighted using engineering judgement when estimating the consequences such as displacements.

6.14.8.2.2 Liquefaction around bridge foundations

Add the following to subclause (b):

Effects of increased spectral accelerations at periods longer than 1 second due to soil liquefaction shall be evaluated by the designer using 2D or 3D rigorous ground response analysis.

6.14.8.2.3 Mitigation measures

Add the following:

Where soil improvement is achieved using vertical reinforcement, as referred to in Clause 6.14.8.2.3 (b), by installing rigid inclusions with a load transfer platform constructed between the underside of the foundation and the top of rigid inclusions, the rigid inclusions shall be designed to carry the vertical and lateral gravity and seismic loads.

The loads transferred from the bridge foundation onto the rigid inclusions shall be assessed based on Rigorous 2D or 3D methods as per Clause 6.14.8.1.3.

Commentary: *There is a recent trend of using rigid inclusions to vertically reinforce foundation soils to enhance both vertical and lateral stiffness of foundation soils. These designs utilize shallow foundations that are not structurally connected to the rigid inclusions. The foundation system involves the construction of a load transfer platform, often in the form of a compacted granular layer, between the underside of the foundation and the rigid inclusions. The composite foundation system eliminates the need to design for large bending moments and tensile forces transferred at the pile cap-deep foundation interface when using conventional pile foundations. Designers should carry checks to confirm that the rigid inclusions have capacity to support the vertical and lateral loads and displacements imposed on them. The unreinforced rigid inclusions are susceptible to brittle failure and therefore the tension and shear stresses in rigid inclusions for both static and seismic loads transferred from the geotechnical/foundation system are required to be evaluated during the design.*

6.14.8.3 Combined kinematics and inertial loads

Delete the second and third paragraphs and replace with:

All bridges in Seismic Performance Category 2 and 3 shall consider the potential simultaneous occurrence of inertial loads from the structure and kinematic loads on foundations, considering the phasing and locations of these loads on foundation elements. The foundations shall be designed such that the structural performance is acceptable when subjected to combined kinematic and inertial loads.

In the absence of a Rigorous soil-structure interaction analysis of the soil-foundation system, the effects of kinematic loading shall be evaluated and combined with inertial loading as follows:

1. 100% kinematic demand ± 50% inertial demand

Inertial demands shall be computed from the requirement in Clause 6.14.8.2.2 (b).

In cases where NTHA is used, kinematic effects may be incorporated by applying displacement time-histories obtained from Rigorous analysis to the base of the non-linear p-y springs (ref. Clause 6.14.8.2.1) distributed along the piles of the structural model.

The percent contribution of inertial loads identified above may be reduced when supported by Rigorous soil-structure interaction analysis of the soil-foundation system. Such analyses shall be subjected to an independent peer review and Consented to by the Ministry. The inertial contribution shall not be less than 25%.

In cases where soil softening does not reduce the inertial effect, then a special assessment shall be undertaken to develop an appropriate combination of inertial plus the applicable kinematic effects.

Commentary: *It is important to recognize that the combination of kinematic loads with inertial loads in a pseudo-static analysis will be subject to considerable uncertainties. Emergence of well-established criteria for the combination of these loads is highly unlikely.*

The contribution of inertial loads to be combined with the kinematic loads has been shown to be dependent on a number of complex factors including, but not limited to, the following (ref. Koshravifar & Nasr, 2021 submitted for publication):

a) Differences in the location of maximum pile bending moments due to kinematic and inertial load: the depth at which the maximum bending moment occurs due to kinematic loads that are distributed along the embedded length of piles is different from the depth at which the maximum bending moment occurs due to inertial loads applied at the top of the piles (ASCE 64-14).
b) Depth of soil liquefaction: the inertial loads on foundations installed in sites with shallow soil liquefaction are anticipated to be larger than for sites with deep soil liquefaction.
c) Duration of ground shaking: the inertial loads from long-duration ground motions are likely to be larger than from short-duration ground motions, since soil liquefaction may be triggered earlier on during strong shaking in sites subjected to long-duration ground motions. On average, the increase is reported to be about 15% (ref. Koshravifar & Nasr, 2021).
d) Pile groups versus individual piles: the inertial loads on piles within pile groups are likely to be smaller than for individual piles due to sheltering effects.
e) Pile stiffness relative to surrounding soil: the inertial loads from foundations supported on large diameter and stiff piles are likely to be larger than on slender small diameter piles.
f) Phase of ground motions: the inertial loads should be added to the kinematic loads unless it can be demonstrated that the ground motions are out-of-phase with the ground displacements.

6.14.9 Associated seismic hazards

6.14.9.1 Stability and deformation of slopes

Delete the first paragraph and replace with:

Embankments comprising soils that are not susceptible to liquefaction or cyclic mobility shall be analyzed using requirements of Clause 6.14.4.2.

Delete the first sentence of the third paragraph.

Delete the fourth paragraph and replace with:

Embankments comprising soils that are susceptible to liquefication, or cyclic mobility shall be analyzed using requirements of Clause 6.14.8.

Commentary: *The factor of safety threshold has been included in the Supplement Cl. 6.14.4.2.*

6.19 Mechanically stabilized earth (MSE) structures

6.19.2 Design

Add the following clause:

6.19.2.1 General

The maximum height for MSE walls using extensible soil reinforcing shall be 9 m. The maximum height of MSE walls using inextensible soil reinforcing shall be 12 m.

Inextensible soil reinforcement shall be steel. Extensible reinforcement shall be geogrid.

Only MSE Wall systems listed in the Ministry Recognized Products List may be used. MSE Walls shall meet all requirements given in the Recognized Products List.

Wire used in wire facing or soil reinforcing components of all MSE walls shall be galvanized and shall have a minimum thickness determined based on a 100-year design life and corrosion-resistance durability requirements.

MSE walls in Seismic Performance Category 2 and 3 must have anchored connections of the facing to the soil reinforcing that do not rely on friction. MSE walls in SPC 2, using facing blocks conforming to SS942, may use friction for their connections when Consented to by the Ministry.

Two-stage MSE walls shall only be used where Approved by the Ministry.

Commentary: *Two-stage MSE walls have had significant performance issues including failure. If a two-stage wall is approved, the designer shall liaise with MSE wall supplier(s) to confirm wall system details prior to tendering. Only wall systems that meet the project-specific criteria shall be shown on the Plans. Two-stage MSE walls shall be constructed so that there is no void space between the initial stage 1 wall and the final stage 2 facing after construction. The connections used to connect the second stage fascia panels to the main gravity wall structure shall be designed to minimize movement between panels during shaking during seismic load cases.*

a) <u>Mechanically Stabilized Earth (MSE) Walls at Bridge Abutments and the embankment bridge interface zone</u>

Inextensible soil reinforcing shall be used. Geogrid extensible soil reinforcing shall only be used with Consent of the Ministry based on a project-specific evaluation.

The walls shall have precast reinforced concrete facing panels. Alternative facings may be acceptable for LVR structures when Consented to by the Ministry.

A reinforced concrete coping shall be used along the top of the walls.

Any portion of an MSE wall within the embankment bridge interface zone (6.14.2.2) shall also be considered as an abutment wall.

b) <u>Other Mechanically Stabilized Earth (MSE) Walls</u>

Inextensible or geogrid extensible soil reinforcing may be used.

Non-geogrid extensible soil reinforcing may only be used with the Consent of the Ministry based on a project specific evaluation.

MSE walls with wire mesh facing, dry cast concrete block facing, or rock stack facing shall only be used when Consented to by the Ministry.

Wire mesh facing shall only be used in Ministry Service Areas 1, 2, 3, 4, 6 and 27 unless otherwise Approved. The design shall include provisions to ensure long-term durability for the wire facing when exposed to spray or surface runoff containing de-icing chemicals.

Commentary for MSE walls: *Corrosion of wire-faced MSE walls has occurred prematurely on Ministry walls. Wire-faced walls need to be carefully designed for site-specific environment and exposure conditions. Exposure to drainage, runoff and spray containing de-icing salts requires a corrosion evaluation during the design phase. The Service Areas listed above where wire-faced walls may be considered have been chosen since they are areas where these facings have not been reported to have premature corrosion in service and where the walls are subject to rain that can help remove de-icing chemicals from the facing. Even in these listed Service Areas, careful consideration of the site-specific corrosion conditions is needed to verify the appropriateness of the use of wire-faced walls.*

The designer needs to consider the extent of quality control and quality assurance testing for the soil reinforcement for the specified wall systems and add these requirements to the Plans.

6.19.9.5 Global and Compound Stability

Add the following:

Global stability is the responsibility of the project geotechnical Engineer of Record. The geotechnical engineer of record shall use Table 6.2b herein for resistance factors and factors of safety for global stability.

Compound stability responsibility can reside with the geotechnical Engineer of Record and/or the MSE wall supplier. If responsibility is to reside with the MSE wall supplier, soil parameters for retained soil required for limit equilibrium stability analysis shall be provided to the supplier and the supplier shall be informed of this responsibility. Factors of safety for compound stability shall not be less than those of Table 6.2b for global stability of geotechnical systems.

Delete the last paragraph and replace with:

For compound stability, the restraining force of each soil reinforcement layer intersected by the failure surface shall be the lesser of either the long-term strength of each extensible reinforcement layer, the 100-year corroded strength of each inextensible reinforcement layer or the strength mobilized by the restrained length.

6.19.14.3 False abutments

Add the following:

False abutments shall consider soil-structure interaction unless the piles are isolated using casing.

Commentary: *Utah Department of Transportation Document UT-13.04, Lateral Resistance of Piles Near Vertical MSE Abutment Walls, presents comprehensive research and analysis related to the complex soil-structure interaction behavior.*

When designing a false abutment MSE wall with an embedded pile without isolation, an integrated design team including the bridge engineer, geotechnical engineer and potential MSE wall suppliers should be involved. Several iterations of the design may be expected for this type of wall configuration.

The lateral pile resistance for loading towards the MSE wall face should be reduced to account for the presence of the wall. Figure 6.14 in UT-13.04 presents p-multipliers to account for this reduction as a function of offset from wall face and MSE strap length. These p-multipliers should be used in conjunction with p-multipliers for group effects.

Lateral loading on the pile will induce additional load in the MSE reinforcement which must be resisted with additional soil reinforcement tied to the pile or pile cap. Appropriate soil-structure interaction analysis that incorporate non-linear soil springs (p-y curves or similar) should be completed to estimate this load. The estimated additional reinforcement load from the soil-structure interaction analysis should be compared with empirical envelopes presented in UT-13.04 Figures 6.15 and 6.16 and the larger of the two values used for design.

After estimating the additional soil reinforcement loads, the MSE wall supplier should be consulted to confirm the estimated additional loading can be reasonably resisted using readily available MSE strap types.

Add the following clause:

6.20 Lightweight fills

All lightweight fills shall be adequately protected against wheel loads, ground water, road salts, weather, fire, flotation under flood conditions and fuel spills.

Where walls are used to contain flammable lightweight fills, the walls shall provide a 2-hour fire rating.

Any geotechnical system or landscaping above the lightweight fills shall be designed such that the protective membrane covers for the lightweight fill shall not be compromised.

Flotation forces corresponding to inundation of the lightweight fill under the 200-year flood level shall be addressed in the design, regardless of any flood protection provided for the area in which the lightweight fill is to be constructed.

Expanded Polystyrene (EPS) lightweight fills shall meet the following requirements:

- EPS shall be supplied in the form of blocks. It shall be classified as to surface burning characteristics in accordance with CAN/ULC-S102.2-03-EN, having a flame spread rating not greater than 500.

- The minimum compressive strength, measured in accordance with ASTM D1621, shall be 125 kPa at a strain of not more than 5%.
- The density of EPS shall not be less than 22 kg/m³.
- EPS blocks shall be fully wrapped with minimum 0.254 mm (10-mil) thick, black polyethylene sheeting.
- Polyethylene sheeting joints shall be overlapped by a minimum of 0.5 m.
- EPS blocks shall have a minimum 1.2 m granular cover vertically and horizontally.

Shredded rubber tires or hog fuel (wood waste) shall not be used as fill.

Add the following clause:

6.21 Retaining Walls

Retaining wall types shall meet the durability and aesthetic requirements specified for the project and shall be subject to the Consent of the Ministry.

Design issues not addressed by S6:19 or herein shall meet the requirements of the latest edition of AASHTO LRFD Bridge Design Specifications.

Surface drainage and drainage of the backfill material shall be addressed in the design of the walls and details shall be shown on the Plans.

Additional full-length anchors, tie-backs, soil reinforcing and/or soil nails shall be installed to allow for future extraction for long-term inspection and testing. The minimum number of additional elements provided for each wall shall be the greater of 2% of the number of elements required by design, or 2. Additional full-length double corrosion protection anchors for long-term inspection and testing may be omitted when Consented to by the Ministry.

Add the following clause:

6.22 Geosynthetic reinforced soil (GRS)

Geosynthetic reinforced soil (GRS) methods may be used when agreed to or directed by the Ministry. GRS design shall follow S6:19, with the exception of methods for internal stability analysis, which shall be in accordance with FHWA-HRT-017-080.

Commentary: *At this time, use of GRS is not considered appropriate on Major or Lifeline routes as the Ministry evaluates this design method. This is to be reevaluated under future revisions of the Code and Supplement.*