MEMO

TO:	Crystal Bleackley, P.Eng.
COMPANY:	British Columbia Ministry of Transportation & Infrastructure (MoTI)
FROM:	Blair Gohl, Ph.D., P.Eng.; Makram El Sabbagh, P.Eng. & Bradchih Cheng, P.Eng.
DATE:	17 October 2023
CC:	Casey Leggett, P.Eng.; Mike Boissonneault & Michael Carreira, P.Eng.
PROJECT NO.:	VG07796A05
SUBJECT:	Bridge No. 2655 – West Abutment Slope Stabilization Using EPS

1 INTRODUCTION

Lightweight, expanded polystyrene (EPS) is proposed to replace portions of abutment fill at the west abutment of Bridge No. 2655. The use of lightweight EPS will reduce abutment fill loading of the underlying clay foundations and improve static and seismic slope stability due to reduction in driving shear stresses in the clay. Details of soil profiles and engineering soil properties are provided in the WSP geotechnical report dated August 4, 2023.

As per direction provided by MoTI, it is our understanding that the Colquitz bridges (both Bridge No. 2655 and Bridge No. 1378) are considered major-route bridges and have a "typical" consequence classification associated with exceeding limit states under static or seismic loading conditions based on CSA S6-19. Based on the latest 2022 site investigation data, MoTI has reviewed the degree of geotechnical understanding of Bridge No. 2655 and has classified the bridge site as having a "high degree of understanding" as defined in the Canadian Highway Bridge Design Code CSA S6-19. This site classification has led to a target factor of safety (FoS) of 1.43 for abutment slope stability under static loading conditions. WSP's evaluation of abutment slope stability under static loading conditions (discussed in the geotechnical report dated August 4, 2023) indicate adequate stability for the east abutment of Bridge No. 2655, assuming undrained and drained soil response in the clay subsoils. It is only at the west abutment of Bridge No. 2655 that computed static factors of safety are less than 1.43. Due to the time involved to obtain a code exemption request for the west abutment of Bridge No. 2655, MoTI requested that WSP evaluate potential methods of slope stabilization for the west abutment of Bridge No. 2655.

2 LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSIS

Limit equilibrium (LE), two dimensional (2D) slope stability analysis has been carried out for the west abutment using the program SLOPE/W (GeoStudio, 2016) and the Morgenstern – Price method of slices. A soil profile along the longitudinal axis of the bridge has been considered. A groundwater table elevation of +3.3 m has been considered. The zone of abutment fill replacement using EPS has been selected to achieve a target FoS of 1.43 under static loading conditions. The critical circular failure surface (with the lowest factor of safety) was selected using an entry and exit method of analysis. Analyses were carried out assuming both undrained and drained shear strengths in the clay foundations. Details of soil shear strength and unit weight properties used are provided for the various granular fill and clay zones in the geotechnical report dated August 4, 2023.

The EPS being considered is the EPS39 GeoSpec® Lightweight Fill by Plasti-Fab, with quoted compressive strengths by Plasti-Fab at 5% and 10% strains of 241 kPa and 276 kPa, respectively. We have adopted the compressive strength at 5% strain of 241 kPa in our stability model calculations. We have used an effective cohesion in the EPS of 30 kPa based on NCHRP (2004) guidelines. However, we have not considered modifications due to strain

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incompatibility since the product quoted does not appear to be brittle and has peak strengths at similar strain levels as peak strengths in the clay.

The geometrical assumptions in the LE model are as follows:

- Ground surface elevation at 12.4 m
- Combined thickness of pavement and approach slab = 0.4 m.
- Thickness of compacted granular base course material on top of the EPS = 0.2 m
- Thickness of EPS = 2.9 m

The above layering is equivalent to the following:

- Average road surface elevation = 12.7 m (based on the west abutment bridge drawings) at road centerline
- Combined thickness of pavement and approach slab = 0.4 m
- Thickness of compacted base course = 0.2 m
- Thickness of EPS = 2.9 m, giving a bottom of EPS elevation of 9.2 m, approximately at the elevation of the bottom of pile cap

The length (east to west) of the EPS at the base is 8 m and slopes up at 1.5H:1V to the ground surface. The approach slab extends over the full length of the EPS. Under the approach slab, the EPS is overlain by 0.2 m of well graded, compacted granular base course followed by a 0.4 m thick slab and pavement structure.

We have also considered a traffic surcharge of 12.5 kPa under static conditions.

A plot of critical slip surface computed using SLOPE/W considering undrained conditions in the clay foundations and no pile reinforcing effect (due to the influence of the battered, steel pipe pile group supporting the west abutment) is shown in Figure 1. The strength of the pavement structure and approach slab has been neglected, which is a conservative assumption. This assumes a critical slip surface that starts from the underside of slab. An approximately vertical slip surface through the EPS is indicated. The computed Factor of Safety of 1.41 is slightly below the target of 1.43. We consider this difference to be minimal. When we include the pile reinforcing effect (52 kN/m, based on the earlier undrained analyses discussed in the geotechnical report) the FS increases to 1.46. The critical failure surface does not pass through the piles.

Drained shear strengths (effective cohesion c and friction angle ϕ) for the various clay zones in the LE model have also been used in the stability assessment. The method of calculating the c-ϕ parameters in the clay has been discussed in the geotechnical report dated August 4, 2023. The computed critical slip surface is shown in Figure 2. Pile reinforcing has no effect on this critical slip surface. The computed Factor of Safety is 1.53, above the target of 1.43. The drained stability analysis indicates that the assumption of an undrained mode of shear failure in the clay soils is more critical.

3 SLOPE STABILITY ANALYSIS USING FINITE ELEMENT MODELING

An additional check of west abutment stability, incorporating the zone of EPS replacement described above, has been carried out using a 2D finite element (FE) model and the program SIGMA/W (GeoStudio, 2023.1.0). This modeling was carried out to check on the pile reinforcing effect of the slope since soil-pile interaction is explicitly considered in the modeling. Strain – strength compatibility of the various geotechnical materials is also directly considered. Undrained shear strengths were used for the various clay zones shown in Figure 1. A plot of the FE model is shown in Figure 3.

Key features of the FE modeling include:

- Use of an elastic – plastic model to characterize the shear stress – shear strain response of the clay zones (assuming undrained response). For the clay zones, the limiting shear stress (τ_{max}) was set equal to the undrained shear strength (S_u) of the clay zone described in the static stability section of the geotechnical report dated August 4, 2023. A maximum shear strain at failure (γ_{max}) was set, depending on degree of over-consolidation of the clay. For normally to lightly over-consolidated clay zones, γ_{max} = 0.10 was used. For moderately to heavily over-consolidated clay, γ_{max} = 0.025 was used. An effective shear modulus G was calculated as τ_{max}/γ_{max}. Using a Poisson's ratio (ν) of 0.495, approximately representative of undrained clay behaviour, gave an effective Young's modulus $E = 2(1+\nu)G$.
- Use of an elastic – plastic model to characterize the shear stress – shear strain response of the EPS. For the EPS, the limiting shear stress(τ_{max}) was set equal to the uniaxial compressive strength of the material (241 kPa for EPS39) at 5% axial strain divided by 2. This was further reduced by 75% in accordance with NCHRP (2004) guidelines to account for potential failure surfaces at EPS block interfaces. This gave τ_{max} = 30 kPa. It was assumed that peak strength in the EPS would be achieved in a uniaxial compression test at an axial strain of 5%. The effective Young's modulus E was computed as $E = 2\tau_{max}/0.05$. The Poisson's ratio (ν) for EPS was estimated from published data relating EPS unit weight to Poisson's ratio. This gave ν = 0.22.
- Use of a hyperbolic model (Duncan and Chang, 1970; Duncan et al, 1980) to characterize the drained stress – strain response of granular fill zones.
- Use of 2D beam elements to model the west abutment wall, pile cap footing and 2 lines of batter piles.
- Abutment wall constrained against lateral movement due to interaction with the bridge deck and the batter pile group.
- Lateral boundaries of the FE model were permitted to settle but were constrained laterally during gravitational (self-weight) loading of the soil mass.
- Bottom boundary of the FE model constrained against lateral and vertical movement during gravitational (self-weight) loading of the soil mass.

Material model properties used to model the elastic-plastic response of the clay zones and EPS are provided in Table 1. Hyperbolic model parameters used to characterize the drained stress – strain response of fill soils are provided in Table 2. Table 3 provides structural model parameters for the abutment wall, pile cap and batter piles.

Table 1 : Summary of Elastic – Plastic Properties for Clay Soils and EPS

MATERIAL TYPE	γ_t (KN/CU.M.)	τ_{max} (KPA)	γ_{max}	G (KPA)	ν	E (KPA)
EPS	0.38	30	0.05**	492	0.22	1200
1-1 west	19.0	33.4	0.1	3.34E+02	0.495	9.99E+02
1-2 west	19.0	118.2	0.025	4.73E+03	0.495	1.41E+04
1-3 west	19.0	45.6	0.1	4.56E+02	0.495	1.36E+03
2-1 west	19.0	42.4	0.1	4.24E+02	0.495	1.27E+03
2-2 west	19.0	82.2	0.025	3.29E+03	0.495	9.83E+03
2-3 west	19.0	35.0	0.1	3.50E+02	0.495	1.05E+03
3-1 west	19.0	24.6	0.1	2.46E+02	0.495	7.36E+02

MATERIAL TYPE	γ_t (KN/CU.M.)	τ_{max} (KPA)	γ_{max}	G (KPA)	ν	E (KPA)
3-2 west	19.0	54.0	0.1	5.40E+02	0.495	1.61E+03
3-3 west	19.0	33.2	0.1	3.32E+02	0.495	9.93E+02
4-1 west	19.0	25.5	0.1	2.55E+02	0.495	7.63E+02
4-2 west	19.0	40.4	0.1	4.04E+02	0.495	1.21E+03
4-3 west	19.0	40.0	0.1	4.00E+02	0.495	1.20E+03
5-1 west	19.0	40.0	0.025	1.60E+03	0.495	4.78E+03
5-2 west	19.0	40.5	0.025	1.62E+03	0.495	4.84E+03
5-3 west	19.0	39.0	0.025	1.56E+03	0.495	4.66E+03
6-1	19.0	40.9	0.025	1.64E+03	0.495	4.89E+03
6-2	19.0	40.9	0.025	1.64E+03	0.495	4.89E+03
6-3	19.0	45.5	0.025	1.82E+03	0.495	5.44E+03

γ_t = total unit weight
τ_{max} = maximum shear strength
γ_{max} = maximum shear strain at failure
G = secant shear modulus = τ_{max}/γ_{max}
ν = Poisson's ratio
E = secant Young's modulus = 2(1+ν)G
**For EPS, axial strain at failure = 0.05 and uniaxial compressive strength of mass of EPS blocks = 2 τ_{max}

Table 2 : Summary of Hyperbolic Model Parameters for Abutment Fills

γ_t (KN/CU.M.)	C (KPA)	Φ (DEG.)	K_E	N	R_F	ν
19.6	0	33	600	0.5	0.8	0.33

γ_t = total unit weight
c = effective cohesion
φ = effective friction angle
K_E = Young's modulus constant
N = Young's modulus exponent
R_F = ratio between the asymptote to the hyperbolic shear stress – shear strain curve and the maximum shear strength
ν = Poisson's ratio

Table 3 : Structural Properties

STRUCTURE TYPE	E (KPA)	AREA (M^2/M)	I (M^4/M)
Pile Cap	2.8e7	1.83	0.511
Abutment Wall	2.8e7	0.71	0.065
Batter Pile	4.28e7	0.050	0.00047

Following set-up of the FE model, a self-weight gravitational loading analysis (gravity turn-on) was carried out using the unfactored soil properties described previously. A groundwater table elevation of +3.3 m has been considered for purposes of computing effective stresses in the model. Subsequent gravity turn-on analyses were carried out in which factors F were used to reduce soil Mohr-Coulomb (c,ϕ) strength parameters , giving new factored strength parameters defined below:

$$\tan(\phi_{new}) = \tan(\phi)/F$$

$$c_{new} = c/F$$

The above approach is termed the Strength Reduction Method. With each factored strength reduction, the FE model would indicate increasing strains and displacements in the soil mass, and progressively increasing bending moments and shear forces in the battered piles at the west abutment. The factor F at which excessive soil displacements occurred at the toe of the abutment fill, and displacement vectors/strain contours in the soil mass indicate development of a critical slip surface, this is termed the Factor of Safety of the slope and may be compared against FoS values determined from Limit Equilibrium analysis. Checks were also made that the maximum bending moments (expressed on a per pile basis, taking into account the out of plane spacing of the piles set equal to a value of 1.65 m) did not exceed the plastic bending moment of the steel pipe piles (approximately 235 kN-m). Computed plots of the deformed shape of the FE mesh and soil displacement vectors are shown in Figure 4 at a strength reduction factor F = 1.45. Plots of bending moment and shear force (per pile) are given in Figures 5 and 6, respectively. A maximum bending moment of 200 to 235 kN-m is computed at two depths along the piles. A maximum shear force (V_{max}) per pile of 285 kN is computed at elevation +4.5 m. Examining the location of the critical slip surface in Figure 1, this passes through the piles at approximately elevation +1 m. A pile shear force of 40 kN is computed at the latter elevation from the SIGMA/W model. The shear resisting force (R_{max}) per unit out of plane length (along the transverse axis of the bridge) is computed as

$$R_{max} = nV_{max}/w$$

where n = number of piles in the west abutment pile group (=16) and w is the transverse width of the bridge (approximately 13.0 m). This gives a mobilized shear resistance of 49 kN/m using the above formula and may be compared to a value of 52 kN/m used in the LE analysis.

Examination of FE model output for various F values indicates a Factor of Safety of the slope in the range of 1.45 to 1.50. This is similar to LE model results using undrained shear strengths in the clay where pile reinforcing of the slope was considered.

4 CONCLUSIONS

The zones of EPS replacement described in Section 2 indicate acceptable Factors of Safety (in the range of 1.45 to 1.5) against static slope instability based on LE and FE methods of analysis. The latter have considered undrained shear strengths in the clay subsoils and pile reinforcing of the slope. The computed FoS values are at or above target FoS values mandated by the 2019 Canadian Highway Bridge Design Code CSA S6-19 and BC MoTI's Supplement to CHBDC S6:19.

Special Provisions (SP's) for EPS placement are currently being prepared, including density and unconfined compressive strength of the EPS, methods of placement including shear connection details between EPS blocks, thickness of granular overlay between the EPS and approach slabs to protect the EPS from dynamic wheel loads, and polyethylene encapsulation of the EPS to protect the EPS from potential future fuel spills on the highway.

5 CLOSURE

The design and recommendations provided in this memorandum are for the purpose of slope stabilization at the west abutment of Bridge No. 2655 using EPS.

Recommendations presented herein are based on a geotechnical evaluation of the available information as noted. If conditions other than those reported arise in subsequent phases of the project, WSP should be notified and be given the opportunity to review and revise the current recommendations if necessary. Recommendations presented herein may not be valid if an adequate level of review or inspection is not provided during construction.

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Yours sincerely,

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Attachments: Figures 1 to 6
Limitations

6 REFERENCES

British Columbia Ministry of Transportation and Infrastructure (2022), "Bridge Standards and Procedures Manual, Volume 1: Supplement to CHBDC S6:19".

Canadian Standard Association (2019), "Canadian Highway Bridge Design Code". CSA S6:19.

Duncan, J.M. and Chang, C.Y., 1970. Nonlinear analysis of stress and strain in soils. Journal of the Soil Mechanics and Foundations Division, ASCE, vol. 96, no. SM5, pp. 1629-1654.

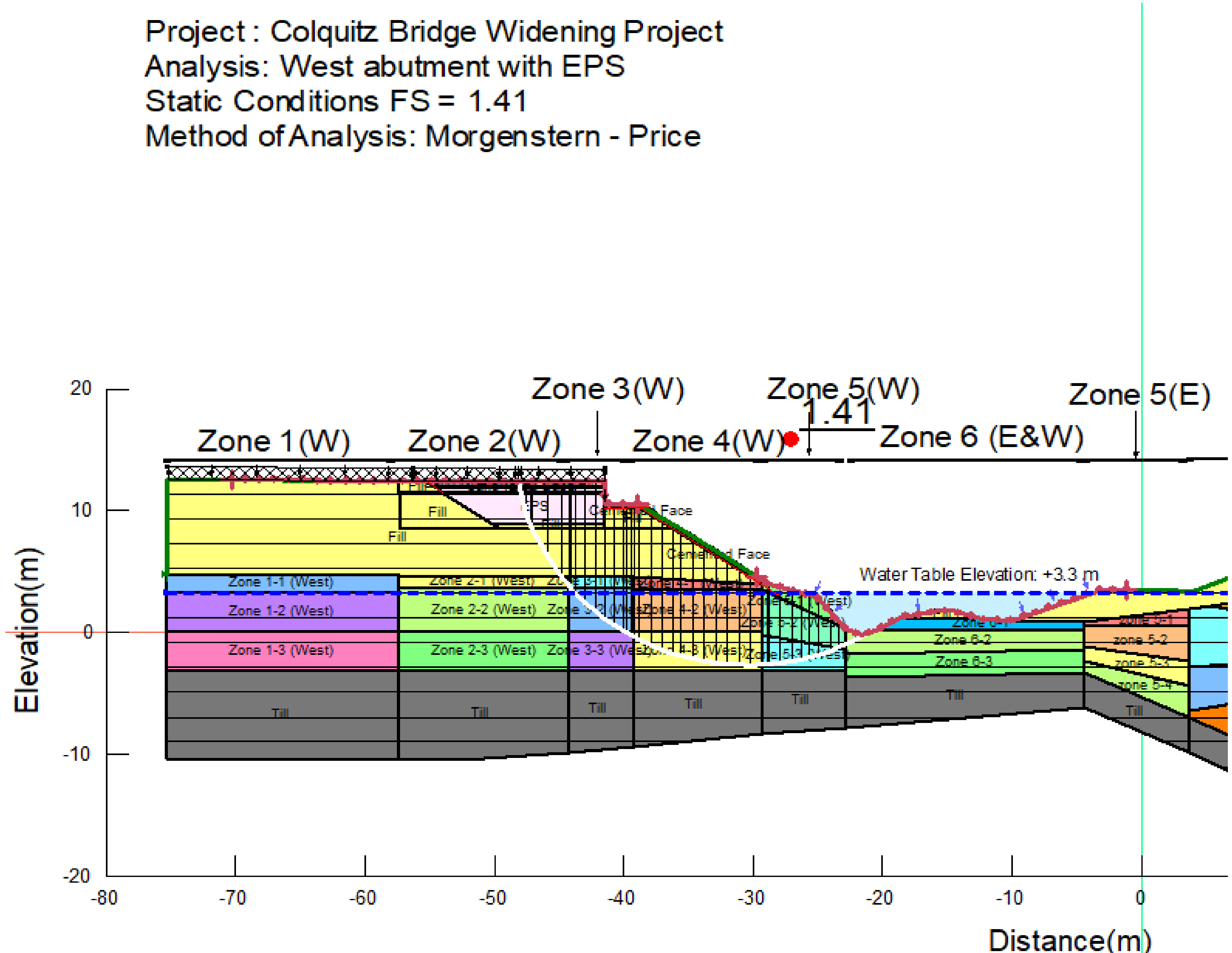
Duncan, J.M., Byrne, P., Wong, K.S., and Mabry, P., 1980. Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analyses of Stresses and Movements in Soil Masses. Report No. UBC/GT/80-01. Department of Civil Engineering, University of California, Berkeley, California (out of print).

GeoStudio (2023), Sigma/W technical manual

GeoStudio (2016), Slope/W technical manual

National Cooperative Highway Research Program (NCHRP) (2004), "Geofoam Applications in the Design and Construction of Highway Embankments", NCHRP Web Document 65 (Project 24-11).

Figures 1 to 6



PROJECT: **Colquitz Bridge Widening Project**
Results of Undrained Static Slope Stability Analysis: Bridge 2655 West Abutment No Pile Reinforcing Effects

CLIENT : British Columbia Ministry of Transportation and Infrastructure

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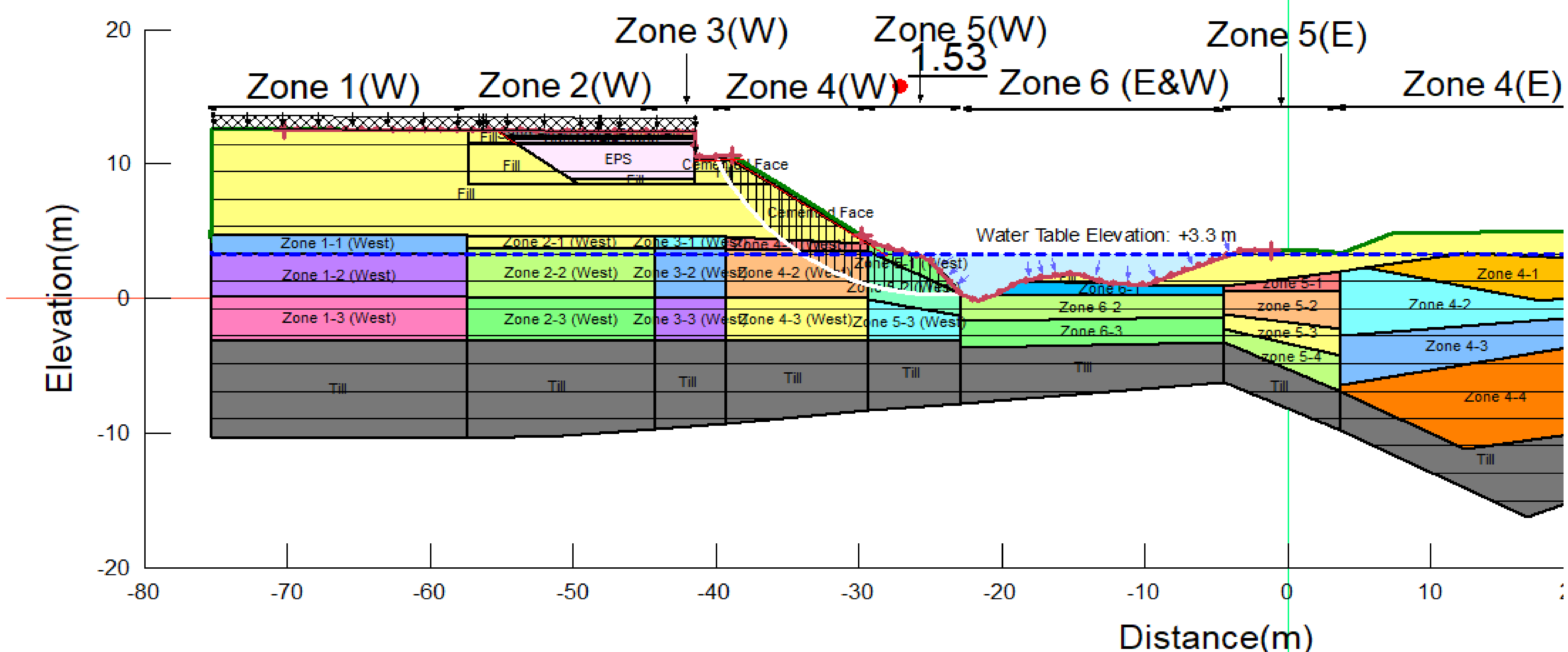
Figure 1-Static West Abutment

Project : Colquitz Bridge Widening Project
Analysis: West abutment with EPS
Static Conditions FS = 1.53
Method of Analysis: Morgenstern - Price



CLIENT : Ministry of Transportation and Infrastructure, British Columbia	PROJECT: **Colquitz Bridge Widening Project**
	Results of Drained Static Slope Stability Analysis: Bridge 2655 West Abutment No Pile Reinforcing Effects

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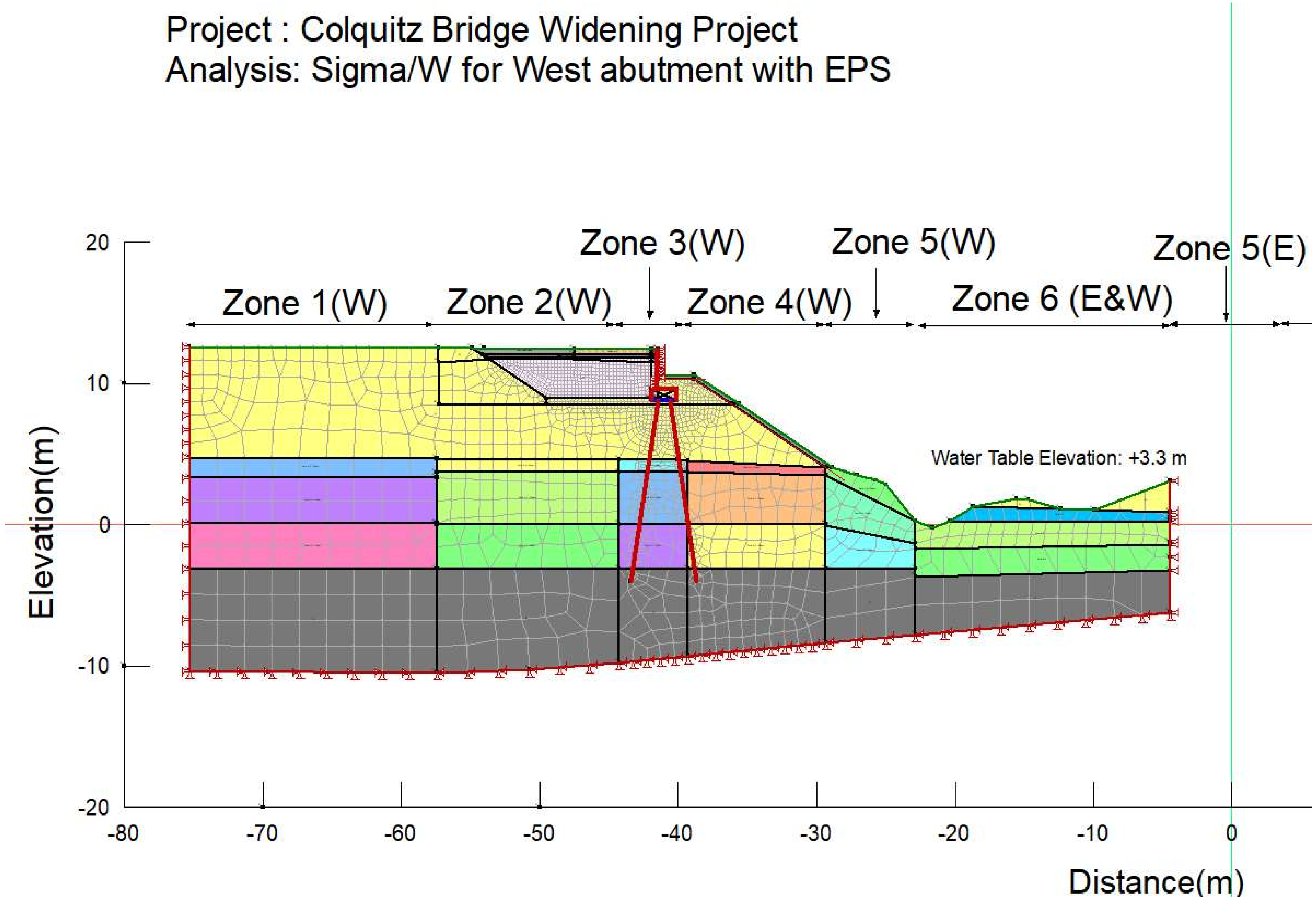
Figure 2-Static West Drained



wsp	PROJECT: **Colquitz Bridge Widening Project**
CLIENT : BRITISH COLUMBIA Ministry of Transportation and Infrastructure	Model of Undrained Static Sigma/W Analysis: Bridge 2655 West Abutment

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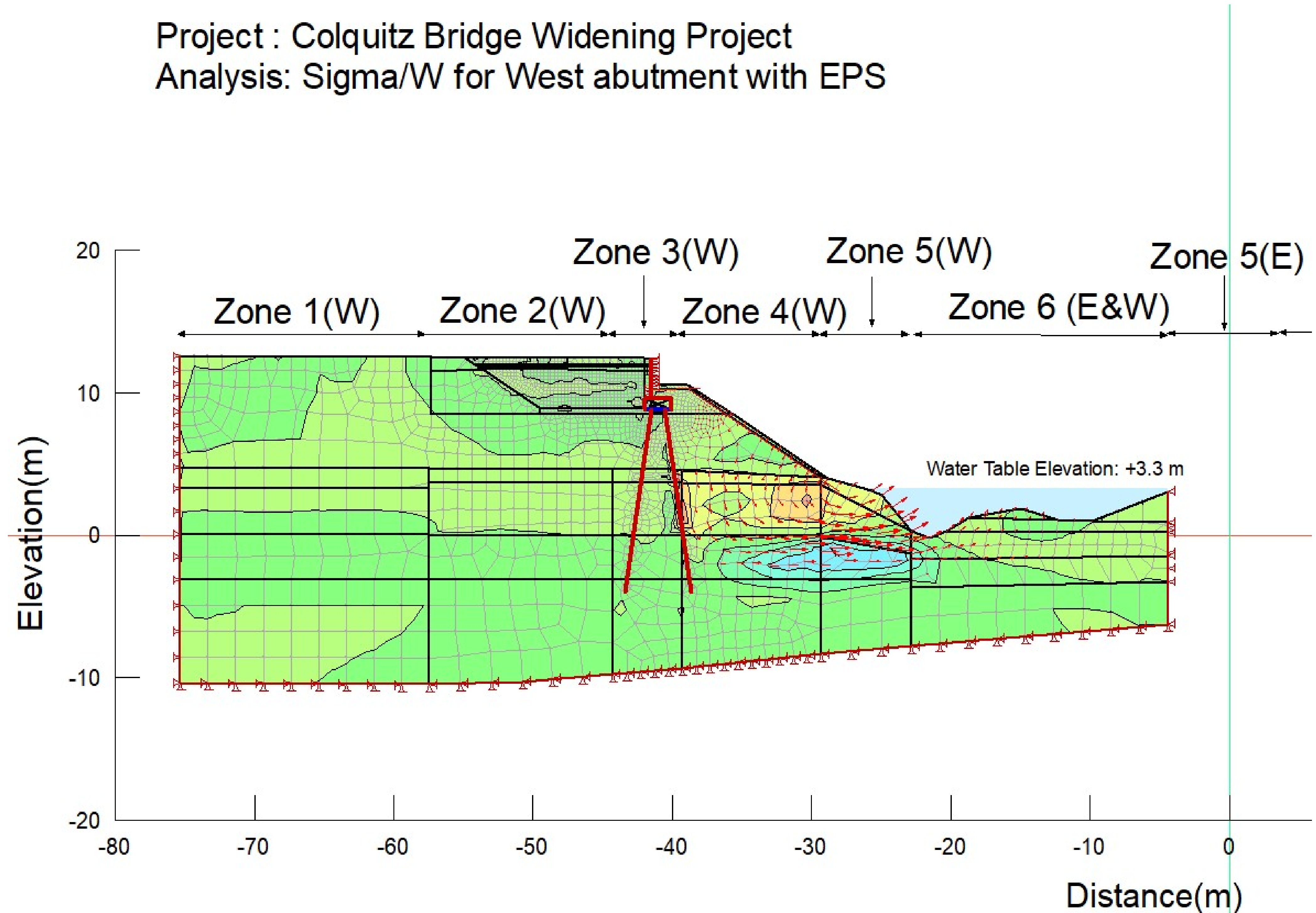

wsp	PROJECT: **Colquitz Bridge Widening Project**
	Results of Undrained Static Sigma/W Analysis: Bridge 2655 West Abutment (Strength Reduction Factor=1.45)
CLIENT : British Columbia, Ministry of Transportation and Infrastructure	DATE: / JOB NO.: / CAD FILE: / FIGURE NO.: 4 / REV 0

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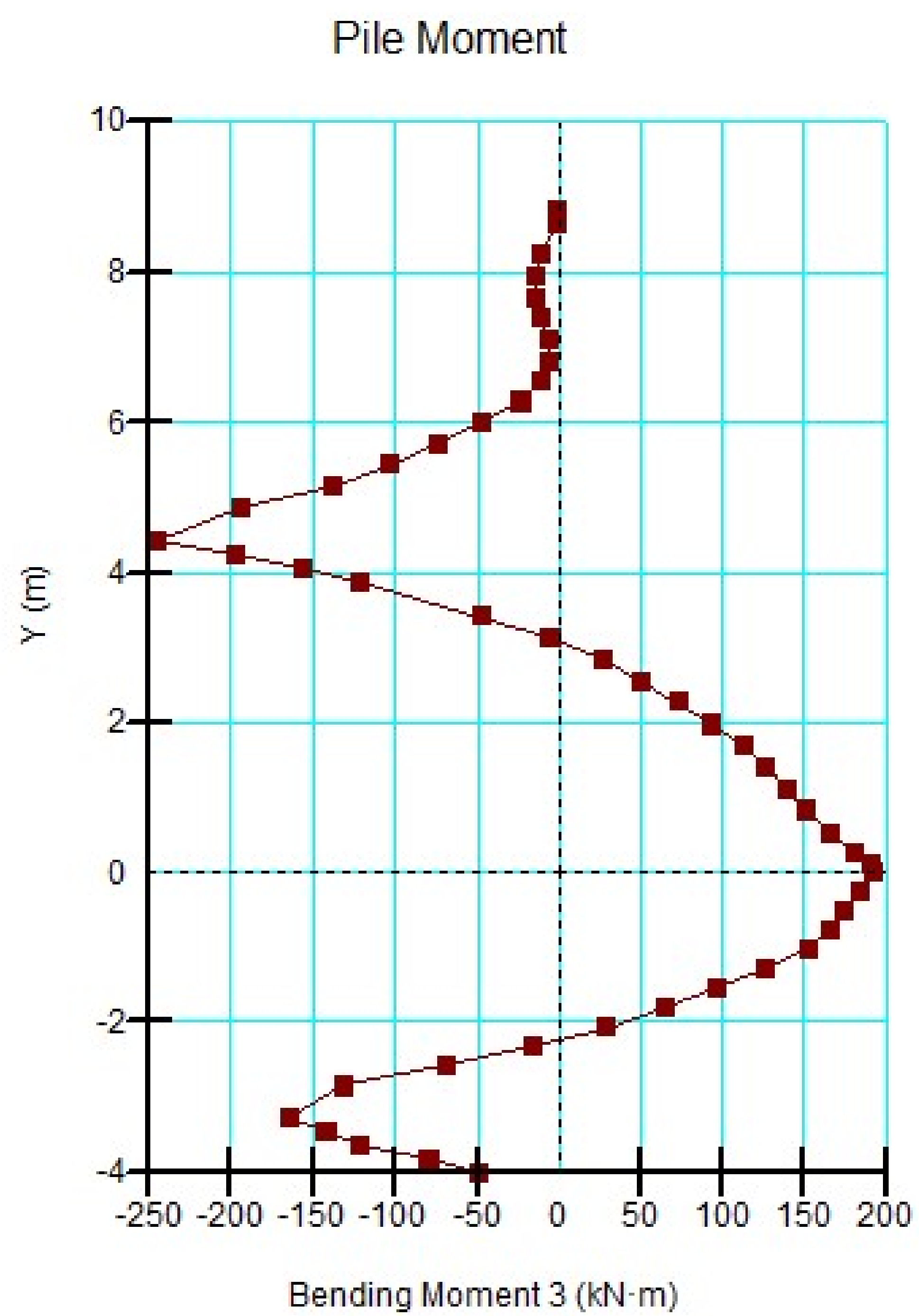
Figure 4-Si



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PROJECT:
Colquitz Bridge Widening Project

Results of Undrained Static Sigma/W Analysis:
Bridge 2655 West Abutment
Pile Moment vs Elevation (Strength Reduction Factor=1.45)

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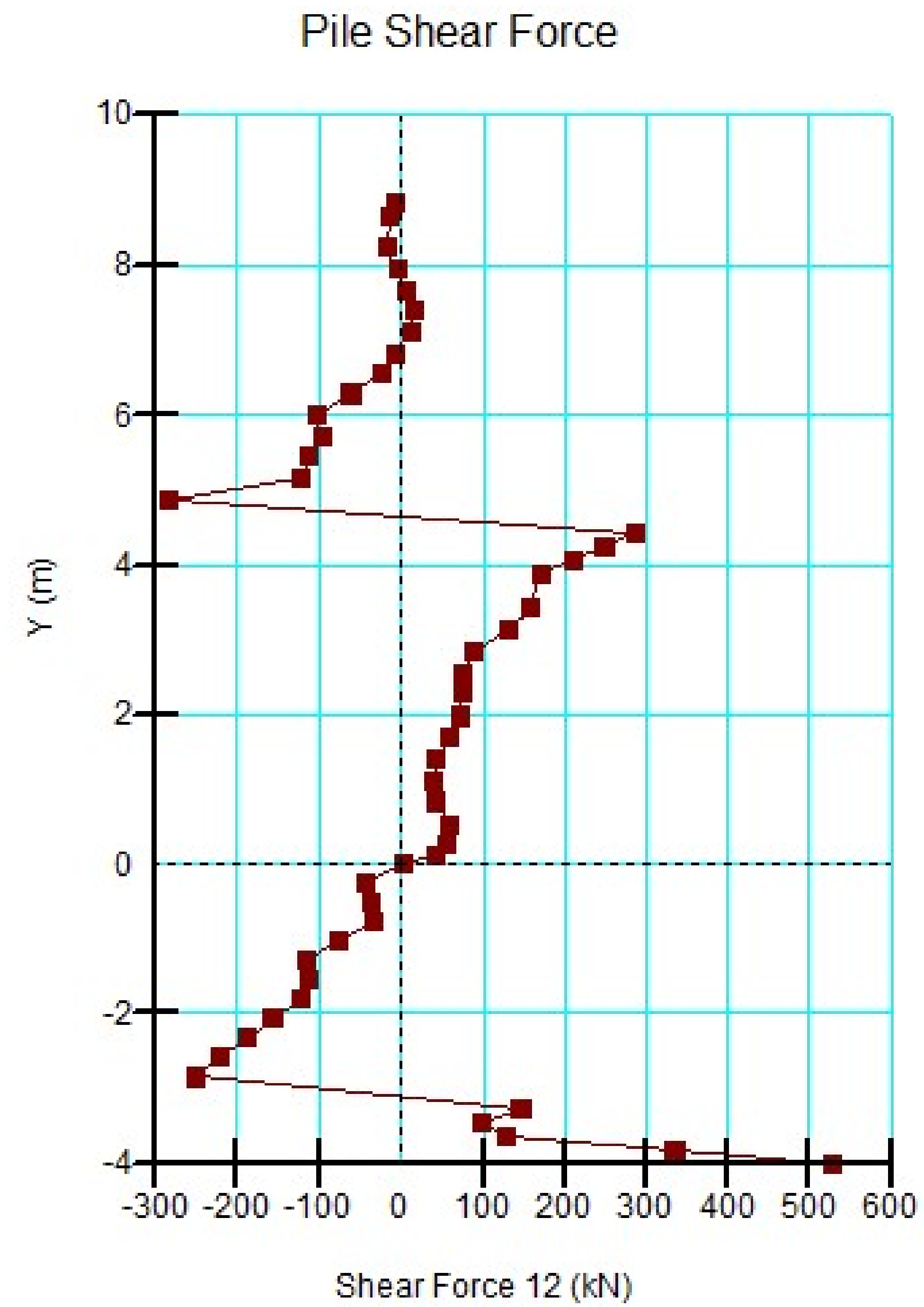
Figure 5-Si



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CLIENT :

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PROJECT:
Colquitz Bridge Widening Project

Results of Undrained Static Sigma/W Analysis:
Bridge 2655 West Abutment
Pile Shear Force vs Elevation (Strength Reduction Factor=1.45)

DATE:	JOB NO.:	CAD FILE:	FIGURE NO.:	REV
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Limitations

Limitations

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11 **Time dependence:** If the project contemplated by the Client is not undertaken within a period of 18 months following the submission of this report, or within the time frame understood by WSP to be contemplated by the Client at the commencement of WSP's assignment, and/or, if any changes are made, for example, to the elevation, design or nature of any development on the site, its size and configuration, the location of any development on the site and its orientation, the use of the site, performance criteria and the location of any physical infrastructure, the conclusions and recommendations presented herein should not be considered valid unless the impact of the said changes is evaluated by WSP, and the conclusions of the report are amended or are validated in writing accordingly.

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WSP will not be liable to update or revise the report to take into account any events or emergent circumstances or facts occurring or becoming apparent after the date of the report.

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13 **Limitations of site investigations:** Site exploration identifies specific subsurface conditions only at those points from which samples have been taken and only at the time of the site investigation. Site investigation programs are a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions.

The data derived from the site investigation program and subsequent laboratory testing are interpreted by trained personnel and extrapolated across the site to form an inferred geological representation and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite this investigation, conditions between and beyond the borehole/test hole locations may differ from those encountered at the borehole/test hole locations and the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

Final sub-surface/bore/profile logs are developed by geotechnical engineers based upon their interpretation of field logs and laboratory evaluation of field samples. Customarily, only the final bore/profile logs are included in geotechnical engineering reports.

Bedrock, soil properties and groundwater conditions can be significantly altered by environmental remediation and/or construction activities such as the use of heavy equipment or machinery, excavation, blasting, pile-driving or draining or other activities conducted either directly on site or on adjacent terrain. These properties can also be indirectly affected by exposure to unfavorable natural events or weather conditions, including freezing, drought, precipitation and snowmelt.

During construction, excavation is frequently undertaken which exposes the actual subsurface and groundwater conditions between and beyond the test locations, which may differ from those encountered at the test locations. It is recommended that WSP be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered at the test locations, that construction work has no negative impact on the geotechnical aspects of the design, to adjust recommendations in accordance with conditions as additional site information is gained, and to deal quickly with geotechnical considerations if they arise.

Interpretations and recommendations presented herein may not be valid if an adequate level of review or inspection by WSP is not provided during construction.

14 **Factors that may affect construction methods, costs and scheduling:** The performance of rock and soil materials during construction is greatly influenced by the means and methods of construction. Where comments are made relating to possible methods of construction, construction costs, construction techniques, sequencing, equipment or scheduling, they are intended only for the guidance of the project design professionals, and those responsible for construction monitoring. The number of test holes may not be sufficient to determine the local underground conditions between test locations that may affect construction costs, construction techniques, sequencing, equipment, scheduling, operational planning, etc.

Any contractors bidding on or undertaking the works should draw their own conclusions as to how the subsurface and groundwater conditions may affect their work, based on their own investigations and interpretations of the factual soil data, groundwater observations, and other factual information.

15 **Groundwater and Dewatering:** WSP will accept no responsibility for the effects of drainage and/or dewatering measures if WSP has not been specifically consulted and involved in the design and monitoring of the drainage and/or dewatering system.

16 **Environmental and Hazardous Materials Aspects:** Unless otherwise stated, the information contained in this report in no way reflects on the environmental aspects of this project, since this aspect is beyond the Scope of Work and the Contract. Unless expressly included in the Scope of Work, this report specifically excludes the identification or interpretation of environmental conditions such as contamination, hazardous materials, wild life conditions, rare plants or archeology conditions that may affect use or design at the site. This report specifically excludes the investigation, detection, prevention or assessment of conditions that can contribute to moisture, mould or other microbial contaminant growth and/or other moisture related deterioration, such as corrosion, decay, rot in buildings or their surroundings. Any statements in this report or on the boring logs regarding odours, colours, and unusual or suspicious items or conditions are strictly for informational purposes.

17 **Sample Disposal:** WSP will dispose of all uncontaminated soil and rock samples after 30 days following the release of the final geotechnical report. Should the Client request that the samples be retained for a longer time, the Client will be billed for such storage at an agreed upon rate. Contaminated samples of soil, rock or groundwater are the property of the Client, and the Client will be responsible for the proper disposal of these samples, unless previously arranged for with WSP or a third party.