APPENDIX 'A'

GEOTECHNICAL REPORT

September 7, 2017 File No. 0022 033 00

Dr. Mike Lau, P.Eng.
Dillon Consulting Ltd.
1558 Willson Place
Winnipeg, Manitoba
R3T 0Y4

RE Fermor Avenue over the Seine River Bridge Rehabilitation
Detailed Design Geotechnical Recommendations – REV1

This letter report provides geotechnical recommendations for the detailed design of bridge rehabilitation works and retaining walls for the Fermor Avenue crossing of the Seine River in Winnipeg, Manitoba. TREK Geotechnical Inc. (TREK) was retained by Dillon Consulting to provide geotechnical engineering services for Detailed Design for the project. The terms of reference for this assignment are included in our proposal to Dillon Consulting Ltd. (Dillon), dated March 19, 2017. The scope of work includes a stability analysis of the final design geometry, lateral earth pressure distributions for sheet pile walls, and detailed design and construction recommendations to supplement TREK's preliminary design report dated September 14, 2017.

The current letter has been revised (1st revision) from previous a version (dated Septemebr 7, 2017) for the following reasons:

- updated girder jacking loads provided by Dillon
- corrected location of the jacking load relative to the bridge abutments
- additional analysis case eliminating a wall at Sta. 6+900 (north side slope)
- provision of frost insulation recommendations

Background and Existing Information

The existing Fermor Avenue bridge over the Seine River is a three-span mixed steel and precast concrete girder structure with cast-in-place concrete (CIPC) deck, barriers and sidewalks. The bridge was originally constructed in 1953, widened in 1969, and rehabilitated in 1984. Recent inspections of the bridge have identified that it is in fair to poor condition and requires rehabilitation or reconstruction to meet current design standards and to accommodate a new Active Transportation Pathway (ATP).

TREK undertook a sub-surface investigation for the preliminary design to supplement existing information from previous bridge construction. Details of the sub-surface investigation are summarized in TREK's Preliminary Design Report, and results of a roadway subsurface investigation performed by TREK (as part of the overall project) along Fermor Avenue between St. Anne's Road and Archibald Street was summarized in a separate report (appended to our Preliminary Design report).

TREK's Preliminary Design Report includes recommendations relative to embankments, foundations, excavations and shoring. All recommendations from our previous preliminary design report remain applicable except for those noted herein.

Proposed Works

Our understanding of the proposed works are based on drawings provided by Dillon (Appendix A). The bridge over the Seine river will be rehabilitated, and the existing foundation and abutments will remain with minor modifications to accommodate the rehabilitation. An underpass structure crossing Fermor Ave. will provide a north-south active transportation pathway (ATP) connection approximately 230 m west of the existing bridge. An east-west ATP connection over the Seine River will be accommodated by an ATP running along and up the north sideslope of Fermor Avenue, crossing the Seine River along the north edge of the rehabilitated bridge. A more detailed description of the major components of work is provided below.

Bridge Rehabilitation

The proposed bridge works involve rehabilitating the existing abutments, piers and girders, and reconstructing the bridge deck and approach slabs. The eastbound sidewalk will be removed, and traffic lanes will be shifted to the south to accommodate the new ATP north of the westbound lanes.

The rehabilitation includes jacking (lifting) the bridge girders approximately 150 mm at the abutments to allow for bearing replacement and concrete repairs. Jacking will be undertaken in two stages: the northern half of the bridge will be rehabilitated in the summer of 2018, and the southern half in the summer of 2019. The proposed works do not include modifying the existing river channel beneath the bridge structure, although recommendations for scour protection will be provided by TREK in a separate report. Headslopes will be maintained at the west abutment, and will be flattened to between 5 horizontal to 1 vertical (5H:1V) and 6H:1V at the east abutment.

Sideslopes

The north sideslope will be regraded to accommodate the ATP, and where necessary to achieve target factors of safety. A portion of the north sideslope within the project area has experienced historical slope instabilities and will be repaired as part of this work. The south sideslope will be stabilized to achieve target factors of safety with a combination of regrading and cantilevered (sheet pile) retaining walls.

Active Transportation Path

An underpass will be constructed beneath Fermor Avenue to connect ATPs to the south and north of Fermor Ave. The concrete box underpass will be founded on a shallow (mat) foundation bearing on stiff to very stiff clay. The underpass will be constructed in two stages, and temporary shoring (to be designed by the Contractor) will be required to accommodate the construction staging. The entrances/exits to the underpass will consist of cantilevered sheet pile headwalls.

North of Fermor Ave., the ATP will climb the north sideslope and cross the Seine River along the north edge of the bridge. A retaining wall will run along the downslope edge of the ATP, and will be constructed as either a mechanically stabilized earth (MSE) wall or a shallow sheet pile retaining wall.

Embankments

Slope Stability Analysis

The stability of the east and west head slopes, and select sections on the embankment side slopes were evaluated to assess detailed design geometry and bridge jacking surcharge loading during construction. Stability model methods, assumptions, parameters, results and recommendations are provided below.

Model Development

The slope stability analysis was conducted using a 2-dimensional limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2016 software package (Geo-Slope International Inc.). The slope stability model used the Morgenstern-Price method of slices with a half-sine inter-slice force function to calculate factors of safety (FS) along potential slip surfaces. A minimum factor of safety (FS) of 1.30 was selected based on the assumed groundwater levels (which are higher than measured levels). Groundwater conditions were represented in the model using a static piezometric line.

Headslopes

The headslope geometry considered in the analysis was provided by Dillon. The river channel beneath the bridge is skewed to the bridge alignment, so the headslope length varies across the width of the bridge. Therefore, three distinct cross-sections were analyzed: the north edge of the bridge, the bridge centerline, and the south edge of the bridge. The headslope analysis considered the proposed final geometry, and the effect of superstructure jacking loads.

Jacking loads and locations were provided by Dillon, and were applied in the model as surcharge loads on the headslopes downslope of the abutments. The loads are greatest at the north and south edges of the bridge to lift the heavier concrete girders, and are reduced in the middle of the bridge to lift the steel girders. TREK modeled both an extreme loading scenario at each abutment which incorporated the highest jacking loads distributed over the width of the concrete girders, and an average loading scenario which distributed all jacking loads over the full width of the bridge. As per discussions with Dillon, it was determined that the jacking loads would be distributed to the foundation soils using timber cribbing and 0.3 m thick by 2.4 m wide rigid timber crane mats of varying length (typically 6 m to 9 m). Table 1 summarizes the range of jacking forces for either concrete or steel girders, and the extreme and average jacking surcharge pressures considered in the analysis. The granular backfill upslope of the abutment was neglected in the analysis, since the abutment will be exposed in an open excavation during the superstructure jacking.

Table 1 – Jacking Forces and Modeled Surcharge Loads

Case	No. of Girders and applicable width	Jacking Force per Girder (kN)	Jacking Pressure (kPa)
Extreme	3 concrete girders over 5 m (at both north and south edges of bridge)	301 to 393 kN	77 kPa
Average	3 concrete girders + 7 steel girders over 25.3 m (full bridge width)	185 to 393 kN	57 kPa

The extreme (upper bound) loading scenario was modeled at the headslope section at which the river was closest to the abutment to assess a lower bound factor of safety for each headslope; at a cross section coincident with the south edge of the east abutment, and the north edge of the west abutment. The average loading scenario was applied to the centerline cross-section.

Sideslopes

The cross-sections used in the sideslope stability models were generated from a combination of ground survey provided by Dillon, and bathymetric survey conducted along the Seine River by Bruce Harding Consulting Ltd. in 2016. The cross-sections reference stationing provided by Dillon, which is shown on Figure 1. The south sideslope cross-sections reference the eastbound Fermor Ave. control line stationing, and the north sideslope cross-sections reference the ATP control line stationing. All cross-sections were cut perpendicular to the sideslopes.

The existing failure along the north sideslope extends from approximately station 6+905 to 6+960. A cross-section at station 6+940 was back-analyzed to assess the observed instability, and a cross-section just outside the failure (Sta. 6+964) was used to estimate the pre-failure slope geometry used in the back-analysis. The observed instability was likely triggered by near-surface saturation and loss of soil suction, which was simulated using a piezometric line at ground surface (i.e. fully saturated) within and downslope of the instability. It should be noted that the fully saturated piezometric line is considered applicable only to shallow slip surfaces and is not considered appropriate for deep-seated global slip surfaces. Residual clay parameters within the failed soil mass were determined based on the back-analysis of pre-failure slope geometry and a slope repair detail (granular ribs) was designed. Additional cross-sections at stations 6+990 and 6+900 were analyzed to assess stability with the ATP at the top of the slope and at mid-slope, respectively, and determine if any stabilization works or deep sheet pile walls are required.

On the south sideslope, the critical cross-section at station 4+850 was analyzed where the Seine River is located in closest proximity to Fermor Avenue. A deep sheet pile wall combined with slope regrading was considered in the analysis for slope stabilization. A cross-section at station 4+910 was modeled to determine the extents of sheet piles required for global stability.

The soil stratigraphy and material parameters assumed in the model is consistent with TREK's Preliminary Design Report, as summarized in Table 2.

Table 2. Material Parameters used in Slope Stability Analysis

Material	Unit Weight (kN/m3)	Cohesion (kPa)	Friction Angle (degrees)
Clay Fill	18	3	20°
Silty Clay	17.5	5	15°
Residual Clay	17.5	2.5	12
Till	20	5	27°
Granular Fill	21	0	35
Lightweight Fill	4.5	500	0
Granular Ribs (50% replacement ratio)	19.25	1.25	28.5

A traffic surcharge load of 2 kPa was incorporated into the embankment side slope analysis.

Piezometric Conditions

Piezometric conditions considered in the analysis are consistent with our Preliminary Design Report, except for the fully saturated level used for back analysis of the existing side slope instability. The piezometric level was modeled at Elev. 227.0 m sloping down to meet the river level at Elev. 225.3 m. The sideslope repair back analysis was modeled with a fully saturated piezometric line downslope of and within the observed instability.

Analysis Results

Results of the stability analysis are summarized in Tables 3 and 4 for head and side slopes, respectively, with stability results shown in the attached figures.

Headslopes

Model outputs for the proposed (final) headslope geometries at the north edge, centerline and south edge of the bridge are shown on Figures 2 to 8. All proposed headslopes met the target factors of safety with granular backfill except the west abutment at the north edge of the bridge, which had a factor of safety of 1.22. Using lightweight fill as backfill behind the abutment (Figure 3) raised the safety factor at this section to 1.31. The factors of safety for all headslope analyses are summarized in Table 3.

The superstructure jacking analyses are shown on Figures 9 to 12. A bench cut (offloading) down to Elev. 229.25 m at the jacking location is required to achieve an adequate factor of safety. In the average loading scenario, all slip surfaces exceeded the target factor of safety. In the extreme loading scenario, two potential slip surfaces on the west headslope did not meet the stability target; slip surfaces originating upslope of the jacking pad and behind the abutment have safety factors of 1.19 and 1.28, respectively (Figure 10). However, because the load for the extreme case is applied over a narrow width (approximately 5 m of timber mat) in comparison with the length of the slip surfaces (between 18 and 36 m long), we anticipate a loading scenario tending toward the average value would be more appropriate. The extreme loading scenario does, however, serve as a check to show that the factor of safety remains above unity for worst-case loading conditions.

Table 3. Summary of Calculated Factors of Safety for Headslopes

Stability Case	Loading Scenario	Headslope	Cross-Section	Slip Surface	Factor of Safety	Figure No.
Final Geometry	N/A	West	North	Critical	1.31**	3
			Centerline	Critical	1.33	4
			South	Critical	1.44	5
		East	North	Critical	1.45	6
			Centerline	Critical	1.38	7
			South	Critical	1.33	8
Superstructure Jacking	Extreme	East	South	Downslope edge of mat	1.55	9
				Upslope edge of mat	1.32	
				Global	1.37	
		West	North	Downslope edge of mat	1.52	10
				Upslope edge of mat	1.19	
				Global	1.28	
	Average	West	Centerline	Downslope edge of mat	1.90	11
				Upslope edge of mat	1.48	
				Global	1.38	
		East	Centerline	Downslope edge of mat	1.63	12
				Upslope edge of mat	1.37	
				Global	1.41	

Meets target factor of safety

Does not meet target factor of safety

** Lightweight fill used to backfill behind abutment

Sideslopes

Sideslope stability analysis cases and associated factors of safety are summarized in Table 4.

The stability analysis showed deep sheet pile retaining walls are not required to achieve the target global factor of safety along the north side slope. Adequate levels of stability can be achieved at station 6+990 through a combination of regrading and a 1.8 m high retaining wall (either MSE or shallow sheet piles) downslope of the ATP (Figure 13). The factor of safety at the retaining wall shown in the model is 1.24 without any strength offered by cantilevered wall or MSE reinforcements. The cross-section at station 6+900 is stable with a 3H:1V slope downslope of the ATP to intersect existing grade (Figure 14).

The existing instability on the north bank was back-analyzed to determine the conditions at failure, and then a slope repair analysis was undertaken. Figure 15 shows the back-analysis of pre-failure conditions using a hybrid section of station 6+940 (downslope of the instability) and 6+964 (within the embankment side slope). The slip surface shown was selected to match the failed geometry, which is shown in red. The post-failure geometry was then modeled using residual strength parameters of c'= 2.5 kPa and ϕ'=12° to achieve a factor of safety of unity, as shown in Figure 16. To repair the instability, granular ribs extending below the anticipated depth of the instability were modeled using a 50% replacement ratio and the slope was re-graded to a 4H:1V slope. The slope repair was analyzed for two groundwater levels: a surficial groundwater level as used in the back-analysis (Figure 17) and applicable to shallow slip surfaces, and the global piezometric level for deep-seated slip surfaces (Figure 18). In both cases, the factor of safety for the proposed repairs meets the design target.

At the critical section on the south sideslope (station 4+850), sheet piles embedded to Elev. 217 m (approximately 15.5 m below the edge of roadway) achieves the target factor of safety of 1.3 (Figure 19), provided the slope is re-graded to 4.5H:1V downslope of the wall. The minimum sheet pile embedment is required to push the slip surface deeper and improve the factor of safety for global slip surfaces. At station 4+910, the target factor of safety can be achieved without deep sheet piles, however re-grading is required along with a 1.6 m high retaining wall (Figure 20). We understand the portion of wall east of Sta. 4+910 will be designed as a moment slab on top of a sheet pile wall.

Table 4. Summary of Calculated Factors of Safety for Sideslopes

Cross-Section and Analysis Case	Sideslope	Groundwater Level	Slip Surface	Factor of Safety	Figure No.
6+990 5H:1V Regrading, 1.8 m Retaining Wall	North	227.0 m	Critical	1.29	13
6+900 Existing slope, mid-bank pathway, 3H:1V fill downslope	North	227.0 m	Critical	1.52	14
6+940 Observed Instability Back Analysis	North	Fully Saturated	Observed	1.14	15
		Fully Saturated	Observed	1.02 **	16
6+940 Granular Ribs, 4H:1V Regrading, No Wall	North	Fully Saturated	Critical	1.29	17
		227.0 m	Critical	1.54	18
4+850 4.5H:1V Regrading 2.0 m Wall with 0.8 m Moment Slab Minimum Sheet Pile Tip Elev. 217 m	South	227.0 m	Global	1.30	19
			Downslope of Sheet Piles	1.34	
4+910 5H:1V Regrading, 1.6 m wall No embedment requirement for global stability		227.0 m	Critical	1.29	20

Meets target factor of safety

Does not meet target factor of safety

** Lightweight fill used to backfill behind abutment

Stability Recommendations

Based on the slope stability analysis, TREK recommends the following:

1. The north half of the west abutment be backfilled with lightweight fill. TREK should be consulted to confirm that the selected lightweight fill geometry is consistent with the assumptions of our analysis.

2. The headslope should be bench cut such that the load transfer platform for the girder jacks is founded at elevation 229.25 m. The load transfer platform should be at least 2.4 m wide (in the east-west direction) as shown in the model outputs, and the jacking loads should not exceed those provided for this analysis.

3. Sheet piles are not required on the north sideslope to achieve adequate global stability. Where walls are required, sheet piles may be used and designed based on the lateral earth pressure recommendations provided herein.

4. External stability of retaining walls on the north sideslope should be checked. Typically, adequate external stability can be achieved using a minimum reinforcement length for MSE

walls of 80% of the exposed wall height, or using a minimum embedment depth for shallow sheet piles of two to three times the exposed wall height. Internal wall stability should also be evaluated (to be completed by others).

5. Granular ribs should be installed to stabilize the north side slope instability with a 50% replacement ratio in plan view from station 6+905 to 6+960, and based on the rib geometry in cross-section shown on Figure 17. The slope should be regraded to a slope of 4H:1V prior to the construction of the ATP along the slope. Granular ribs should be constructed using 100 to 150 mm down crushed limestone rockfill topped with a 0.6 m thick clay cap to prevent infiltration. The rockfill should be placed in lifts and compacted using vibratory techniques (e.g. hoe-pack) to achieve a maximum apparent density. The lift thickness and compaction duration should be determined by a field trial at the time of construction.

6. At locations along the north sideslope west of station 6+960 where the ATP is above existing grade, fill may be placed downslope of the ATP at a slope of 3H:1V (as shown in Figure 14). A shallow retaining wall may also be utilized.

7. Vegetation should be established on any regraded slopes to prevent saturation of the near surface soils.

8. Sheet piles are required for global slope stability on the south sideslope from station 4+850 to 4+910. The sheet piles should have a minimum tip elevation of 217 m, and the ground should be regraded such that a 4.5H:1V slope angle and a 2.8 m cantilever is achieved. The sheet piles should be designed using the lateral earth pressure recommendations noted herein.

Settlement

Upon review of the proposed final grade, minimal fill is planned and therefore settlement is not considered to be a concern for the proposed works.

Permanent Cantilevered Walls

Permanent cantilevered sheet pile retaining walls are proposed along the south sideslope of the west approach embankment from station 4+850 to 4+910, and at either end of the pedestrian-cyclist underpass. Table 5 provides the recommended earth pressure coefficients and bulk unit weights of each soil layer for calculation of lateral earth pressures. Surcharge loads and hydrostatic water pressure below the groundwater table should be incorporated into the design of cantilevered walls, as well as an adequate factor of safety against instability. Figure 21 shows the recommended earth pressure diagram for preliminary design of the sheet pile wall. The surcharge pressure should be selected by the structural engineer for any sustained loads.

An active earth pressure coefficient (K_a) should be used to calculate lateral loads against cantilevered walls which are free to translate horizontally away from the retained soil by more than 0.2% of the wall height. A passive earth pressure coefficient (K_p) should be used if the wall is free to translate horizontally towards the retained soil by more than 2% of the wall height. An at-rest earth pressure coefficient (K_o) should be used if the walls undergo less than 2% movement of the wall height towards the retained soil and less than 0.2% of the wall height away from the retained soil. The table below provides K_a, K_p, and K_o values for calculation of lateral earth pressures acting on below grade walls.

Table 5. Recommended Design Parameters for Cantilevered Walls

Design Parameter	Granular Fill	Clay Fill	Silty Clay	Till
Active Earth Pressure Coefficient (K_a)	0.2	0.5	0.6	0.4
Passive Earth Pressure Coefficient (K_p)	4.6	2.0	1.7	2.7
At-Rest Earth Pressure Coefficient (K_o)	0.4	0.7	0.7	0.5
Bulk Unit Weight, Y (kN/m^3)	21.0	18.0	17.5	20.0
Effective Unit Weight, Y' (kN/m^3)	11.2	8.2	7.7	10.2

Underpass Mat Foundation

Our previous foundation recommendations, provided in our PD report, should be supplemented with the following design and construction recommendations:

1. A minimum thickness of 0.3 m of granular base course should be used to limit frost penetration (150 mm of sub-base and 150 mm of base material). Base layers should be compacted to 98% SPMDD.
2. A minimum thickness of 125 mm of rigid polystyrene insulation should be placed on top of the base course and beneath the concrete slab. The insulation should extend a minimum distance of 2.4 m beyond the edges of the slab and should be buried a minimum of 0.3 m below final grade.

Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation, laboratory testing, geometries). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

If you have any questions regarding the findings or recommendations presented, please contact the undersigned at your earliest convenience.

Kind Regards,

TREK Geotechnical
Per:

PROVINCE OF MANITOBA
2017/09/22
M. J. VAN HELDEN
Member
24729
REGISTERED PROFESSIONAL ENGINEER

Michael Van Helden, Ph.D., P.Eng.
Geotechnical Engineer

Reviewed By:

for **Ken Skaftfeld, M.Sc., P.Eng.**
Senior Geotechnical Engineer

APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: Sep 22/17

PLAN OF PROPOSED RETAINING WALLS
1:500

NOTE:
ADDITIONAL CROSS SECTIONS PROVIDED EVERY 10m ALONG THE EASTBOUND (EB) FERMOR AND ATP ALIGNMENTS, SEE REF-CS-007 TO REF-CS-010.

Figure 1

DILLON CONSULTING	TITLE	DRAWING NO.
SCALE: AS NOTED	RETAINING WALLS LAYOUT PLAN	REF-CS-002

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Fermor Avenue over Seine River Bridge

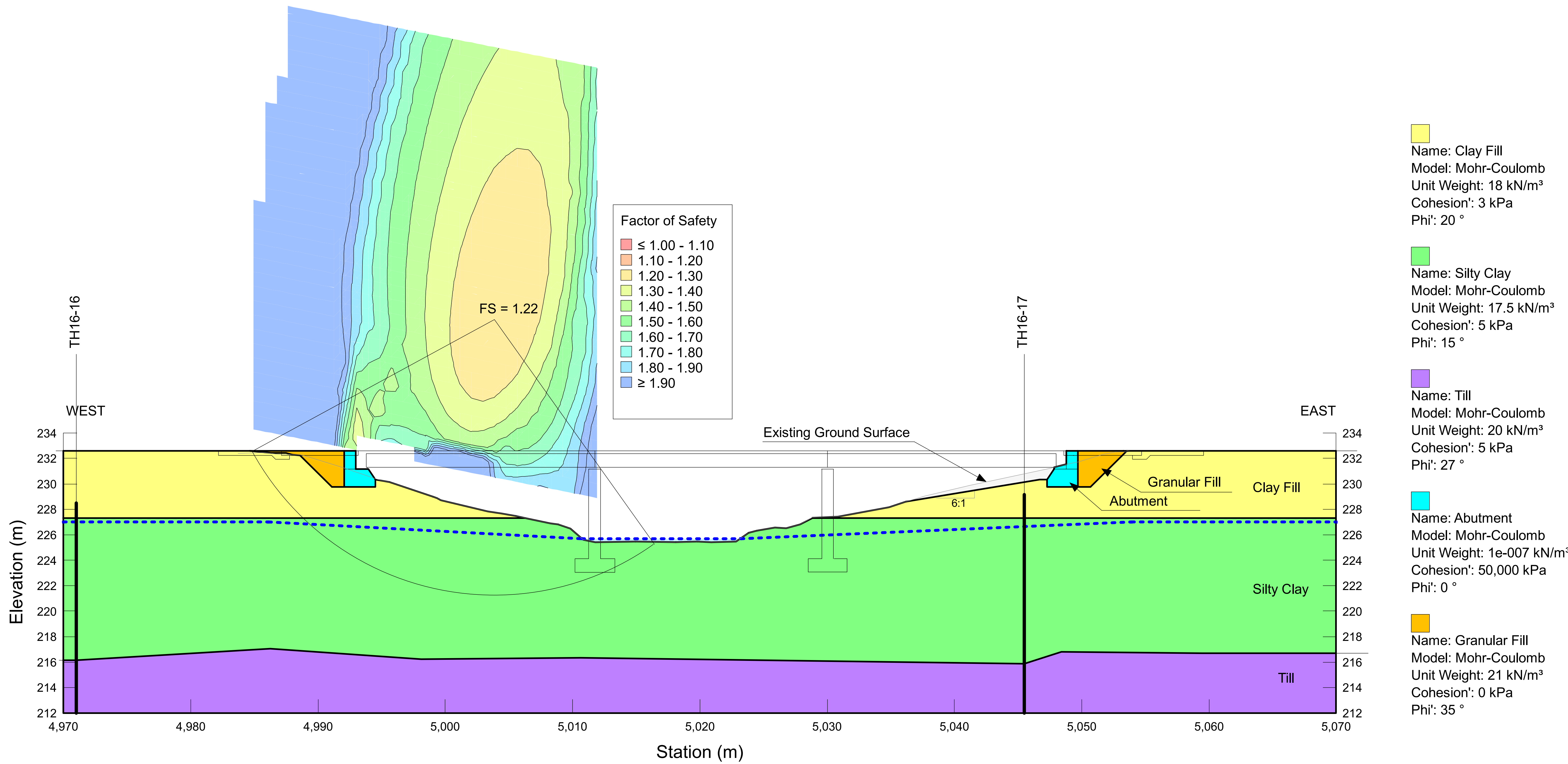

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Figure 2
Proposed Geometry
North Edge West Headslope

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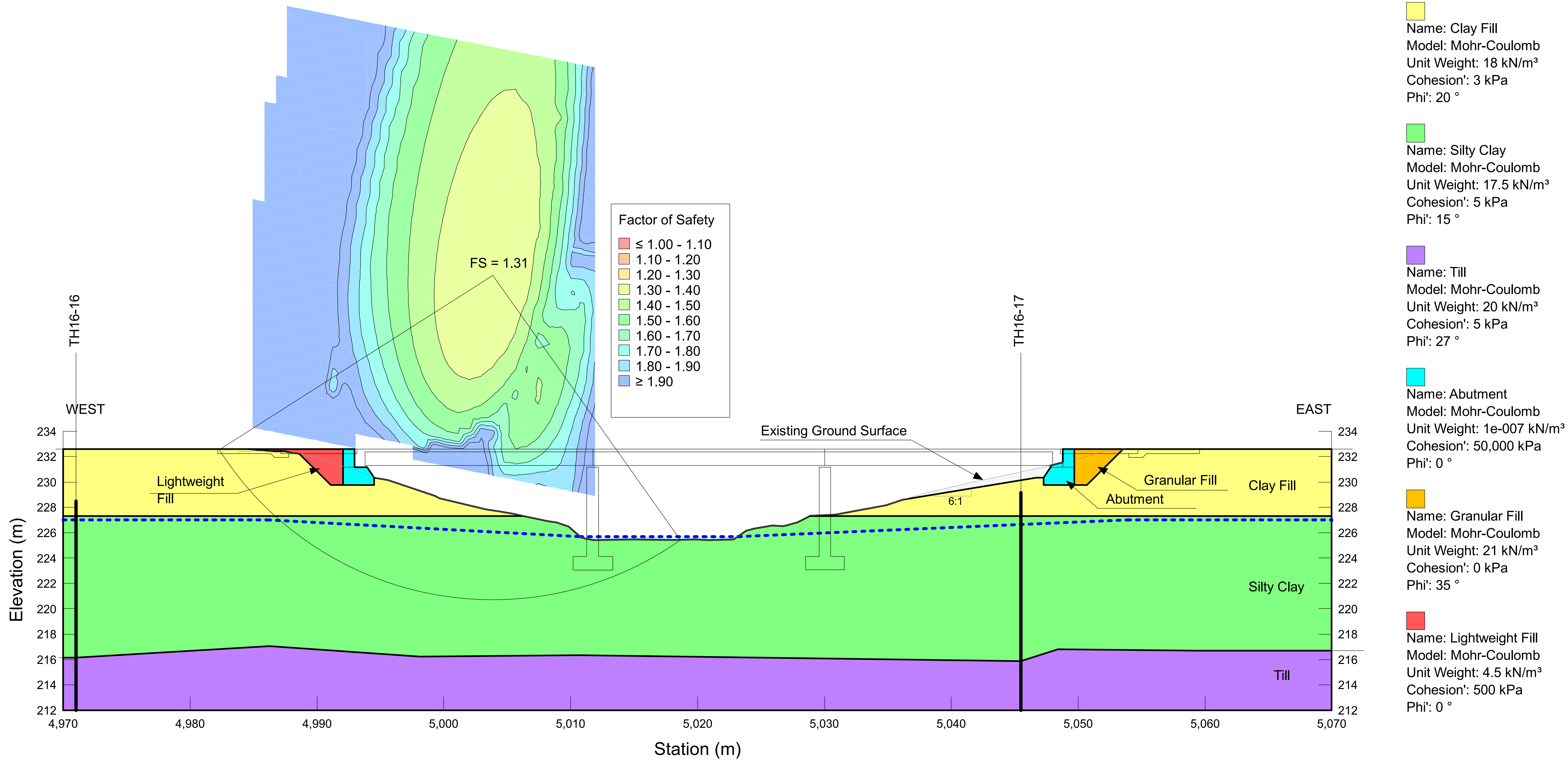

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Figure 3
Lightweight Fill
North Edge West Headslope

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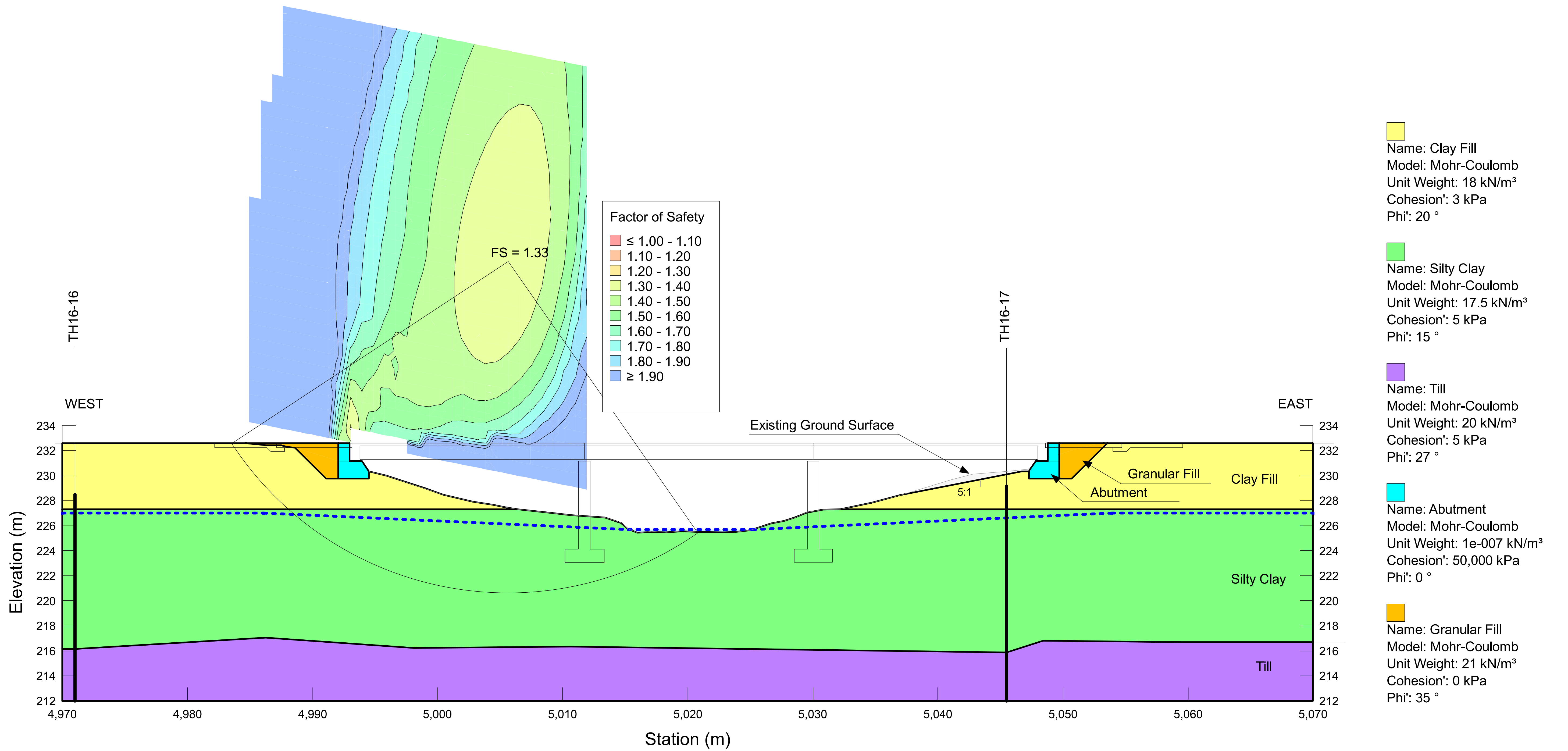

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Figure 4
Proposed Geometry
Centerline West Headslope

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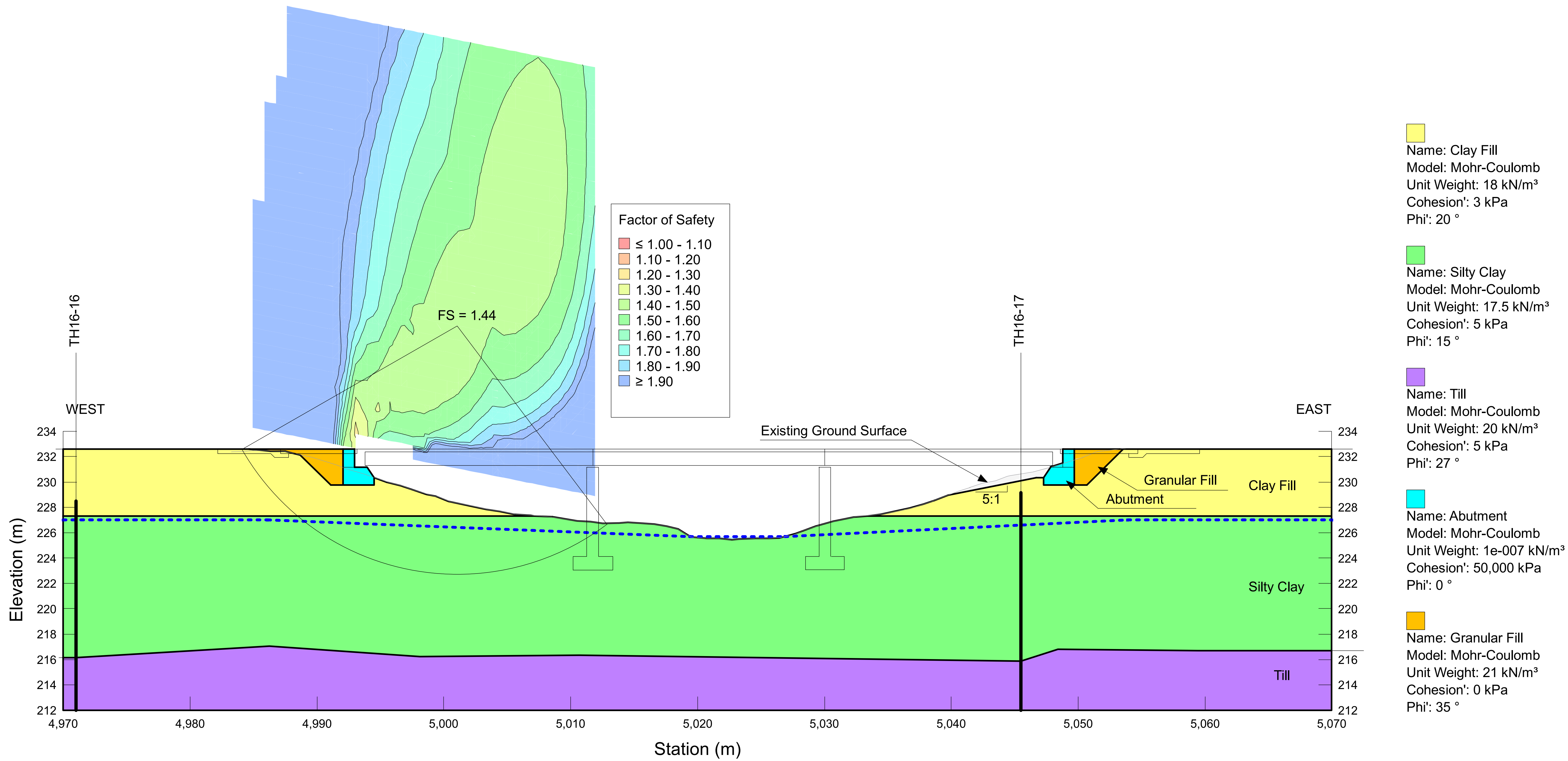

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Figure 5
Proposed Geometry
South Edge West Headslope

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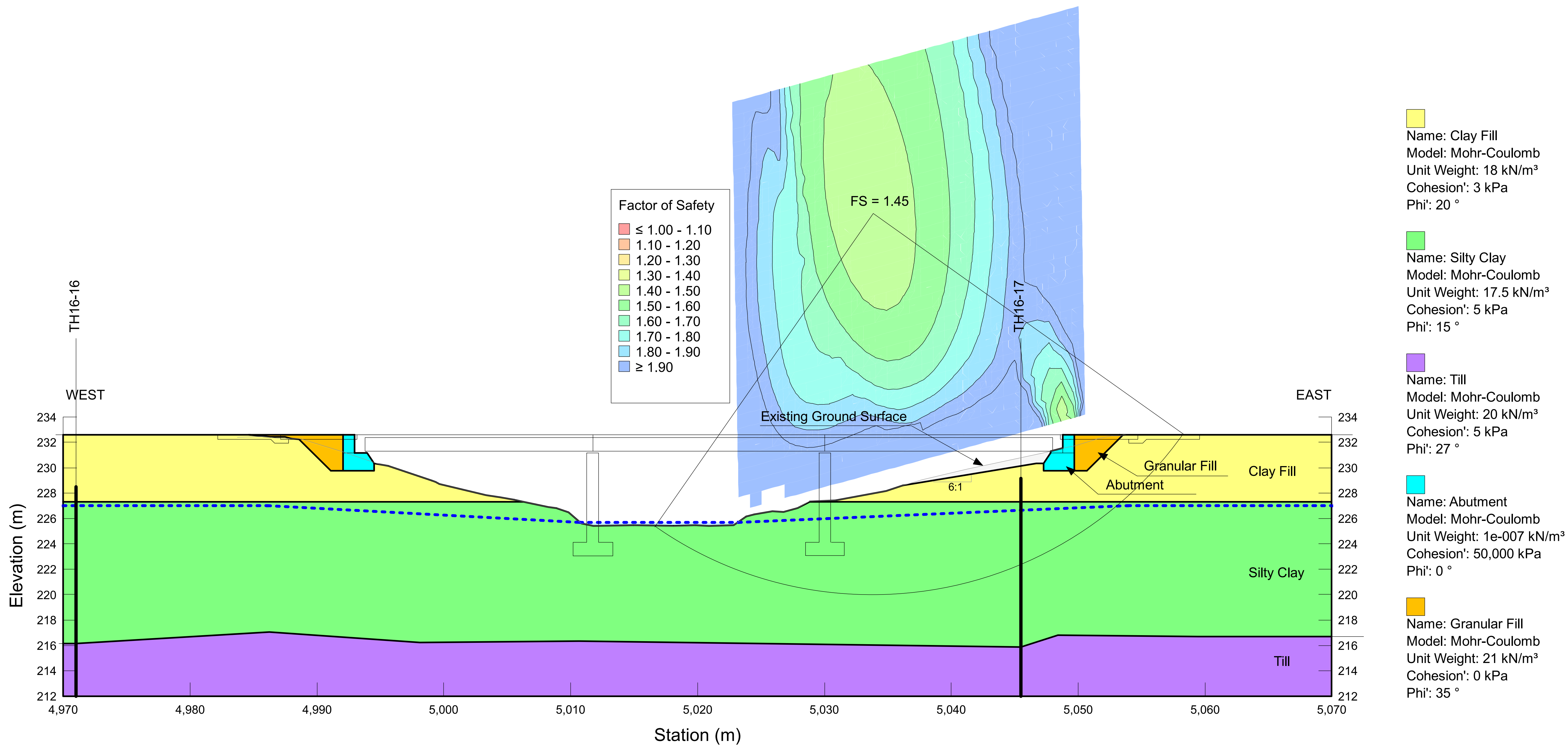

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Figure 6
Proposed Geometry
North Edge East Headslope

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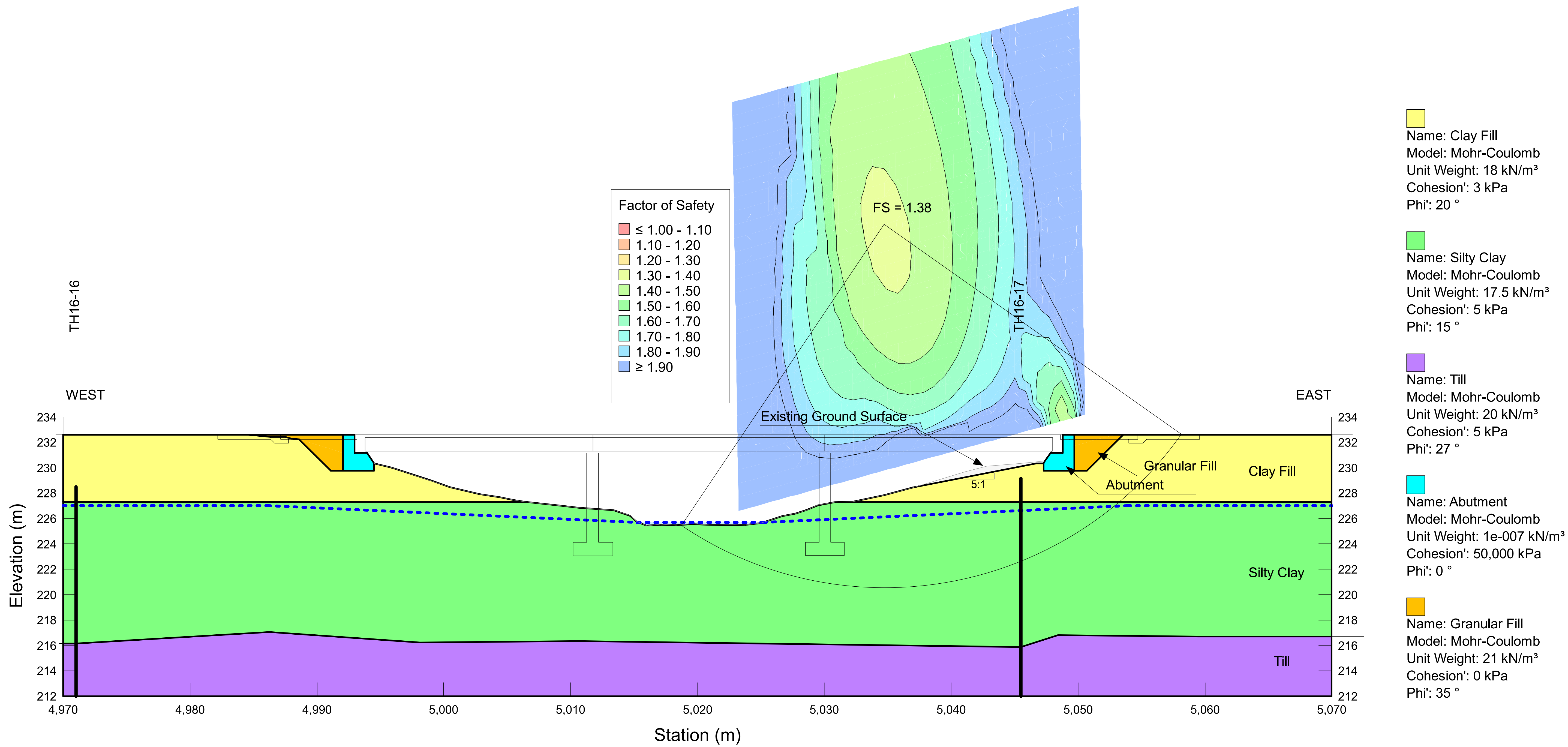

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Figure 7
Proposed Geometry
Centerline East Headslope

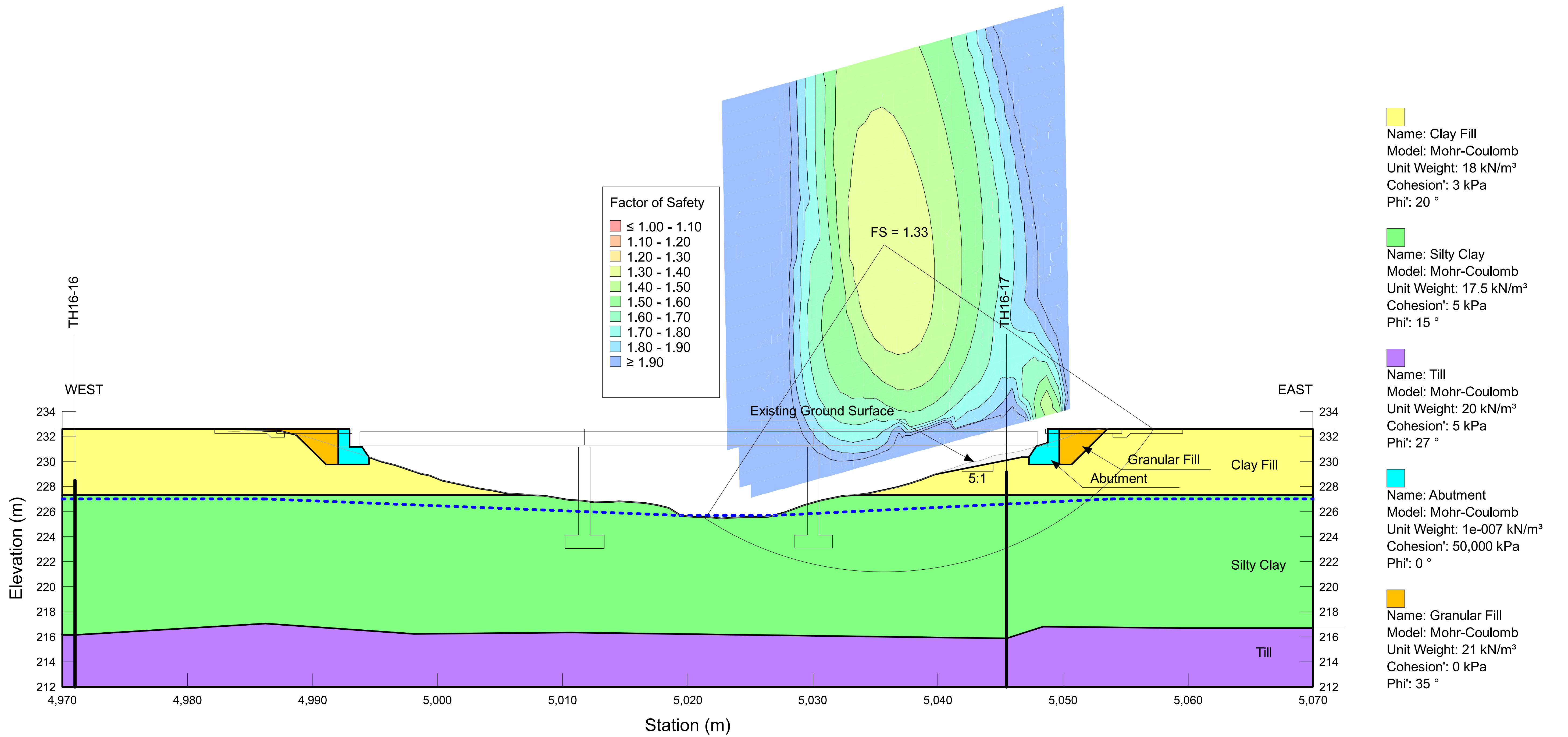

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Figure 8
Proposed Geometry
South Edge East Headslope

Jacking Pressure: 46.9 kPa
2.4 m wide jacking mat at 229.25 m

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar
	Clay Fill	Mohr-Coulomb	18	3	20	0
	Silty Clay	Mohr-Coulomb	17.5	5	15	0
	Till	Mohr-Coulomb	20	5	27	0

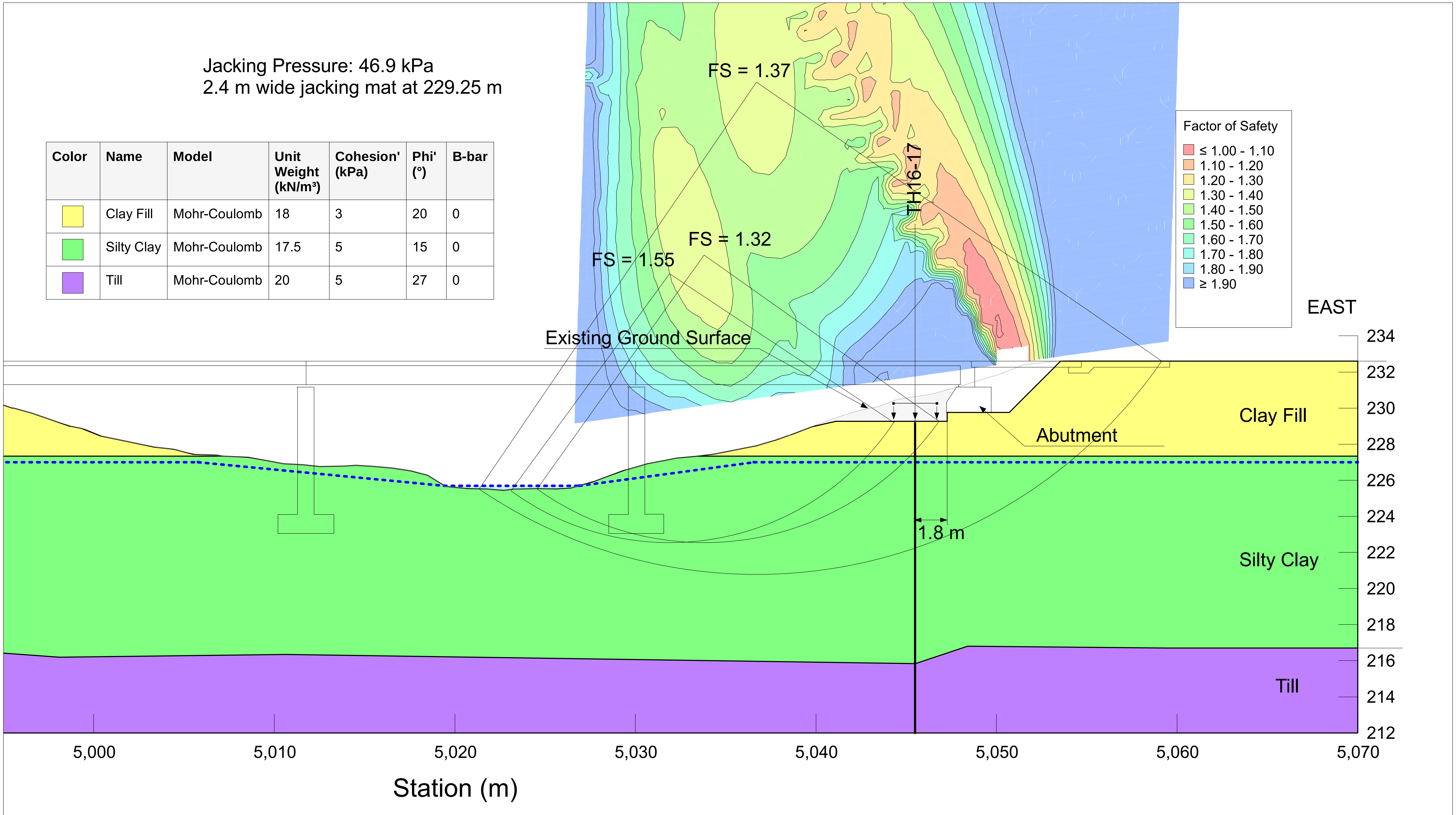

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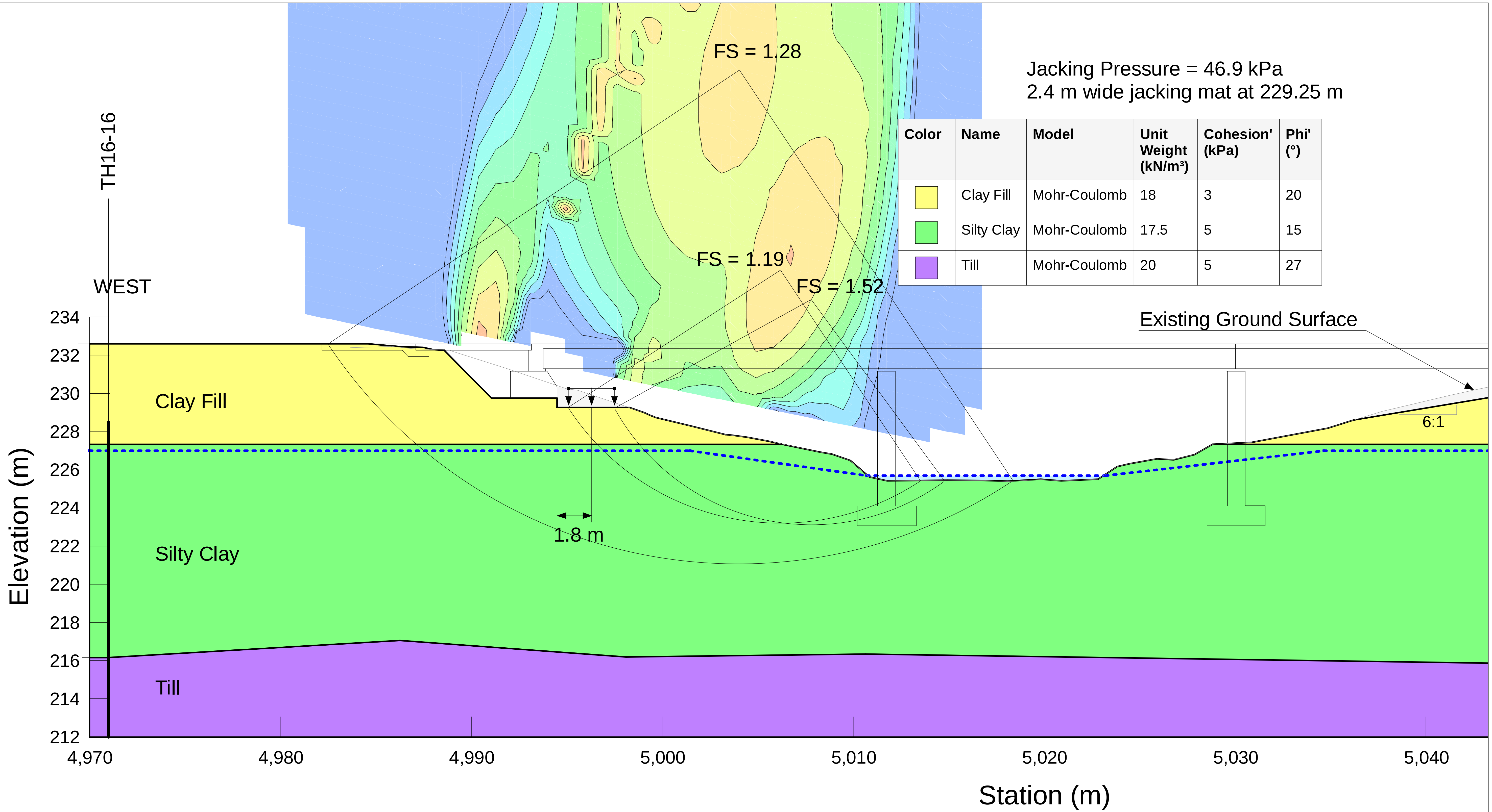
Figure 9
Jacking Loads (Extreme)
South Edge of Bridge (East Headslope Worst Case)



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Clay Fill	Mohr-Coulomb	18	3	20
	Silty Clay	Mohr-Coulomb	17.5	5	15
	Till	Mohr-Coulomb	20	5	27

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Figure 10
Jacking Loads (Extreme)
North Edge of Bridge (West Headslope Worst Case)

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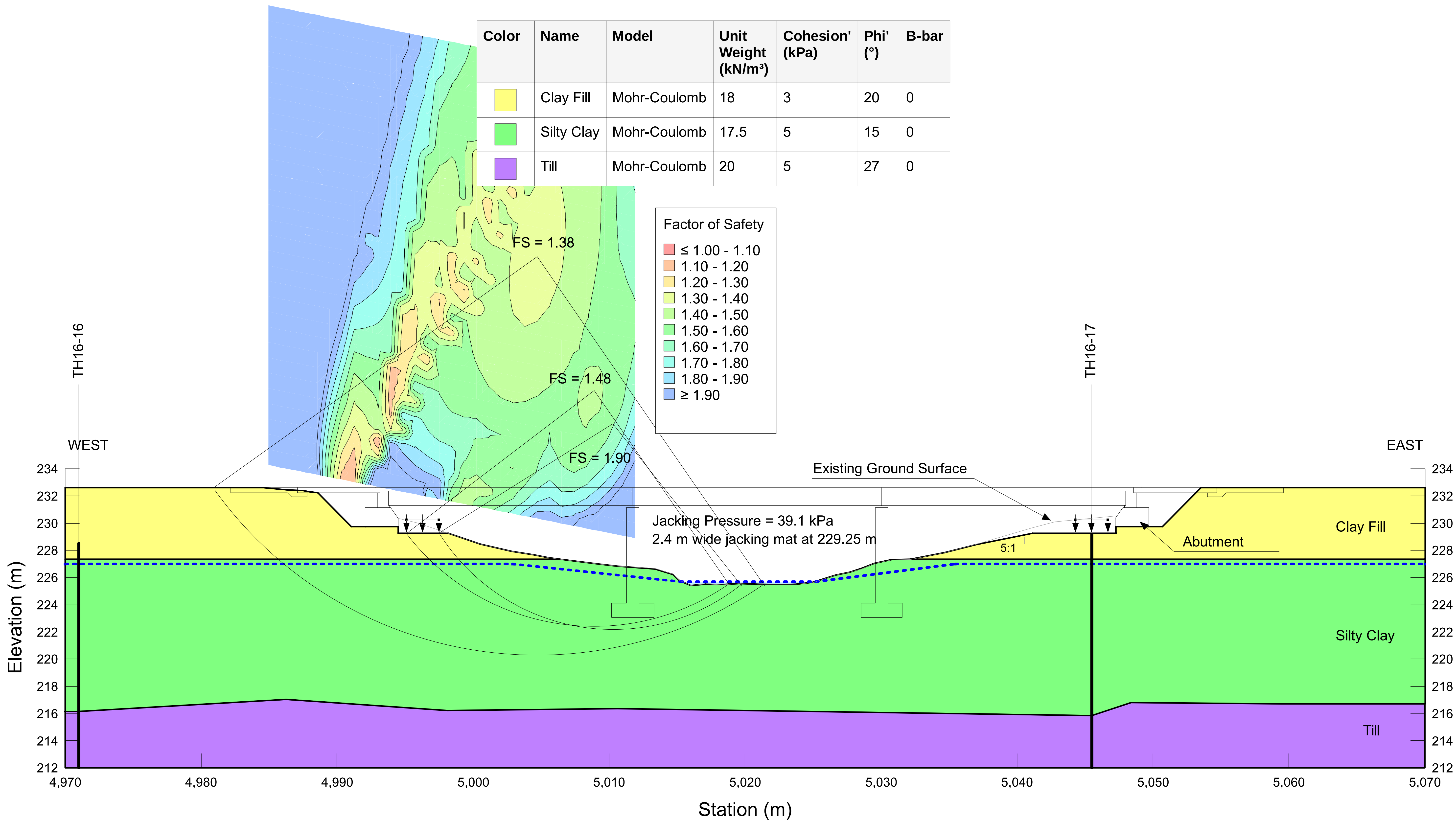
Dillon Consulting Ltd.
Fermor Avenue over Seine River Bridge

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar
	Clay Fill	Mohr-Coulomb	18	3	20	0
	Silty Clay	Mohr-Coulomb	17.5	5	15	0
	Till	Mohr-Coulomb	20	5	27	0



SCALE: 1:275 (279mm x 432mm)

Figure 11
Jacking Loads (Average)
Bridge Centerline (West Abutment)

0022 033 00
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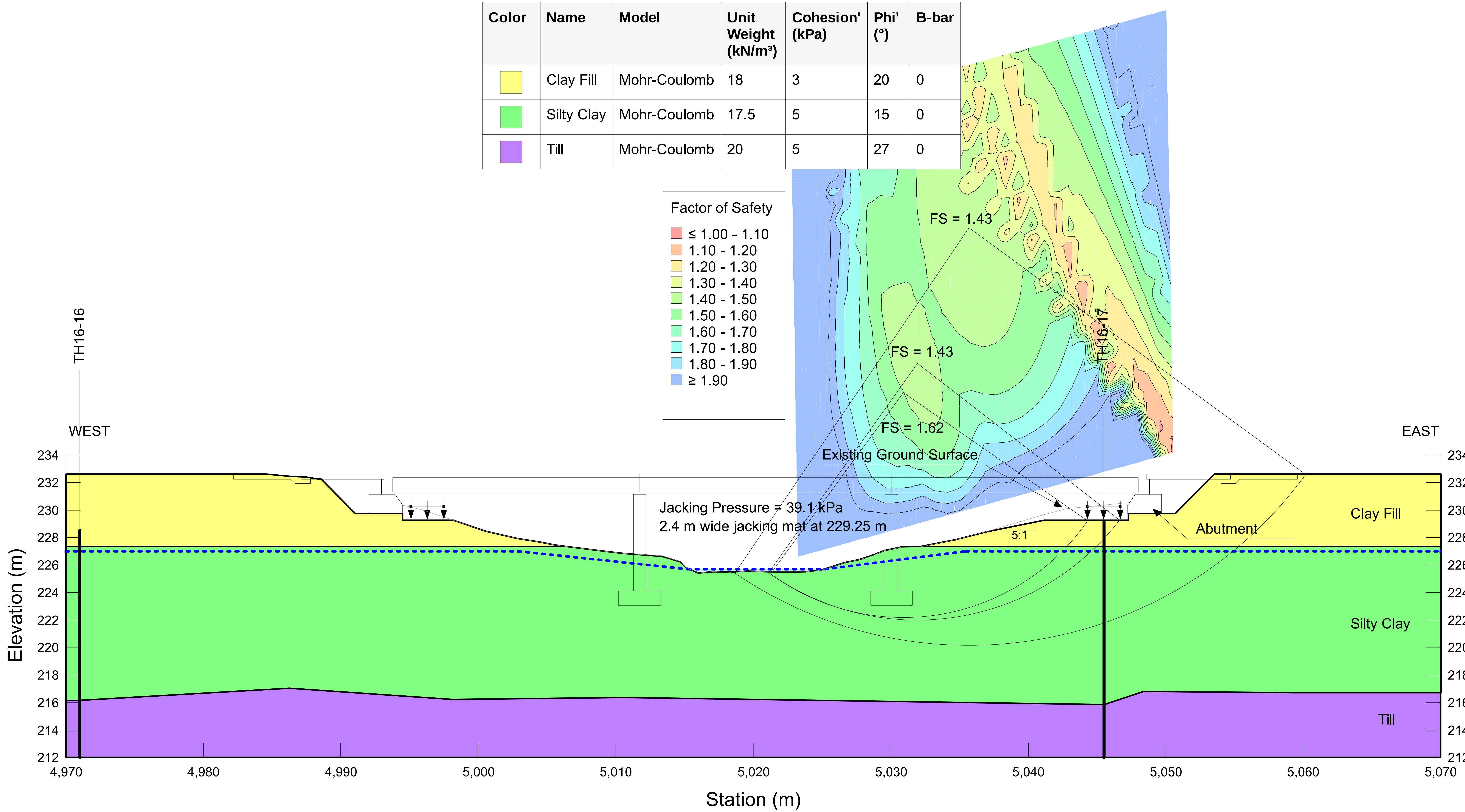
Fermor Avenue over Seine River Bridge

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar
	Clay Fill	Mohr-Coulomb	18	3	20	0
	Silty Clay	Mohr-Coulomb	17.5	5	15	0
	Till	Mohr-Coulomb	20	5	27	0



Tabloid (279mm x 432mm)
SAVED: 18/09/2017 12:27:03 PM
FILE NAME: HC003.gsz

SCALE: 1:275 (279mm x 432mm)

Figure 12
Jacking Loads (Average)
Bridge Centerline (East Abutment)

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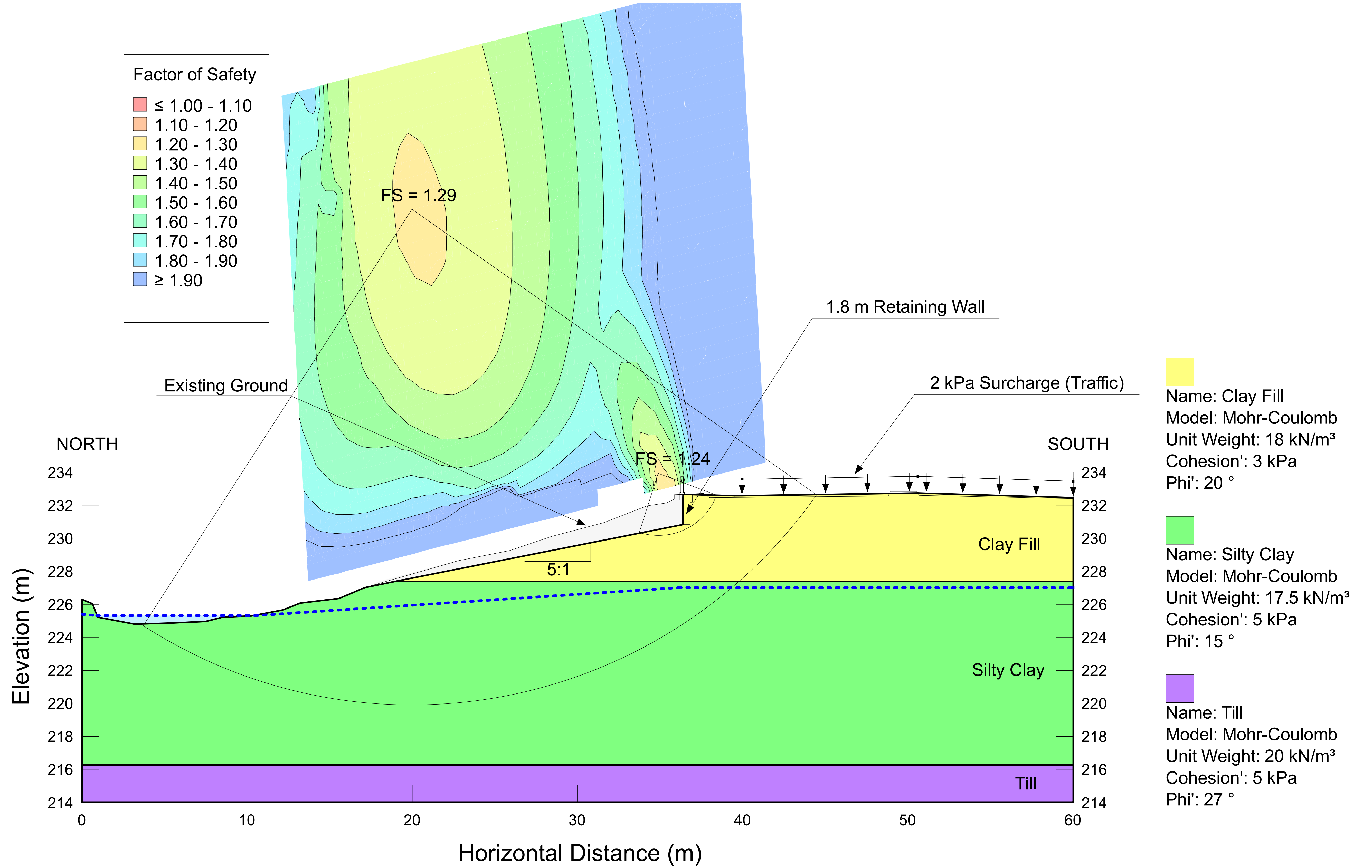

Tabloid (279mm x 432mm)
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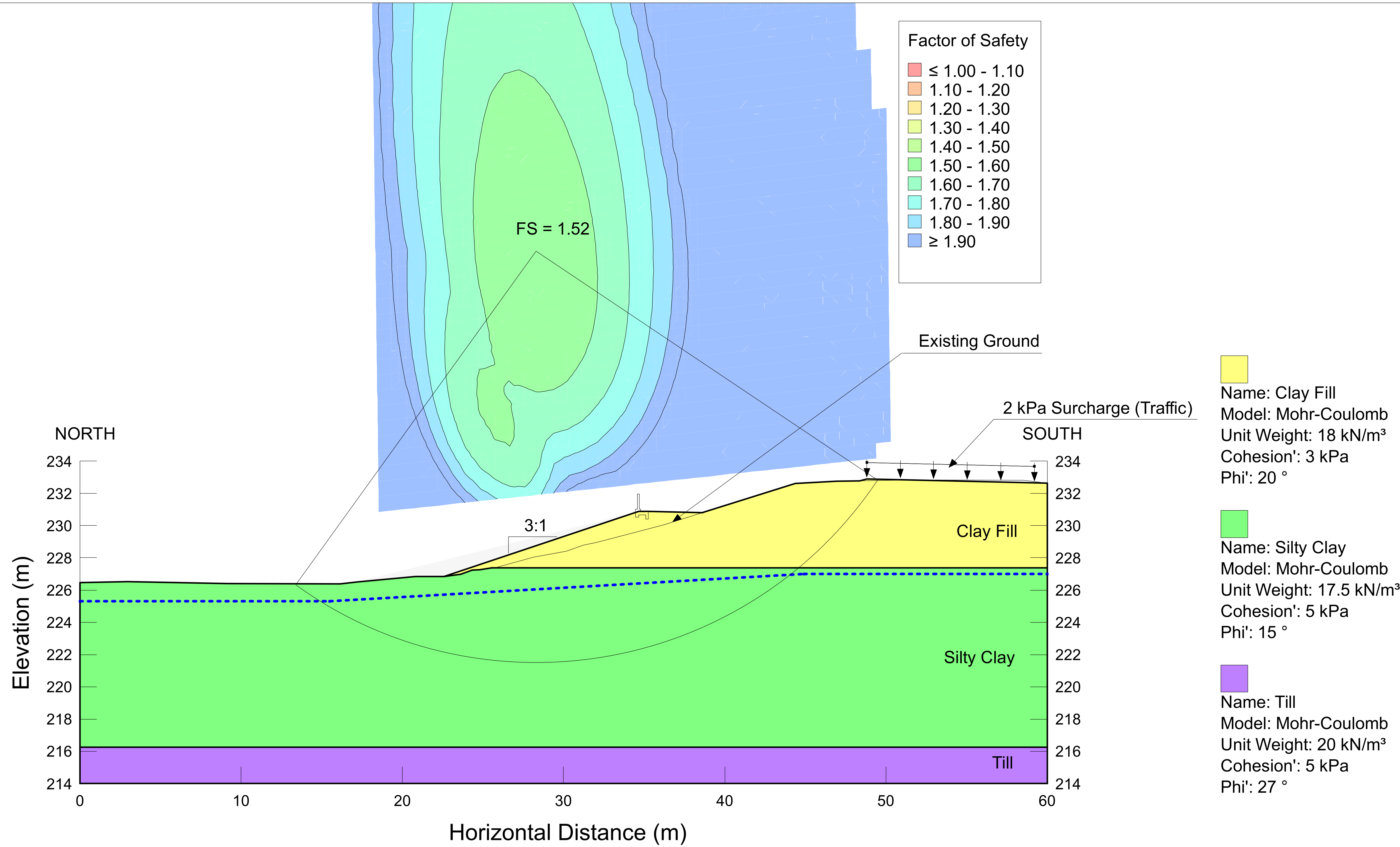
FILE NAME: 6990 - S003.gsz
SCALE: 1:235 (279mm x 432mm)

Figure 13
Sta. 6+990
Modified Geometry



Tabloid (279mm x 432mm)
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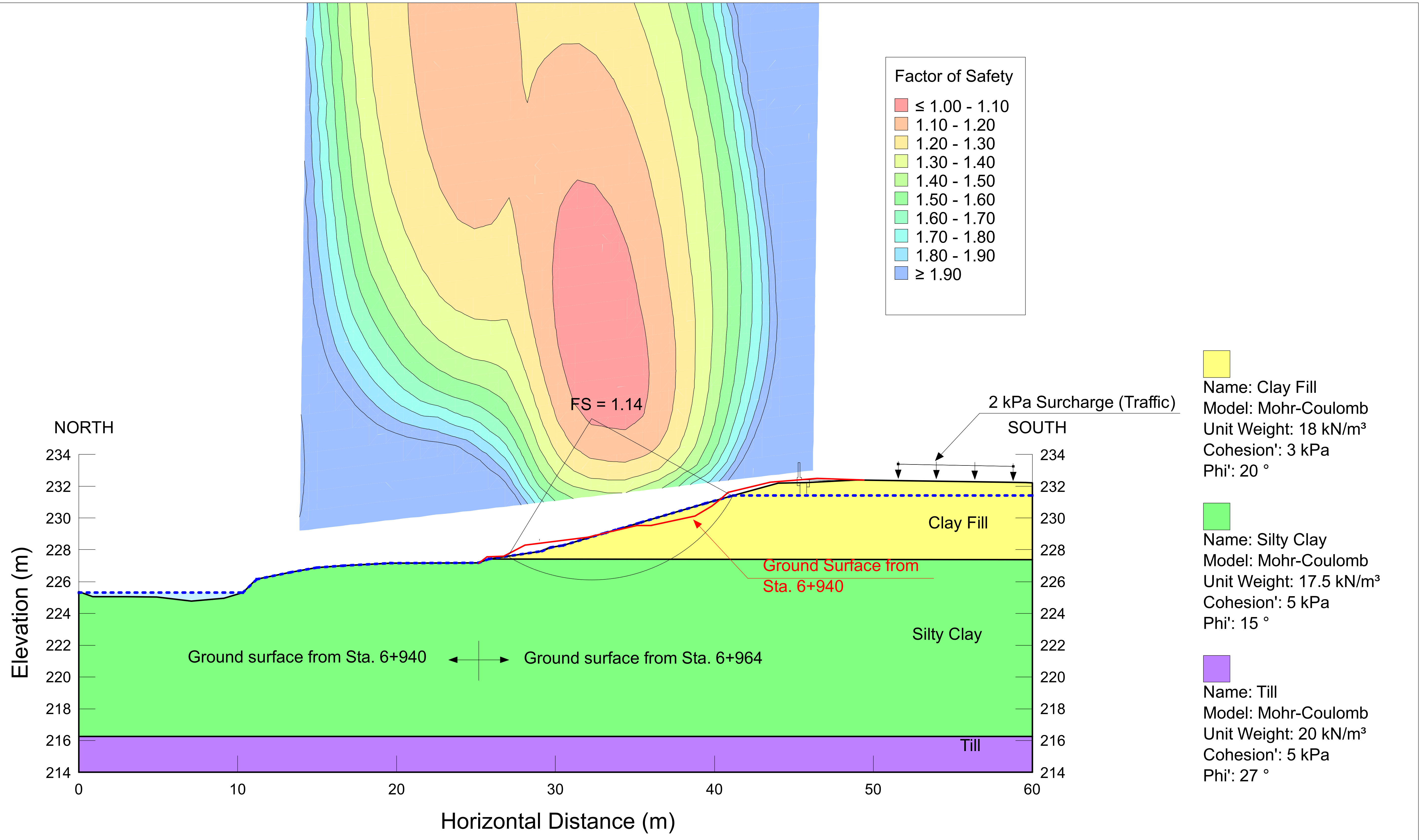
FILE NAME: 6900 - S002.gsz
SCALE: 1:235 (279mm x 432mm)

Figure 14
Sta. 6+900
Modified Geometry

0022 033 00
Dillon Consulting Ltd.
Fermor Avenue over Seine River Bridge



Tabloid (279mm x 432mm)
SAVED: 28/07/2017 3:55:22 PM
FILE NAME: 6964 - S006.gsz
SCALE: 1:235 (279mm x 432mm)

Figure 15
Hybrid Cross-Section
Pre-Failure Back-Analysis

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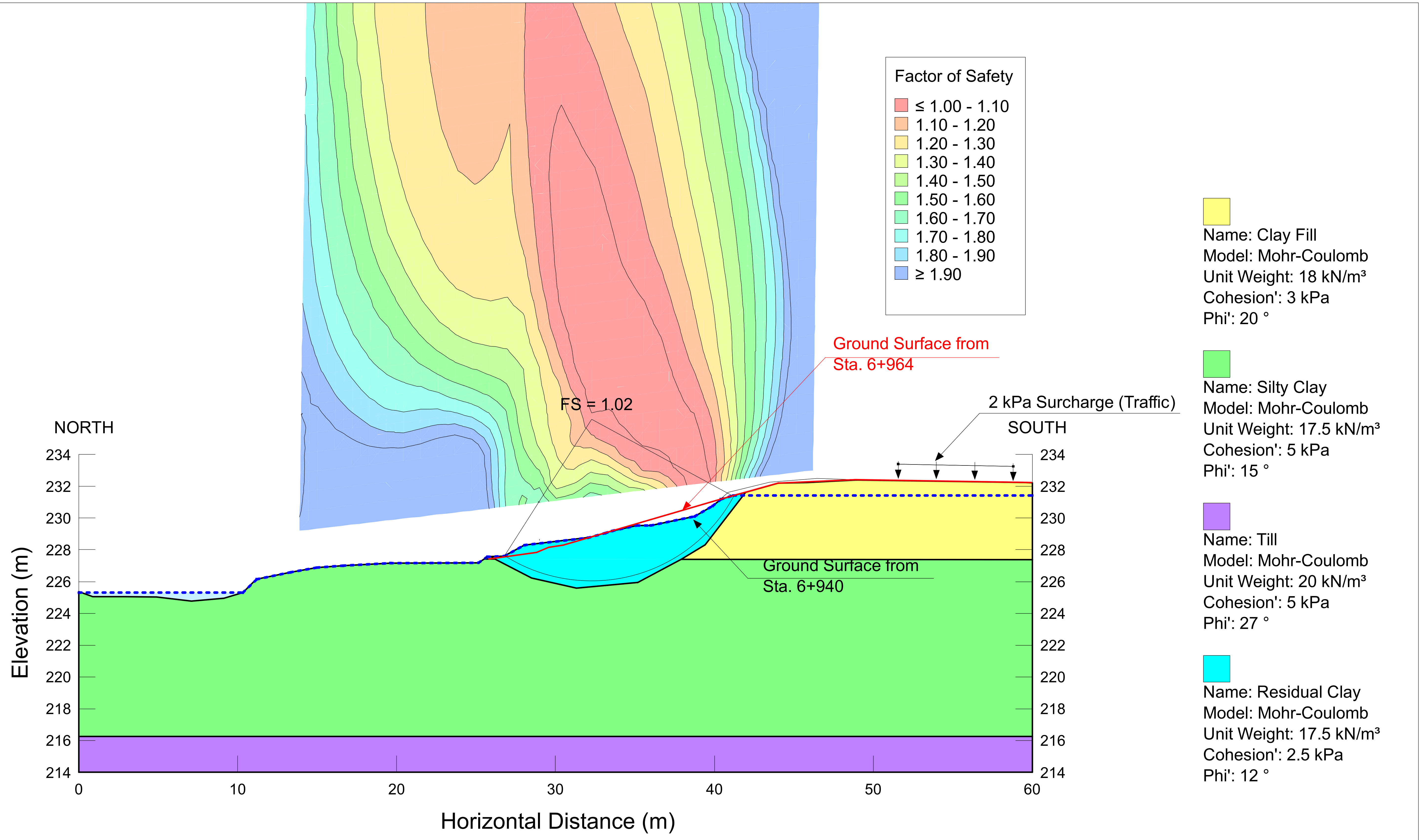
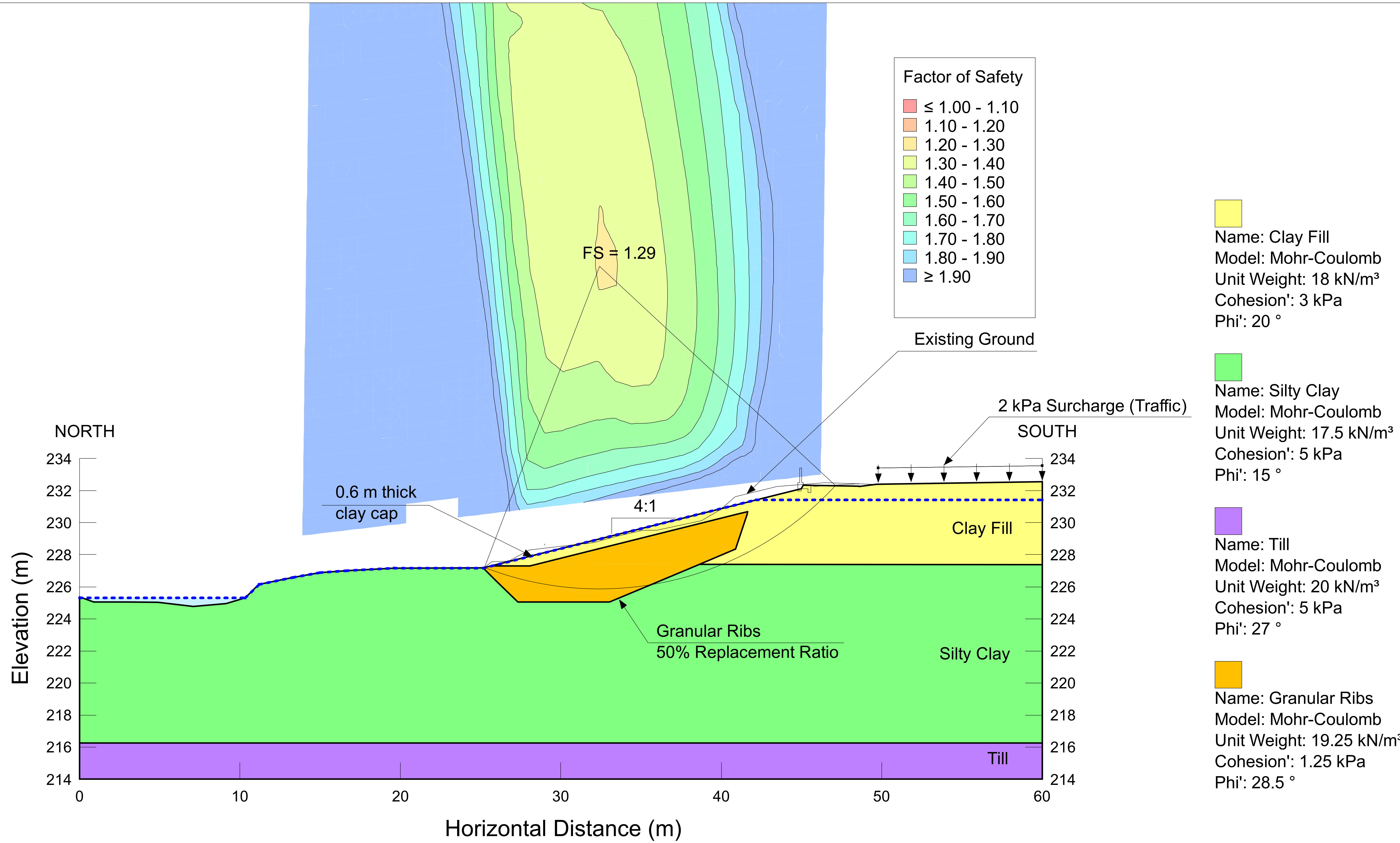
Dillon Consulting Ltd.
Fermor Avenue over Seine River Bridge



Tabloid (279mm x 432mm)
SAVED: 01/08/2017 2:51:10 PM
FILE NAME: 6964 - S007a.gsz
SCALE: 1:235 (279mm x 432mm)

Figure 16
Hybrid Cross-Section
Residual Parameter Back-Analysis



Tabloid (279mm x 432mm)
SAVED: 01/08/2017 4:10:16 PM
FILE NAME: 6940 - S002.gsz
SCALE: 1:235 (279mm x 432mm)

Figure 17
Sta. 6+940
Slope Repair - Shallow Failure

0022 033 00
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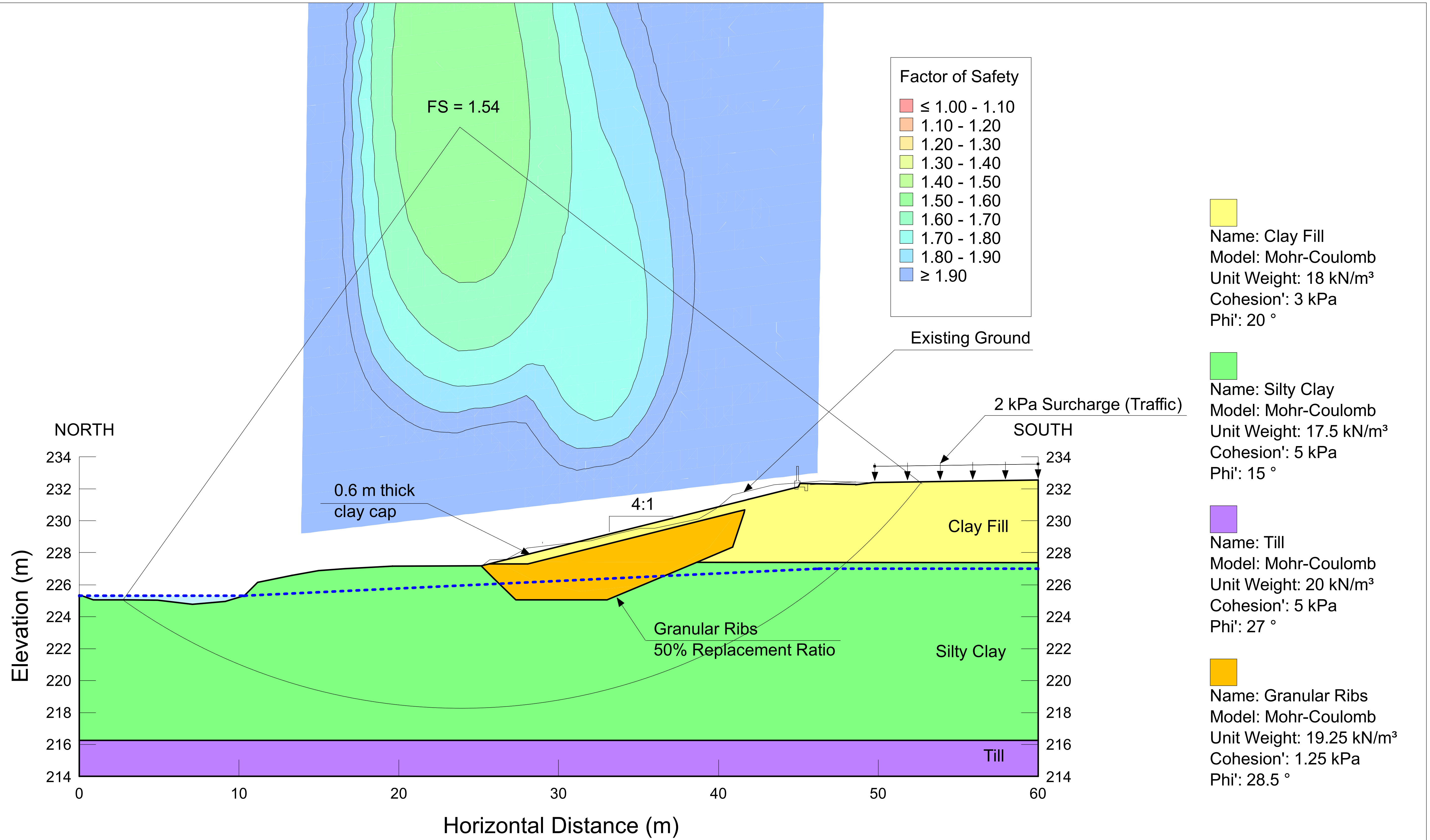
Fermor Avenue over Seine River Bridge



Tabloid (279mm x 432mm)
SAVED: 01/08/2017 4:15:01 PM
FILE NAME: 6940 - S003.gsz
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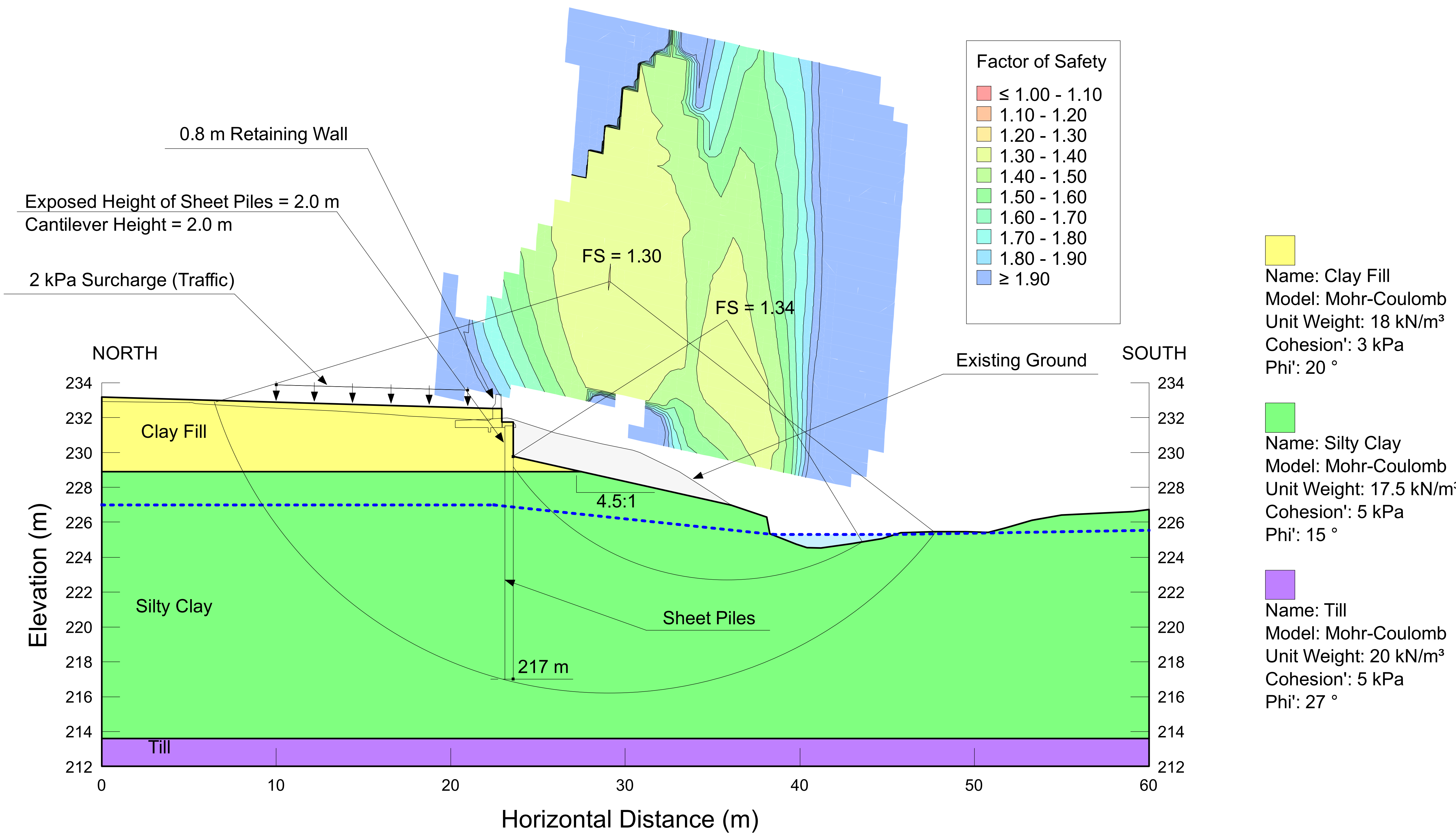
Figure 18
Sta. 6+940
Slope Repair - Normal GWL

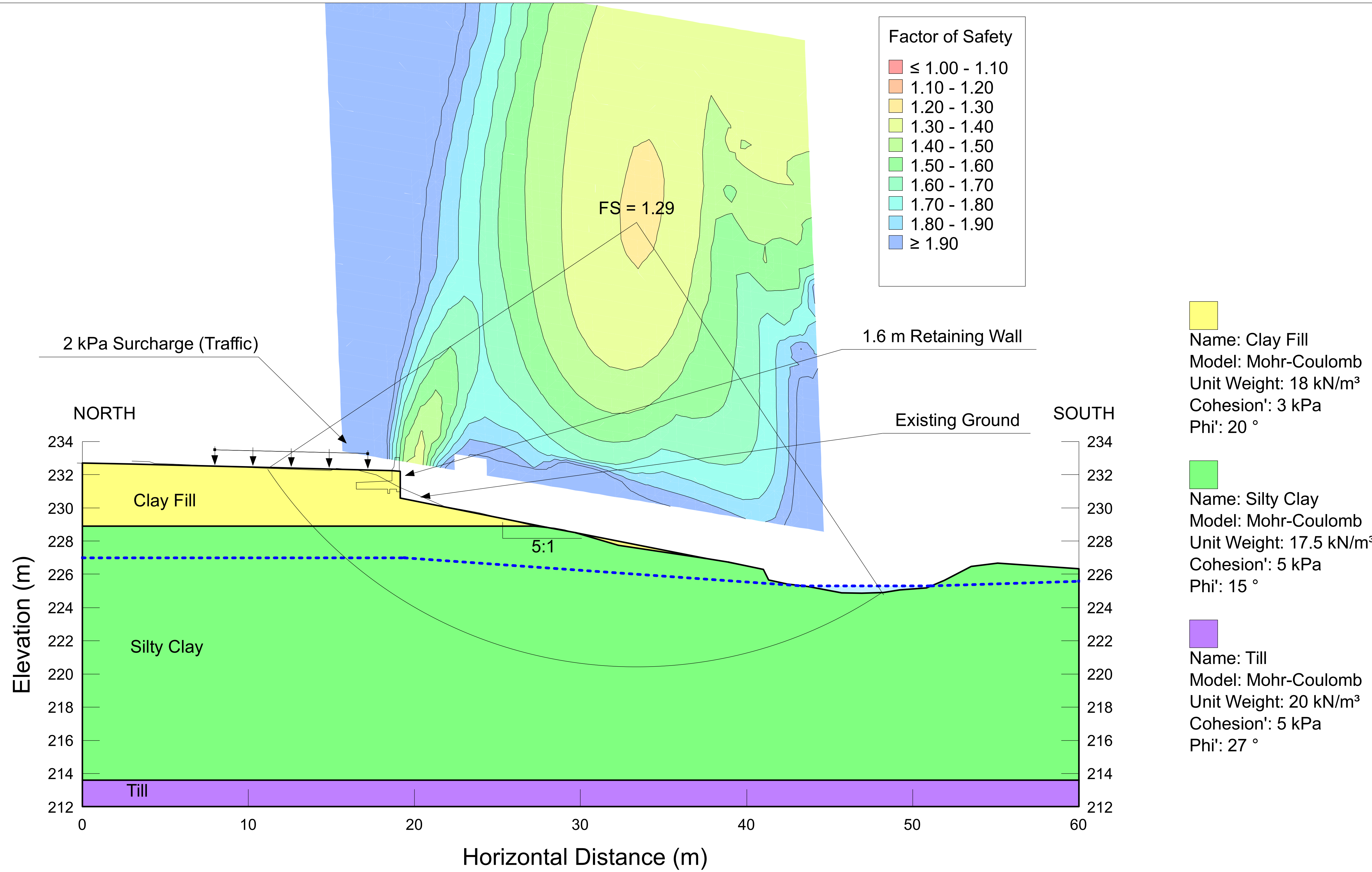
0022 033 00
Dillon Consulting Ltd.
Fermor Avenue over Seine River Bridge



Tabloid (279mm x 432mm)
SAVED: 18/08/2017 2:05:52 PM
FILE NAME: 4850 - S004.gsz
SCALE: 1:235 (279mm x 432mm)

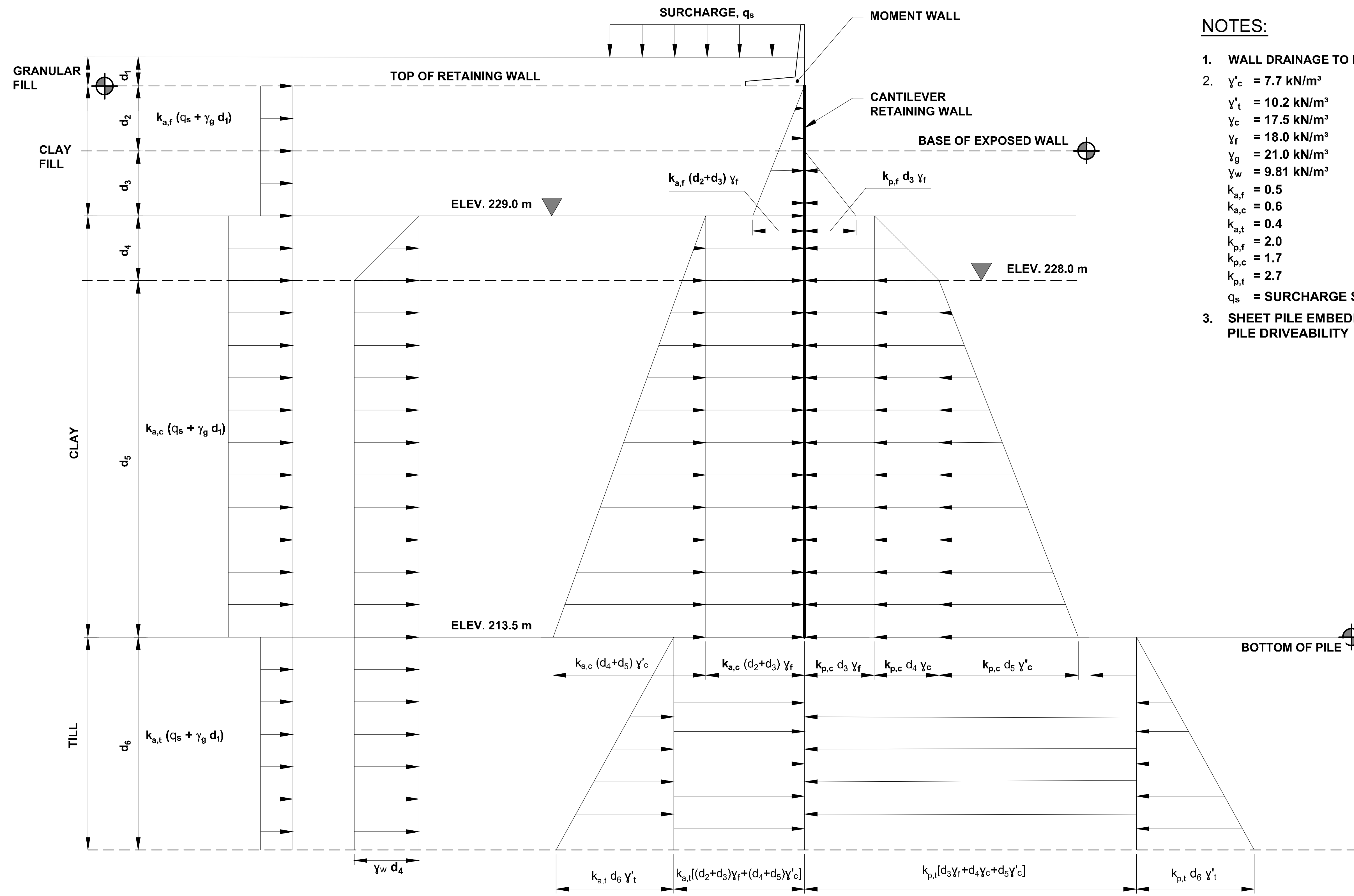
Figure 19
Sta. 4+850
Modified Geometry

0022 033 00
Dillon Consulting Ltd.
Fermor Avenue over Seine River Bridge



Tabloid (279mm x 432mm)
SAVED: 18/08/2017 1:46:53 PM
FILE NAME: 4910 - S002b.gsz
SCALE: 1:235 (279mm x 432mm)

Figure 20
Sta. 4+910
Modified Geometry



NOTES:

1. WALL DRAINAGE TO BE PROVIDED TO BASE OF EXPOSED WALL
2. γ'_c = 7.7 kN/m³
   γ'_t = 10.2 kN/m³
   γ_c = 17.5 kN/m³
   γ_f = 18.0 kN/m³
   γ_g = 21.0 kN/m³
   γ_w = 9.81 kN/m³
   $k_{a,f}$ = 0.5
   $k_{a,c}$ = 0.6
   $k_{a,t}$ = 0.4
   $k_{p,f}$ = 2.0
   $k_{p,c}$ = 1.7
   $k_{p,t}$ = 2.7
   q_s = SURCHARGE SELECTED BY WALL DESIGNER
3. SHEET PILE EMBEDMENT IN TILL SHOULD BE EVALUATED FOR PILE DRIVEABILITY

Figure 21
Lateral Earth Pressure Distribution Figure