

15 September 2025

Mark Hoffman
Garden Homes Management
820 Morris Turnpike, Suite 301
Short Hills, New Jersey 07078

**Re: Global Stability Study for Eastern Slope near Building D
West Essex – Highlands Development
West Orange, New Jersey
Langan Project No.: 101049801**

Dear Mr. Hoffman:

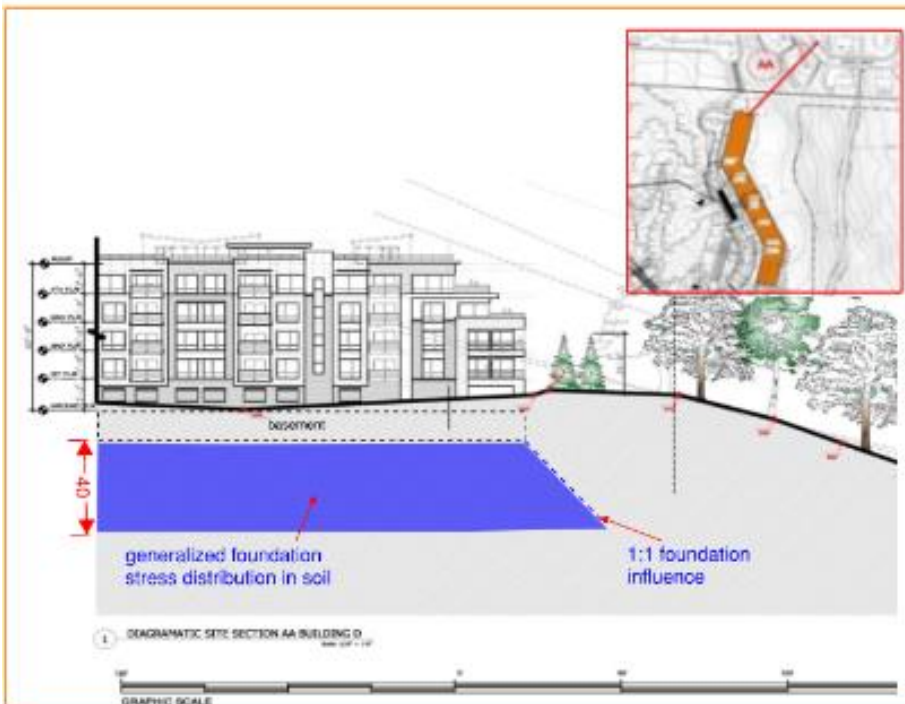
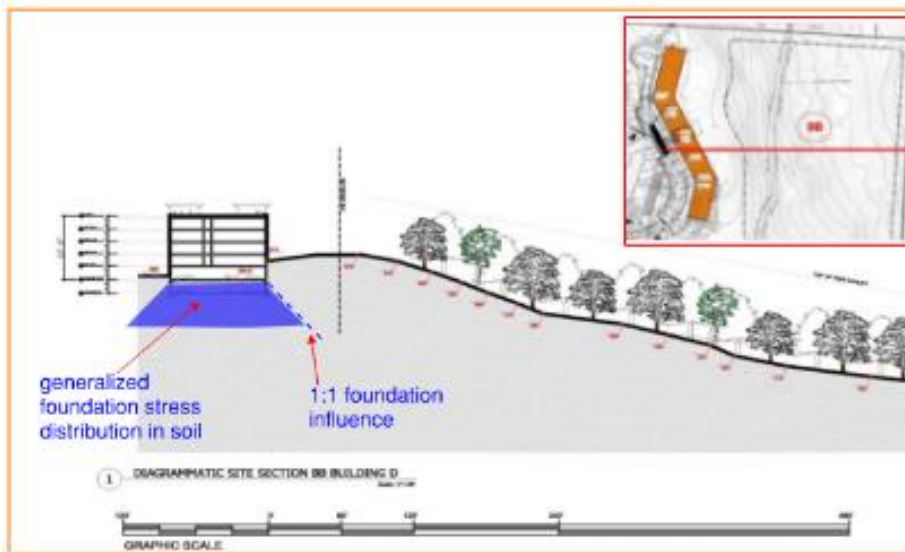
As requested, in conjunction with the above-referenced study Langan Engineering and Environmental Services, LLC (Langan) has reviewed letters prepared by Princeton Hydro ("Princeton"), who has been retained by Objectors to the proposed Development. The Princeton letter, dated 16 July 2025, summarized their comments under four primary issues (below):

1. Building Layout Currently Proposed Closer than Concept
2. Parameters of Slope Stability Analysis Do Not Match Actual Conditions
3. Geologic Hazard Review Unjustified in Dismissing the Risks
4. Blasting Implications and Threats to Stability of the Slope Must be Addressed Before Approval

In a follow up letter dated 12 August 2025, Princeton provided a slope stability analysis at site locations they defined as "Model Cross Sections C and D", which loosely correspond to analysis sections C-C and D-D from Langan's letter report dated 18 June 2025. In the following sections, Langan provides responses to the four primary issues raised in Princeton's 16 July 2025 letter.

In general, it is very important to note that Princeton's slope stability analyses at the two above referenced cross sections indicated the slope to be stable with adequate factors of safety, even after Princeton assigned a much lower strength value to the rock at the site than was used in Langan's analysis. Princeton's slope stability results, therefore, are in general agreement with the Langan conclusions and contradict the basis for Princeton's slope concerns which they expressed in their aforementioned letters and 16 July 2025 testimony by Geoffrey Goll.

To make abundantly clear, our analysis demonstrates that the existing slopes are stable and that the new building does not change the loading condition on the slope; therefore, no change in the slope stability. Schematics below show the general area beneath the central and northern portions of the building where foundation loading occurs and will be distributed in soil. These schematics show the limited influence of the building loading, and specifically no influence on the slope.



Princeton Comment: Building Layout Currently Proposed Closer than Concept

Langan Response: It is common for slight adjustments to be made between conceptual site layout and final layout. Langan utilized the most recent building layout in our slope stability analysis and concluded the slope is currently stable and will be stable under the proposed loading.

Princeton Comment: Parameters of Slope Stability Analysis Do Not Match Actual Conditions

Langan Response: Princeton initially identified seven subparts associated with their position. Five of 7 relate to rock properties used in our model, one relates to building loading, and one relates to the presence of a sewer. In Princeton's 12 August letter they provided analyses using their interpretation of our data. Overall, Princeton repeatedly claims our design values are incorrect, yet they clearly misinterpret the data and mis-apply it.

Princeton is mistakenly focused on the rock parameters as being the primary controlling factor in the slope stability analysis. Although rock strength is important, the soil properties control the stability of this slope. Princeton implies that the critical section for slope stability would extend through the underlying rock stratum. Our output files, provided in our 18 June report, clearly demonstrate that all slope stability critical surfaces (i.e., lowest factors of safety) pass through the overlying soil. Princeton's slope stability output files also clearly demonstrate that the overlying soil properties control the slope stability. The only way for the rock to govern the stability under the design loading conditions would be if the rock strength were lower than the overlying soil strength. Despite both Langan and Princeton proving that point, Princeton incorrectly maintains the rock strength is the primary controlling factor in the slope stability analysis. As there is no grading proposed on the slope, the soil properties do not change between the existing and post development conditions (i.e., no change in the stability).

Princeton also incorrectly stated that Rock Quality Designation (RQD) defines rock mass quality. ASTM D6032 is the Standard Test Method for Determining Rock Quality Designation of Rock Core. The significance and use of this standard is to provide a "simple and inexpensive general indication of rock mass quality". However, RQD, which is determined from 2-inch diameter rock cores, is one basic component of rock mass classification systems which are used in engineering analysis and design to better understand rock mass quality. Fundamentally, RQD is used to identify potential problems within a rock mass and to provide a basis for making preliminary decisions. The Standard clearly states that RQD alone is not sufficient to provide an adequate description of rock mass quality; and, RQD must be used in combination with other geological and geotechnical input. The Rock Mass Rating (RMR) System (aka, Geomechanics Classification System by Bieniawski, 1989) is a widely accepted and utilized method of classifying/categorizing rock masses in terms of general "quality" based on physical and mechanical properties and is used to aid in the prediction of rock mass behavior. The RMR System assigns numerical values to six important parameters (UCS, RQD, Joint Spacing, Joint Condition, Groundwater, and Joint Orientation). It is universally agreed that the RMR system is appropriate for rock mass of good to reasonable quality (i.e., $GSI > 25$, per Hoek and Brown). We assigned values to the RMR System (see Exhibit A) and find the rock mass to be predominantly Fair to Good Quality, not Very Poor Quality as erroneously suggested by Princeton.

Princeton claimed we chose the most competent rock to select model properties, yet our model clearly shows we assigned design values (GSI, UCS, Disturbance, etc.) in our slope stability analysis that are appropriate based on our borings, laboratory testing and evaluation of the rock mass quality and characteristics. Princeton recognized that basalt should have considerably higher uniaxial compressive strength values, but they incorrectly correlated the

relatively lower unconfined compressive strengths to “significant weathering”, which failed to recognize that other properties such as porosity and microstructural attributes (grain size and shape) tend to govern the unconfined compressive strength of brittle rock (Gueguen & Bouteica, 2004).

Princeton’s 12 August model of sections C and D utilized a GSI of 20 and a UCS of about 2300 psi. Rock material having the Princeton assigned values would equate to a very poor quality seamy/disturbed rock or a poor quality disintegrated rock. However, our summary of the rock structure and quality in the table below shows a narrow range in structure from predominantly blocky to very blocky (at one location) and generally good quality to fair quality rock. The upper 5 feet of the rock at NLB-1 (our analysis section C-C and Princeton’s model section C) exhibits rough slightly weathered joint conditions, which are considered Good surface quality within the GSI system, not very poor surface quality as suggested by Princeton. In addition, field estimates of typical intact rock strength are shown to range from roughly 3,625 psi to 36,250 psi, the upper end of which is more closely associated with basalt. Nonetheless, we modeled analysis section C-C using a GSI value of 37 and a comparatively low intact rock strength and there was no change in the factor of safety (Exhibit B). Rock core photos are provided in Exhibit C. Princeton’s reference to using a factor of safety of 2.0 is simply not applicable. The EM 1110-1-2908 reference is applicable to US Corp of Engineer facilities built on rock that experience a lateral load (e.g., a dam), which if the slope failed would be catastrophic to neighboring properties. This development clearly does not apply. A factor of safety of 1.5 is applicable for the subject slope.

Rock deformation modulus, which is measure of a rock mass’s resistance to deformation under load, calculated using a GSI of 20 and Disturbance factor of 1 (Hoek and Brown, 2018), indicates a range in values of 35 to 68 MPa, which is only 2 to 4 times greater than the unconfined compressive strength of the rock. The modulus should be hundreds of times greater than the unconfined compressive strength of the rock. Therefore, using a GSI of 20 is not representative of the rock mass throughout the site. For comparison, the estimated deformation modulus of rock having a GSI of 65 ranges from about 2,000 to 8,000 MPa depending on the Disturbance factor.

LOCATION	RQD	JOINT CONDITION		STRUCTURE	GSI Range	Grade ¹ and Range in Uniaxial Strength (psi)		RMR ²
LB-1	58-82	Good	A	Blocky	55-75	R3-R5	3625-36250	Fair to Good
LB-2	67-77	Good	A	Blocky	55-75	R3-R5	3625-36250	Fair to Good
NLB-1	0-23	Fair	B	Very Blocky	37-55	R3-R5	3625-36250	Fair
NLB-2	72-93	Good	A	Blocky	55-75	R3-R5	3625-36250	Good
NLB-3	42-48	Good	A	Blocky	55-75	R3-R5	3625-36250	Fair
NLB-4	52-83	Good	A	Blocky	55-75	R3-R5	3625-36250	Fair to Good

1. Brown, 1981; Hoek and Karzulovic, 2000

R3: Medium Strong. Cannot be scraped with a pocket knife. Can be fractured with a single blow from a geological hammer.

R4: Strong. Specimen requires more than one blow from a geological hammer to break.

R5: Strong. Specimen requires many blows from a geological hammer to break.

2. Rock Mass Rating System, Bieniawski, 1989

The RMR varies from Fair to Good rock; but, is classified overall as Fair.

Joint Description:

A. Rough, slightly weathered, some iron staining

B. Rough, moderately weathered

Princeton also claimed we used incorrect values for the overlying soil in their 12 August letter. Their model of sections C and D use a thicker medium dense section than exists. We provide in Exhibit D a summary of the Standard Penetration Test N-values and their associated correlations to strength parameters for the soils encountered in our borings. We have stated that our design parameters are lower bound (conservative) values, as clearly shown in the graphic. Yet, Princeton continues to misinterpret the available data. In their 12 August models for sections C and D, they assigned 32 degrees friction to a 20-foot-thick section of soil, despite there being only one boring location (NLB-3) where a 4-ft-thick-layer of the upper sands was encountered to have N-values that correlate to 32 degrees. The average friction value for these soils in our Sections C-C and D-D are 39 degrees and 45 degrees, respectively.

Princeton mistakenly stated Langan dismissed Building D loading at the top of the slope. Princeton also mistakenly stated Langan did not model the excavation. We modelled the proposed loading conditions, just as we said we did in our report. We also modelled a strip load of 4 ksf; neither a strip load nor a uniform load had an impact on the slope stability.

Princeton also stated we should not ignore the Feltville Formation. The borings (LB-2 and LB-4) in our analysis sections C-C and D-D were terminated in decomposed rock (roughly 50 feet below ground) based on split-spoon refusal values. The critical surfaces for all our slope stability models did not extend into the decomposed rock stratum for any of the design cases. As demonstrated above, the Feltville Formation also does not factor into the slope stability due to its depth and the fact that decomposed rock is stronger than the overlying soils.

We modeled our analysis sections A-A and B-B with the discontinuity of the wastewater utility line as a trench of medium dense sand having a friction value of 32 degrees extending from ground surface to the decomposed rock over 30 feet below the existing grade. Our models show that there is no change in the location of the critical failure surfaces or corresponding factors of safety (Exhibit B).

Princeton Comment: Geologic Hazard Review Unjustified in Dismissing the Risks

Langan Response: Langan has not dismissed any risk of a slide. We were retained to investigate and analyze the slope subsurface conditions to assess the slope stability risk. Langan performed the necessary investigations including borings, test pits and outcrop mapping. Langan concluded that the outcrops and overall slopes are stable under existing and proposed conditions.

Furthermore, Princeton mistakenly dismissed rock outcrop mapping and stated that the slopes would be susceptible to landslides based solely on RQD results. Rockfall hazard was assessed by Langan and was found to be low based on the predominance of favorable joint orientations. Furthermore, Mr. Goll's testimony suggested a fictitious rock mass instability could develop during the limited excavation of rock in the building footprint beyond the top of the ridge and further suggested that rock anchors would likely be used to stabilize the rock mass. This idea that the rock in the slope would fail without any disturbance in the slope completely ignores the fact that the rock structure/jointing is both relatively rough and

favorably oriented, which are conditions that control stability in jointed hard rock formations such as the basalt.

Princeton Comment: Blasting Implication and Threats to Stability of the Slope Must be Addressed Before Approval

Langan Response: There has been much discussion concerning blasting; however, Langan is not aware of any plan to blast rock at the site. Based on the available subsurface information, the homes closest to rock to be excavated are roughly 450-500 feet away. Langan maintains its opinion that if blasting is used to excavate rock, controlled techniques should be used to control impacts to the rock mass such as overbreak. Langan has no knowledge of the means and methods that will be selected by the Developer's contractor to excavate soil and rock at Building D. However, the following paragraphs address methods of rock excavation that would be appropriate for the conditions encountered.

As indicated in our 18 June report that the upper rock could not be penetrated by the excavation equipment used during the investigation which included John Deere 75G, CAT 308 and Hitachi 7X85 excavators, all of which are smaller than the type of equipment we expect to be used during construction. Nonetheless, specialized equipment is expected to be required for ripping rock. Hoek-Karzulovic, 2000; and, Abdullatif and Cruden, 1983 indicate rock masses having GSI values up to 40 (1 MPa) can be excavated by conventional means, rock masses having GSI values up to 60 (10 MPa) can be ripped, and for rock masses having GSI over 60 (>15 MPa) blasting has typically been utilized. Splitters and chemical expanders can generate pressures up to 43 MPa and 30-44 MPa, respectively. These methods work based on the tensile strength of rock, which is typically 6-15% of the rock's compressive strength (Al-Bakri and Hefni, 2021).

If the basalt at the subject slope has a GSI value of 20 as suggested by Princeton, we would expect the contractor to use conventional excavation methods, not blasting. Our findings indicate most of the section of rock mass to be excavated (i.e., the large outcrop roughly 450-500 feet from Howell Drive) has a GSI value of roughly 65 but is expected to range as low as about 50. Such rock is expected to require some combination of mechanical methods to excavate, including large excavators or loaders equipped with ripping teeth or shanks, hoe rams, and splitters. However, locally and at a minimum as a starter method, splitters and chemical expanders would also be effective on the higher GSI material, as would controlled blasting methods.

If controlled blasting is used to excavate the rock (including properly designed and implemented channel and line drilling), the blast should be designed to produce a locally (i.e., the building limits) fractured rock mass having the maximum quality such as a GSI value of 40-60 (i.e., least disturbance) that allows for mechanical removal. Regardless of the method used, the excavation should be designed by the contractor to prevent over breakage of the rock and damage to nearby structures.

CLOSURE

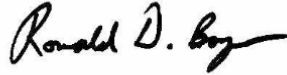
We maintain the analysis and conclusions presented in our 18 June 2025 report. Please call with questions.

Sincerely,

Langan Engineering and Environmental Services, LLC



Mark K. Seel, P.E.
Associate Principal / Vice President



Ronald D. Boyer, P.E.
Senior Principal / Senior Vice President

MKS/RDB:mks

Enclosure(s): Exhibit A – RMR Output files
Exhibit B – SlopeW Output files
Exhibit C – Rock Core Photos
Exhibit D – N-value – Friction correlations

EXHIBIT A

ROCK MASS RATING (RMR)

ROCK MASS RATING (RMR) - UPPER RANGE

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of Values						
1	Strength of intact rock material	Point-load strength index	> 1,450 psi	580 psi - 1,450 psi	290 psi - 580 psi	145 psi - 290 psi	For this low range - Uniaxial Compressive Strength Test is preferred		
		Uniaxial comp. strength	> 36,300 psi	14,500 psi - 36,300 psi	7,250 psi - 14,500 psi	3,630 psi - 7,250 psi	725 psi - 3,630 psi	145 psi - 725 psi	< 145 psi
	Rating		15	12	7	4	2	1	0
12									
2	Drill Core Quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
8									
3	Spacing of Joints (See E)		> 78.7 in	23.6 in - 78.7 in	7.87 in - 23.6 in	2.36 in - 7.87 in	< 2.36 in		
	Rating		20	15	10	8	5		
8									
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <0.04 in Slightly weathered walls	Slightly rough surfaces Separation <0.04 in Highly weathered walls	Sickensided surfaces or Gouge <0.2 in thick or Separation 0.04-0.2 in Continuous	Soft gouge >0.2 in thick or Separation >0.04 in Continuous		
	Rating		30	25	20	10	3		
25									
5	Groundwater	Inflow per 32.8 ft tunnel length (L/min)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General Conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
15									
Rating Value for Part A: 68									
B. RATING ADJUSTMENTS FOR DISCONTINUITY ORIENTATIONS (See E & F)									
Strike and dip orientations			Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable		
Ratings	Tunnel & mines		0	-2	-5	-10	-12		
	Foundations		0	-2	-7	-15	-25		
	Slopes		0	-5	-25	-50	-60		
ENTER RATING NUMBERS TO THE RIGHT SEPARATELY			Tunnel & mines ENTER RATING HERE	Foundations ENTER RATING HERE	Slopes -5	Rating Value for Part B: -5			
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS ("number of points")									
Rating			100 <-- 81	80 <-- 61	60 <-- 41	40 <-- 21	< 21		
Class			I	II	III	IV	V		
Description			Very good rock	Good rock	Fair rock	Poor rock	Very poor rock		
CLASS II; Good Rock									
D. MEANING OF ROCK CLASSES									
Class number			I	II	III	IV	V		
Average stand-up line			20 yrs for 49.2 ft span	1 year for 32.8 ft span	1 week for 16.4 ft span	10 hrs for 8.2 ft span	30 min for 3.28 ft span		
Cohesion of rock mass (psi)			> 58 psi	43.5 psi - 58 psi	29 psi - 43.5 psi	14.5 psi - 29 psi	< 14.5 psi		
Friction angle of rock mass (deg)			> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°		
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS									
Discontinuity length (persistence)			< 3.28 ft	3.28 ft - 9.84 ft	9.84 ft - 32.8 ft	32.8 ft - 65.6 ft	> 65.6 ft		
Rating			6	4	2	1	0		
Separation (aperture)			None	< 0.004 in	0.004 in - 0.04 in	0.04 in - 0.2 in	> 0.2 in		
Rating			6	5	4	1	0		
Roughness			Very rough	Rough	Slightly rough	Smooth	Sickensided		
Rating			6	5	3	1	0		
Infilling (gouge)			None	Hard filling < 0.2 in	Hard filling > 0.2 in	Soft filling < 0.2 in	Soft filling > 0.2 in		
Rating			6	4	2	2	0		
Weathering			Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed		
Rating			6	5	3	1	0		
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING**									
Strike perpendicular to tunnel axis					Strike parallel to tunnel axis				
Drive with dip - Dip 45 - 90°			Drive with dip - Dip 20 - 45°			Dip 45 - 90°		Dip 20 - 45°	
Very favourable			Favourable			Very unfavourable		Fair	
Drive against dip - Dip 45 - 90°			Drive against dip - Dip 20 - 45°			Dip 0-20 - Irrespective of strike*			
Fair			Unfavourable			Fair			

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al (1972).

ROCK MASS RATING (RMR) - AVERAGE RANGE

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of Values						
1	Strength of intact rock material	Point-load strength index	> 1,450 psi	580 psi - 1,450 psi	290 psi - 580 psi	145 psi - 290 psi	For this low range - Uniaxial Compressive Strength Test is preferred		
		Uniaxial comp. strength	> 36,300 psi	14,500 psi - 36,300 psi	7,250 psi - 14,500 psi	3,630 psi - 7,250 psi	725 psi - 3,630 psi	145 psi - 725 psi	< 145 psi
	Rating		15	12	7	4	2	1	0
7									
2	Drill Core Quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
8									
3	Spacing of Joints (See E)		> 78.7 in	23.6 in - 78.7 in	7.87 in - 23.6 in	2.36 in - 7.87 in	< 2.36 in		
	Rating		20	15	10	8	5		
8									
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <0.04 in Slightly weathered walls	Slightly rough surfaces Separation <0.04 in Highly weathered walls	Sickensided surfaces or Gouge <0.2 in thick or Separation 0.04-0.2 in	Soft gouge >0.2 in thick or Separation >0.04 in Continuous		
	Rating		30	25	20	10	3		
25									
5	Groundwater	Inflow per 32.8 ft tunnel length (L/min)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General Conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
10									
Rating Value for Part A: 58									
B. RATING ADJUSTMENTS FOR DISCONTINUITY ORIENTATIONS (See E & F)									
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable			
Ratings	Tunnel & mines	0	-2	-5	-10	-12			
	Foundations	0	-2	-7	-15	-25			
	Slopes	0	-5	-25	-50	-60			
ENTER RATING NUMBERS TO THE RIGHT SEPARATELY		Tunnel & mines ENTER RATING HERE	Foundations ENTER RATING HERE	Slopes -3	Rating Value for Part B: -3				
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS ("number of points")									
Rating		100 <-- 81	80 <-- 61	60 <-- 41	40 <-- 21	< 21			
Class		I	II	III	IV	V			
Description		Very good rock	Good rock	Fair rock	Poor rock	Very poor rock			
CLASS III; Fair Rock									
D. MEANING OF ROCK CLASSES									
Class number		I	II	III	IV	V			
Average stand-up line		20 yrs for 49.2 ft span	1 year for 32.8 ft span	1 week for 16.4 ft span	10 hrs for 8.2 ft span	30 min for 3.28 ft span			
Cohesion of rock mass (psi)		> 58 psi	43.5 psi - 58 psi	29 psi - 43.5 psi	14.5 psi - 29 psi	< 14.5 psi			
Friction angle of rock mass (deg)		> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°			
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS									
Discontinuity length (persistence)		< 3.28 ft	3.28 ft - 9.84 ft	9.84 ft - 32.8 ft	32.8 ft - 65.6 ft	> 65.6 ft			
Rating		6	4	2	1	0			
Separation (aperture)		None	< 0.004 in	0.004 in - 0.04 in	0.04 in - 0.2 in	> 0.2 in			
Rating		6	5	4	1	0			
Roughness		Very rough	Rough	Slightly rough	Smooth	Sickensided			
Rating		6	5	3	1	0			
Infilling (gouge)		None	Hard filling < 0.2 in	Hard filling > 0.2 in	Soft filling < 0.2 in	Soft filling > 0.2 in			
Rating		6	4	2	2	0			
Weathering		Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed			
Rating		6	5	3	1	0			
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING**									
Strike perpendicular to tunnel axis				Strike parallel to tunnel axis					
Drive with dip - Dip 45 - 90°		Drive with dip - Dip 20 - 45°		Dip 45 - 90°			Dip 20 - 45°		
Very favourable		Favourable		Very unfavourable			Fair		
Drive against dip - Dip 45 - 90°		Drive against dip - Dip 20 - 45°		Dip 0-20 - Irrespective of strike°					
Fair		Unfavourable		Fair					

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al (1972).

ROCK MASS RATING (RMR) - LOWER RANGE

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of Values						
1	Strength of intact rock material	Point-load strength index	> 1,450 psi	580 psi - 1,450 psi	290 psi - 580 psi	145 psi - 290 psi	For this low range - Uniaxial Compressive Strength Test is preferred		
		Uniaxial comp. strength	> 36,300 psi	14,500 psi - 36,300 psi	7,250 psi - 14,500 psi	3,630 psi - 7,250 psi	725 psi - 3,630 psi	145 psi - 725 psi	< 145 psi
	Rating		15	12	7	4	2	1	0
2									
2	Drill Core Quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3									
3	Spacing of Joints (See E)		> 78.7 in	23.6 in - 78.7 in	7.87 in - 23.6 in	2.36 in - 7.87 in	< 2.36 in		
	Rating		20	15	10	8	5		
8									
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <0.04 in Slightly weathered walls	Slightly rough surfaces Separation <0.04 in Highly weathered walls	Sickensided surfaces or Gouge <0.2 in thick or Separation 0.04-0.2 in Continuous	Soft gouge >0.2 in thick or Separation >0.04 in Continuous		
	Rating		30	25	20	10	3		
25									
5	Groundwater	Inflow per 32.8 ft tunnel length (L/min)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General Conditions	Completely dry	Damp	Wet	Dripping	Flowing		
		Rating	15	10	7	4	0		
10									
Rating Value for Part A: 48									
B. RATING ADJUSTMENTS FOR DISCONTINUITY ORIENTATIONS (See E & F)									
Strike and dip orientations			Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable		
Ratings	Tunnel & mines		0	-2	-5	-10	-12		
	Foundations		0	-2	-7	-15	-25		
	Slopes		0	-5	-25	-50	-60		
ENTER RATING NUMBERS TO THE RIGHT SEPARATELY			Tunnel & mines ENTER RATING HERE	Foundations ENTER RATING HERE	Slopes -5	Rating Value for Part B: -5			
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS ("number of points")									
Rating			100 <-- 81	80 <-- 61	60 <-- 41	40 <-- 21	< 21		
Class			I	II	III	IV	V		
Description			Very good rock	Good rock	Fair rock	Poor rock	Very poor rock		
CLASS III; Fair Rock									
D. MEANING OF ROCK CLASSES									
Class number			I	II	III	IV	V		
Average stand-up line			20 yrs for 49.2 ft span	1 year for 32.8 ft span	1 week for 16.4 ft span	10 hrs for 8.2 ft span	30 min for 3.28 ft span		
Cohesion of rock mass (psi)			> 58 psi	43.5 psi - 58 psi	29 psi - 43.5 psi	14.5 psi - 29 psi	< 14.5 psi		
Friction angle of rock mass (deg)			> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°		
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS									
Discontinuity length (persistence)			< 3.28 ft	3.28 ft - 9.84 ft	9.84 ft - 32.8 ft	32.8 ft - 65.6 ft	> 65.6 ft		
Rating			6	4	2	1	0		
Separation (aperture)			None	< 0.004 in	0.004 in - 0.04 in	0.04 in - 0.2 in	> 0.2 in		
Rating			6	5	4	1	0		
Roughness			Very rough	Rough	Slightly rough	Smooth	Sickensided		
Rating			6	5	3	1	0		
Infilling (gouge)			None	Hard filling < 0.2 in	Hard filling > 0.2 in	Soft filling < 0.2 in	Soft filling > 0.2 in		
Rating			6	4	2	2	0		
Weathering			Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed		
Rating			6	5	3	1	0		
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELING**									
Strike perpendicular to tunnel axis					Strike parallel to tunnel axis				
Drive with dip - Dip 45 - 90°			Drive with dip - Dip 20 - 45°			Dip 45 - 90°		Dip 20 - 45°	
Very favourable			Favourable			Very unfavourable		Fair	
Drive against dip - Dip 45 - 90°			Drive against dip - Dip 20 - 45°			Dip 0-20 - Irrespective of strike*			
Fair			Unfavourable			Fair			

* Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

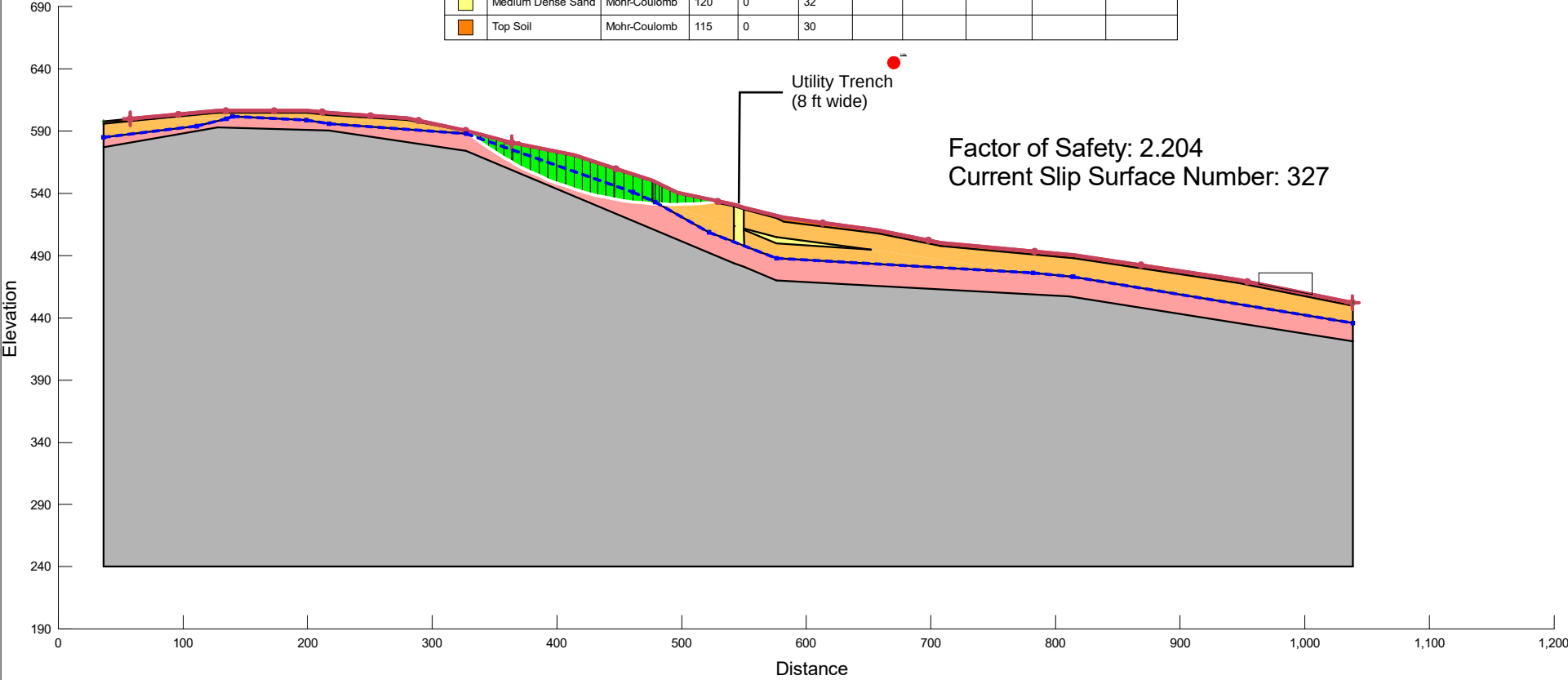
** Modified after Wickham et al (1972).

EXHIBIT B

ADDITIONAL SECTIONS

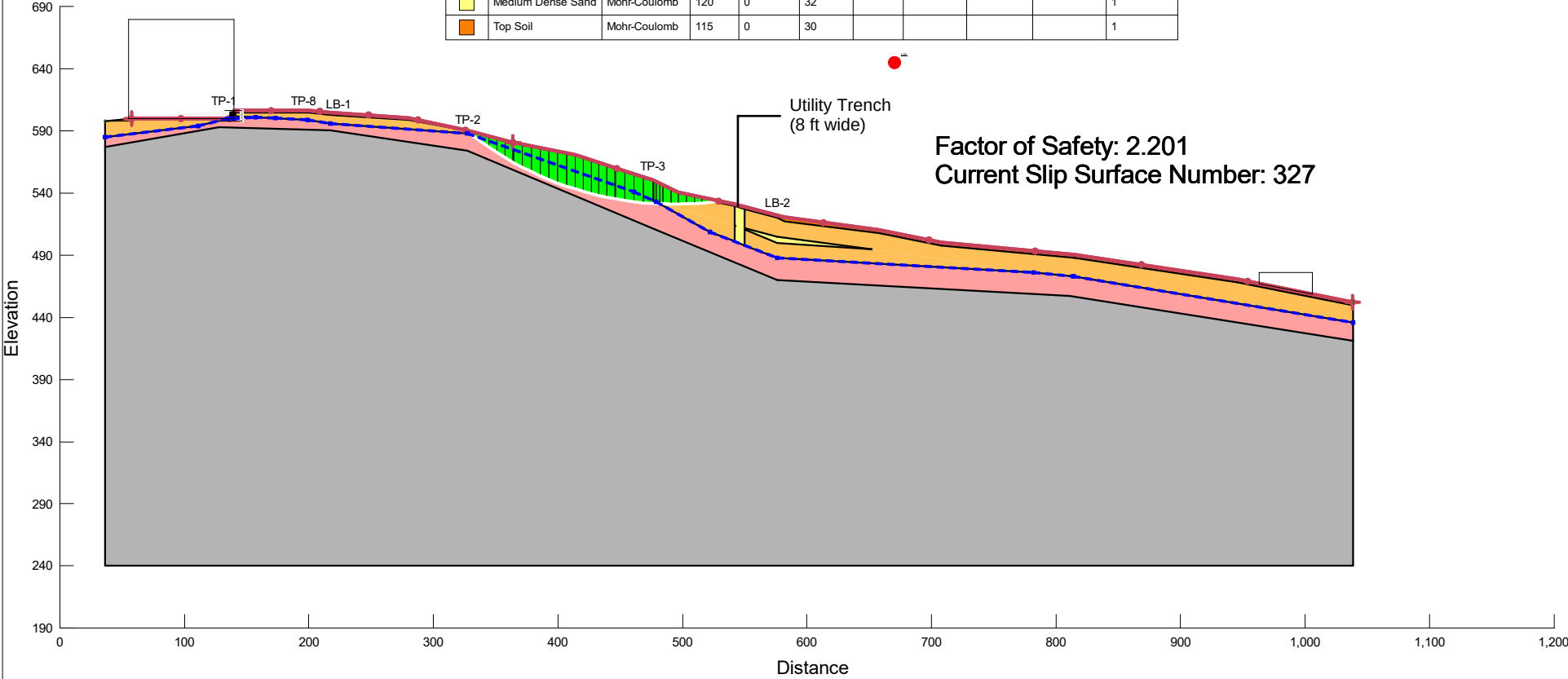
Cross Section A (Static Condition)

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
■	Bedrock	Hoek-Brown	180			323,263	17	65	1	1
■	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
■	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					
■	Medium Dense Sand	Mohr-Coulomb	120	0	32					
■	Top Soil	Mohr-Coulomb	115	0	30					



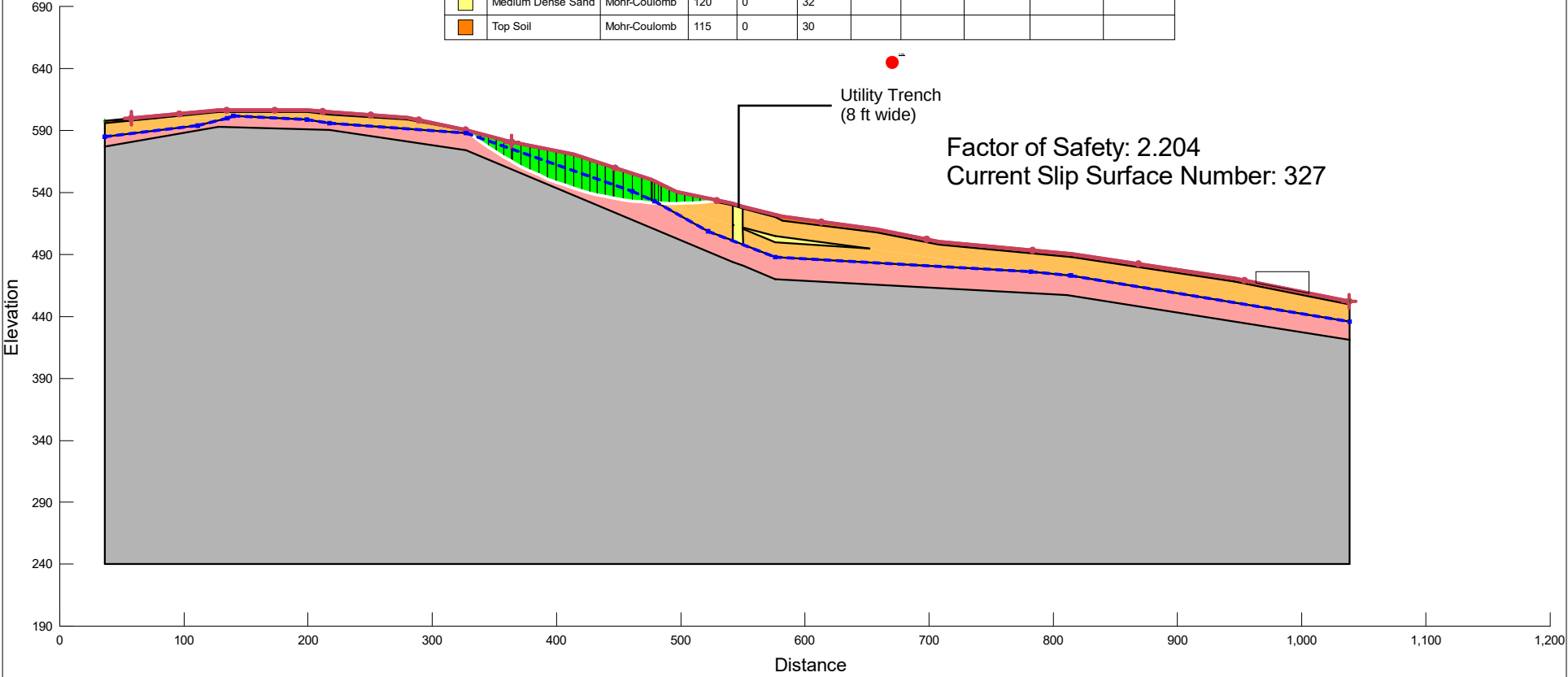
Cross Section A_Existing Condition
(GSI = 65)

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	Hoek-Brown	180			323,263	17	65	1	1
Red	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
Orange	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					1
Yellow	Medium Dense Sand	Mohr-Coulomb	120	0	32					1
Dark Orange	Top Soil	Mohr-Coulomb	115	0	30					1



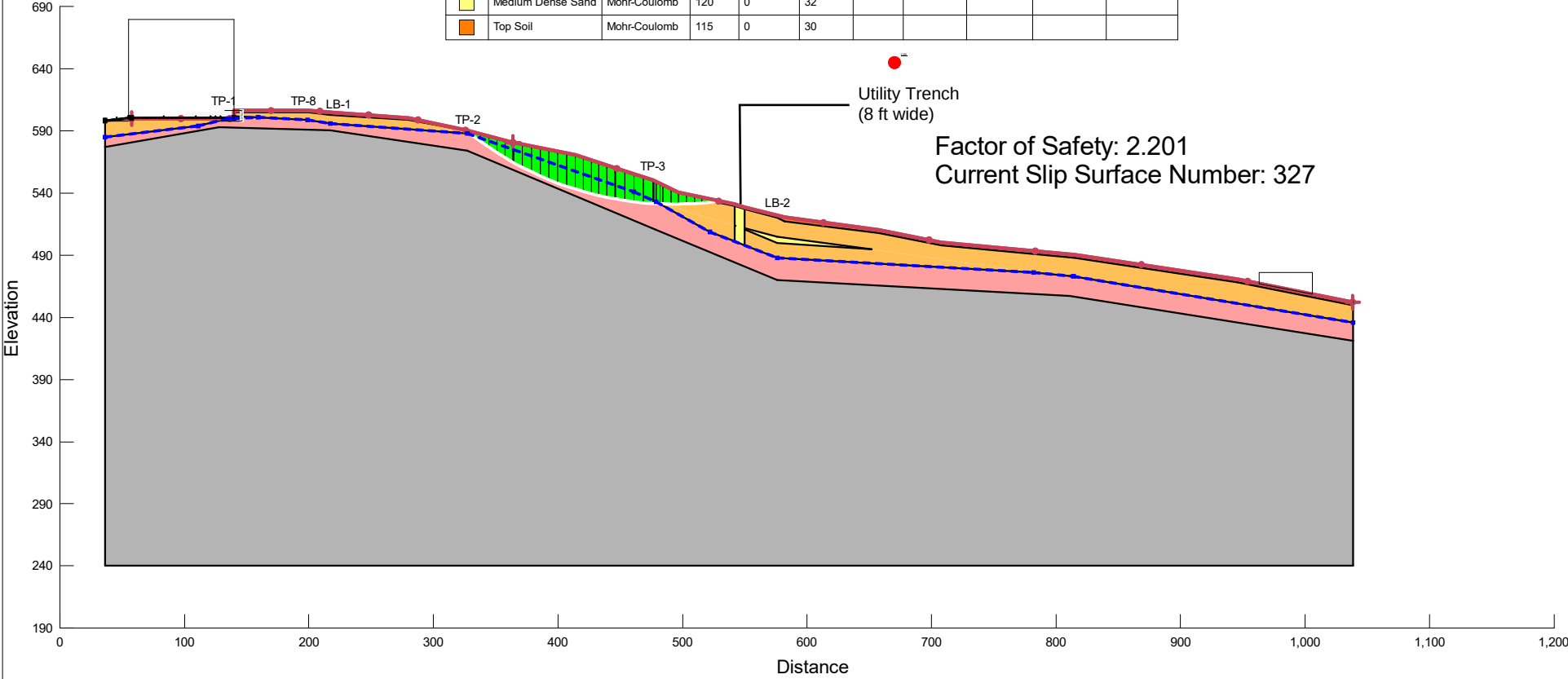
Cross Section A_Proposed Condition
(GSI = 65)

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	Hoek-Brown	180			323,263	17	50	1	1
Red	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
Orange	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					
Yellow	Medium Dense Sand	Mohr-Coulomb	120	0	32					
Brown	Top Soil	Mohr-Coulomb	115	0	30					



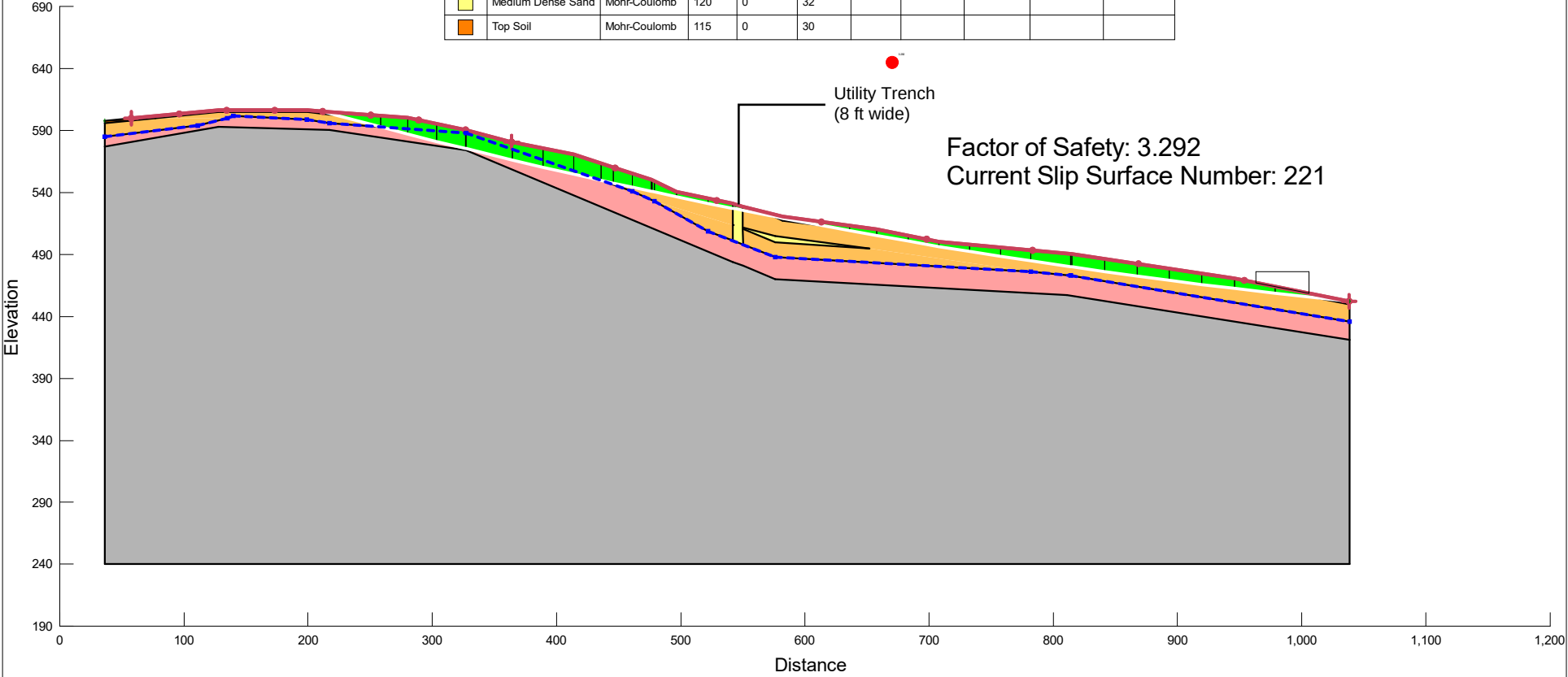
**Cross Section A_Existing Condition
(GSI = 50)**

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	Hoek-Brown	180			323,263	17	50	1	1
Red	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
Orange	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					
Yellow	Medium Dense Sand	Mohr-Coulomb	120	0	32					
Dark Orange	Top Soil	Mohr-Coulomb	115	0	30					



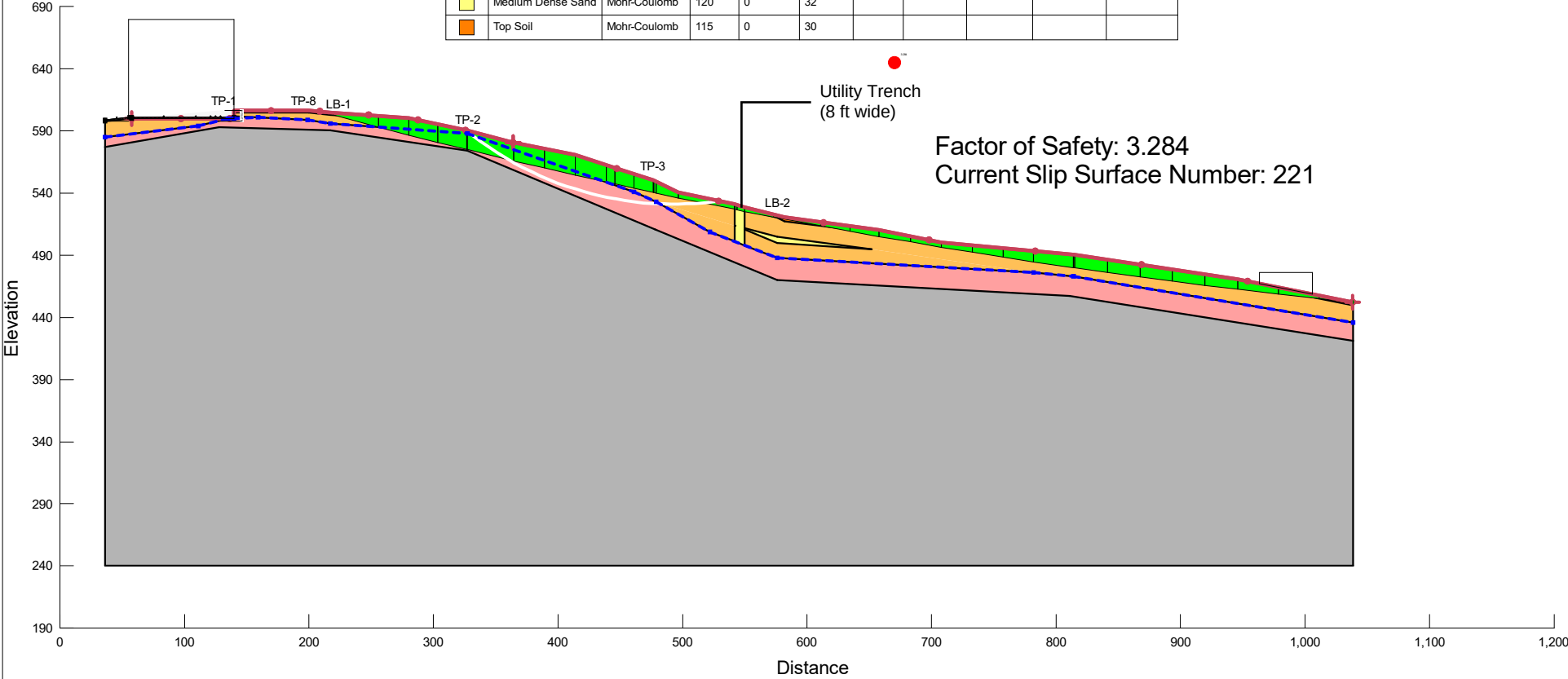
Cross Section A_Proposed Condition
(GSI = 50)

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	Hoek-Brown	180			323,263	17	65	1	1
Light Red	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
Orange	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					
Yellow	Medium Dense Sand	Mohr-Coulomb	120	0	32					
Dark Orange	Top Soil	Mohr-Coulomb	115	0	30					



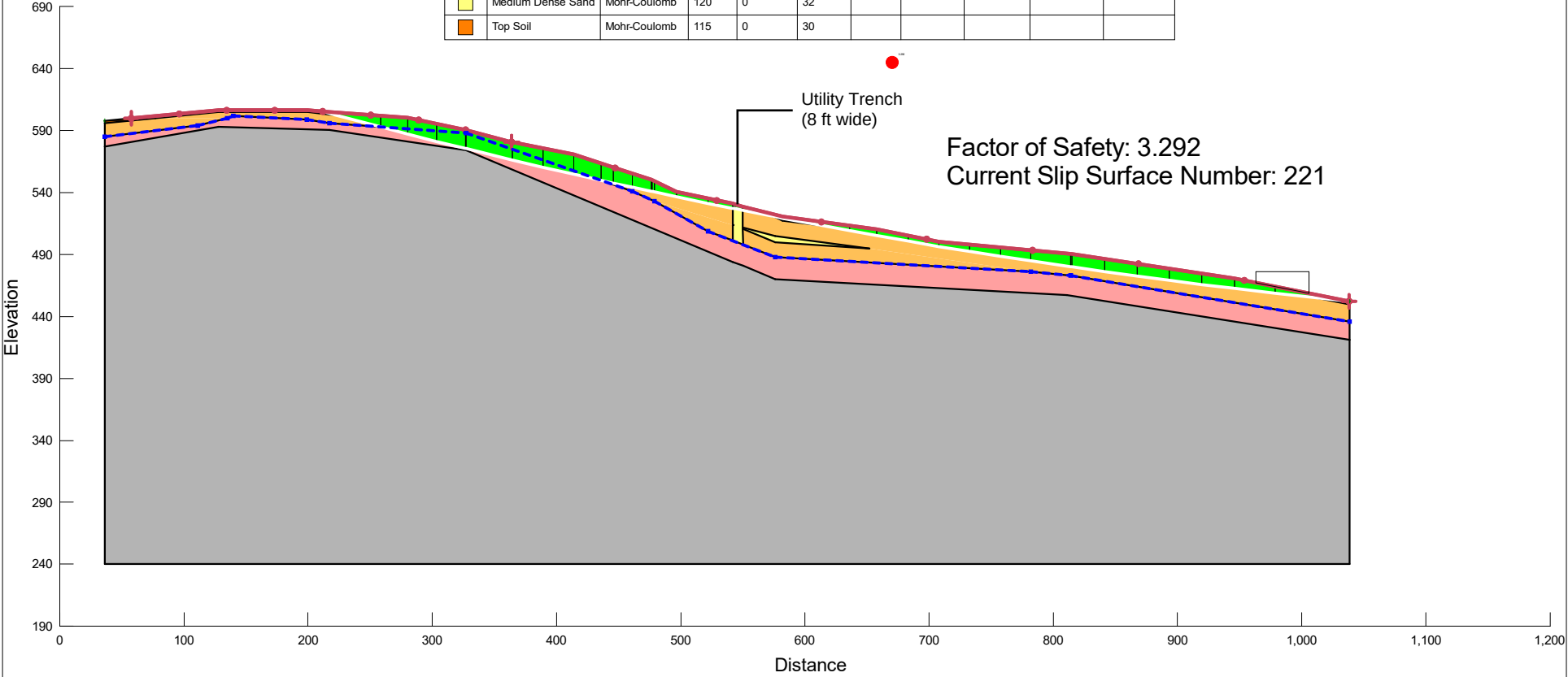
**Cross Section A_Proposed Condition
(GSI = 65)**

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	Hoek-Brown	180			323,263	17	65	1	1
Red	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
Orange	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					
Yellow	Medium Dense Sand	Mohr-Coulomb	120	0	32					
Dark Orange	Top Soil	Mohr-Coulomb	115	0	30					



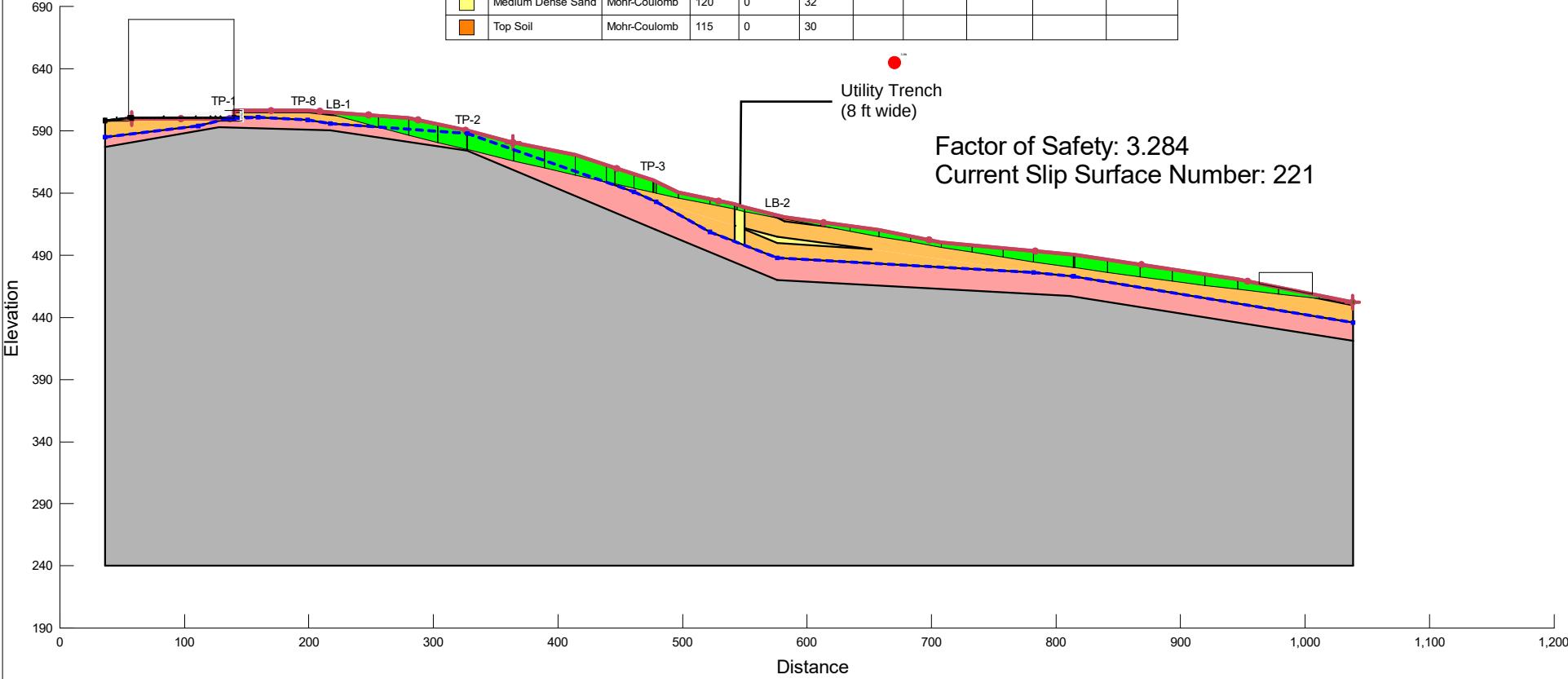
Cross Section A_Proposed Condition
(GSI = 65)

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	Hoek-Brown	180			323,263	17	50	1	1
Light Red	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
Orange	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					
Yellow	Medium Dense Sand	Mohr-Coulomb	120	0	32					
Dark Orange	Top Soil	Mohr-Coulomb	115	0	30					



Cross Section A_Existing Condition
(GSI = 50)

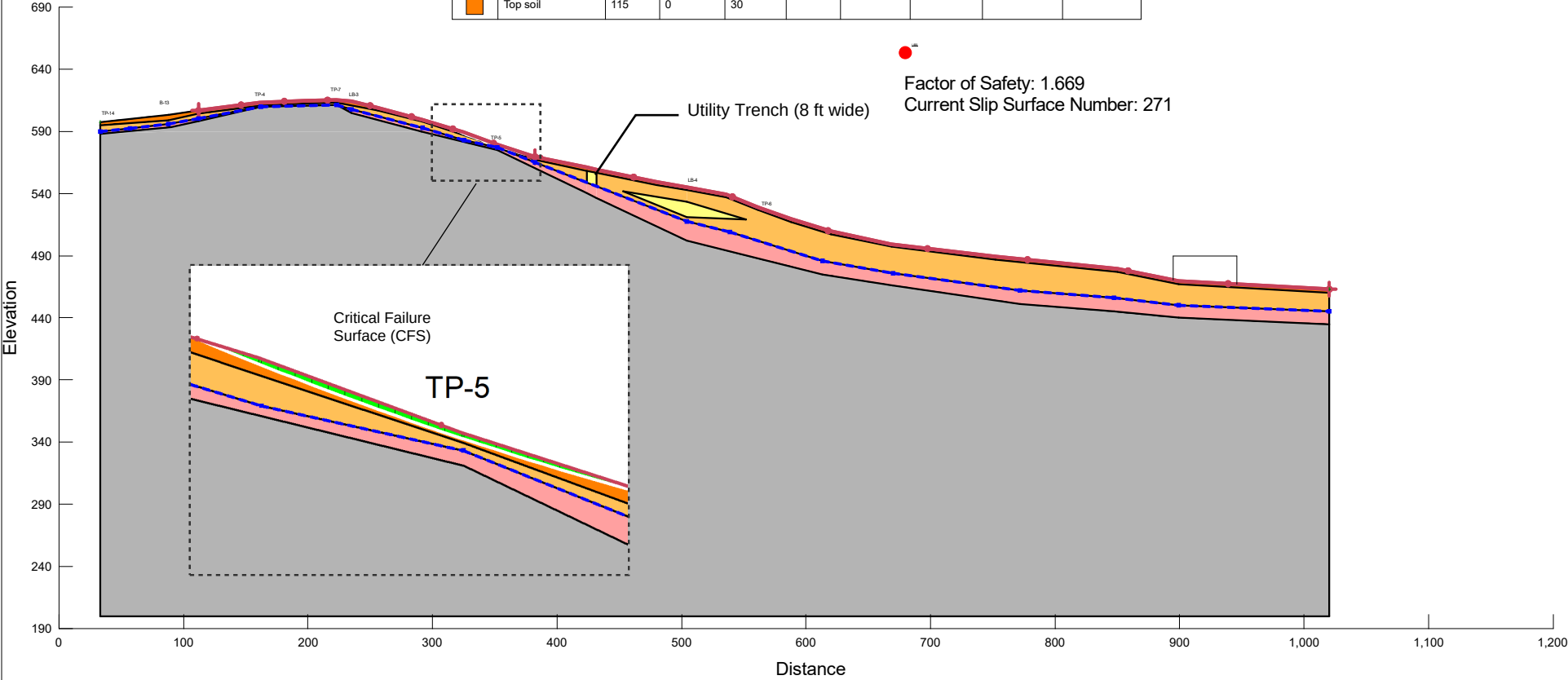
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	Hoek-Brown	180			323,263	17	50	1	1
Red	Decomposed Rock/Very Dense Granular Soil	Mohr-Coulomb	135	0	38					1
Orange	Dense to Very Dense Sand	Mohr-Coulomb	125	0	34					
Yellow	Medium Dense Sand	Mohr-Coulomb	120	0	32					
Dark Orange	Top Soil	Mohr-Coulomb	115	0	30					



Cross Section A_Proposed Condition
(GSI = 50)

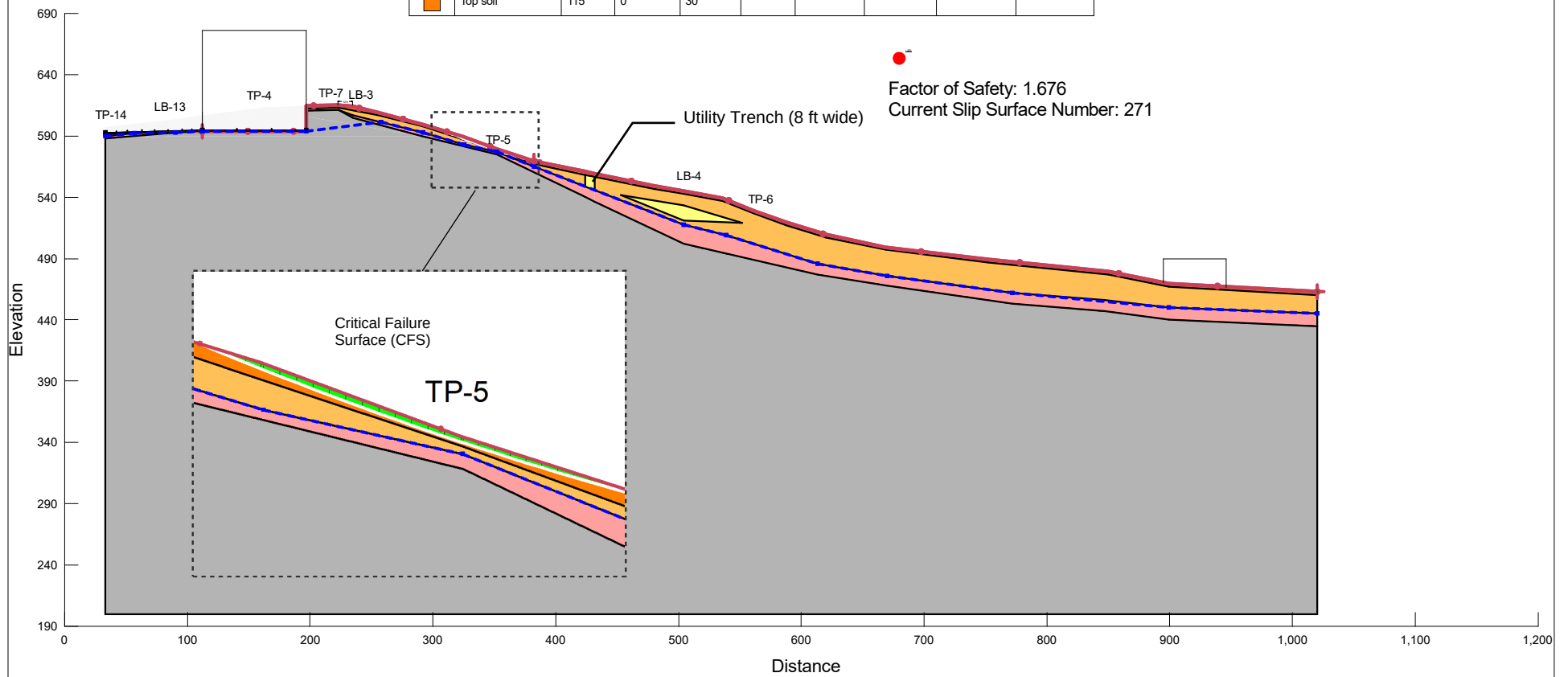
Cross Section B (Static Condition)

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	65	1	1
Red	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Orange	Dense to Very Dense Sand	125	0	34					
Yellow	Medium sand	120	0	32					
Dark Orange	Top soil	115	0	30					



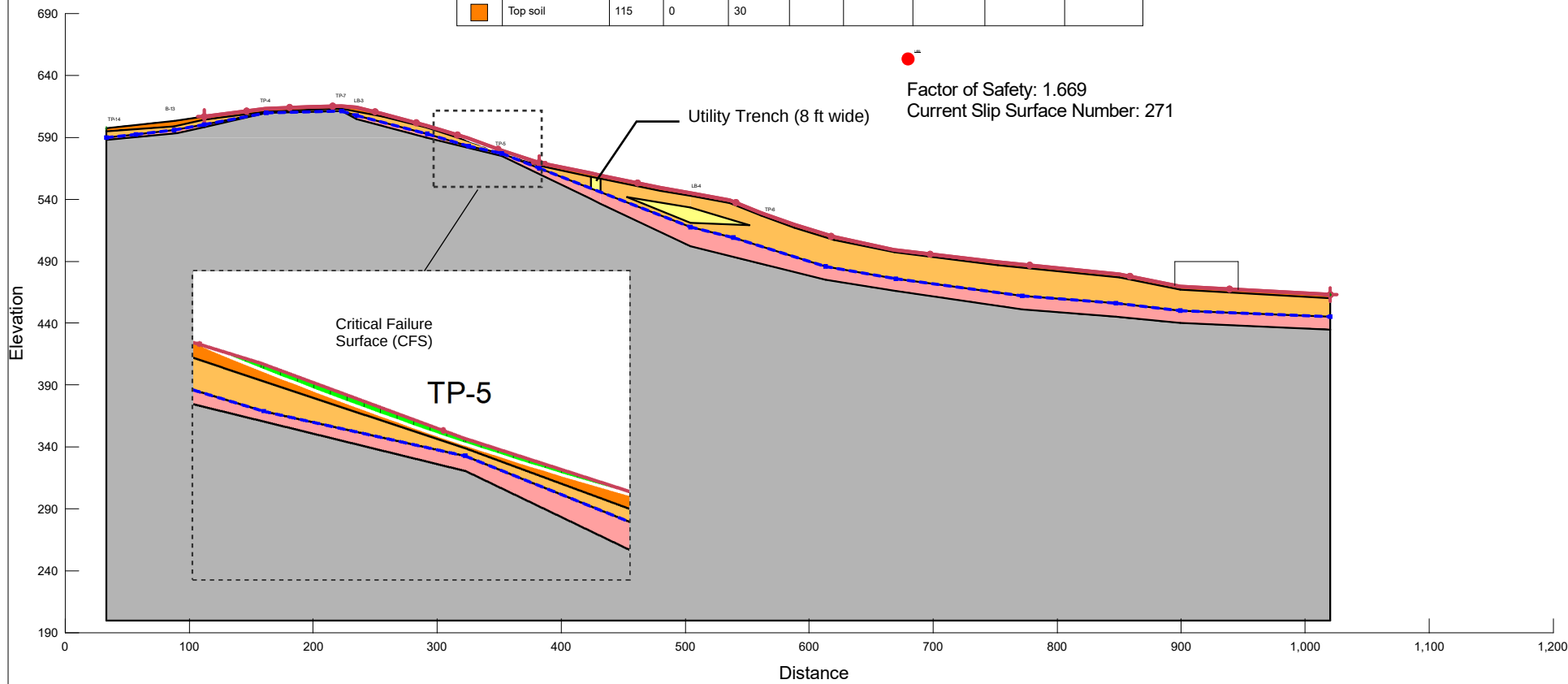
**Cross Section B_Existing Condition
(GSI = 65)**

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	65	1	1
Pink	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Orange	Dense to Very Dense Sand	125	0	34					
Yellow	Medium sand	120	0	32					
Brown	Top soil	115	0	30					



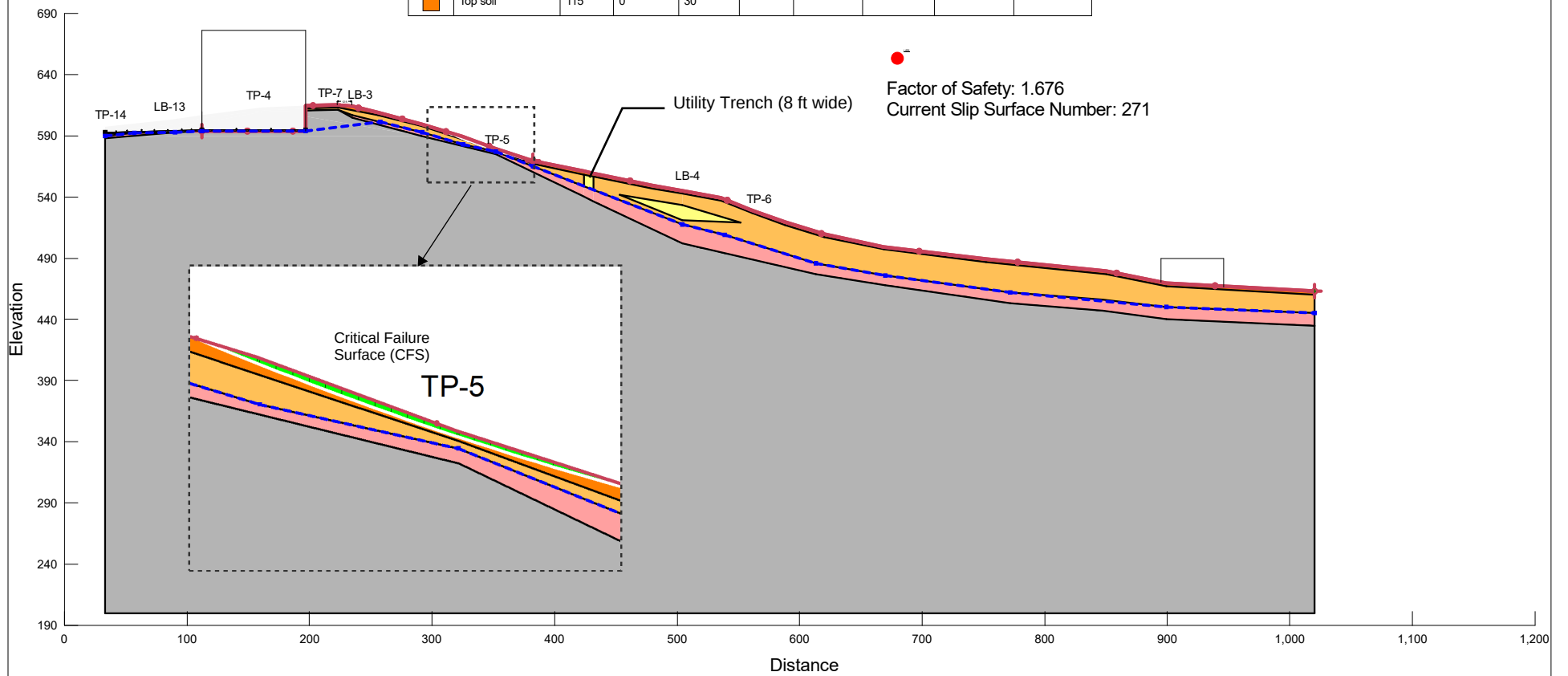
**Cross Section B_Proposed Condition
(GSI = 65)**

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Grey	Bedrock	180			323,263	17	50	1	1
Red	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Orange	Dense to Very Dens Sand	125	0	34					
Yellow	Medium sand	120	0	32					
Dark Orange	Top soil	115	0	30					



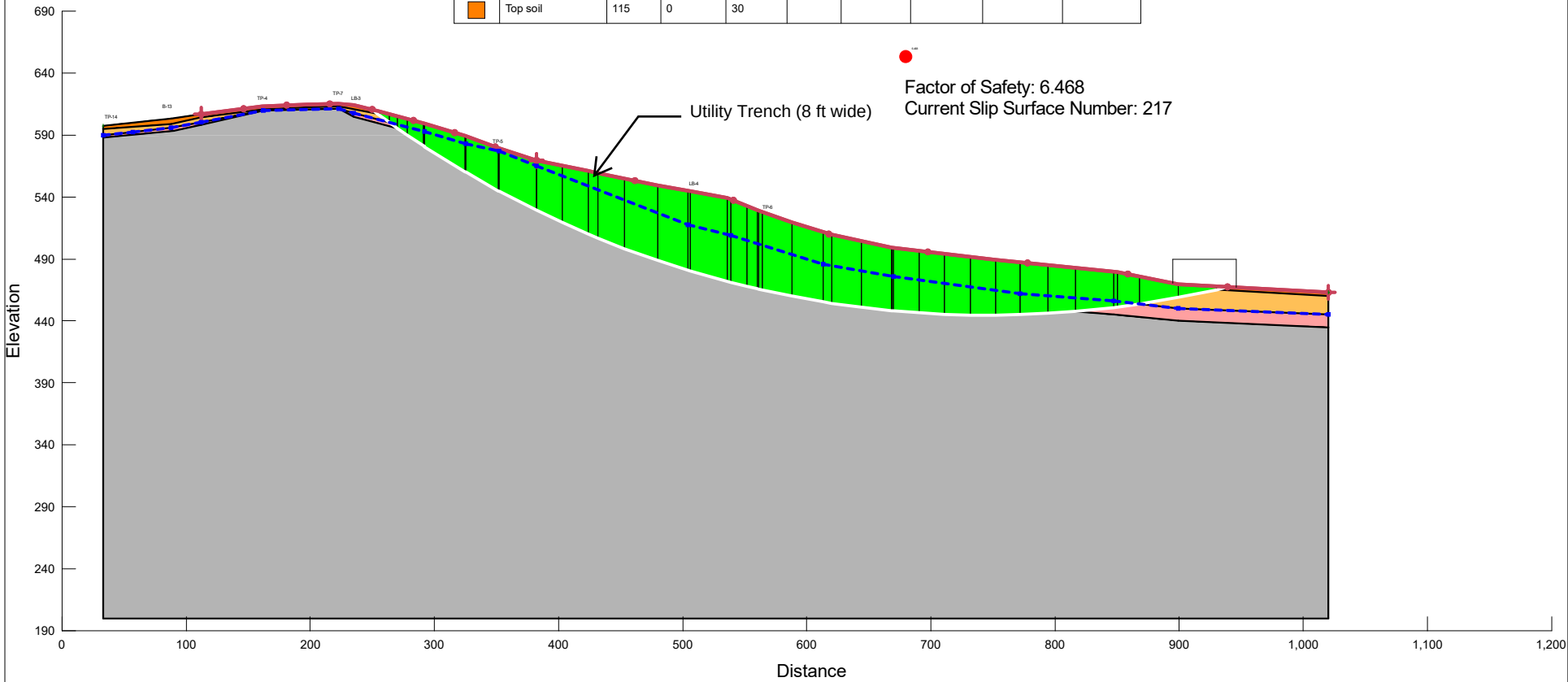
Cross Section B_Existing Condition
(GSI = 50)

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	50	1	1
Pink	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Orange	Dense to Very Dense Sand	125	0	34					
Yellow	Medium sand	120	0	32					
Dark Orange	Top soil	115	0	30					



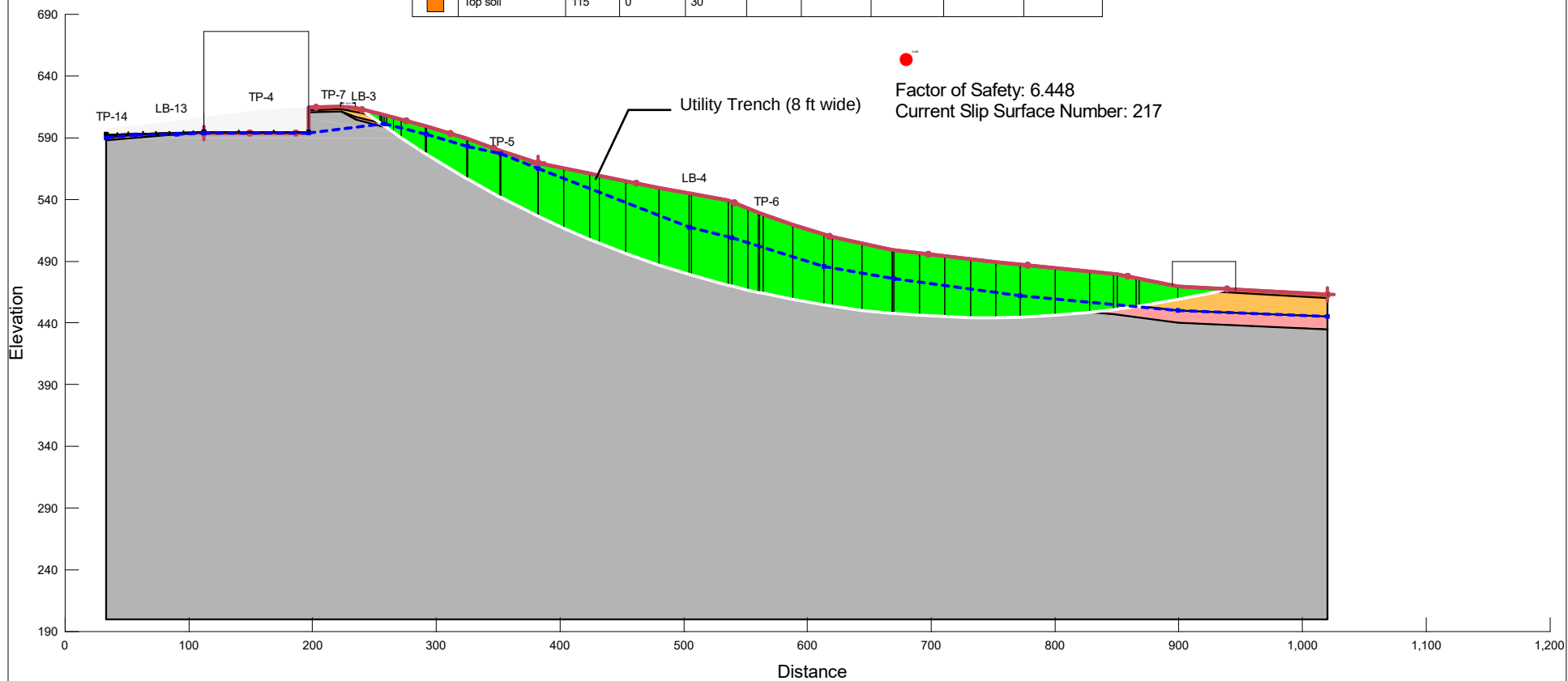
**Cross Section B_Proposed Condition
(GSI = 50)**

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
	Bedrock	180			323,263	17	65	1	1
	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
	Dense to Very Dense Sand	125	0	34					
	Medium sand	120	0	32					
	Top soil	115	0	30					



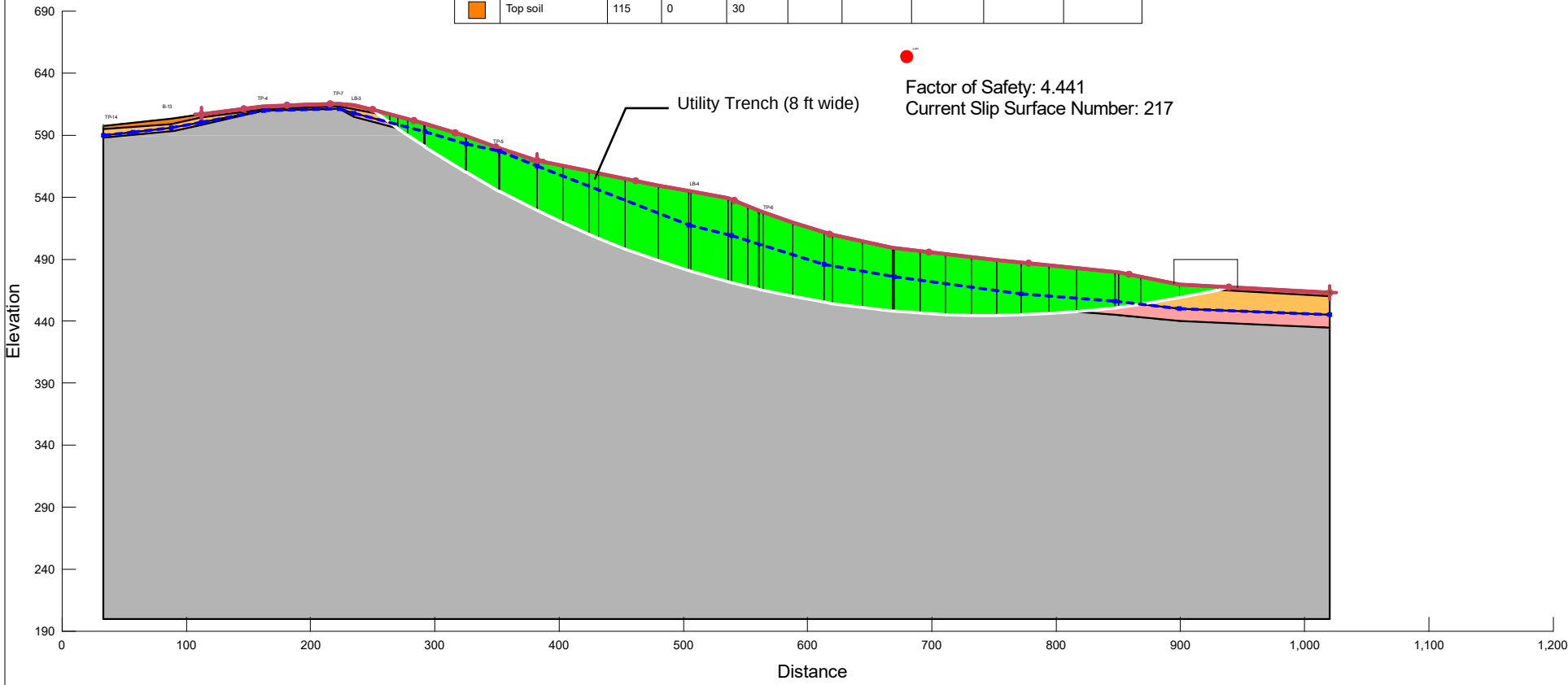
Cross Section B_Existing Condition
(GSI = 65)

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	65	1	1
Light Red	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Orange	Dense to Very Dense Sand	125	0	34					
Yellow	Medium sand	120	0	32					
Dark Orange	Top soil	115	0	30					



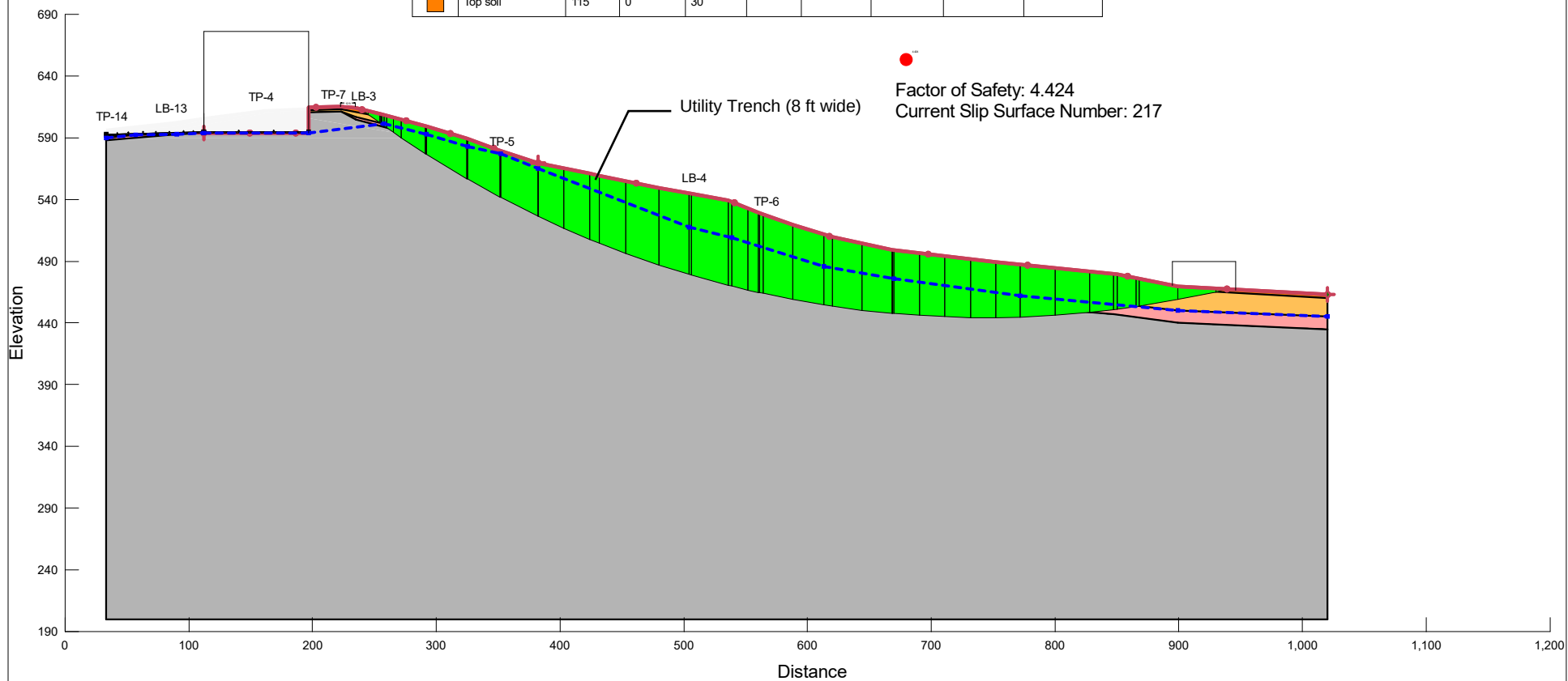
**Cross Section B_Proposed Condition
(GSI = 65)**

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	50	1	1
Red	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Orange	Dense to Very Dense Sand	125	0	34					
Yellow	Medium sand	120	0	32					
Dark Orange	Top soil	115	0	30					



Cross Section B_Existing Condition
(GSI = 50)

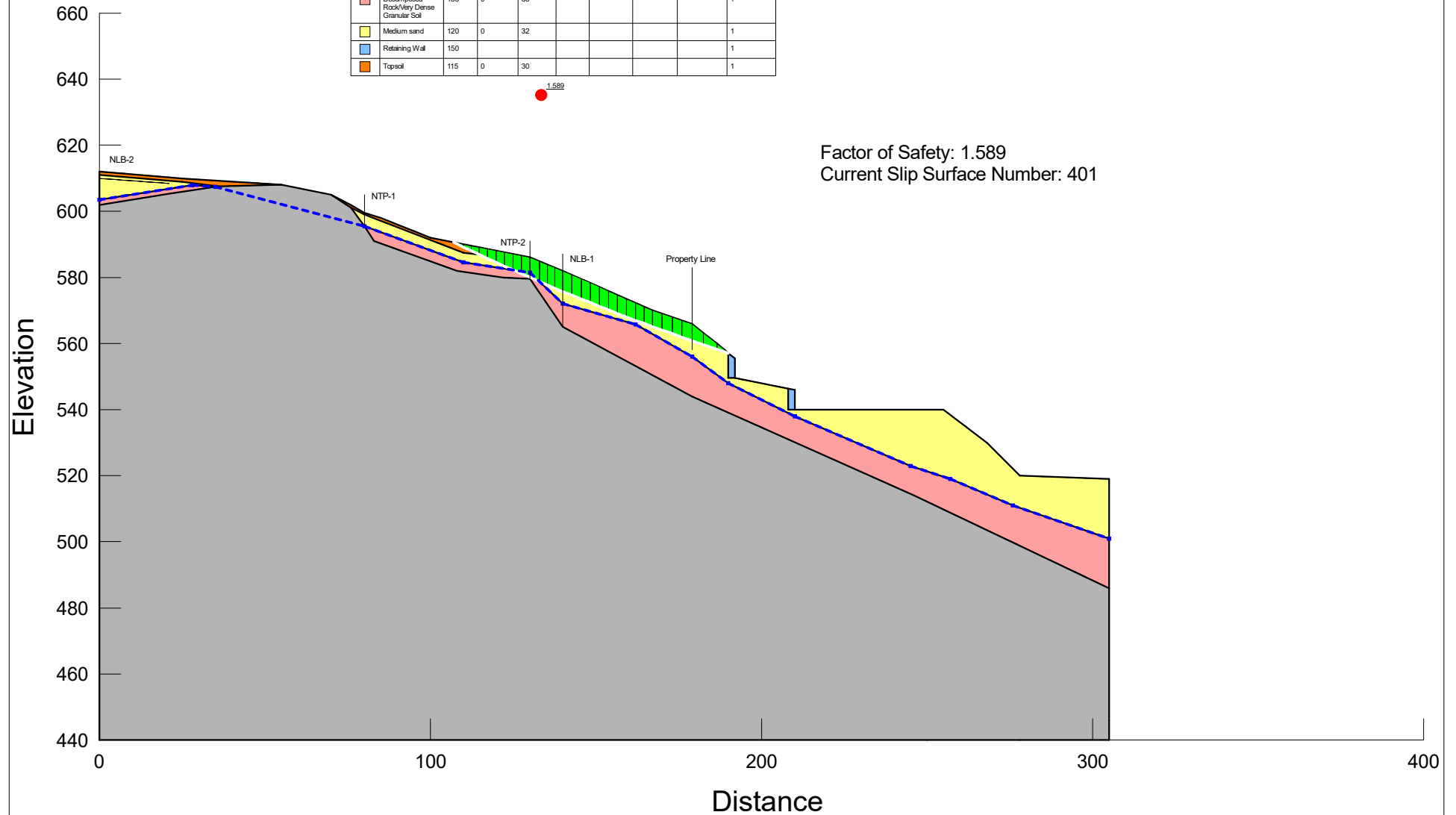
Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter mi	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	50	1	1
Light Red	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Orange	Dense to Very Dense Sand	125	0	34					
Yellow	Medium sand	120	0	32					
Dark Orange	Top soil	115	0	30					



**Cross Section B_Proposed Condition
(GSI = 65)**

Cross Section C (Static Condition)

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter m	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	37	1	
Red	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Yellow	Medium sand	120	0	32					1
Blue	Retaining Wall	150							1
Orange	Topsoil	115	0	30					1



Factor of Safety: 1.589
Current Slip Surface Number: 401

**Cross Section C_Existing Condition
(GSI = 37)**

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	UCS Intact (psf)	Intact Rock Parameter m	Geological Strength Index GSI	Disturbance Factor D	Piezometric Surface
Gray	Bedrock	180			323,263	17	37	1	1
Red	Decomposed Rock/Very Dense Granular Soil	135	0	38					1
Yellow	Medium sand	120	0	32					1
Blue	Retaining Wall	150							1
Orange	Topsoil	115	0	30					1

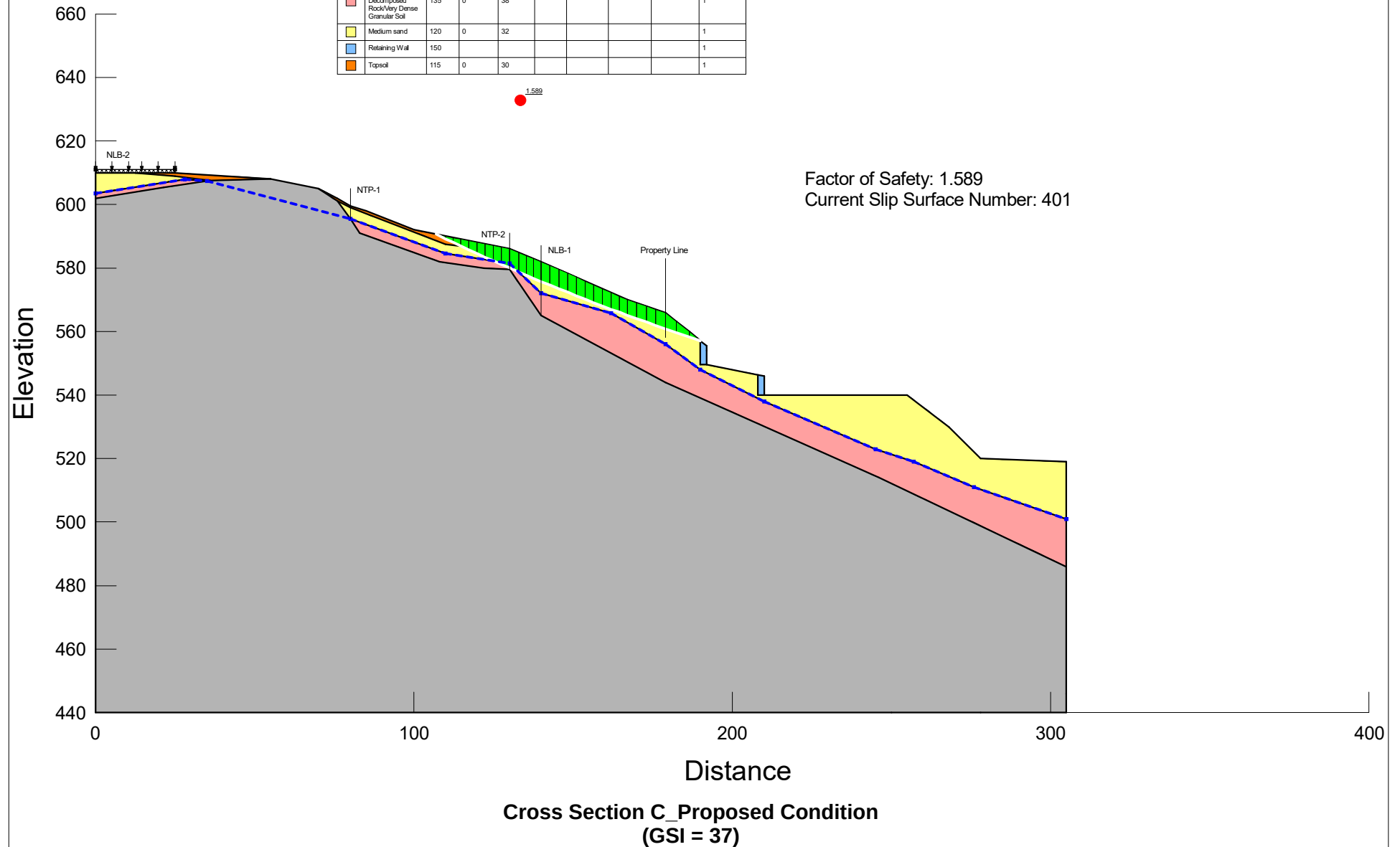


EXHIBIT C

ROCK CORE SAMPLE PHOTOGRAPHS

[illegible][illegible]

LANGAN

Exhibit C – Select Rock Core Photographs



NLB-1 and NLB-3 – Rock Core Samples



Exhibit C – Select Rock Core Photographs

NLB-2 and NLB-4 – Rock Core Samples



EXHIBIT D

SPT N-VALUE AND INTERNAL ANGLE OF FRICTION CORRELATION FOR OVERBURDEN SOILS

	SECTION A-A		AVERAGE N-VALUE (blows/ft)	ANGLE OF INTERNAL FRICTION (see note 7) (degrees)
	WEST	EAST		
	LB-1	LB-2		
	604	522		
surface elevation	604	522		
total depth (feet)	24	50		
0				
1		4	4	29
2				
3	70		70	45
4				
5				
6	40	50	45	40
7				
8	99	99	99	45
9				
10				
11		40	40	39
12				
13		78	78	45
14				
15				
16		53	53	41
17				
18				
19				
20				
21		22	22	34
22				
23				
24				
25				
26		45	45	40
27				
28				
29				
30				
31		62	62	43
32				
33				
34				
35				
36		99	99	45
37				
38				
39				
40				
41		99	99	45
42				
43				
44				
45				
46			65	43
47				
48				
49				
50				
51				
52				

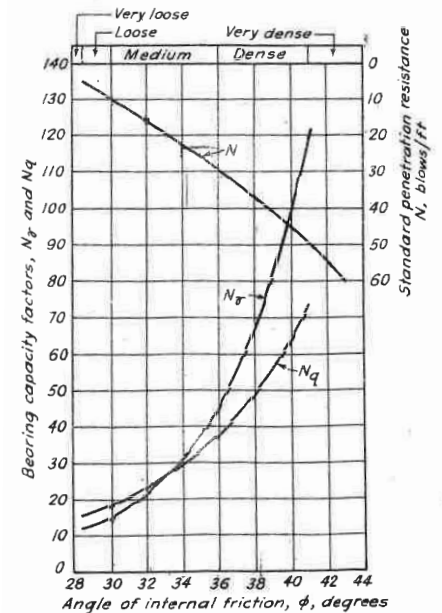


FIGURE 19.5. Curves showing the relationship between bearing-capacity factors and ϕ , as determined by theory, and rough empirical relationship between bearing capacity factors or ϕ and values of standard penetration resistance N_s .

NOTES

- 1 Subsurface information provided is generalized and is shown for illustration purposes only.
- 2 Refer to location plan for actual locations.
- 3 Refer to logs for actual soil descriptions and details.
- 4 The N-values tabulated are in blows/ft.
- 5 Split spoon refusal values are represented by 99 blows/ft.
- 6 Split spoon refusal values in the decomposed rock layer are excluded in this table.
- 7 The corresponding angle of internal friction was taken from Figure 19.5 of the "Foundation Engineering" (second edition) written by Ralph B. Peck, Walter E. Hanson and Thomas H. Thornburn

	SECTION B-B		AVERAGE N-VALUE (blows/ft)	ANGLE OF INTERNAL FRICTION (see note 7) (degrees)
	WEST	EAST		
	LB-3	LB-4		
surface elevation	614	548		
total depth (feet)	19	45		
0				
1				
2	5	5	5	29
3				
4	53	47	50	41
5				
6		23	23	34
7	35		35	37
8		56	56	39
9				
10				
11		99	99	45
12				
13		52	52	39
14				
15				
16		22	22	34
17				
18				
19				
20				
21				
22				
23				
24				
25		21	21	34
26				
27				
28				
29				
30				
31		99	99	45
32				
33				
34				
35				
36		99	99	45
37				
38				
39				
40				
41		99	99	45
42				
43				
44				
45				
46			60	43
47				

DEPTH FROM SURFACE GRADE (FT)

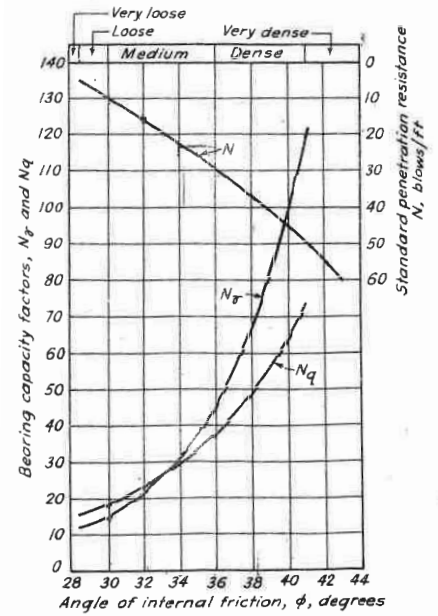


FIGURE 19.5. Curves showing the relationship between bearing-capacity factors and ϕ , as determined by theory, and rough empirical relationship between bearing capacity factors or ϕ and values of standard penetration resistance N_s .

NOTES

- 1 Subsurface information provided is generalized and is shown for illustration purposes only.
- 2 Refer to location plan for actual locations.
- 3 Refer to logs for actual soil descriptions and details.
- 4 The N-values tabulated are in blows/ft.
- 5 Split spoon refusal values are represented by 99 blows/ft
- 6 Split spoon refusal values in the decomposed rock layer are excluded in this table.
- 7 The corresponding angle of internal friction was taken from Figure 19.5 of the "Foundation Engineering" (second edition) written by Ralph B. Peck, Walter E. Hanson and Thomas H. Thornburn

	SECTION C-C			AVERAGE N-VALUE (blows/ft)	ANGLE OF INTERNAL FRICTION (see note 7) (degrees)
	WEST		EAST		
	NLB-2		NLB-1		
	612		582		
surface elevation	20		26		
total depth (feet)					
0					
1	4		6	5	29
2					
3	18		56	37	37
4					
5	29		27	28	35
6					
7	25		65	45	40
8					
9	25		99	62	43
10					
11				43	39
12					
13					
14					
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20					
21					
22					
23					
24					
25					
26					
27					
28					

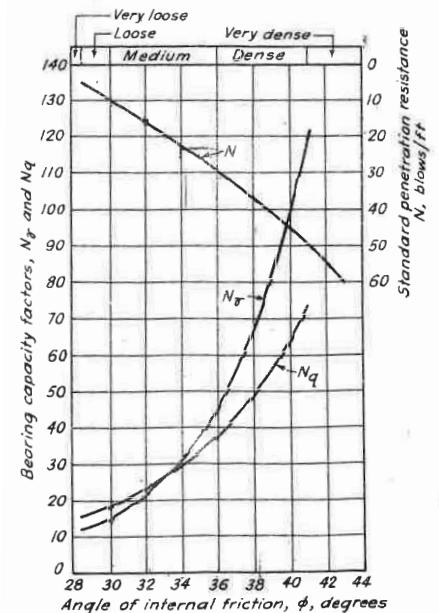


FIGURE 19.5. Curves showing the relationship between bearing-capacity factors and ϕ , as determined by theory, and rough empirical relationship between bearing capacity factors or ϕ and values of standard penetration resistance N .

NOTES

- 1 Subsurface information provided is generalized and is shown for illustration purposes only.
- 2 Refer to location plan for actual locations.
- 3 Refer to logs for actual soil descriptions and details.
- 4 The N-values tabulated are in blows/ft.
- 5 Split spoon refusal values are represented by 99 blows/ft
- 6 Split spoon refusal values in the decomposed rock layer are excluded in this table.
- 7 The corresponding angle of internal friction was taken from Figure 19.5 of the "Foundation Engineering" (second edition) written by Ralph B. Peck, Walter E. Hanson and Thomas H. Thornburn

	SECTION D-D		AVERAGE N-VALUE (blows/ft)	ANGLE OF INTERNAL FRICTION (see note 7) (degrees)
	WEST	EAST		
	NLB-4	NLB-3		
	609	566		
surface elevation	22	32		
total depth (feet)				
0				
1	2	6	4	29
2				
3	20	9	15	32
4				
5	16	11	14	32
6				
7	99	35	67	45
8				
9		50	50	41
10				
11	99	99	99	45
12				
13				
14				
15				
16		43	43	39
17				
18				
19				
20				
21		99	99	45
22				
23			72	45
24				
25				
26				
27				
28				
29				
30				
31				
32				
33				

DEPTH FROM SURFACE GRADE (FT)

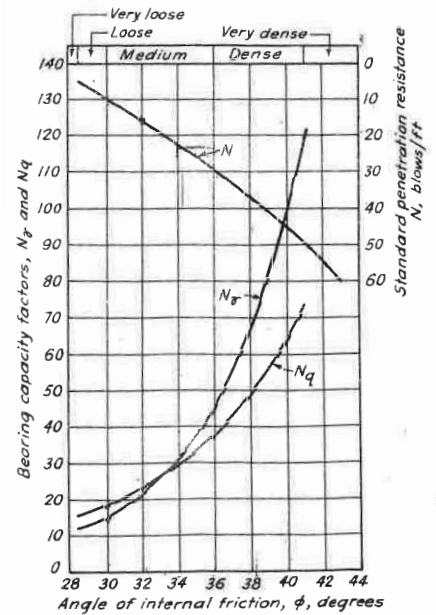


FIGURE 19.5. Curves showing the relationship between bearing-capacity factors and ϕ , as determined by theory, and rough empirical relationship between bearing capacity factors or ϕ and values of standard penetration resistance N_s .

NOTES

- 1 Subsurface information provided is generalized and is shown for illustration purposes only.
- 2 Refer to location plan for actual locations.
- 3 Refer to logs for actual soil descriptions and details.
- 4 The N-values tabulated are in blows/ft.
- 5 Split spoon refusal values are represented by 99 blows/ft
- 6 Split spoon refusal values in the decomposed rock layer are excluded in this table.
- 7 The corresponding angle of internal friction was taken from Figure 19.5 of the "Foundation Engineering" (second edition) written by Ralph B. Peck, Walter E. Hanson and Thomas H. Thornburn