

Dr. Robert Bagoff, Chair
West Orange Township Planning Board
West Orange Township
66 Main Street
West Orange, NJ 07052

July 16, 2025

RE: Review of Langan Slope Stability Analysis, January 14, 2025, revised June 18, 2025
PB-24-01 West Essex Highlands, Inc
Block: 179 Lot: 32 Zone: IH-1
Warner Road
Project 2151.002

Dear Chair Bagoff,

I have been tasked with reviewing the above referenced executive summary and letter report and the basis of analysis prepared by Langan, dated January 14, 2025, last revised June 18, 2025. As will be discussed, the slope stability analysis is flawed, specifically not reflecting the actual conditions on the eastern slope of the site. In fact, the analysis picked the data obtained from the soil borings with the most competent parameters, when, in fact, the areas most vulnerable to initiating slope failure, the toe or bottom of the steep slope, especially in the northern area of the slopes analyzed found rock quality as "very poor"¹. This review is focused on this eastern area of the site, the location of proposed Building "D".

Building Layout Currently Proposed Closer to Slope than Concept

As we understand, the Settlement of Litigation Agreement (Settlement Agreement), signed by Mark Hoffman on April 23, 2020, an application before the Planning Board must be developed "...consistent with the 'West Essex Highlands Concept Plan 16 for West Essex Highlands, Block 179, Lot 32, Township of West Orange, Essex County, New Jersey' dated March 3, 2020 prepared by Anderson Consulting Services, LLC (the 'Concept Plan')"².

A review of the building layout provided as Figure 1 in the Langan report reveals that Proposed Building 'D' is 50 feet closer (east) to the slope, and atop the knob on the eastern end of the building and over 30 feet closer (east) to the slope on the western end of the building; in addition to the change in building layout. The relocation of this building adds additional risk to the stability of the slope in question.

Parameters of Slope Stability Analysis Do Not Match Actual Conditions

We reviewed the approach and parameters to the slope stability analysis and found significant inconsistencies with the data obtained in the field, as well as mapped geologic conditions. As illustrated in the table below, the borings identified in red were the ones chosen to include as the chosen parameters, with 67% RQD chosen to develop the strength of the overall rock.

¹ An actual designation of the quality of bedrock as defined by ASTM D603 and US Army Corps of Engineers Engineering Manual, EM-1110-1-1804.

² Page 2 of the Settlement Agreement.



Boring	Cross Section	Slope Location	Sample/ Depth (ft)	REC %	RQD %
LB-1	A	Upper	C-1 / 14-19 C-2 / 19-24	97 97	58 82
LB-2	A	Middle	n/a	n/a	n/a
LB-3	B	Upper	C-1 / 9-14 C-2 / 14-19	93 87	77 67
LB-4	B	Middle	n/a	n/a	n/a
Average (LB)				94	71
Highest Value (LB)				97	82
Lowest Value (LB)				87	58
NLB-1	C	Middle	C-1 / 17-22 C-2 / 22-26	78 100	0 23
NLB-2	C	Upper	C-1 / 10-15 C-2 / 15-20	100 93	72 93
NLB-3	D	Middle	C-1 / 22-27 C-2 / 27-32	100 75	42 48
NLB-4	D	Upper	C-2 / 12-17 C-3 / 17-22	95 97	52 83
Average (NLB)				92	52
Highest Value (NLB)				100	93
Lowest Value (NLB)				75	0
Average (LB and NLB)				92	58
Highest Value (LB and NLB)				100	93
Lowest Value (LB and NLB)				75	0

- As stated above, the rock core samples obtained from the boring investigation used two rock samples to test for compressive strength from the most competent rock found above the top of the subject slope. For example, the cores chosen had RQD values of 77% and 67%, which are greater than the average values, and significantly greater than the lowest values obtained from the site. And, when tested for compressive strength, even the values obtained were found to have strengths indicative of a lower strength type rock than basalt. As stated in the subject report (pdf page 17),

Unconfined compressive tests were performed on 2 rock core samples taken from boring LB-3 located at the upper portion of the slope. The tests suggest that the unconfined compressive strength of the basalt rock samples varied between approximately 2,200 pounds per square inch (psi) and 3,500 psi or approximately 316,800 pounds per square foot (psf) and 504,000 psf.

These values equate to 15 Megapascals (MPa) and 24 MPa, respectively. When compared to known strengths of rock material, as shown in the table below (table source from Rockscience slope stability program), the material has more of a quality of chalk, rocksalt, or potash material. This is an indication that the basalt is significantly weathered, even in the intact portions of the rock than typically found. Typical un-weathered basalts are normally almost 10 times the values obtained from the testing of the specimens chosen.

Intact Uniaxial Compressive Strength		
Field Estimate of Strength	Examples	Strength (MPa)
Specimen can only be chipped with a geological hammer.	Fresh basalt, chert, diabase, gneiss, granite, quartzite.	>250
Specimen requires many blows of a geological hammer to fracture it.	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff.	100-250
Specimen requires more than one blow of a geological hammer to fracture it.	Limestone, marble, phyllite, sandstone, schist, shale.	50-100
Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer.	Claystone, coal, concrete, schist, shale, siltstone.	25-50
Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer.	Chalk, rocksalt, potash.	5-25
Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife.	Highly weathered or altered rock.	1-5
Indented by thumbnail.	Stiff fault gouge.	0.25-1

- The model sections provided assumed the highest RQD values for the entire slope section in every scenario tested, instead of using the actual RQDs obtained from the vicinity of each boring. For example, at modeled Section C-C that contains one of the steepest sections of the slope, and immediately above 15 Howell Drive, the RQD values in boring NLB-1, were 0% and 23", not including the fragmented rock above the start of the cores, that indicate a very weak rock mass, significantly weathered and susceptible to slides. And for the borings at LB-2 and LB-4 obtained at the middle of the slope, the average RQDs for this portion of the would be less than 23%³ which would lower the GSI value from the 67 chosen by Langan to 15 to 20, reflecting poor and very poor joint conditions, common in heavily fractured rock.
- Of particular concern is the use of a relatively high Geological Strength Index (GSI) in the slope stability model. While the model uses a Disturbance Factor (DF) of 1.0, which indicates a site that is disturbed by construction activity, it is the combination with an appropriate GSI factor that will truly dictate the overall stability of the slope. As illustrated in borings NLB-1, NLB-3, LB-2, and LB-4, the RQDs average
- While Langan stated they used a uniform loading of 1,000 psf for the model (page 8 of 13 of the Langan report), they dismissed proposed Building D's loading on the top of the slope, as they considered it "far enough from the top of the slope" to have surcharge impact (page 3 of 5 of the report), however the proposed building sits at the top of the slope, indicating that there would be an impact to loading on the slope. Langan did not discuss or model the impact of rock excavation to achieve the below-building garage (up to 25 feet of excavation, requiring rock excavation or blasting) subgrades on the stability of the slope.

³ due to the highly decomposed nature of the rock in borings LB-2 and LB-4, these borings should use a RQD of 0%.

5. The model omitted the representation of the Feltville Formation, defined by the geologic map (Figure 2 of the Langan report). It is notable that contacts between rock formations (such as between the Preakness and Feltville Formations), are generally the location of significant access of surface water to the subsurface and resulting weathering at depth. Therefore, the marginal quality of the rock, such as in borings NLB-1 (RQDs 0% and 23%), NLB-3 (RQDs 42% and 48%), LB-2 (no RQDs due to deteriorated nature of rock⁴), and LB-4 (no RQDs due to deteriorated nature of rock⁵).
6. The analysis did not include the discontinuity in the subsurface cross section that results from the existence of the wastewater utility line that runs parallel to and at the toe of the steepest portion of the subject slopes. This utility, based on review of historic aerial photographs, was constructed in 1995 or earlier. Such a break in the subsurface conditions of the slope will have an impact and add to the probable failure mechanisms of the slope.
7. The depth of decomposed rock of a maximum of seven (7) feet is underestimated based on the extent of the depth of decomposed rock found in NLB-1, NLB-3, LB-2, and LB-4.

Geologic Hazard Review Unjustified in Dismissing the Risks

Langan dismissed the risk associated with the Landslide Class B III localized area directly below the building and on the slope due to their observation that the “typically favorable joint orientation in the outcrops”. It is our interpretation that their observation that the rock fracturing is tilted into the slope and therefore, would not be a hazard. However, based on the results of the borings and test pits completed for the site, on the downslope areas of the subject slope under analysis, the bedrock was found to have RQDs equivalent of “Fair” to “Poor”, based on the RQD analysis method, these slopes would, indeed, be susceptible to landslides. Below is a table of the classification of Landslide Susceptibility of Geologic Groups. A review of the soil borings with highly fractured and decomposed rock in the downhill formation of the subject slopes, it is highly probable that the rock acts as “weakly cemented” and does have a moderate chance of slope failure. And, as the application is proposing to impose significant excavations and likely, blasting to achieve the necessary garage subgrade elevation of proposed Building D, this will further increase the probability of landslide activity in this location.

Blasting Implications and Threats to Stability of the Slope Must be Addressed Before Approval

Unless the Applicant provides sufficient information and defined specifications, simply adhering to “State regulations” as stated at the Planning Board meeting of June 30, such regulations only apply to the safety of conducting the blasting activities, not for the long-term stability and potential landslide potential that may occur because of changes to the bedrock condition adjacent to the site. Any rock stabilization and long-term stability is governed in NJ by the International Building Code, as adopted via NJAC 7:23, which requires that all site grading, excavation and foundation systems be designed based on property geotechnical investigation and engineering principles. A condition of approval stating that the Applicant will comply with the State regulations for blasting will not absolve the Applicant or Township from liability associated with the short and long term effects of blasting, or development, for that matter without blasting on the requirement to ensure that an approved project protects the health, safety, and welfare of the public from the result of landslides, rockfalls, and other related slope failures.

⁴ No RQDs as the rock was so deteriorated and fragmented as cores could not be progressed.

⁵ Same as previous footnote.

Table 6-4: Landslide Susceptibility of Geologic Groups

Geologic Group		Landslide Susceptibility* (Slope Angle, degrees)					
		0–10	10–15	15–20	20–30	30–40	>40
DRY (groundwater below level of sliding)							
A	Strongly Cemented Rocks (crystalline rocks and well-cemented sandstone)	None	None	I	II	IV	VI
B	Weakly Cemented Rocks and Soils (sandy soils and poorly cemented sandstone)	None	III	IV	V	VI	VII
C	Argillaceous Rocks (shales, clayey soil, existing landslides, and poorly compacted fills)	V	VI	VII	IX	IX	IX
WET (groundwater level at ground surface)							
A	Strongly Cemented Rocks (crystalline rocks and well-cemented sandstone)	None	III	VI	VII	VIII	VIII
B	Weakly Cemented Rocks and Soils (sandy soils and poorly cemented sandstone)	V	VIII	IX	IX	IX	X
C	Argillaceous Rocks (shales, clayey soil, existing landslides, and poorly compacted fills)	VII	IX	X	X	X	X

* Landslide susceptibility is measured on a scale of I to X, with X being the most susceptible

In conclusion, the recent slope stability analysis is inadequate to assess the conditions of the subject slope at proposed Building. The parameters used only reflect the most competent locations above the top of the slope of the investigation and not the degraded bedrock conditions found in the middle slopes on the site. By also leaving out the impact of the subsurface wastewater utility line and the less competent Feltville Formation provides an unrealistic evaluation of the site and provides a false sense of security that the subject proposed Building D can be constructed without initiating landslide activity.

I will be available for the July 8 16, 2025 Planning Board meeting to discussion my observations and conclusions.

Thank you.

Sincerely,



Geoffrey M. Goll, P.E.
President
Princeton Hydro, LLC

cc: WeCare