APPENDIX I-3

CORRESPONDENCE BETWEEN AECOM AND DYNAMIC EARTH

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Memorandum

Date:	10/23/19
To:	IHCC Files
From:	David M. Cregger, P.E., P.G.
Subject:	Review of Appendix H

AECOM responds by this letter to a request from Town of Huntington for Geotechnical Engineering Services for a Review of Slope Stability Evaluation Report Related to Indian Hills Country Club. The scope of work includes Review & Evaluation of Appendix H – Geotechnical Engineering Reports and Correspondence. We received 3 documents for review:

1. Appendix H-1 Geotechnical Engineering Services Report, Phase I of the Bluff Area Stability Evaluation, PS&S, dated July 25, 2008 *(does not include Appendix)*
2. Appendix H-2 Geotechnical Engineering Investigation and Slope Stability Analysis, PS&S, dated April 15, 2019 (with associated Appendices A&B)
3. Appendix H-3 Dynamic Earth Correspondence, dated July 8, 2019

<u>Methodology Used by PS&S</u>

Document H-1, also known as Phase I, appears to have been done in a competent manner in general conformance to geotechnical practice where test borings and detailed observations have yet to be made (see detailed comments attached).

However, document H-2 appears to ignore the original plan for the investigation in a rush to complete the borings without senior level oversight which then impacted the ability to complete a credible slope evaluation (see detailed comments attached). We offer this opinion based on several deficiencies:

- Test borings were drilled with drilling mud which does not allow for the collection of groundwater table data,
- Separate groundwater observation wells were not installed by means of either hollow stem augers or with bio-degradable drilling mud,
- Continuous samples of the clay were not made to identify potential horizon of sliding,
- Undisturbed samples of the clay were not taken within the clay zone to determine the shear strength of the clay by means of laboratory testing,
- In-situ vane shear tests or pocket penetrometer tests on clay samples were not taken to determine shear strength, and
- Inadequate laboratory testing was completed.

These field investigation deficiencies then led to improper office evaluations including:

- Unjustified selection of shear strengths of the clay based on empirical textbook relations rather than development of site-specific properties formulated by lab and/or field testing,

- Selection of a sliding surface at the base of the clay zone forcing the defined sliding without evidence of shear failure at that level,
- Assuming a groundwater table at sea level across the entire site resulting in heavier soil loads on the slip surface increasing resistance incorrectly, and
- Assuming an unacceptable Factor of Safety (FS) equal to 1.0 for slopes.

Our professional opinion is that these results do not follow conventional practice.

<u>Recommendation of the 120-ft Buffer Zone</u>
Our professional opinion, for the reasons cited above, is that the 120-ft buffer is inadequate at this time in a known landslide zone (see Figure 1 below). GeoStudio software of SEEP/W and SLOPE/W is appropriate for this work because it allows a seepage analysis and the application of the surcharge can be set back beyond the worst case slip circle. We would recommend that an FS=1.3 or greater be maintained to develop the setback.

Due to the nature of the historical sliding in the area, it will be necessary to develop soil shear strengths based on residual soil parameters. It may be possible to use the existing soil samples, if they were saved in jars, to prepare remolded samples in the laboratory. Simple direct shear testing of remolded samples may provide appropriate parameters if it can be determined what the depth of sliding may be for appropriate normal loads during the test. Triaxial testing of remolded samples may also provide useful results, but may be more expensive.

At least one extra boring upslope of the failure zone should be made with hollow stem augers to determine a reasonable groundwater level for the slope stability analyses. It would also be prudent to take a second boring or test pit through the land sliding mass just below the existing scarp to see if the depth of sliding can be determined to verify the slope stability model.

AECOM is pleased to provide these recommendations and looks forward to answering any further questions you may have.

Figure 1. Google Earth Photograph Circa 2012 with Interpreted Landslide Scarps, Tension Cracks, Approximate Boring Locations, and Presumed Slope Stability Profile Location

PEER REVIEW COMMENT AND RESOLUTION FORM

PROJECT NO.: Town of Huntington, Indian Hills Country Club

DESCRIPTION: Geotechnical Engineering Service for Review of Slope Stability Evaluation Report

DESIGNER: Paulus, Sokolowski and Sartor Engineering, P.C.

SUBMITTAL: Appendix H1 - Geotechnical Engineering Reports and Correspondence, July 25, 2008

DATE: October 17, 2019

REVIEW SECTION: Geotechnical

REVIEWER NAME: DMCregger

NO.	SHEET OR ITEM	COMMENT	INITIAL ACTION	RESPONSE	QC REVIEW INITIAL	FINAL ACTION VERIFIED
COMPLETED BY REVIEWER			COMPLETED BY DESIGNER			BY REVIEWER
1	p.4	April 2001 photo shows caisson retaining wall at northeast end of site and a groin at the property line; stone riprap is not considered to be a wall, but does show up in 2004 air photos.				
2	p.5	Phase I tasks appear to be reasonable and non-invasive				
3	p.5	Phase II tasks are more invasive, but appear reasonable				
4	p.7	good historical reference and reporting				
5	p.7	17 feet of movement 2002-2008 should be shown on map				
6	p.8	there are existing borings; depth to clay is not stated				
7	p.8	good correlation to rainfall				
8	p.10	good correlation to groundwater				
9	p.10	more evaluation of drywell impact could have been made to elaborate on how the accelerated movements are documented				
10	p.12	recommendation of 2 borings and 2 inclinometers and 2 piezometers apperas to be reasonable				
11	p.13	figures were not attached				
12						
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21						

"SHEET OR ITEM" PREFIX FOR COMMENT NO'S - PLANS =P, SPEC. PROVS=S, EST.=E, CALC BOOK=C, DESIGN REPORT=D, OTHER=O
"ACTION" A=WILL INCORPORATE, B=WILL EVALUATE, C=DELETE COMMENT

PEER REVIEW COMMENT AND RESOLUTION FORM

PROJECT NO.: Town of Huntington, Indian Hills Country Club

DESCRIPTION: Geotechnical Engineering Service for Review of Slope Stability Evaluation Report

DESIGNER: Paulus, Sokolowski and Sartor Engineering, P.C.

SUBMITTAL: Appendix H-2 - Geotechnical Engineering Reports and Correspondence, April 15, 2019

REVIEW SECTION: Geotechnical

AECOM

DATE: October 17, 2019

REVIEWER NAME: DMCregger

NO.	SHEET OR ITEM	COMMENT	INITIAL ACTION	RESPONSE	QC REVIEW INITIAL	FINAL ACTION VERIFIED
COMPLETED BY REVIEWER			COMPLETED BY DESIGNER			BY REVIEWER
1	p.1	is the date of this appendix correct?				
2	p.2	boring samples at 5-ft intervals was proposed to be continuous for delineation of the clay layer as stated in H-1				
3	p.3	what is thickness of the clay stratum				
4	p.4	groundwater readings were not obtained?				
5	p.4	FS should be > 1.3				
6	p.5	300 psf surcharge is a reasonable design				
7	p.6	is section 1-1 the SLOPE/W stability profile?				
8	p.9	what is the recovery of the sample?				
9	p.9	inspector should describe plasticity and stiffness of the clay				
10	p.10	what is difference between S10A and S10B? Is this a wet zone that could be the sliding zone?				
11	p.10	why is there poor recovery at S12? Is this the sliding zone?				
12	p.11	typo on S18 remark S8 indicates logs were not reviewed properly				
13	p.13	no water table given; no sample recovery data provided				
14	p.15	no water table given; no sample recovery data provided				
15	p.15	testing of lignite or other organic materials is important				
16	p.19	not reasonable to assume groundwater at sea level on the left side				
17	p.20	why is crest of slope not shown at highest point?				
18	p.21	only data on which to base the cohesion of the clay is the Atterberg limits; selection of 400, 500, 650, and 800 psf appears to be conservative; however, if the water table is higher than shown it may not be conservative				
19	p.22	groundwater line nearly intersects ground surface, but there is no report of water seeping out of the bluff				
20	p.22	cohesion parameters may not be correct for the higher water table				
21	p.23	why are there two water tables?				

"SHEET OR ITEM" PREFIX FOR COMMENT NO'S - PLANS =P, SPEC. PROVS=S, EST.=E, CALC BOOK=C, DESIGN REPORT=D, OTHER=O
"ACTION" A=WILL INCORPORATE, B=WILL EVALUATE, C=DELETE COMMENT

DYNAMIC
ENGINEERING EARTH SURVEY TRAFFIC

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Date: November 11, 2019
Via email: *jtsunis@northwindgroup.com*

THE PRESERVE AT INDIAN HILLS, LLC
c/o THE NORTHWIND GROUP, LLC
One Rabro Drive, Suite 100
Hauppauge, NY 11788

Attn: Jim Tsunis

Regarding: RESPONSE TO COMMENTS FROM AECOM
The Preserve at Indian Hills
Northport, Town of Huntington, Suffolk County, New York
Dynamic Earth Project No.: 3114-99-001EC

Dear Mr. Tsunis,

As requested, this letter summarizes our discussion with AECOM and the Town of Huntington on October 29, 2019 in response to geotechnical-related comments dated October 23, 2019, provided by AECOM regarding their review of *Appendix H – Geotechnical Engineering Reports and Correspondence* for the above-referenced project. The following documents were reportedly reviewed by AECOM.

- Appendix H-1: Geotechnical Engineering Services Report, Phase I of the Bluff Area Stability Evaluation, prepared by PS&S, dated July 25, 2008 (Appendix not included);
- Appendix H-2: Geotechnical Engineering Investigation and Slope Stability Analysis prepared by PS&S, dated April 15, 2019;
- Appendix H-3: Correspondence by Dynamic Earth, dated July 8, 2019.

Appendix H-1: AECOM had no comments regarding the report but did indicate that the Appendix to the report was not included in the information for their review. Nelson Pope & Voorhis, LLC agreed to provide the Appendix for AECOM's review.

Appendix H-2: Based on AECOM's comments, it was clear that AECOM did not understand the purpose and methodology of the analyses performed. The comments from AECOM are only applicable if a conventional stability analysis of the existing slope was being performed to determine the Factor of Safety (FS) of a slope before and after construction. It was clear that AECOM did not understand that the existing slope is actively moving and has been moving for over 100 years as documented in Appendix H-1 and is already at a Factor of Safety (FS) equal to 1.0 or less, so trying to establish and maintain a post development FS equal to 1.3 is not practical unless one is trying to design mitigation measures to prevent further slope movement.

The purpose of PS&S's analyses was not to evaluate the existing slope movement, nor was it to analyze and propose mitigation measures to prevent further movement. The purpose of PS&S's analyses was to evaluate how close to the existing slope a development could safely be constructed. Everyone agrees that it would not be prudent to build at the crest of the slope along the existing hazard line, but everyone also agrees that there is some distance away from the hazard line that would be considered safe for construction, as evidenced by existing dwellings to the east and west of the bluff. So, the question to be answered was, "What is that safe distance?" Appendix H-2 presented the conclusion of numerous analyses that modeled the slope and proposed construction, the conclusion of which was that maintaining a 120-foot buffer zone would have no impact on the existing slope. For comparison purposes, the current dwellings to the east of the bluff were constructed within 100 feet of the existing scarp.

During the conference call with AECOM, the above methodology and analyses were explained as follows. The first step was to build a computer model of the existing slope using existing topography, available boring data, and assumed design parameters that appropriately modeled the existing conditions with a FS equal to 1.0. As previously discussed, a FS equal to 1.0 was chosen because the existing slope is currently moving and has been for 100+ years. Once the model was established that appropriately represented the existing conditions, multiple iterations of stability analyses were performed using worst case scenarios (such as groundwater at the surface to model a long-term heavy precipitation event) and a 300 psf surcharge load at varying distances from the crest of the slope to represent the proposed development. The purpose of performing the many iterations was to determine the distance from the crest at which the surcharge load would have no affect on the existing stability of the slope. The results of the analyses indicated that the closer the surcharge was to the existing crest of the slope, the more the FS decreased, but the surcharge load applied at distances greater than 120 feet from the crest of the slope had no affect on the FS and current conditions. AECOM agreed that this was a different methodology, and they would evaluate further upon receipt of the missing information from Appendix H-1.

Appendix H-3: No comments provided by AECOM.

Conclusion: It is clear that AECOM did not have all the project information and did not understand the purpose and methodology of the analyses being performed (i.e., the analysis was not performed to design and build on a stable slope; the analyses was performed to determine a safe distance to remain away from a known failing slope). The misunderstanding of the purpose and scope resulted in inappropriate conclusions that would be more appropriate if someone were proposing to build upon a slope, as opposed to determining the distance away from said slope so as not to affect the slope. Therefore, the comments presented in AECOM's letter to the Town of Huntington unnecessarily and inappropriately raised unjustifiable concerns as related to this analysis. (We also noted that Mr. David Cregger, the author of the AECOM letter, is apparently not listed as a licensed professional engineer in the State of New York.)

We trust this correspondence will address and clarify the concerns noted in the referenced letter.

Feel free to contact us with any questions regarding these matters.

Sincerely,

DYNAMIC EARTH, LLC

Marc G. Dyer, P.E.
Senior Geotechnical Engineer
NY PE License No. 083672

Jeffrey W. Schaumburg, P.E.
Principal

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Memorandum

Date:	3/24/2020
To:	IHCC Files
From:	David M. Cregger, P.E. (NY), P.G. (NY)
Subject:	Review of Appendix H Response

AECOM responds by this memorandum to a request from Town of Huntington for Geotechnical Engineering Services for a re-review of slope stability at Indian Hills Country Club. We originally received 3 documents for review:

1. Appendix H-1 Geotechnical Engineering Services Report, Phase I of the Bluff Area Stability Evaluation, PS&S, dated July 25, 2008 *(did not include Appendix)*
2. Appendix H-2 Geotechnical Engineering Investigation and Slope Stability Analysis, PS&S, dated April 15, 2019 (with associated Appendices A&B)
3. Appendix H-3 Dynamic Earth Correspondence, dated July 8, 2019

Subsequently, we prepared a memorandum dated 10/23/2019 and had a phone conversation with the development consultant, followed by their response to our questions in a 4th document:

4. 2019-11-18 Response to AECOM memo with attachments which includes a response letter from NP&V and the Dynamic Earth Consultants; the attachment is a report prepared on July 25, 2008 titled Bluff Area Stability Evaluation - Phase I which now includes the figures and surface monitoring data which are first reviewed here.

Site location map in Figure 1 clearly shows the development area is a large hillside at the escarpment area with elevations more than 140 feet above the sea level of Long Island Sound. Topographic contours drop off significantly to the southwest where a chain of lakes is present at elevations 40 to 60 feet above sea level.

Sheet 2 shows the escarpment line, locations of cross section A-A’, B-B’, C-C’, and 16 monitoring points from the 2008 season. Except for one monitoring point M5 between the 6th Green and the escarpment, all the monitoring points are inside of the landslide. At M5, the vertical movements range from 0 to 0.7 inches of settlement. Horizontal movement vectors are lowest at M5, being less than 0.3 inches southward, but are significant elsewhere within the landslide. Several monitoring points (M1, M3, and M6) are showing similar behavior to M5, but moving slightly northward, possibly due to being located within the scarp itself. Inside the scarp, for example, the monitoring points M15, M16, and M7 show 4 inches of movement below the 13th Green, while M11 at the 12th Green is showing 7 inches of lateral movement.

Sheet 3 illustrates the topography of cross sections A-A’, B-B’, and C-C’ estimated from the year 2000, 2002, 2003, and from survey measurements made in 2008. Topographic surveys are made based on isolated points which are then contoured. Reliable conclusions can not be made from such topographic surveys without permanent benchmarks along each cross-section line.

Daily rainfall data indicates heavy ground saturation in late April 2008. Movement at the 6th Green and M5 is minimal as indicated by the numerous plots; however, movements within the landslide are significant and it remains unknown what the long-term effect would be outside the escarpment line should this material continue to slide towards Long Island Sound.

It is a common practice to back-calculate the soil shear strength parameters from a known landslide failure. However, there is no way to determine if the factor of safety is equal to 1.0 or equal to a lesser value. Insufficient laboratory testing of soil parameters was conducted, as noted in our original memorandum, to justify the selected soil parameters from back-analysis.

Appendix H-2 presents the soil landslide model in a series of cases which deserve further comment subsequent to our phone call. Case 2 does show an enlargement of the failure zone (green sliced area) when a 300 psf surcharge is applied directly upon the escarpment. Using standard practice limit-equilibrium slope modeling software such as SLOPE/W, different peak and residual soil strength parameters should be used on either side of the slip zone when surcharge is placed at the greater distances.

We see no evidence that the consultant has investigated the depth of the shear zone or monitored pore pressures to verify the landslide model developed in Phase II. Appendix H-1 indicates that 2007 Boring SB-C2 near the 12th tee encountered a very soft silty clay layer at a depth of 5 feet which is not incorporated into the landslide model.

In our opinion, which is based on our review of the data provided, we disagree with the approach of placing the surcharge from new building construction at any distance from the escarpment above the chain of lakes saturating the ground and potentially softening the clay at this site which will continue to slide as it has done for over 100 years. We conclude that 120 feet may not be enough of a buffer in a pre-existing landslide zone.

AECOM is pleased to provide this opinion and trust that this clarifies our position.