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LETTER OF TRANSMITTAL

March 27, 2023

20615-001

Mr. Rob Vanden Noven, Director of Public Works
Port Washington City Hall
100 W Grand Avenue
P.O. Box 307
Port Washington, WI 53074

Subject: **Schematic Plan Summary Report**
North Beach Restoration & Bluff Rehabilitation

Rob:

The attached *Schematic Plan Summary Report* summarizes our analysis and Schematic Plan, which we have prepared in accordance with our December 6, 2022, proposal.

The purpose of this evaluation, with support of the Fund for Lake Michigan grant, is to provide a Schematic Plan to mitigate the soil slides that have been recurring along the southern half of Upper Lake Park's shoreline bluff, protect the bluff's toe from future recession due to storm wave erosion during periodic high-water levels in the lake, and improve the beach's recreational quality.

Based on this Schematic Plan, we have prepared a preliminary estimate of construction cost for budgeting purposes and have summarized on-going and potential sources of grant funding. The next steps in planning will be to prepare grant applications and develop detailed plans, specifications, and bid documents for construction.

If you have any questions, please call.

Sincerely,

MILLER ENGINEERS & SCIENTISTS

Emily Blum, P.E.,
Project Manager

CC: Mr. Roger Strohm, City Engineer

If enclosures are not as noted, please notify us immediately.

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Schematic Plan Summary Report

NORTH BEACH RESTORATION AND BLUFF REHABILITATION

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March 2023

Fund for Lake Michigan
Clearly making a deep impact.

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EXECUTIVE SUMMARY

Port Washington's North Beach occupies the toe of a glacial till bluff along 1,900 lineal feet of Upper Lake Park's Lake Michigan shoreline. During high water levels the beach is very narrow, and the bluffs are too steep to remain stable. Existing bluff slopes range from 1.4 to 1.8 parts horizontal to 1 part vertical (1.4 to 1.8H:1V) with the bluff crest gradually rising in height from 88 feet at the south end adjacent to the Wastewater Treatment Plant to 113 feet at the north end of the park. This bluff is naturally in a condition of instability to periodic quasi-stability due to the natural process of bluff toe erosion by storm waves during episodic high water level periods that cause storm wave erosion of the bluff toe. This process initiates successive relatively shallow sliding of soils propagating upward along the slope that eventually cause westward recession of the crest.

Along the southern portion of the park, the bluff has been chronically unstable due to water seepage emanating from sand and silt strata located below mid-height in the bluff's soil profile. The soil profiles both above and below these pervious strata consist of stiff to hard lean clay that has been pre-consolidated by glaciers.

To mitigate the potential for landslides that have jeopardized use of the beach and to improve beach quality throughout the large range of fluctuations of the lake's water level, we recommend:

- **Cutting back the bluff from the present unstable slopes to 2.25 parts horizontal to 1 part vertical (2.25H:1V) along the southern 300 lineal feet** of the park shoreline's most unstable area and **transitioning to 2H:1V for the remainder of the 900 lineal feet** of recommended bluff cutback. Prior to regrading, approximately 600 lineal feet of the northbound drive at the top of the bluff will need to be relocated westward.
- Installing **several directionally drilled subsurface drainage lines** to internally intercept seeping ground water in the sand and silt strata. This should be completed at least several months prior to commencing bluff excavation in order to allow time for these seams to drain. Additionally, **an unknown** (at this time) **number of "sand points"** will likely need to be **driven approximately 20' horizontally** into the cutback bluff face at locations to the north where excavation may reveal localized seeps. Discharge from these would be incorporated in the final surface drainage design for the bluff that is integrated with an asphalt paved path that we recommend traverse the bluff face.
- Installing **a revetment along the southern 950 lineal feet** of the bluff toe and placing **gravelly sand "beach nourishment" along 1,300 lineal feet** of the park's shoreline.

These features are depicted on the *Schematic Plan* included in the Appendix.

The majority of construction cost will be to remove the large volume of soil (approximately 80,000 cubic yards) to reduce the bluff slope so as to provide a relatively stable condition. **Construction cost for the amount of work described above is preliminary estimated to be in the range of $7 to $10 million dollars,** as outlined in the Preliminary Estimate of Construction Cost section of this report. **Potential sources of matching fund grant application opportunities are summarized in Section 6.**

PROJECT DESCRIPTION

North beach along the base of the Upper Lake Park bluff system has had very limited usability during the last several years because of the high lake water level from 2017 through 2021 during which storm waves advanced erosion of the toe of the bluff. This steepening of the toe precipitated landslides that pose a safety hazard to occupants of the beach that is very narrow during high water levels.

To varying degrees, conditions were similar during prior high-water episodes: 1996-1997, 1986-1987, 1974-1975, 1953-1954, as well as numerous previous times throughout the contemporary record (refer to the graph in the Appendix of *Historic Annual Water Levels – Lake Michigan & Huron -1860 through Present*). During this time water levels have fluctuated throughout a range of about six feet, cycling on an average of every 10 to 12 years. This has been a naturally reoccurring pattern throughout the last 4,000 years while the lake has been at the present water level regime (refer to the *Lake Michigan Water Levels in Recent Geologic Time* graph in the Appendix).

Because 4,000 years is a relatively brief period of time geomorphologically speaking, the Lake Michigan shoreline is continuing to be modified from erosion caused by storm waves when water levels are high. This results in a contemporary shoreline recession rate averaging 50 to 100 feet per century as indicated by past regional evaluations that are summarized in the Evaluation Methodology section. This recession will continue unless appropriate action is taken to prevent additional loss of the park area above and maintain a safe beach of high recreational quality.

The groundwater seep from the silt and sand strata in the southernmost portion of the subject bluff has resulted in gradual slumping of soil over the asphalt path approaching the south end of the beach. Additionally, last spring a sudden relatively large slide had enough momentum to cover the beach and project out into the lake. Although most of the accumulation of that slide has since washed away by storm waves, this slide was just cause for the City of Port Washington to post notice of public closure of the beach for public safety because similar additional slides are no less likely to occur until appropriate bluff stabilization measures are implemented.

Additionally, the crest at the top of the bluff has, within recent years, advanced to within 15 feet along the southern portion of the eastern (northbound) lane of the asphalt drive through Upper Lake Park.

Making the beach useful, stabilizing the bluff so that both the beach below and the east drive in the park above are safe, and improving pedestrian connectivity between Upper Lake Park and North Beach can be accomplished by:

a) **Relocating westward about 650 lineal feet of the northbound drive** along the top to make way for step c). We recommend this be done **in advance of the project** to avoid interruption of traffic circulation in Upper Lake Park that is presently at risk due to proximity of the bluff crest to this portion of road. The remainder of the existing east drive is currently safely distant from the bluff crest.

b) **Installing several directionally drilled subsurface drainage lines** to intercept groundwater that is seeping from natural sand and silt strata located below mid-height in the southernmost portion of the bluff that has been least stable. These should be installed at least several months before bluff excavation begins.

c) **Cutting back and revegetating the southern 900 lineal feet of presently bare and unstable bluff face.** Cutting back the bluff slope can accommodate **construction of a traversing paved pathway to provide more central access to the beach from Upper Lake Park.** The existing concrete promenade around the lake (east) side of the Wastewater Treatment Plant and the existing wooden stairway extending down from the south end of Upper Lake Park will continue to provide additional points of access to North Beach. The proposed bluff traversing pathway will also provide small utility vehicle access on the slope face to conduct appropriate long term vegetation management on the stabilized bluff slope. In addition to providing a third public access route to the beach, **final design details of the traversing pathway should include drainage elements to intercept and convey rain runoff from the slope face as well as discharge from any horizontal sand points** that are installed in order to discharge this water into the revetment stabilized bluff toe to minimize the potential for surface erosion and future shallow sliding of soil.

d) **Installing a relatively small cross section of quarry stone revetment** (as compared to the Wastewater Treatment Plant's revetment) along the southern 950 lineal feet of the toe of the bluff to prevent storm wave erosion along the stabilized bluff area during high water levels. This will become at least partially buried by placement of the "beach nourishment" described below. We do not recommend armoring the northern portion of the park's shoreline because the east (northbound) drive lane in that area is sufficiently distant (75' to 125') from the bluff crest to accommodate the natural rate of shore recession for many decades to come. Additionally, there are disadvantages on a regional scale in unnecessarily or prematurely armoring the shoreline.

e) **Augmenting the natural materials** (predominantly sand) **along the southern 1,300 lineal feet of beach** with prescribed (specified) compatible locally mined native sand and gravel which can make North Beach usable even during present and future high-water levels. This is commonly referred to as "beach nourishment" and is practical at this particular location for the reasons explained in more detail in the Bluff and Shoreline Analysis section. Although natural migration of sand along this region of shoreline is expected to be from north to south, the lakeward projection of the wastewater treatment plant and its revetment forms a convenient pocket that will aid in beach "nourishment" retention.

Due to the cyclic nature of Lake Michigan water levels and the uncertain effects of ongoing global warming, our planning of this project anticipated episodic high-water levels potentially exceeding the peaks that have historically occurred.

EVALUATION METHODOLOGY

Beginning with a sound concept plan based on extensive related experience, we performed the following described site-specific data collection as the basis for analysis necessary to develop the Schematic Plan presented in the next section of this report on which to base preliminary estimates of construction cost that are summarized in the Preliminary Estimate of Construction Cost section of this report:

1) Collected available pertinent background information to develop an initial **Concept Plan** that was completed and previously provided to the City as a necessary step in preparing the Fund for Lake Michigan grant application that is the primary source of funding for the following scope of work to develop the Schematic Plan.

2) Performed a current **topographic survey** of the park's shoreline conditions extending north from the north end of the WWTP, including the land along the top of the bluff, the bluff, and the beach. Refer to the *Existing Condition Map* included in the Appendix.

3) Performed a **bathymetric survey** of the lakebed along the park's shoreline out to a depth of approximately 20 feet. Thie information is included in the Existing Conditions Map in the Appendix. Lakebed topography is important because the near shore bottom profile limits breaking wave height and determines the potential wave energy during high water level storms. Numerous **lakebed and beach sediment samples** were obtained to correlate sediment grain size to the area's cross-shore wave energy dissipation profile. Refer to the *Existing Condition Map* and *Lakebed Sediment Grain Size Analyses* included in the Appendix. Based on this **we developed the recommended grain size distribution for beach nourishment** that will be compatible with the existing beach.

4) Performed three deep **soil exploration borings (B3, B4, and B5)** across the southern half of the park and installed a ground water level monitoring well in both the southern and northern of these borings. This information is included in the Appendix along with the data from the **two borings (B1and B2)** performed by Wisconsin Testing Laboratories (WTL) **in 2014** at the crest of the bluff opposite the north end of the Wastewater Treatment Plant (refer to the *WTL Geotechnical Report Boring Logs & Laboratory Test Results* in the Appendix). These borings as well as SWRPC's 1995 *Lake Michigan Shoreline Recession and Bluff Stability in Southeastern Wisconsin Technical Report* No. 36 **identify a silt stratum below mid-height in the bluff that is water bearing** and remains visible along portions of the presently failing bluff face. Additionally, both 2014 borings revealed wet silt and sand strata. Based on this soil information we **analyzed slope stability** to provide the basis for the bluff stabilization components of the Schematic Plan. Those calculations are included in the Appendix.

5) Performed **wave breaking wave height analyses** based on our bathymetric and topographic surveys to determine the wave energy reaching the shoreline during high water levels. This information was utilized in determining the appropriate rip rap size and revetment geometry to provide adequate wave energy absorption. This analysis is included in the Appendix.

6) Developed a **Schematic Plan** for cutting back the bluff face along the southern 900 lineal feet of the park's shoreline and incorporating a traversing path, relocating the adjacent portion of east (northbound) drive that's presently at risk in Upper North Lake Park, proportioning and positioning a revetment along 1,300 lineal feet of bluff toe, and providing beach nourishment (refer to the Appendix).

7) Consulted with qualified local contactors experienced in this type of work to develop a **Preliminary Estimate of Construction Cost**.

8) Researched and identified **appropriate matching grant potential sources** to which applications can be submitted to seek potential matching fund sources for project final design and construction, as outlined in the Potential Grant Fund Sources section.

BLUFF & SHORELINE ANALYSIS

Considerable information on the bluff and shoreline conditions of the park is included in the 1977 Shore Erosion Study (*Shoreline Erosion and Bluff Stability Along Lake Michigan and Lake Superior Shorelines of Wisconsin*) that was performed on behalf of Wisconsin Coastal Management and the 1995 report (*Lake Michigan Shoreline Recession and Bluff Stability in Southeastern Wisconsin: 1995 – Technical Report No. 36*) that was performed on behalf of the Southeastern Wisconsin Regional Planning Commission.

In general, except for limited areas where bedrock is exposed, shoreline erosion and recession occur at varying rates around the shores of Lakes Michigan for two reasons:

1) The present regime of water level fluctuations of Lake Michigan (and Huron) has been ongoing for just the last 4,000 years (a relatively short period of time geomorphologically), so **the shoreline is still evolving.** Prior to this, water levels at different times had been both much higher and much lower as the most recent glaciers receded from the area about 12,000 years ago.

2) The three western **Great Lakes are large enough for sustained strong winds** to **develop very tall waves out in deep water**.

In deep water, offshore wave height and wavelength are determined by the strength and duration of wind up to a physical limit confined by the exposed fetch (the length of the path of the wind over the water). At Port Washington, the fetch of south-southeast to southeast wind directions is 100 to 120 statute miles and the fetch ranges from 75 to almost 200 miles under wind from the northeast to north-northeast. For these lengths of fetch, and sustained wind durations of 6 to 24 hours, **deep water wave characteristics are**:

Wind Speed (knots*)	**Height (ft)**	**Length (ft)**	**Period (seconds)**
15 – moderate breeze (fun sailing)	3 to 5	45 to 120	3.0 to 4.8
25 – strong breeze (small craft warning)	8 to 12	90 to 230	4.2 to 6.7
35 – fresh gale	13 to 24	150 to 300	5.3 to 7.6

1 knot*=nautical mile (n. mi.=6,080 ft) per hour=1.151 statute miles (s. mi.=5,280 ft) per hour

However, the **large wave heights that are generated out in deep water are greatly reduced as they approach the shore** because **diminishing water depth causes them to break.** This limits their height to about two-thirds to three-quarters of the water depth. As waves break throughout the surf zone, most of their energy is dissipated by water turbulence. With wave height limited by the diminishing water depth, **it is the bottom profile of the surf zone that is most critical to protecting the shoreline** itself from on shore storm wave erosion. Beach nourishment will contribute to this.

To maximize the amount of time that a usable beach is exposed throughout the lake's 6 foot range of fluctuating water levels, **revetments should be placed as far inland as is possible.** This position also minimizes the duration of time that the revetment is exposed to storm waves, thereby increasing its longevity and reducing its interference with the natural dynamics of the beach and near shore sand bars. It also minimizes the necessary scale of the revetment's cross section, stone sizes and quantity, and consequently its cost. Refer to the Appendix for the related analysis.

Slope stability analyses that we performed to develop our recommendations for bluff stabilization are included in the Appendix of this report. **The most common mode of lakeshore bluff slope failure consists of relatively shallow,** commonly two to six feet thickness of surficial **soil sliding, that has softened** due to weather wetting-drying and seasonal freeze-thaw cycles **over years of exposure**. This "weathering" action eventually reduces the soil shear strength of the upper several feet to a "soft" (shear strength less than 500 psf) condition in contrast to the naturally very stiff (shear strength greater than 3,000 psf in this case) parent clay comprising the majority of the bluff. Section I of the *Stability Analysis* in the Appendix provides a quantitative evaluation of this mode for the schematically designed slope range of 2H:1V to 2.25H:1V for the areas recommended to be graded back. Section II provides this analysis throughout the common regional range of bluff slopes extending from as steep as 1V:1H to as gentle as 3H:1V.

The results of this analysis are consistent with nearby, as well as predominant regional examples, of long-term field performance and are the basis for the design slopes we have recommended. These results indicate the **critical role of the roots of well-**

maintained vegetation in providing surficial soil reinforcement to compensate for the inevitable loss of shallow soil strength due to weathering. Additionally, the above-grade portions of vegetation remove soil pore water through evapotranspiration as well as prevent soil erosion.

A much less common mode of bluff slope failure can occur as deep failure surfaces in locations where some strata within the clay profile are just "normally" consolidated and therefore have much less shear strength than the substantially "over-consolidated" profile of the bluff system along Upper Lake Park. Section III of the stability analyses in the Appendix quantitatively indicates why this mode of slope instability is not a controlling design criterion for this project.

Slope stability analyses compute "Factors of Safety" which is the numerical ratio of the soil shear strength available along any potential failure surface, divided by the gravitational forces exerted on the overlying soil mass. A Factor of Safety of 1.0 indicates incipient failure. Therefore, an absolute minimum "design" safety factor of 1.3 might be used to provide some allowance for variations in soil strength and drainage conditions for slopes where the cost of repair is no more than the cost of slide prevention, but more commonly 1.5 is used. At locations where the cost of repair would be substantially more than that of prevention, or valuable infrastructure could be at risk, a Factor of Safety of 2 may be used to direct slope stabilization design.

However, the Factor of Safety for potential deep slope failure modes of the existing steep slopes of the park (that are undergoing shallow sliding) are already in the range of 2 to 3. For this reason, the "deep" analyses are not controlling. In this analysis, a uniform profile of "undrained" soil shear strength in the range of 3,333 psf ("very stiff") to 4,267 psf (borderline "hard") as is indicated by the consistent lower bound and median values of Standard Penetration Blow Counts (N values) throughout all the boring logs. The "undrained" condition of soil strength is appropriate for the analysis in this case because the predominant inciting factor of deep slope failure surfaces in stiff clay profiles is a sudden increase in the "pore pressure" profile that commonly occurs each spring.

RECOMMENDATIONS AND SCHEMATIC PLAN

This section of the report includes a brief synopsis of the relevant background information and salient considerations that are the basis of the recommendations and Schematic Plan presented herein.

Bluff Stabilization

The bluffs throughout this region consist predominately of lean clay of stiff to hard consistency due to pre-consolidation by glaciers. The clay's inherently high strength temporarily allows very steep scarps at the bluff toe caused by the powerful erosion forces of storm waves. However, successive "weathering" (seasonal freeze-thaw and wetting-drying) causes weakening of surficial soil shear strength that results in upward propagation of relatively shallow sliding that, within a decade or two, often develops quasi-equilibrium slopes commonly of about 2 parts horizontal to 1 part vertical (2H:1V) provided adequate surface vegetation. By that time, a new cycle of bluff toe erosion has typically occurred, starting that natural process over again.

Any water-bearing sand or silt seams cause periodic to continuous soil slides of considerably flatter proportions as have been occurring at the southern end of the park's bluff. The water bearing sand and silt strata visible in the *Oblique Aerial Photographs* (included in the Appendix) were also encountered between elevation 615 and 630 feet (NAVD) in Boring 3 and Boring 5 that were performed. Due to the spatial variation inherent in the sand and silt deposits, that stratum was not encountered in Boring 4 despite visible signs of the layer on the face of the bluff in that area and indicates the seam tapers out in the vicinity of Boring 4. We expect the source of water seeping from the relatively pervious silt and sand strata is the corresponding elevation of the creek bed from a position that's a considerable distance to the north-northwest. Although the pervious seams exposed farther north along the park's bluff are consistently moist, they appear to be seeping much less water so the natural, varyingly vegetated slopes there are relatively stable.

Water seeping within the sand and silt strata can be intercepted before it emerges from the slope face by installing several directionally drilled, perforated pipes deep within the bluff at the southern end of the park's shoreline. Recommended locations and profiles are depicted in the Schematic Plan included in the Appendix. Because it takes time for ground water dewatering systems to take effect, these should be installed at least several months before bluff excavation commences.

The average quasi-stable slope of well vegetated slopes in this region that do not have water seeping from them is about 2H:1V; therefore, we recommend the bluff slope along **the southern 300 feet** that is presently most unstable **be cut back** from its present unstable slope **to the Schematic Plan's 2.25H:1V recommended slope,** and **another 600 feet to the north** be cut back to **2H:1V. Revegetation of this reduced slope face is a critical element** of slope stabilization final design, the detailed specifications for which should be included in any final design plans for construction.

Excavation of the bluff face should include construction of a traversing pathway wide enough to accommodate at least small equipment to access the slope face for long term vegetation management. This can be asphalt paved to also provide pedestrian access. In any case, design details should include drain lines that intercept rain runoff and convey that water safely into the top of the revetment at the bluff toe. These elements are shown in the *Schematic Plan* in the Appendix.

Access to the south end of the beach has been provided along the concrete paved promenade around the lake (east) side of the Wastewater Treatment Plant as well as from a wooden stairway extending down from the south end of Upper Lake Park. The promenade is wide enough to provide emergency vehicle, construction equipment, and trucking access to the beach and is protected by a substantial revetment comprised of large quarry stone. The wooden stairway is on a relatively stable earth slope. These will remain as points of public access to the beach in combination with the multi-function pathway that we propose traversing the bluff along the southern third of the park.

“Buried” Revetment

Although future loss of land from natural shore erosion can be mitigated by installing revetments, that has the overall disadvantage of stopping the natural supply of soil particles that form the beach and near shore bottom sediment. Therefore, revetments should be constructed along no more length of shoreline than is necessary to protect important infrastructure or resources, even though the predominantly clay bluffs are an inefficient supply of beach sand. It is for this reason we recommend that revetment installation be avoided along the northern portion of the park’s shoreline where ample space remains between the northbound loop of the drive through Upper Lake Park that traverses parallel to the bluff crest.

Based on our analysis (refer to Appendix) it is our opinion that **a quarry stone revetment placed against the toe of the bluff and ranges from 5 to 6 tons per lineal foot of material**, including armor stone and a bedding stone layer, is suitable for bluff toe protective revetment at this location. Design details and specifications for this should be included in final design for the project.

Beach Nourishment

The shoreline extending from the City of Port Washington north to Harrington Beach State Park is relatively sand starved, as are many reaches along the west shore of Lake Michigan. This is because the lakeshore bluffs are comprised primarily of silt and clay size soil particles, with only about 15% to 20% of the bluff soil eroded by storm waves during high water level periods consisting of sand that forms the beaches. Less than 10% consists of rounded gravel size particles. Therefore, **shoreline bluff erosion is not an efficient ongoing supply of sand to compensate for the natural longshore and cross shore displacement of sand.**

With the average long term shore recession rate in this area in the range of 50 to 100 feet per century, and an average bluff height commonly ranging from 80 to 100 feet, the long term average natural beach sand “nourishment” contribution due to unimpeded natural shore erosion north of the City is in the range of 1 to 2 tons per lineal foot of shoreline per year. However, this occurs in relatively large increments only during high water levels that advance shore erosion. **Wherever bluff toe natural recession is prevented by construction of revetments, the natural supply of sand at those locations is halted**. Additionally, increasing proportions of shoreline north of the City of Port Washington can be expected to become “hardened” by construction of revetments in future decades. Therefore, **beach nourishment is the only way to compensate for this in a manner that doesn’t interfere with natural shore dynamics**, and it should become an increasingly common thing to do elsewhere in the future.

The natural cross shore wave dynamics hydraulically sort out the wide range of soil particle sizes (occasional boulders and cobbles, a little gravel, some sand, but mostly silt and clay) of the eroding bluff material so that the cobbles and gravel are retained closest to the bluff toe. The sand forms the variable beach width and the shifting sand bars throughout the surf zone. The silt and clay settle on the lakebed farther out. Altogether, these sorted out materials form a cross section of diminishing particle size with increasing distance and depth of water offshore which, aside from shifting of the beach geometry and width as well as the sand bars with each storm and varying water level, is relatively consistent through time.

The sand comprising the beach along the park has an average grain size of 0.24 mm and there are spatially varying amounts of gravel. Sand particle size diminishes with increasing water depth and distance offshore to an average of about 0.16 mm at 6' to 8' depth and 0.12mm at 10' to 15' depth. Results of *Grain Size Analyses* of the near shore lakebed *and* beach samples is the Lakebed Sediment of the existing beach and near shore bottom sediments are included in the Appendix.

Even though some gravel is present along Port Washington's North Beach, it is the sand grain size that comprises the relevant longshore migration material as well as the capillary rise of water above the calm water line at any time that affects beach aesthetic quality and affinity for growing bacteria. **Beach quality can be optimized by introducing beach "nourishment" material consisting of well-graded sand** that includes 10% to 20% of non-crushed gravel and falls within the Particle Size Grading Band boundaries described below:

Particle & Sieve Size	% Passing by Weight
3"	100
1"	95-100
#4	80-90
#10	65-85
#20	45-75
#40	20-55
#100	0-15
#200	0-5

This can be supplied by local sand and gravel quarries by appropriate screening and blending of non-crushed materials.

The longshore migration of sand along the open, unobstructed reaches of shoreline along of the north half of Lake Michigan generally nets out on a long-term basis from north to south due to slightly more bias in NE vs SE onshore storm wind origins as well as the longer exposed fetch from the north. However, subtle variations in the bathymetric contours of the lakebed may be causing this area to have little bias in either northward or southward longshore sand migration. In any case, large scale harbor structures that are vital to all Great Lakes port cities obstruct any longshore migration. Therefore, some accumulation of sand would be expected along the base of the north breakwater.

However, historical aerial photos and our bathymetric survey show that no accumulation has been occurring here. This may be due to the plan view orientation of the harbor's north breakwater closely coinciding with the direction of SE storm driven waves that parallel the breakwater until they are reflected off the face of the substantial revetment protecting the wastewater treatment plant. This creates a somewhat chaotic zone of relatively high wave energy that drives northward any sand that would otherwise be accumulating at the base of the breakwater. It instead comes to rest along the north edge of the Wastewater Treatment Plant's revetment.

At this location, there appears to be insignificant longshore migration of sand, with the losses tending to be relatively small in the cross-shore direction as the bluff toe recedes over time. This is advantageous to the longevity of any beach nourishment done at this location to provide some usable beach during future highwater periods. The amount of beach nourishment **we recommend (10 to 11 tons per lineal foot)** is expected to provide some usable width of beach during even highwater levels.

The width of beach at any time varies with each storm as the near shore sand profile, including the nearshore bars, reshape in response to fluctuating wave patterns. And with beach nourishment, during low water levels the beach at this location will be wider than it has been in the past throughout both high and low water levels.

Final design of beach nourishment should include planting of marram ("dune") grass in the upper portion of the beach nourishment profile to hold in place blowing sand and to self-propagate into dune formation during low water levels.

PRELIMINARY ESTIMATE OF CONSTRUCTION COST

Bluff work at this scale is regionally not a common enough occurrence, and each project is unique enough, so that usual unit costs of excavation are not available. Nevertheless, we have endeavored to estimate the cost of bluff excavation, which is the major cost component of this project based on conversations with local contractors experienced in this type of work. We expect actual construction bids are likely to cover a wider spread of bid costs than is usual for more common types of major infrastructure work.

Regarding the bluff stabilization, there can be some choices in soil erosion control revegetation that cover a substantial range in unit cost, but with diminishing reliability or performance with decrease in initial cost. Therefore, the choices in this aspect during final design should weigh initial capital versus long term maintenance costs.

Due to the damage caused by recent high-water levels, revetment work has become quite common. Therefore, the cost estimate of this component is likely to be more reliable. And although beach nourishment is not common, it relies on locally quarried sand and gravel for which unit costs are well established and placement of this material is easy.

For preliminary capital planning for this project and to prepare grant applications for potential sources of construction funds, we estimate the following cost components:

TASK ITEM	ESTIMATED BUDGET (in thousands of $)
1) **Final design**, specifications and preparation of bidding and construction contract documents, preparations of grant applications, and permit applications	$150
2) Install directionally **drilled subsurface drain lines**	$80 to $100
3) **Bluff excavation**, pathways, horizontal sand points, surface drainage features, erosion control, and revegetation	$4,600 to $7,000
4) **Revetment construction**	$1,700 to $2,300
5) **Beach nourishment** and dune grass	$500 to $700
6) Periodic **inspection and administration** during construction	$150
TOTAL ESTIMATED BUDGET	**$7,180 to $10,400**

Approximately $7 to $10 Million.

This preliminary cost estimate does not include the relocation of the necessary portion of the roadway at the top of the bluff in preparation for this project. While the City has not indicated a desire to provide a restroom near the bottom of the existing wooden stairway, we have included that on the Schematic Plan. Due to the large distance from any other restroom facility, we recommend continued use of seasonal "port-a-potties" at this location.

POTENTIAL GRANT FUND SOURCES

The City of Port Washington was awarded a grant from Fund for Lake Michigan to cover the majority of this initial data gathering, analysis, and Schematic Plan development. It is our understanding that the City currently has submitted a 2023-2024 grant application from Wisconsin Coastal Management Program for preparing a final engineering design for regrading the bluff slope, subsurface and surface drainage systems, revetment, and beach nourishment. Additionally, UW-Whitewater has spearheaded the 2024-2026 grant application to Wisconsin Sea Grant for public education of the project and slope stability research along the northern portion of the park that will be left unaltered based on our Schematic Plan.

Other grant applications to be considered consists of Sustain our Great Lakes, FEMA, and Knowles-Nelson Stewardship.

NEXT STEPS

The purpose of the Schematic Plan presented herein is to provide the basis for next steps in planning that should include:

1) **City financial planning to develop a capital budget target** in consideration of a range of potential matching grant fund sources. This may affect the lineal footage of revetment to be constructed as well as the amount of beach nourishment to provide at this time versus the future.

2) **Application to appropriate grant fund sources** for potential cost share funding of construction.

3) **Preparation of Design Plans, Specifications, and Bid Documents for construction**, which may be required to qualify for some of the potential grant funds.

Concurrent with but independent of the above steps, we recommend the portion of road at the top of the bluff that is presently at jeopardy be relocated. This will avoid interruption of public use of Upper Lake Park that may otherwise occur any time soon in the event of just a minor amount of additional bluff crest recession which may occur this spring.

Because the directionally drilled subsurface drain lines to intercept ground water seepage emanating from sand and silt seams in the bluff is a relatively small cost component of the remainder of bluff stabilization work, it too should be considered in advance. This, in combination with a relatively minor amount of excavation of unstable soils, may allow re-opening to the public this summer to take advantage of usable beach width as the water level may continue to drop in the near future.

This report has been prepared for the exclusive use of the City of Port Washington. Our recommendations are applicable only to the project as described and conditions disclosed herein. It was not prepared for uses or parties other than those specifically named or for applications other than those enumerated herein. For purposes or uses other than those specifically named, this report may contain information that is insufficient or inaccurate.

We appreciate participating in this project with you. Please call if you have any questions or comments pertaining to our work.

Prepared by,

MILLER ENGINEERS & SCIENTISTS

Emily Blum, P.E.	Roger G. Miller, P.E.
Project Manager	President

\\Fs01\sys\DATA\20600\20615 - Port Washington Upper Lake Park\001 - Bluff and Shoreline Stabilization\DRAFT REPORT\Shoreline and Bluff Rehabilitation Report draft.doc

APPENDIX

Oblique photographs herein are from the Shoreline Oblique Viewer website (https://floodscience.maps.arcgis.com/apps/instant/minimalist/index.html?appid=c47ab45bb8c046e099a46df28837ca88).

Photo 1: 1976 South end of the park.

Photo 2: 1976 Middle of the park.

Photo 3: 1976 North end of the park.

Photo 4: 2022 South end of the park.

Photo 5: 2022

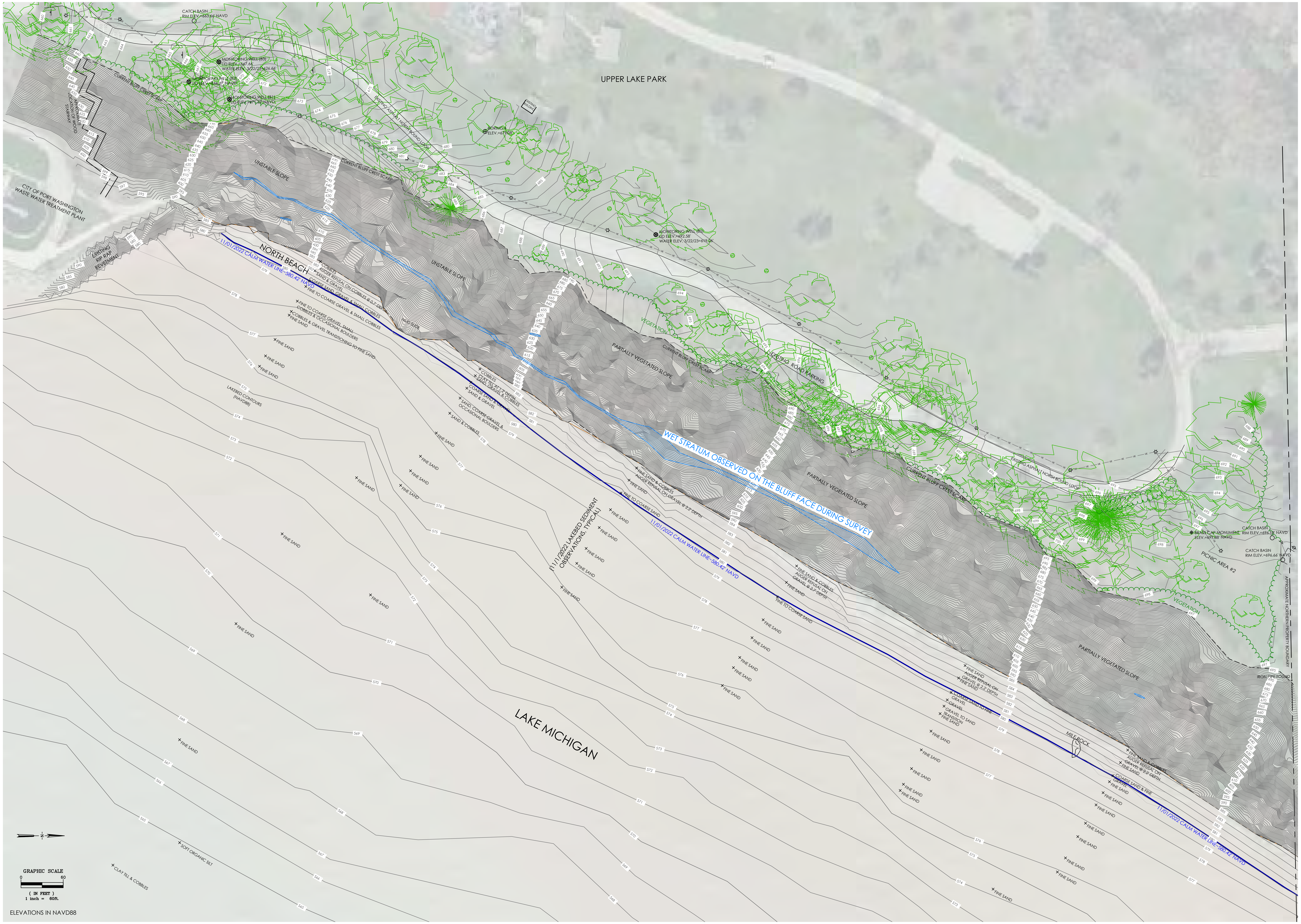
Photo 6: 2022

Photo 7:   2022

Photo 8:   2022

Photo 9: 2022

Photo 10: 2022 North end of the park.



ELEVATIONS IN NAVD88

EXISTING CONDITIONS MAP

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UPPER LAKE PARK BEACH & BLUFF REHABILITATION
CITY OF PORT WASHINGTON
498 N LAKE STREET
PORT WASHINGTON, WI

SCALE
HOR. 1"=60'
VER. 1"=60'

DATE
03/24/2023

JOB
20615-B

BY
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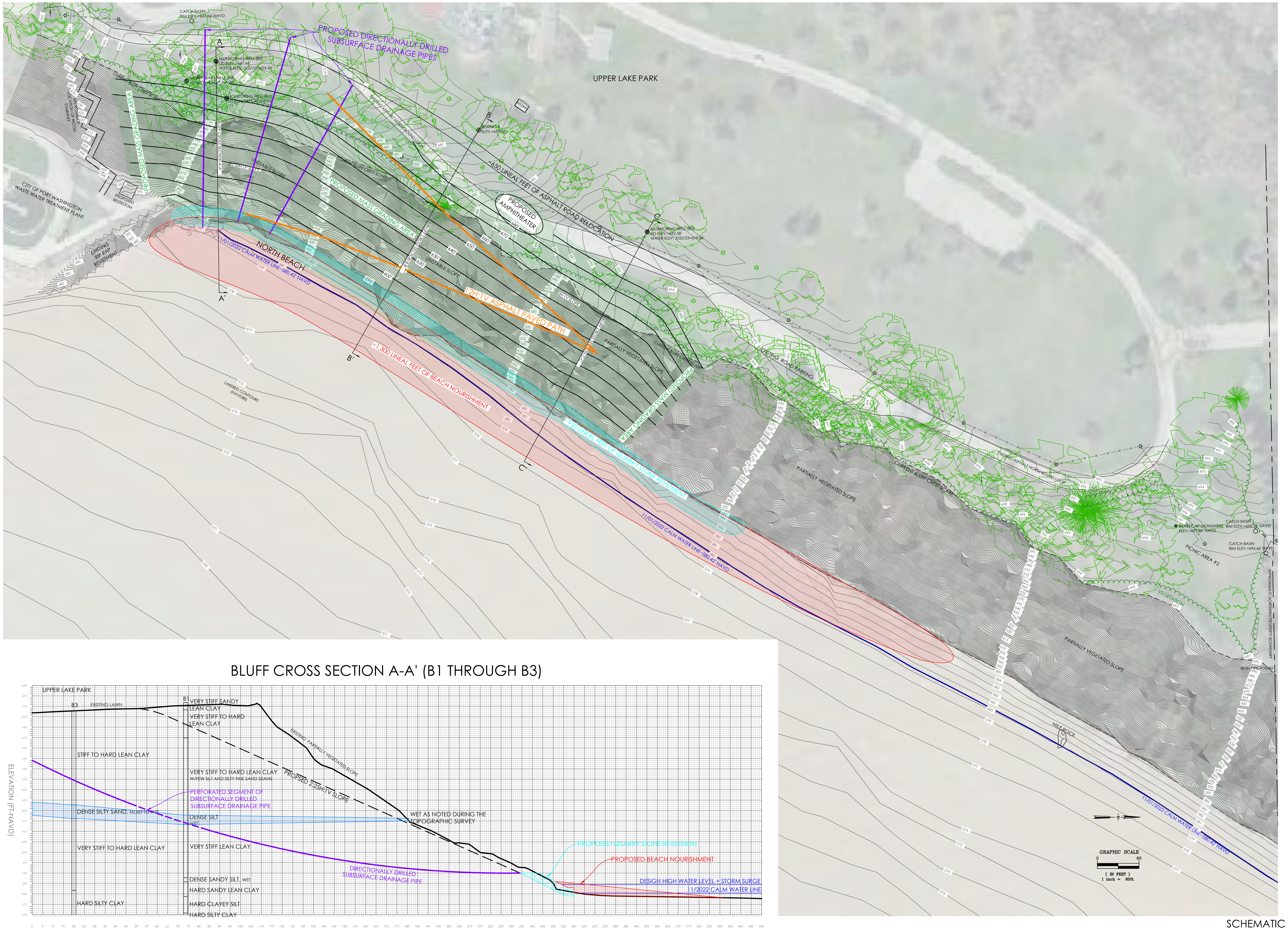
SHEET
1 OF 2

I:\DATA\20600\20615 - Port Washington Upper Lake Park\CAD\DESIGN\20615 - B Bluff Stability.dwg 3/24/2023 3:33 PM



BLUFF CROSS SECTION A-A' (B1 THROUGH B3)

SCHEMATIC PLAN

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SCALE
HOR. 1"=60'
VER. 1"=60'
DATE
03/24/2023
JOB
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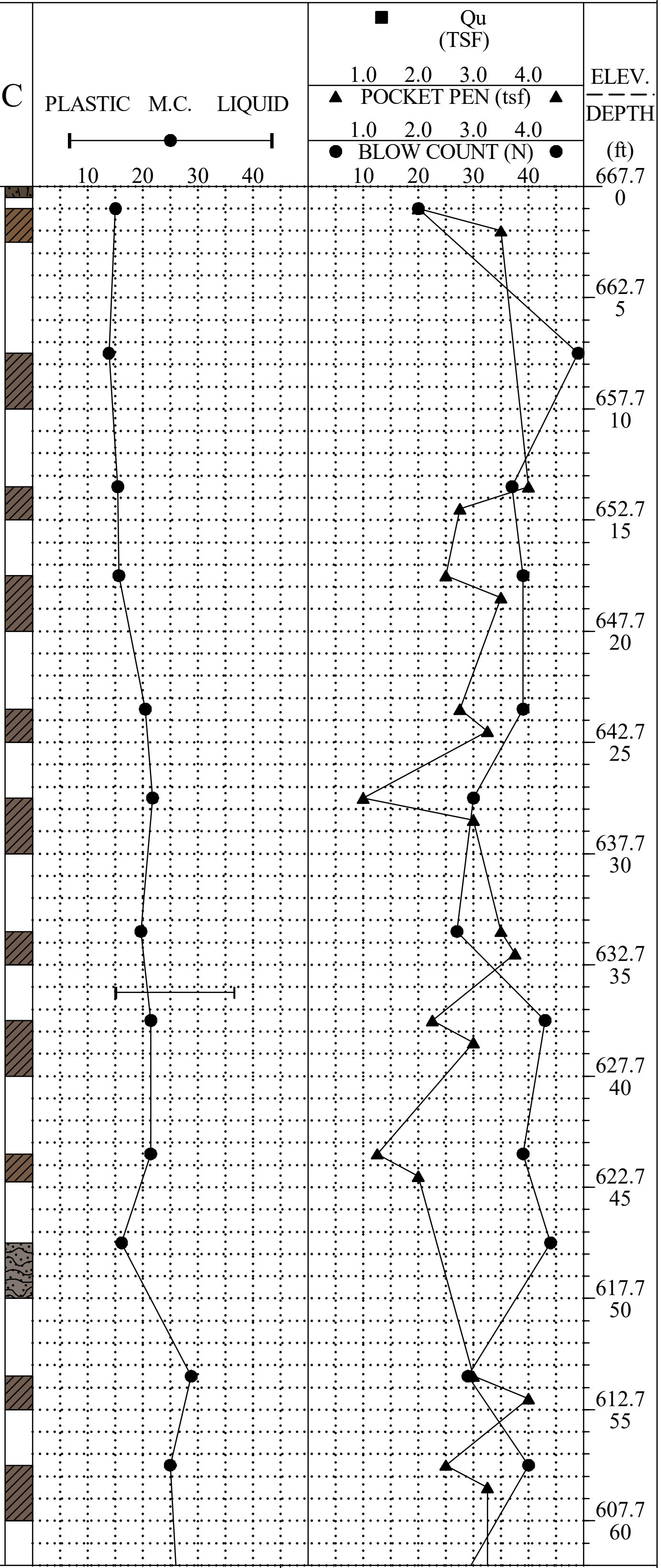
SHEET
2 OF 2

Project: **UPPER LAKE PARK**	Job No: **20615-001**	Boring No: **Boring 3**
Client: **City of Port Washington**	Drilled By: **Geotest**	Elevation: **667.7'**
Location: **Port Washington, WI**	Drilling Begun: **3/1/23**	Drilling Completed: **3/3/23**

SAMPLE TYPE ▮ 1" Geoprobe ○ No Recovery Ⓖ Grab Sample ▮ Auger Sample ■ 3" Shelby Tube ◩ 2" Split Spoon

ELEV. / DEPTH (ft) 667.7	SAMPLE NO.	SAMPLE TYPE	RECOVERY (in.)	SPT (N)	SOIL DESCRIPTION	USC
0					TOPSOIL	
	1	◩	10	20	LEAN CLAY - moist, very stiff, little fine to coarse sand, trace fine to coarse gravel, brown (7.5YR 4/4)	CL
662.7 / 5						
	2	◩	3	49	...moist, ribbon sample, brown (7.5YR 4/2)	CL
657.7 / 10						
652.7 / 15	3	◩	18	37	...moist, very stiff to hard, few fine to coarse sand, brown (7.5YR 4/2)	CL
	4	◩	18	39	...moist, very stiff to hard, few fine to coarse gravel, brown (7.5YR 4/2)	CL
647.7 / 20						
642.7 / 25	5	◩	18	39	...moist, very stiff to hard, trace fine to coarse gravel, brown (7.5YR 4/2)	CL
	6	◩	18	30	...moist, stiff to very stiff, trace fine to coarse sand, trace wet pocket, brown (7.5YR 4/2)	CL
637.7 / 30						
632.7 / 35	7	◩	18	27	...moist, very stiff, trace fine sand lens, brown (7.5YR 4/2)	CL
	8	◩	14	43	...moist, very stiff to hard, trace fine sand lens, brown (7.5YR 4/2)	CL
627.7 / 40						
622.7 / 45	9	◩	16	39	...moist, stiff, brown (7.5YR 4/3)	CL
					...thin silt layer and wet poorly graded sand with silt layer	
	10	◩	12	44	SILTY SAND - moist to wet, dense, few clay lens, grey (7.5YR 5/1)	SM
617.7 / 50						
612.7 / 55	11	◩	16	29	LEAN CLAY - moist, very stiff, trace fine to coarse sand and gravel, trace thin fine sand lens, brown (7.5YR 4/2)	CL
	12	◩	18	40	...moist, very stiff to hard, brown (7.5YR 4/2)	CL
607.7 / 60						



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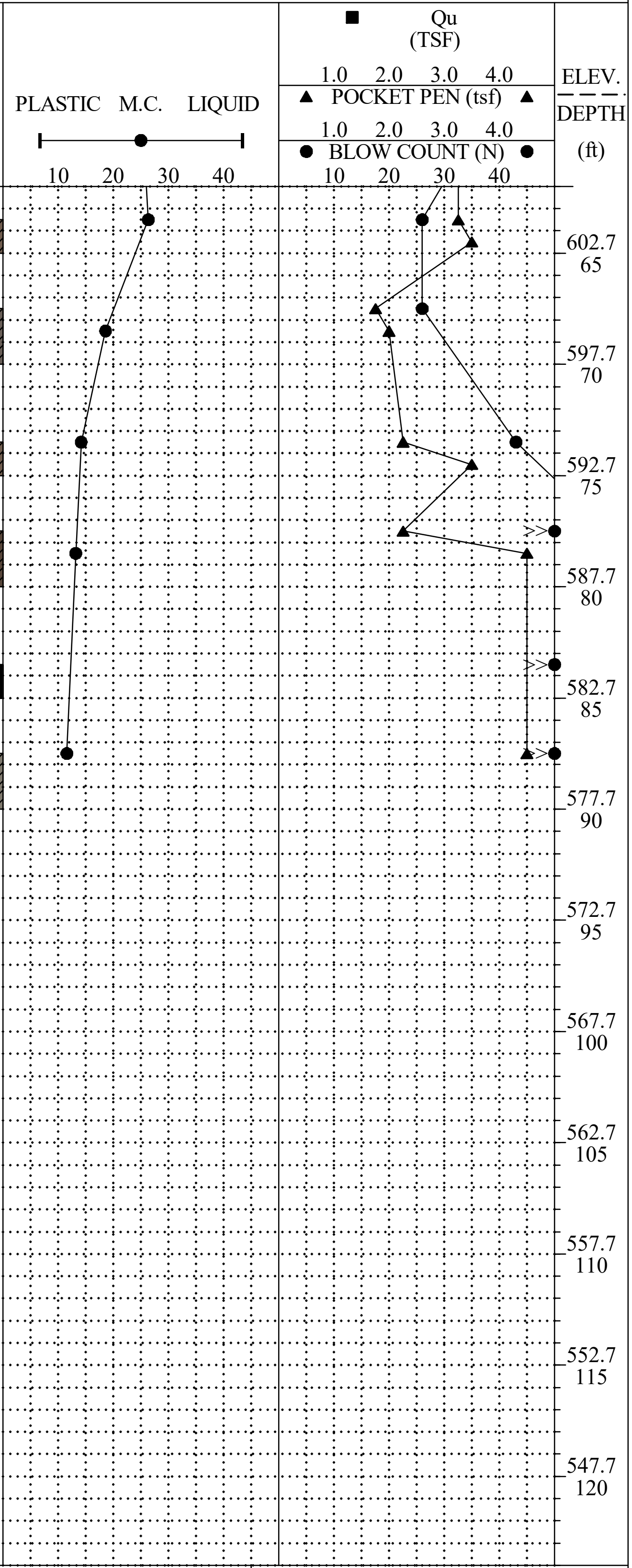
	Water Level	Cave-in Depth	Borehole Abandonment	
Date ________ Time ______	______ft.	______ ft.		Crew: **Geotest**
Date ________ Time ______	______ft.	______ ft.	Date: **3/3/2023**	Rig: **Geoprobe 7822DT**
Date ________ Time ______	______ft.	______ ft.	Material: **Well Casing**	Method: **Mud Rotary 6"**

Project: **UPPER LAKE PARK**	Job No: **20615-001**	Boring No: **Boring 3**
Client: **City of Port Washington**	Drilled By: **Geotest**	Elevation: **667.7'**
Location: **Port Washington, WI**	Drilling Begun: **3/1/23**	Drilling Completed: **3/3/23**

SAMPLE TYPE ▮ 1" Geoprobe ○ No Recovery G Grab Sample ▮ Auger Sample ■ 3" Shelby Tube ◩ 2" Split Spoon

ELEV. / DEPTH (ft)	SAMPLE NO.	SAMPLE TYPE	RECOVERY (in.)	SPT (N)	SOIL DESCRIPTION	USC
602.7 / 65	13	◩	18	26	...moist, very stiff, brown (7.5YR 4/2)	CL
	14	◩	18	26	...moist, very stiff, dark grey (7.5YR 4/1) with many mottles	CL
597.7 / 70						
592.7 / 75	15	◩	18	43	...moist, very stiff to hard, few fine to coarse sand, brown (7.5YR 4/2)	CL
	16	◩	16	120/10	...moist, very stiff to hard, trace silty sand seams, dark brown (7.5YR 3/2)	CL
587.7 / 80						
582.7 / 85	17	◩	0	71/1	...no recovery	
	18	◩	3	124/9	SILTY CLAY - moist, hard, few fine to coarse sand, trace fine to coarse gravel, trace gravel seam, grey (2.5Y 5/1)	CL ML
577.7 / 90					Boring Terminated @ 90' Depth.	
572.7 / 95						
567.7 / 100						
562.7 / 105						
557.7 / 110						
552.7 / 115						
547.7 / 120						



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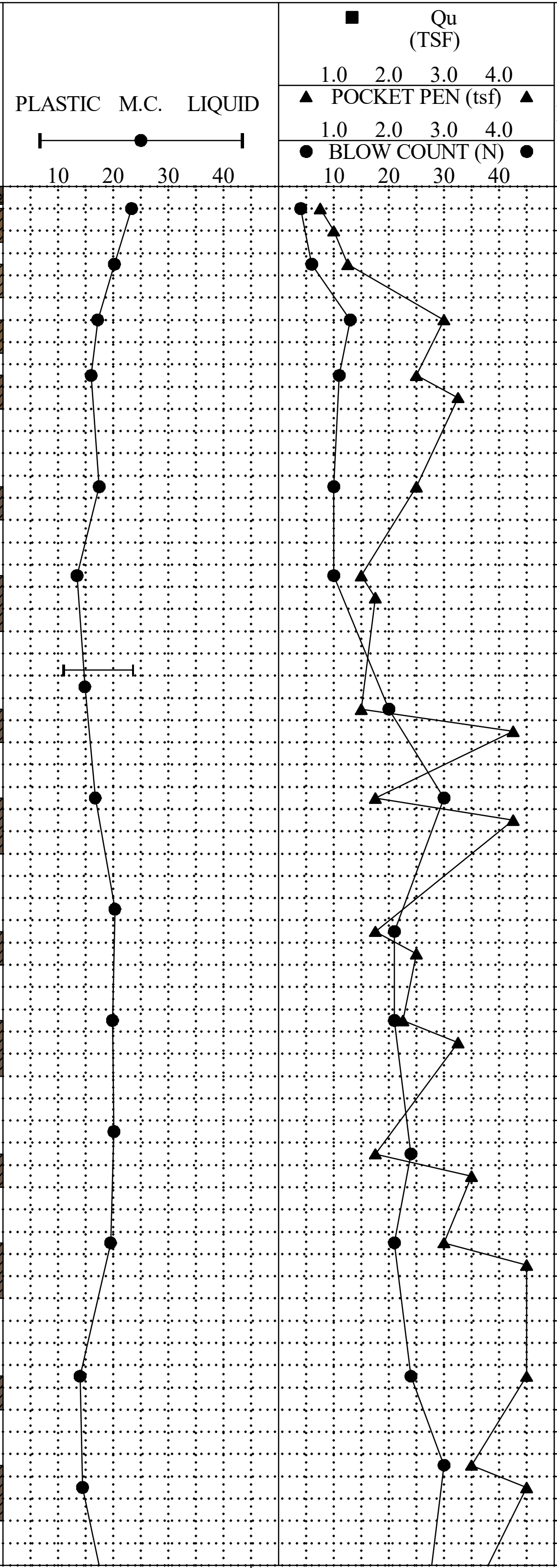
Water Level		Cave-in Depth	Borehole Abandonment	
Date ________ Time ______	______ft.	_______ ft.	Date: **3/3/2023**	Crew: **Geotest**
Date ________ Time ______	______ft.	_______ ft.		Rig: **Geoprobe 7822DT**
Date ________ Time ______	______ft.	_______ ft.	Material: **Well Casing**	Method: **Mud Rotary 6"**

Project:	**UPPER LAKE PARK**	Job No:	**20615-001**	Boring No:	**Boring 4**
Client:	**City of Port Washington**	Drilled By:	**Geotest**	Elevation:	**679.1'**
Location:	**Port Washington, WI**	Drilling Begun:	**2/13/23**	Drilling Completed:	**2/28/23**

SAMPLE TYPE ▮ 1" Geoprobe ○ No Recovery Ⓖ Grab Sample ▯ Auger Sample ■ 3" Shelby Tube ◩ 2" Split Spoon

ELEV. / DEPTH (ft) 679.1	SAMPLE NO.	SAMPLE TYPE	RECOVERY (in.)	SPT (N)	SOIL DESCRIPTION	USC	ELEV. / DEPTH (ft) 679.1
0					TOPSOIL		0
	1	◩	12	4	LEAN CLAY - moist, medium, few fine to coarse sand, trace fine to coarse gravel, brown (7.5YR 4/4) with common mottles	CL	
674.1 / 5	2	◩	14	6	...wet, medium, brown (10YR 5/3) with many mottles	CL	674.1 / 5
	3	◩	18	13	...moist, stiff to very stiff, brown (7.5YR 4/4)	CL	
669.1 / 10	4	◩	16	11	...moist, stiff to very stiff, brown (7.5YR 4/3)	CL	669.1 / 10
664.1 / 15	5	◩	18	10	...moist, stiff to very stiff, trace fine sand lens, brown (7.5YR 4/2)	CL	664.1 / 15
	6	◩	18	10	SANDY LEAN CLAY - moist, stiff, trace fine to coarse gravel, trace fine sand lens, brown (7.5YR 4/2)	CL	
659.1 / 20							659.1 / 20
654.1 / 25	7	◩	14	20	...moist, very stiff, brown (7.5YR 4/2)	CL	654.1 / 25
	8	◩	18	30	LEAN CLAY - moist, stiff to very stiff, few fine to coarse sand, trace fine to coarse gravel, brown (7.5YR 4/2)	CL	
649.1 / 30							649.1 / 30
644.1 / 35	9	◩	18	21	...moist, very stiff, trace fine to coarse sand, brown (7.5YR 4/2)	CL	644.1 / 35
	10	◩	16	21	...moist, very stiff, brown (7.5YR 4/2)	CL	
639.1 / 40							639.1 / 40
634.1 / 45	11	◩	16	24	...moist, very stiff, brown (7.5YR 4/2)	CL	634.1 / 45
	12	◩	18	21	...moist, very stiff, brown (7.5YR 4/2)	CL	
629.1 / 50							629.1 / 50
624.1 / 55	13	◩	18	24	...moist, very stiff to hard, few fine to coarse gravel, brown (7.5YR 4/2)	CL	624.1 / 55
	14	◩	16	30	...moist, very stiff to hard, trace fine to coarse gravel, brown (7.5YR 4/2)	CL	
619.1 / 60							619.1 / 60



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	Water Level	Cave-in Depth
Date **2/13/2023** Time ______	**15** ft.	______ ft.
Date ________ Time ______	______ft.	______ ft.
Date ________ Time ______	______ft.	______ ft.

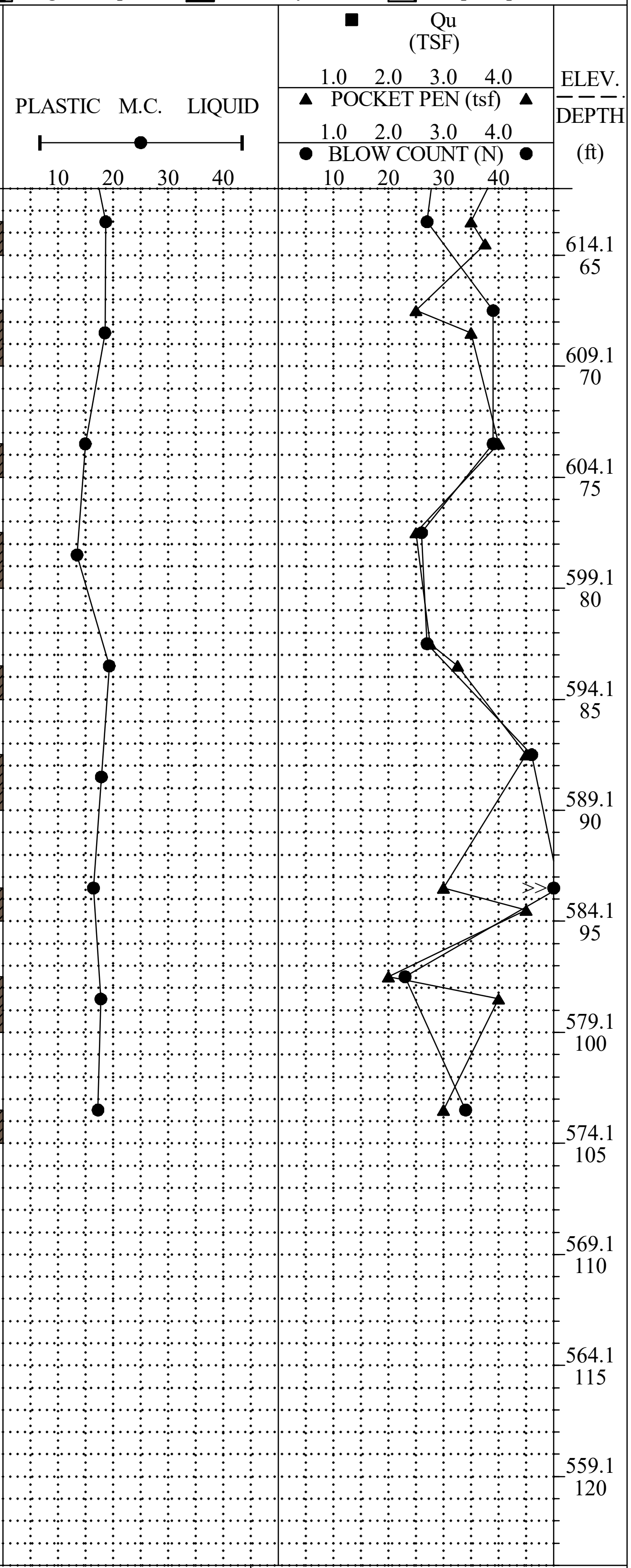
Borehole Abandonment
Date: **2/28/2023**
Material: **Bentonite**

Crew: **Geotest**
Rig: **Geoprobe 7822DT**
Method: **HSA**

Project:	**UPPER LAKE PARK**	Job No:	**20615-001**	Boring No:	**Boring 4**
Client:	**City of Port Washington**	Drilled By:	**Geotest**	Elevation:	**679.1'**
Location:	**Port Washington, WI**	Drilling Begun:	**2/13/23**	Drilling Completed:	**2/28/23**

SAMPLE TYPE: ▮ 1" Geoprobe ○ No Recovery G Grab Sample ▯ Auger Sample ■ 3" Shelby Tube ◣ 2" Split Spoon

ELEV. / DEPTH (ft)	SAMPLE NO.	SAMPLE TYPE	RECOVERY (in.)	SPT (N)	SOIL DESCRIPTION	USC
614.1 / 65	15	◣	18	27	...moist, very stiff, brown (7.5YR 4/2)	CL
	16	◣	6	39	...moist, very stiff, brown (7.5YR 4/2) partially drove stone	CL
609.1 / 70						
604.1 / 75	17	◣	16	39	...moist, hard, brown (7.5YR 4/2)	CL
	18	◣	18	26	...moist, very stiff, little fine to coarse sand, brown (7.5YR 4/2)	CL
599.1 / 80						
	19	◣	18	27		
594.1 / 85					...moist, very stiff, trace fine to coarse gravel, brown (7.5YR 4/2)	CL
	20	◣	16	46	...moist, hard, brown (7.5YR 4/2)	CL
589.1 / 90						
584.1 / 95	21	◣	8	51	...moist, very stiff to hard, trace silt, brown (7.5YR 4/2)	CL
	22	◣	18	23	...moist, very stiff, brown (7.5YR 4/2)	CL
579.1 / 100						
574.1 / 105	23	◣	18	34	...moist, very stiff to hard, brown (7.5YR 4/2)	CL
					Boring Terminated @ 105' Depth.	
569.1 / 110						
564.1 / 115						
559.1 / 120						



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	Water Level	Cave-in Depth
Date 2/13/2023 Time ______	15 ft.	______ ft.
Date ________ Time ______	______ ft.	______ ft.
Date ________ Time ______	______ ft.	______ ft.

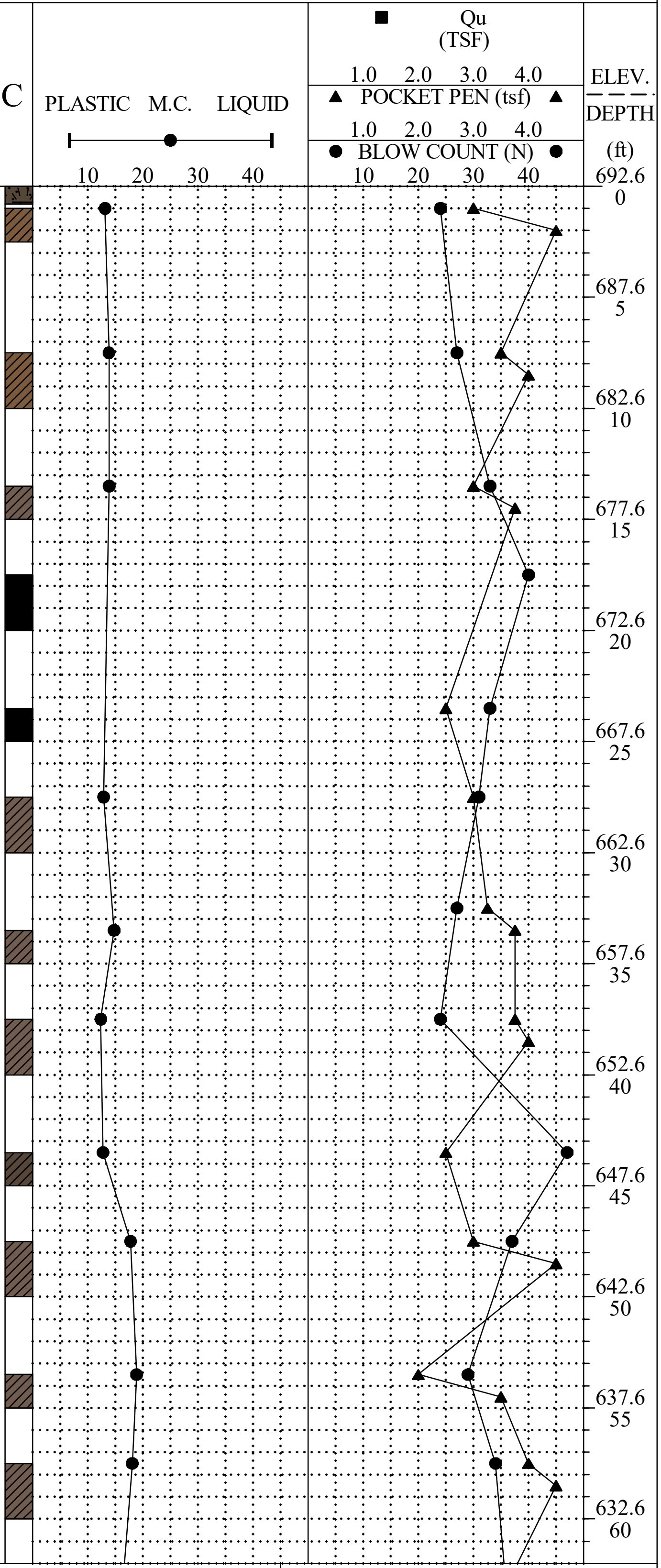
Borehole Abandonment
Date: **2/28/2023**
Material: **Bentonite**

Crew: **Geotest**
Rig: **Geoprobe 7822DT**
Method: **HSA**

Project: **UPPER LAKE PARK**	Job No: **20615-001**	Boring No: **Boring 5**
Client: **City of Port Washington**	Drilled By: **Geotest**	Elevation: **692.6'**
Location: **Port Washington, WI**	Drilling Begun: **3/4/23**	Drilling Completed: **3/4/23**

SAMPLE TYPE ▮ 1" Geoprobe ○ No Recovery Ⓖ Grab Sample ▮ Auger Sample ■ 3" Shelby Tube ◩ 2" Split Spoon

ELEV. / DEPTH (ft) 692.6	SAMPLE NO.	SAMPLE TYPE	RECOVERY (in.)	SPT (N)	SOIL DESCRIPTION	USC
0					TOPSOIL	
	1	◩	16	24	LEAN CLAY - moist, very stiff to hard, little fine to coarse sand, trace fine to coarse gravel, brown (7.5YR 4/4) with common mottles	CL
687.6 5						
	2	◩	10	27	...moist, very stiff to hard, few fine to coarse sand, brown (7.5YR 4/4) with common mottles	CL
682.6 10						
677.6 15	3	◩	18	33	...moist, very stiff to hard, trace fine to coarse sand, trace silt lens, brown (7.5YR 4/2)	CL
	4	◩	6	40	...no recovery	
672.6 20						
667.6 25	5	◩	10	33	...no recovery	
	6	◩	16	31	...moist, few fine to coarse sand, brown (7.5YR 4/2) drove stone, ribbon sample	CL
662.6 30						
657.6 35	7	◩	18	27	...moist, very stiff, few fine to coarse sand, trace silt lens, trace fine sand seam, brown (7.5YR 4/2)	CL
652.6 40	8	◩	18	24	...moist, very stiff, little fine to coarse sand, brown (7.5YR 4/2) with trace mottles	CL
647.6 45	9	◩	3	47	...moist, little fine to coarse sand, dark brown (7.5YR 3/2) drove stone, ribbon sample	CL
642.6 50	10	◩	18	37	...moist, very stiff to hard, trace fine to coarse sand, brown (7.5YR 4/2)	CL
637.6 55	11	◩	18	29	...moist, very stiff, brown (7.5YR 4/2)	CL
632.6 60	12	◩	18	34	...moist, hard, brown (7.5YR 4/2)	CL



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	Water Level	Cave-in Depth
Date ________ Time ______	______ft.	_______ ft.
Date ________ Time ______	______ft.	_______ ft.
Date ________ Time ______	______ft.	_______ ft.

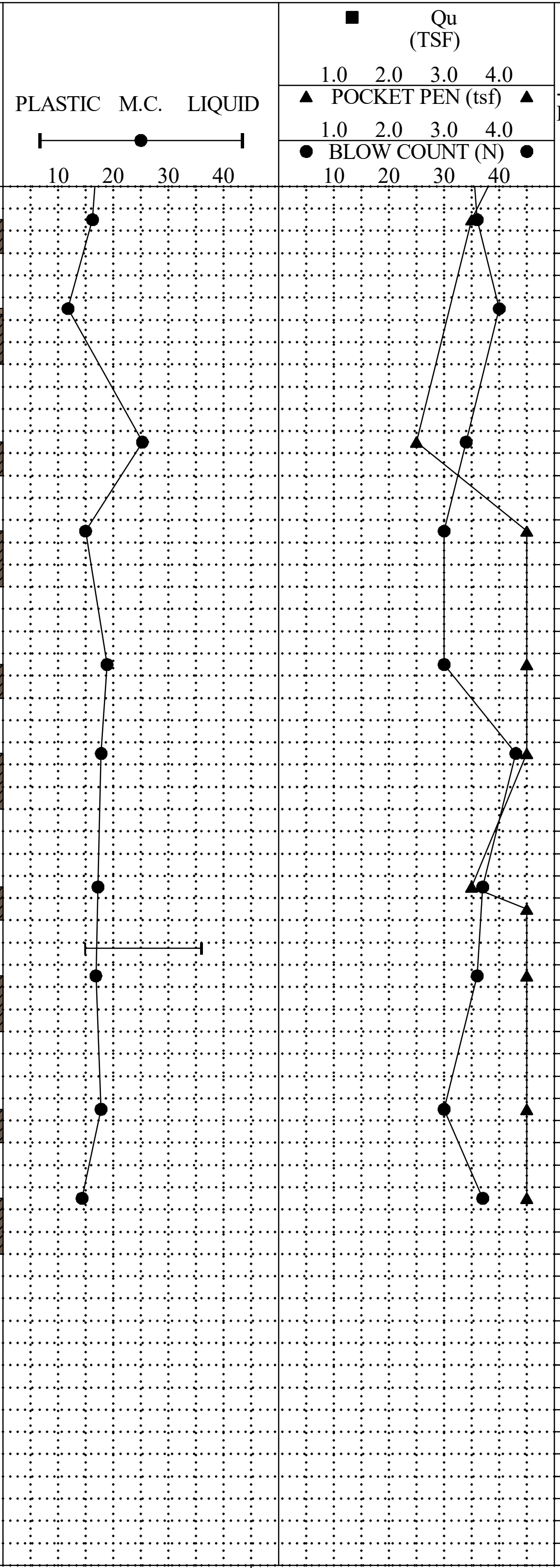
Borehole Abandonment
Date: **3/4/2023**
Material: **Well Casing**

Crew: **Geotest**
Rig: **Geoprobe 7822DT**
Method: **Mud Rotary 6"**

Project: **UPPER LAKE PARK**	Job No: **20615-001**	Boring No: **Boring 5**
Client: **City of Port Washington**	Drilled By: **Geotest**	Elevation: **692.6'**
Location: **Port Washington, WI**	Drilling Begun: **3/4/23**	Drilling Completed: **3/4/23**

SAMPLE TYPE ▮ 1" Geoprobe ○ No Recovery G Grab Sample ▮ Auger Sample ■ 3" Shelby Tube ◩ 2" Split Spoon

ELEV. / DEPTH (ft)	SAMPLE NO.	SAMPLE TYPE	RECOVERY (in.)	SPT (N)	SOIL DESCRIPTION	USC
627.6 / 65	13	◩	18	36	...moist, very stiff to hard, brown (7.5YR 4/2)	CL
	14	◩	18	40	POORLY GRADED SAND SEAM - moist, dense, fine grain, trace silt, brown (10YR 5/3) underlain by thin wet sandy silt seam	SP
622.6 / 70					LEAN CLAY - moist, stiff, trace fine to coarse sand and gravel, brown (7.5YR 4/2)	CL
617.6 / 75	15	◩	18	34	...moist, very stiff to hard, few silt, brown (7.5YR 4/2)	CL
	16	◩	16	30	...moist, hard, brown (7.5YR 4/2)	CL
612.6 / 80						
607.6 / 85	17	◩	18	30	...moist, hard, brown (7.5YR 4/2)	CL
	18	◩	16	43	...moist, hard, brown (7.5YR 4/2)	CL
602.6 / 90						
597.6 / 95	19	◩	18	37	...moist, hard, brown (7.5YR 4/2)	CL
	20	◩	16	36	...moist, hard, brown (7.5YR 4/2)	CL
592.6 / 100						
587.6 / 105	21	◩	12	30	...moist, hard, brown (7.5YR 4/2)	CL
	22	◩	16	37	...moist, hard, brown (7.5YR 4/2)	CL
582.6 / 110					Boring Terminated @ 110' Depth.	
577.6 / 115						
572.6 / 120						



ELEV. / DEPTH (ft)
627.6 65
622.6 70
617.6 75
612.6 80
607.6 85
602.6 90
597.6 95
592.6 100
587.6 105
582.6 110
577.6 115
572.6 120

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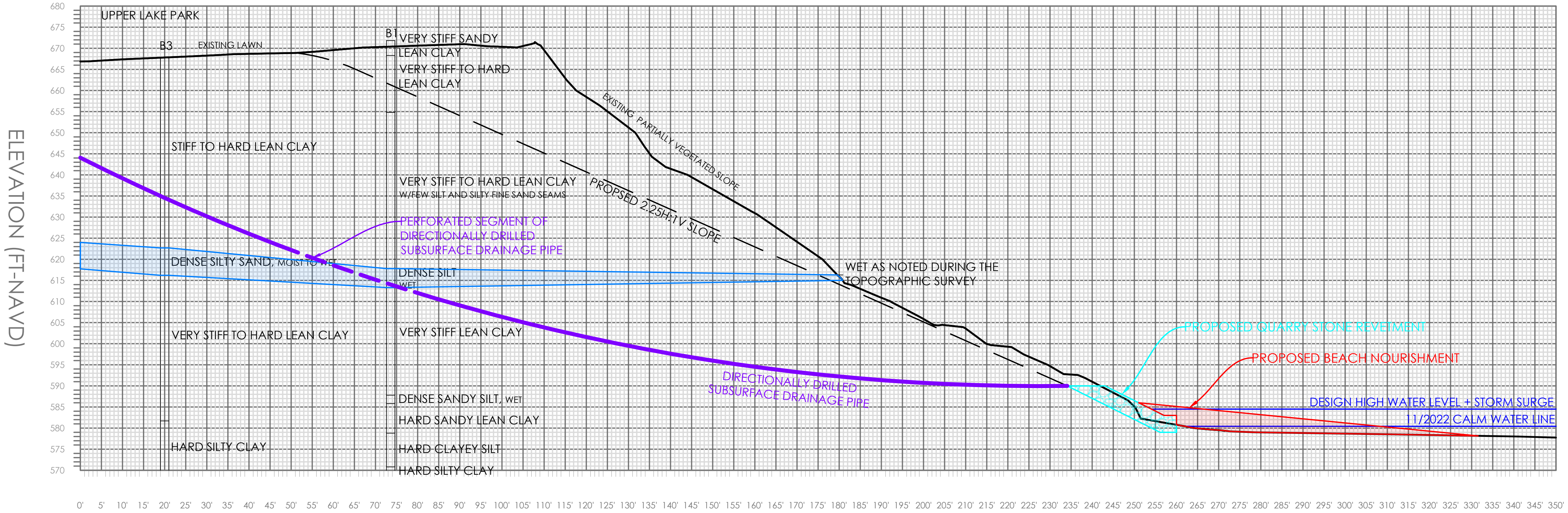
	Water Level	Cave-in Depth
Date ________ Time ______	______ft.	______ ft.
Date ________ Time ______	______ft.	______ ft.
Date ________ Time ______	______ft.	______ ft.

Borehole Abandonment
Date: **3/4/2023**
Material: **Well Casing**

Crew: **Geotest**
Rig: **Geoprobe 7822DT**
Method: **Mud Rotary 6"**

BLUFF CROSS SECTION A-A' (B1 THROUGH B3)



BLUFF CROSS SECTION A-A'

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NO.	DATE	DESCRIPTION	BY

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498 N LAKE STREET
PORT WASHINGTON, WI

SCALE
HOR. 1"=30'
VER. 1"=30'

DATE
03/24/2023

JOB
20615-B

BY
EBB

CK
RGM

SHEET
1 OF 3

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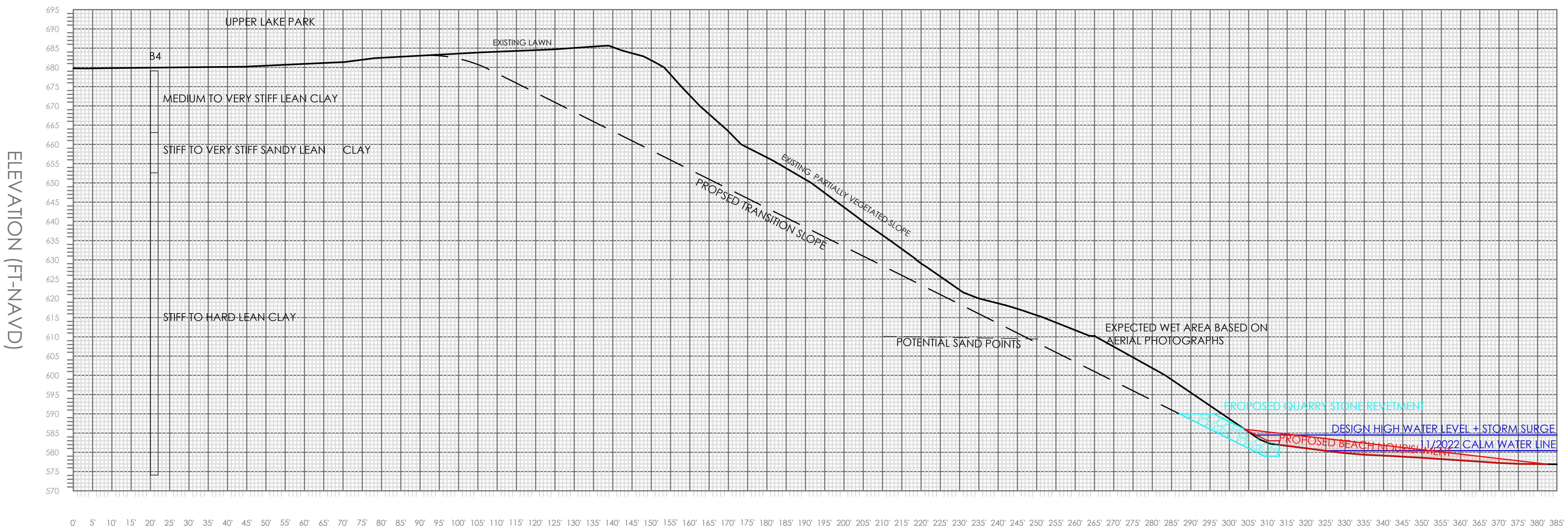
BLUFF CROSS SECTION B-B'



BLUFF CROSS SECTION B-B'

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NO.	DATE	DESCRIPTION	BY

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498 N LAKE STREET
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SHEET
2 OF 3

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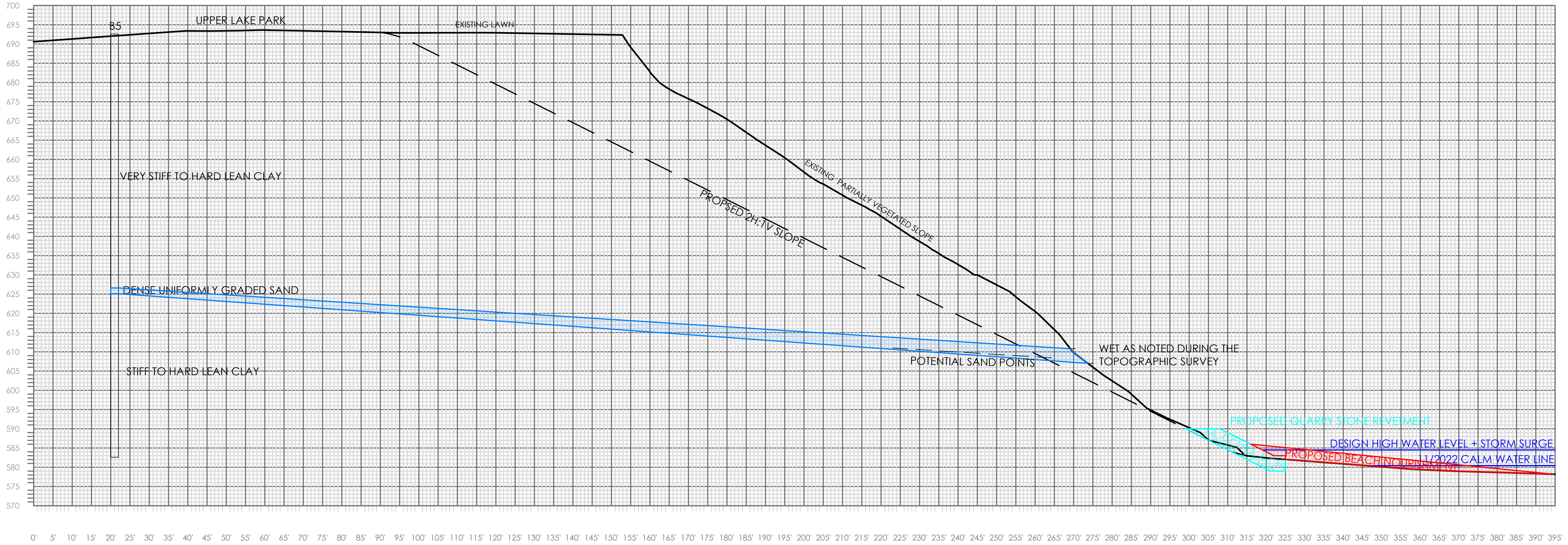
BLUFF CROSS SECTION C-C'



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BLUFF CROSS SECTION C-C'

SHEET
3 OF 3

I:\DATA\20600\20615 - Port Washington Upper Lake Park\CAD\DESIGN\20615 - B Bluff Stability.dwg 3/24/2023 2:24 PM

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Client: CITY OF PORT WASHINGTON Project: NORTH BEACH & BLUFF
Subject: BLUFF SHALLOW SLIDING ANALYSIS Project No.: 20615-001
By: RBM Date: 3/21/23 Checked By: ECB Date: 3/24/2023 Page No.: 1/4

REFER TO FIGURE 10 - STABILITY CHART FOR INFINITE SLOPES

Ⓘ DESIGN SLOPES RANGING FROM 2¼ H : 1V AT SOUTH END TO 2H : 1V ONWARD NORTH

$\therefore$ $2.0 \leq \cot\beta \leq 2.25$

SHALLOW SLIDING OCCURS AS THE FIRM CLAY COMPRISING A BLUFF CUT SLOPE "WEATHERS" (softens) OVER TIME DUE TO SEASONAL WETTING - DRYING AND FREEZING - THAWING. SO USE A "RESIDUAL" ANGLE OF SOIL INTERNAL FRICTION: $20° \leq \phi'_{res} \leq 23°$, AND "COHESION" (c') DIMINISHES FROM "VERY STIFF" TO "MEDIUM", THEN "SOFT"

ALTHOUGH THE UNWEATHERED V.STIFF CLAY IS ~ 140 pcf = γ_s, IT "WEATHERS" TO ~135 pcf

SO FOR $2.0 \leq \cot\beta \leq 2.25$ $\therefore$ $\tan\bar{\beta} = 0.47$
$\phi'_{res} \sim 21.5°$ $\gamma_s \sim 135$ pcf

THE RATIO OF FORCES RESISTING SLIDING DIVIDED BY THE SLIDING FORCES IS DEFINED AS THE "FACTOR OF SAFETY" (F)

$$F = A\frac{\tan\phi'}{\tan\beta} + B\frac{c'}{\gamma_s H}$$

SEE FIG 10 (ATTACHED)

TO DERIVE A & B BASED ON r_u (pore pressure ratio) AND $\cot\beta$.

$\bar{r}_u \sim 0.43 \Rightarrow A = 0.47$

AND $B = 2.6$

$$F = 0.47\,(\tan 21.5°)/\tan\bar{\beta} + 2.6\left(\frac{c'}{\gamma_s H}\right)$$

$$F = 0.47(0.394)/(0.47) + 2.6\left(\frac{c'}{\gamma_s H}\right)$$

$$\boxed{F = \sim 0.4 + 2.6\left(\frac{c'}{\gamma_s H}\right)}$$

F (FACTOR OF SAFETY) FOR $\cot\bar{\beta} = 2\frac{1}{8}$ & FOR $\gamma_s = 135$ pcf; $F = 0.4 + \left(\frac{c'}{52H}\right)$:

H (ft)	V.STIFF c' = 3600 PSF	STIFF c' = 1500	MEDIUM c' = 750	SOFT c' = 250
2	35	14.8	7.6	2.8
4	18	7.6	4.0	1.6
6	12	5.2	2.8	1.2
8	9	4.0	2.2	1.0
12	6.2	2.8	1.6	0.8
24	3.2	1.2	1.0	
48	1.8	1.0	0.7	
96	1.1	0.7		

COMMON DEPTHS OF SHALLOW SLIDING

SLOPE VEG. ROOT DEPTH

F ~ 2

INCIPIENT FAILURE

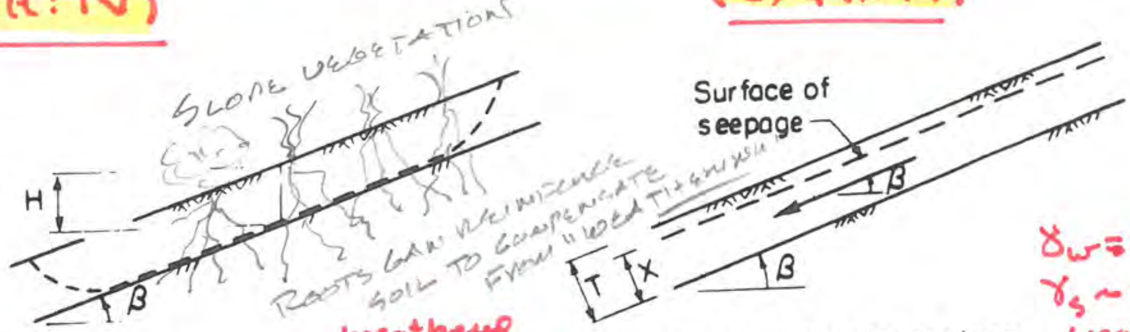
→ IS CRITICAL TO COMPENSATE FOR THE INEVITABLE SHALLOW SOIL WEATHERING OVER TIME.

SO MOST SHALLOW SLIDES OCCUR DURING SPRING THAW & SATURATION AS SOIL "SOFTENS" BELOW A SHEAR STRENGTH (c') LESS THAN 200 PSF, OFTEN AT ABOUT 150 PSF

SHALLOW SLIDING ANALYSIS

cot $\beta = 2.0 \leq$ **DESIGN SLOPES** $\leq$ cot $\beta = 2\frac{1}{4}$

(2H:1V) (2¼H:1V)



weathered

γ_s = total unit weight of soil ~ 135 pcf

γ_W = unit weight of water = 62.4 pcf

c′ = cohesion intercept } Effective Stress

$20^\circ \leq \phi_{res.} \leq 23^\circ$ φ′ = friction angle } Effective Stress

r_u = pore pressure ratio = $\frac{u}{\gamma H}$

u = pore pressure at depth H

Seepage parallel to slope

$$r_u = \frac{X}{T}\,\frac{\gamma_W}{\gamma_s}\cos^2\beta$$

$\gamma_W = 62.4$ pcf

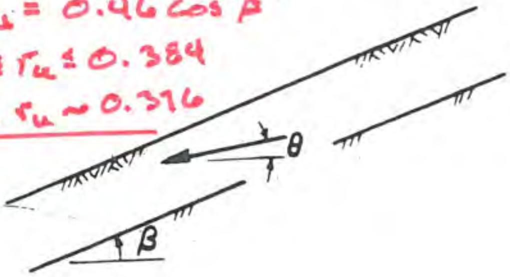
$\gamma_s \sim 135$ pcf weathered

Use X = T AS WORST CASE

$r_u = 0.46\cos^2\beta$

$0.368 \leq r_u \leq 0.384$

use $r_u \sim 0.376$



Steps:

① Determine r_u from measured pore pressures or formulas at right

② Determine A and B from charts below

③ Calculate $F = \overset{0.47}{\bar{A}}\,\frac{\tan\phi'}{\tan\bar{\beta}} + B\,\frac{C'}{\gamma_s H}$ 0.384

$\tan\bar{\beta} = 0.470$
↑
25°

Seepage emerging from slope

$$r_u = \frac{\gamma_W}{\gamma}\,\frac{1}{1+\tan\beta\tan\theta}$$ ← use θ = zero ∴ tan θ = zero

↑
$\bar{\beta} = 25^\circ$

$\bar{r}_u = 0.46\left(\frac{1}{1}\right) = 0.46 = \bar{r}_u$

$F \approx 0.4 + 2.6\left(\frac{c'}{\gamma_s H}\right)$

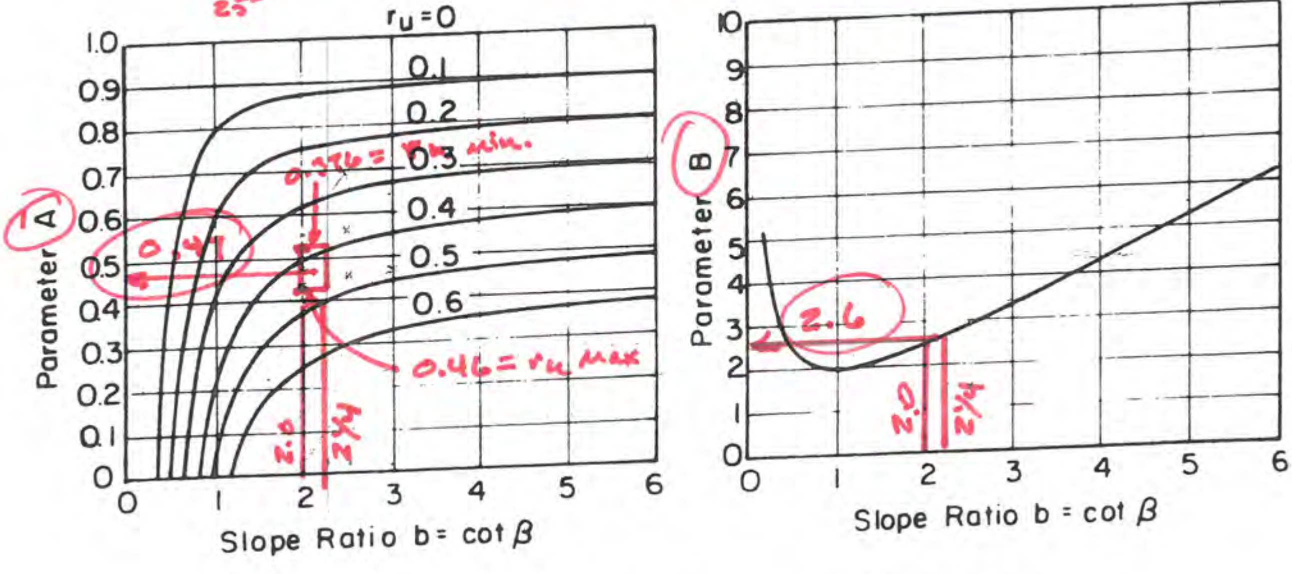


Fig. 10 STABILITY CHARTS FOR INFINITE SLOPES

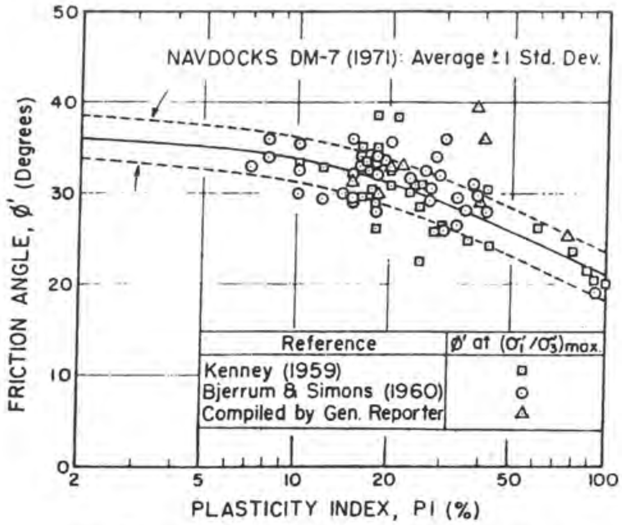


Fig. 14 Empirical correlation between friction angle and plasticity index from triaxial compression tests on normally consolidated undisturbed clays.

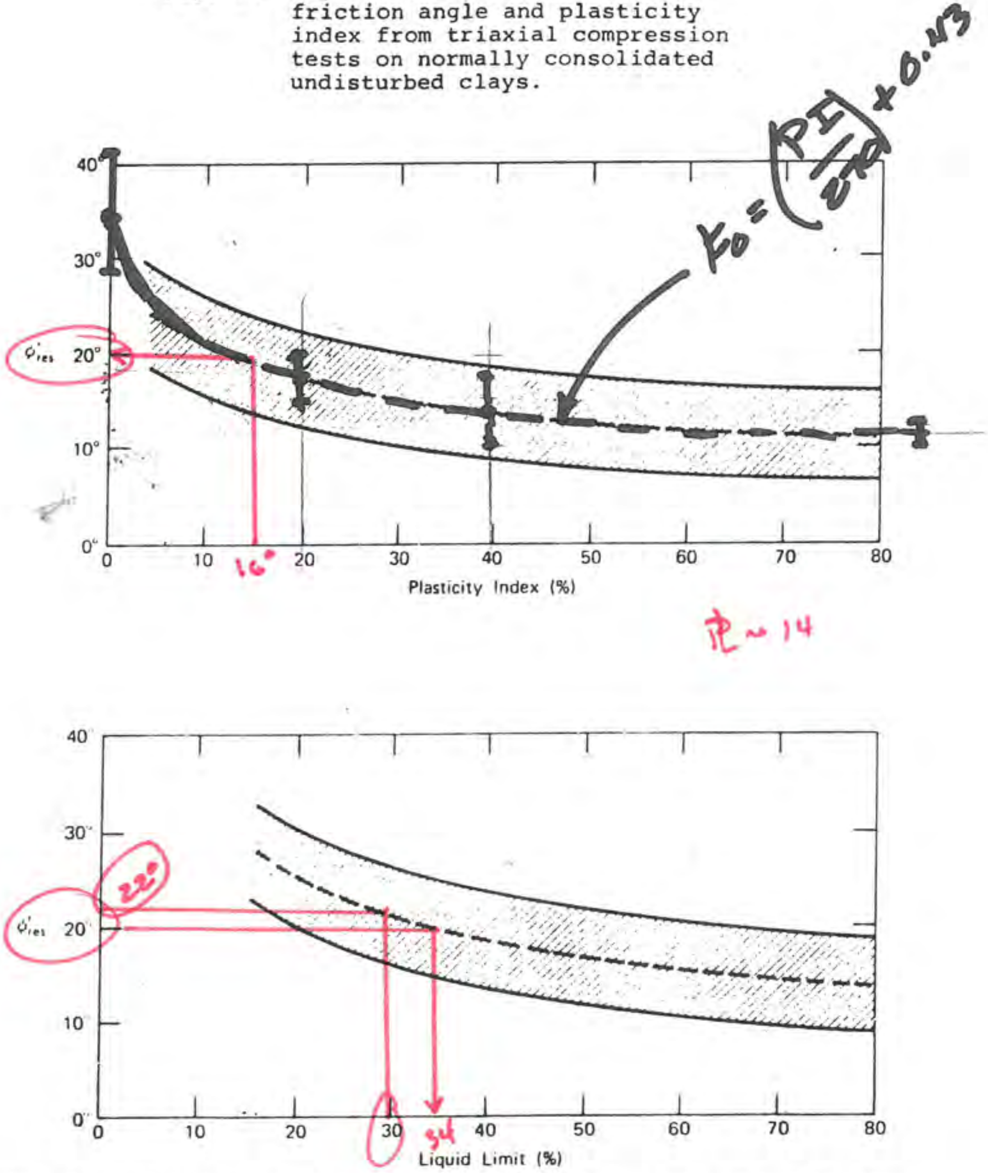


Fig. 15 Relationship between residual friction angle and plasticity (Compiled by Deere, 1974).

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LOG OF TEST BORING
GENERAL NOTES

Descriptive Soil Classification

GRAIN SIZE TERMINOLOGY

Soil Fraction		Particle Size	U.S. Sieve Size
Boulders		Larger Than 12"	Larger Than 12"
Cobbles		3" to 12"	3" to 12"
Gravel:	Coarse	3/4" to 3"	3/4" to 3"
	Fine	4.76mm to 3/4"	#4 to 3/4"
Sand:	Coarse	2.00mm to 4.76mm	#10 to #4
	Medium	0.42mm to 2.00mm	#40 to #10
	Fine	0.074mm to 0.42mm	#200 to #40
Fines		Less Than 0.074mm	Smaller Than #200
Silt		0.005mm to 0.074mm	Smaller Than #200
Clay		Smaller Than 0.005mm	

(Plasticity characteristics differentiate between silt and clay.)

COMPOSITION TERMINOLOGY (ASTM D2487)

Primary Constituent:

Gravel	Sand	Fines (Silt or Clay)
with sand...>=15% sand	with gravel.....>=15% gravel	with gravel....15-29% gravel
with silt.......5-12% silt	with silt...........5-12% silt	gravelly..........>=30% gravel
with clay.....5-12 clay	with clay..........5-12% clay	with sand......15-29% sand
silty............>12% silt	silty.................>12% silt	sandy............>=30%sand
clayey........>12% clay	clayey............>12% clay	

RELATIVE DENSITY
COHESIONLESS SOILS

Term	"N" Value
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	over 50

The penetration resistance, N, is the summation of the number of blows required to affect two successive 6" penetrations of the 2" split-barrel sampler. The sampler is driven with a 140 lb. weight falling 30" and is seated to a depth of 6" before commencing the standard penetration test (ASTM 1586).

CONSISTENCY
COHESIVE SOILS

Term	pp (tons/sq. ft.)	"N" Value
Very Soft	0.00 to 0.25	<2
Soft	0.25 to 0.50	2-4
Medium	0.50 to 1.00	4-8
Stiff	1.00 to 2.00	8-15
Very Stiff	2.00 to 4.00	15-30
Hard	over 4.00	>30

PLASTICITY

Term	Plasticity Index
None to slight	0 to 4
Slight	5 to 7
Medium	8 to 22
High to Very High	over 22

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SYMBOLS

DRILLING AND SAMPLING

CS--Continuous Sampling
RC--Rock Coring: Size AW, BW, NW, 2" W
RQD--Rock Quality Designator
RB--Rock Bit
FT--Fish Tail
DC--Drove Casing
C--Casing: Size 2 1/2", NW, 4", HW
CW--Clear Water
DM--Drilling Mud
HSA--Hollow Stem Auger
FA--Flight Auger
HA--Hand Auger
SS--2" Diameter Split-Barrel Sample
2ST--2" Diameter Thin-Walled Tube Sample
3ST--3" Diameter Thin-Walled Tube Sample
PT--3" Diameter Piston Tube Sample
AS--Auger Sample
PS--Pitcher Sample
NR--No Recovery
VS--Vane Shear Test

LABORATORY TESTS

pp--Penetrometer Reading, tons/sq.ft.
qu--Unconfined Strength, tons/sq. ft.
MC--Moisture Content, %
LL--Liquid Limit, %
PL--Plastic Limit, %
PI--Plasticity Index, %
SL--Shrinkage Limit, %
LI--Loss on Ignition, %
D--Dry Unit Weight, lbs./cu. ft.
pH--Measure of Soil Alkalinity or Acidity
FS--Free Swell, %
HNu--ppmv as Benzene
TLV--ppmv as Hexane
TPH--Total Petroleum Hydrocarbons, ppm

WATER LEVEL MEASUREMENTS

▽--Water Table Interpretation

Note: Water level measurements recorded in notes on the boring logs represent conditions at the time indicated and may not reflect static levels, especially in cohesive soils.

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Subject: BLUFF SHALLOW SLIDING ANALYSIS Project No.: 20615-001
By: RGM Date: 3/21/23 Checked By: EBB Date: 3/24/2023 Page No.: 4/3

(II) SHALLOW SLIDING ANALYSIS FOR RANGE OF COMMON SLOPES: 1H:1V TO 3H:1V

$1 \le \cot\beta \le 3$

PARAMETER (A) WITH SEEPAGE PARALLEL TO SLOPE WHICH IS COMMON

PORE PRESSURE RATIO $\left(r_u = \frac{X}{T}\,\frac{\gamma_w}{\gamma_s}\cos^2\beta\right)$ WHERE $X = T$ IS WORST BUT COMMON CONDITION IN SPRING.

cot β	cos²β	r_u	(A) FROM FIG 10
1.0	0.50	0.23	0.55
1.5	0.69	0.32	0.54
1.75	0.75	0.35	
2.0	0.80	0.37	0.54
2.25	0.835	0.39	
2.5	0.863	0.40	
3.0	0.90	0.42	0.54

SO (A) IS CLOSE TO A CONSTANT THAT CAN CONSERVATIVELY BE APPROXIMATED @ 0.5

$\gamma_w = 62.4$ pcf

$r_u = \frac{62.4}{\gamma_s}\cos^2\beta$

FOR γ_s WEATHERED ~135 pcf

$r_u \sim 0.46\cos^2\beta$

$$\therefore F \sim 0.5\left(\frac{\tan\bar{\phi}'_{res}}{\tan\beta}\right) + B\left(\frac{c'}{\gamma_s H}\right),\ \text{for } \bar{\phi}'_{res} = 21.5° :$$

$$F = 0.5(0.394)(\cot\beta) + (0.725\cot\beta + 1.08)\left(\frac{c'}{\gamma_s H}\right)$$

$$F = 0.2\cot\beta + (0.725\cot\beta + 1.08)\left(\frac{c'}{\gamma_s H}\right)\ \text{FOR } 1 \le \cot\beta \le 3$$

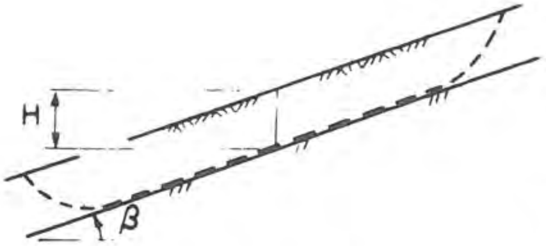
cot β		$\gamma_s = 135$ pcf $H = 2'$, $c' = 125$ ↳ $k_1 = 0.46$	$\gamma_s = 135$ pcf $H = 6'$, $c = 500$ psf ↳ $k_1 = 0.62$
1.0	$F = 0.2 + 1.8\,(c'/\gamma_s H)$:	F = 1.0	F = 1.3
1.5	$F = 0.3 + 2.2\,(c'/\gamma_s H)$:	F = 1.3	F = 1.7
2.0	$F = 0.4 + 2.5\,(c'/\gamma_s H)$:	F = 1.5	F = 2.0
2.5	$F = 0.5 + 2.9\,(c'/\gamma_s H)$:	F = 1.8	F = 2.3
3.0	$F = 0.6 + 3.2\,(c'/\gamma_s H)$:	F = 2.1	F = 2.6

$\underbrace{c'/\gamma_s H}$ "constant" = k_1

SEE GRAPH ON NEXT PG

II

2/3



γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept } Effective Stress

ϕ' = friction angle } Effective Stress

$\bar{\phi}'_{res} \sim 21.5°$

r_u = pore pressure ratio = $\frac{u}{\gamma H}$

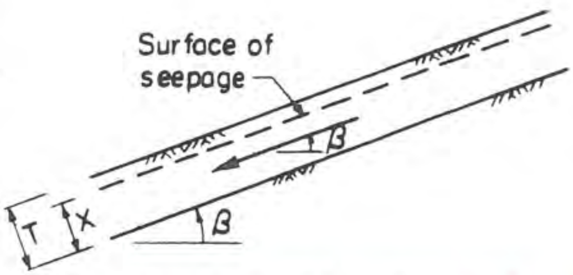
u = pore pressure at depth H

Steps:

① Determine r_u from measured pore pressures or formulas at right

② Determine A and B from charts below

③ Calculate $F = A \frac{\tan\phi'}{\tan\beta} + B \frac{C'}{\gamma H}$

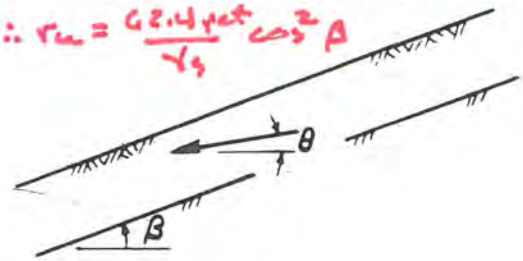


Seepage parallel to slope = COMMON CONDITION

$r_u = \frac{X}{T} \frac{\gamma_w}{\gamma_s} \cos^2\beta$

& X = T IS COMMON SPRING CONDITION

$\therefore r_u = \frac{62.4 \text{ pcf}}{\gamma_s} \cos^2\beta$



Seepage emerging from slope

$r_u = \frac{\gamma_w}{\gamma} \frac{1}{1 + \tan\beta \tan\theta}$

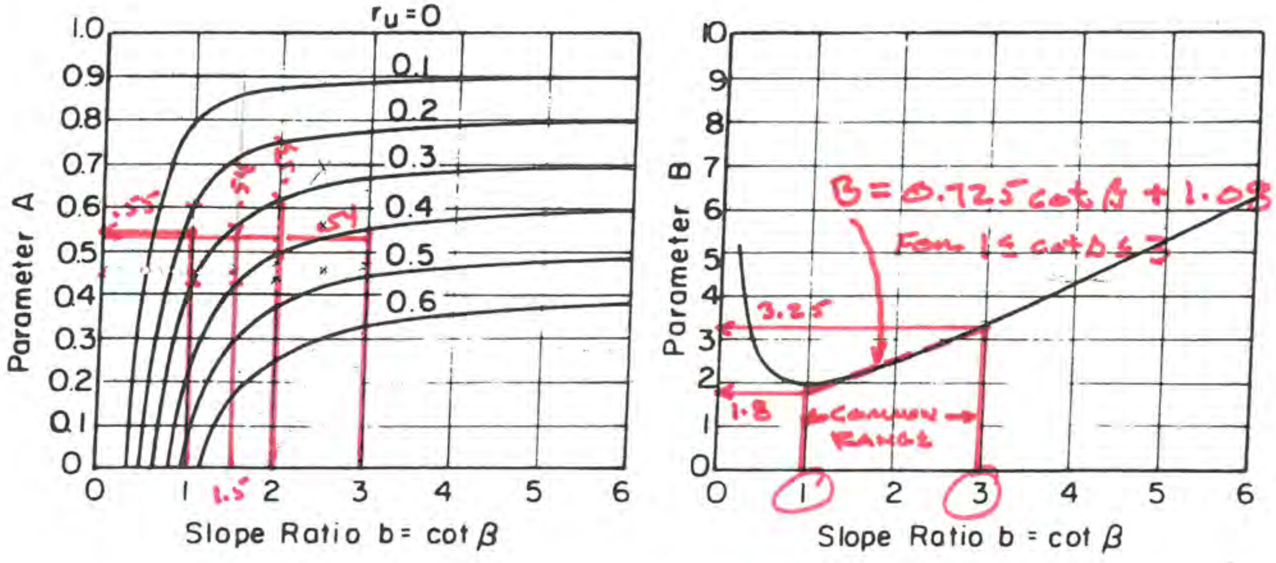


Fig. 10 STABILITY CHARTS FOR INFINITE SLOPES.

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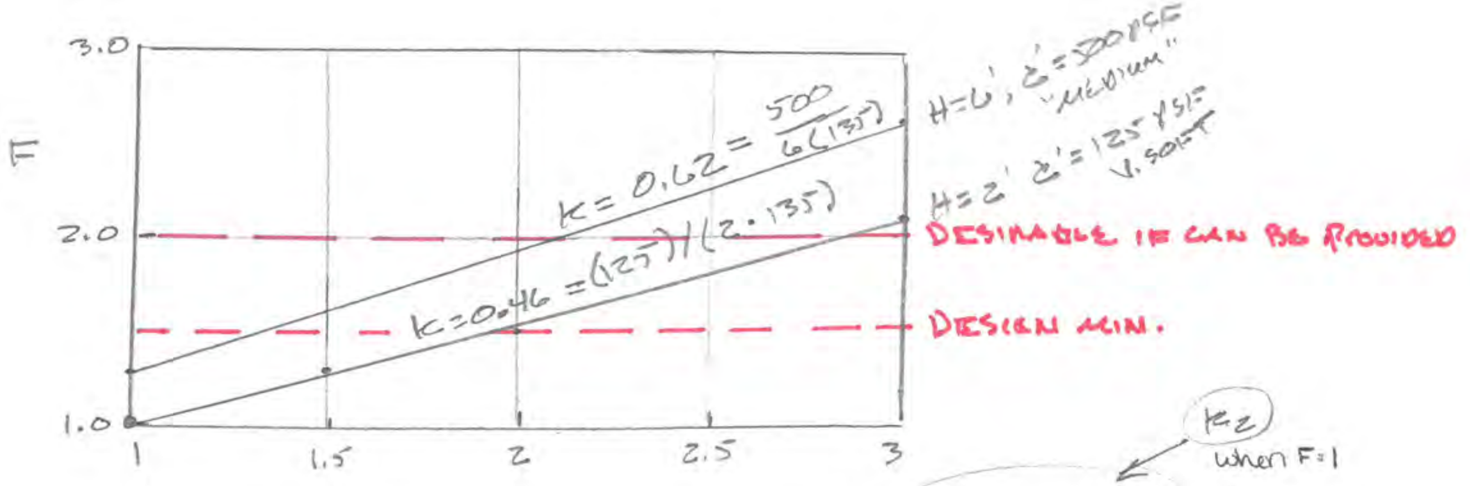
Client: City of Port Washington Project: N. Beach & Bluff

Subject: Bluff Shallow Sliding Analysis Project No.: 20615-001

By: EGM Date: 3/21/23 Checked By: EBB Date: 3/24/2023 Page No.: 3/3



K_z when F = 1

$$\text{SHALLOW SLIDING THICKNESS } (H) = \frac{(0.725\cot\beta + 1.05)}{(1 - 0.2\cot\beta)}\left(\frac{c'}{\gamma_s}\right)$$

$\gamma_s \sim 135$ pcf weathered

		V. soft	soft	medium		STIFF
K_z	cot β	c' = 125 psf	250	500	1000	1500
2.256	1.0	2.1'	4.2'	8.4'	16.7'	25'
3.096	1.5	2.9'	5.7'	11.4'	23'	34'
4.217	2.0	3.9'	7.8'	15.6'	31'	47'
5.785	2.5	5.3'	10.7'	21'	43'	64'
8.138	3.0	7.5'	15.0	30'	60'	90'

2'

3'

4'

6'

8'

10'

20'

H

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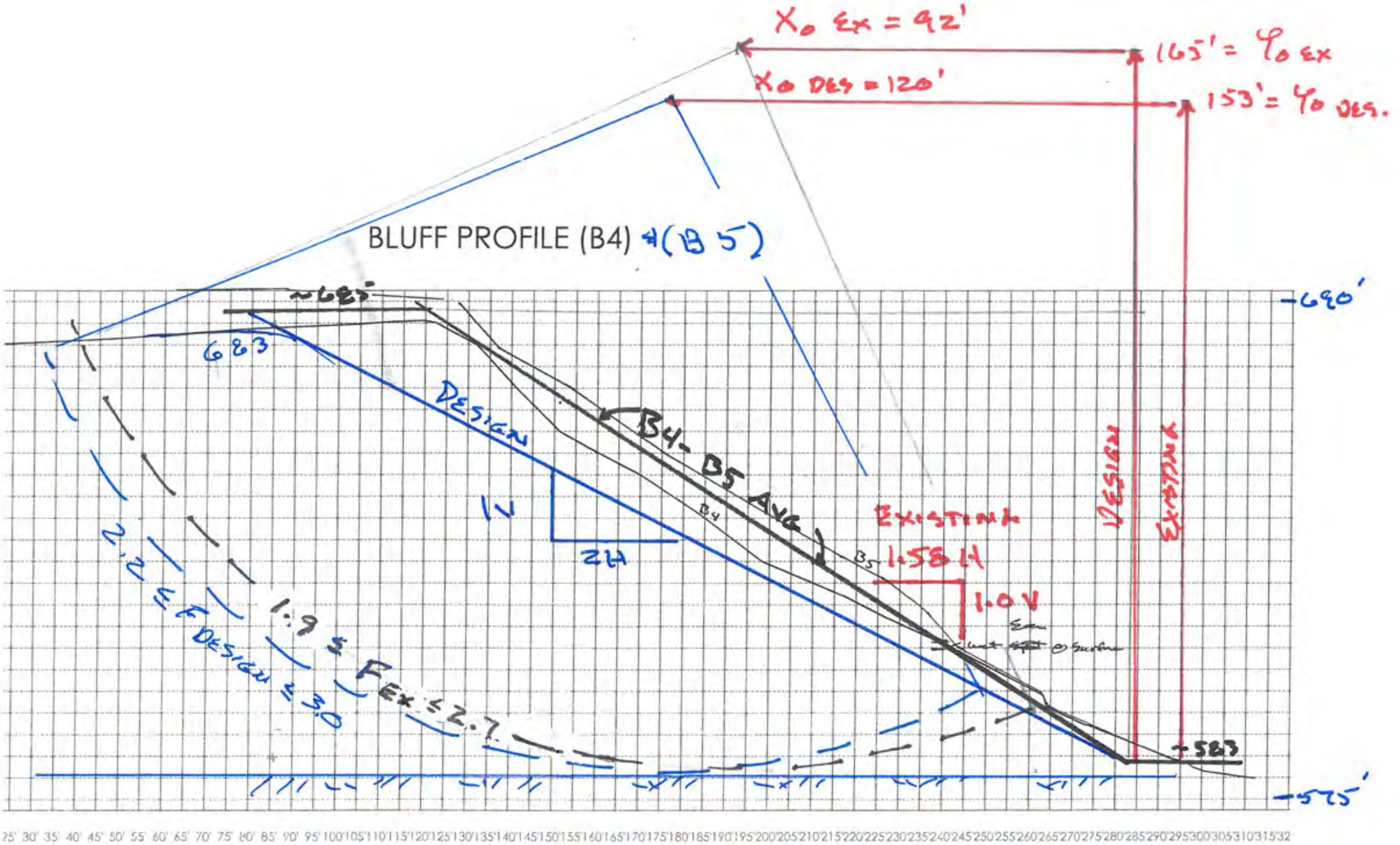
Client: CITY OF PORT WASHINGTON Project: NORTH BEACH & BLUFF
Subject: DEEP SEATED SLOPE STABILITY ANALYSIS Project No.: 20615-001
By: RGM Date: 3/21/22 Checked By: EBB Date: 3/24/2023 Page No.: 3/3



$\sigma_{1\,max\,EX} = 91'(140) = 12,740$ PSF = 88 psi

$\sigma_{1\,max\,DES} = 76'(140) = 10,640$ PSF = 74 psi

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Client: CITY OF PORT WASHINGTON Project: NORTH BEACH & BLUFF

Subject: DEEP SEATED SLOPE STABILITY ANALYSIS Project No.: 20615-001

By: RGM Date: 3/22/23 Checked By: EBB Date: 3/24/2023 Page No.: Y4

(III) DEEP SEATED SLOPE STABILITY (EXISTING & DESIGN)

SOUTH END (B1, B2, B3) EXISTING

$H_{EXISTING} = 672' - 586' = 86'$ $\cot\beta = 1.5$ $(1.5H : 1V)$

"FACTOR OF SAFETY" $(F) = N_o \frac{c}{P_d}$, $P_d = \gamma_s \cdot H = 140(86)$

N_o from FIG. 6 = 7.4 $P_d = 12{,}040$

$\bar{S}_u = \bar{c} = 4267$ PSF (USE UU ENVELOPE - SEE "TRIAX. TEST" FIGURE)

$S_{u_{nominal}} = c_n = 3333$ PSF

$$\bar{F} = \frac{7.4(4267)}{12040} = 2.62, \quad F_{nominal} = \frac{7.4(3333)}{12040} = 2.05$$

WHICH IS WHY DEEP SEATED FAILURES HAVEN'T OCCURRED HERE

SOUTH END (B1, B2, B3) DESIGN

$H_{DESIGN} = 668 - 586 = 82'$, $\cot\beta = 2\,1/4$ $(2.25H : 1V)$

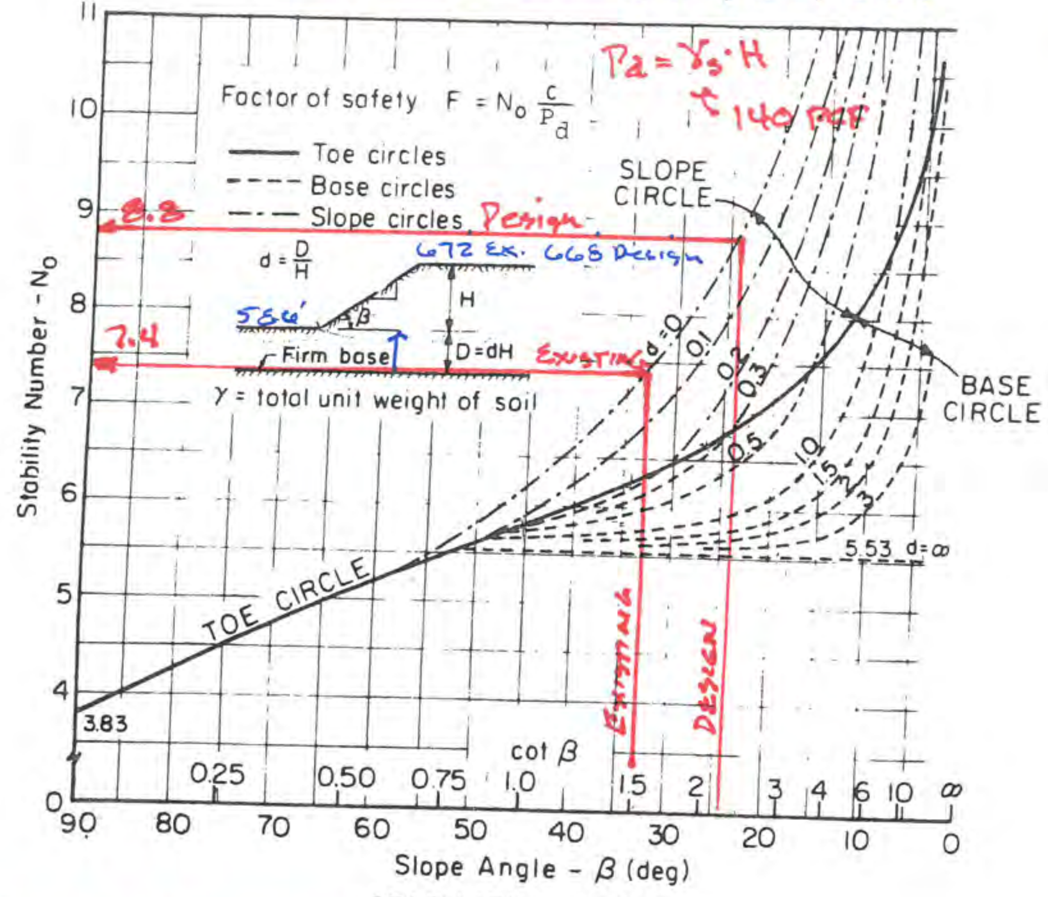
N_o FROM FIG 6 = 8.8 $P_d = 140(82) = 11480$

$$\bar{F} = \frac{8.8(4267)}{11480} = 3.27 \quad F_{nominal} = \frac{8.8(3333)}{11480} = 2.55$$

BECAUSE 2.0 IS A MORE THAN ADEQUATE FACTOR OF SAFETY, THE SLOPE IS BEING CUT BACK TO MITIGATE SHALLOW SLIDING AS DESCRIBED BY SECTIONS I & II OF THIS CALCULATION.

DEEP SEATED SLOPE STABILITY ANALYSIS
@ SOUTH END (B1, B2, B3)

III
2/4



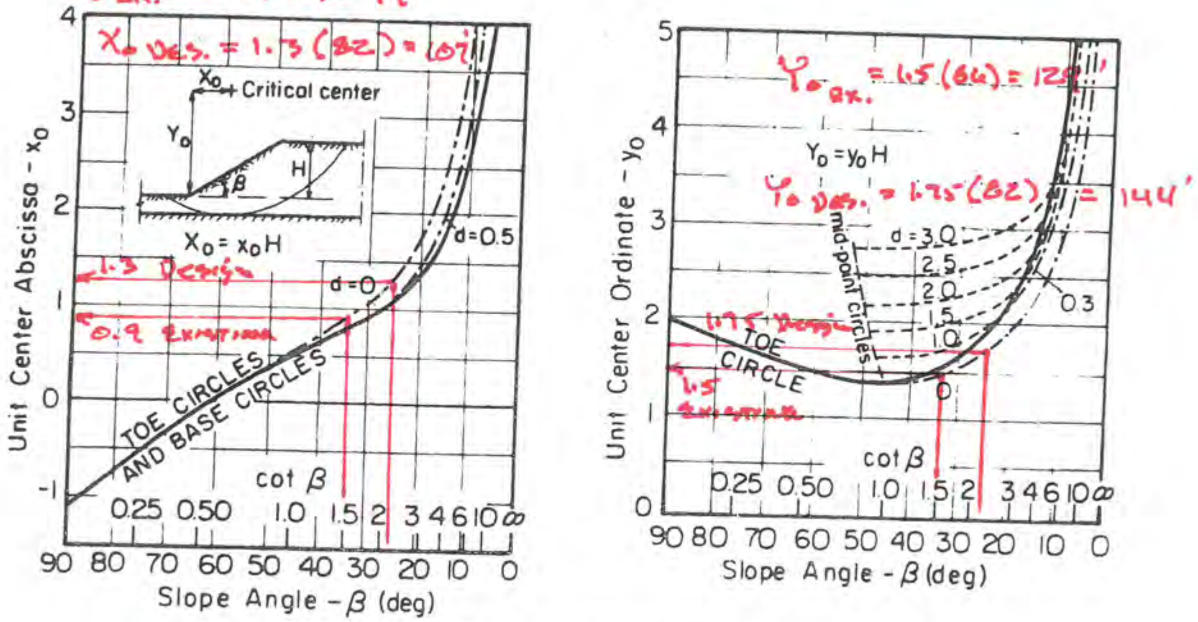
STABILITY NUMBER

$X_{o\,EX.}$ = 0.9 (86) = 77'

$X_{o\,DES.}$ = 1.3 (82) = 107'

$Y_{o\,EX.}$ = 1.5 (86) = 129'

$Y_{o\,DES.}$ = 1.75 (82) = 144'



CENTER COORDINATES FOR CRITICAL CIRCLE

Fig. 6 SLOPE STABILITY CHARTS FOR φ = 0 SOILS (after Janbu, 1968)

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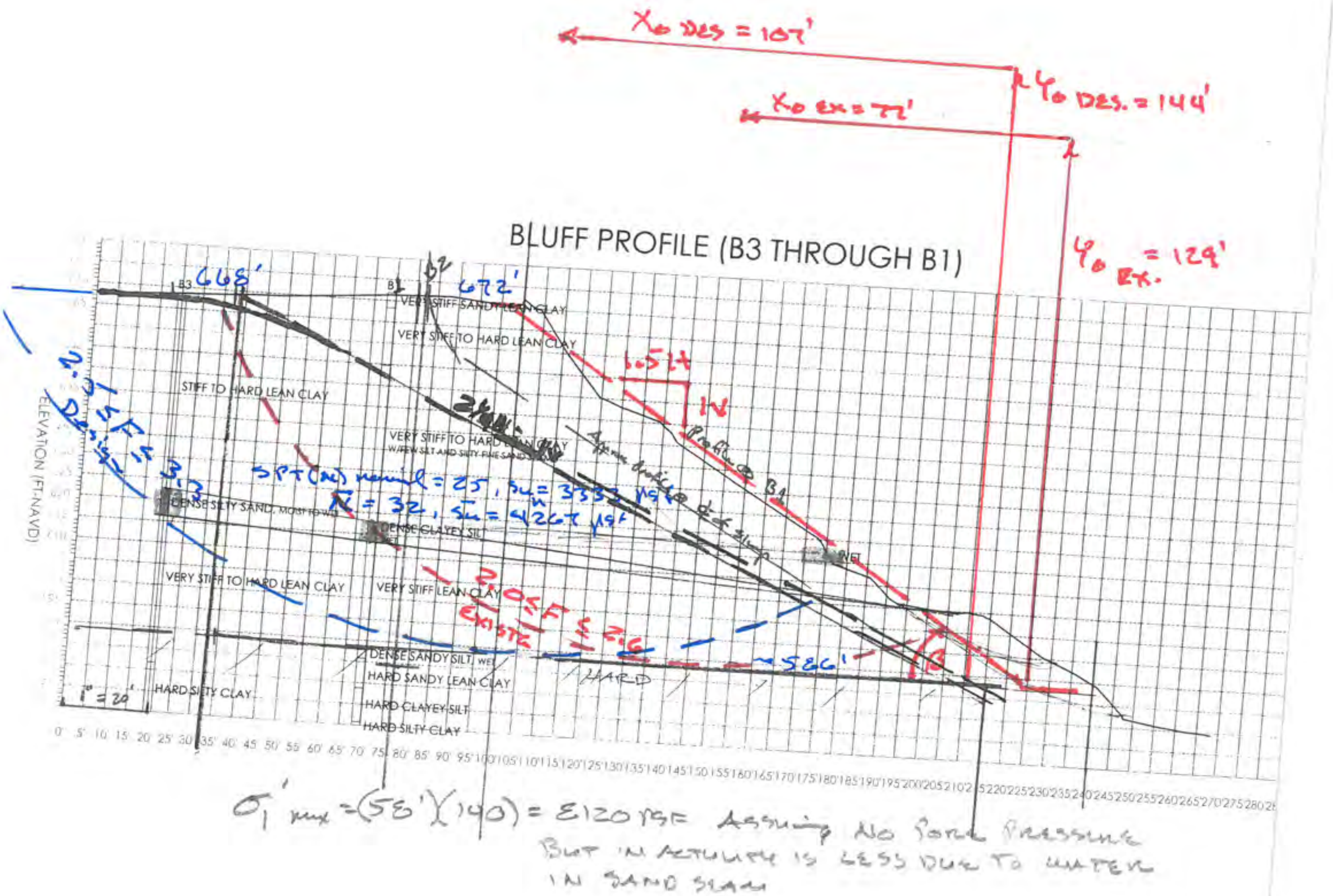
Client: City of Port Washington Project: North Beach & Bluff
Subject: Deep Seated Slope Stability Analysis Project No.: 20615-001
By: DBM Date: 3/21/23 Checked By: EBB Date: 3/24/2023 Page No.: 3/4



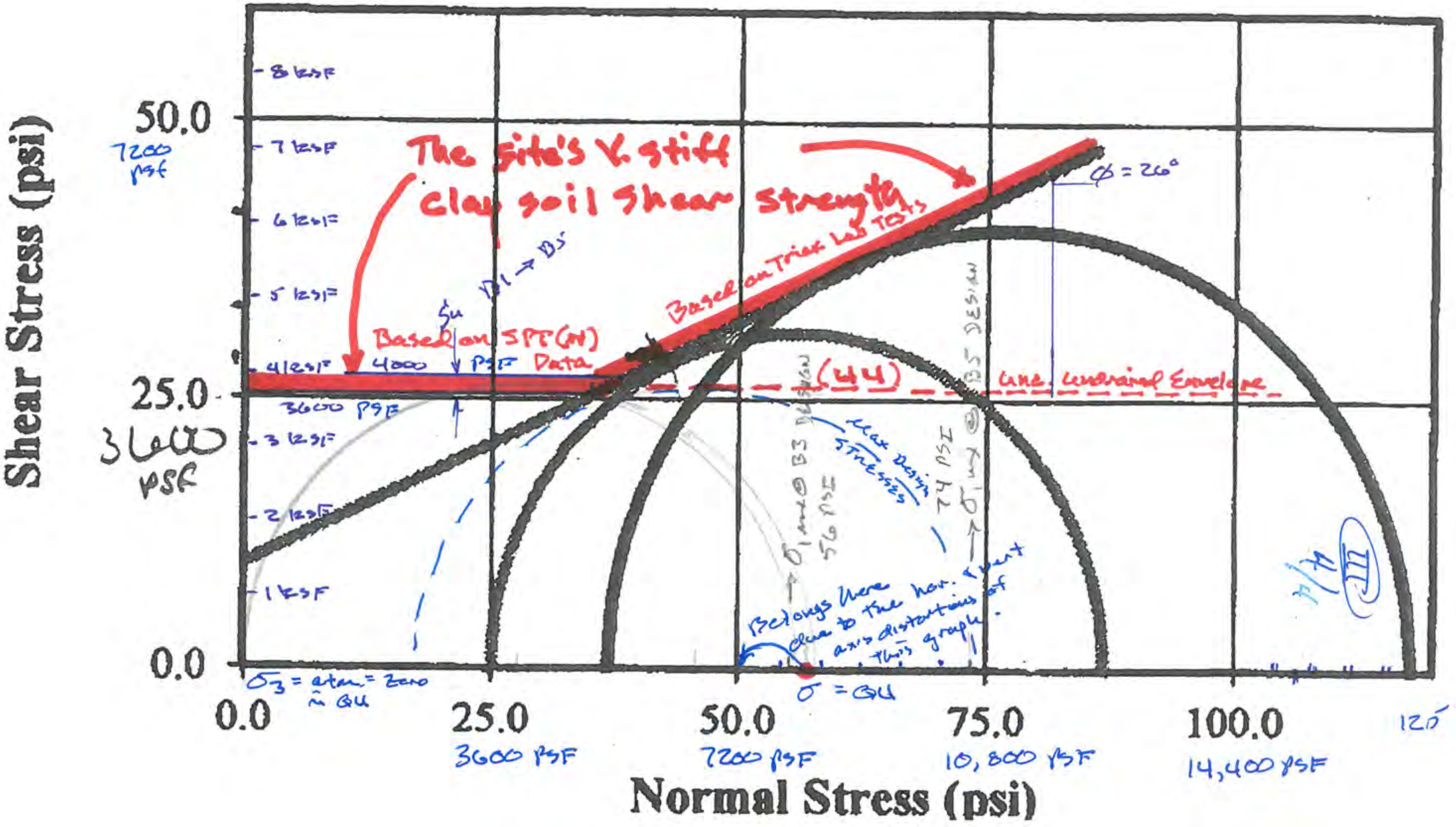
σ_1' max = (58')(140) = 8120 PSF Assuming No Pore Pressure But in Actuality is less due to water in sand seam

Wisconsin Testing Laboratories 2014 Geotechnical Report Laboratory Data

GeoTest, Inc.

nsolidated Undrained Triaxial Test (ASTM D4767)

Mohr Stress Circles at 15% Axial Strain Criterion



Client: City of Port Washington Project: North Beach & Bluff
Subject: Deep Seated Slope Stability Analysis Project No.: 20615-001
By: RGM Date: 3/22/23 Checked By: EBB Date: 3/24/2023 Page No.: 1/3

(III) Cont. - DEEP SEATED SLOPE STABILITY (B4 & B5)

SPT(N) PROFILES

ELEV	B4	B5
		$692^{\underline{6}}$
690		24
680	$679^{\underline{1}}$	27
	4	
	6	33
	13	
670	11	40
	10	32
660	10	31
	20	27
650	30	24
	21	47
640	21	37
	24	29
630		
	21	31
	24	36
620	30	40
	27	34
610	39	30
	39	30
600	26	43
	27	37
590	Clays ↓ 46	36
	51	30
580	23	37
	34	E.O.B
	E.O.B.	

$\overline{N} = 24$ $\overline{N} = 33.5$
$\sigma = 12$ $\sigma = 5.6$

$\overline{S_u} = 3200$ PSF B4

$\overline{S_u} = 4467$ PSF B5

$H = 679 - 592$ $H = 692 - 580$

$P_d = (140)(87) = 12,180$ $(112')(140) = 15,680 = P_d$

$$F = N_o \frac{c}{P_d}$$

cot β EXISTING = 1.58
∴ $N_o = 7.5$ from Fig 6

@ B4, H = 87'
$\overline{P_d} = 12180$
$\overline{c} = 3200$ PSF

cot β DESIGN = 2.0
$N_o = 8.3$ from Fig 6

$$F_{EXISTING} = \frac{7.4(3200)}{12180} = 1.94 \quad F_{Des} = \frac{8.3(3200)}{12180} = 2.1$$

@ B5 H = 112'
$P_d = 15680$
$\overline{c} = 4467$ psf

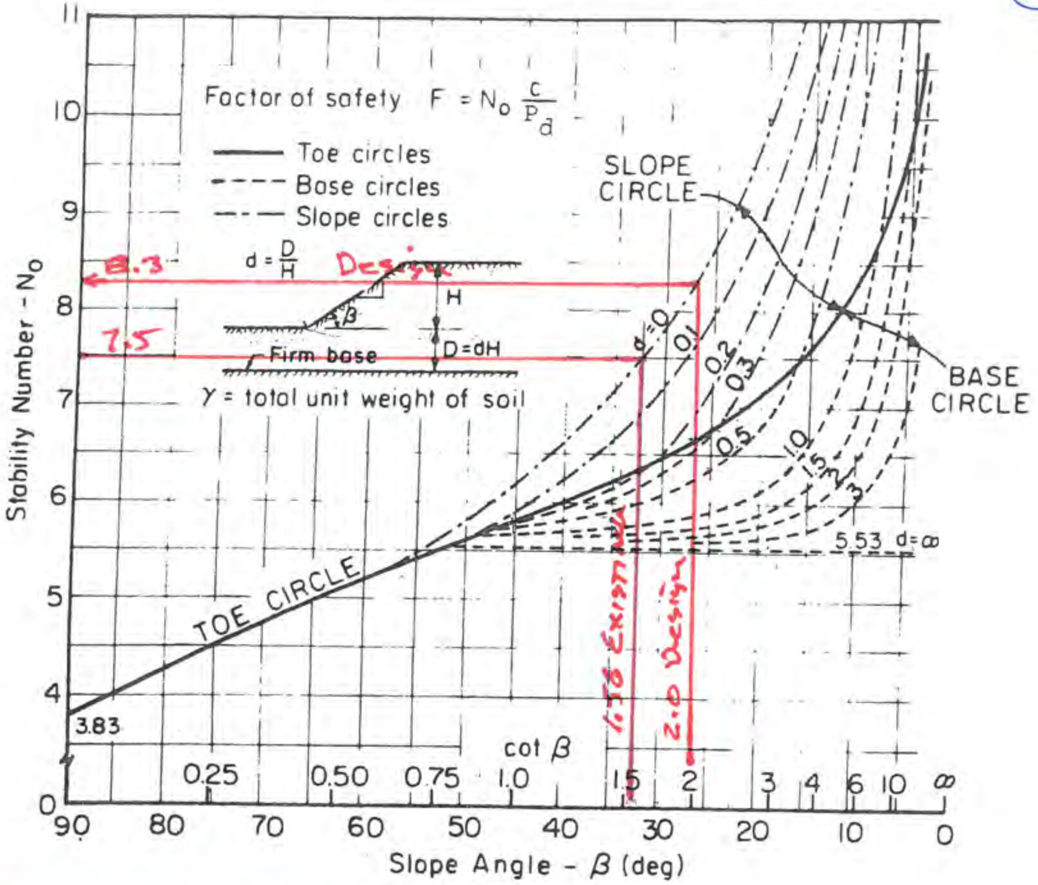
$$F_{EXISTING} = \frac{7.4(4467)}{12180} = 2.71 \quad F_{DES} = \frac{8.3(4467)}{12180}$$

$F_{DES.} = 3.0$

AND BLUFF SLOPES TO NORTH ARE SIMILAR, SO TO NORTH SLOPES CAN BE LEFT ALONE AS THEY'LL ONLY EXPERIENCE SHALLOW SLIDING WHERE BEACH IS NOT FREQUENTED

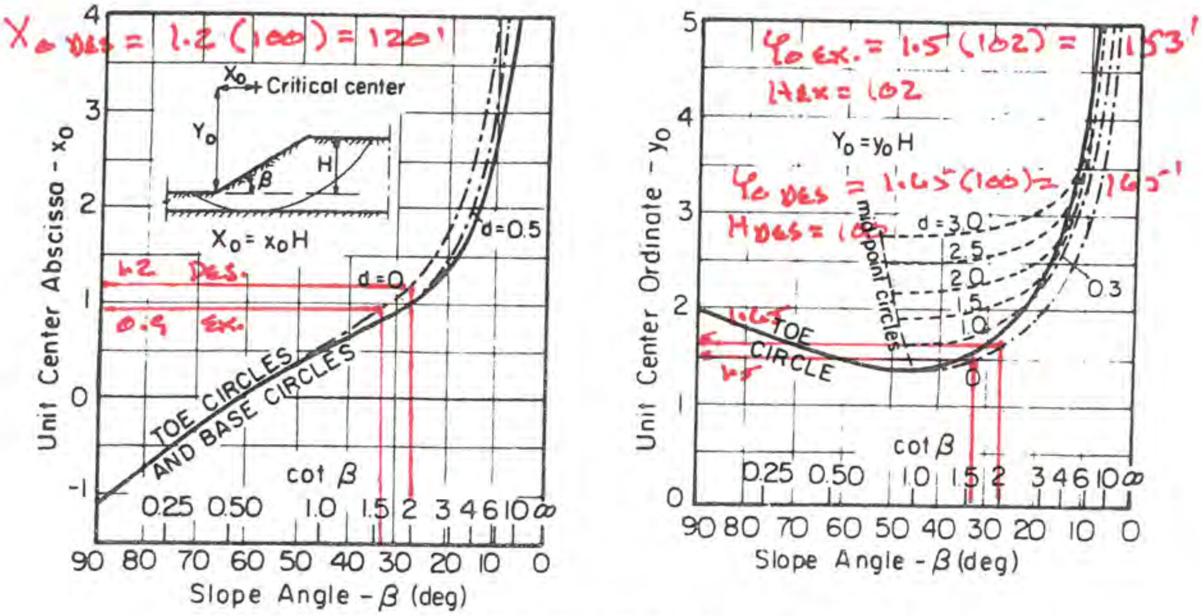
DEEP SEATED POTENTIAL SLOPE FAILURE

III cont.
2/3



STABILITY NUMBER

$X_{o\,ex} = 0.9(102) = 92'$



$X_{o\,DES} = 1.2(100) = 120'$

$Y_{o\,EX.} = 1.5(102) = 153'$

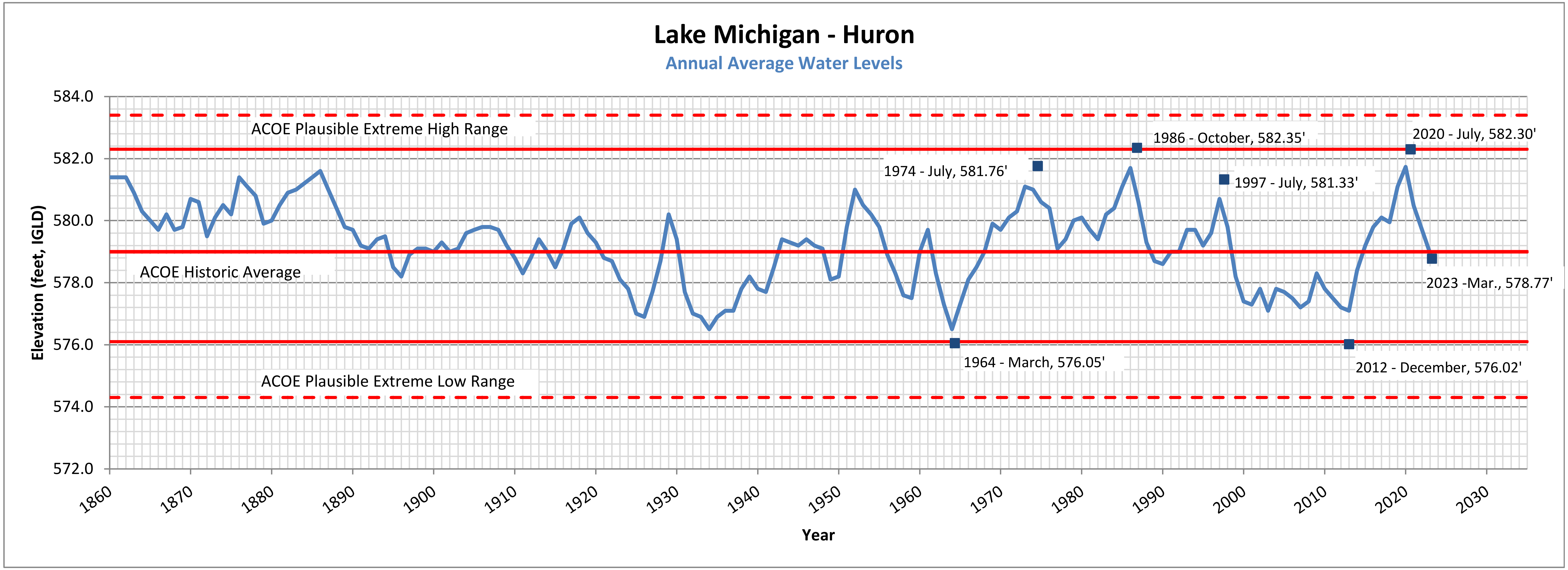
$H_{EX} = 102$

$Y_{o\,DES} = 1.65(100) = 165'$

$H_{DES} = 100$

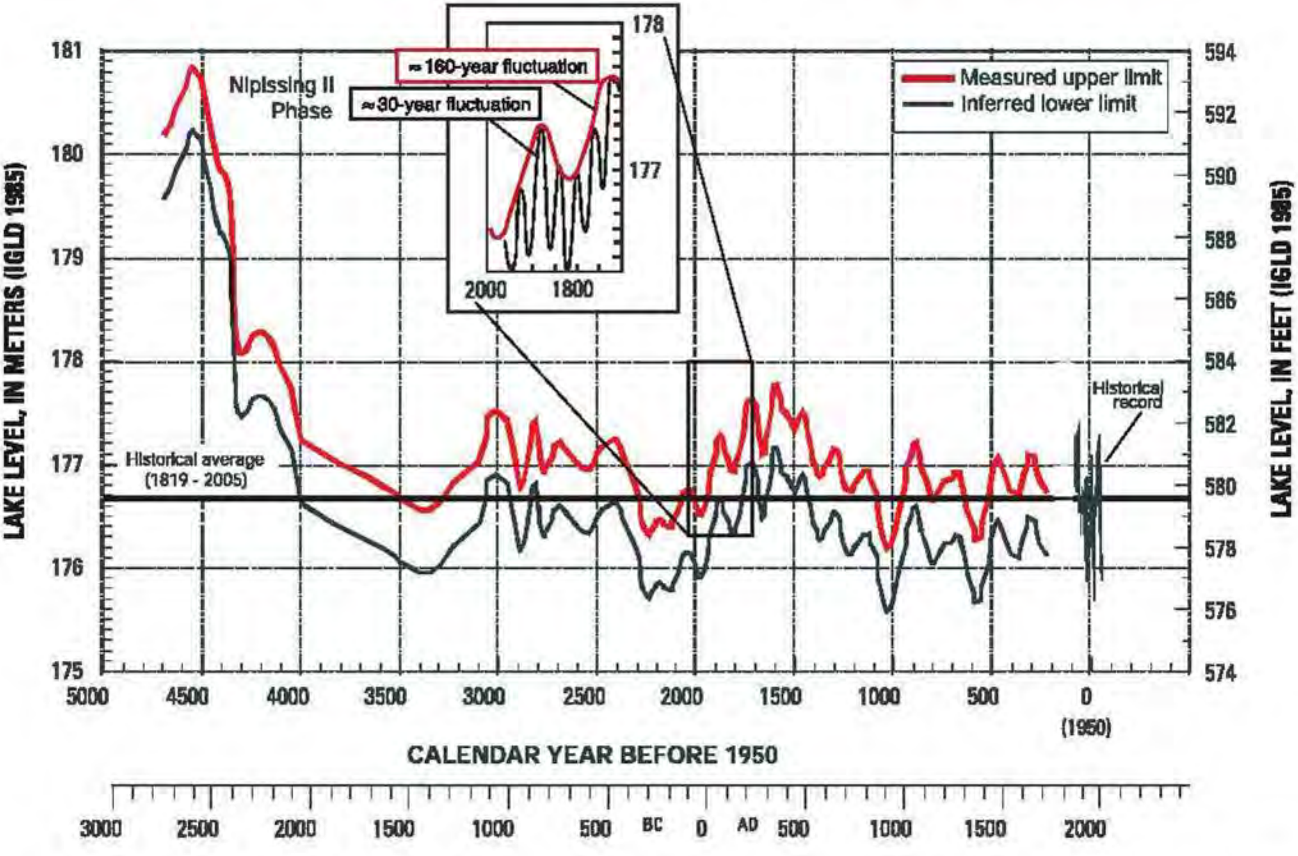
CENTER COORDINATES FOR CRITICAL CIRCLE

Fig. 6 SLOPE STABILITY CHARTS FOR $\phi = 0$ SOILS (after Janbu, 1968)


Lake Michigan - Huron
Annual Average Water Levels
584.0
582.0
580.0
578.0
576.0
574.0
572.0
Elevation (feet, IGLD)
ACOE Plausible Extreme High Range
1986 - October, 582.35'
2020 - July, 582.30'
1974 - July, 581.76'
1997 - July, 581.33'
ACOE Historic Average
2023 -Mar., 578.77'
1964 - March, 576.05'
2012 - December, 576.02'
ACOE Plausible Extreme Low Range
1860
1870
1880
1890
1900
1910
1920
1930
1940
1950
1960
1970
1980
1990
2000
2010
2020
2030
Year

Lake Michigan Water Levels in Recent Geologic Time

Lake-Level Variability and Water Availability in the Great Lakes:
U.S. Geological Survey Circular 1311, Wilcox et al., 2007



Hydrograph of late Holocene lake level and historical lake level for Lake Michigan-Huron. The red line is interpreted from beach-ridge studies, whereas the lower black line is an inferred lower limit using the range of the historical record as a guide.

Grain Size Analyses - Lakebed Samples at Port Washington North Beach

Water Elevation (ft NAVD): 580.35

Sample ID	43 deg. N. Latitude	87 deg. E. Longitude	Bottom Elev. (ft NAVD88)	Water Depth (ft)	Bottom Material	Percent Passing by Weight: Sieve #: / Opening (mm):	6 / 3.35	10 / 2.00	16 / 1.18	20 / 0.85	30 / 0.60	40 / 0.425	50 / 0.30	60 / 0.25	80 / 0.178	100 / 0.15	200 / 0.075	D50% (mm)	Moisture Cont.(%)	Dry Weight Bulk Density* (pcf)	m	b	c
Cross Section located several hundred yards north of park																							
	23.822'	51.421'		19.5			Well graded gravel																
A	23.822	51.465'	562.85	17.5	Just W of gravel-sand transition				100	99.8	99.3	97.6	96	95.3	75.7	56.6	2.4	0.141	25.5	99.8	257.0	268.3	-0.850
B	23.847	51.478'	565.85	14.5	Fine sand		100	99.1	98.8	98.5	98.1	97.6	96.9	96	85.3	70.9	7.3	0.117	23.6	102.9	193.7	230.5	-0.932
	23.877'	51.483'	565.85	14.5	Gravel & cobbles																		
C	23.910'	51.505'	570.65	9.7	Fine sand			100	100	100	99.9	98.7	96.2	95.5	73.1	55.5	2.4	0.142	23.5	103.1	236.8	250.6	-0.847
D	23.923'	51.507'	571.85	8.5	Fine sand			100	99.4	99.2	98.6	97.4	92.9	91.9	48.3	26.3	0.6	0.180	23.5	103.1	296.0	270.2	-0.744
E	23.941'	51.508'	573.65	6.7	Fine sand			100	99.9	99.8	99.2	98.4	94.5	94.1	54.4	32.7	0.3	0.172	24	102.2	291.9	273.2	-0.765
Cross Section off Mile Rock																							
F	23.823'	51.634'	574.05	6.3	Fine sand			100	99.7	99.5	99.2	98.3	94.2	93.8	59.9	36.2	1.1	0.166	27.4	96.8	318.9	298.9	-0.781
Cross Section off south portion of park																							
G	23.674'	51.768'	574.55	5.8	Fine sand		100	99.7	99.4	98.9	97.7	94.8	85.8	85.2	76.1	59.1	0.8	0.137	21.5	106.6	228.7	247.5	-0.864
H	23.657'	51.750'	571.85	8.5	Fine sand			100	99.9	99.8	99.6	99.4	98.6	98.4	73.8	49.5	1.4	0.151	25.5	99.8	326.9	318.9	-0.822
I	23.648'	51.721'	569.95	10.4	Fine sand			100	99.8	99.7	99.5	99.4	99.2	99.1	88.3	74.3	8.2	0.111	28	95.9	188.4	229.5	-0.953
J	23.633	51.684'	567.85	12.5	Fine sand			100	100	99.9	99.8	99.7	99.5	99.4	91.6	78.4	11.9	0.104	25.4	99.9	177.6	224.7	-0.984
						Average:		99.9	99.7	99.5	99.1	98.2	95.3	94.8	72.3	53.7	3.8	0.142	24.8	101.0	251.59	261.2	-0.854
	23.617'	51.644'	563.05	17.3	Clay till & cobbles																		
K?	23.633'	51.651'							100	100	99.9	99.8	99.6	99.5	95.1	86.9	32.0	0.094	21.7	106.2	182.4	237.2	-1.026
K muck	23.633'	51.651'	565.05	15.3	Soft organic silt				100	99.9	99.8	99.4	98.6	98.5	94.5	86.8	47.2	0.066	47.1	74.2	103.6	172.2	-1.179
								Average:	100.0	100.0	99.9	99.6	99.1	99.0	94.8	86.9	39.6	0.080	34.4	90.2	142.984	204.656	-1.103

*Gs: 2.7

Grain Size Analyses - Samples at mid-beach in cross section and at 3 feet water depth

Stake ID	Sieve #:	2"	1-1/2"	1"	3/4"	1/2"	3/8"	1/4"	4	10	20	30	40	50	60	80	100	200	D50% (mm)		m	b	c
	Opening (mm):	51	38	25	19	12.7	9.5	6.3	3.35	2.00	0.85	0.60	0.425	0.30	0.25	0.178	0.15	0.075					
1	Mid-Beach:									100	99.9	99.3	93.1	**59.7**	**44.7**	3.7	0.8	0.0	**0.267**		189.4	158.8	-0.574
	3 feet Depth:									100	99.5	98.9	96.8	79.3	**74.3**	**13.4**	3.9	0.0	**0.218**		412.8	322.8	-0.661
2	Mid-Beach:									100	99.6	97.6	**85.4**	**48.4**	39.3	5.8	1.2	0.0	**0.215**		244.6	213.3	-0.668
	3 feet Depth:									100	99.5	98.6	94.3	**67.7**	**49.4**	6.8	1.9	0.0	**0.251**		231.1	188.5	-0.599
3	Mid-Beach:								100	99.6	97.9	95.5	**85.7**	**45**	31.3	3.00	0.9	0.0	**0.221**		269.1	226.4	-0.656
	3 feet Depth:								100	99.4	98.5	97.9	96.3	83.0	**75.7**	**13.9**	4.1	0.0	**0.217**		418.9	327.9	-0.663
4	Mid-Beach:								100	99.1	97.8	96.9	93.5	**66.1**	**47.1**	3.4	0.7	0.0	**0.257**		240.0	191.6	-0.590
	3 feet Depth:								100	99.3	98.5	98.1	96.6	85.8	**80.7**	**19.6**	5.8	0.0	**0.211**		414.2	330.1	-0.676
5	Mid-Beach:								100	99.9	99.8	99.5	96.8	66.0	**50.9**	**4.3**	0.8	0.0	**0.248**		315.9	241.1	-0.605
	3 feet Depth:								100	98.3	97.7	97.4	96.0	82.3	**78.3**	**15.4**	4.2	0.0	**0.215**		426.4	335.0	-0.668
6	Mid-Beach:									100	99.7	98.8	95.7	74.5	**62**	**8.6**	2.4	0.0	**0.232**		362.0	279.9	-0.635
	3 feet Depth:								100	99.4	97.0	95.3	91.2	71.3	**66.4**	**12.3**	3.8	0.0	**0.226**		366.7	287.2	-0.647
								Average:	**100**	**99.6**	**98.8**	**97.8**	**93.5**	**69.1**	**58.3**	**9.2**	**2.5**	**0.0**	**0.240**	**Mid-Beach Average**			
																			0.217	**3 feet Depth Average**			
7	Mid-Beach:	100	78.1	51.9	46.7	38.7	36.8	33.4	32.1	29.1	26.0	22.9	17.8		6.6	1.1	0.6	0.3					
						Sand Portion Only:		100	96.1	87.1	77.8	68.6	**53.3**		**19.8**	3.3	1.8	0.9	**0.403**		145.5	107.4	-0.394
	3 feet Depth:								100	99.9	99.5	99.3	98.1	88	**72.7**	**8.8**	2.5	0.0	**0.222**		433.2	333.5	-0.654
8	Mid-Beach:	100	82.0	61.4	52.1	41.3	38.4	36.6	36.0	35.3	34.2	31.3	22.4	9.0	4.7	0.6	0.3	0.1					
						Sand Portion Only:		100	98.4	96.4	93.4	85.5	**61.2**	**24.6**	12.8	1.6	0.8	0.3	**0.270**		242.0	187.8	-0.569
	3 feet Depth:								100	99.6	98.6	97.6	95.4	85.0	**81.9**	**36.2**	17.0	0.0	**0.197**		309.8	268.4	-0.705
Composite Sample from Stockpile of Sand that had been Dredged from the south side of the south breakwater adjacent to the Powerplant							100	97.8	96.3	93.9	91.6	90.4	88	78.9	77.3	33.6	16.1	1.1	**0.202**		296.2	255.6	-0.694

MILLER
ENGINEERS
SCIENTISTS

Client: City of Port Washington Project: North Beach + Bluff
Subject: Breaking Wave Height Analysis + Revetment Proportioning Project No.: 20615-001
By: ERB Date: 3/27/2023 Checked By: RGM Date: 3/27/23 Page No.: 1/1

* Breaking wave height calculation based on USACOE 1984 Shore Protection Manual

Water Level Range

1986 high water = 582.35' IGLD + 0.5' to NAVD = 582.85 NAVD
1.5' storm surge typical + high water level = 584.35' NAVD ∴ say 584.50' NAVD
RFE100

Breaking Wave Height

max. water height based on bathymetric survey at one wavelength out (40' to 60') compared to design high water level of 584.50' $H = 5'$ (see near shore profile)

Lakebed slope ~0.033%

See Figure 7-3 for the relationship to breaking wave height (H_b) prior to reaching the revetment
$H_b \approx 5.9'$
See Figure 7-4 for the relationship to breaking wave height (H_b) at the revetment
$H_b \cong 4.1'$

∴ use $H_b = 5'$ ⇒ Top of revetment = 584.50 + 5 = 589.50 ∴ $\boxed{590' \text{ NAVD}}$

* Armor stone size calculation based on USACOE engineering manual 1110-2-1614

Armor Stone Size

$$W_{50} = \frac{\gamma H_b^3}{K_D\left(\frac{\gamma_r}{\gamma_w}-1\right)\cot\theta}$$

use 2H:1V slope for revetment ∴ $\cot\theta = 2.0$
γ_r for limestone = 175 lb/ft³
$\gamma_{water} = 62.3$ lb/ft³
$H_b = 5'$
$K_D = 2.0$ (see table 2-3)

$W_{50} = 3{,}023$ lb

$D_{sphere} = \left(\frac{6W}{539}\right)^{0.33}$ Diameter of perfectly spherical stone $D_{sphere} = 3.2'$

$D_{cube} = \left(\frac{W}{171.6}\right)^{0.33}$ Diameter of perfectly cubic stone $D_{cube} = 2.6'$

∴ W_{50} stone diameter of 2.9' ⇒ $\boxed{3'}$

Armor Layer Thickness

$r = nk_\Delta\left(\frac{W}{\gamma_r}\right)^{0.33}$

r = layer thickness in feet
$n = 2$
$K_\Delta = 1.0$ (see table 2-4)

$r = \boxed{5.2'}$

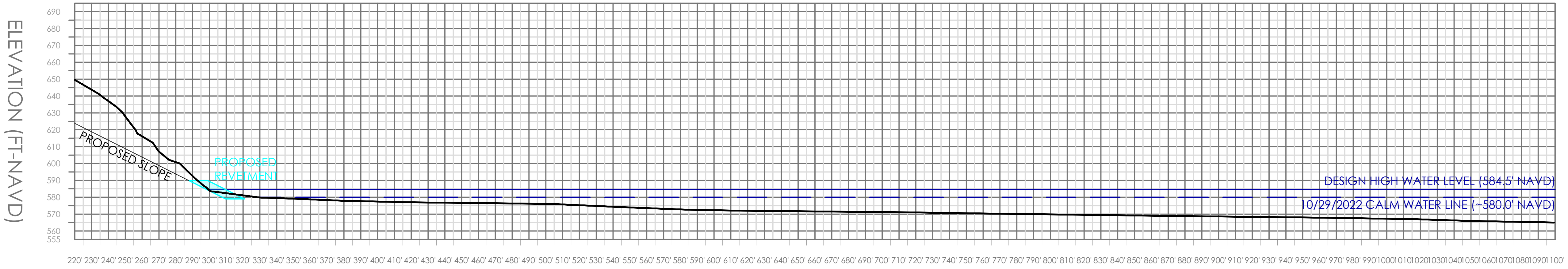
Revetment Proportions

$\underset{\text{(top of revetment)}}{590'} - \underset{\text{(bottom of slope)}}{584'} = 6'$ ∴ slope length = 14' + revetment toe length $(2 * \overset{\to 3'}{d_{50}}) = 20'$

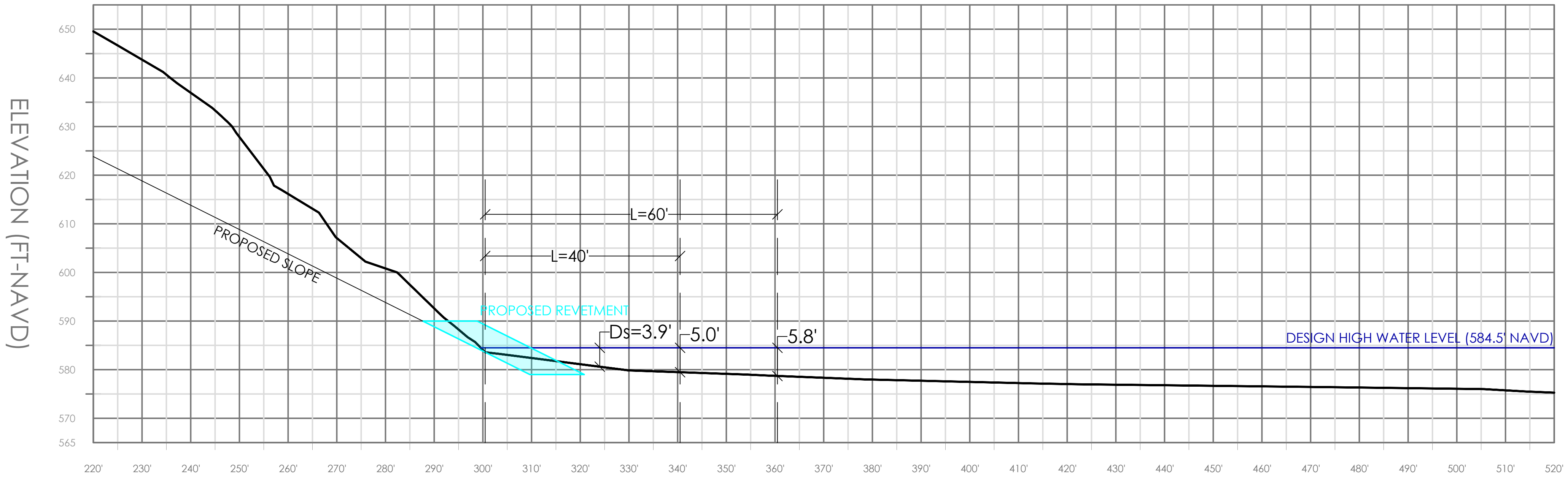
~4' to 5' thick revetment * 20ft * 175 $\frac{lb}{ft^3}$ * $\overset{\text{account for void space}}{0.7}$ * $\frac{1 \text{ ton}}{2000 \text{ lbs}}$ = 4.9 to 6.1 tons/LF of revetment ∴ say $\boxed{\text{5-6 tons/LF of revetment}}$

NEAR SHORE PROFILE (BATHYMETRIC SURVEY PERFORMED ON 10/29/2022)

SCALE:1"=70'



SCALE: 1"=30'



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NO.	DATE	DESCRIPTION	BY

UPPER LAKE PARK BEACH & BLUFF REHABILITATION
CITY OF PORT WASHINGTON
498 N LAKE STREET
PORT WASHINGTON, WI

SCALE
HOR. VARIES
VER. VARIES

DATE
03/24/2023

JOB
20615-B

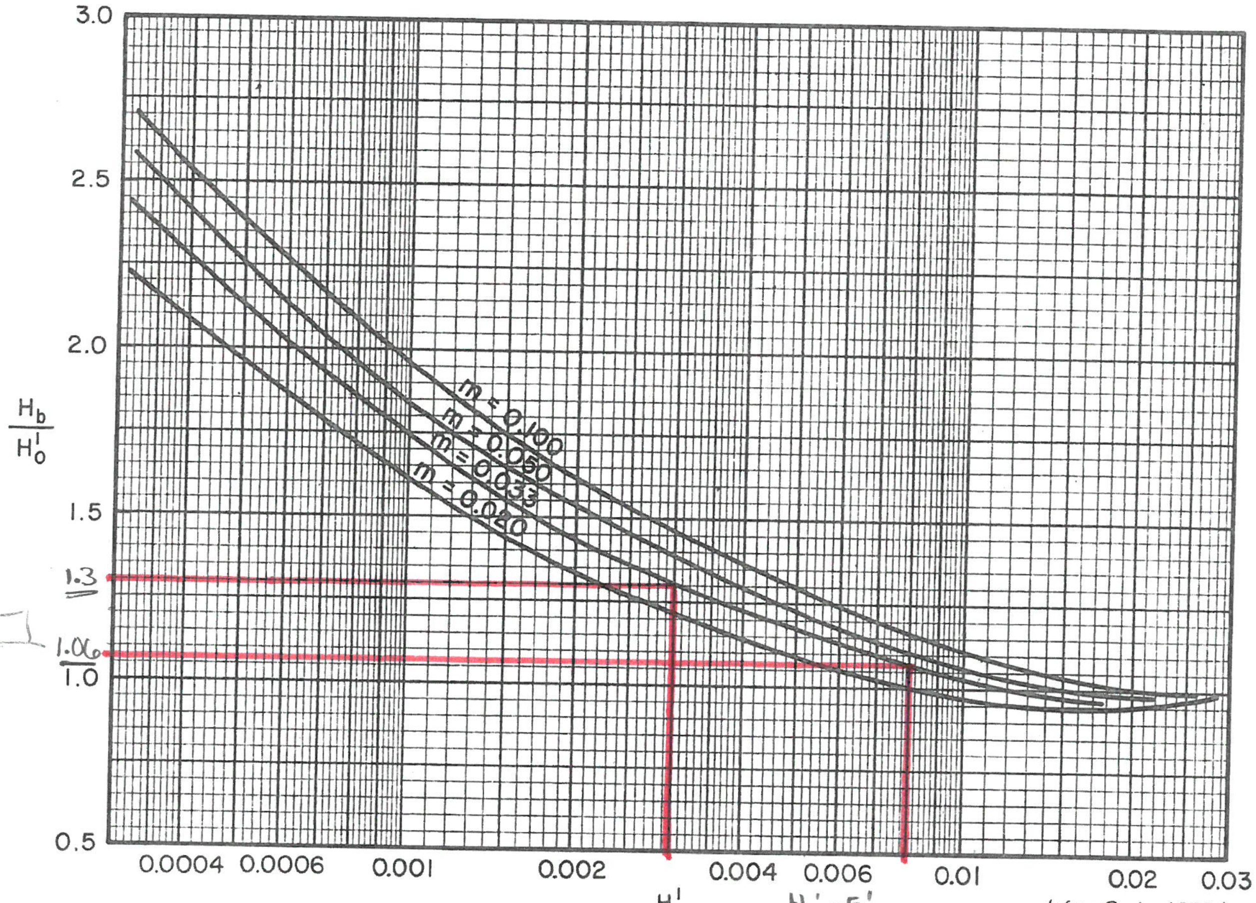
BY
EBB

CK
RGM

SHEET
1 OF 1

NEAR SHORE PROFILE

I:\DATA\20600\20615 - Port Washington Upper Lake Park\CAD\DESIGN\20615 - B Bluff Stability.dwg 3/27/2023 11:58 AM



∴ use 1.18

$0.003 \leq \frac{H'_o}{gT^2} \leq 0.008$

$H_o' = 5'$

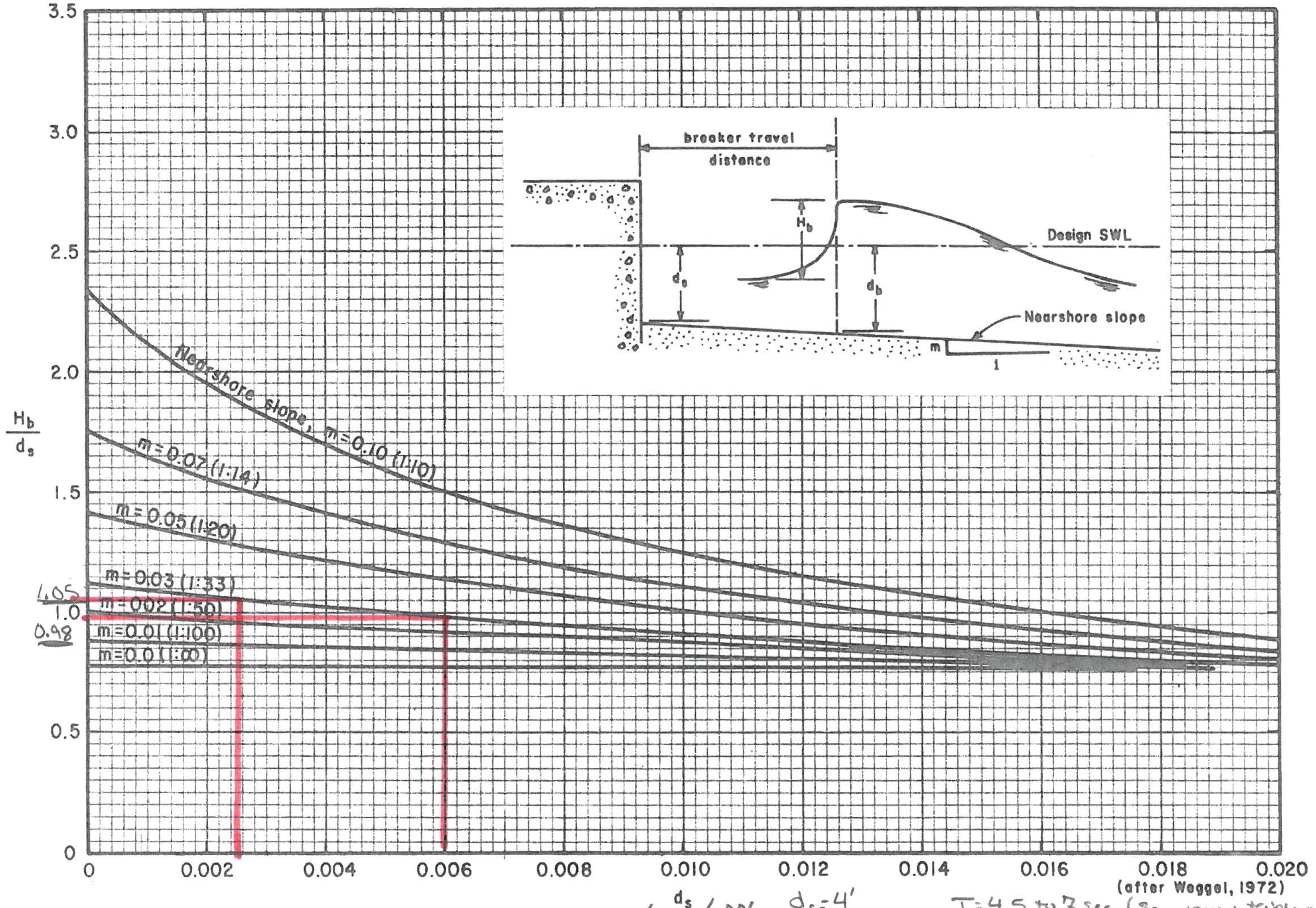
g = 32.2 ft/sec²

(after Goda, 1970a)

T = 4.5 to 7 sec (see report table on pg 6)

Figure 7-3. Breaker height index H_b/H_o' versus deepwater wave steepness H_o'/gT^2 .

∴ use 1.02



$.0025 \leq \frac{d_s}{gT^2} \leq .006$ $d_s = 4'$

$g = 32.2$ ft/sec^2

T = 4.5 to 7 sec (see report table on pg 6.)

Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

Table 2-3
Suggested Values for Use In Determining Armor Weight (Breaking Wave Conditions)

Armor Unit	n[1]	Placement	Slope (cot θ)	K_D
Quarrystone				
Smooth rounded	2	Random	1.5 to 3.0	1.2
Smooth rounded	>3	Random	1.5 to 3.0	1.6
Rough angular	1	Random	1.5 to 3.0	Do Not Use
Rough angular	2	Random	1.5 to 3.0	2.0
Rough angular	>3	Random	1.5 to 3.0	2.2
Rough angular	2	Special[2]	1.5 to 3.0	7.0 to 20.0
Graded riprap[3]	2[4]	Random	2.0 to 6.0	2.2
Concrete Armor Units				
Tetrapod	2	Random	1.5 to 3.0	7.0
Tripod	2	Random	1.5 to 3.0	9.0
Tripod	1	Uniform	1.5 to 3.0	12.0
Dolos	2	Random	2.0 to 3.0[5]	15.0[6]

[1] n equals the number of equivalent spherical diameters corresponding to the median stone weight that would fit within the layer thickness.

[2] Special placement with long axes of stone placed perpendicular to the slope face. Model tests are described in Markle and Davidson (1979).

[3] Graded riprap is not recommended where wave heights exceed 5 ft.

[4] By definition, graded riprap thickness is two times the diameter of the minimum W_{50} size.

[5] Stability of dolosse on slope steeper than 1 on 2 should be verified by model tests.

[6] No damage design (3 to 5 percent of units move). If no rocking of armor (less than 2 percent) is desired, reduce K_D by approximately 50 percent.

Meer (1988a, 1988b). Two stability equations were presented. For plunging waves,

$$N_s = 6.2\, P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_z^{0.5} \tag{2-19}$$

and for surging or nonbreaking waves,

$$N_s = 1.0\, P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot\theta}\; \xi_z^{P} \tag{2-20}$$

where

P = permeability coefficient

S = damage level

N = number of waves

P varies from $P = 0.1$ for a riprap revetment over an impermeable slope to $P = 0.6$ for a mound of armor stone with no core. For the start of damage $S = 2$ for revetment slopes of 1:2 or 1:3, or $S = 3$ for revetment slopes of 1:4 to 1:6. The number of waves is difficult to estimate, but Equations 2-19 and 2-20 are valid for $N = 1{,}000$ to $N = 7{,}000$, so selecting 7,000 waves should provide a conservative estimate for stability. For structures other than riprap revetments, additional values of P and S are presented in van der Meer (1988a, 1988b).

e. Equations 2-19 and 2-20 were developed for deepwater wave conditions and do not include a wave-height truncation due to wave breaking. van der Meer therefore recommends a shallow water correction given as

$$\frac{N_{s\ (shallow\ water)}}{N_{s\ (deep\ water)}} = \frac{1.40\, H_s}{H_2} \tag{2-21}$$

where H_2 is the wave height exceeded by 2 percent of the waves. In deep water, $H_2 \approx 1.40\, H_s$, and there is no correction in Equation 2-21.

Table 2-4
Layer Coefficients and Porosity for Various Armor Units

Armor Unit	n	Placement	K_Δ	P (%)
Quarrystone (smooth)	2	Random	1.00	38
Quarrystone (rough)	2	Random	1.00	37
Quarrystone (rough)	≥3	Random	1.00	40
Graded riprap	2[a]	Random	N/A	37
Tetrapod	2	Random	1.04	50
Tribar	2	Random	1.02	54
Tribar	1	Uniform	1.13	47
Dolos	2	Random	0.94	56

[a] By definition, riprap thickness equals two cubic lengths of W_{50} or 1.25 W_{100}.

Table 2-5
$H/H_{D=0}$ for Cover Layer Damage Levels for Various Armor Types ($H/H_{D=0}$ for Damage Level in Percent)

Unit	$0 \le \%_D < 5$	$5 \le \%_D < 10$	$10 \le \%_D < 15$	$15 \le \%_D < 20$	$20 \le \%_D \le 30$
Quarrystone (smooth)	1.00	1.08	1.14	1.20	1.29
Quarrystone (angular)	1.00	1.08	1.19	1.27	1.37
Tetrapods	1.00	1.09	1.17	1.24	1.32
Tribars	1.00	1.11	1.25	1.36	1.50
Dolos	1.00	1.10	1.14	1.17	1.20

calculated, and a ratio with the site's wave height can be used to estimate the damage that can be expected with the available stone. All values in the table are for randomly placed units, n=2, and minor overtopping. The values in Table 2-5 are adapted from Table 7-8 of the SPM. The SPM values are for breakwater design and nonbreaking wave conditions and include damage levels above 30 percent. Due to differences in the form of damage to breakwaters and revetments, revetments may fail before damages reach 30 percent. The values should be used with caution for damage levels from breaking and nonbreaking waves.

c. *Graded riprap.* Information on riprap reserve stability can be found in Ahrens (1981a). Reserve stability appears to be primarily related to the layer thickness although the median stone weight and structure slope are also important.

2-19. Toe Protection

a. *General.* Toe protection is supplemental armoring of the beach or bottom surface in front of a structure which prevents waves from scouring and undercutting it. Factors that affect the severity of toe scour include wave breaking (when near the toe), wave runup and backwash, wave reflection, and grain-size distribution of the beach or bottom materials. The revetment toe often requires special consideration because it is subjected to both hydraulic forces and the changing profiles of the beach fronting the revetment. Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Specific guidance for toe design based on either prototype or model results has not been developed. Some empirical suggested guidance is contained in Eckert (1983).

b. *Revetments.*

(1) Design procedure. Toe protection for revetments is generally governed by hydraulic criteria. Scour can be caused by waves, wave-induced currents, or tidal currents. For most revetments, waves and wave-induced currents will be most important. For submerged toe stone, weights can be predicted based on Equation 2-25:

Upper Lake Park Buff Investigation
Port Washington, Wisconsin

For

City of Port Washington
100 West Grand Avenue
P.O. Box 307
Port Washington, WI 53074-0307

Job Number 0709-13-001

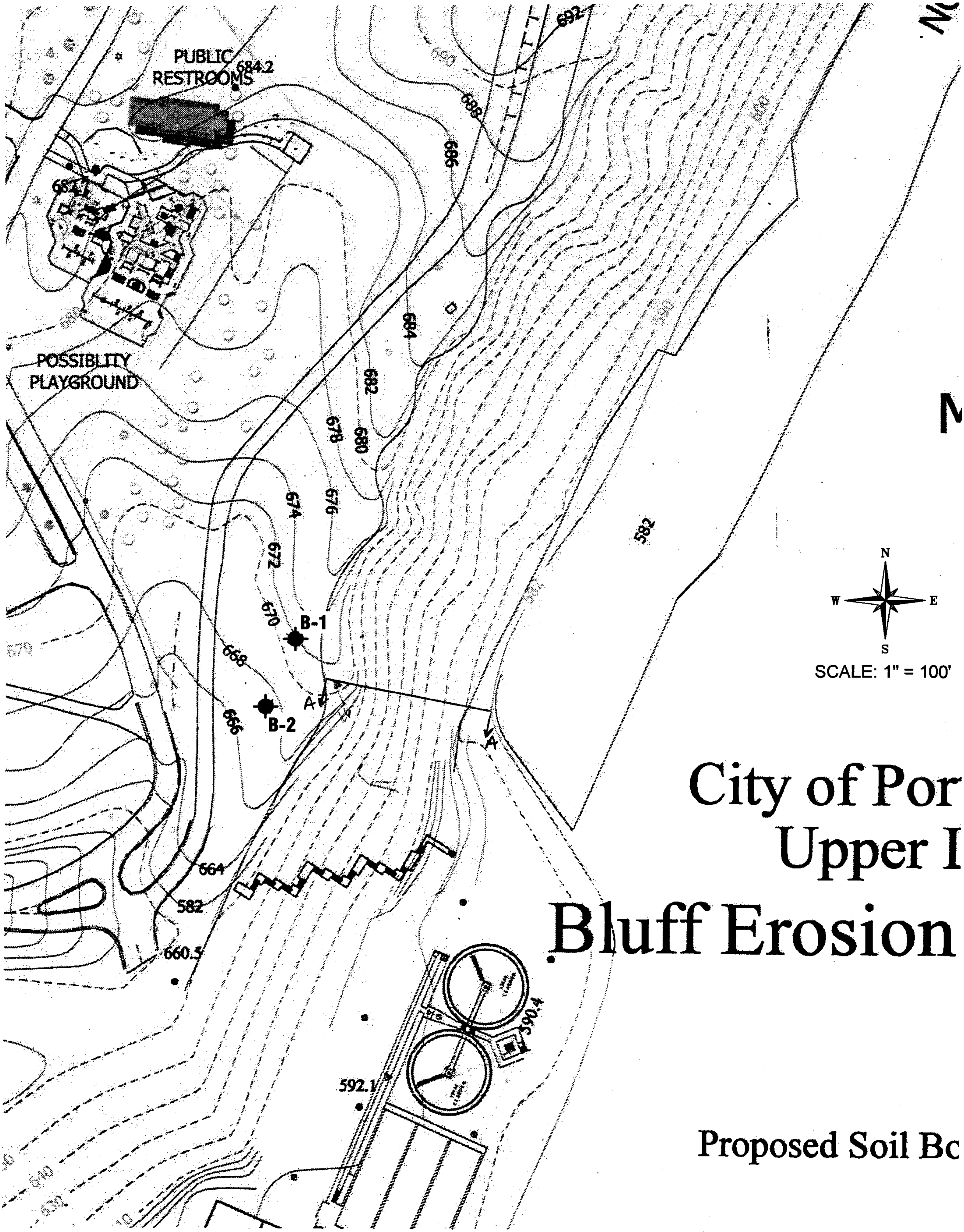
January 6, 2014

Prepared By:

Wisconsin Testing Laboratories LLC

GEOTECHNICAL ENGINEERING - MATERIALS TESTING

W140 N5886 Lilly Road
Menomonee Falls, WI 53051-6046
Phone (262) 252-3300   Fax (262) 252-5373
witestlab.com



City of Por
Upper I
Bluff Erosion

Proposed Soil B

LOG OF BORING

WISCONSIN TESTING LABORATORIES

PROJECT: Upper Lake Park Buff Investigation
CLIENT: City of Port Washington
LOCATION: Lake Street, North of Jackson Street
Port Washington, Wisconsin

JOB NO.: 0709-13-001
BORING NO.: 1
GROUND ELEVATION: 671.83
BORING LOCATION: See Diagram

BORING STARTED 9/11/2013
BORING COMPLETED 9/11/2013
TOTAL BORING DEPTH 109'

Groundwater: During Drilling
Completion of Drilling
24 Hours After Completion

Notes and Laboratory Test Results					Sample No	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
M$_c$ %	D$_d$ pcf	O$_c$ %	Q$_u$ tsf	Q$_p$ tsf							
										TOPSOIL moist. (2-1/2")	671.6
9.1				>4.5	1	12"	12 12 12		■	VERY STIFF REDDISH BROWN SILTY CLAY moist, some fine to coarse sand, little fine to coarse gravel. (Possible Fill)	
9.7				>4.5	2	12"	12 23 24	5	■	HARD TO VERY STIFF REDDISH BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel. (CL)	668.3
10.4	126.4		6.0	>4.5	3	14"	11 13 23		■		
8.5				4.5	4	16"	11 13 13	10	■		
14.1				3.5	5	16"	6 11 11		■		
13.5	120.8		3.5	4.0	6	16"	9 11 12	15	■		
13.5				3.2	7	10"	12 13 15	20	■	VERY STIFF AND HARD BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel, few seams of silt and silty fine sand. (CL)	654.8
16.2				>4.5	8	14"	9 13 20	25	■		

continues...

LOGGED BY AP & JF
CHECKED BY Soils Engineer
CLASSIFICATION SYSTEM WTL
METHOD OF DRILLING 4-1/4 in. I.D. hollow stem auger casing with carbide toothed finger bit used to 15' depth.
Mud-rotary with 6-1/4 in. roller bit used below 15' depth.
MACHINE MODEL Truck-mounted Diedrich D-120
SPT HAMMER TYPE Safety hammer, rope and cathead

■ SPLIT-SPOON
☒ TUBE
A AUGER
W WASH

LOG OF BORING

WISCONSIN TESTING LABORATORIES

PROJECT: Upper Lake Park Buff Investigation
CLIENT: City of Port Washington
LOCATION: Lake Street, North of Jackson Street
Port Washington, Wisconsin

JOB NO.: 0709-13-001
BORING NO.: 1
GROUND ELEVATION: 671.83
BORING LOCATION: See Diagram

Notes and Laboratory Test Results					Sample No.	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
M_c %	D_d pcf	O_c %	Q_u tsf	Q_p tsf							
										VERY STIFF AND HARD BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel, few seams of silt and silty fine sand. (CL)	
16.5	115.4		2.4	>4.5	9	16"	9 13 20	30	■		
18.5				3.5	10	14"	10 13 7	35	■		
19.7				3.7	11	16"	9 11 13	40	■		
20.2	108.8		1.9	3.2	12	16"	9 11 12	45	■		
20.8				1.5	13	16"	9 11 11	50	■		
21.5					14	14"	15 25 43	55	■	DENSE BROWN CLAYEY SILT wet, trace to some fine sand, few seams of silty clay. (ML)	617.8

continues...

LOG OF BORING

WISCONSIN TESTING LABORATORIES

PROJECT: Upper Lake Park Buff Investigation

CLIENT: City of Port Washington

LOCATION: Lake Street, North of Jackson Street
Port Washington, Wisconsin

JOB NO.: 0709-13-001

BORING NO.: 1

GROUND ELEVATION: 671.83

BORING LOCATION: See Diagram

Notes and Laboratory Test Results					Sample No.	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
M_c %	D_d pcf	O_c %	Q_u tsf	Q_p tsf							
17.6					15	12"	23 19 15			DENSE BROWN CLAYEY SILT wet, trace to some fine sand, few seams of silty clay. (ML)	
19.8				2.2	16	14"	8 10 11	60		VERY STIFF BROWN SILTY CLAY moist, trace fine to coarse sand, trace fine to coarse gravel. (CL)	613.3
24.0	103.6		1.8	3.0	17	16"	9 9 11	65			
26.6				2.2	18	16"	9 11 14	70			
27.3				3.5	19	16"	9 12 13	75			
19.7	109.7		1.5	2.7	20	16"	7 8 15	80			
14.1 12.7				1.2	21	14"	9 23 50/4"	85		DENSE BROWN FINE SANDY SILT wet, few seams of silty clay. (ML)	587.8

continues...

LOG OF BORING
WISCONSIN TESTING LABORATORIES

PROJECT: Upper Lake Park Buff Investigation
CLIENT: City of Port Washington
LOCATION: Lake Street, North of Jackson Street
Port Washington, Wisconsin

JOB NO.: 0709-13-001
BORING NO.: 1
GROUND ELEVATION: 671.83
BORING LOCATION: See Diagram

Notes and Laboratory Test Results					Sample No.	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
M_c %	D_d pcf	O_c %	Q_u tsf	Q_p tsf							
										DENSE BROWN FINE SANDY SILT wet, few seams of silty clay. (ML)	585.8
16.3				>4.5	22		25 50/5"			HARD GRAYISH BROWN SILTY CLAY moist, little to some fine to coarse sand, little fine to coarse gravel. (CL)	
14.3	116.7		1.6	>4.5	23	12"	43 50/5"				
								90			
											578.8
9.0				>4.5	24	4"	50/3"			HARD GRAYISH BROWN CLAYEY SILT moist, some fine to coarse sand, little fine to large gravel. (ML)	
								95			
8.7				>4.5	25	6"	50/5"				
								100			
											570.8
										HARD GRAYISH BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel. (CL)	
12.7				>4.5	26	4"	50/5"				
								105			
No Recovery				>4.5	27	0"	50/4"				562.8
										END OF BORING	
								110			
								115			

LOG OF BORING

WISCONSIN TESTING LABORATORIES

PROJECT: Upper Lake Park Buff Investigation
CLIENT: City of Port Washington
LOCATION: Lake Street, North of Jackson Street
Port Washington, Wisconsin

JOB NO.: 0709-13-001
BORING NO.: 2
GROUND ELEVATION: 667.65
BORING LOCATION: See Diagram

BORING STARTED 9/12/2013
BORING COMPLETED 9/12/2013
TOTAL BORING DEPTH 109'-5"

Groundwater: During Drilling
Completion of Drilling
24 Hours After Completion

Notes and Laboratory Test Results											
M_c %	D_d pcf	O_c %	Q_u tsf	Q_p tsf	Sample No	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
										TOPSOIL moist. (3")	667.4
					1	2"	7 11 12		■	MEDIUM DENSE DARK BROWN CLAYEY SILT moist, little fine to coarse sand. (Possible Fill)	
13.4	113.7		3.4	3.5	2	10"	6 7 8	5	■	VERY STIFF REDDISH BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel. (CL)	664.7
14.5				3.0	3	12"	8 11 13		■		
13.0	117.4		3.2	4.0	4	16"	6 15 11	10	■	VERY STIFF TO HARD BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel, few silt seams. (CL)	659.2
12.8				>4.5	5	14"	10 15 18		■		
12.9				4.5	6	14"	7 13 18	15	■		
35.1					7	10"	15 11 18	20	■	wet clayey silt layer at 19' depth	
17.0	111.3		1.7	3.5	8	16"	9 11 18	25	■	continues...	

LOGGED BY AP & JF
CHECKED BY Soils Engineer
CLASSIFICATION SYSTEM WTL
METHOD OF DRILLING 4-1/4 in. I.D. hollow stem auger casing with carbide toothed finger bit used to 15 ft. depth.
Mud-rotary with 6-1/4 in. roller bit used below 15' depth.
MACHINE MODEL Truck-mounted Diedrich D-120
SPT HAMMER TYPE Safety hammer, rope and cathead

■ SPLIT-SPOON
☒ TUBE
A AUGER
W WASH

LOG OF BORING
WISCONSIN TESTING LABORATORIES

PROJECT:	Upper Lake Park Buff Investigation	**JOB NO.:**	0709-13-001
CLIENT:	City of Port Washington	**BORING NO.:**	2
LOCATION:	Lake Street, North of Jackson Street Port Washington, Wisconsin	**GROUND ELEVATION:**	667.65
		BORING LOCATION:	See Diagram

Notes and Laboratory Test Results					Sample No.	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
M_c %	D_d pcf	O_c %	Q_u tsf	Q_p tsf							
										VERY STIFF TO HARD BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel, few silt seams. (CL)	
20.7				4.5	9	16"	9 13 20	30	■		
20.3	103.8		1.7	2.5	10	16"	10 13 7	35	■		
20.2				3.5	11	14"	9 11 13	40	■		
					12	22"		45	☒		
										DENSE PALE BROWN SILTY FINE SAND wet, trace clay. (SM)	621.2
17.1					13		34 50/6"	50	■		
										DENSE GRAYISH BROWN CLAYEY SILT wet, some fine sand, with seams of brown silty clay. (ML & CL)	616.7
14.7					14	16"	25 39 30		■		
18.4				4.5	15	18"	15 17 20	55	■		

continues...

LOG OF BORING

WISCONSIN TESTING LABORATORIES

PROJECT: Upper Lake Park Buff Investigation
CLIENT: City of Port Washington
LOCATION: Lake Street, North of Jackson Street
Port Washington, Wisconsin

JOB NO.: 0709-13-001
BORING NO.: 2
GROUND ELEVATION: 667.65
BORING LOCATION: See Diagram

Notes and Laboratory Test Results					Sample No.	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
M_c %	D_d pcf	O_c %	Q_u tsf	Q_p tsf							
										DENSE GRAYISH BROWN CLAYEY SILT wet, some fine sand, with seams of brown silty clay. (ML & CL)	611.7
										HARD AND VERY STIFF BROWN SILTY CLAY moist, trace fine to coarse sand, trace fine to coarse gravel, few silt seams. (CL)	
24.0				>4.5	16		20 22 27		■		
					17			60	☒		
27.4	96.4		1.8	3.2	18	18"	8 9 13	65	■		
28.7				3.7	19	18"	10 15 17	70	■		
18.4				3.0	20	16"	12 18 21	75	■		
				4.5	21	20"		80	☒		
										HARD BROWN SILTY CLAY moist, little to some fine to coarse sand, trace to little fine to coarse gravel, with seams of grayish brown silt. (CL)	586.7
14.0				>4.5	22	12"	37 49 50/1"	85	■		

continues...

LOG OF BORING

WISCONSIN TESTING LABORATORIES

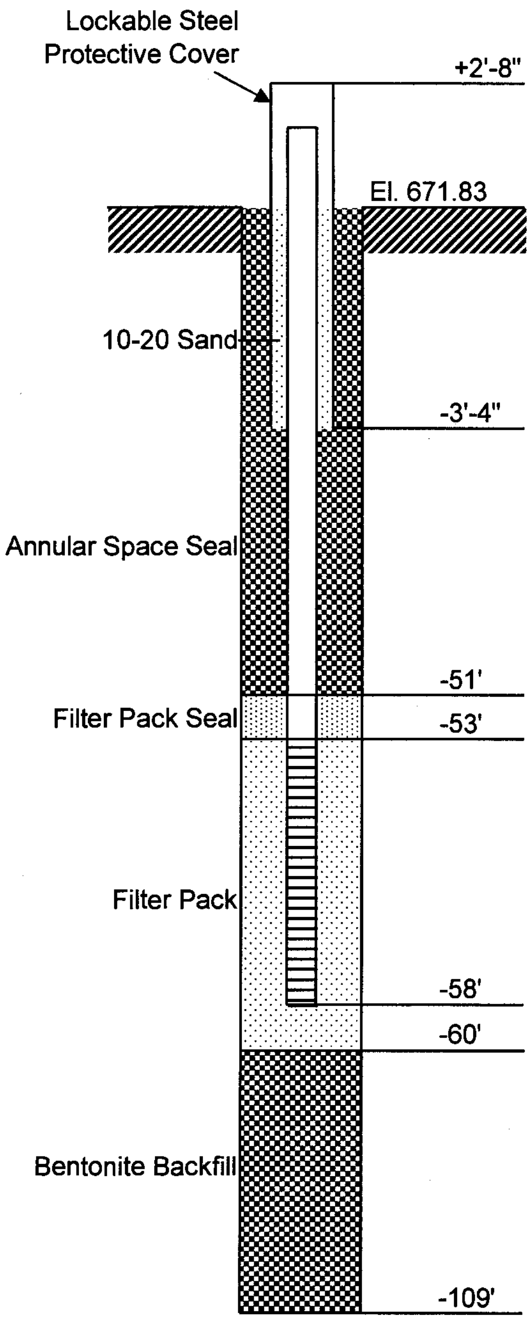
PROJECT: Upper Lake Park Buff Investigation
CLIENT: City of Port Washington
LOCATION: Lake Street, North of Jackson Street
Port Washington, Wisconsin

JOB NO.: 0709-13-001
BORING NO.: 2
GROUND ELEVATION: 667.65
BORING LOCATION: See Diagram

Notes and Laboratory Test Results					Sample No.	Recovery	Blow Count	Depth Feet	Sample	Material Classification	Elevation
M_c %	D_d pcf	O_c %	Q_u tsf	Q_p tsf							
										HARD BROWN SILTY CLAY moist, little to some fine to coarse sand, trace to little fine to coarse gravel, with seams of grayish brown silt. (CL)	580.7
										HARD GRAYISH BROWN CLAYEY SILT moist, some fine to coarse sand, trace to little fine to large gravel. (ML)	
				>4.5	23	1"	50/2"				
								90			
8.8				>4.5	24	10"	47 50/4"				
								95			
										gravel layers at 96'-97.5' depth	570.2
										HARD GRAYISH BROWN SILTY CLAY moist, little fine to coarse sand, trace fine to coarse gravel. (CL)	
9.9				>4.5	25	3"	50/3"				
								100			
10.5	129.5		9.0	>4.5	26	3"	50/3"				
								105			
13.1				>4.5	27	12"	24 50/4"				558.2
										END OF BORING	
								110			
								115			

Wisconsin Testing Laboratories

Piezometer Diagram



WTL Job. Number: 0709-13-001

Project: Upper Lake Park Buff Investigation
Port Washington, Wisconsin

Piezometer No.: 1

Date Installed: 9/12/13

Annular Space Seal Material: 3/8" Bentonite Chips

Filter Pack Seal Material: #15 Flint Sand

Filter Pack Material: 10-20 Ohio Sand

Borehole Diameter: 6"

Well Material: Flush-Threaded Sch. 40 PVC

Well I.D.: 2.00"

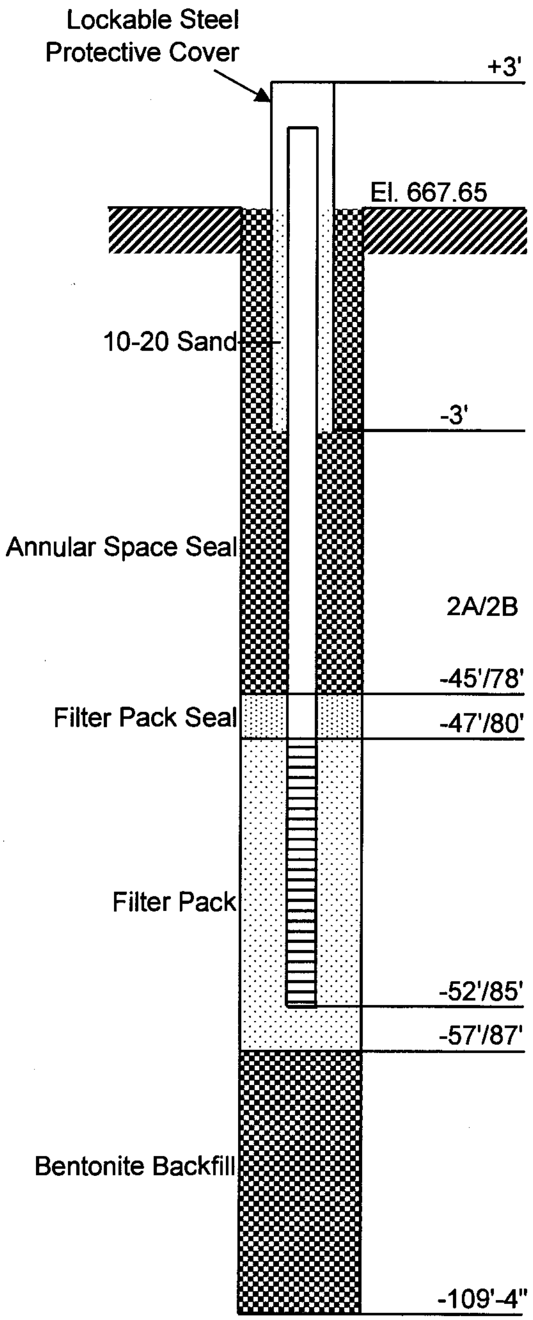
Well Screen Slot Size: 0.01"

Water Level Observations

Date	Time	Depth	Water Elev.	Comments
9/18/13	11:35am	53'-10"	618.0	
10/9/13	1:55pm	55'-10"	616.0	
11/4/13	2:00pm	55'-10"	616.0	

Remarks:

Wisconsin Testing Laboratories
Piezometer Diagram



WTL Job. Number: 0709-13-001

Project: Upper Lake Park Buff Investigation
Port Washington, Wisconsin

Piezometer No.: 2A & 2B

Date Installed: 9/16/13

Annular Space Seal Material: 3/8" Bentonite Chips

Filter Pack Seal Material: #15 Flint Sand

Filter Pack Material: 10-20 Ohio Sand

Borehole Diameter: 6"/10"

Well Material: Flush-Threaded Sch. 40/80 PVC

Well I.D.: 2.00"

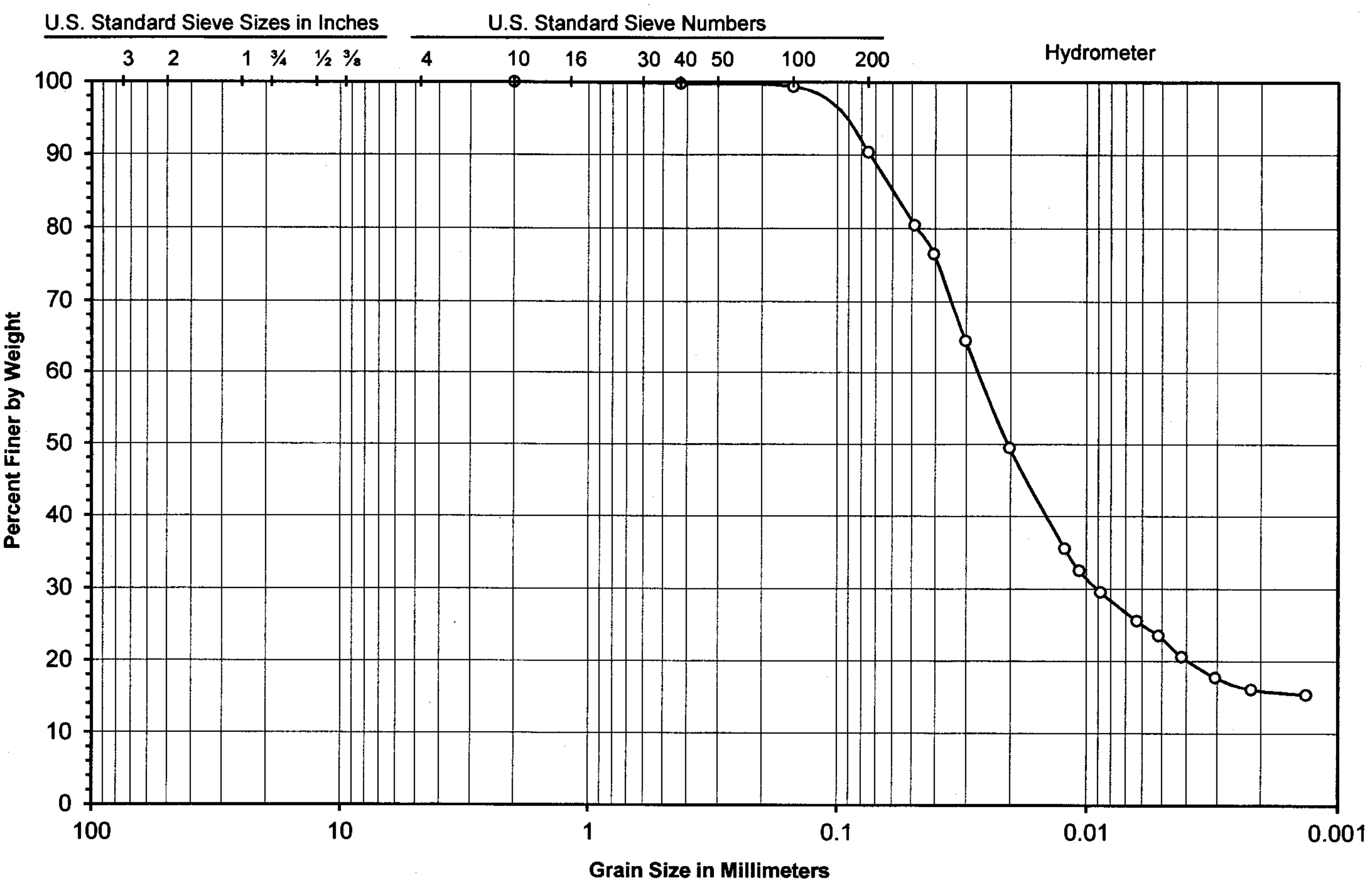
Well Screen Slot Size: 0.01"

Water Level Observations

Date	Time	Depth	Water Elev.	Comments
2A:				
9/18/13	11:45am	34'-6"	633.2	
10/9/13	2:00pm	50'-11"	616.7	
11/4/13	2:03pm	51'-4"	616.3	
2B:				
9/18/13	11:48am	25'-8"	642.0	
10/9/13	2:03pm	60'-10"	606.8	
11/4/13	2:06pm	61'-0"	606.7	

Remarks:

GRAIN SIZE ANALYSIS TEST RESULTS



UNIFIED	GRAVEL	SAND	SILT AND CLAY
AASHTO	GRAVEL	SAND	SILT AND CLAY

SIEVE SIZE	PERCENT FINER	SPEC
125 mm (5")	-	-
100 mm (4")	-	-
90 mm (3½")	-	-
75 mm (3")	-	-
63 mm (2½")	-	-
50 mm (2")	-	-
37.5 mm (1½")	-	-
31.5 mm (1¼")	-	-
25.0 mm (1")	-	-
19.0 mm (3/4")	-	-
12.5 mm (1/2")	-	-
9.5 mm (3/8")	-	-
4.75 mm (# 4)	-	-
2.36 mm (# 8)	-	-
2.00 mm (# 10)	100.0	-
1.18 mm (# 16)	-	-
600 µm (# 30)	-	-
425 µm (# 40)	99.8	-
300 µm (# 50)	-	-
180 µm (# 80)	-	-
150 µm (# 100)	99.4	-
75 µm (# 200)	90.4	-

SAMPLE ID: B-1, S-14, 53.5'-55'

Description/Classification

Clayey Silt, trace fine sand (ML)

Coefficients

D_{60} = 0.027 mm
D_{30} = 0.0090 mm
D_{10} = -- mm
C_u = --
C_c = --

Other Test Data

Moisture Content = 21.5 percent
Dry Unit Weight = lbs./cu.ft.
Hydraulic Conductivity = cm./sec.
Liquid Limit =
Plastic Limit =

Wisconsin Testing Laboratories LLC

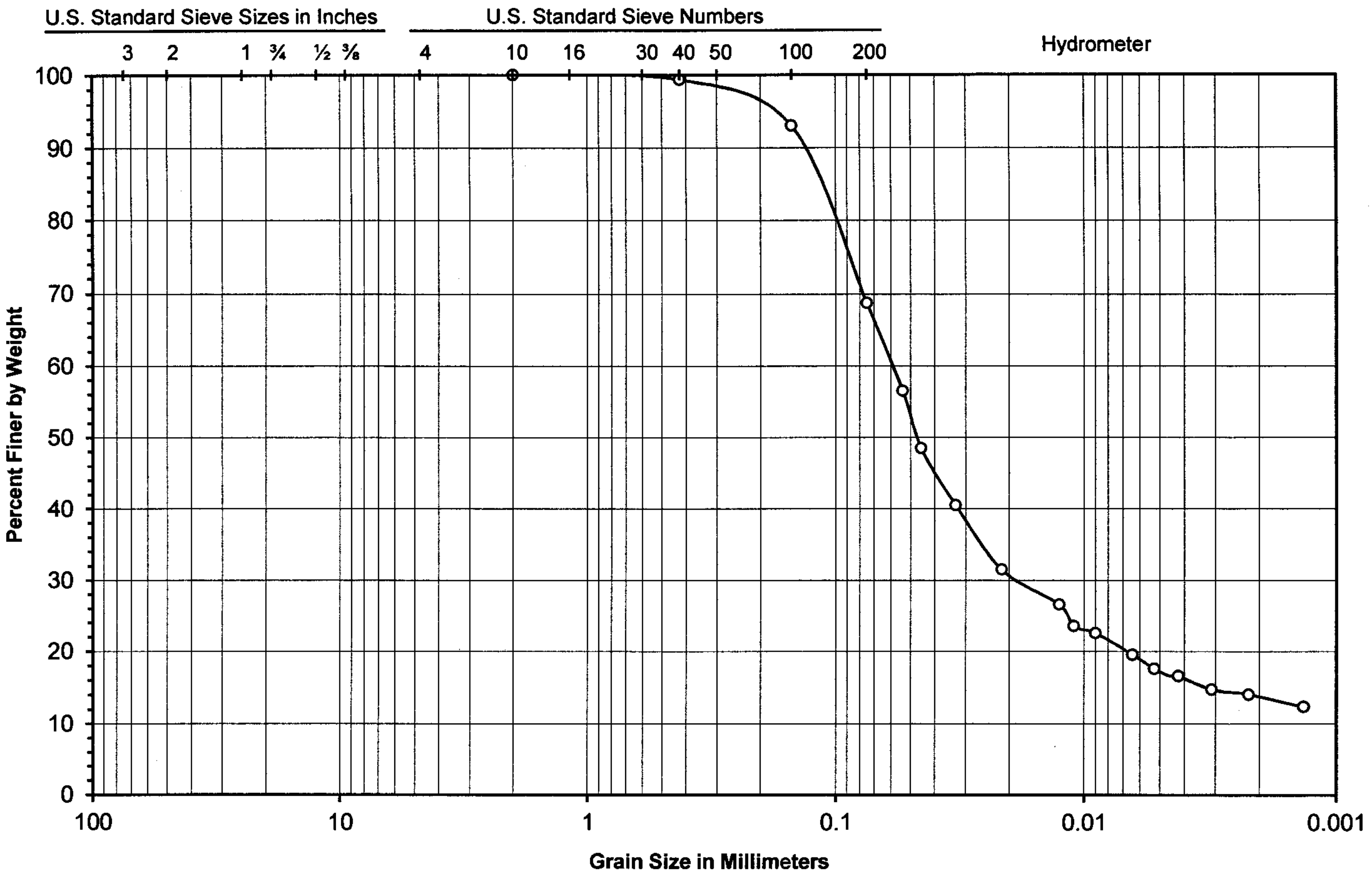
Job No. 0709-13-001

UPPER LAKE PARK BUFF INVESTIGATION
Lake Street, North of Jackson Street
Port Washington, Wisconsin

For City of Port Washington

October 4, 2013

GRAIN SIZE ANALYSIS TEST RESULTS



UNIFIED	GRAVEL	SAND	SILT AND CLAY
AASHTO	GRAVEL	SAND	SILT AND CLAY

SIEVE SIZE	PERCENT FINER	SPEC
125 mm (5")	-	-
100 mm (4")	-	-
90 mm (3½")	-	-
75 mm (3")	-	-
63 mm (2½")	-	-
50 mm (2")	-	-
37.5 mm (1½")	-	-
31.5 mm (1¼")	-	-
25.0 mm (1")	-	-
19.0 mm (3/4")	-	-
12.5 mm (1/2")	-	-
9.5 mm (3/8")	-	-
4.75 mm (# 4)	-	-
2.36 mm (# 8)	-	-
2.00 mm (# 10)	100.0	-
1.18 mm (# 16)	-	-
600 µm (# 30)	-	-
425 µm (# 40)	99.4	-
300 µm (# 50)	-	-
180 µm (# 80)	-	-
150 µm (# 100)	93.0	-
75 µm (# 200)	68.8	-

SAMPLE ID: B-1, S-15B, 56'-57.5'

Description/Classification

Fine Sandy Clayey Silt (ML)

Coefficients

D_{60} =	0.058	mm
D_{30} =	0.018	mm
D_{10} =	--	mm
Cu =	--	
Cc =	--	

Other Test Data

Moisture Content =	17.6	percent
Dry Unit Weight =		lbs./cu.ft.
Hydraulic Conductivity =		cm./sec.
Liquid Limit =		
Plastic Limit =		

Wisconsin Testing Laboratories LLC

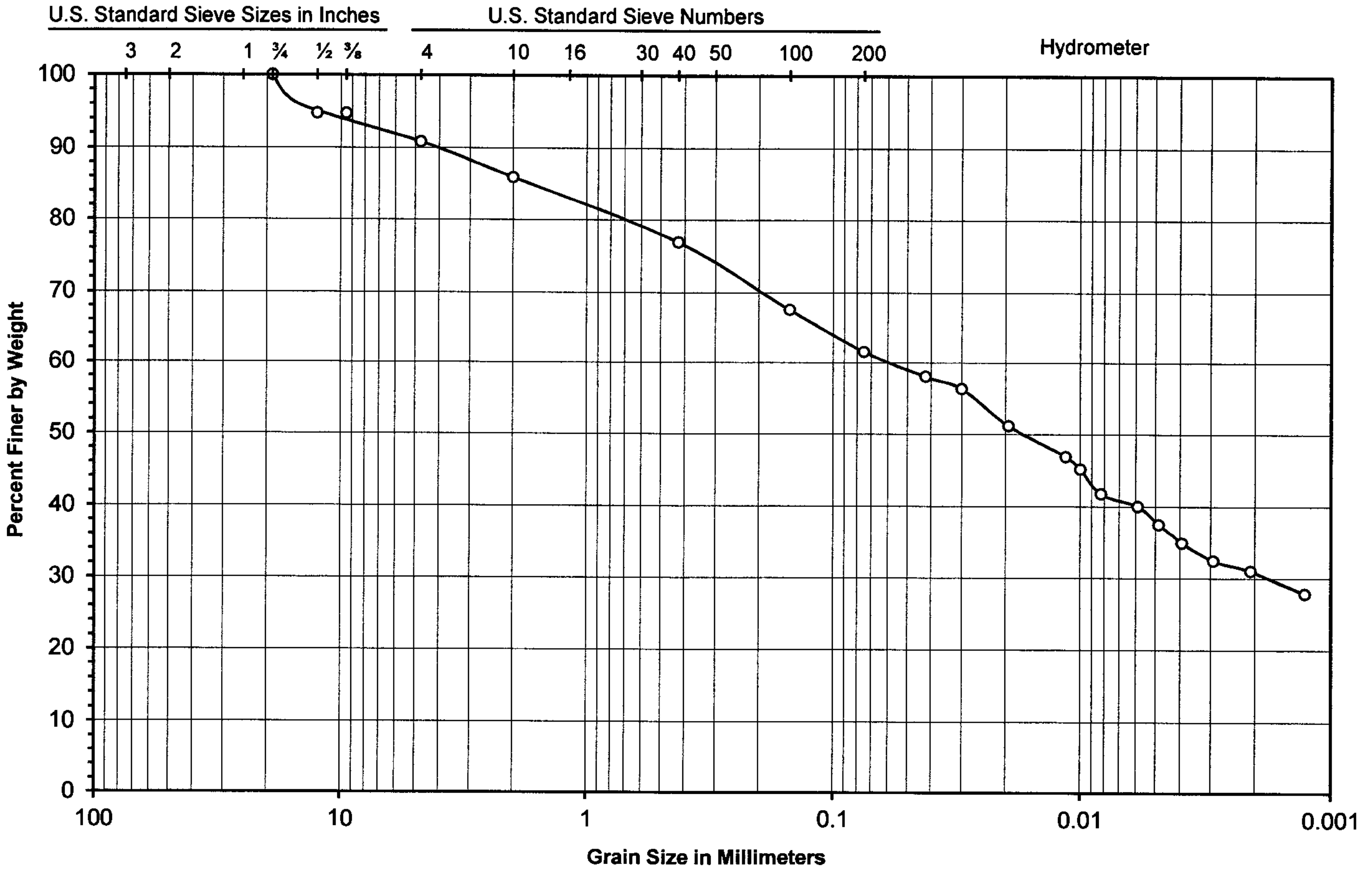
Job No. 0709-13-001

UPPER LAKE PARK BUFF INVESTIGATION
Lake Street, North of Jackson Street
Port Washington, Wisconsin

For City of Port Washington

October 4, 2013

GRAIN SIZE ANALYSIS TEST RESULTS



UNIFIED	GRAVEL	SAND	SILT AND CLAY
AASHTO	GRAVEL	SAND	SILT AND CLAY

SIEVE SIZE	PERCENT FINER	SPEC
125 mm (5")	-	-
100 mm (4")	-	-
90 mm (3½")	-	-
75 mm (3")	-	-
63 mm (2½")	-	-
50 mm (2")	-	-
37.5 mm (1½")	-	-
31.5 mm (1¼")	-	-
25.0 mm (1")	-	-
19.0 mm (3/4")	100.0	-
12.5 mm (1/2")	94.7	-
9.5 mm (3/8")	94.7	-
4.75 mm (# 4)	90.8	-
2.36 mm (# 8)	-	-
2.00 mm (# 10)	85.9	-
1.18 mm (# 16)	-	-
600 µm (# 30)	-	-
425 µm (# 40)	76.9	-
300 µm (# 50)	-	-
180 µm (# 80)	-	-
150 µm (# 100)	67.5	-
75 µm (# 200)	61.5	-

SAMPLE ID: B-2, S-7, 18.5'-20'

Description/Classification

Sandy Clayey Silt (ML)

Coefficients

D_{60} = 0.060 mm
D_{30} = 0.0018 mm
D_{10} = -- mm
C_u = --
C_c = --

Other Test Data

Moisture Content = 35.1 percent
Dry Unit Weight = lbs./cu.ft.
Hydraulic Conductivity = cm./sec.
Liquid Limit =
Plastic Limit =

Wisconsin Testing Laboratories LLC

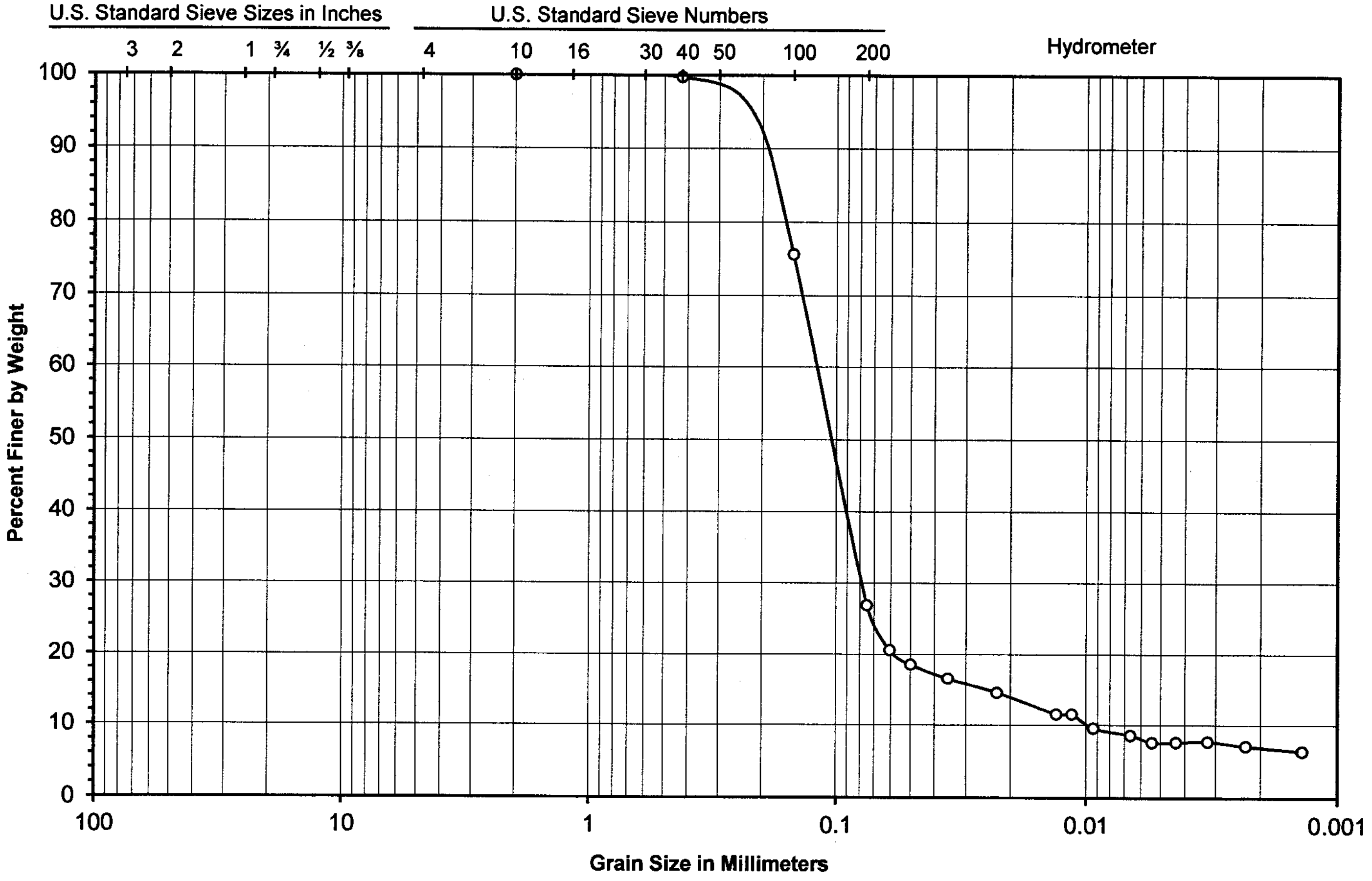
Job No. 0709-13-001

UPPER LAKE PARK BUFF INVESTIGATION
Lake Street, North of Jackson Street
Port Washington, Wisconsin

For City of Port Washington

October 4, 2013

GRAIN SIZE ANALYSIS TEST RESULTS



UNIFIED	GRAVEL	SAND	SILT AND CLAY
AASHTO	GRAVEL	SAND	SILT AND CLAY

SIEVE SIZE	PERCENT FINER	SPEC
125 mm (5")	-	-
100 mm (4")	-	-
90 mm (3½")	-	-
75 mm (3")	-	-
63 mm (2½")	-	-
50 mm (2")	-	-
37.5 mm (1½")	-	-
31.5 mm (1¼")	-	-
25.0 mm (1")	-	-
19.0 mm (3/4")	-	-
12.5 mm (1/2")	-	-
9.5 mm (3/8")	-	-
4.75 mm (# 4)	-	-
2.36 mm (# 8)	-	-
2.00 mm (# 10)	100.0	-
1.18 mm (# 16)	-	-
600 µm (# 30)	-	-
425 µm (# 40)	99.7	-
300 µm (# 50)	-	-
180 µm (# 80)	-	-
150 µm (# 100)	75.6	-
75 µm (# 200)	26.9	-

SAMPLE ID: B-2, S-13, 48.5'-50'

Description/Classification

Silty Fine Sand, trace clay (SM)

Coefficients

D_{60} = 0.12 mm
D_{30} = 0.078 mm
D_{10} = 0.010 mm
C_u = 12.00
C_c = 5.07

Other Test Data

Moisture Content = 17.1 percent
Dry Unit Weight = lbs./cu.ft.
Hydraulic Conductivity = cm./sec.
Liquid Limit =
Plastic Limit =

Wisconsin Testing Laboratories LLC

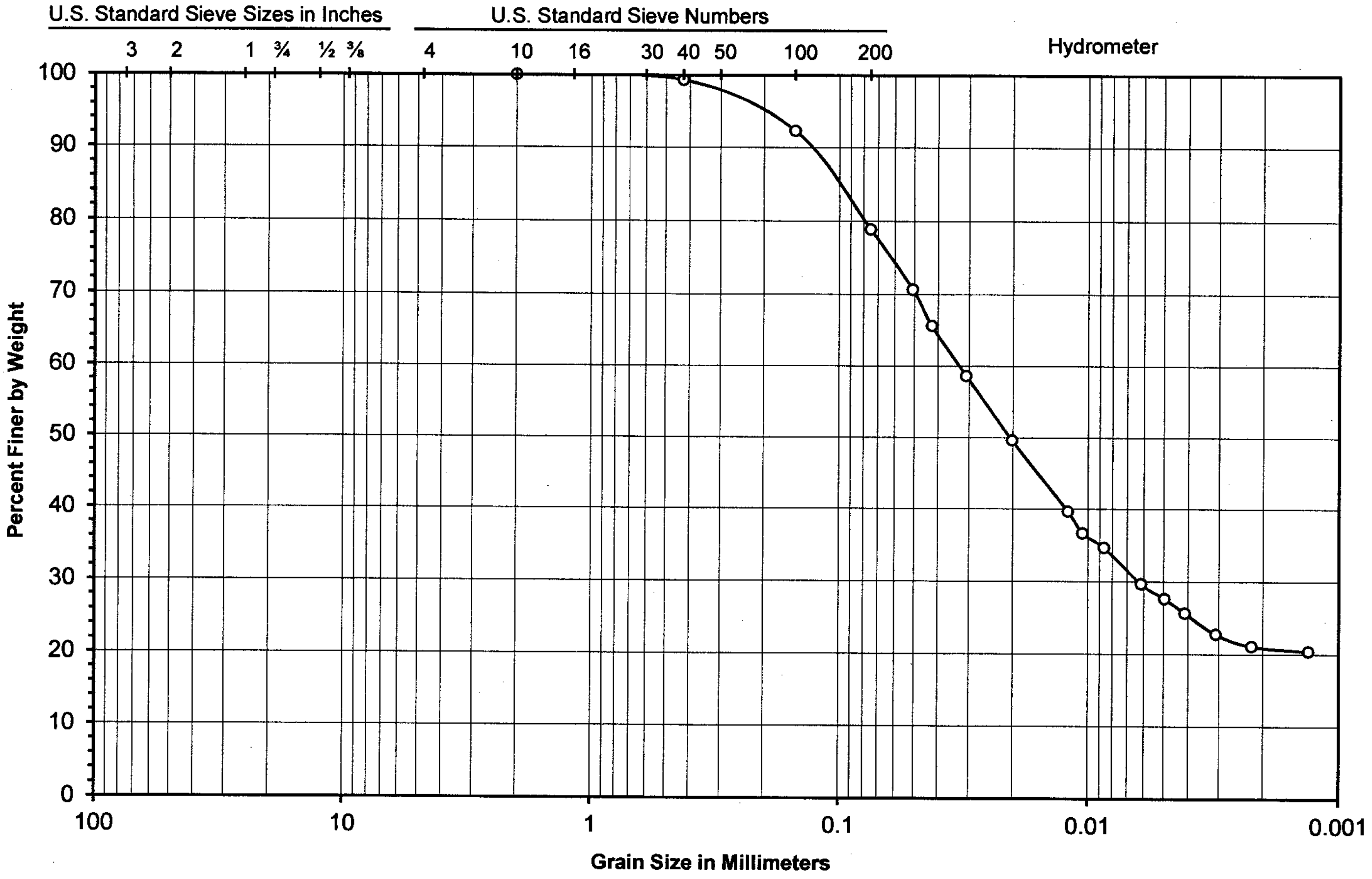
Job No. 0709-13-001

UPPER LAKE PARK BUFF INVESTIGATION
Lake Street, North of Jackson Street
Port Washington, Wisconsin

For City of Port Washington

October 4, 2013

GRAIN SIZE ANALYSIS TEST RESULTS



UNIFIED	GRAVEL	SAND	SILT AND CLAY
AASHTO	GRAVEL	SAND	SILT AND CLAY

SIEVE SIZE	PERCENT FINER	SPEC
125 mm (5")	-	-
100 mm (4")	-	-
90 mm (3½")	-	-
75 mm (3")	-	-
63 mm (2½")	-	-
50 mm (2")	-	-
37.5 mm (1½")	-	-
31.5 mm (1¼")	-	-
25.0 mm (1")	-	-
19.0 mm (3/4")	-	-
12.5 mm (1/2")	-	-
9.5 mm (3/8")	-	-
4.75 mm (# 4)	-	-
2.36 mm (# 8)	-	-
2.00 mm (# 10)	100.0	-
1.18 mm (# 16)	-	-
600 µm (# 30)	-	-
425 µm (# 40)	99.3	-
300 µm (# 50)	-	-
180 µm (# 80)	-	-
150 µm (# 100)	92.3	-
75 µm (# 200)	78.9	-

SAMPLE ID: B-2, S-14, 51'-52.5'

Description/Classification

Fine Sandy Clayey Silt (ML)

Coefficients

D_{60} = 0.033 mm
D_{30} = 0.0060 mm
D_{10} = -- mm
C_u = --
C_c = --

Other Test Data

Moisture Content =	14.7	percent
Dry Unit Weight =		lbs./cu.ft.
Hydraulic Conductivity =		cm./sec.
Liquid Limit =		
Plastic Limit =		

Wisconsin Testing Laboratories LLC

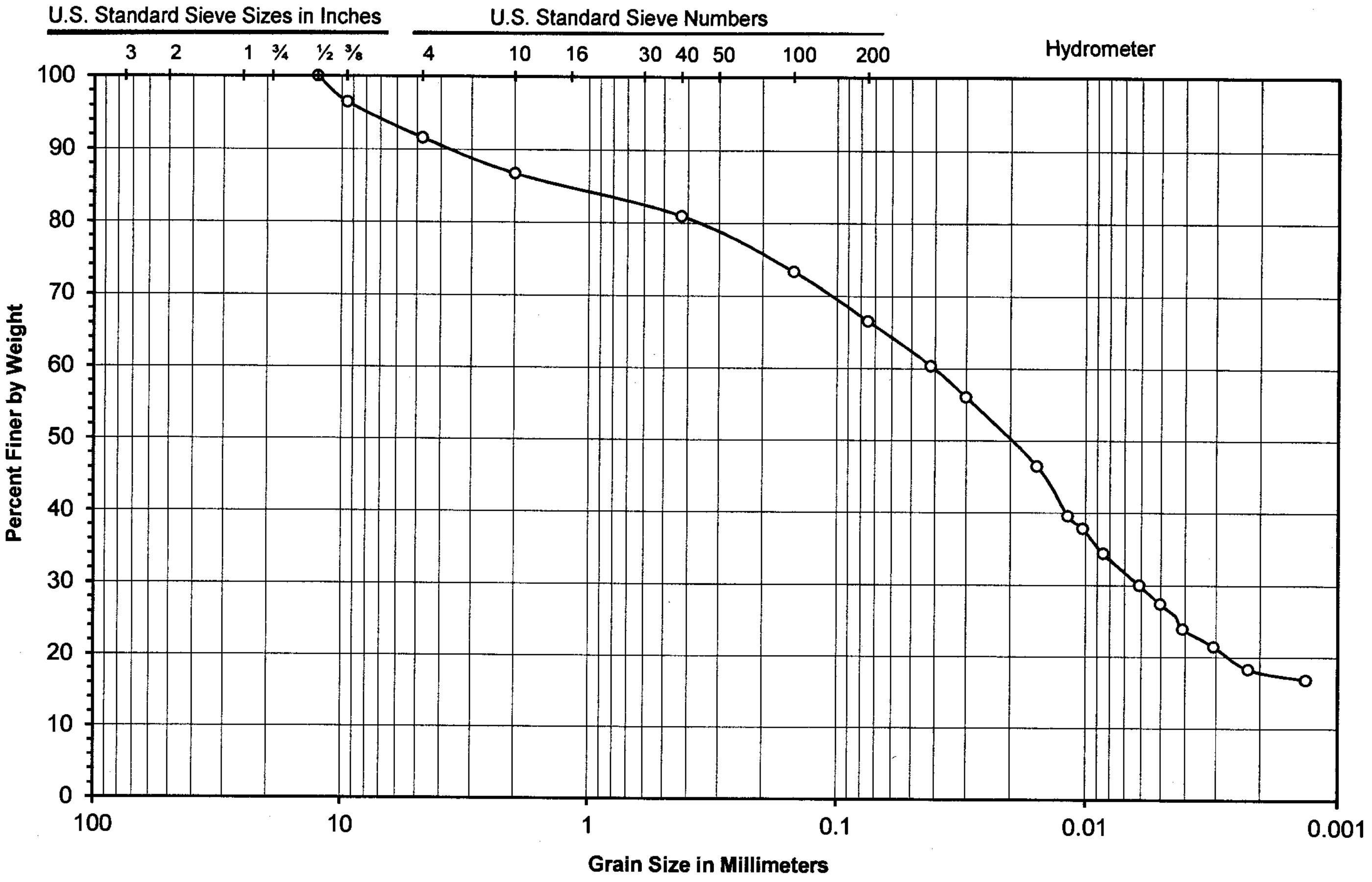
Job No. 0709-13-001

UPPER LAKE PARK BUFF INVESTIGATION
Lake Street, North of Jackson Street
Port Washington, Wisconsin

For City of Port Washington

October 4, 2013

GRAIN SIZE ANALYSIS TEST RESULTS



UNIFIED	GRAVEL	SAND	SILT AND CLAY
AASHTO	GRAVEL	SAND	SILT AND CLAY

SIEVE SIZE	PERCENT FINER	SPEC
125 mm (5")	-	-
100 mm (4")	-	-
90 mm (3½")	-	-
75 mm (3")	-	-
63 mm (2½")	-	-
50 mm (2")	-	-
37.5 mm (1½")	-	-
31.5 mm (1¼")	-	-
25.0 mm (1")	-	-
19.0 mm (3/4")	-	-
12.5 mm (1/2")	100.0	-
9.5 mm (3/8")	96.5	-
4.75 mm (# 4)	91.6	-
2.36 mm (# 8)	-	-
2.00 mm (# 10)	86.7	-
1.18 mm (# 16)	-	-
600 µm (# 30)	-	-
425 µm (# 40)	80.9	-
300 µm (# 50)	-	-
180 µm (# 80)	-	-
150 µm (# 100)	73.3	-
75 µm (# 200)	66.5	-

SAMPLE ID: B-2, S-24, 93.5'-95'

Description/Classification

Sandy Clayey Silt (ML)

Coefficients

D_{60} = 0.040 mm
D_{30} = 0.0060 mm
D_{10} = -- mm
Cu = --
Cc = --

Other Test Data

Moisture Content = 8.8 percent
Dry Unit Weight = lbs./cu.ft.
Hydraulic Conductivity = cm./sec.
Liquid Limit =
Plastic Limit =

Wisconsin Testing Laboratories LLC

Job No. 0709-13-001

UPPER LAKE PARK BUFF INVESTIGATION
Lake Street, North of Jackson Street
Port Washington, Wisconsin

For City of Port Washington

October 4, 2013

GeoTest, Inc.

Consolidated Undrained Triaxial Test (ASTM D4767)

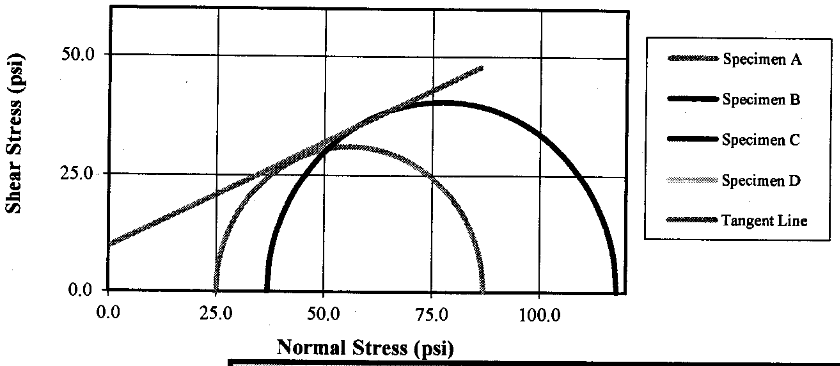
Date: 10/21/2013

Checked By: Jeff Anderson

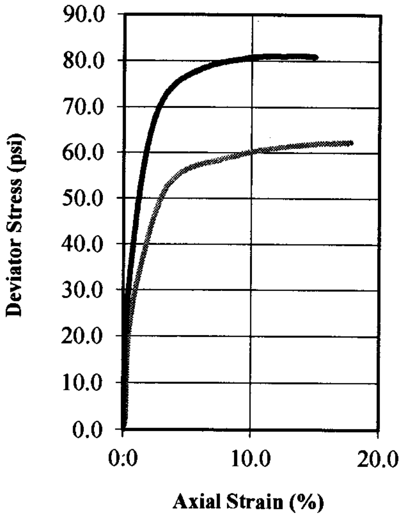
Date: 10/18/2013

Tested By: Jeffery R. Anderson

Mohr Stress Circles at 15% Axial Strain Criterion



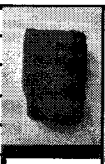
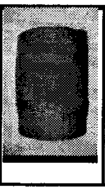
Deviator Stress Vs. Axial Strain



Initial	A	B	C	D
Water Content (%)	17.9	15.9		
Dry Density (pcf)	114.0	119.4		
Saturation (%)	99.31	102.25		
Void Ratio	0.487	0.420		
Diameter (in)	2.834	2.787		
Height (in)	5.615	5.562		
Specific Gravity	2.72	2.72		
Liquid Limit	0	0		
Plastic Limit	0	0		
After Consolidation	**A**	**B**	**C**	**D**
B-Value	1.00	1.00		
Water Content (%)	21.7	19.8		
Dry Density (pcf)	122.24	123.88		
Saturation (%)	100.00	100.00		
Void Ratio	0.389	0.371		
Effective Stress (psi)	32.5	34.6		
Back Press. (psi)	97.5	97.6		
Rate of Strain	0.01146	0.00555		
After Shear	**A**	**B**	**C**	**D**
σ'1 at Failure (psi)	86.97	117.81		
σ'3 at Failure (psi)	25.00	36.90		

15% Axial Strain Criterion	
C (psi)	4.4
C' (psi)	9.7
Ø (deg)	25.1
Ø' (deg)	23.9

Project:	Upper Lake Park Bluff
Location:	
Project Number:	3884
Boring Number:	B-2
Sample Number:	S-12
Depth:	43.5-45.5'
Sample Type:	Undisturbed
Description:	Dark red brown silty CLAY, trace sand and gravel
Test Type	Consolidated Undrained
Remarks	

		N/A	N/A

Failure Photographs

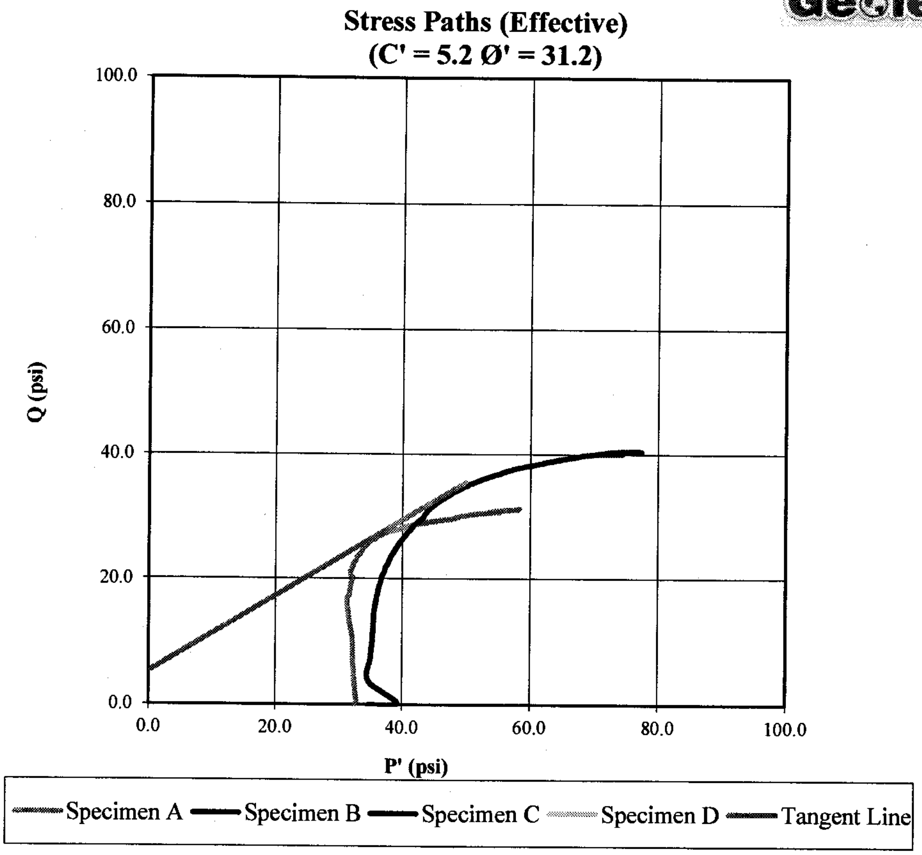
GeoTest

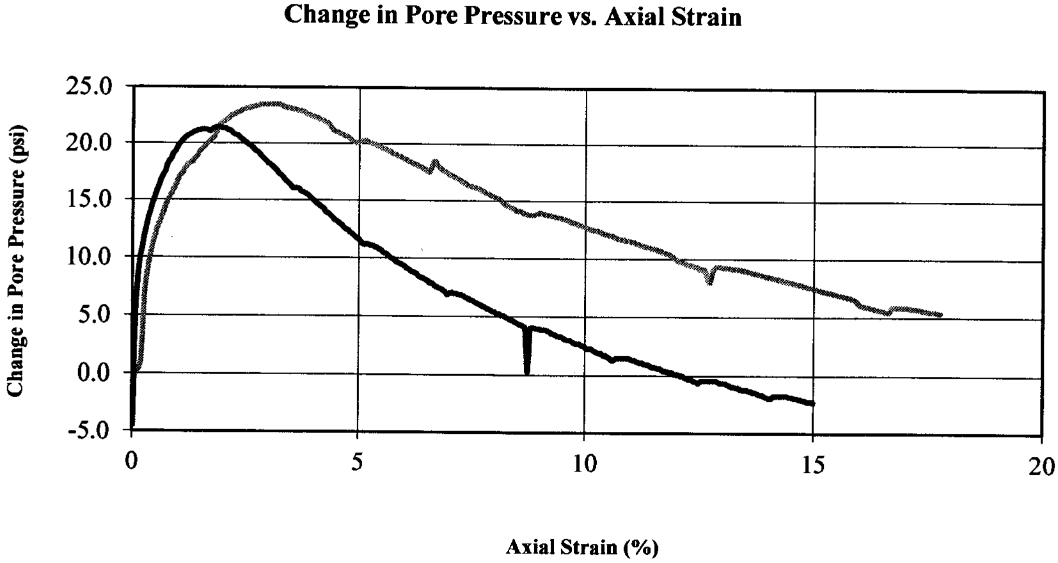
Date:

Checked By:

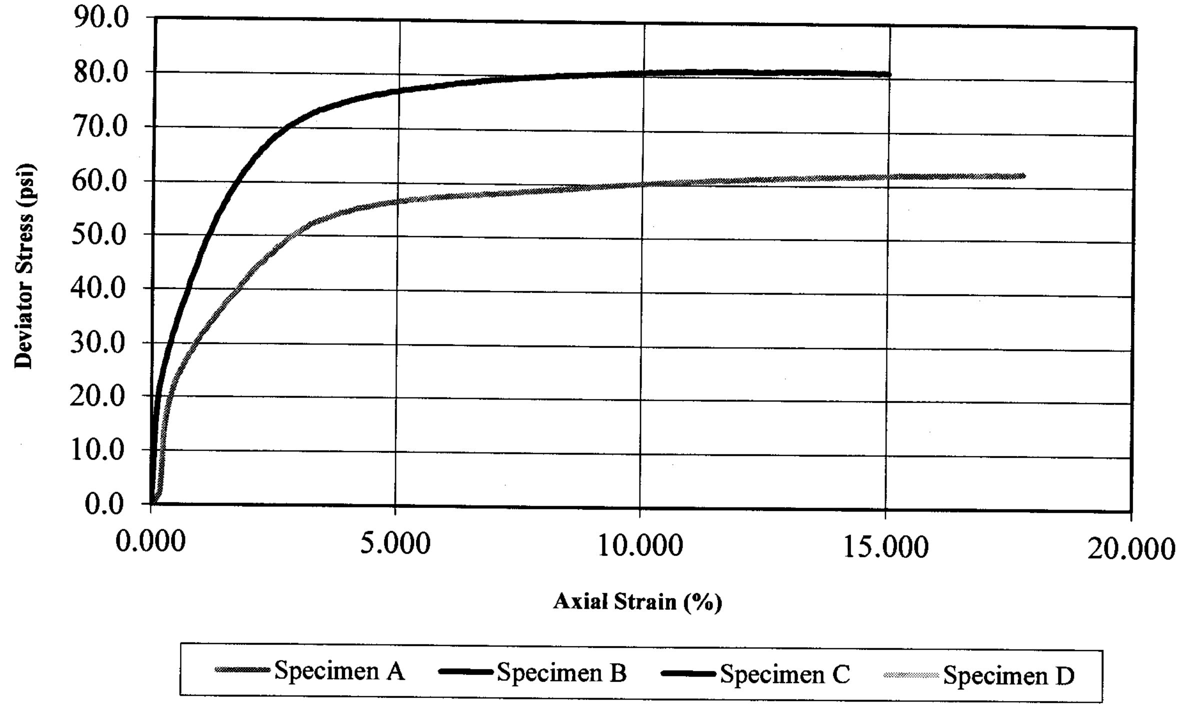
Date:

Tested By:

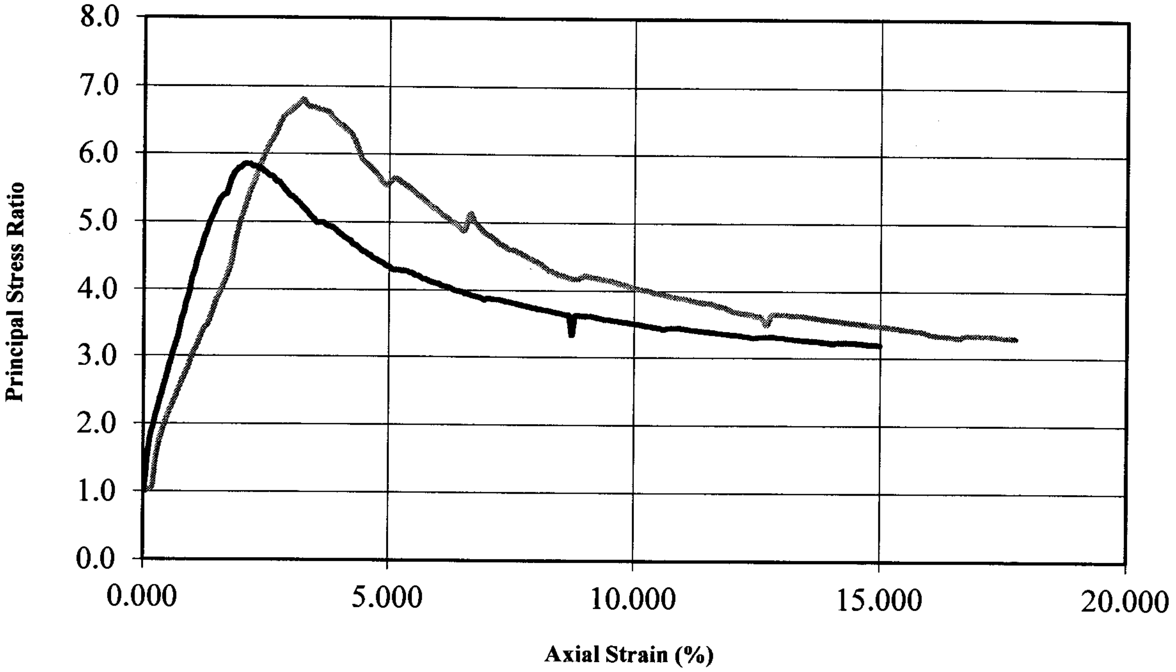


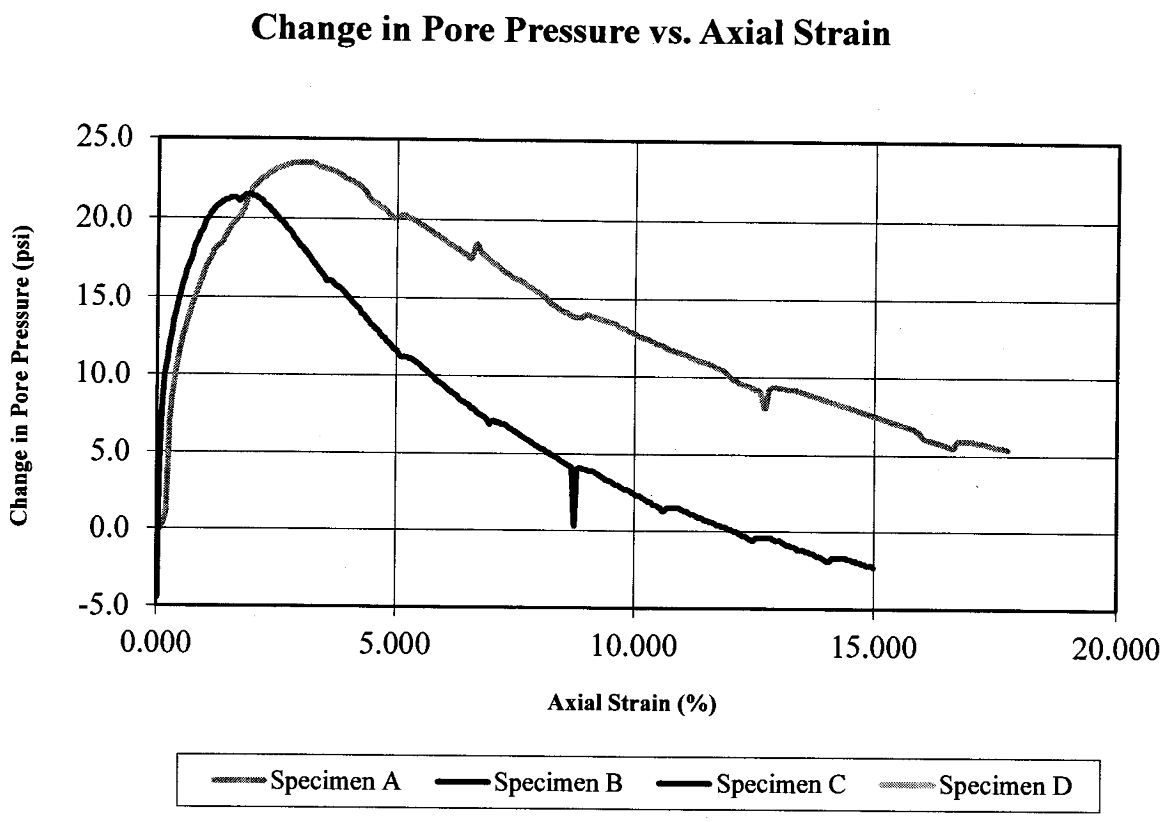


Deviator Stress vs. Axial Strain



Principal Stress Ratio vs. Axial Strain

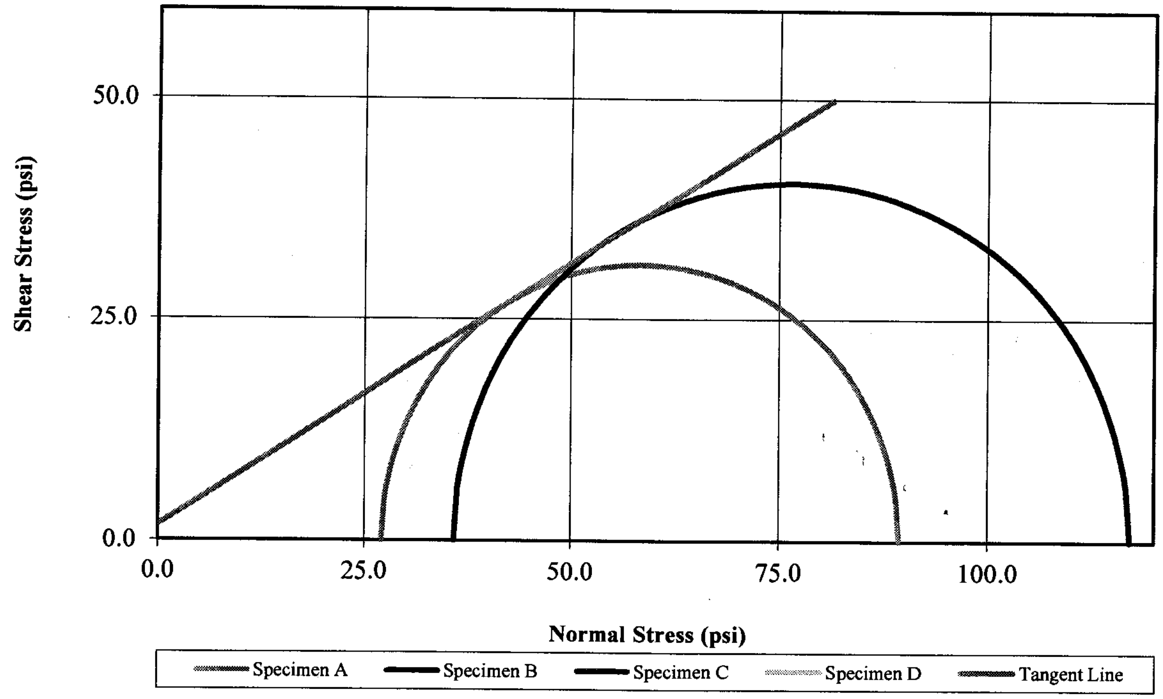



Change in Pore Pressure vs. Axial Strain
Change in Pore Pressure (psi)
25.0
20.0
15.0
10.0
5.0
0.0
-5.0
0.000
5.000
10.000
15.000
20.000
Axial Strain (%)
Specimen A
Specimen B
Specimen C
Specimen D

Mohr Stress Circles at Maximum Deviator Stress Criterion

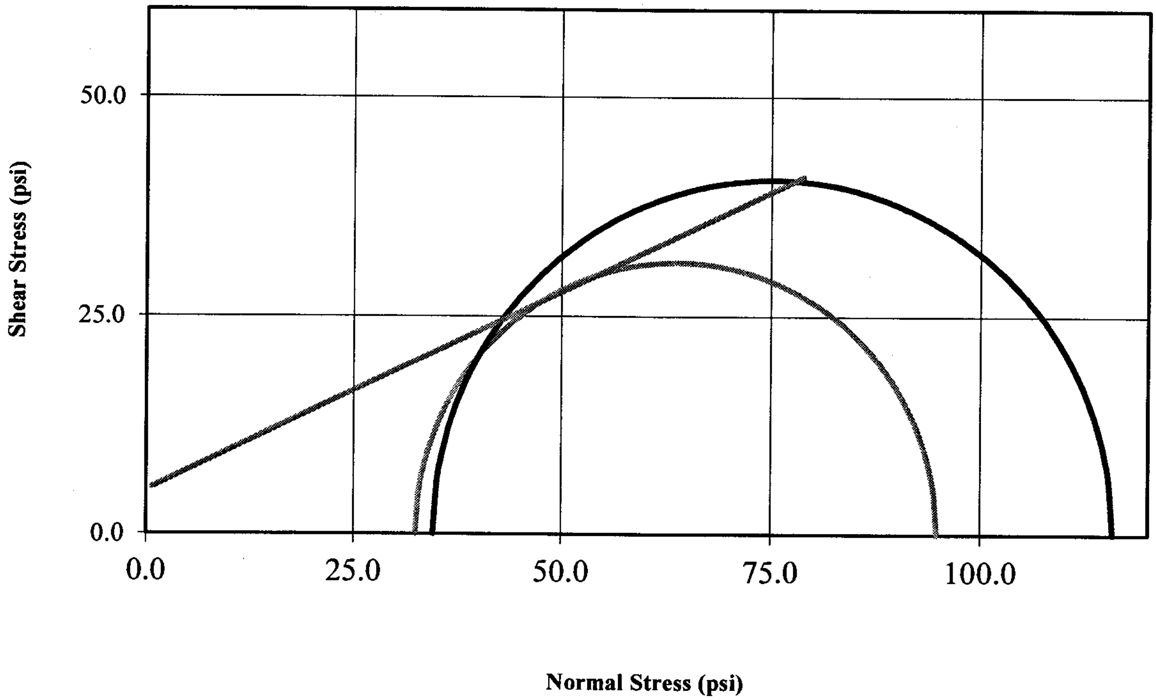
Effective Stress

(C' = 1.7 Ø' = 30.7)



Total Stress

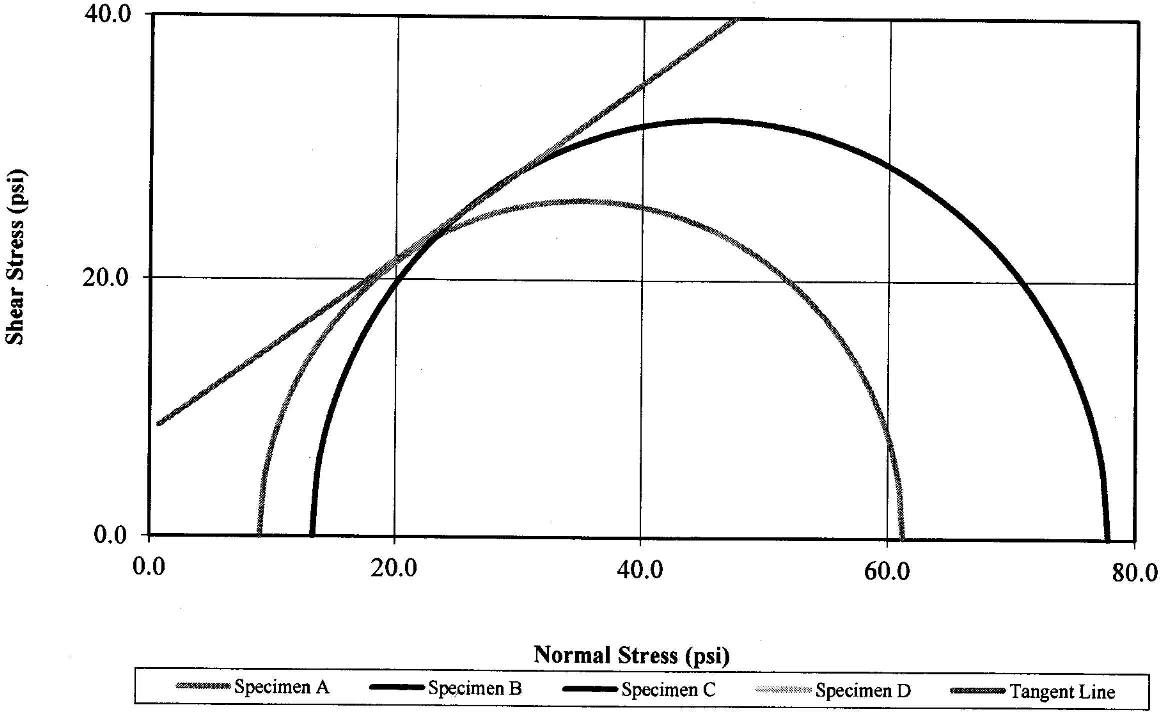
(C = 5.0 Ø = 24.5)



Mohr Stress Circles at Maximum Principal Stress Ratio Criterion

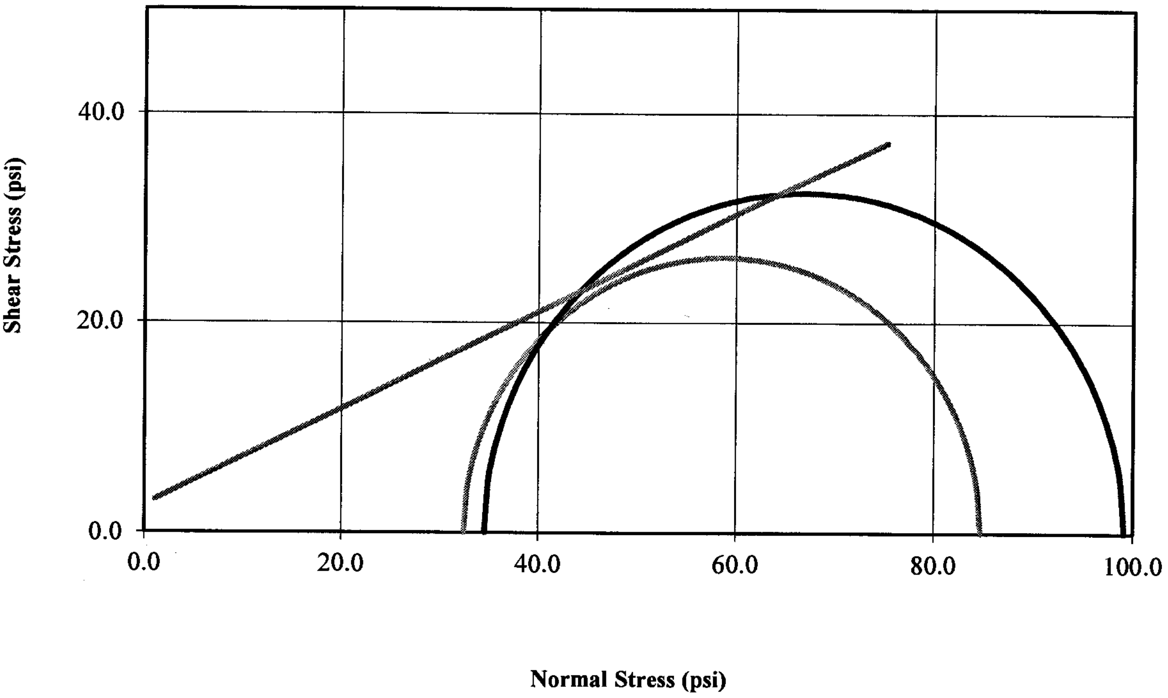
Effective Stress

(C' = 8.1 Ø' = 33.9)



Total Stress

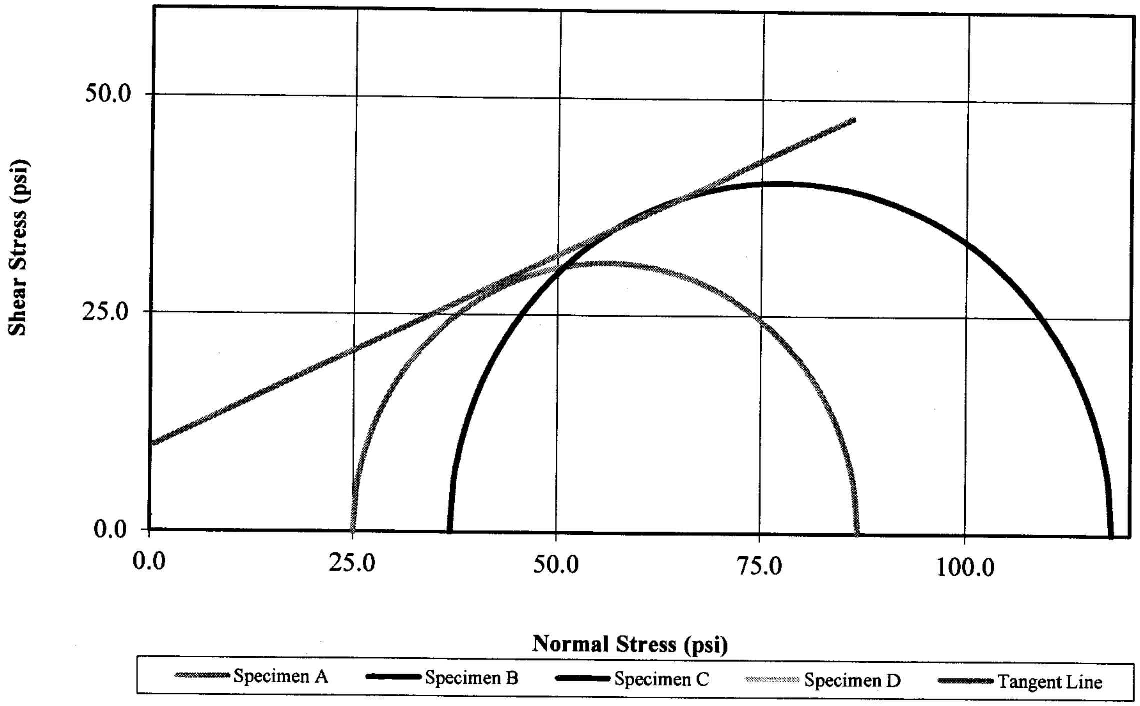
(C = 2.6 Ø = 24.7)



Mohr Stress Circles at 15% Axial Strain Criterion

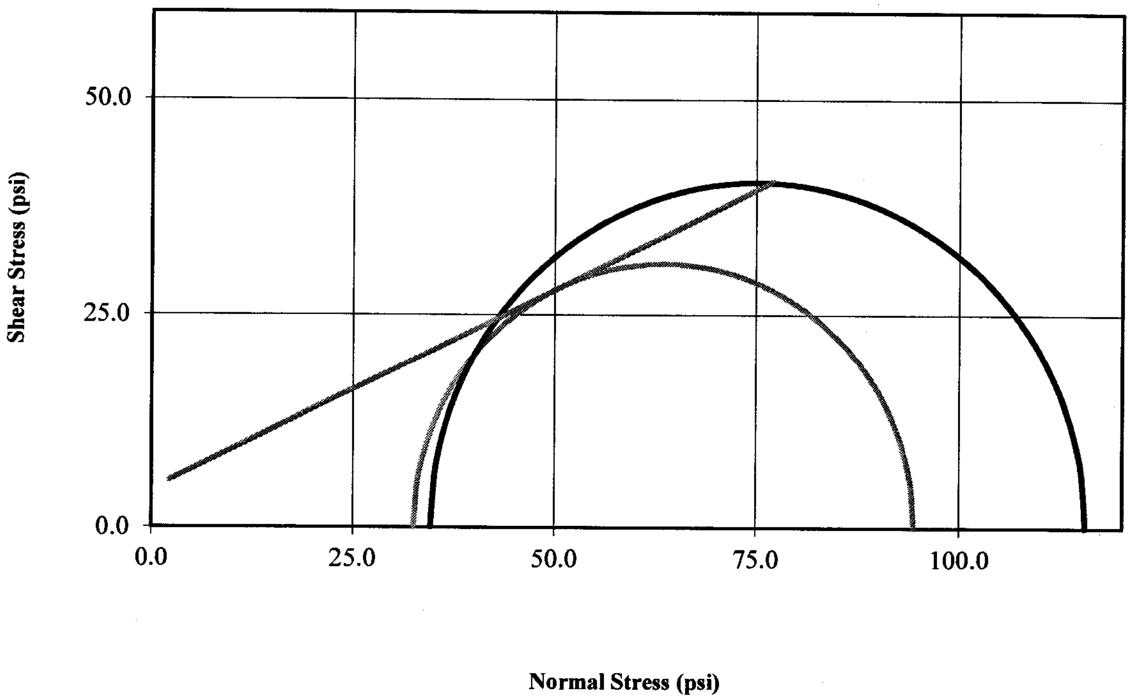
Effective Stress

(C' = 9.7 Ø' = 23.9)

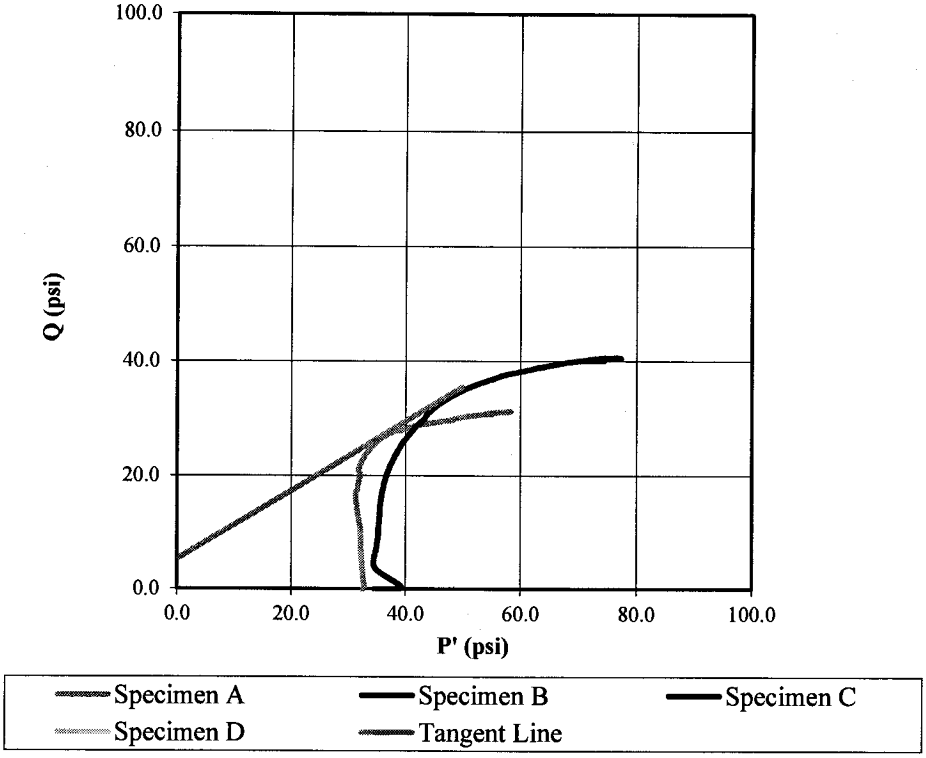


Total Stress

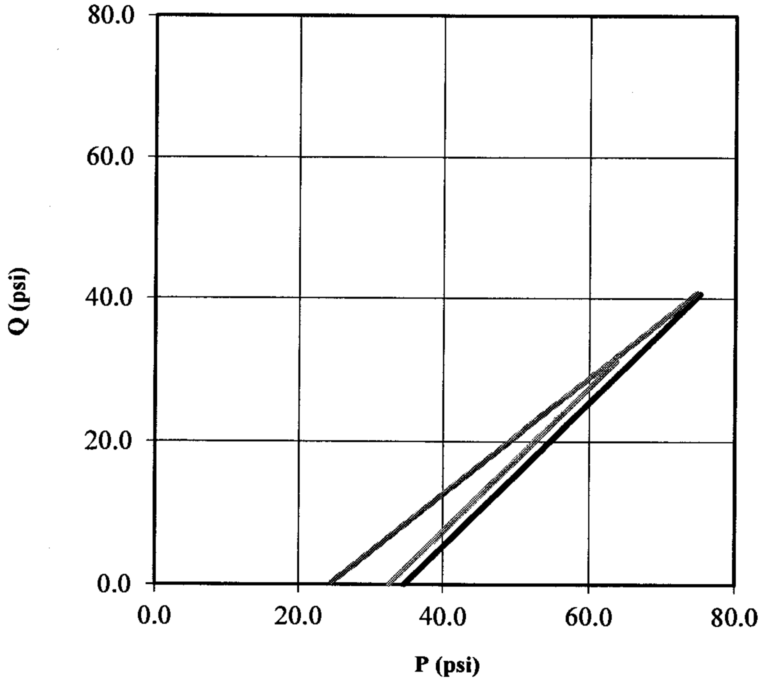
(C = 4.4 Ø = 25.1)



Stress Paths (Effective)
(C' = 5.2 Ø' = 31.2)



Stress Paths (Total)
(C' = -19.5 Ø' = 38.8)



GeoTest, Inc.

Consolidated Undrained Triaxial Test (ASTM D4767)

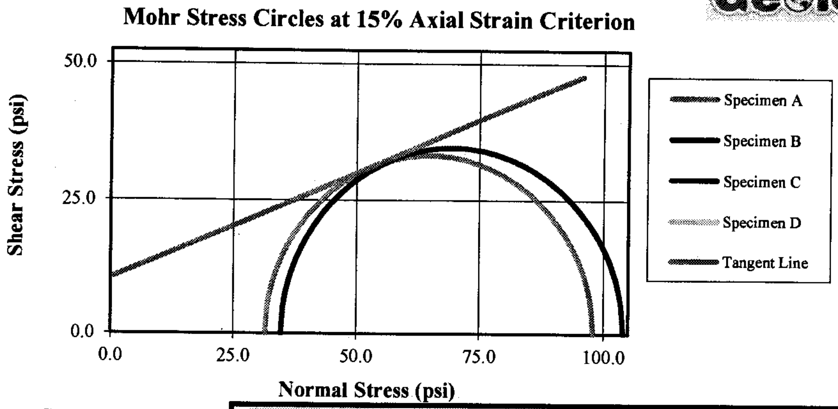
GeoTest

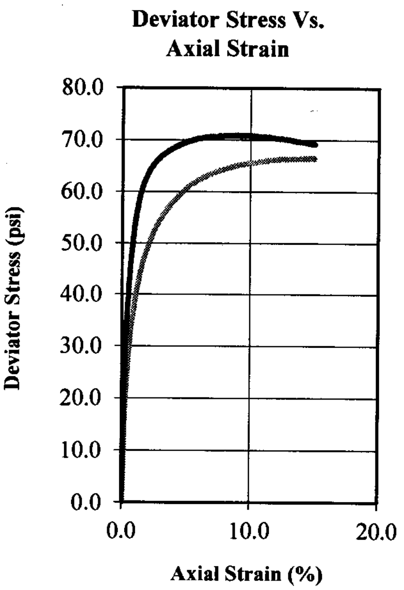
Date:

Checked By:

Date:

Tested By:





Initial	Specimen A	B	C	D
Water Content (%)	19.1	19.3		
Dry Density (pcf)	111.8	111.8		
Saturation (%)	100.51	100.91		
Void Ratio	0.516	0.516		
Diameter (in)	2.852	2.861		
Height (in)	5.611	5.605		
Specific Gravity	2.72	2.72		
Liquid Limit	0	0		
Plastic Limit	0	0		
After Consolidation	**A**	**B**	**C**	**D**
B-Value	1.00	1.00		
Water Content (%)	24.3	23.7		
Dry Density (pcf)	122.84	121.63		
Saturation (%)	100.00	100.00		
Void Ratio	0.382	0.396		
Effective Stress (psi)	41.9	44.8		
Back Press. (psi)	78.1	77.7		
Rate of Strain	0.00606	0.0042		
After Shear	**A**	**B**	**C**	**D**
σ'1 at Failure (psi)	97.92	103.94		
σ'3 at Failure (psi)	31.50	34.70		

15% Axial Strain Criterion	
C (psi)	8.4
C' (psi)	10.5
Ø (deg)	19.6
Ø' (deg)	21.3

Project: Upper Lake Bluff Park

Location:

Project Number: 3884

Boring Number: B-2

Sample Number: S-17

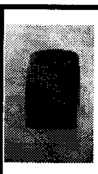
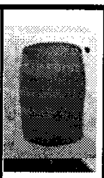
Depth: 58.6-60.6'

Sample Type: Undisturbed

Description: Gray brown silty CLAY

Test Type Consolidated Undrained

Remarks

		N/A	N/A

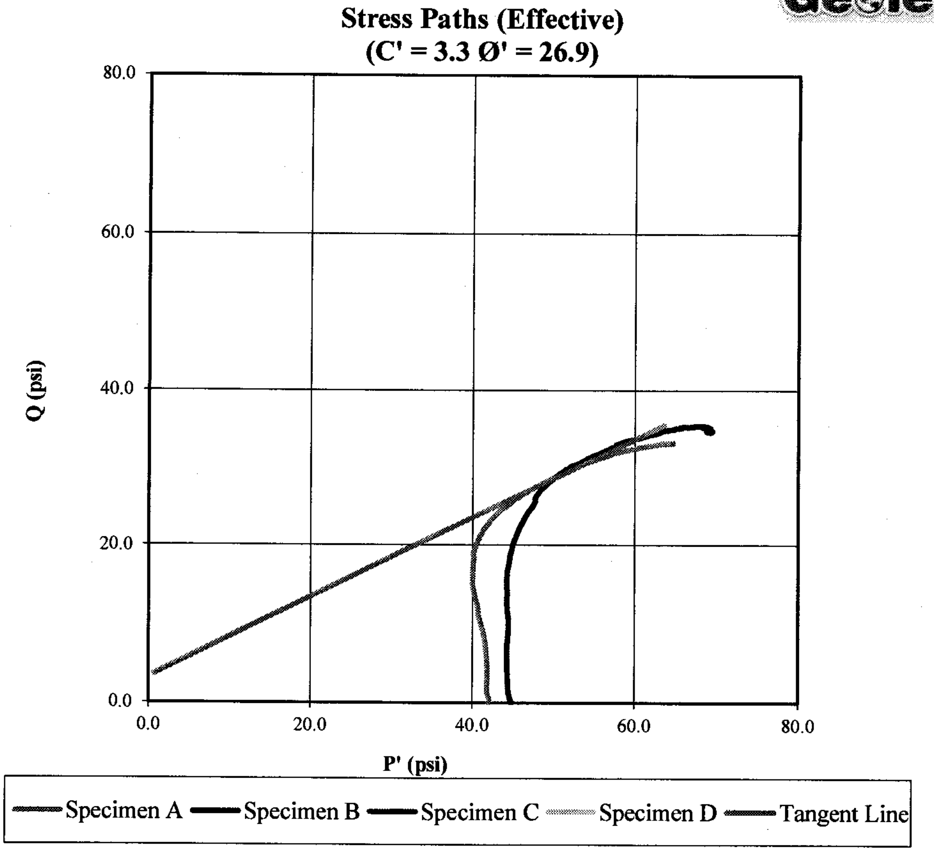
Failure Photographs

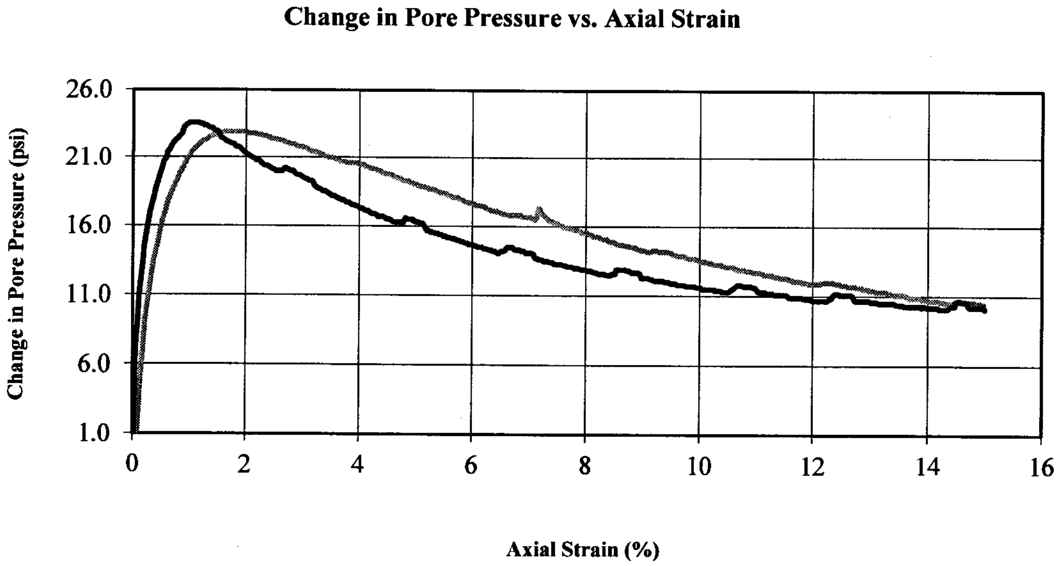
Date:

Checked By:

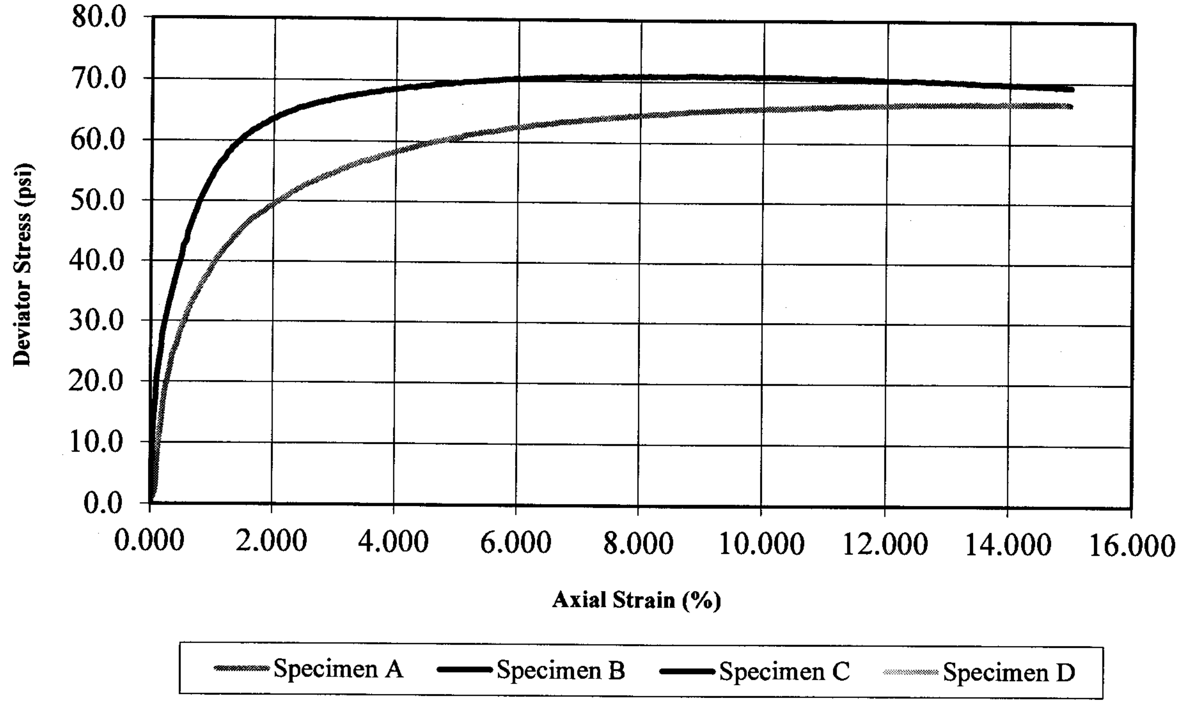
Date:

Tested By:

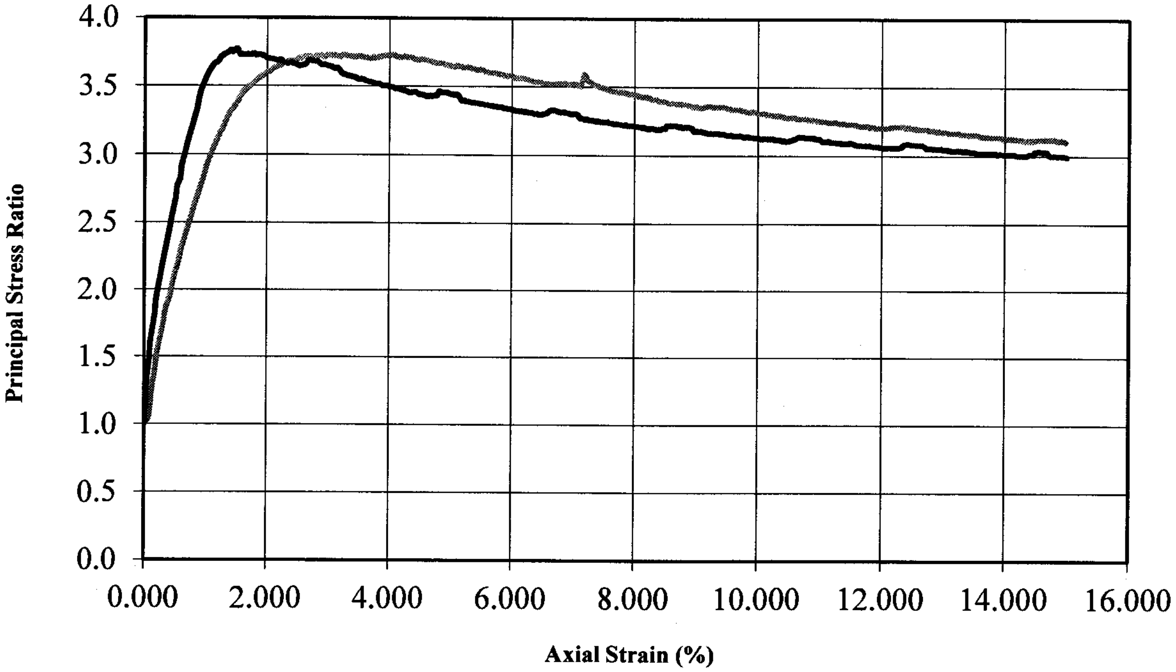


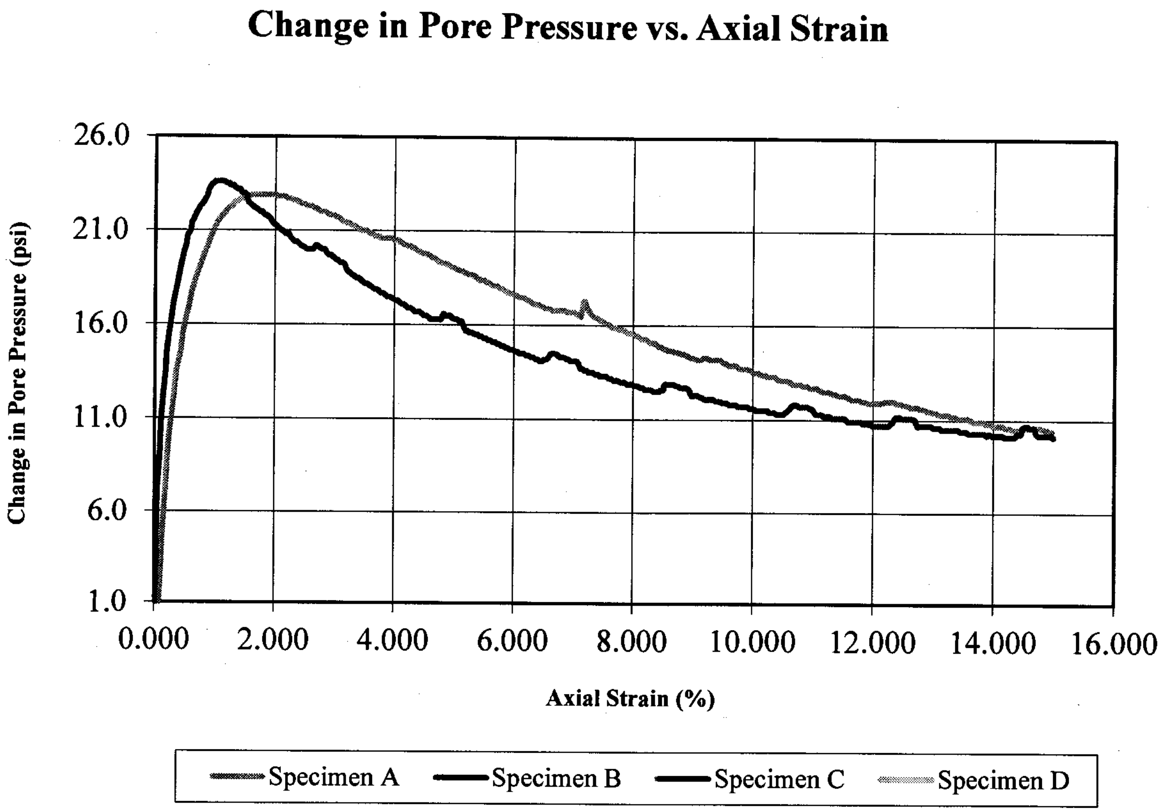


Deviator Stress vs. Axial Strain



Principal Stress Ratio vs. Axial Strain

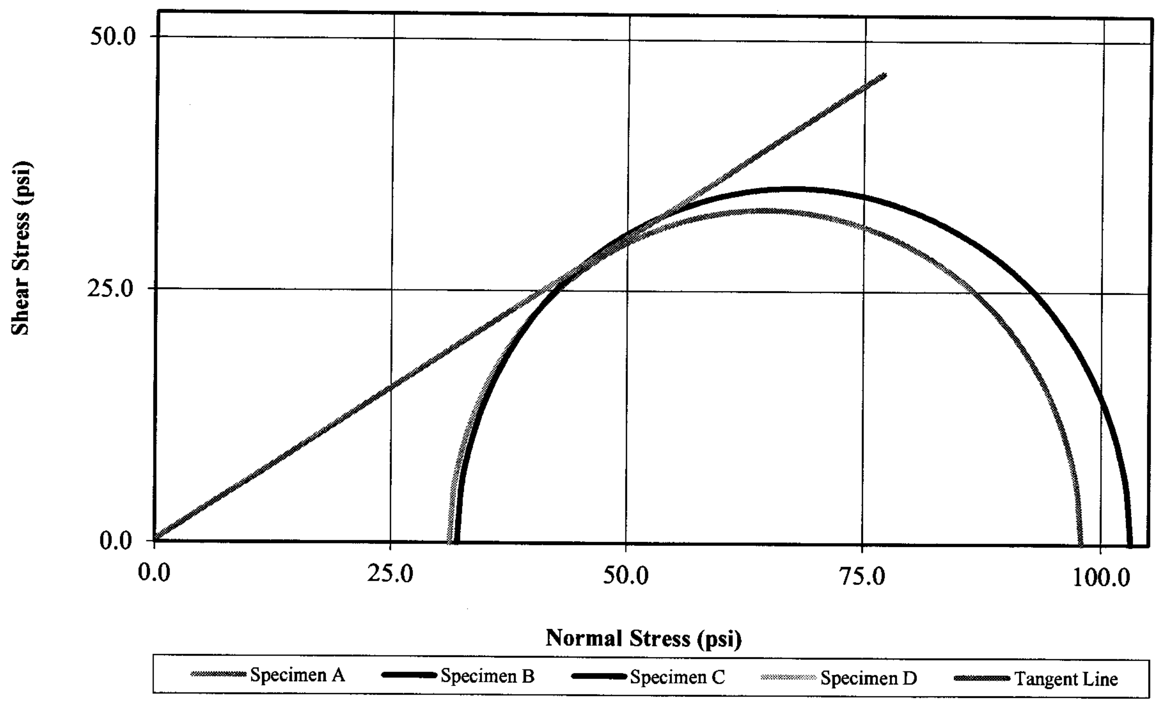



Change in Pore Pressure vs. Axial Strain
Change in Pore Pressure (psi)
26.0
21.0
16.0
11.0
6.0
1.0
0.000
2.000
4.000
6.000
8.000
10.000
12.000
14.000
16.000
Axial Strain (%)
Specimen A
Specimen B
Specimen C
Specimen D

Mohr Stress Circles at Maximum Deviator Stress Criterion

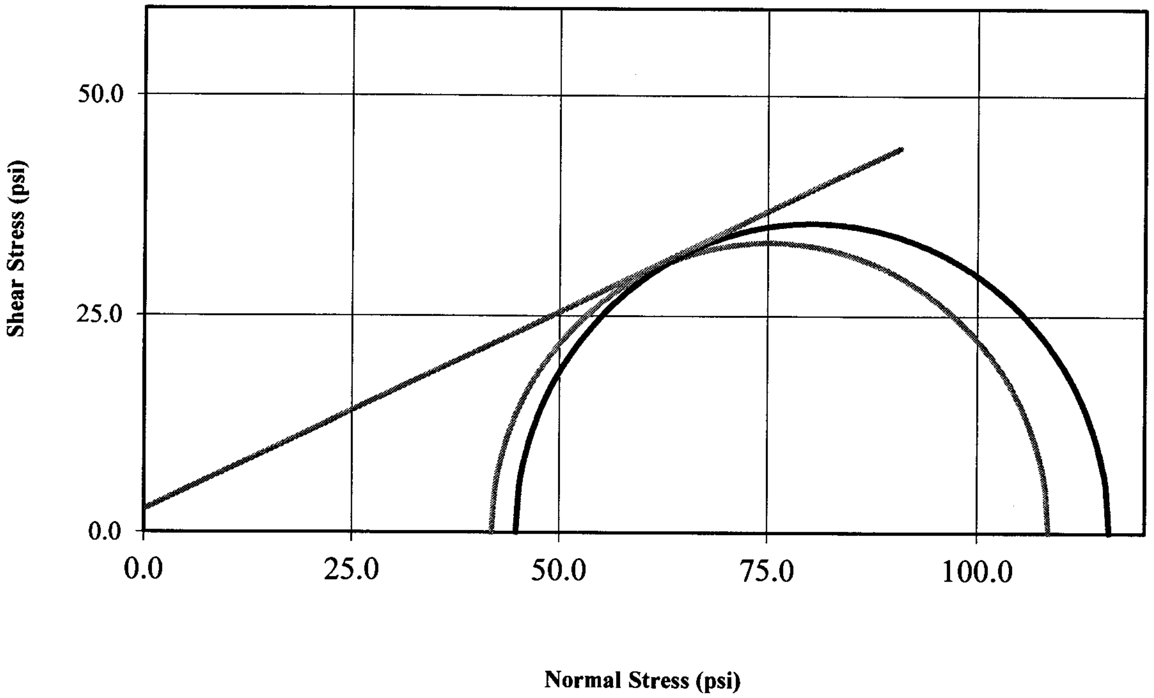
Effective Stress

(C' = 0.2 Ø' = 31.2)



Total Stress

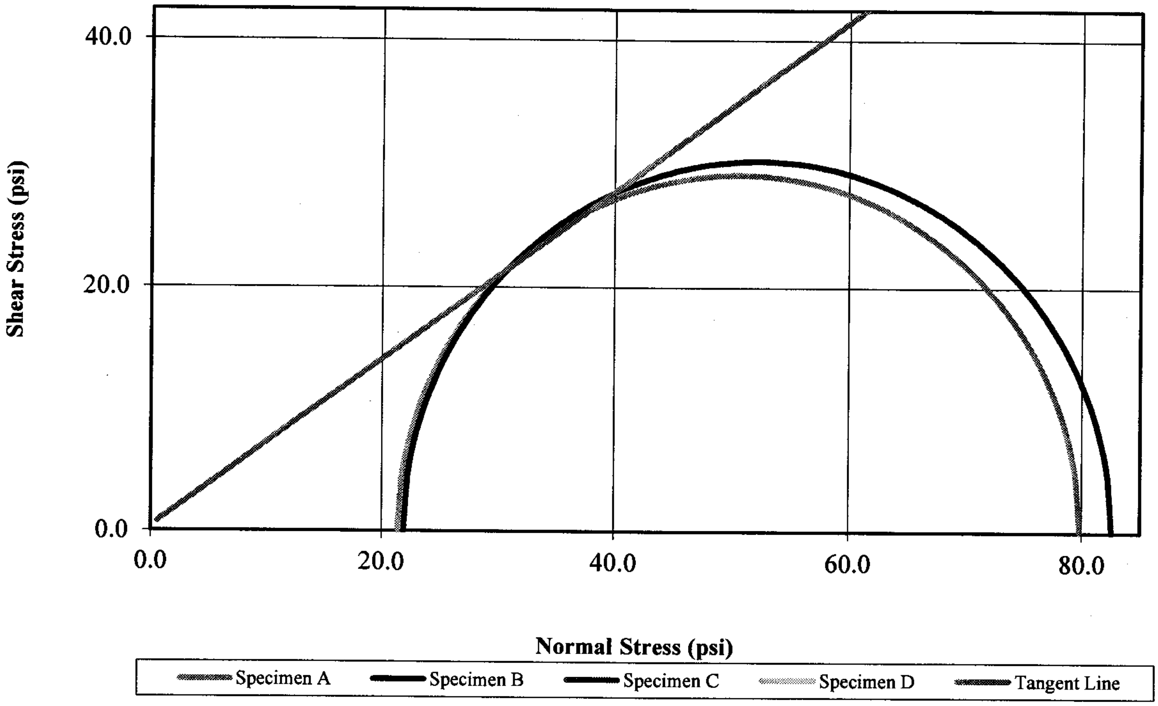
(C = 2.6 Ø = 24.5)



Mohr Stress Circles at Maximum Principal Stress Ratio Criterion

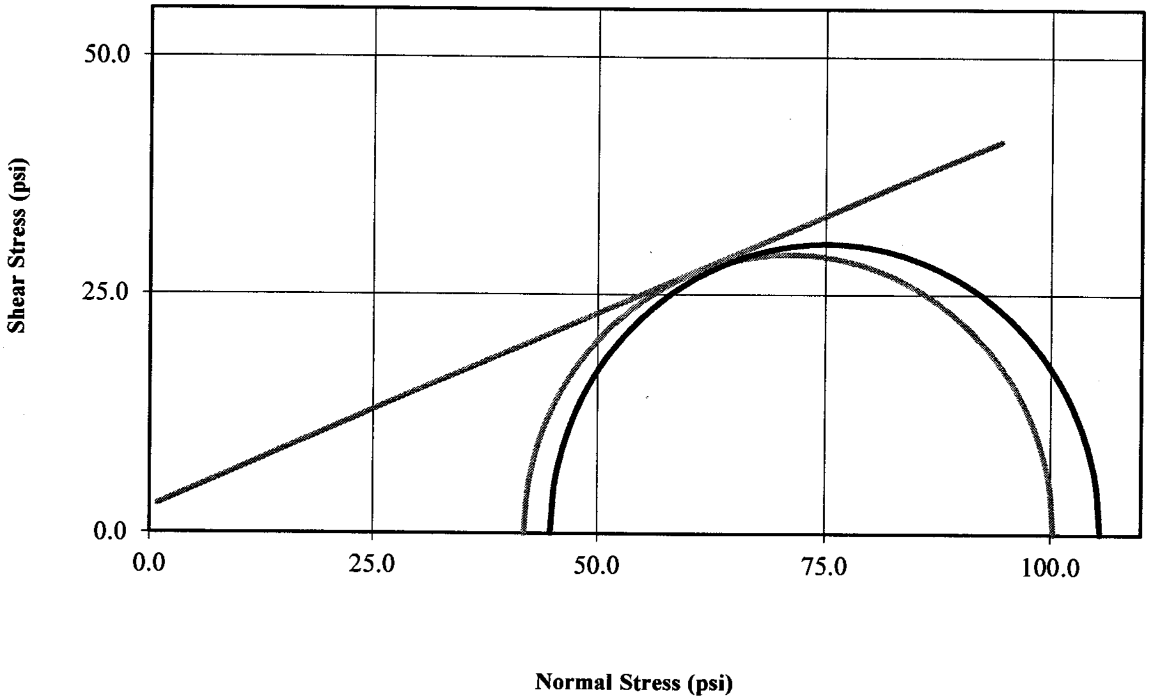
Effective Stress

(C' = 0.4 Ø' = 34.4)



Total Stress

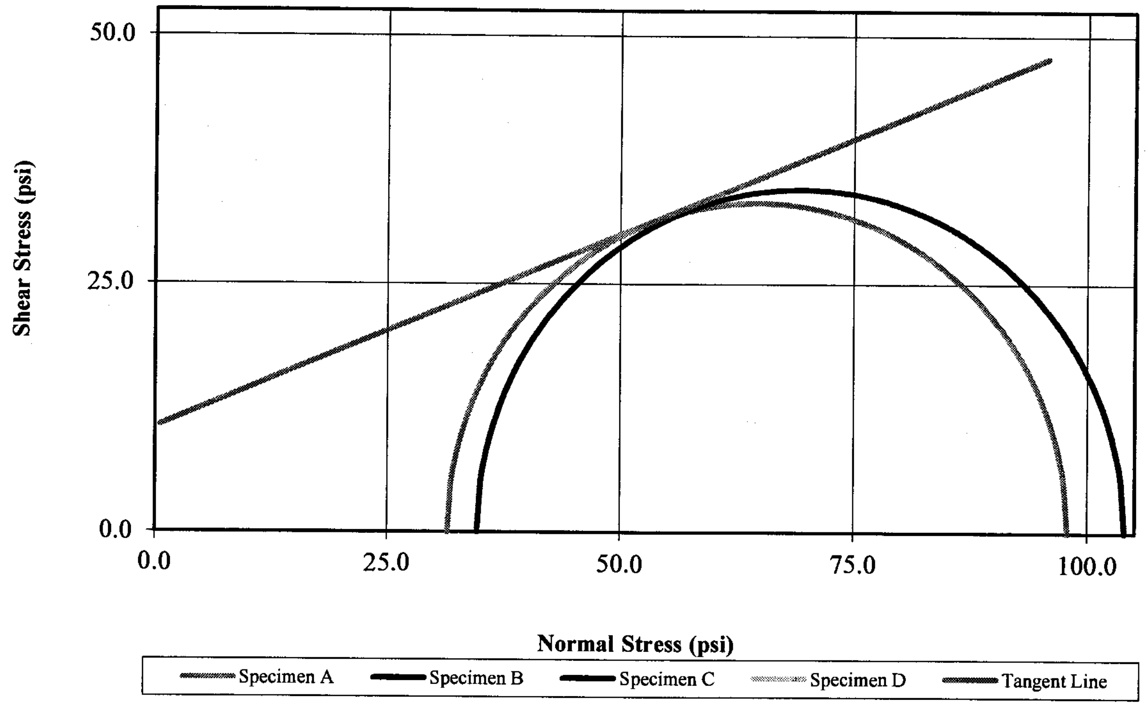
(C = 2.6 Ø = 22.2)



Mohr Stress Circles at 15% Axial Strain Criterion

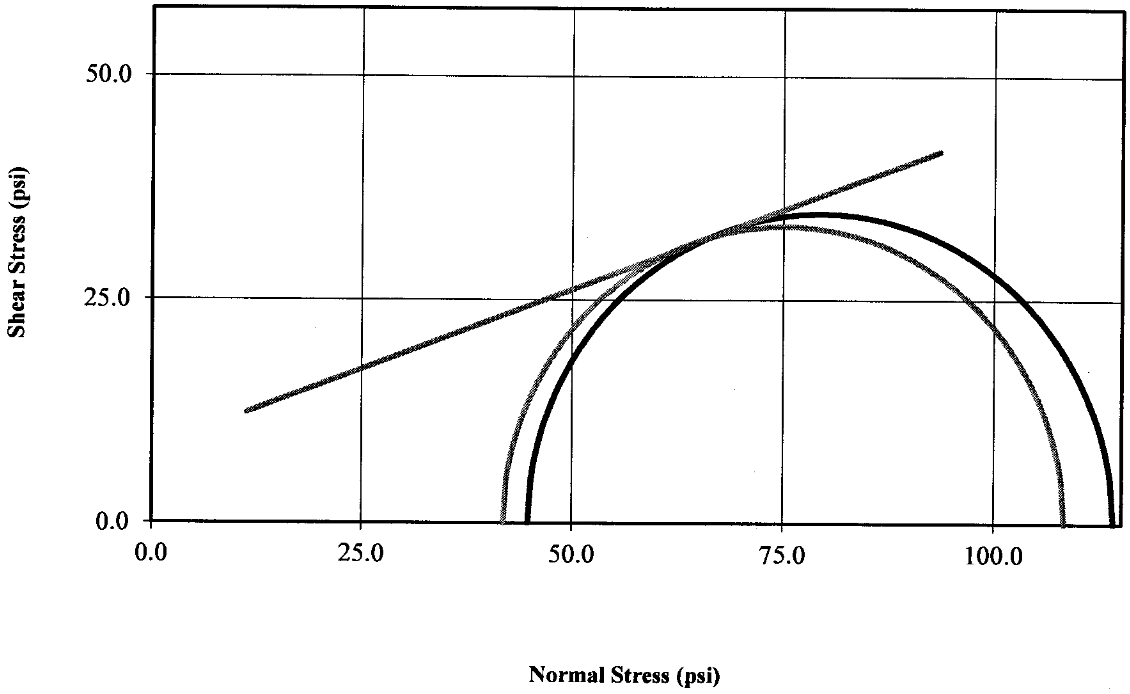
Effective Stress

(C' = 10.5 Ø' = 21.3)

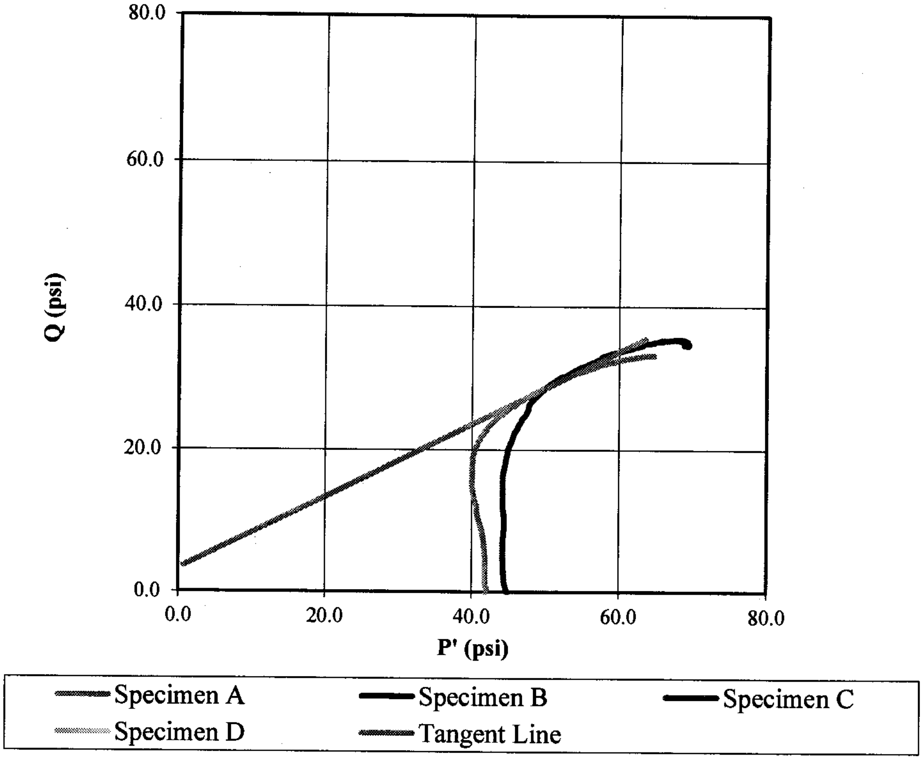


Total Stress

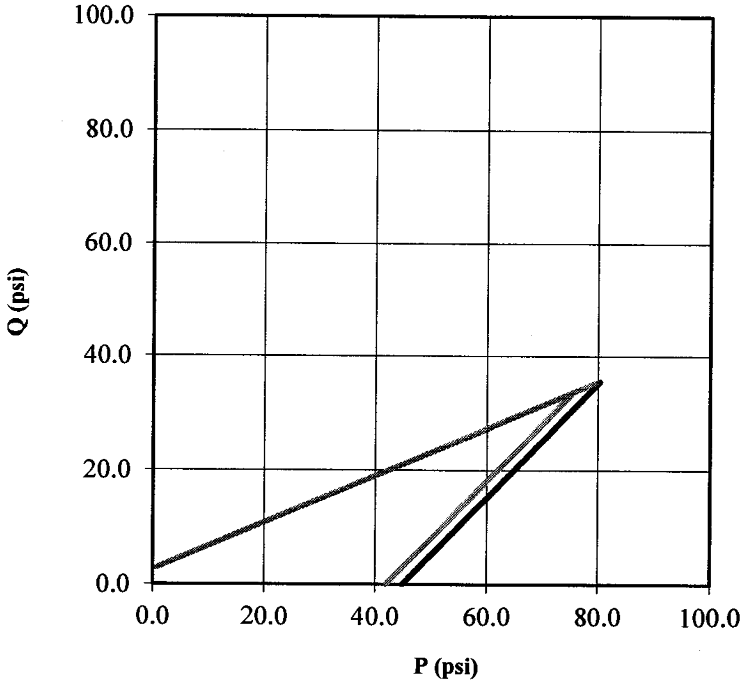
(C = 8.4 Ø = 19.6)



Stress Paths (Effective)
(C' = 3.3 Ø' = 26.9)



Stress Paths (Total)
(C' = 2.6 Ø' = 22.3)



GeoTest, Inc.

Consolidated Undrained Triaxial Test (ASTM D4767)

GeoTest

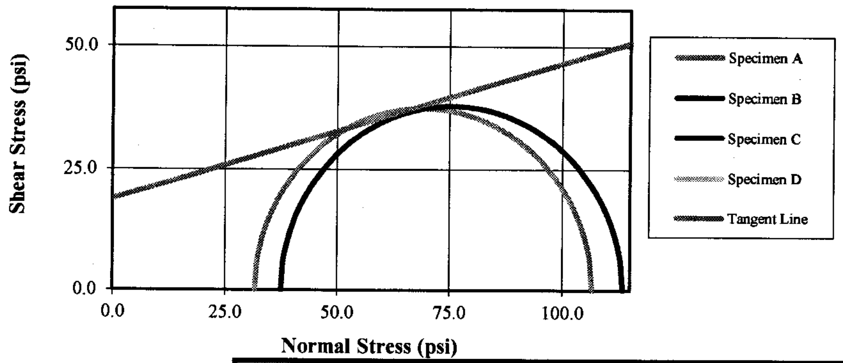
Date:

Checked By:

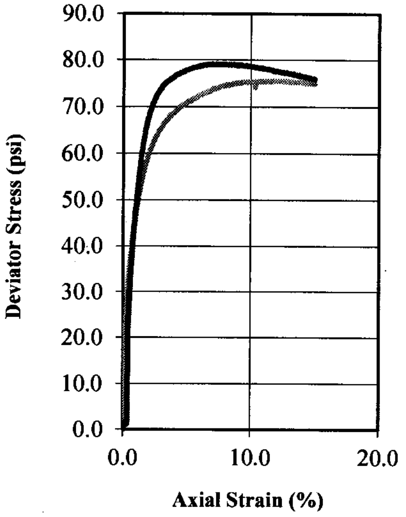
Date:

Tested By:

Mohr Stress Circles at 15% Axial Strain Criterion



Deviator Stress Vs. Axial Strain

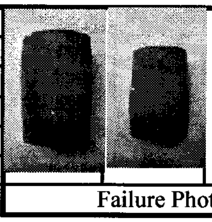


	Specimen			
Initial	**A**	**B**	**C**	**D**
Water Content (%)	12.3	14.2		
Dry Density (pcf)	125.4	121.6		
Saturation (%)	101.87	104.73		
Void Ratio	0.317	0.358		
Diameter (in)	2.866	2.863		
Height (in)	5.595	5.589		
Specific Gravity	2.65	2.65		
Liquid Limit	0	0		
Plastic Limit	0	0		
After Consolidation	**A**	**B**	**C**	**D**
B-Value	1.00	1.00		
Water Content (%)	15.6	17.1		
Dry Density (pcf)	132.86	128.12		
Saturation (%)	100.00	100.00		
Void Ratio	0.245	0.291		
Effective Stress (psi)	40.8	43.2		
Back Press. (psi)	79.2	79.3		
Rate of Strain	0.00521	0.00536		

15% Axial Strain Criterion	
C (psi)	10.4
C' (psi)	19.0
Ø (deg)	20.5
Ø' (deg)	15.4

After Shear	A	B	C	D
σ'1 at Failure (psi)	106.55	113.29		
σ'3 at Failure (psi)	31.60	37.40		

Project:	Upper Lake Bluff Park
Location:	
Project Number:	3884
Boring Number:	B-2
Sample Number:	ST-22
Depth:	78.6-80.6'
Sample Type:	Undisturbed
Description:	Gray brown silty CLAY
Test Type	Consolidated Undrained
Remarks	



N/A N/A

Failure Photographs

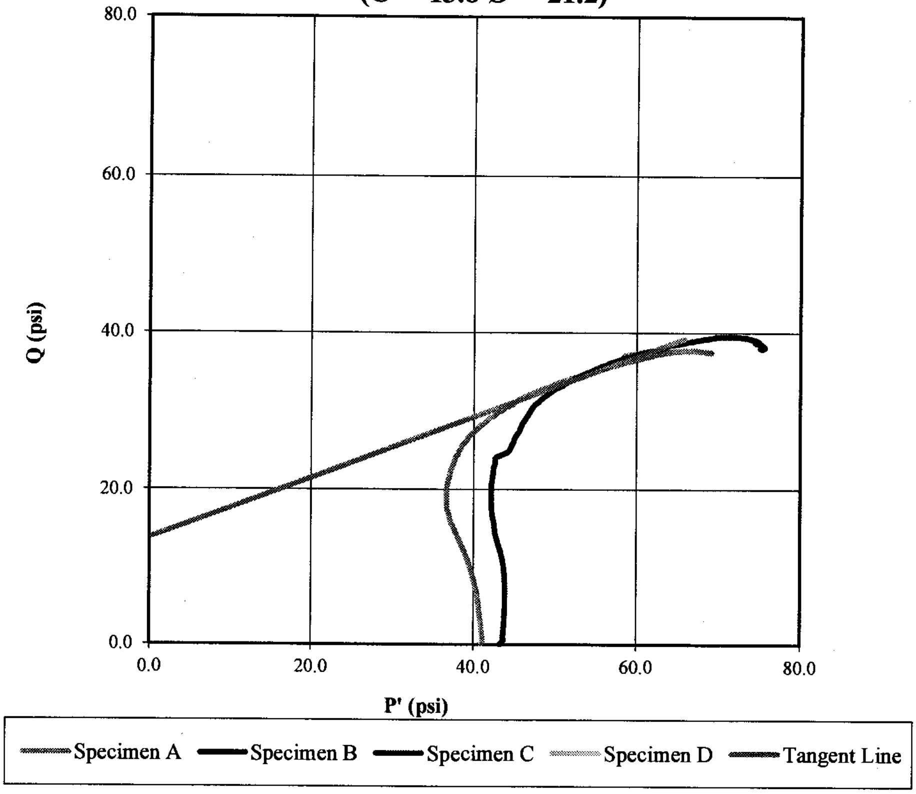
Date:

Checked By:

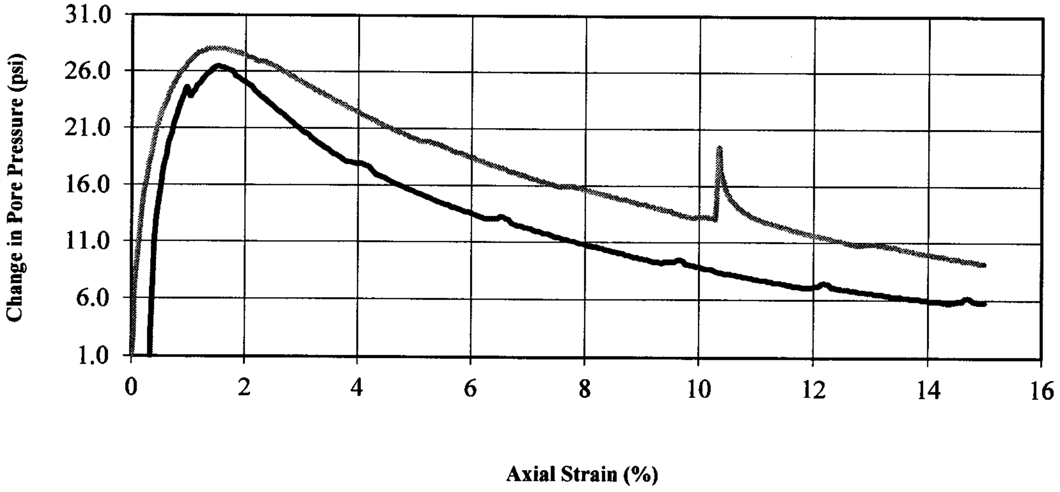
Date:

Tested By:

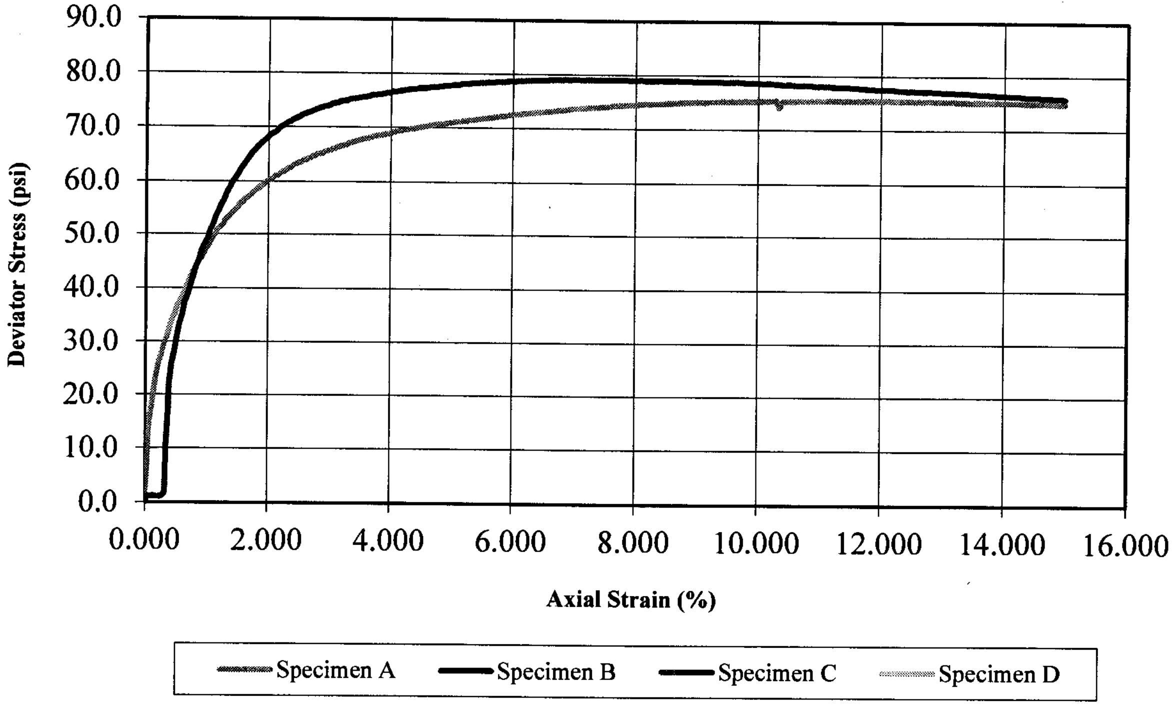
Stress Paths (Effective)
(C' = 13.8 Ø' = 21.2)



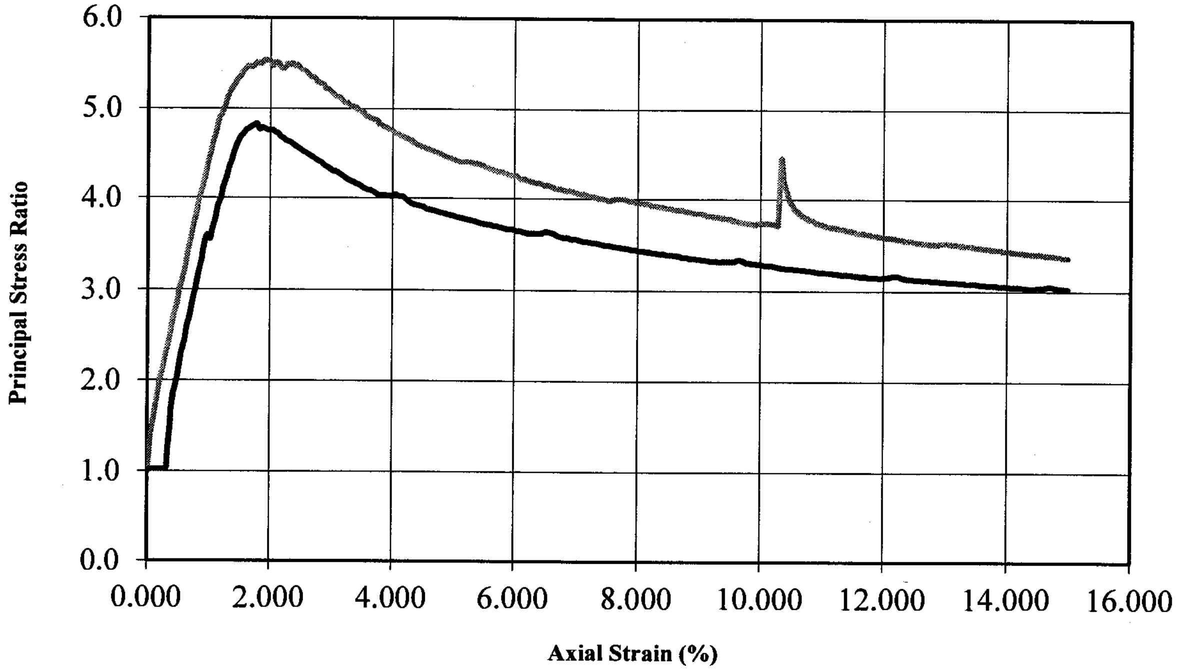
Change in Pore Pressure vs. Axial Strain



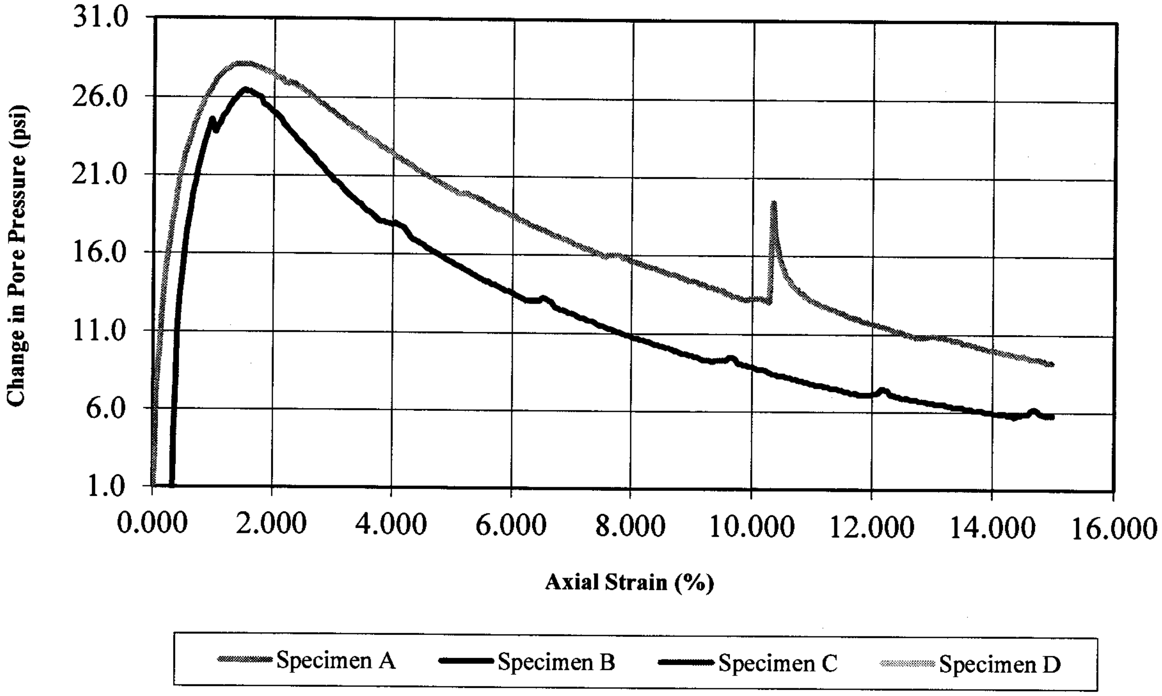
Deviator Stress vs. Axial Strain



Principal Stress Ratio vs. Axial Strain



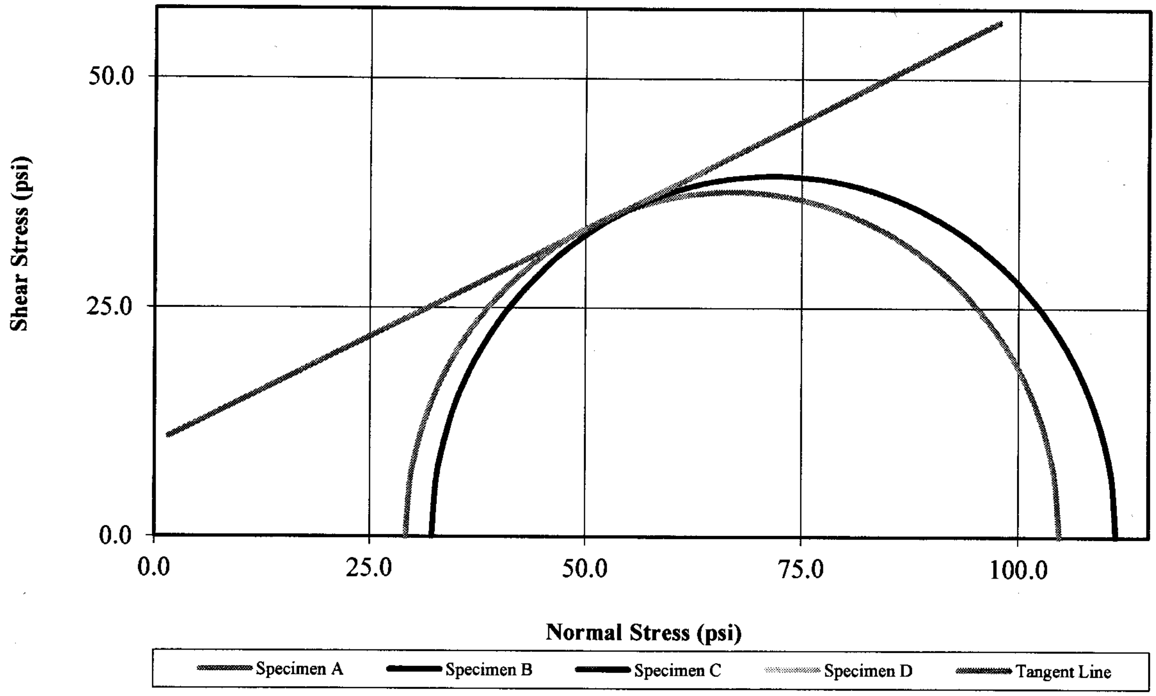
Change in Pore Pressure vs. Axial Strain



Mohr Stress Circles at Maximum Deviator Stress Criterion

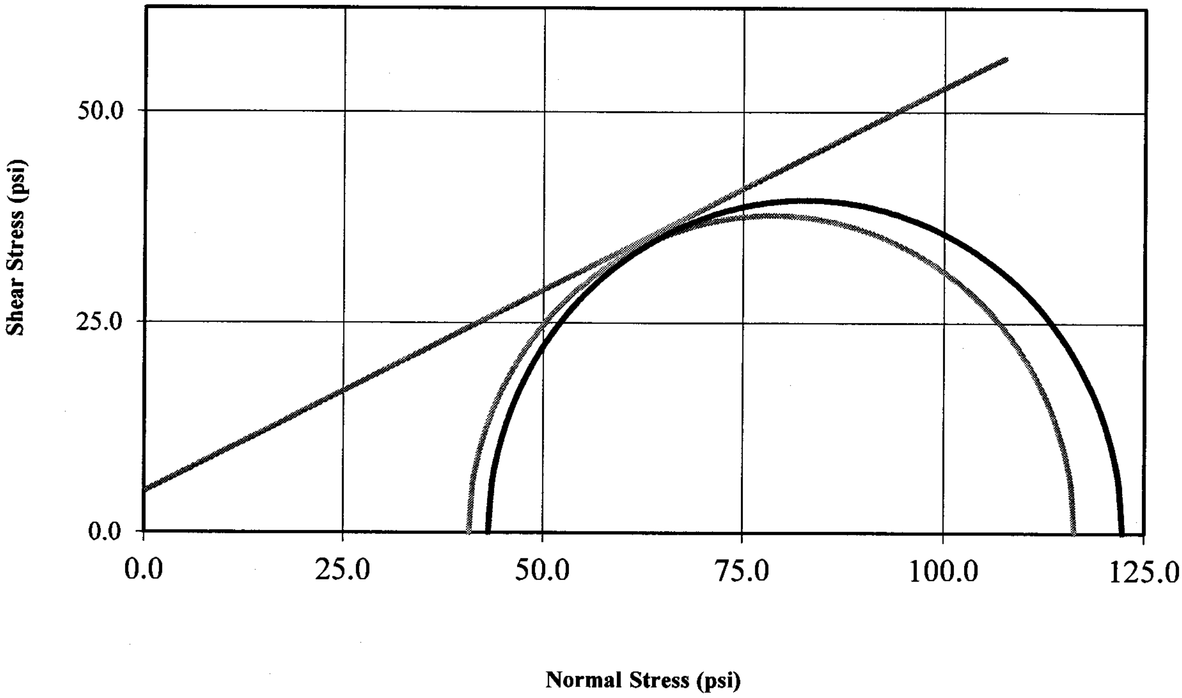
Effective Stress

(C' = 10.1 Ø' = 25.2)



Total Stress

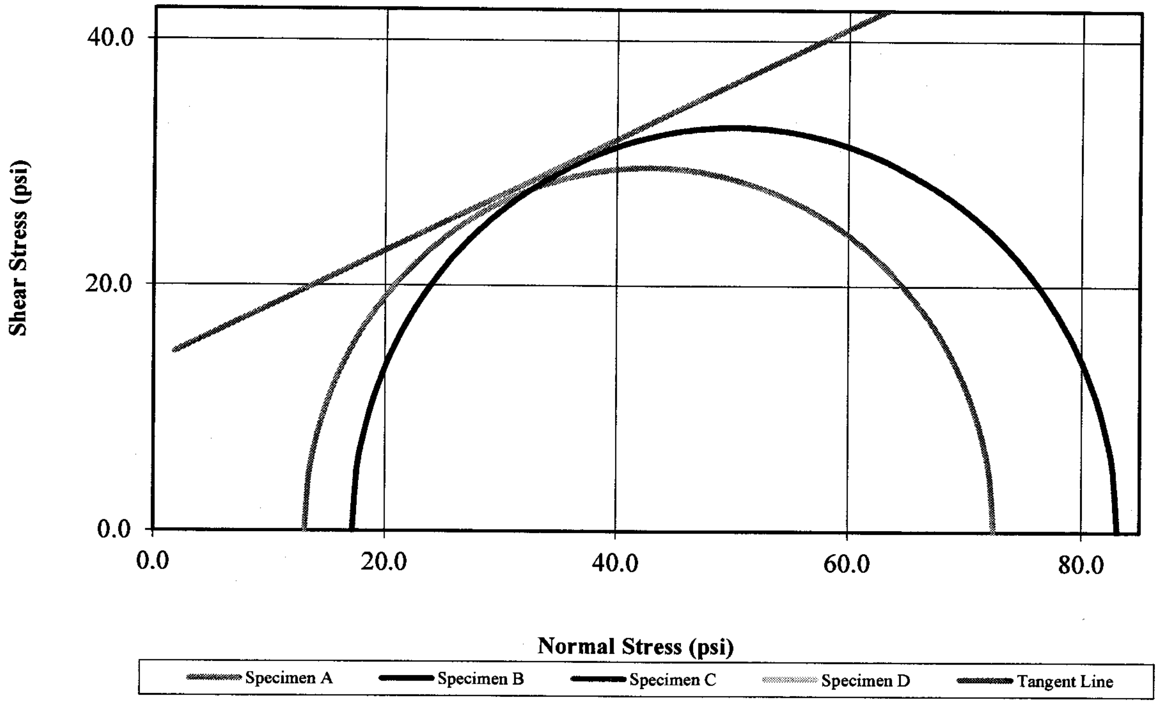
(C = 4.9 Ø = 25.6)



Mohr Stress Circles at Maximum Principal Stress Ratio Criterion

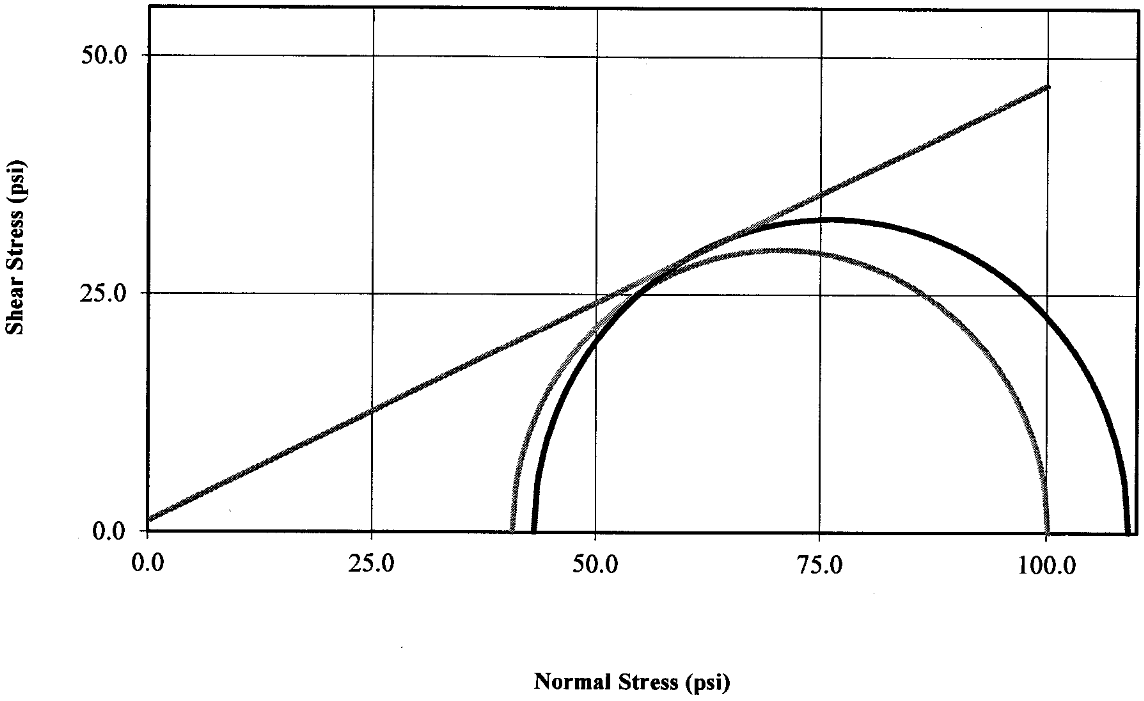
Effective Stress

(C' = 13.7 Ø' = 24.5)



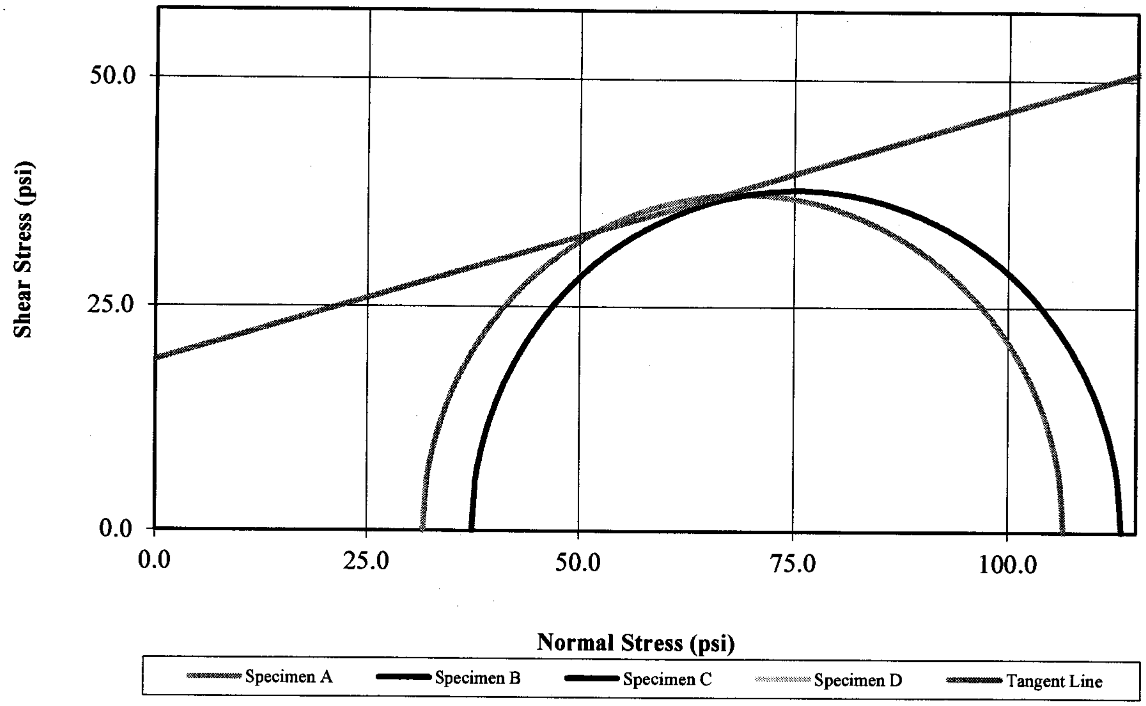
Total Stress

(C = 1.1 Ø = 24.7)

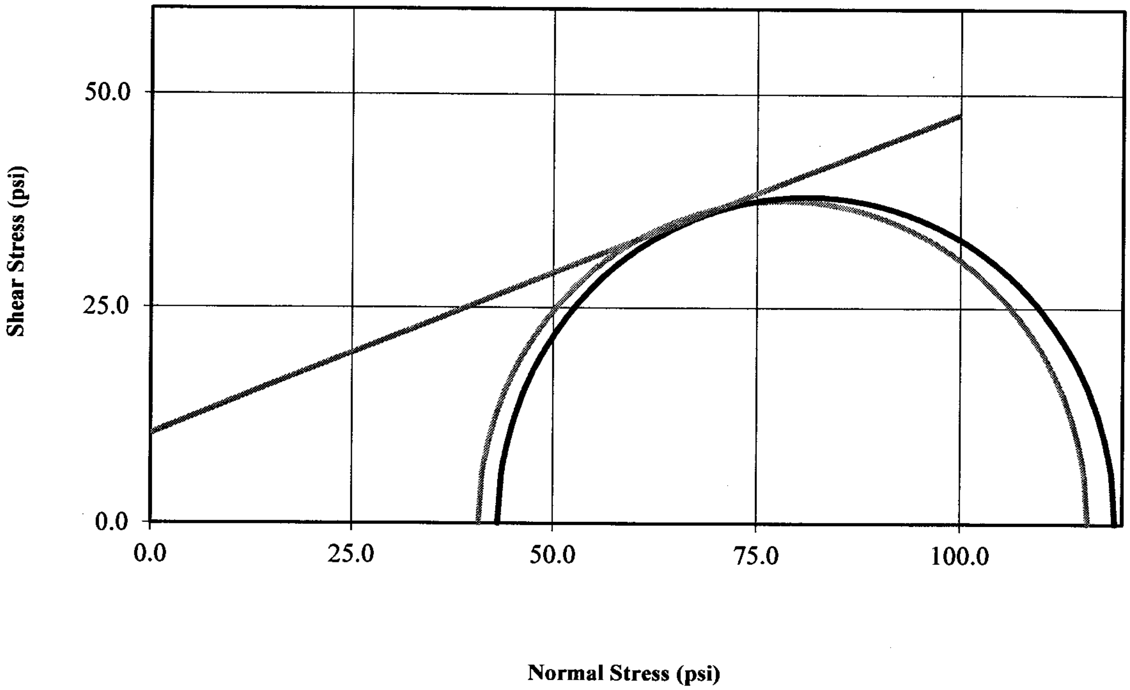


Mohr Stress Circles at 15% Axial Strain Criterion

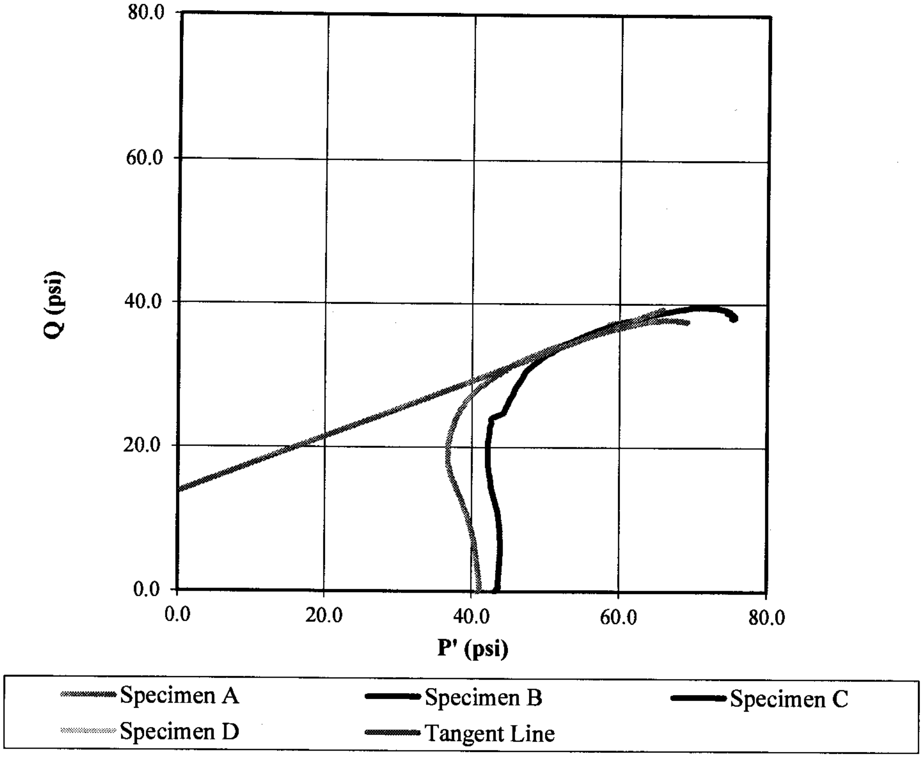
Effective Stress
(C' = 19.0 Ø' = 15.4)



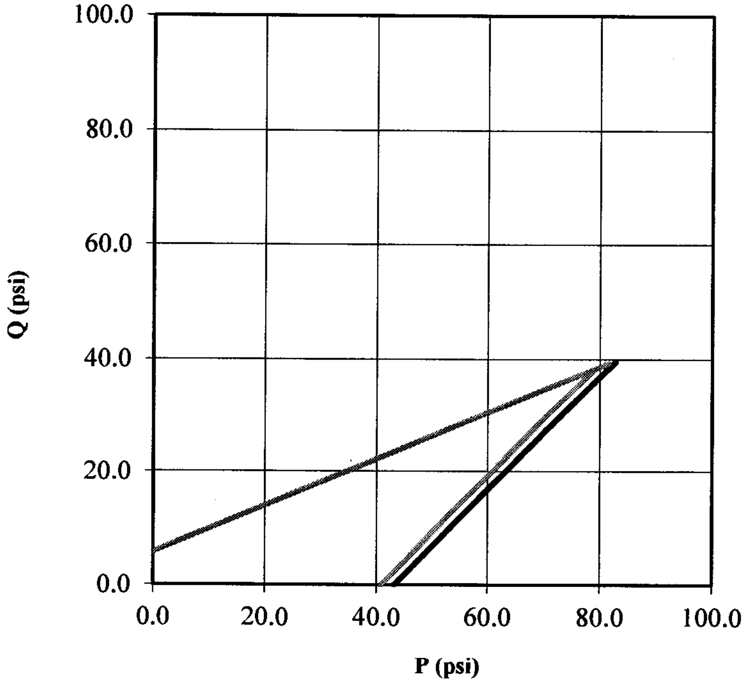
Total Stress
(C = 10.4 Ø = 20.5)



Stress Paths (Effective)
(C' = 13.8 Ø' = 21.2)



Stress Paths (Total)
(C' = 5.7 Ø' = 22.3)



WISCONSIN TESTING LABORATORIES

FIELD EXPLORATION STANDARD SAMPLING PROCEDURES

Soil sampling was performed in general accordance with ASTM method D-1586. Using this method, a 140 lb. weight (hammer) free-falling a distance of 30 in. is used to drive a 2 in. O.D. by 1-3/8 in. I.D. split-barrel sampler into the soil. The sampler is first driven 6 in. into the soil for seating purposes. The sampler is then driven an additional 12 in., and the number of blows required to drive the sampler the final 12 in. is known as the penetration resistance or "N" value. The number of hammer blows used in making the test is reported on the drill logs for all three 6 in. increments of penetration (example: 7/8/9 where 8 + 9 = 17 is the standard penetration resistance or "N" value). "N" values are used to indicate relative densities of cohesionless (sand and gravel soils) and to a lesser degree the consistencies of cohesive soils.

All soil samples recovered from the test borings were preliminarily classified in the field by the drill crew. Representative portions of the samples were enclosed in glass jars, labeled and returned to the laboratory for further examination and final classification by a geotechnical engineer.

Please note that the boring logs show the subsurface conditions at the dates, locations and depths indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times, and to greater depths than penetrated by the borings. It should also be noted that water level determinations made in clean, cohesionless soil are generally quite reliable, whereas water level determinations made in cohesive soils may not indicate true static water levels even after several days or weeks observation.

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Field Classification System for Soil Exploration

Non Cohesive Soils
(Silt, Sand, Gravel and Combinations)

Relative Density	Blows Per Ft.
Very Loose	5 or less
Loose	6 to 10
Firm	11 to 15
Medium Dense	16 to 30.
Dense	31 to 50
Very Dense	51 or more

Relative Proportions

Descriptive Term	Percent
Trace	1 to 10
Little	11 to 20
Some	21 to 35
And	36 to 50

Cohesive Soils
(Clay, Silt and Combinations)

Consistency	Blows Per Foot
Very Soft	3 or less
Soft	4 to 5
Medium Stiff	6 to 10
Stiff	11 to 15
Very Stiff	16 to 30
Hard	31 or more

Laboratory Test Symbols

M_c: Natural Moisture Content

D_d: Natural Dry Density

O_c: Organic Content

Q_u: RIMAC Unconfined Compressive Strength

Q_p: Calibrated Penetrometer

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- Large 1 to 3 inch
	- Medium	- 1/2 to 1 inch
	- Fine	- 4.76mm to 1/2 inch
Sand	- Coarse	- 2.0mm to 4.76mm (dia. of pencil lead)
	- Medium	- 0.42mm to 2.0mm (dia. of broom straw)
	- Fine	- 0.074mm to 0.42mm (dia. of human hair)
Silt		- 0.002mm to 0.074mm (Cannot see particles)

Plasticity

Degree of Plasticity	Plasticity Index
None to Slight	0 to 4
Slight	5 to 7
Medium	8 to 22
High to Very High	Over 22

Classification on logs are made by visual inspection in the absence of classification tests.

Standard Penetration Test - A 2.0 in. O.D. by 1-3/8 in. I.D. sampler (split-spoon) is driven a distance of 1.5 ft. with a 140 lb. hammer free falling a distance of 30.0 in. The number of hammer blows required for each 6.0 in. of penetration are recorded on the boring log (Example - 6/8/9). The Standard Penetration Resistance (N value) can be obtained by adding the last two figures (i.e. N = 8+9 = 17).

Strata Changes - In the column "Material Classification" on the boring log, the horizontal lines represent strata changes. A solid line (——) represents an actually observed change, a dashed line (------) represents an estimated change.

Groundwater Observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause the water levels to vary from those indicated on the logs.

Interbedded Strata Descriptions

Very thin seams	- Paper thin to 1/8 in. thick
Thin seams	- 1/8 in. to 1 in. thick
Medium seams	- 1 in. to 6 in. thick
Large seams	- 6 in. to 12 in. thick