Mueser Rutledge
Consulting Engineers
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August 8, 2013

Beatty Development Group
1000 Wills Street
Baltimore, MD 21231

Attention: Mr. Jonathan Flesher

Re: Engineering Evaluation Report
Harbor Point Development (Exelon Tower)
Baltimore, Maryland
MRCE File No. 11896A-40

Gentlemen:

Mueser Rutledge Consulting Engineers (MRCE) provides this Engineering Evaluation document summarizing analysis of planned development construction for protection of the corrective measures. The analyses and evaluations are presented in the attached memoranda which summarize detailed assumptions, calculations, and findings. Analysis subjects and findings are summarized below:

1. Estimated Settlement Under Development Fill

Fill is proposed for street areas to raise grades. Utilities will be buried in the fill. Pre-loading was performed before MMC construction in some areas to allow development fill.

Where planned grades are below the pre-load elevation, and OCR is greater than about 1.05, fill settlement results only from recompression, and long term secondary compression (3.8" in sixty-five years). Settlement magnitude can be tolerated by the MMC and does not result in negative slope at the geomembrane. The location where street areas should be supported on piles was determined by this rule.

The former timber frame bulkhead structure was abandoned below Dock St. The bulkhead was preloaded, but its existing condition and longevity is not known. Soil below the pile supported structure is compressible, and would result in unacceptable settlement if the bulkhead structure degrades with time and overburden loads are transferred from the bulkhead to the underyling compressible soil. As described in EE Memo #9, a new pile-supported platform will be placed above the abandoned bulkhead to support the MMC, HMS and development infrastructure.

2. Storm Water Storage Demand

After the MMC geomembrane layer is removed, storm water collected in excavations must be managed to prevent water which contacts soil below the geomembrane from rising to the capillary break. The water will be collected and stored for testing to determine disposal criteria. The volume of water collected relies on the area open at any one time. Two tanks are needed to permit storm water testing and disposal (day 1 water) simultaneous with storm water collection (day 2). A construction scenario having the large shear wall foundation fully open and 60 pile caps open found one 4 feet deep 75 feet x 75 feet temporary tank will provide storage for 24 hours of a 100 year storm event.

Pumping rates were established for the maximum intensity period within the 100 year storm. Pumping rates are reasonable and can be managed with standard construction equipment. Pumping rates and storage quantity required can be managed by reducing the number of open areas at one time, and by covering open areas to prevent storm water contact with exposed subgrades.

3. Flow in Drainage Net from Development Area

MMC drainage requires revision in order to accommodate development and to provide the pile support improvement to the MMC and HMS systems on Dock St. in the development area. Development revisions consider:

- The risk of infiltration to the HMS pumps is greatly reduced because development roof and street drainage will remove direct storm water from 87.5% of the development area.
- Only 14.7% of the drainage net area is obstructed by pile cap construction.
- Drainage net flow from 90% of the drainage net area will pass through sampling points SSP4 or SSP4A (new) so that the drainage net water may continue to be used to evaluate the MMC performance after development foundations are in place.

4. Hydraulic Conductivity of Sheet Pile Barrier

Sealed interlock steel sheet piles are proposed to allow pile driving in close proximity to the barrier. Sheet pile installation should remove any existing arching stresses within the backfill. Calculations demonstrate that an interlocking sheet pile barrier performs as well as the existing soil-bentonite backfill if the soil-bentonite was to fail to perform due to arching or long-term chemical degradation.

5. Spill Control Volume of New Loading Dock

HMS groundwater is removed in 5,000 gal tank trucks. A new interior loading dock will be constructed as secondary storage to contain 6,000 gal. The loading dock and collection/discharge sump will be made of structural concrete supported on pile foundations.

6. Plaza Garage Slab over Multimedia Cap

   A slab-on-grade parking floor will replace the existing MMC cover soil. The concrete will mechanically protect the synthetic layers from tow truck and car parking. A 1 inch thickness of styrofoam is sufficient to provide thermal insulation of the MMC synthetic layers equal to the existing soil cover. The 5” thick concrete slab on grade was evaluated to adequately support a tow truck with car in tow within the allowable bearing pressure at the geomembrane. Larger trucks and heavy construction equipment will be excluded from garage use by the limited 7 ft headroom below the Central Plaza deck above. The slab on grade will be reinforced with #3 bars at 10 in spacing so that wheel loads will be distributed, even with concrete cracking.

7. Protection of Multimedia Cap from Construction Vehicle Loading

   This analysis evaluated loads from construction vehicles and equipment/concrete supply trucks. A dynamic load was added to the static load. HS-20 and 12 cy concrete truck loading distributed through the 30 inch soil cover imposes bearing stresses below 2,000 lb/sf at the synthetic layers. The cover soil provides a stable environment at the synthetic layers by virtue of high bearing capacity safety factor. Material storage containers and 16,000 gal water storage containers impose a low bearing stress. Rutting should be repaired to maintain the existing 30 inches of cover soil. Paving is recommended at primary vehicle pathways and where material containers will be repeatedly loaded onto truck carriages to protect against rutting and reduce dust. Large construction equipment such as the pile driver crawler cranes will require mats to spread concentrated loads. The tower cranes will be independently pile supported.

8. Environmental Assessment (by ERM)

   Details are provided in Appendix A.

9. Pile-Supported MMC & HMS above Dock Street Bulkhead

   The multimedia cap (MMC) and replacement head maintenance system (HMS) is supported by an interconnected structural system consisting of a pile supported concrete mat. The purpose of the structure is to prevent future settlement caused by the proposed roadway loading and raised grades along Dock Street. The MMC and HMS are supported on this structural system.

10. Protection Of HMS Systems For Continuous Operation During Construction (No Memorandum Attached)

    The office wing and truck loading dock of the Honeywell Transfer Station will be demolished and rebuilt within the footprint of the future Trading Floor Garage. The groundwater storage tanks and their containment, and the maintenance area will remain in place for future use. Piles supporting the development structures will be driven in close proximity to the tanks and maintenance areas, which are to remain operational throughout construction period. Also, construction of the Dock St. platform which provides pile support for the HMS vaults and conveyance lines (V11, V12, and MJ1) requires pile driving in close proximity to these HMS components.

The Tank pad is a heavily reinforced mat with integral concrete walls which can tolerate minor ground movement and vibrations. The primary components of the Transfer Station maintenance area include power supply and compressed air supply to the perimeter vaults, and support data systems recording and monitoring HMS performance. Utilities are largely above grade and supported on the structure. Vibration and crack width monitoring will be performed, and damage sustained will be repaired after pile driving is complete. These components are flexible, and contract drawings require protection during demolition and construction. The data computer systems will be relocated to temporary office space adjacent to the site. Temporary groundwater storage tanks will be provided and the primary tanks will be emptied during adjacent pile driving activity.

The vaults and conveyance lines within the Dock St. and Wills St. development area are below the multimedia cap. Surveys and test pits will be performed to locate the conveyance lines to prevent direct pile contact damage. The vaults are robust concrete structures bearing on timber frames of the former bulkhead structures and the conveyance lines are buried in fill above these timber structures so that these components should undergo little settlement as a result of pile driving. The conveyance lines contain pressurized fluids in flexible pipes, power, and data cables. These pipes and power cables are housed within oversized conduits. The conduits will isolate the active components from ground vibration. Monitoring of system performance will be performed during construction, and damage will be repaired to maintain operation throughout and after construction.

The contingency plan for the Head Maintenance System and Transfer Station identifies the mechanical, plumbing, and data components and their performance mechanics, and provides requirements for monitoring and repair during the construction period. The Contingency Plan provides required details of the components and strong monitoring and maintenance performance criteria, and is an acceptable means for management of these systems during construction.

We trust that the analyses will document allowable construction conditions questions regarding the proposed development on the corrective measures. Please do not hesitate to contact us with any questions.

Very truly yours,

MUESER RUTLEDGE CONSULTING ENGINEERS

By: ______________________________
Peter W. Deming, P.E.

AMD\PWD\11896A-40\Engineering Evaluation Summary Letter
Attachments
cc: Michael L. Ricketts (BDG)
Chris French (Honeywell)
Ken Biles (CH2M Hill)
Jeff Boggs (ERM)

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MEMORANDUM

Date: August 8, 2013
To: Office
From: Alexandra Patrone and Adam M. Dyer
Re: EE Memo 1 – Estimated Settlement Under Development Fill
Exelon Building & Plaza Garage, Baltimore, MD
File: 11896A

MRCE has reviewed available information for the Exelon Building and Plaza Garage and has estimated settlement resulting from fill placed for development. The purpose of these estimates is to determine if the proposed grading scheme will cause settlement which may influence the integrity of the multi-media cap (MMC) and Head Maintenance System (HMS) components.

Exhibits

Figure 1 Key Plan
Figure 2 Historic Filling Grading and Surcharging of Dock Street
Figure 3 Results of Analysis
Figure 4 Geomembrane Slope Analysis

Appendix A Settlement Calculations
Appendix B Assessment of Compressibility Characteristics
Appendix C Geologic Sections

References

1. "Corrective Measures Implementation Construction Completion Report, Phase I: Soil-Bentonite Hydraulic Barrier Wall, Phase II: Final Remedial Construction" prepared by Black and Veatch, Volumes I and II, February 2000.
2. "An Engineering Manual for Settlement Studies" by J.M. Duncan and A.L. Buchignani, June 1976, revised October 1987.

Site Description

The proposed development includes a high-rise tower, a multi-use plaza, parking garage, roadways and streetscapes. The development is situated in Area 1 of the Honeywell (formerly Allied Signal Site) and is bounded by Dock, Block Street (future), Point Street (future), and Wills Street. Generally, the existing ground surface for the proposed development slopes gently to the north, existing ground surface varies from Elev. +9 to +14. The proposed development includes raised grades for roadways and streetscapes from approximately Elev. +13 to Elev. +27.

Subsurface Conditions

Subsurface conditions consist of a layer of fill underlain by a compressible organic clay layer ranging in thickness from 4 to 20 ft. This compressible layer is generally described as a soft brown to black organic silty clay with trace vegetation and fine sand, and is typically given a USCS designation of OH or OL. This clay layer is underlain by a series of sand and silt layers. Bedrock is at approximately Elev. -80. Groundwater is managed at low tide approximately Elev. 0 to Elev. +1.

A buried timber bulkhead structure is present below the MMC, and immediately abuts the existing soil-bentonite barrier. The bulkhead consists of either a timber or granite block headwall supported by piles terminating in the underlying sand or silt strata with unknown tip elevation. A series of timber deadmen and support framing are also part of the bulkhead structure. The timber structural elements were constructed at low water to prevent decay. They are between Elev. -1 and Elev. +1, and are buried in soil.

Historic Earthwork

As part of the corrective measures during the 1990s Honeywell pre-loaded the site in areas of potentially high settlement, see Figure 1. A schematic of historic earthwork operations in the vicinity of Dock Street west of Wills Street is shown on Figure 2. These operations included:

Prior to 1988:

Back Basin north of Dock Street consisted of a bulkhead adjacent to open water.

Back Basin Surcharge c. 1991:

To make way for the construction of the Soil-Bentonite barrier, the back basin was filled in and pre-loaded to an elevation that sloped from the west end at Elev. +19 feet to the east end at +14 feet.

Transfer Station Surcharge c. 1996:

To make way for the Transfer Station and Multimedia Cap (MMC), Dock Street and the area of the Transfer Station were pre-loaded to between Elev. +20 to + 24 feet.

S-B Barrier Construction c. 1999:

The S-B Barrier trench was excavated in close proximity to the north side of the buried bulkhead structure.

MMC Construction c. 1999:

After completion of the S-B Barrier, the MMC was constructed including soil cover to the present grade.

In general, pre-loading included installation of vertical wick drains to shorten the drainage path, and it is assumed that the preloading successfully consolidated the clay to the surcharge load in all of the surcharge schemes.

This historic surcharging is significant to the current settlement analysis when determining whether the compressible clay will be in a recompression or virgin compression loading condition as a result of fill placement to achieve the proposed grades. If the proposed new grade is above that of the historic pre-load, a significant magnitude of settlement can be expected due to virgin compression of the underlying material. If the proposed new grades are below the historic pre-load only recompression settlement will occur.

Assessment of Settlement Potential

An overlay of proposed grades, existing conditions, historical conditions, and buried structures was examined to analyze areas of settlement concern. Four areas were identified to potentially impact the corrective measures; areal extents can be seen on Figure No. 1.

These areas include:

1. Wills Street roadway grading, analyses include:
    a. Recompression only, all pre-loaded (adjacent to Vault 1);
    b. Virgin compression, partially pre-loaded (near Vault 2);
    c. Location of division between recompression and virgin compression;
2. Exelon Tower moment slab excavation, analysis includes:
    a. Fluid weight of concrete prior to load transfer to driven piles, t = 1 day;
3. Point Street roadway grading, analysis includes:
    a. Virgin compression, not pre-loaded;
4. Dock Street overlying buried bulkhead structure, analysis includes:
    a. Existing grade with a deteriorated bulkhead, portions recompression, virgin compression;
    b. Proposed grade with a deteriorated bulkhead, virgin compression;

Compressibility Characteristics

Previous laboratory testing indicates a strong correlation between natural water content and compression ratio, swell index, and initial void ratio, see Attachment B. To assess the compressibility characteristics of Stratum O, natural water content of borings within the vicinity of each Area was investigated. The data for Areas 1, 2, and 3 indicates a good correlation for increase of water content with depth. The data for Area 4 did not provide a good correlation and included significant scatter. This is reasonably attributable to the presence of the buried bulkhead structure that helps to attract load locally. For Area 4, average water content was used and settlement was estimated $\pm 1\sigma$. Elastic moduli of granular strata were estimated based on the EPRI *Manual on Estimating Soil Properties for Foundation Design*.

Analysis and Assumptions

In general, settlement is computed as the sum of three contributors. These include elastic compression, consolidation, and secondary compression. For this analysis, in areas where re-compression only is anticipated, it is assumed that secondary compression is negligible. In areas where virgin compression is anticipated, elastic compression and secondary compression are negligible with respect to engineering improvements necessary to alleviate settlement concerns. It was assumed that strata below the hard silty clay of Stratum M were incompressible under the potential loadings.

Sample hand calculations and Excel calculation sheets are attached as Appendix A.

Elastic Compression

Elastic compression of granular fill strata was modeled as a one-dimensional loading on medium dense granular strata. A typical calculation of elastic compression is included in Appendix B, Area 1, Analysis a. In general, elastic compression of approximately 0 to ¾ inch can be expected.

<u>Consolidation</u>

Consolidation settlement compressible strata estimates were developed using one-dimensional consolidation theory after Terzaghi (1947). Idealized profiles were determined for analysis based on the geologic sections presented in Appendix C. The compressible stratum was divided into sub-layers no greater than four feet in thickness. The ground water table was assumed to be at El. 0. A construction sequence was identified for each analysis, and settlement was calculated for the loading conditions during each phase of the construction sequence. In areas where a historic preload was present, the maximum past pressure was calculated based on this preload. In locations where a preload was not present, the maximum past pressure was computed assuming existing conditions. Primary settlement was determined for each phase of the construction sequence in each sub-layer, and a total primary settlement estimate at each section was determined.

Area 1: Wills Street Roadway Grading (Section 1-1)

Settlement will result from raising grades to accommodate the proposed grading scheme. Portions of this area will be in re-compression and transition to virgin compression based on the pre-loaded to Elev. +20. Three analyses were performed to assess re-compression settlement adjacent to Vault 1, virgin compression near Vault 2 and the threshold elevation where virgin compression is risked. This threshold was defined as the location at which the maximum past pressure is 5% greater than the existing overburden pressure (i.e. OCR = 1.05). The results are:

- Adjacent to Vault 1, the added fill height of 5 feet from Elev. +14 to Elev. +19 does not exceed the pre-load at Elev. +20 and results in approximately 0.2 inches of consolidation settlement;
- Near Vault 2, the added fill height of 12 feet from Elev. +14 to + 26 exceeds the pre-load at Elev. +20 and results in approximately 3.9 inches of consolidation settlement;
- For the pre-load at Elev. +20, depth and thickness of Stratum O in the vicinity, it was determined that fill below Elev. +18.5 will result in an OCR > 1.05.

Area 2: Exelon Tower Moment Slab Excavation (Section 2-2)

The construction sequence in Area 2 consists of excavation from existing grade at Elev. +13 to the bottom of slab at Elev. +9 and installation of a seven foot reinforced concrete pile cap to top of slab to Elev.+16. The compressible material was not surcharged in this area, therefore the material undergoes an unloading during excavation, a reload to the equivalent height of concrete to reach existing stress conditions, and virgin compression due to the remaining height of concrete.

During the 24-hour period when the concrete is first poured, the fluid weight of concrete will be resting directly on the subgrade. This fluid weight will produce settlement that is a percentage of the total primary settlement if this weight was a permanent increase in stress on the subgrade. To determine this partial settlement over the short period when the concrete is fluid, the time to primary consolidation of Stratum O was calculated, and the percent consolidation was calculated by dividing the 24 hour period by the time to primary. This percent consolidation was then multiplied by the total settlement resulting from the weight of the fluid concrete to obtain the settlement occurring over the 24 hour set-up time. This sequences results in approximately 0.1 inches of consolidation settlement.

Area 3: Point Street Roadway Grading (Section 3-3)

Settlement will result from raising grades to accommodate the proposed grading scheme. This area was not pre-loaded and fill placed will result in significant virgin compression. An average fill of 9 feet was estimated from approximately Elev. +10 to Elev. +19 and results in approximately 10.5 inches of consolidation settlement.

Area 4: Dock Street overlying Buried Bulkhead Structure (Section 4-4)

Settlement may result from the potential for the buried bulkhead structure to deteriorate. Historically, the bulkhead structure has allowed the fill above it to arch and shed load to the timber piles and passes some portion on to the soft compressible Stratum O soil below, see Figure 2. Based on the wide scatter of laboratory data and S-B barrier documentation from Reference 1, many unknowns exist regarding the present stress state of Stratum O within the buried bulkhead structure. For this analysis, it was assumed that the bulkhead structure has carried and currently carries roughly 50% of the load placed on/above it at Elev. 0 and passes the remaining 50% on to Stratum O below. This area was preloaded to Elev. +23 and thus Stratum O was consolidated to an equivalent fill height of 11.5 feet above Elev. 0.

Two analyses were performed to assess consolidation settlement in the event the bulkhead deteriorates and no longer carries load. These analyses include, consolidation settlement under existing grades and under subsequent grading. The results are:

- Bulkhead deteriorates under existing grade and carries no load, Stratum O thus feels the full height of fill from Elev. 0 to Elev. +9, which is equivalent to 9 feet of fill above Elev. 0. This does not exceed the pre-load and results in approximately 0.75 inches of consolidation settlement;
- Bulkhead deteriorates under proposed grades and carries no load, Stratum O thus feels the full height of fill from Elev. 0 to Elev. +18, which is an equivalent to 18 feet of fill above Elev. 0. This exceeds the pre-load and results in approximately 10.75 inches of consolidation settlement;

Secondary Compression

The magnitude of secondary compression was computed under Wills Street, at the location where the applied load on the MMC due to fill placement is the greatest. Boring No. MR-801 was used as the basis for this analysis because it is directly adjacent to the area of interest and was drilled after surcharging, and therefore captures the stress history at Wills Street. The coefficient of secondary compression was determined using the results of consolidation testing performed on a sample from MR-801, and it was assumed that all primary consolidation occurred prior to the start of construction under the previous surcharge.

Given these assumptions, the magnitude of secondary compression fifteen years after construction is approximately 1.05 inches, and thirty-five years after construction is approximately 1.7 inches. The details of this calculation can be seen in Appendix A.

Results

Settlement estimates summarized below in Table 1 indicate that in areas where fill is placed that were not pre-loaded or where the buried bulkhead structure shadows load, results in settlement between 7 and 18 inches. Settlement of this magnitude risks substantially damaging the geomembrane within the MMC and HMS components. In areas where fill is placed that was pre-loaded and exceeds the pre-load, results in settlement ranging from 3.5 to 5 inches. Settlement of this magnitude risks damaging the geomembrane within the MMC and HMS components. In areas where fill is placed that was pre-loaded and does not exceed the pre-load, results in settlement ranging from ¼ to 1 inch. Settlement of this magnitude can be accommodated by the geomembrane. In Area 1, fill above Elev. +18.5 will result in detrimental settlement.

Area	**Permanent Settlement Sources**	**Estimated Settlement, inches**
1a	Elastic Compression and Re-compression, pre-loaded	¼ to 1
1b	Elastic Compression, Re-compression and Virgin Compression, pre-loaded	3 ½ to 5
2	Short Duration Virgin Compression, not pre-loaded	< 1/8
3	Elastic Compression and Virgin Compression, not pre-loaded	9 to 12
4a	Elastic Compression and Re-compression, pre-loaded and sheltered load	½ to 1 ¼
4b	Elastic Compression, Re-compression and Virgin Compression, pre-loaded and sheltered load	7 to 18

The resulting slope of the geomembrane was assessed assuming areas that would experience virgin compression would be founded on pile foundations and results are shown on Figure 4. The resulting re-compression settlement will not significantly alter the slope of the geomembrane.

Discussion

In general, areas that will experience virgin compression will result in settlement that is detrimental to the integrity of the multimedia cap and HMS components and will require redistribution of loading to strata that can support the load. Areas 1b, 3, and 4b should be supported by pile foundations. Areas that will experience re-compression only will not result in settlement that is detrimental to the multimedia cap.

A. Patrone

By: ______________________

Alexandra E. Patrone

By: ______________________

Adam M. Dyer

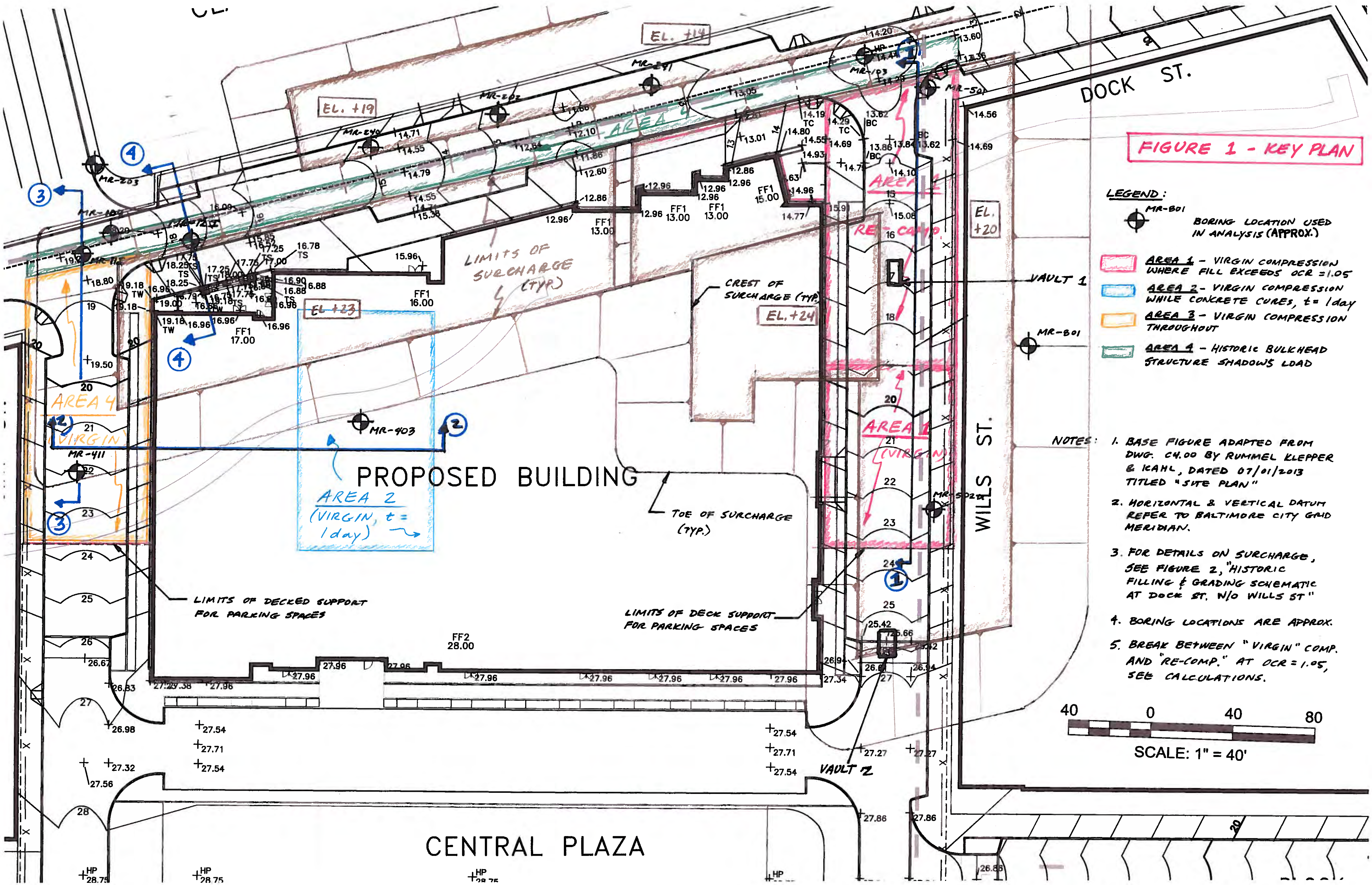


FIGURE 1 - KEY PLAN
LEGEND:
MR-801
BORING LOCATION USED IN ANALYSIS (APPROX.)
AREA 1 - VIRGIN COMPRESSION WHERE FILL EXCEEDS OCR = 1.05
AREA 2 - VIRGIN COMPRESSION WHILE CONCRETE CURES, t = 1 day
AREA 3 - VIRGIN COMPRESSION THROUGHOUT
AREA 4 - HISTORIC BULKHEAD STRUCTURE SHADOWS LOAD
NOTES:
1. BASE FIGURE ADAPTED FROM DWG. C4.00 BY RUMMEL KLEPPER & KAHL, DATED 07/01/2013 TITLED "SITE PLAN"
2. HORIZONTAL & VERTICAL DATUM REFER TO BALTIMORE CITY GRID MERIDIAN.
3. FOR DETAILS ON SURCHARGE, SEE FIGURE 2, "HISTORIC FILLING & GRADING SCHEMATIC AT DOCK ST. N/O WILLS ST"
4. BORING LOCATIONS ARE APPROX.
5. BREAK BETWEEN "VIRGIN" COMP. AND "RE-COMP." AT OCR = 1.05, SEE CALCULATIONS.
40 0 40 80
SCALE: 1" = 40'
DOCK ST.
WILLS ST.
CENTRAL PLAZA
PROPOSED BUILDING
AREA 2 (VIRGIN, t = 1 day)
AREA 1 (VIRGIN)
AREA 1 RE-COMP.
AREA 4
AREA 4 VIRGIN
LIMITS OF SURCHARGE (TYP.)
CREST OF SURCHARGE (TYP.)
TOE OF SURCHARGE (TYP.)
LIMITS OF DECKED SUPPORT FOR PARKING SPACES
LIMITS OF DECK SUPPORT FOR PARKING SPACES
VAULT 1
VAULT 2
EL. +19
EL. +19
EL. +23
EL. +24
EL. +20
MR-203
MR-189
MR-411
MR-403
MR-240
MR-202
MR-291
MR-103
MR-501
MR-502
MR-801
FF1 17.00
FF1 16.00
FF1 13.00
FF1 13.00
FF1 13.00
FF1 15.00
FF2 28.00

MUESER RUTLEDGE CONSULTING ENGINEERS

SHEET NO. 1 OF 2
FILE 11896A-10

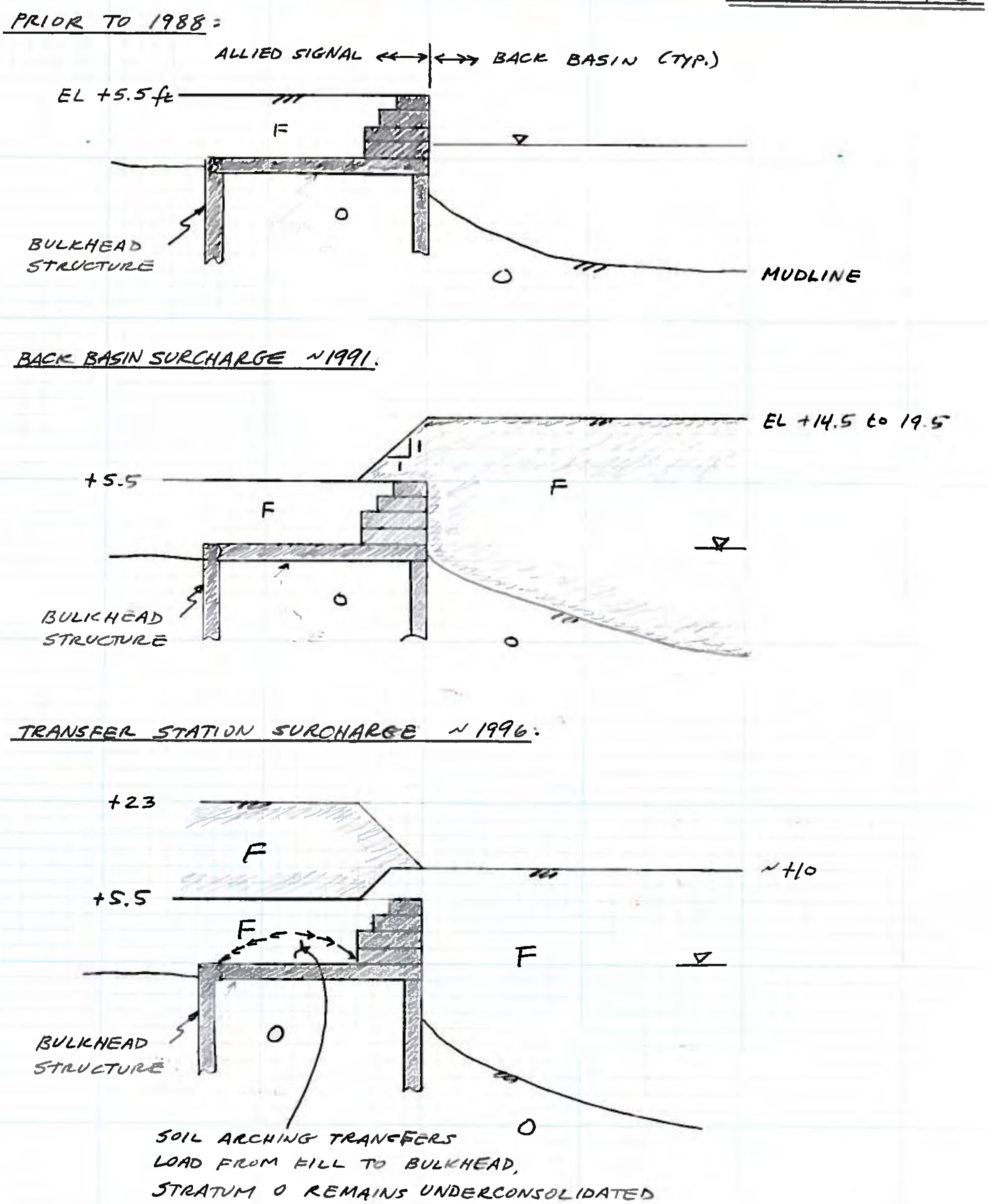
FORM 3

FOR EXELON TOWER & TF GARAGE

MADE BY AMD DATE 6/26/13
CHECKED BY PWD DATE 7/3/13

SUBJECT FIGURE 2: HISTORIC FILLING GRADING, AND SURCHARGING OF DOCK ST W/O WILLS ST.

NOT TO SCALE



MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896A-10

FORM 3

FOR EXELON TOWER & TF GARAGE

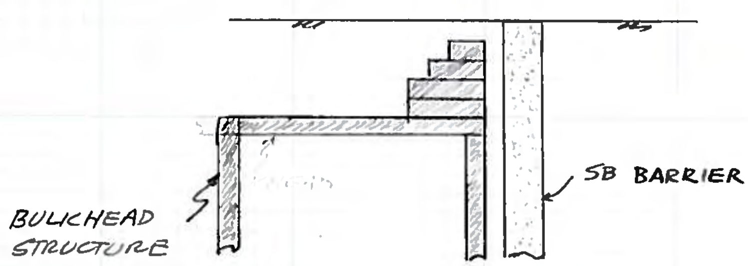
MADE BY AMD DATE 6/26/13

CHECKED BY PWD DATE 7/3/13

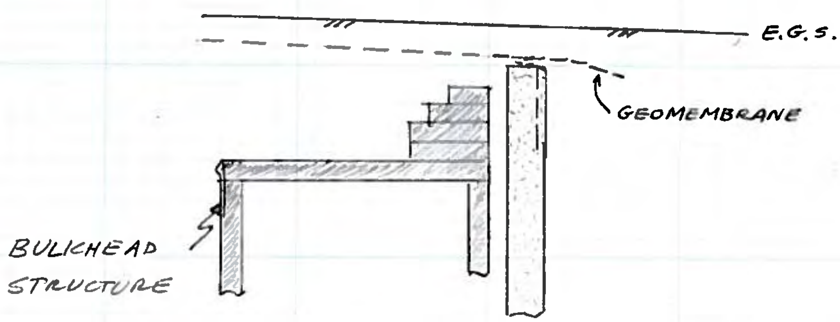
SUBJECT FIGURE 2: HISTORIC FILLING, GRADING, AND SURCHARGING OF DOCK ST, W/O WILLS ST.

SB BARRIER CONSTRUCTION ~1999

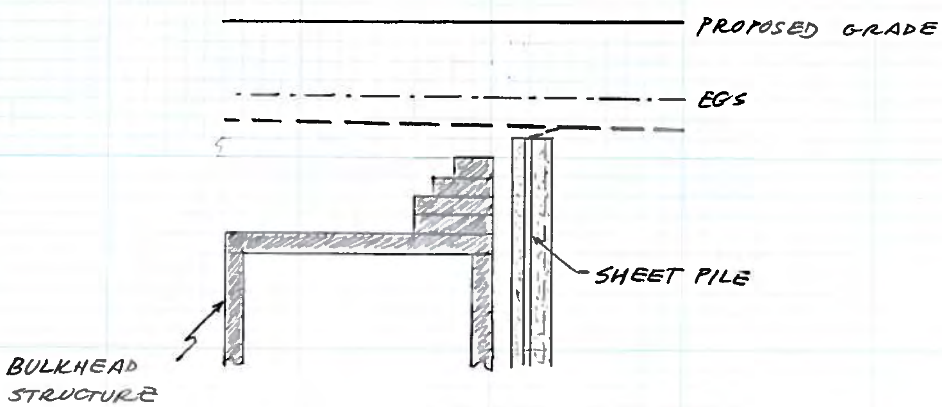
NOT TO SCALE



PRESENT DAY 2013



PROPOSED



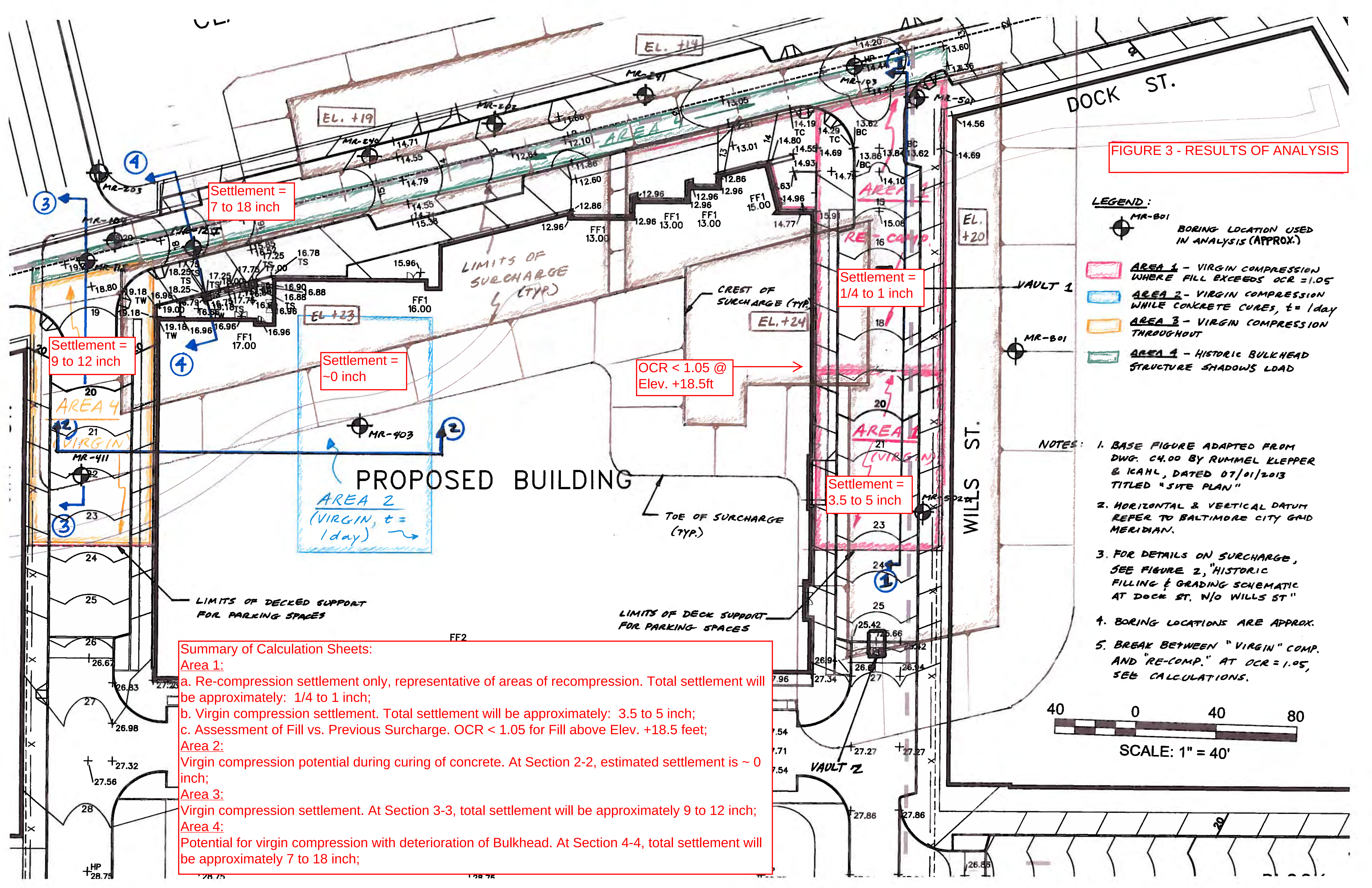


FIGURE 3 - RESULTS OF ANALYSIS
DOCK ST.
WILLS ST.
PROPOSED BUILDING
Settlement = 7 to 18 inch
Settlement = 9 to 12 inch
Settlement = ~0 inch
Settlement = 1/4 to 1 inch
Settlement = 3.5 to 5 inch
OCR < 1.05 @ Elev. +18.5ft
AREA 1
AREA 1 (VIRGIN)
AREA 1 RE-COMP.
AREA 2 (VIRGIN, t = 1 day)
AREA 4
AREA 4 VIRGIN
LIMITS OF SURCHARGE (TYP.)
CREST OF SURCHARGE (TYP.)
TOE OF SURCHARGE (TYP.)
LIMITS OF DECKED SUPPORT FOR PARKING SPACES
LIMITS OF DECK SUPPORT FOR PARKING SPACES
VAULT 1
VAULT 2
EL. +19
EL. +19
EL. +20
EL. +23
EL. +24
MR-201
MR-202
MR-203
MR-103
MR-110
MR-112
MR-240
MR-403
MR-411
MR-501
MR-502A
MR-801
LEGEND:
MR-801 BORING LOCATION USED IN ANALYSIS (APPROX.)
AREA 1 - VIRGIN COMPRESSION WHERE FILL EXCEEDS OCR = 1.05
AREA 2 - VIRGIN COMPRESSION WHILE CONCRETE CURES, t = 1 day
AREA 3 - VIRGIN COMPRESSION THROUGHOUT
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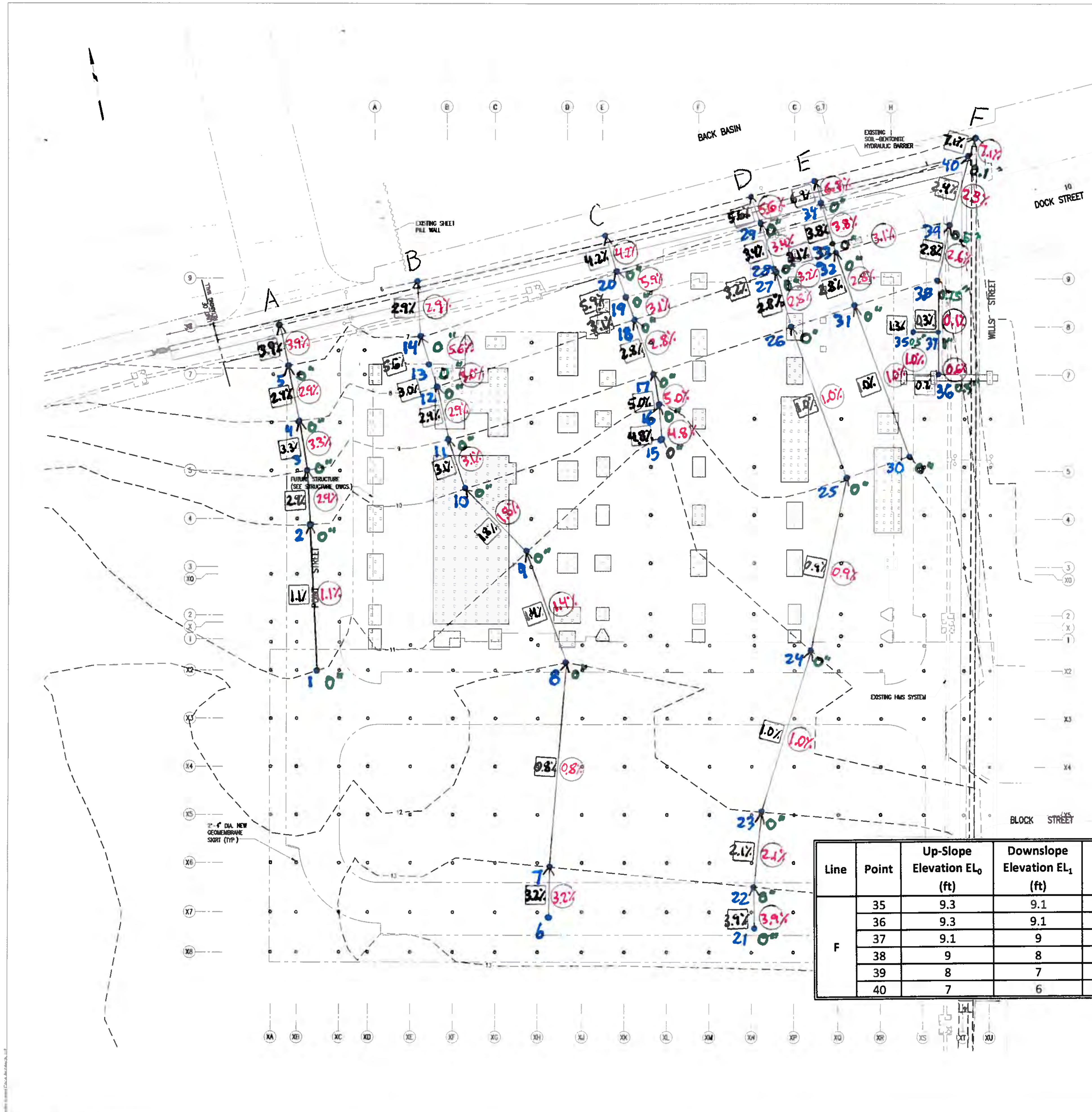
5. BREAK BETWEEN "VIRGIN" COMP. AND "RE-COMP." AT OCR = 1.05, SEE CALCULATIONS.
40 0 40 80
SCALE: 1" = 40'
Summary of Calculation Sheets:
Area 1:
a. Re-compression settlement only, representative of areas of recompression. Total settlement will be approximately: 1/4 to 1 inch;
b. Virgin compression settlement. Total settlement will be approximately: 3.5 to 5 inch;
c. Assessment of Fill vs. Previous Surcharge. OCR < 1.05 for Fill above Elev. +18.5 feet;
Area 2:
Virgin compression potential during curing of concrete. At Section 2-2, estimated settlement is ~ 0 inch;
Area 3:
Virgin compression settlement. At Section 3-3, total settlement will be approximately 9 to 12 inch;
Area 4:
Potential for virgin compression with deterioration of Bulkhead. At Section 4-4, total settlement will be approximately 7 to 18 inch;



Notes:

1. Elevation contours of Geomembrane taken from Phase II Construction Completion Report, dated February 2000.
2. Elevation contours continue beyond actual extent of Geomembrane, which terminates approximately 6.5 feet outboard of Hydraulic Barrier center line.
3. Settlement estimates taken from MRCE's Engineering Evaluation Memorandum "Estimated Settlement under Development Fill", dated June 28, 2013.
4. Settlement below structure and deck along north wall is assumed negligible as structure floors and slab for deck are pile-supported.
5. Slope of Geomembrane computed from contours shown on plan. Slope % = Rise/Run.
6. Projected slope of Geomembrane assumes settlement only occurs at up-slope location as indicated

Legend.

13 – As-Built elevation contour of Synthetic Cap from survey by Morris Ritchies Associate

1.8% — Existing Slope
1" — Assumed Settlement
1 — Settlement Point Number
1.7% — Projected Slope After Settlement

Line	Point	Up-Slope Elevation EL_0 (ft)	Downslope Elevation EL_1 (ft)	Slope Distance (ft)	Projected Settlement (in)	Existing Slope (%)	Projected Slope (%)
F	35	9.3	9.1	16	0.5	1.3	1.0
	36	9.3	9.1	27	0.5	0.7	0.6
	37	9.1	9	32	1	0.3	0.1
	38	9	8	36	0.75	2.8	2.6
	39	8	7	42.5	0.5	2.4	2.3
	40	7	6	14	0.1	7.1	7.1

GRAPHIC SCALE

PROGRESS SET NOT FOR CONSTRUCTION

BHC ARCHITECTS

EXELON BLDG & PLAZA GARAGE

HARBOR POINT AREA 1 PHASE 1 DDP SUBMISSION 7/1/13

GEOMEMBRANE SLOPE ANALYSIS

DDP SKETCH 1

MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896A

MADE BY AMD DATE 7/2/13

PROJECT EXELON TOWER & TF GARAGE

CHECKED BY ALS DATE 7/3/13

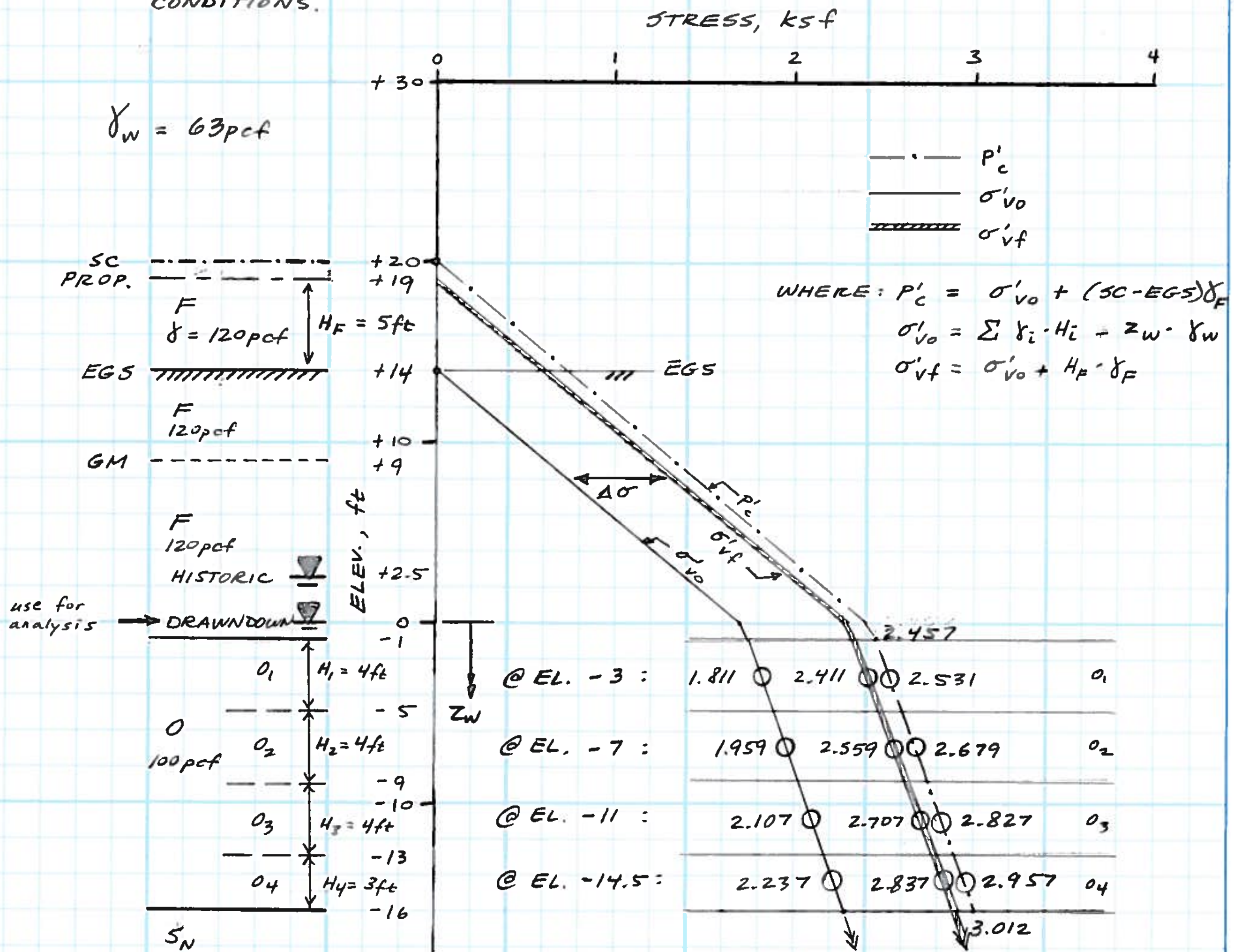
SUBJECT ATTACHMENT A: SETTLEMENT CALCULATIONS (SAMPLE HAND CALC)

AREA 1 : RAISED GRADES ALONG WILLS ST.

FOR EXTENTS SEE FIGURE 1 & SECTION 1-1

STRATIGRAPHY & STRESS STATE :

CALCULATE STRESS STATE FOR HISTORIC, EXISTING, & PROPOSED CONDITIONS.



NOTE : $\sigma'_{vf} < P'_c$.

DEFINITIONS:

SC = MAX. PREVIOUS SURCHARGE

PROP. = PROPOSED GRADE

EGS = EXISTING GROUND SURFACE

GM = GEOMEMBRANE

F = GRANULAR FILL STRATUM

O = ORGANIC CLAY STRATUM

S_N = NATURAL SAND STRATUM

Z_W = DEPTH BELOW GROUNDWATER

H_N = THICKNESS OF LAYER N

P'_c = MAXIMUM PAST VERTICAL STRESS

$\Delta\sigma$ = CHANGE IN STRESS = $\sigma'_{vf} - \sigma'_{vo}$ = $\gamma_F \cdot H_F$

σ'_{vo} = EXISTING VERTICAL EFFECTIVE OVERBURDEN STRESS

σ'_{vf} = FINAL VERTICAL EFFECTIVE OVERBURDEN STRESS

MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896A

MADE BY AMD DATE 7/2/13

PROJECT EXELON TOWER & TF GARAGE

CHECKED BY ALS DATE 7/3/13

SUBJECT ATTACHMENT A : SETTLEMENT CALCULATIONS (SAMPLE HAND CALC)

<u>CALCULATION OF SETTLEMENT</u> : $\delta_T = \delta_I + \delta_c + \delta_s$

WHERE: $\delta_I = \Delta\sigma \sum \frac{H_i I}{E_i}$; ("ELASTIC" COMPRESSION) (FOR GRANULAR, FREE DRAINING)

FOR $\sigma'_{vf} < P'_c$: (RECOMPRESSION ONLY)

$$\delta_c = \frac{H_i}{1+e_{oi}}\left[C_{si} \cdot \log_{10}\left[\frac{\sigma'_{vf}}{\sigma'_{vo}}\right]\right] ;$$

FOR $\sigma'_{vf} > P'_c$: (RECOMPRESSION & VIRGIN COMPRESSION)

$$\delta_c = \frac{H_i}{1+e_{oi}}\left[C_{si} \cdot \log_{10}\left[\frac{P'_c}{\sigma'_{vo}}\right] + C_{ci} \cdot \log_{10}\left(\frac{\sigma'_{vf}}{P'_c}\right)\right] ;$$

$$\delta_s = H_i \cdot C_\alpha \log_{10}\left(\frac{\Delta t}{t_p}\right)$$; (SECONDARY COMPRESSION, NEGLIGIBLE FOR RE-COMPRESSION)

SETTLEMENT COMPUTED AFTER: "AN ENGINEERING MANUAL FOR SETTLEMENT STUDIES" BY J.M. DUNCAN AND A.L. BUCHIGNANI (1987)

BY INSPECTION OF N-VALUES, E_i FOR STRATA M AND BELOW >> E_i FOR STRATA F, S_1, S_2 ∴ δ_I FROM STRATA F, S_1, S_2

<u>COMPRESSIBILITY PARAMETERS</u> : (SEE ATTACHMENT B)

STRATA F, S_1, S_2 E_i = 740 ksf

STRATUM O : $w_i = (5 - El.)/0.3404$

$e_o = 0.0272w$; $C_c = 0.0112w$; $C_s = 0.005$

STRATUM M : ASSUMED TO BE HEAVILY OVERCONSOLIDATED AND HENCE $P'_{CM} >> P'_{CO}$, $C_{CM} << C_{CO}$, $C_{SM} << C_{SO}$

<u>DEFINITIONS</u>:

δ_T	=	TOTAL SETTLEMENT	C_s	=	SWELL INDEX
δ_I	=	IMMEDIATE ELASTIC SETTLEMENT	w	=	NATURAL WATER CONTENT
δ_c	=	CONSOLIDATION SETTLEMENT	Δt	=	TIME TO OBSERVE SECONDARY COMPRESSION
δ_s	=	SECONDARY COMPRESSION			
E_i	=	ELASTIC MODULUS OF SUBLAYER i	t_p	=	TIME FOR PRIMARY CONSOLIDATION TO OCCUR.
e_o	=	INITIAL VOID RATIO			
C_c	=	VIRGIN COMPRESSION INDEX	C_α	=	SECONDARY COMPRESSION RATIO, STRAIN PER LOG CYCLE
I	=	INFLUENCE FACTOR			

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MADE BY AMD DATE 7/2/13
CHECKED BY ALS DATE 7/3/13

PROJECT EXELON TOWER & TF GARAGE

SUBJECT ATTACHMENT A: SETTLEMENT CALCULATION (SAMPLE HAND CALC)

CALCULATION OF CONSOLIDATION SETTLEMENT, δ_c:

$$\sigma'_{vf} < P'_c \quad \therefore \delta_c = \sum_{i=1}^{4} \frac{H_i}{1+e_{oi}} \left[C_{si} \cdot \log_{10} \left(\frac{\sigma'_{vfi}}{\sigma'_{voi}} \right) \right] \quad \text{(STRATUM O ONLY)}$$

LAYER O_i	H_i (ft)	ELEV. OF MIDPT (ft)	σ'_{voi} (ksf)	σ'_{vfi} (ksf)	W_i (%)	e_{oi} (--)	C_{si} (--)	δ_{ci} (in)
O_1	4	-3	1.811	2.411	23.5	0.639	0.012	0.043
O_2	4	-7	1.959	2.559	35.3	0.959	0.018	0.050
O_3	4	-11	2.107	2.707	47.0	1.278	0.024	0.054
O_4	3	-14.5	2.237	2.837	57.3	1.558	0.029	0.041

δ_c = $\boxed{0.188}$ (in)

EXAMPLE CALC.: FOR LAYER O_1

$w_i = (5-(-3))/0.3404 = 23.5\%$

$e_{oi} = 0.0272 \cdot 23.5 = 0.639$

$C_{ri} = 0.0005 \cdot 23.5 = 0.012$

$$\delta_{ci} = \frac{4\,ft \cdot 12\,in/ft}{1+0.639} \cdot \left[0.012 \cdot \log_{10} \left(\frac{2.411\,ksf}{1.811\,ksf} \right) \right] = 0.043\,in$$

CALCULATION OF IMMEDIATE SETTLEMENT, δ_I:

$\delta_I = \Delta\sigma \cdot H_{F,S_1,S_2} \cdot I / E$ Influence factor, $I = 1.0$ FOR 1-D LOADING

FROM SECTION 1-1, H_{F,S_1,S_2} ABOVE STRATUM M ~ 20 ft

$\therefore \delta_I = 0.60\,ksf \cdot 20\,ft \cdot 12\,in/ft \cdot 1.0 / 740\,ksf = \underline{0.195\,in}$

IN GENERAL, δ_I WILL VARY BETWEEN NO AND 3/4 in BASED ON MEDIUM DENSE STRATA THICKNESS UP TO 30 ft THICK AND BETWEEN 0 AND 13 ft OF FILL.

CALCULATION OF SECONDARY COMPRESSION, δ_s:

$\sigma'_{vf} < P'_c \quad \therefore \underline{\delta_s \sim 0\,in}$ (δ_s IS NEGLIGIBLE IN RECOMPRESSION)

TOTAL ESTIMATED SETTLEMENT, δ_T:

$\delta_T = \delta_I + \delta_c + \delta_s = 0.195 + 0.188 + 0\,in = \underline{0.383\,in}$ SAY $\boxed{1/4 \text{ to } 1 \text{ in}}$

MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896A

PROJECT EXELON TOWER & TF GARAGE

MADE BY AMD DATE 7/2/13

CHECKED BY ALS DATE 7/3/13

SUBJECT ATTACHMENT 4: SETTLEMENT CALCULATION (SAMPLE HAND CALC)

<u>AREA: 4:</u> RAISED GRADES ALONG DOCK ST. CONSIDERING PRESENCE OF HISTORIC BULKHEAD STRUCTURE. FOR EXTENTS, SEE FIGURE 1 & SECTION 4-4.

<u>ASSUMPTIONS:</u>

1. DURING PREVIOUS SURCHARGING, BULKHEAD STRUCTURE CARRIED 50% OF LOAD PLACED ON/ABOVE IT. AND SHED 50% TO STRATUM O AROUND/BELOW IT.
2. ASSUMPTION 1 CURRENTLY HOLDS AT TODAY'S GRADE.

FROM 1 & 2: $\Delta q_{SOIL} = \Delta q_{BULKHEAD} = \gamma_F \cdot H_F \cdot 0.5$

FOR SCHEMATIC OF HISTORICAL GRADING & FILLING, SEE FIGURE 2.

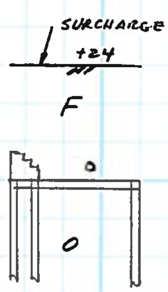
<u>DEFINITIONS:</u> AS FROM P. 1, ADDITIONALLY:

Δq_{SOIL} = LOAD IMPARTED TO STRATUM O BELOW BULKHEAD

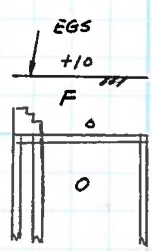
$\Delta q_{BULKHEAD}$ = LOAD IMPARTED TO BULKHEAD

<u>STRESS STATES:</u> (SCHEMATICALLY)

P'_c:

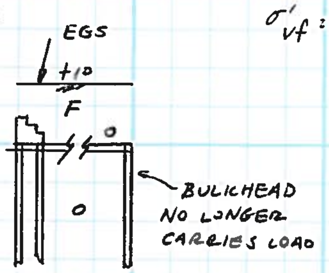


σ'_{vo}:

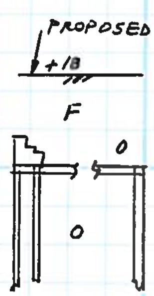


NOTE: FOR P'_c & σ'_{vo}, ASSUMPTIONS 1 & 2 HOLD

σ'_{v1}:



σ'_{vf}:



NOTE: FOR σ'_{v1} & σ'_{vf}, ASSUMPTIONS 1 & 2 DO <u>NOT</u> HOLD

σ'_{v1} = VERTICAL EFFECTIVE OVERBURDEN STRESS WHERE BULKHEAD CARRIES <u>NO</u> LOAD UNDER EXISTING GRADES

σ'_{vf} = SAME AS σ'_{v1} UNDER PROPOSED GRADES

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FILE 118964

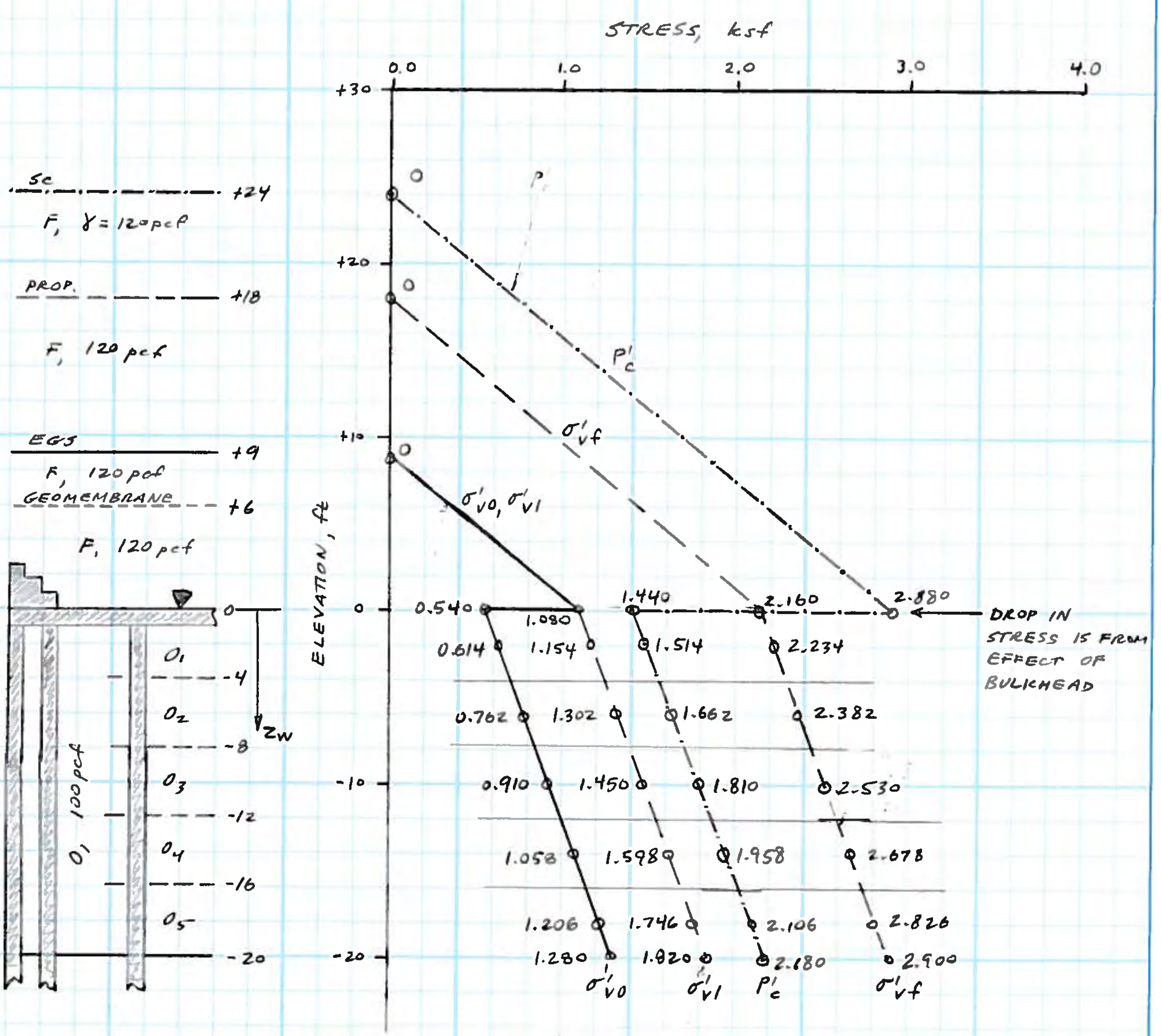
PROJECT EXELON TOWER & TF GARAGE

MADE BY AMD DATE 7/3/13

CHECKED BY ALS DATE 7/3/13

SUBJECT ATTACHMENT A: SETTLEMENT CALCULATIONS (SAMPLE HAND CALC)

STRATIGRAPHY & STRESS STATE:



BASED ON HISTORIC, PRESENT, PROPOSED, AND DETERIORATED STRESS STATES WILL ANALYZE δ_c FOR TWO CASES.

1. σ'_{v0} TO σ'_{v1} : BULKHEAD DETERIORATES UNDER EXISTING GRADES AND SHEDS ALL LOAD TO STRATUM O.

2. σ'_{v1} TO σ'_{vf} : RAISE GRADES TO PROPOSED WITH DETERIORATED BULKHEAD CARRYING NO LOAD.

SUBJECT ATTACHMENT A: SETTLEMENT CALCULATIONS (SAMPLE HAND CALC)

CALCULATION OF SETTLEMENT:

METHODS AS DESCRIBED ON P. 2.

COMPRESSIBILITY PARAMETERS:

STRATUM F, S_1, S_2 : $E_i = 740$ ksf

STRATUM O : $w_{AVG} = 94\% \pm 36\%$ ∴ $w_{min} = 58\%$, $w_{max} = 130\%$

$e_o = 0.0272w = 2.557$, $e_{min} = 1.578$, $e_{max} = 3.536$

$C_c = 0.0112w = 1.053$, $C_{c\,min} = 0.650$, $C_{c\,max} = 1.456$

$C_s = 0.0005w = 0.047$, $C_{s\,min} = 0.029$, $C_{s\,max} = 0.065$

CALCULATION OF CONSOLIDATION SETTLEMENT, δ_c:

1) σ'_{vo} TO σ'_{v1} : $\Delta\sigma = 540$ psf $\sigma'_{v1} < P'_c$ ∴ RE-COMPRESSION

LAYER O_i	H_i (ft)	EL. OF MIDPT (ft)	σ'_{vo} (ksf)	P'_c (ksf)	σ'_{v1} (ksf)	w (%)	e_o (--)	C_c (--)	C_s (--)	δ_{ci} (in)
O_1	4	-2	0.644	1.514	1.154	94	2.557	1.053	0.047	0.174
O_2	4	-6	0.762	1.662	1.302	94	↓	↓	↓	0.148
O_3	4	-10	0.910	1.810	1.450	94	↓	↓	↓	0.128
O_4	4	-14	1.058	1.958	1.598	94	↓	↓	↓	0.114
O_5	4	-18	1.206	2.106	1.746	94	↓	↓	↓	0.102
									δ_c	0.665 in

FOR $P'_c > \sigma'_{vn}$

$$\delta_c = \frac{H_i}{1+e_o} \cdot \left[C_c \log_{10}\left[\frac{\sigma'_{v1}}{P'_c}\right]\right]$$

FOR O_1 :

$$\delta_{c1} = \frac{4\text{ft} \cdot 12^{\text{in}}/\text{ft}}{1+2.557}\left[0.047 \cdot \log 10\left[\frac{1.154\text{ ksf}}{0.614\text{ ksf}}\right]\right] = \underline{0.174\text{ in}}$$

∴ SETTLEMENT OF EXISTING GRADE ALONG DOCK ST. IF BULKHEAD STRUCTURE DETERIORATES AND CARRIES NO LOAD, $\delta_{c\,0 \to 1} \approx$ 0.75 in

CONSIDERING VARIATION IN WATER CONTENT $\delta_{c\,0\to1} \sim$ 0.50 in to 1.25 in

MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896A

MADE BY AMD DATE 7/3/13

CHECKED BY ALS DATE 7/3/13

PROJECT EXELON TOWER & TF GARAGE

SUBJECT ATTACHMENT A: SETTLEMENT CALCULATION (SAMPLE HAND CALC)

2) σ'_{v1} TO σ'_{vf} $\Delta\sigma$ = 1080 psf $\sigma'_{vf} > P'_c$ $\therefore$ VIRGIN COMP.

LAYER O_i	H_i (ft)	El. of Mid Pt. (ft)	σ'_{v1} (ksf)	P'_c (ksf)	σ'_{vf} (ksf)	w (%)	e_o (--)	C_c (--)	C_s (--)	δ_{ci} (in)
O_1	4	-2	1.154	1.514	2.234	94	2.557	1.053	0.047	2.476
O_2	4	-6	1.302	1.662	2.382	↓	↓	↓	↓	2.288
O_3	4	-10	1.450	1.810	2.530	↓	↓	↓	↓	2.128
O_4	4	-14	1.598	1.958	2.678	↓	↓	↓	↓	1.988
O_5	4	-18	1.746	2.106	2.826	↓	↓	↓	↓	1.866
									δ_c (in) =	10.747

$\therefore$ ADDITIONAL SETTLEMENT FROM RAISING GRADES FOR PROPOSED DEVELOPMENT ASSUMING BULKHEAD CARRIES NO LOAD, $\delta_{c1\to f} \sim$ 11 in

$\delta_{c,TOT} \approx 0.75 + 11$ in $\approx$ 11.75 in

CONSIDERING VARIATION IN w, $\delta_{c,TOT} \sim$ 7 to 18 in

e.g. FOR $\sigma'_{vf} > P'_c$: $\delta_{ci} = \dfrac{H_i}{1+e_o}\left[C_s \log\left(\dfrac{P'_c}{\sigma'_{v1}}\right) + C_c \log\left(\dfrac{\sigma'_{vf}}{P'_c}\right)\right]$

FOR O_1 : $\delta_{c1} = \dfrac{4\,ft \cdot 12\,in/ft}{1 + 2.557}\left[0.047 \log\left(\dfrac{1.514}{1.154}\right) + 1.053\left[\log \dfrac{2.234}{1.154}\right]\right]$ = 2.476 in

MUESER RUTLEDGE CONSULTING ENGINEERS

File No.: 11896A

Made by: AEP Date: 6/6/13

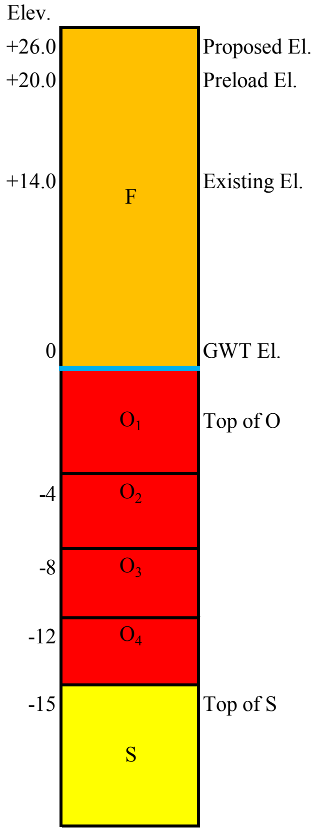
FOR EXELON

Checked by: AMD Date: 6/27/13

SUBJECT: 1-D SETTLEMENT ESTIMATE AREA 1 -DIFFERENTIAL SETTLEMENT ALONG WILLS ST. BETWEEN VAULTS 1 AND 2

ANALYSIS AT VAULT 2

IDEALIZED PROFILE:



REFERENCES:

1. GEOLOGIC SECTION 1-1
2. WATER CONTENT CORRELATIONS BASED ON MRCE LABORATORY TESTING

ASSUMPTIONS:

1. ANALYSIS BASED ON SUBSURFACE CONDITIONS PRESENTED IN SECTION 1-1
2. BY INSPECTION, SETTLEMENT WILL OCCUR DUE TO NEW FILL PLACEMENT TO ACHIEVE PROPOSED GRADE

CONSTRUCTION SEQUENCE

1. RELOAD TO HISTORIC PRELOAD ELEVATION
2. VIRGIN COMPRESSION TO PROPOSED ELEVATION EXCEEDING PRELOAD

GEOTECHNICAL PARAMETERS

LAYER	ELEV. OF MID. (FT)	σ'_{V0} (PSF)	ω_N (%)	e_0 (-)	Cc (-)	Cs (-)
O_1	-2.0	1794	21	0.56	0.23	0.01
O_2	-6.0	1942	32	0.88	0.36	0.02
O_3	-10.0	2090	44	1.20	0.49	0.02
O_4	-13.5	2220	54	1.48	0.61	0.03

LOADING

CONSTRUCTION PHASE	DESCRIPTION	LOADING CONDITION	Δh (FT)	$\Delta\sigma$ (PSF)
1	FILL TO PRELOAD EL.	RELOAD	6.0	720
2	FILL TO PROPOSED EL	VIRGIN	6.0	720

SETTLEMENT ESTIMATE

LAYER	H (FT)	$\sigma'_{VF(1)}$ (PSF)	$\sigma'_{VF(2)}$ (PSF)	P'c (PSF)	$\delta_{c,Cs}$ (in.)	$\delta_{c,Cc}$ (in.)	δ_c (in.)
O_1	4	2514	3234	2514	0.0	0.8	0.8
O_2	4	2662	3382	2662	0.1	1.0	1.0
O_3	4	2810	3530	2810	0.1	1.1	1.1
O_4	3	2940	3659.5	2939.5	0.0	0.8	0.9
				Σ	0.2	3.6	3.9

Approximately 3.5 to 5in

DETERMINE ELEVATION AT WHICH OVERCONSOLIDATION RATIO (OCR) = 1.05

$$\boxed{OCR = \frac{P'_c}{\sigma'_{v0}}} \qquad \frac{P'_c}{OCR} = \sigma'_V = \sigma'_{V0} + H_F * \gamma_F$$

$$H_F = \left(\frac{\frac{P'_c}{OCR} - \sigma'_{V0}}{\gamma_F} \right)$$

MAXIMUM PAST PRESSURE AT CENTER OF STRATUM O

P'c	2677.5	psf

EXISTING OVERBURDEN STRESS AT CENTER OF STRATUM O

σ'_{v0}	1957.5	psf

HEIGHT OF FILL (Hf) AT WHICH OCR = 1.05

H_f	4.5	feet

ELEVATION AT WHICH OCR = 1.05

EL	+18.5

Therefore, virgin compression settlement can be expected for fill grades higher than approximately Elev. +18.5

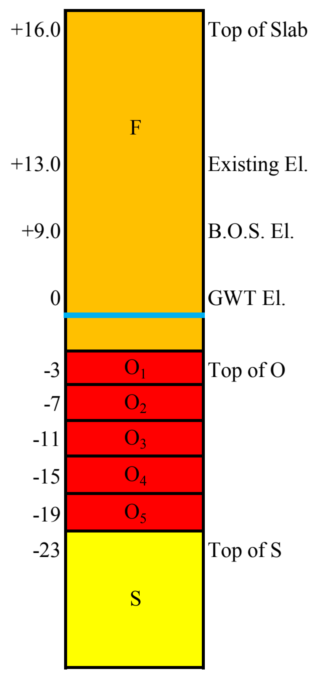
MUESER RUTLEDGE CONSULTING ENGINEERS

		File No.:	11896A
Made by:	AEP	Date:	6/26/13
Checked by:	AMD	Date:	6/27/13

FOR EXELON

SUBJECT: 1-D SETTLEMENT ESTIMATE — AREA 2 - MOMENT SLAB EXCAVATION

IDEALIZED PROFILE:



REFERENCES:

1. GEOLOGIC SECTION 2-2
2. WATER CONTENT CORRELATIONS BASED ON MRCE LABORATORY TESTING

ASSUMPTIONS:

1. ANALYSIS BASED ON SUBSURFACE CONDITIONS PRESENTED IN SECTION 2-2
2. BY INSPECTION, SETTLEMENT WILL OCCUR DUE TO EXCAVATION AND SUBSEQUENT CONCRETE SLAB PLACEMENT FOR 24-HOUR PERIOD
3. ASSUME STRATUM O IS NORMALLY CONSOLIDATED AND HAS NOT BEEN PRELOADED, DOUBLE DRAINAGE

CONSTRUCTION SEQUENCE

1. UNLOAD FROM EXISTING EL. TO BOTTOM OF SLAB ELEVATION
2. RELOAD TO EQUIVALENT HEIGHT OF CONCRETE
3. VIRGIN COMPRESSION TO TOP OF SLAB ELEVATION

GEOTECHNICAL PARAMETERS

LAYER	ELEV. OF MID. (FT)	σ'_{V0} (PSF)	ω_N (%)	e_0 (-)	Cc (-)	Cs (-)
O_1	-5.0	1805	26	0.70	0.29	0.01
O_2	-9.0	1953	41	1.11	0.46	0.02
O_3	-13.0	2101	55	1.51	0.62	0.03
O_4	-17.0	2249	70	1.91	0.79	0.04
O_5	-21.0	2397	85	2.31	0.95	0.04

LOADING

CONSTRUCTION PHASE	DESCRIPTION	LOADING CONDITION	Δh (FT)	$\Delta\sigma$ (PSF)
1	EXC. TO SUBGRADE	UNLOAD	-4.0	-480
2	POUR TO EQUIV. HEIGHT	RELOAD	3.2	480
3	POUR TO TOP OF SLAB	VIRGIN	3.8	570
		NET LOAD (FOR 24HR):		570

SETTLEMENT ESTIMATE

LAYER	H (FT)	σ'_{VF} (PSF)	P'c (PSF)	δ_c (in.)
O_1	4	2375	1805	1.0
O_2	4	2523	1953	1.2
O_3	4	2671	2101	1.2
O_4	4	2819	2249	1.3
O_5	4	2967	2397	1.3
			Σ	4.6

FOR U = 100%

FOR 1-DAY OF CONSOLIDATION:

Coeff. Of Consol., c_v	0.02	FT^2/DAY
Time, t	1.0	DAY
Time Factor, T	0.0002	--
Consolidation, U	0.02	%
$S_{p(1),\ 1DAY}$	0.07	IN

Approximately 0 to 0.125in

T, U AFTER TAYLOR'S SQUARE ROOT METHOD

MUESER RUTLEDGE CONSULTING ENGINEERS

File No.: 11896A

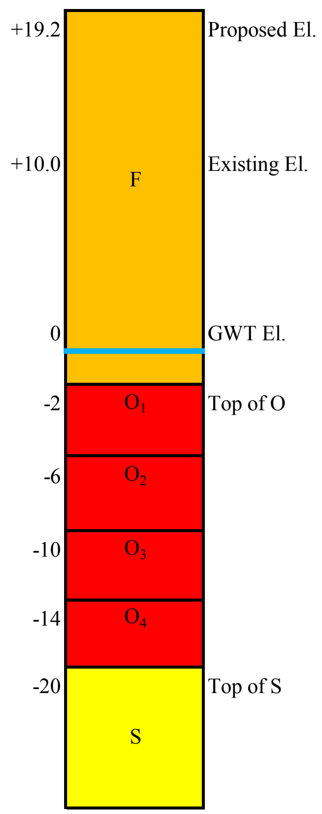
Made by: AEP Date: 6/26/13

Checked by: AMD Date: 6/27/13

FOR EXELON

SUBJECT: 1-D SETTLEMENT ESTIMATE AREA 3 - SETTLEMENT UNDER RAISED GRADES ALONG POINT ST.

IDEALIZED PROFILE:



REFERENCES:

1. GEOLOGIC SECTION 3-3
2. WATER CONTENT CORRELATIONS BASED ON MRCE LABORATORY TESTING

ASSUMPTIONS:

1. ANALYSIS BASED ON SUBSURFACE CONDITIONS PRESENTED IN SECTION 3-3
2. BY INSPECTION, SETTLEMENT WILL OCCUR DUE TO NEW FILL PLACEMENT TO ACHIEVE PROPOSED GRADE
3. ASSUME STRATUM O IS NORMALLY CONSOLIDATED AND HAS NOT BEEN PRELOADED, DOUBLE DRAINAGE

CONSTRUCTION SEQUENCE

1. VIRGIN COMPRESSION TO PROPOSED EL.

GEOTECHNICAL PARAMETERS

LAYER	ELEV. OF MID. (FT)	σ'_{V0} (PSF)	ω_N (%)	e_0 (-)	Cc (-)	Cs (-)
O_1	-4.0	1388	22	0.59	0.24	0.01
O_2	-8.0	1536	36	0.98	0.40	0.02
O_3	-12.0	1684	50	1.37	0.56	0.03
O_4	-17.0	1869	68	1.85	0.76	0.03

LOADING

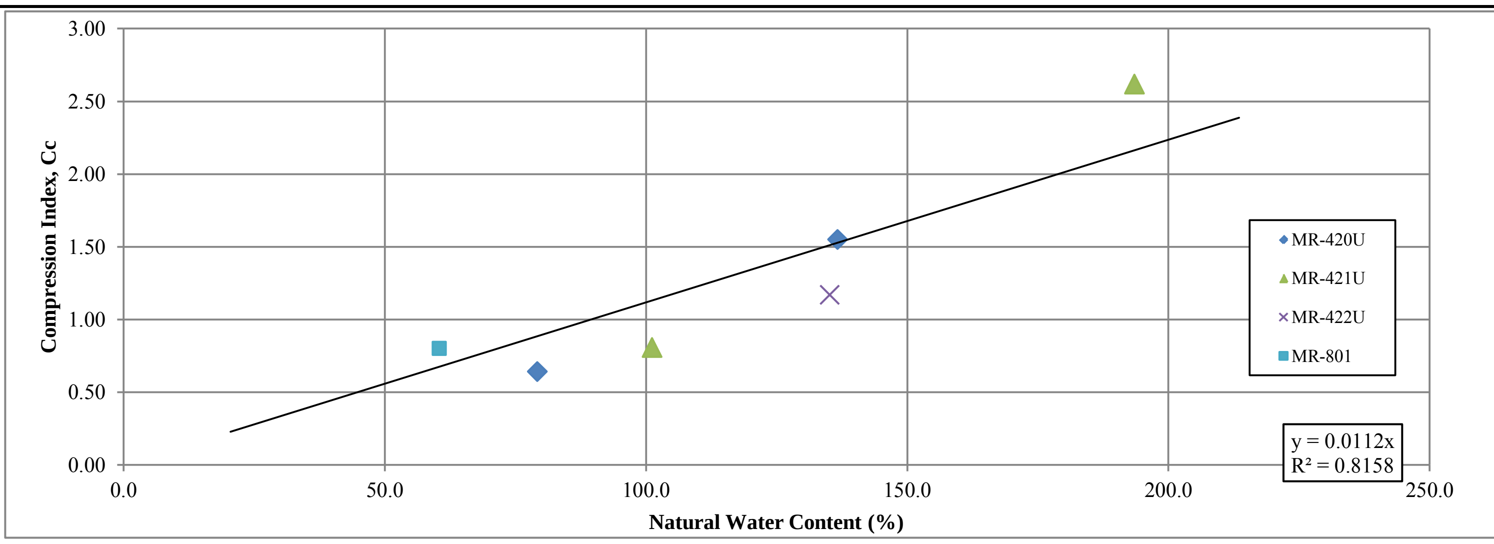
CONSTRUCTION PHASE	DESCRIPTION	LOADING CONDITION	Δh (FT)	$\Delta\sigma$ (PSF)
1	FILL TO PROPOSED EL.	VIRGIN	9.2	1104

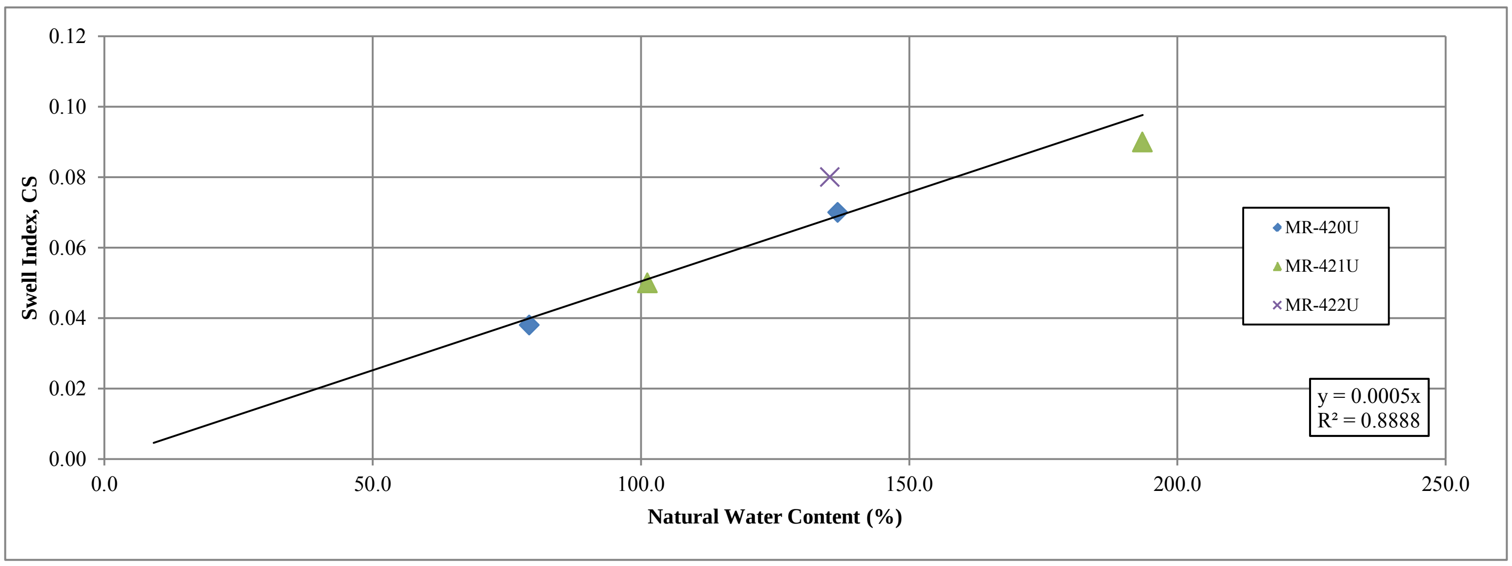
SETTLEMENT ESTIMATE

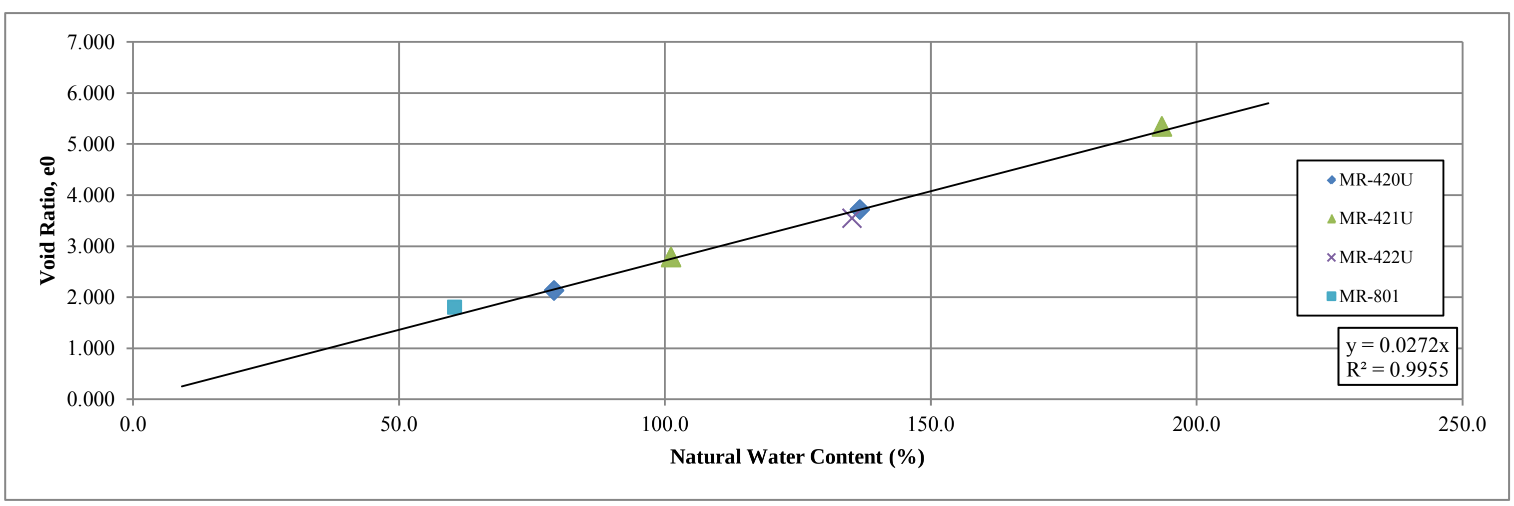
LAYER	H (FT)	σ'_{VF} (PSF)	P'c (PSF)	δ_c (in.)
O_1	4	2492	1388	1.9
O_2	4	2640	1536	2.3
O_3	4	2788	1684	2.5
O_4	6	2973	1869	3.9
			Σ	10.5

Approximately 9 to 12in

SUBJECT: 1-D SETTLEMENT ESTIMATE APPENDIX B - ASSESSMENT OF COMPRESSIBILITY CHARACTERISTICS







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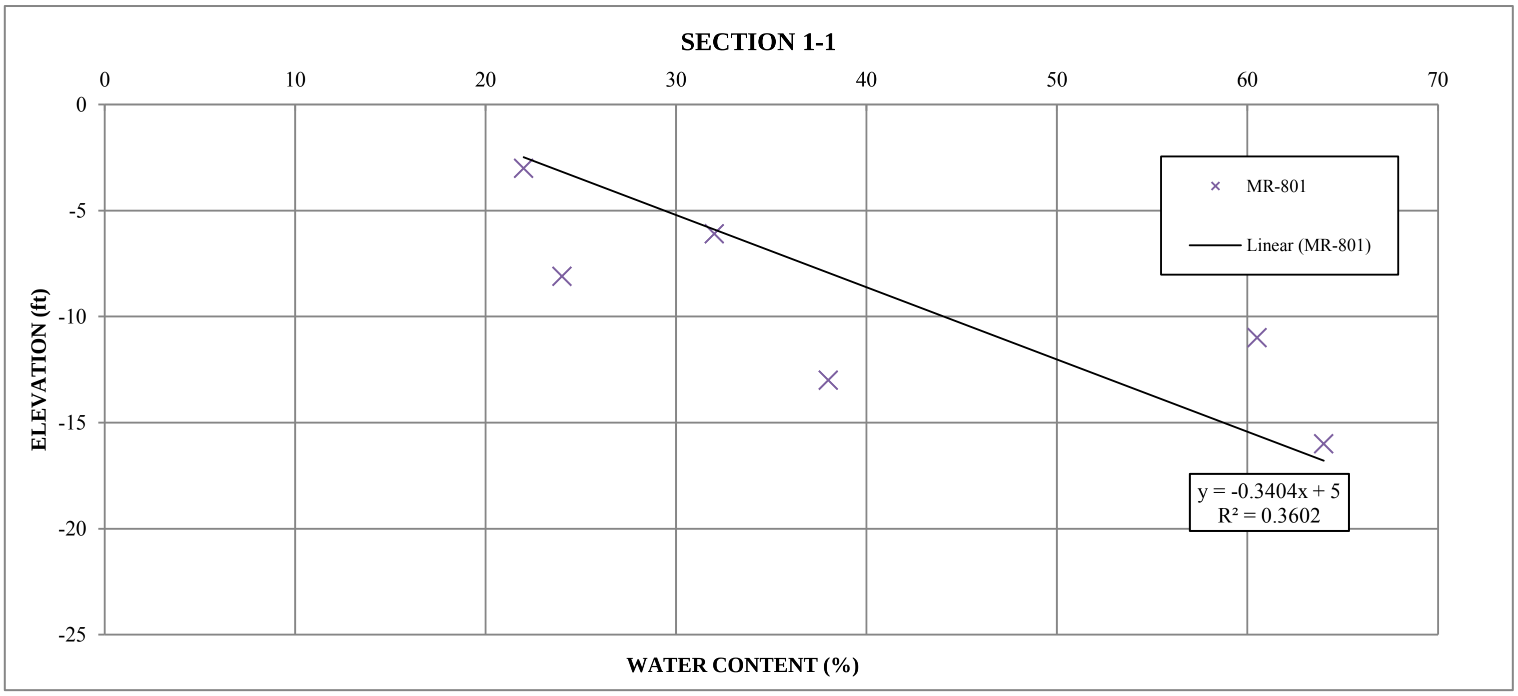
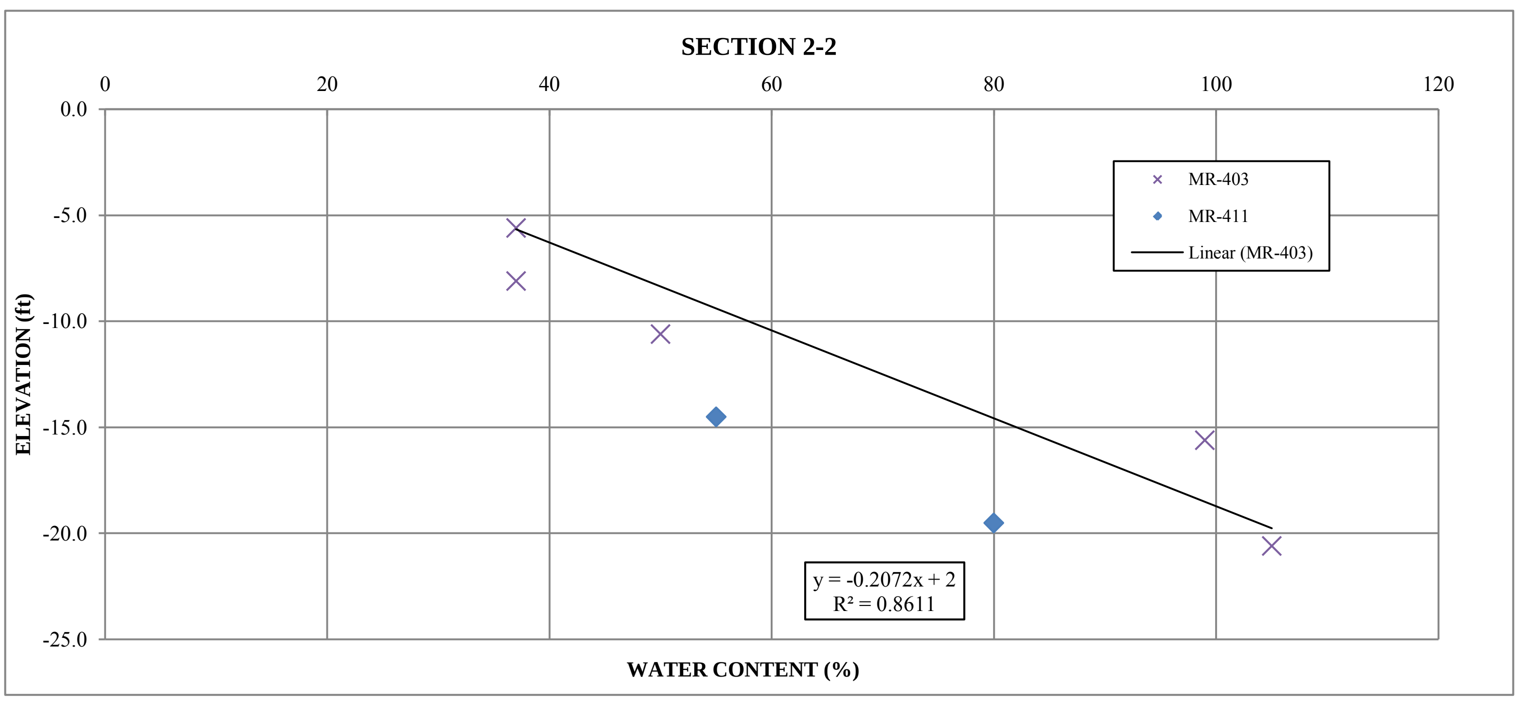
		File No.:	11896A
Made by:	AEP	Date:	6/6/13
Checked by:	AMD	Date:	6/27/13

FOR ____________

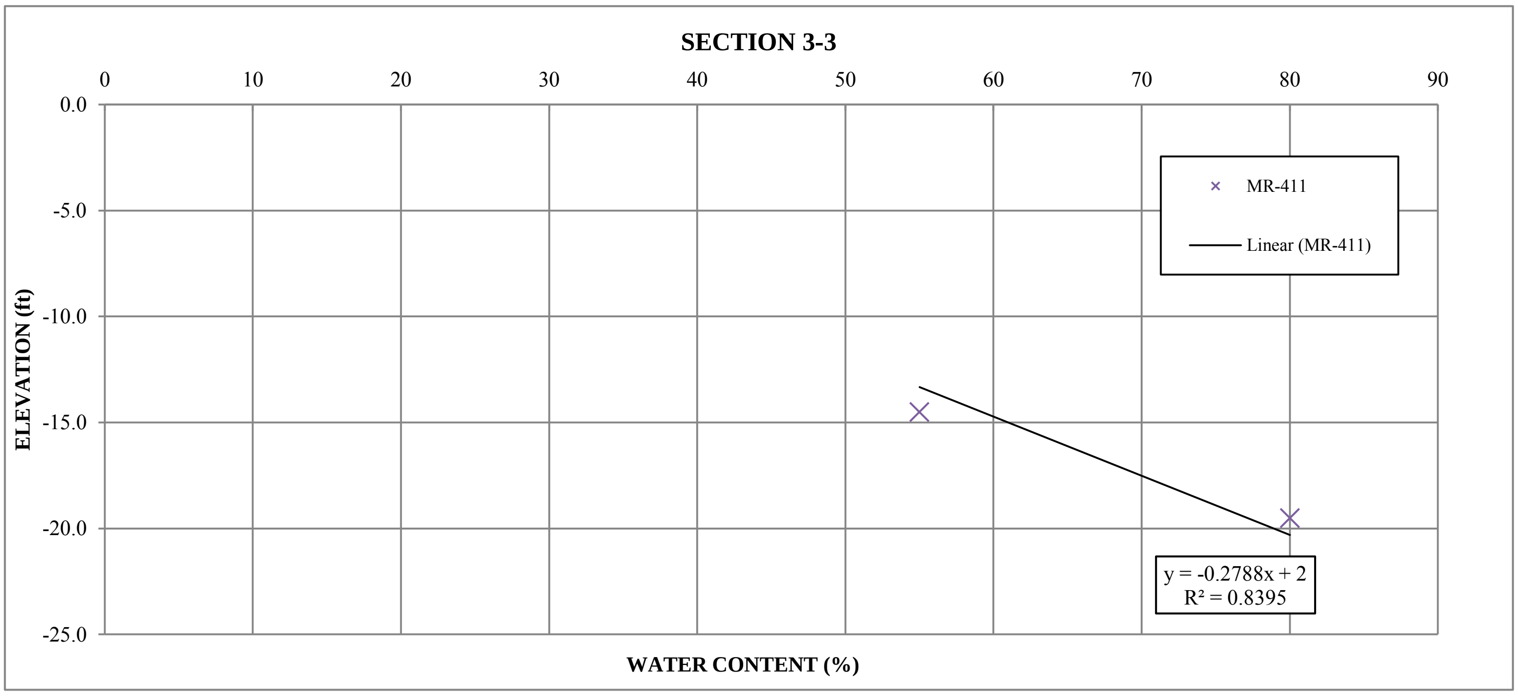
SUBJECT: 1-D SETTLEMENT ESTIMATE — APPENDIX B - ASSESSMENT OF COMPRESSIBILITY CHARACTERISTICS



Trendline: Elev. = -0.3404 * w+5
Therefore: w = (5 - Elev.) / 0.3404

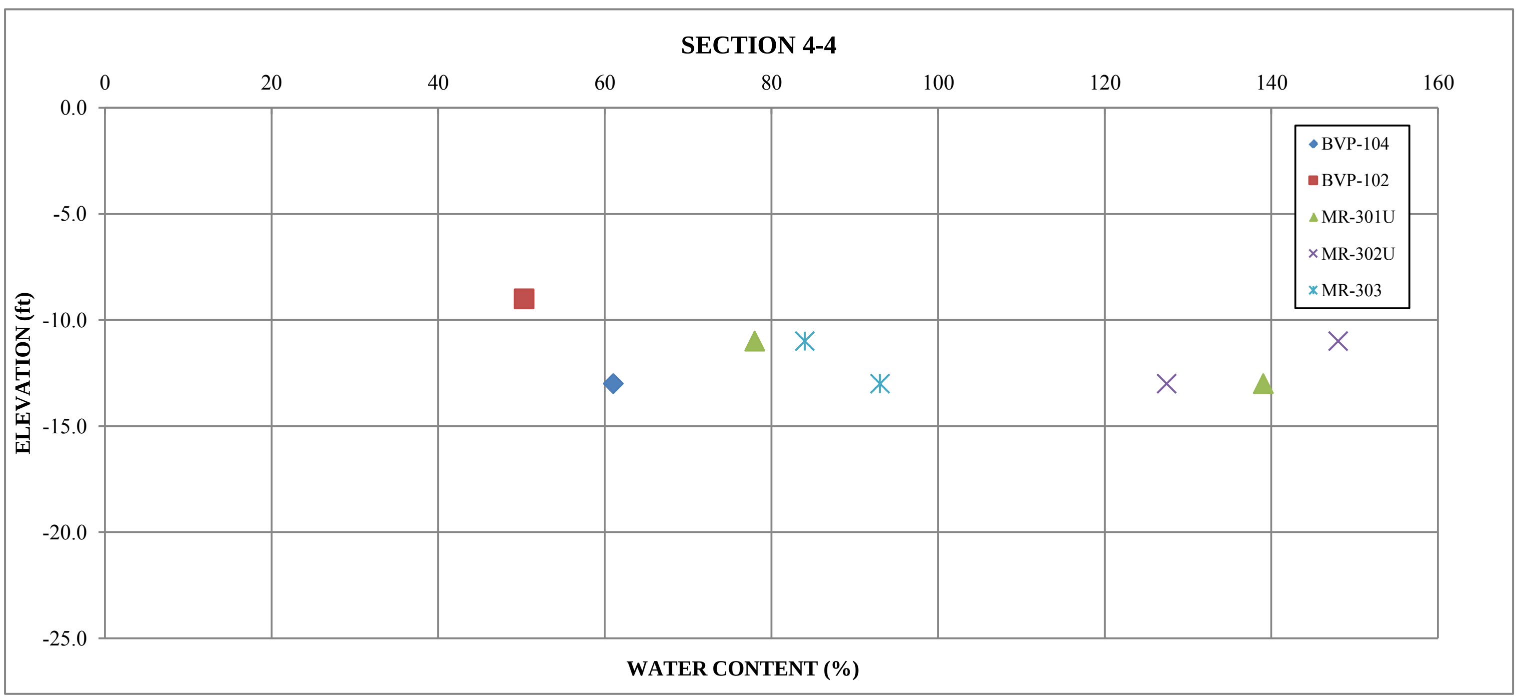


Trendline: Elev. = -0.27072 * w + 2
Therefore: w = (2 - Elev.) / 0.27072



Trendline: Elev. = -0.2788 * w + 2

Therefore: w = (2 - Elev.) / 0.2788



Average w: 98 %

Sigma 36 %

APPENDIX B:
FROM EPRI:
MANUAL ON ESTIMATING SOIL PROPERTIES FOR FOUNDATION DESIGN (1990)

Table 5-5

TYPICAL RANGES OF DRAINED MODULUS FOR SAND

Consistency	Normalized Elastic Modulus, E_d/p_a Typical	Driven Piles[a]
loose	100 to 200	275 to 550
medium	200 to 500	550 to 700
dense	500 to 1000	700 to 1100

a - Source: Poulos (17), p. 207.

STRATUM F, S_1, S_2
use $E_d/P_a = 350$
$\therefore$ E = 740 ksf

$$E_t = \kappa p_a (\bar{\sigma}_3/p_a)^n [1 - R_f (1 - \sin \bar{\phi}_{tc})(\bar{\sigma}_1 - \bar{\sigma}_3)/(2 \bar{\sigma}_3 \sin \bar{\phi}_{tc})]^2 \qquad (5\text{-}21)$$

in which $\bar{\sigma}_1$ and $\bar{\sigma}_3$ = effective major and minor principal stresses, respectively, $\bar{\phi}_{tc}$ = effective stress friction angle in triaxial compression, and κ, n, and R_f = modulus parameters given in Table 5-6. For convenience in computer code implementation, Trautmann and Kulhawy (1) approximated κ as follows:

$$\kappa \approx 300 + 900\ \phi_{rel} \qquad (5\text{-}22)$$

with ϕ_{rel} defined in Equation 5-8.

Correlations with Strength

The shear modulus commonly is correlated to the effective soil strength through the rigidity index (I_r), as defined below for drained loading:

$$I_r = G/(\bar{\sigma} \tan \bar{\phi}_{tc}) \qquad (5\text{-}23)$$

Selected values for I_r are given in Table 5-7. Of particular interest to note is that I_r increases with increasing relative density and decreases with increasing normal stress. It also is lower with more compressible soil minerals.

When using the rigidity index (I_r) for drained loading, volume changes normally have to be considered. Therefore, I_r must be corrected for the volumetric strains (ϵ_v) to yield a reduced rigidity index (I_{rr}), as given below by Vesić (20):

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SHEET ____ OF ____

FILE 11896A

PROJECT EXELON

MADE BY AEP DATE 6/6/13

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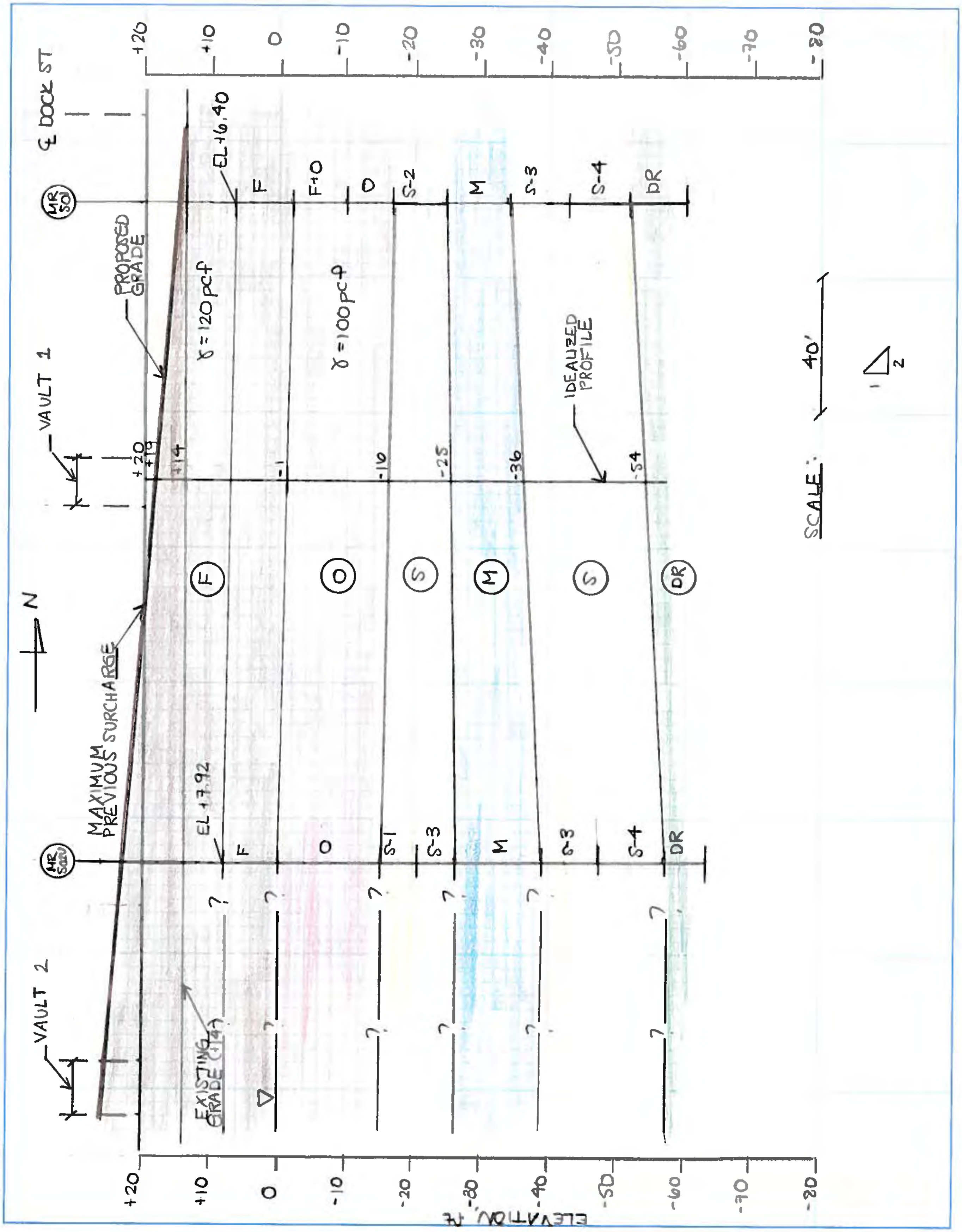
SUBJECT PRELIMINARY SETTLEMENT ESTIMATE - SECTION 1-1



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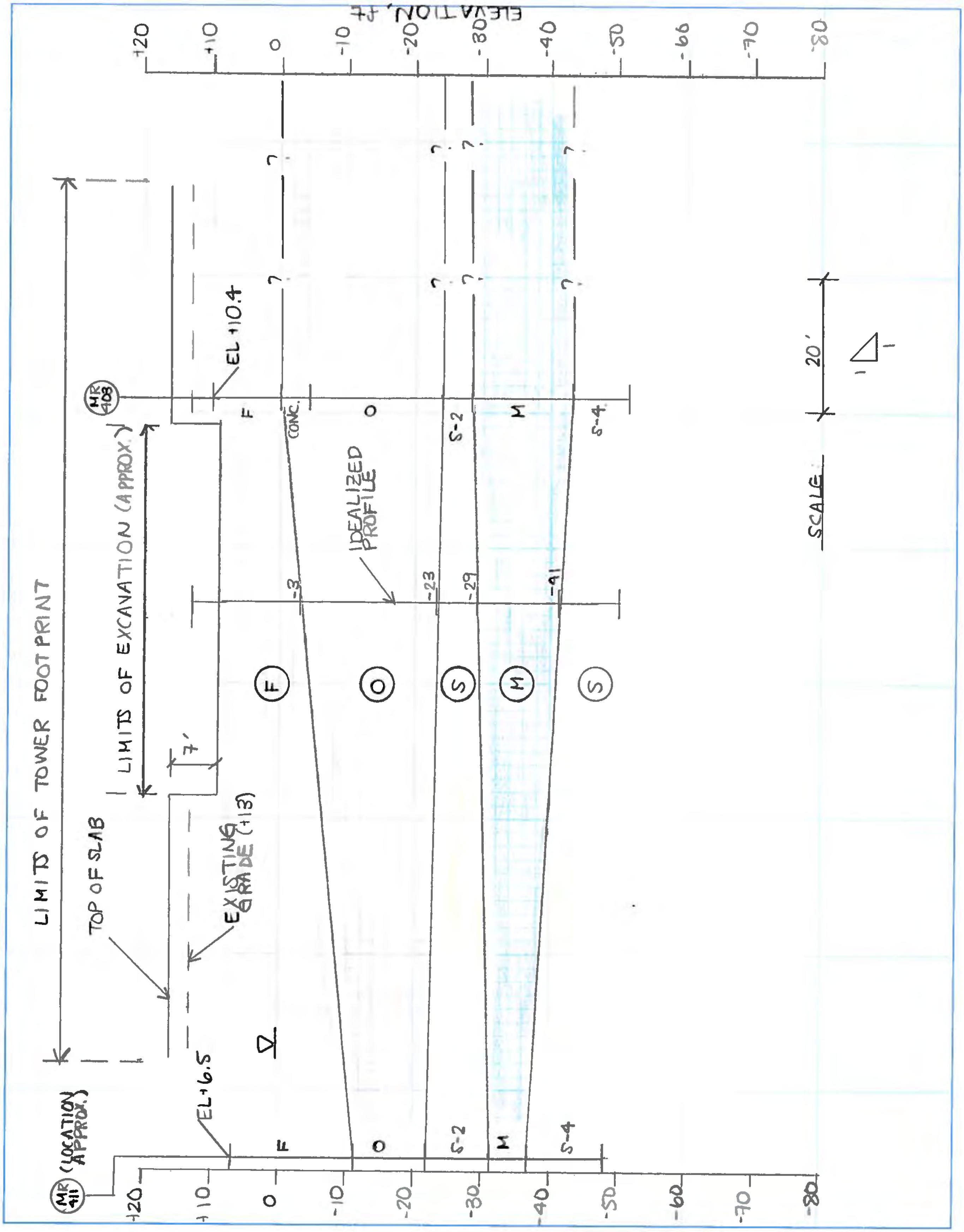
FILE 11896A

PROJECT EXELON

MADE BY AEP DATE 6/8/13

CHECKED BY AMD DATE 6/27/13

SUBJECT PRELIMINARY SETTLEMENT ESTIMATE - SECTION 2-2



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SHEET ____ OF ____

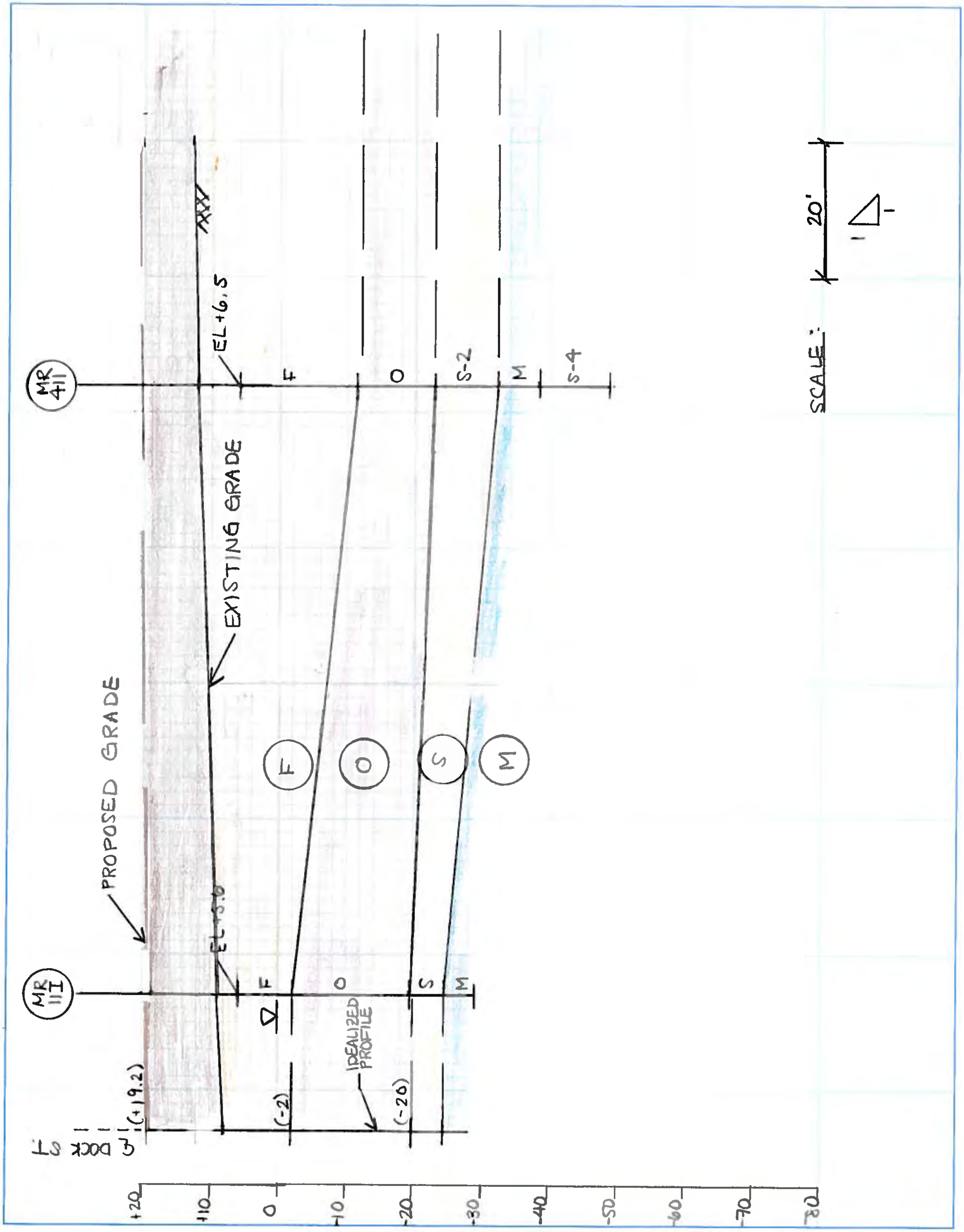
FILE AEP

MADE BY AEP DATE 6/16/13

CHECKED BY AMD DATE 6/27/13

PROJECT EXELON

SUBJECT PRELIMINARY SETTLEMENT ESTIMATE - SECTION 3-3



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SHEET ___ OF ___

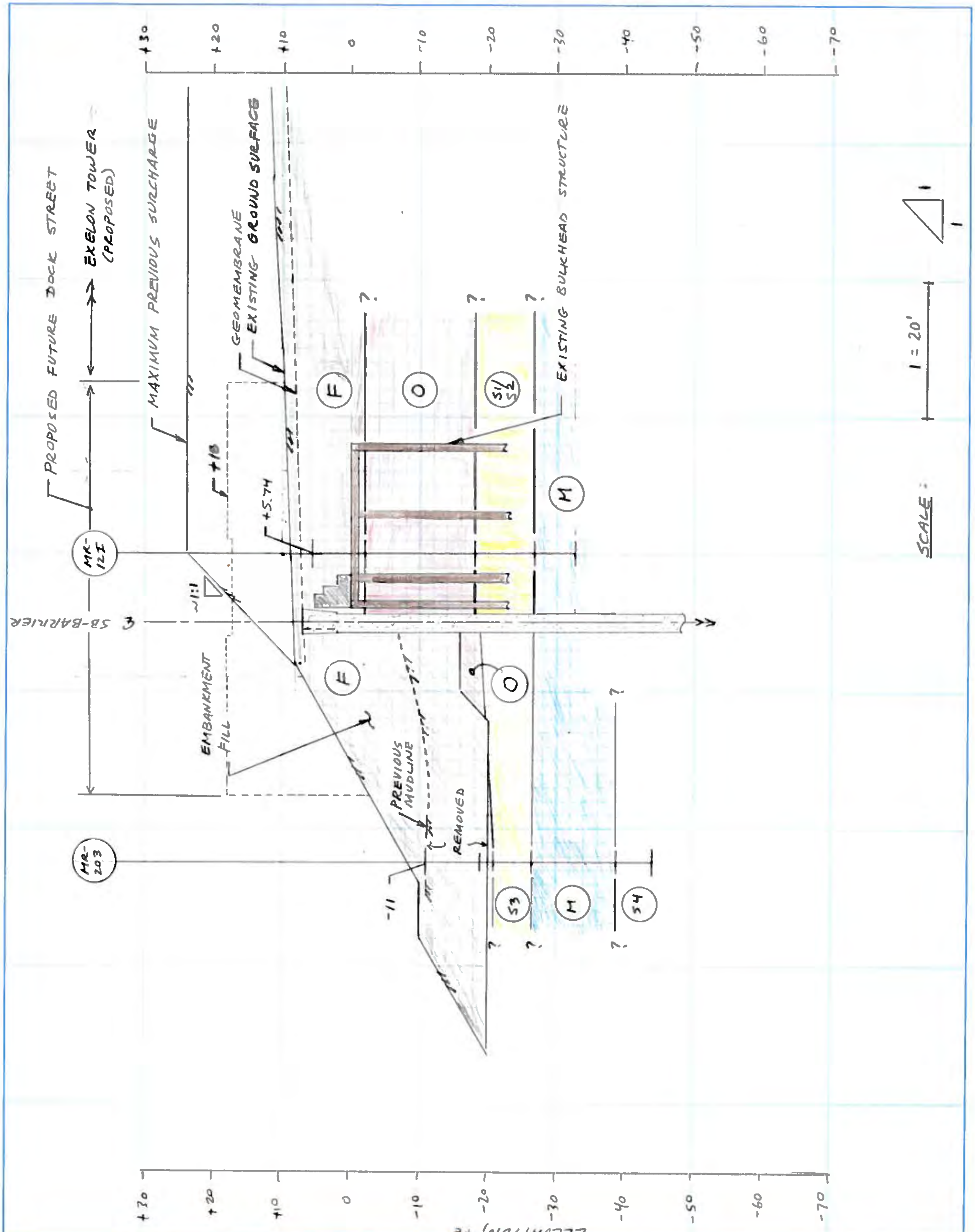
FILE 11896A

PROJECT EXELON TOWER & TF GARAGE

MADE BY AMD DATE 7/3/13

CHECKED BY AEP DATE 7/3/13

SUBJECT SECTION 4-4 ESTIMATE OF SETTLEMENT AT DOCK ST. E/o POINT ST



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FILE 11896A

PROJECT EXELON TOWER + TF GARAGE

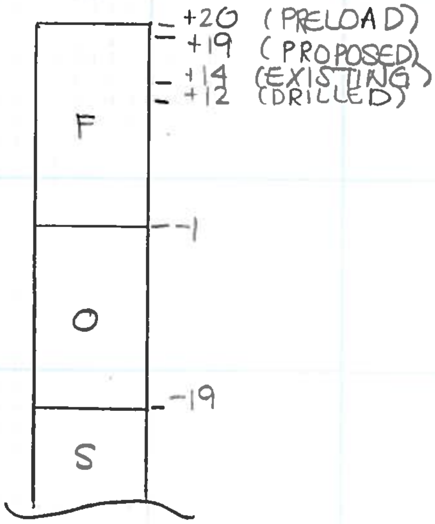
MADE BY AEP DATE 8/8/13

CHECKED BY LL DATE 8/8/13

SUBJECT SECONDARY COMPRESSION UNDER WILLS ST.

- COMPUTE MAGNITUDE OF SECONDARY COMPRESSION UNDER WILLS ST., AT LOCATION WHERE THE APPLIED LOAD ON MMC DUE TO FILL LOAD IS THE LARGEST.
- USE BORING NO. MR-801, LOCATED DIRECTLY ADJACENT TO LOCATION OF CALCULATION AND WAS DRILLED AFTER SURCHARGING (CAPTURES STRESS HISTORY)

SOIL PROFILE: MR-801



RESULTS OF CONSOLIDATION TEST

$P_c = 20$ TSF $\quad e_o = 1.798$

$P_o = 1.0$ TSF $\quad w_n = 60.5\%$

$C_c = 0.801 \quad C_\alpha = 0.032$

$C_s = 0.136 \quad e_p = 1.708$

COMPUTATION OF SECONDARY COMP

$$S_s = C_\alpha' H \log(t_2/t_1)$$

$$C_\alpha' = \frac{C_\alpha}{1+e_p} = \frac{0.032}{1+1.708} = 0.012$$

$H = 19$ FT

$t_1 \equiv t_p$

$$t_p = \frac{H_{DR}^2 T_V}{C_v}$$

$H_{DR} = H/2 = 19'/2 = 9.5'$

$T_V = 0.848$ @ $U = 90\%$

$C_v = 0.02$ FT²/DAY

$\Rightarrow t_p = 3827$ DAYS $= 10.5$ YRS

t_2 = TIME AFTER END OF PRIMARY

S_s @ $t_2 = 15$ YRS AFTER END OF PRIMARY

$$S_s = (0.012)(19')\log\left(\frac{15+10.5}{10.5}\right)$$

$S_s = 0.09$ FT $= 1.05$ IN.

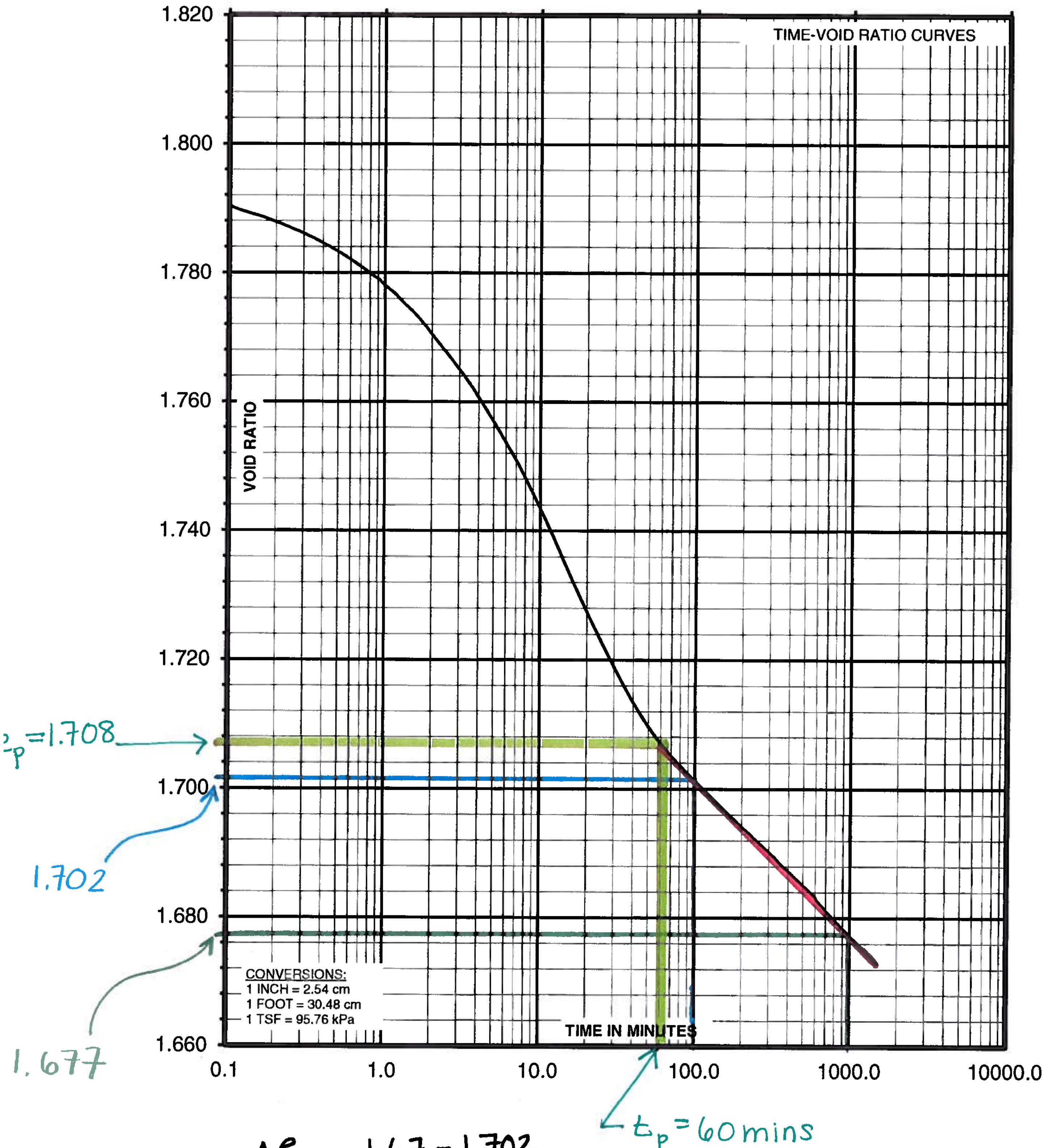
S_s @ $t_2 = 35$ YRS AFTER END OF PRIMARY

$$S_s = (0.012)(19')\log\left(\frac{35+10.5}{10.5}\right)$$

$= 0.15$ FT $= 1.7$ IN.

ATTACHMENT

VOID RATIO-TIME CURVE FOR MR-801



$$C_\alpha = \frac{\Delta e}{\Delta t} = \frac{1.67 - 1.702}{\log(1000) - \log(100)}$$

$$= 0.032$$

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MEMORANDUM

Date: August 8, 2013
To: Office
From: Alexandra Patrone
Re: EE Memo 2 – Storm Water Storage Demand
Exelon Building & Plaza Garage, Baltimore, MD
File: 11896A-40

This memorandum summarizes analyses of storm water management for exposed areas of the cap as a result of foundation construction during a 25-year and 100-year storm event. The purpose of this analysis is to estimate the quantity of "impacted" water that comes in contact with soil below the membrane, which must be stored for analytical testing. It was also necessary to determine the required pumping rate of impacted water for a 25-year and 100-year frequency storm event. Storm water must be removed from excavations to prevent storm water from within an excavation from rising above the lowest membrane level in the excavation.

Attachments
We have attached the following to illustrate our analyses:

Figure 1 Open Excavation Areas
Figure 2 Rainfall – Intensity – Duration – Frequency Curves for Baltimore, MD (McCuen, 2004)

Appendix A Computation of Pile Cap Excavation Areas
Appendix B Required Storage and Pumping Rates Calculation
Appendix C Containment Berm Design

References:
1. "Hydrologic Analysis and Design" by R.H. McCuen, 2004.
2. "Urban Hydrology for Small Watersheds TR-55", United States Department of Agriculture, Natural Resources Conservation Service (1986).

Design Rain Events
Two design rain events were provided as the foundation for this analysis. The 25-year storm event is described as having a total precipitation of 5.5 inches over 24 hours, and the 100-year storm event is described as having a total precipitation of 7.1 inches over 24 hours (TR-55, 1986). The amount of impacted water was estimated for both storm events in terms of total gallons accumulated over a 24-hour period to determine the amount of on-site storage necessary for each design storm. The critical rainfall intensity is 3.0 in/hr. and 3.9 in/hr. for a 25-year and 100-year frequency storm event, respectively, and occurs during 1-hour duration (McCuen, 2004). The required pumping rates were determined based on this rainfall intensity.

Proposed Storm Water Management System

The storm water management plan was examined for the 25-year storm event and 100-year storm event. When a storm event occurs, the only water that will come in contact with soil below the membrane will be storm water falling directly into an excavation. All water that falls outside of the excavations is treated as surface runoff and will be deflected away from open excavations by diversion berms. Infiltration through the cover soil into the drainage net was not considered because the drainage net is dammed at the edge of each excavation. The bottom of each excavation is open to soil below membrane, so that any storm water collected in the excavation may be impacted.

Each excavation will be sloped and a sump will be installed to collect storm water to prevent it from rising above the capillary break gravel at the down-slope side of the excavation. The entire footprint of the excavation, including the sloped portions, was considered to catch storm water in the excavation. Contact and non-contact water testing and proper disposal procedures are described in the Material Handling and Management Plan.

Analysis Method and Assumptions

As previously described, it was assumed that the only source of contaminated water during a storm event will come from direct catchment of storm water in open excavations.

The area of an average pile cap excavation of 310 ft^2 was examined to determine the total volume of impacted water generated over a 24 hour period for both a 25-year and 100-year storm event. The total volume of impacted water produced over 24 hours is 1,064 gal/day and 1,374 gal/day for a 25-year and 100-year frequency storm event, respectively. The maximum required pumping rate was also computed for a rainfall intensity of 3.9 in./hr., corresponding to a one-hour duration 100-year frequency storm event. The maximum required pumping rate in a typical excavation is 755 gal/hr. or 13 gal/min.

The shear wall foundation excavation was also analyzed for total impacted water volume produced per day and maximum required pumping rate in the event of a one hour duration 100-year frequency storm event. The total volume of impacted water generated per day in the largest shear wall foundation excavation is 52,633 gal/day and 69,236 gal/day for a 25-year and 100-year frequency storm event, respectively. The maximum required pumping rate for a 3.9in/hr. intensity one hour duration storm is 38,031 gal/hr. or 634 gal/min.

The amount of on-site storage required was determined by examining a likely excavation scenario. In order to maintain the project schedule, it will be necessary to keep a large number of excavations open at one time. It was assumed that at the time of a storm event, all excavations below the tower footprint and half of all excavations below the trading floor garage footprint will be open, creating an open excavation area of 32,244 ft^2. The total volume of impacted water generated during a 24-hour 25-year and 100-year frequency storm event is 107,129 gal/day and 138,294 gal/day, respectively. Based on this volume of impacted water, two 75’ x 75’ x 4’ ModuTank storage containers were selected to store the impacted water. A single 75’ x 75’ tank can store 168,323 gallons of water. One tank will store the first 24 hours of rainfall and a second tank will store a second day while treatment and disposal are performed for the first tank.

Given this excavation scenario that leaves 32,244 ft^2 of excavation area open during a storm event, it was necessary to determine the pumping rate required across the site in the event of a 1-hour duration

3.9in/hr. intensity storm. If a storm with these parameters were to pass over the site, the pumping requirement is 75,964 gal/hr. or 1,266 gal/min. It should be noted that this required pumping rate can be easily reduced by limiting the excavation area open at the onset of a 100-year storm event.

In the event of a 100-year storm, 7.1" of rain will fall directly into both tanks. The empty tanks each have a capacity of 22,500 ft^3, but when the depth of storm water falling into the tanks is accounted for, the "effective" storage capacity of one tank is 19, 172 ft^3. The total volume of impacted water generated at the site during a 100-year storm event is 18,486 ft^3 based on the excavation scenario described. This leaves a "reserve" capacity of 686 ft^3 in one tank, or 1.4" of additional rainfall that can occur before one tank is filled to capacity. The second tank will have 19,172 ft^3 of filling capacity available for treatment and disposal.

If the 100-year storm lasts two days, two 75'x75'x4' ModuTanks will not have enough capacity to store the amount of impacted water generated from the open excavations and the amount of rain water falling directly into the two tanks. If the storm were to last two days, the total excavation area that can be left open without exceeding the capacity of the two ModuTanks is 26,777 ft^2. These calculations can be found in Appendix B.

If we assume that two 75'x75'x4' ModuTank storage containers will be present on-site during a 100-year storm event, the allowable open excavation area based on this storage volume is 38,008 ft^2.

A containment berm was designed to handle the volume of one storage tank in the event that one of the ModuTanks fails. The total volume that the containment berm will need to hold is the volume of one ModuTank, or 22,500 ft^3, and the volume of rain water falling into the containment berm during a 100-year storm event. Considering this scenario, a 120'x208'x22" containment berm will be sufficient to handle the total volume of water from the failed tank, the volume of water falling directly into the berm during the storm, and maintain a 4" reserve capacity. Details of this calculation can be found in Appendix C.

<u>Discussion</u>

Water volume collected can be managed by restricting the number of pile caps open at one time. Inactive excavations can be lined with 6 mil plastic to prevent contact of water with the underlying soil. The large volume storage containers recommended will allow collection of storm water for 2-1/2 days of a 100-year storm event. However, disposal of this water will require several days if the water cannot be discharged to the sanitary sewer. All efforts should be made to prevent contact of storm water with contamination below the geomembrane, including additional site housekeeping measures in advance of forecasted storm events.

The total required pumping rate for the excavation scenario described of 75,964 gallons per hour can be accommodated by an array of multiple pumps with hose connections into each excavation. Pumps similarly sized can be used to control storm water in smaller pile cap excavations. As with the amount of impacted water generated, the required pumping rate can be managed by restricting the number of pile cap excavations open at one time.

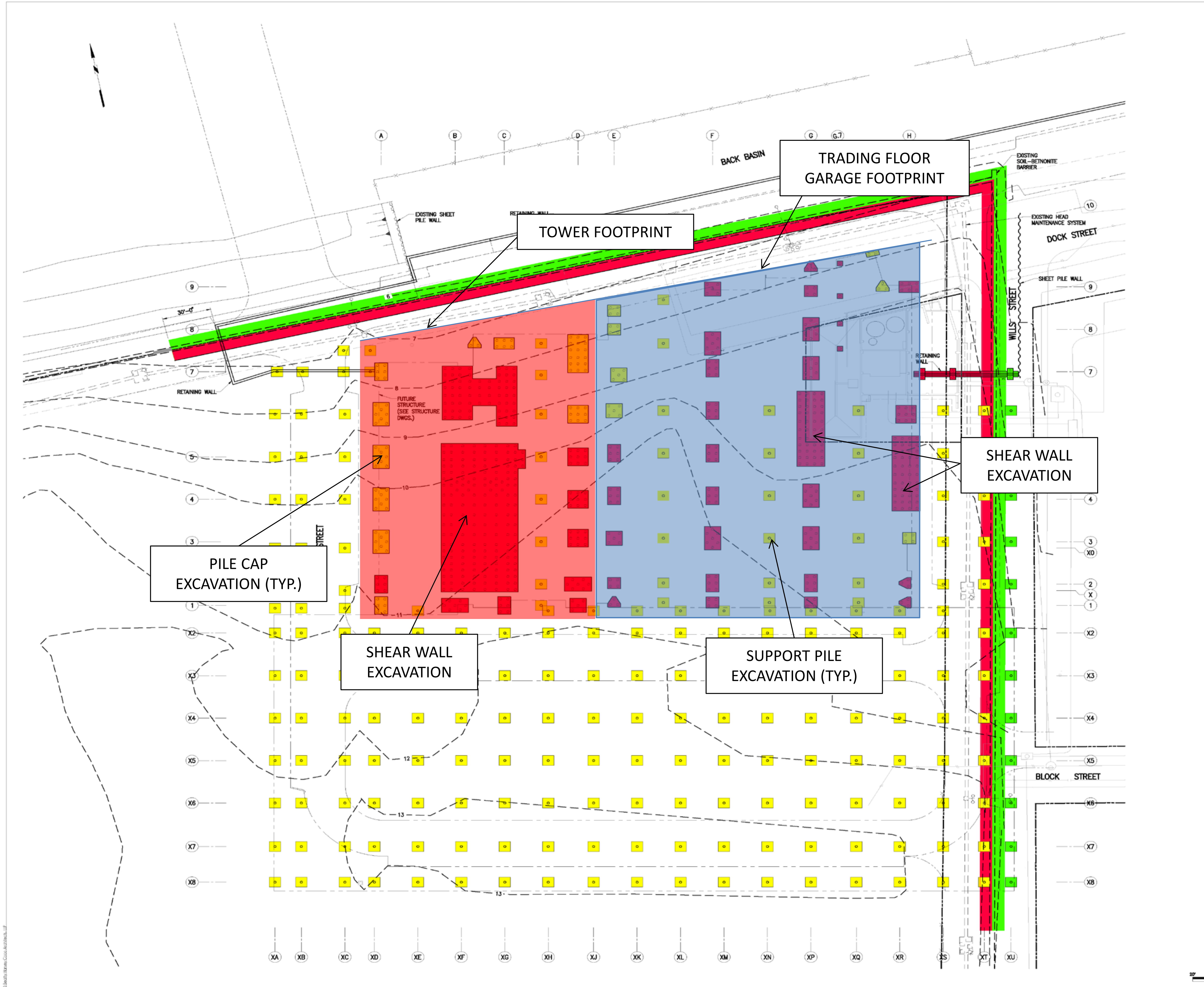
The pumping rate required for an individual pile cap, 13 gal/min, is easily accommodated with a submersible electric construction sump pump. The pumping rate required for the large shear wall excavation, if the entire area is open during the storm is on the order of 650 gal/min. An organized pattern of storm water collection swales and sumps with large construction pumps will be needed. These pumping rates assume there is no infiltration into the ground at pile cap subgrade. Infiltration will be handled by the HMS system after some time lag to account for groundwater flow to the piezometer and pump locations.

Because water collected is potentially impacted by contact with the bottom of the excavation, the water conveyance pipes must be double walled from the pump location to the storage tanks. Leakage water collected in the containment pipe should discharge at the pump location where it will be removed for discharge to the storage tank.

a. Patrone

By: ____________________
Alexandra E. Patrone

AEP:AMD: PWD\11896A-40\Storm Water Storage Demand



NOTES:

1. Demolish and remove obstructions and materials to obtain sufficient depth for pile cap and MMC below pile cap.
2. Obstructions may be encountered at any pile location.
3. Where obstruction demolition or removal is not feasible (as approved by Owner), relocate piles to avoid obstruction. Owner will re-design pile cap to support columns.
4. Demolish concrete obstructions and remove timber piles necessary to place new pile at designated location. (concentric pile: Plaza Garage, Point St. Deck, Wills St. Deck)
5. See Materials Management Plan.
6. See Location B on Drawing F1.30.

LEGEND:

- Column location as per BHC
- Concentric Pile
- Potential excavation
- Concentric Pile (Plaza Garage, Point St. Deck, Wills St. Deck)
- Remove spoils to create volume for foundation / pile cap
- Proposed Retaining Wall

BHC ARCHITECTS

650 S. Exeter Street, Suite 200, Baltimore, MD 21202
Phone 410 752 2759

owner/developer

developer:
Beatty Development Group

consultants

design consultant:
Elkus Manfredi Architects

structural engineer:
Tadjer Cohen Edelson & Associates

mep & fp engineer:
Vanderweil

foundation engineers:
Mueser Rutledge Consulting Engineers

civil engineers:
Rummel, Klepper & Kahl

landscape architect:
Mahan Rykiel Associates

environmental engineer:
Environmental Resources Management

project

EXELON BLDG & PLAZA GARAGE

HARBOR POINT
AREA 1 PHASE 1
DDP SUBMISSION
7/1/13

FOUNDATION EXCAVATION AREAS PLAN

DDP
F1.07

ATTACHMENT
FIGURE NO. 2 : RAINFALL - INTENSITY - DURATION - FREQUENCY CURVES FOR BALTIMORE, MD. (McCUEN, 2004)

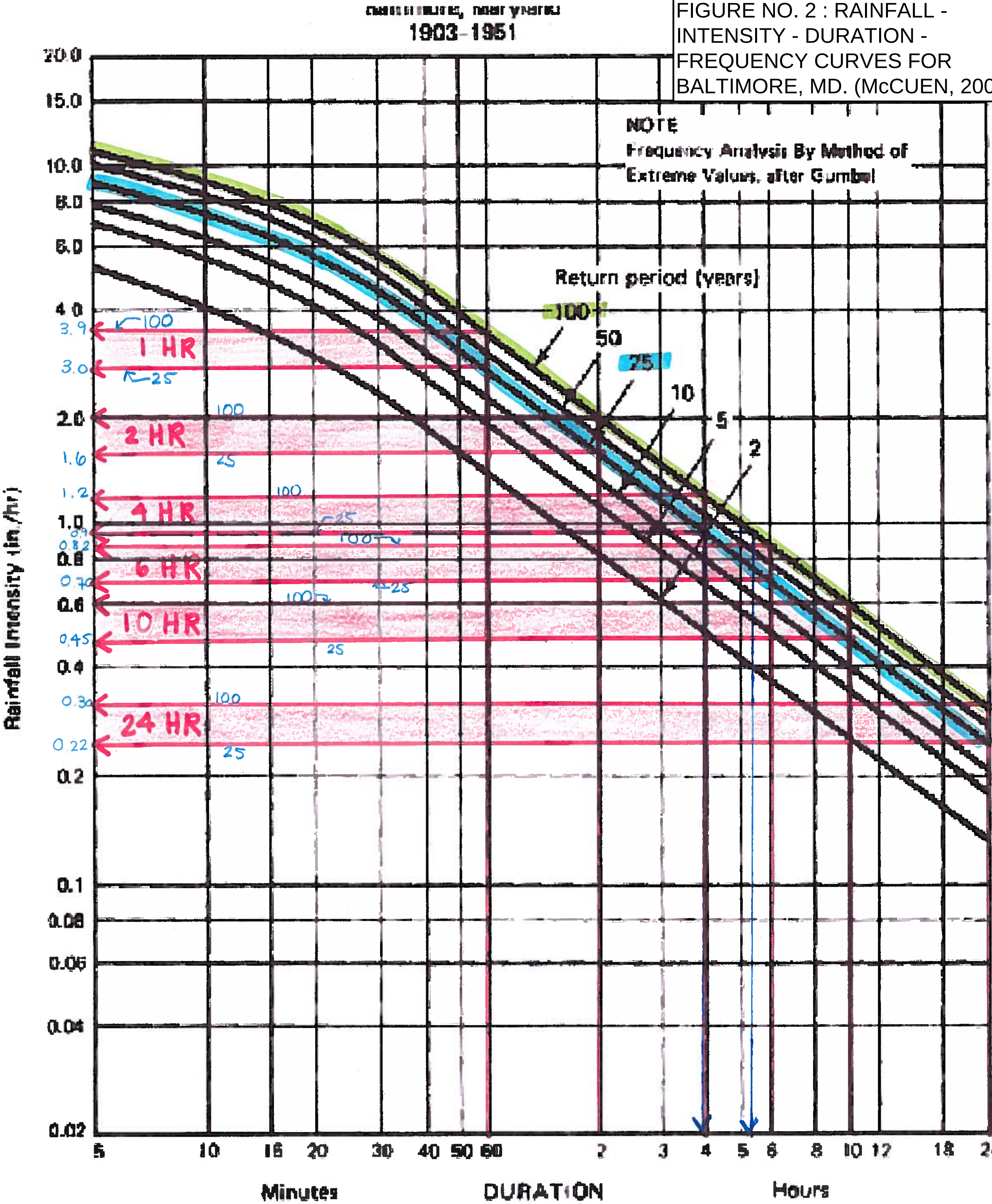


FIGURE 4–4 Rainfall intensity-duration-frequency. (National Weather Service, 1961.)

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File No.: 11896A
Made by: AEP Date: 6/26/13
FOR Exelon
Checked by: Date:

SUBJECT: Stormwater Management Appendix A - Computation of Pile Cap Excavation Areas

AREA	Pile Cap	# of Piles	Top of Slab Elev.	Slab Thickness (feet)	Pile Cap Depth (feet)	Bottom of Pile Cap Elev.	Bottom of Excav. (1.5 ft below Pile Cap)	MMC Elev.	Depth of Excav. Below MMC	Distance from Pile Cap to Excav. Edge (FT)	Orig. CAD info	Length of Pile Cap (ft)	With of Pile Cap (ft)	Excavation Length at Ground Surface (ft)	Excav. Width at G.S. (ft)	Excav. area (ft^2)
	A-7	6	17.0	0.0	5.25	11.8	10.3	7.75	0.0	2.0	A-7	12.5	8	17	12	198
	A-6	7	17.0	0.0	5	12.0	10.5	8.5	0.0	2.0	A-6	16.5	10	21	14	287
	A-5	7	17.0	0.0	5	12.0	10.5	9.4	0.0	2.0	A-5	16.5	10	21	14	287
	A-4	7	17.0	0.0	5	12.0	10.5	10.1	0.0	2.0	A-4	16.5	10	21	14	287
	A-3	7	17.0	0.0	5	12.0	10.5	10.5	0.0	2.0	A-3	16.5	10	21	14	287
	A-2	6	17.0	0.0	5.25	11.8	10.3	10.8	0.6	2.8	A-2	12.5	8	18	13.65	248
	A-1	5	17.0	0.0	4.25	12.8	11.3	11	0.0	2.0	A-1	10	10	14	14	196
	B-1	6	17.0	0.0	5.25	11.8	10.3	11.2	0.9	3.4	B-1	12.5	8	19	14.85	287
	B-2		17.0	0.0	#N/A	#N/A	#N/A	10.9	#N/A	#N/A	B-2	#N/A	#N/A	#N/A	#N/A	
	C-1	5	17.0	0.0	4.25	12.8	11.3	11.5	0.3	2.4	C-1	10	10	15	14.75	218
	C-2		17.0	0.0	#N/A	#N/A	#N/A	11.3	#N/A	#N/A	C-2	#N/A	#N/A	#N/A	#N/A	
TOWER	C-5	7	17.0	0.0	5	12.0	10.5	9.9	0.0	2.0	C-5	16.5	10	21	14	287
	B.1-7		17.0	0.0	#N/A	#N/A	#N/A	7.3	#N/A	#N/A	B.1-7	#N/A	#N/A	#N/A	#N/A	
	C-7		16.0	0.0	#N/A	#N/A	#N/A	8.1	#N/A	#N/A	C-7	#N/A	#N/A	#N/A	#N/A	
	D-7.8	18	16.0	0.0	7	9.0	7.5	8	0.5	2.8	D-7.8	12.5	12.5	18	18	324
	D-6	9	17.0	0.0	6	11.0	9.5	9.4	0.0	2.0	D-6	12.5	12.5	17	16.5	272
	D-5	9	17.0	0.0	6	11.0	9.5	10.3	0.8	3.2	D-5	12.5	12.5	19	18.9	357
	D-4	9	17.0	0.0	6	11.0	9.5	11	1.5	4.3	D-4	12.5	12.5	21	21	441
	D-3.1	9	17.0	0.0	6	11.0	9.5	11.3	1.8	4.7	D-3.1	12.5	12.5	22	21.9	480
	D-2	7	17.0	0.0	5	12.0	10.5	11.7	1.2	3.8	D-2	16.5	10	24	17.6	424
	D-1	5	17.0	0.0	4.25	12.8	11.3	11.8	0.6	2.8	D-1	10	10	16	15.65	245
	B/C-7.8	3	17.0	0.0	4	13.0	11.5	7.3	0.0	2.0	B/C-7.8	8	7.5	12	11.5	138
	C-7.8	6	16.0	0.0	5.25	10.8	9.3	7.5	0.0	2.0	C-7.8	12.5	8	17	12	198
	D-7	18	16.0	0.0	0	16.0	14.5	8.4	0.0	2.0	D-7	12.5	12.5	17	16.5	272
	E-7.1	5	16.0	0.0	4.25	11.8	10.3	8.8	0.0	2.0	E-7.1	10	10	14	14	196
	E-8	4	16.0	0.0	4	12.0	10.5	7.9	0.0	2.0	E-8	8	8	12	12	144
	E-10	3	16.0	0.0	4	12.0	10.5	7.6	0.0	2.0	E-10	8	7.5	12	11.5	138
	E-6.1	5	16.0	0.0	4.25	11.8	10.3	9.8	0.0	2.0	E-6.1	10	10	14	14	196
	E-5.1	6	16.0	0.0	5.25	10.8	9.3	10.8	1.6	4.3	E-5.1	12.5	8	21	16.65	352.148
	E-4.1	6	16.0	0.0	5.25	10.8	9.3	11.2	2.0	4.9	E-4.1	12.5	8	22	17.85	398.948
	E-3.1	5	16.0	0.0	4.25	11.8	10.3	11.4	1.2	3.7	E-3.1	10	10	17	17.45	304.503
	E-2.1	4	16.0	0.0	4	12.0	10.5	11.7	1.2	3.8	E-2.1	8	8	16	15.6	243.36
	E-1.2	3	16.0	0.0	4	12.0	10.5	11.8	1.3	4.0	E-1.2	8	7.5	16	15.4	244.86
	F-1.2	4	15.5	0.0	4	11.5	10.0	11.5	1.5	4.3	F-1.2	8	8	17	16.5	272.25
	F-2.1	6	15.5	0.0	5.25	10.3	8.8	11.4	2.7	6.0	F-2.1	12.5	8	24	19.95	487.778
	F-3.1	7	15.5	0.0	5	10.5	9.0	11.1	2.1	5.2	F-3.1	16.5	10	27	20.3	544.04
	F-4.1	6	15.5	0.0	5.25	10.3	8.8	10.8	2.1	5.1	F-4.1	12.5	8	23	18.15	411.098
	F-5.1	6	15.5	0.0	5.25	10.3	8.8	10.5	1.8	4.6	F-5.1	12.5	8	22	17.25	375.188
	F-6.1	6	13.00	0.0	5.25	7.8	6.3	9.9	3.7	7.5	F-6.1	12.5	8	27	22.95	629.978
	F-7.1	6	13.00	0.0	5.25	7.8	6.3	9.1	2.9	6.3	F-7.1	12.5	8	25	20.55	514.778
	F-7.8	7	13.00	0.0	5	8.0	6.5	8.7	2.2	5.3	F-7.8	16.5	10	27	20.6	558.26
	F-8		13.00	0.0	#N/A	#N/A	#N/A	8.5	#N/A	#N/A		#N/A	#N/A	#N/A	#N/A	
TRADING FLOOR GARAGE	F-10	5	13.0	0.0	4.25	8.8	7.3	7.7	0.5	2.7	F-10	10	10	15	15.35	235.623
	G-10	3	13.0	0.0	4	9.0	7.5	7.8	0.3	2.5	G-10	8	7.5	13	12.4	159.96
	G-8.9	4	13.0	0.0	4	9.0	7.5	8.3	0.8	3.2	G-8.9	8	8	14	14.4	207.36
	G-8	7	13.0	0.0	5	8.0	6.5	9	2.5	5.8	G-8	16.5	10	28	21.5	602
	G-7.1	7	13.0	0.0	5	8.0	6.5	9.3	2.8	6.2	G-7.1	16.5	10	29	22.4	647.36
	G-6.1		16.0	0.0	#N/A	#N/A	#N/A	9.6	#N/A	#N/A		#N/A	#N/A	#N/A	#N/A	
	G-5.1		16.0	0.0	#N/A	#N/A	#N/A	9.8	#N/A	#N/A		#N/A	#N/A	#N/A	#N/A	
	G-4.1	7	15.5	0.0	5	10.5	9.0	10.2	1.2	3.8	G-4.1	16.5	10	24	17.6	424.16
	G-3.1	7	15.5	0.0	5	10.5	9.0	10.5	1.5	4.3	G-3.1	16.5	10	25	18.5	462.5
	G-2.1	6	15.5	0.0	5.25	10.3	8.8	10.8	2.1	5.1	G-2.1	12.5	8	23	18.15	411.098
	G-1.2	4	15.5	0.0	4	11.5	10.0	11	1.0	3.5	G-1.2	8	8	15	15	225
	G.9-1.2	3	16.0	0.0	4	12.0	10.5	10.7	0.2	2.3	G.9-1.2	8	7.5	13	12.1	152.46
	G.9-2.1	3	16.0	0.0	4	12.0	10.5	10.6	0.1	2.2	G.9-2.1	8	7.5	12	11.8	145.14
	G.9-3.1	3	16.0	0.0	4	12.0	10.5	10.5	0.0	2.0	G.9-3.1	8	7.5	12	11.5	138
	G.9-4.1		16.0	0.0	#N/A	#N/A	#N/A	10.2	#N/A	#N/A		#N/A	#N/A	#N/A	#N/A	
	G.9-5.1		16.0	0.0	#N/A	#N/A	#N/A	10	#N/A	#N/A		#N/A	#N/A	#N/A	#N/A	
	G.9-6.0	9	15.5	0.0	6	9.5	8.0	9.7	1.7	4.6	G.9-6.0	12.5	12.5	21.6	21.6	466.56
	G.7-9	3	15.5	0.0	4	11.5	10.0	8.6	0.0	2.0	G.7-9	8	7.5	12	11.5	138
	G.9-9	6	15.5	0.0	5.25	10.3	8.8	8.8	0.1	2.1	G.9-9	12.5	8	16.65	12.15	202.298
	G.7-10	2	15.5	0.0	4	11.5	10.0	7.9	0.0	2.0	G.7-10	8	3.5	12	7.5	90

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File No.: 11896A
Made by: AEP Date: 7/31/13
FOR Exelon Checked by: AMD Date: 7/31/13

SUBJECT: Stormwater Management — Appendix B - Required Storage and Pumping Rates

ASSUMPTIONS:

1. ALL EXCAVATIONS WITHIN THE TOWER FOOTPRINT AND HALF OF ALL EXCAVATIONS WITHIN THE TRADING FLOOR GARAGE WILL BE OPEN AT ONE TIME.
2. BECAUSE THE DRAINAGE NET WILL BE DAMMED AT EACH EXCAVATION, THE ONLY SOURCE OF IMPACTED STORM WATER GENERATED DURING A 25-YEAR AND 100-YEAR FREQUENCY STORM EVENT WILL BE FROM RAIN WATER FALLING DIRECTLY INTO OPEN EXCAVATIONS (DIRECT CATCHMENT).

NOTES:

1. THE TOTAL RAINFALL FOR A 25-YEAR AND 100-YEAR STORM ARE AS FOLLOWS:

STORM FREQUENCY	TOTAL RAINFALL OVER 24 HOURS (IN.)
25-YEAR	5.5
100-YEAR	7.1

REFERENCES:

1. "HYDROLOGIC ANALYSIS AND DESIGN" BY R.H. McCUEN, 2004
2. "URBAN HYDROLOGY FOR SMALL WATERSHEDS" TR-55, UNITED STATES DEPARTMENT OF AGRICULTURE, NATURAL RESOURCES CONSERVATION SERVICE, 1986

ATTACHMENTS:

1. FIGURE NO. 1: SCENARIO - TOWER FOOTPRINT AND TRADING FLOOR GARAGE FOOTPRINT EXCAVATIONS
2. FIGURE NO. 2: RAINFALL-INTENSITY-DURATION-FREQUENCY CURVE FOR BALTIMORE, MD

TYPICAL EXCAVATION AREAS

NOTES:

1. EXAMINE THE VOLUME OF WATER GENERATED FROM AN AVERAGE PILE CAP EXCAVATION AND FROM THE LARGEST EXCAVATION AREA, THE EXCAVATION TO CONSTRUCTION THE SHEAR WALL FOUNDATIONS.

Volume of Impacted Water Generated from a Single Pile Cap Excavation

Average Pile Cap Area	310	ft^2
Volume Generated per day:		
25-year storm	1,064	gal/day
100-year storm	1,374	gal/day

BASED ON FIGURE NO. 2, THE CRITICAL RAINFALL INTENSITY IS 3.0 IN/HR AND 3.9 IN/HR FOR A 25-YEAR AND 100-YEAR STORM, RESPECTIVELY. THIS INTENSITY WILL OCCUR OVER A STORM DURATION OF 1 HOUR. THIS CRITICAL INTENSITY WILL DETERMINE THE REQUIRED PUMPING RATE AT EACH EXCAVATION.

REQUIRED PUMPING RATE FOR A SINGLE PILE CAP EXCAVATION - CRITICAL RAINFALL INTENSIT

AVERAGE PILE CAP AREA	310	ft^2
RAINFALL INTENSITY	3.9	IN/HR
	0.325	FT/HR
	100.9	FT^3/HR
REQUIRED PUMPING RATE	755	GAL/HR
	13	GAL/MIN.

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Made by:	AEP	Date:	7/31/13
Checked by:	AMD	Date:	7/31/13

FOR Exelon

SUBJECT: Stormwater Management — Appendix B - Required Storage and Pumping Rates

Volume of Impacted Water Generated from Largest Excavation (Shear Wall Foundation)

Shear Wall Excavation Area	15642	ft^2
Volume Generated per day:		
25-year storm	53,633	gal/day
100-year storm	69,236	gal/day

REQUIRED PUMPING RATE FOR SHEAR WALL EXC. - CRITICAL RAINFALL INTENSITY

SHEAR WALL EXC. AREA	15642	ft^2
RAINFALL INTENSITY	3.9	IN/HR
	0.325	FT/HR
	5083.7	FT^3/HR
REQUIRED PUMPING RATE	38,031	GAL/HR
	634	GAL/MIN.

STORAGE REQUIRED FOR LIKELY EXCAVATION SCENARIO

IF WE EXAMINE AN EXCAVATION SCENARIO IN WHICH ALL EXCAVATIONS WITHIN THE TOWER FOOTPRINT AND HALF OF ALL EXCAVATIONS WITHIN THE TRADING FLOOR GARAGE FOOTPRINT (SEE FIGURE NO. 1), WE CAN DETERMINE THE AMOUNT OF ON-SITE STORAGE IS REQUIRED TO HANDLE THE VOLUME OF IMPACTED WATER GENERATED DURING 24-HOURS.

OPEN EXCAVATION AREAS:

	Area (ft^2)
Tower Footprint:	
Shear Wall Excavation	15642
Pile Caps	5733
Support Piles	754
TOTAL	22129
Trading Floor Garage (entire footprint):	
Shear Wall Excavations	5499
Pile Caps	10719
Support Piles	2011
TOTAL	18229
HALF FOOTPRINT	9115
Total Open Excavation Area:	$\boxed{31,244}$ ft^2

TOTAL VOLUME OF IMPACTED WATER GENERATED OVER A 24-HOUR PERIOD

NOTES:

1. THE TOTAL VOLUME OF IMPACTED WATER GENERATED IS THE PRODUCT OF THE OPEN EXCAVATION AREA AND THE TOTAL RAINFALL OVER 24 HOURS

STORM FREQUENCY	EXC. AREA [FT^2]	TOTAL PRECIP. [IN.]	TOTAL PRECIP. [FT.]	IMPACTED VOLUME [FT^3]	IMPACTED VOLUME [GAL]*
25-YEAR	31,244	5.5	0.46	14,320	107,129
100-YEAR	31,244	7.1	0.59	18,486	138,294

*1 FT^3 = 7.481 GAL

ON-SITE STORAGE REQUIRED

NOTES:

1. 75 FT X 75FT X 4 FT STORAGE CONTAINERS WILL BE USED ON SITE TO STORE AND TREAT IMPACTED WATER

VOLUME OF A SINGLE STORAGE CONTAINER:

LENGTH	75	FT
WIDTH	75	FT
HEIGHT	4	FT
$V_{STORAGE}$	22,500	FT^3
	168,323	GAL

$V_{STORAGE} > V_{IMPACTED}$ FOR BOTH 25-YEAR AND 100-YEAR STORM EVENTS

NUMBER OF 75' X 75' STORAGE CONTAINERS REQUIRED FOR 24 HOURS OF COLLECTION

STORM FREQUENCY	REQUIRED QUANTITY
25-YEAR	0.6
100-YEAR	0.8

PUMPING RATE REQUIRED FOR GIVEN EXCAVATION SCENARIO

WE CAN DETERMINE THE REQUIRED PUMPING RATE FOR THE AMOUNT OF OPEN EXCAVATION AREA IN THE GIVEN SCENARIO.

OPEN EXCAVATION AREA	31244	ft^2
RAINFALL INTENSITY	3.9	IN/HR
	0.325	FT/HR
	10154.3	FT^3/HR
REQUIRED PUMPING RATE	75,964	GAL/HR
	1,266	GAL/MIN

THE REQUIRED PUMPING RATE CAN BE REDUCED BY REDUCING THE EXCAVATION AREA OPEN AT THE ONSET OF THE STORM.

ALLOWABLE OPEN EXCAVATION AREA BASED ON ON-SITE STORAGE

IF TWO 75'X75'X4' STORAGE TANKS ARE PRESENT ON THE SITE TO HANDLE IMPACTED WATER, THE MAXIMUM AMOUNT OF OPEN EXCAVATION AREA THAT IS ACCEPTABLE DURING A 100-YEAR STORM IS

ALLOWABLE STORAGE	22,500	FT^3
TOTAL RAINFALL IN 24 HRS	7.1	IN/24 HR
	0.59	FT/HR
ALLOWABLE OPEN AREA	38,028	FT^2

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FILE 11896A

PROJECT EXELON TOWER + TF GARAGE

MADE BY AEP DATE 8/7/13

CHECKED BY GS DATE 8/8/13

SUBJECT CHECK MODUTANK CAPACITY FOR 2-DAY STORM.

ACCOUNT FOR DIRECT CATCHMENT INTO TANKS DURING 100-YEAR STORM

$(75' \times 75') \times (7.1''/12) = 3{,}328 \text{ FT}^3$

TOTAL STORAGE AVAILABLE IN ONE TANK DURING 100-YEAR STORM:

$(75' \times 75' \times 4') - 3328 \text{ FT}^3 = 19{,}172 \text{ FT}^3$

TOTAL VOLUME REQUIRED BASED ON SCENARIO DESCRIBED:

$V_{IMPACTED} = 18{,}486 \text{ FT}^3$

"RESERVE" VOLUME $= 19{,}172 \text{ FT}^3 - 18{,}486 \text{ FT}^3 = 686 \text{ FT}^3$

$$\frac{686 \text{ FT}^3}{(75' \times 75')} = 0.12 \text{ FT} = 1.4''$$

IN SECOND TANK, REMAINING STORAGE IS.

$(75' \times 75') \times (4' - 7.1''/12) = 19172 \text{ FT}^3$

TOTAL "RESERVE" VOLUME BETWEEN THE TWO TANKS IS

$19{,}172 \text{ FT}^3 + 686 \text{ FT}^3 = 19{,}858 \text{ FT}^3$

IF THE 100-YEAR STORM LASTS A SECOND DAY, AN ADDITIONAL $18{,}486 \text{ FT}^3$ WILL NEED TO BE STORED IN THE TWO TANKS.

$19{,}858 \text{ FT}^3 - 18{,}486 \text{ FT}^3 = 1{,}372 \text{ FT}^3$

$$\frac{1{,}372 \text{ FT}^3}{(75' \times 75')} = 0.24 \text{ FT} = 2.9''$$

THE TWO TANKS WOULD NEED TO HANDLE AN ADDITIONAL 14.2" OF DIRECT CATCHMENT RAINFALL

$2.9'' \ll 14.2''$

SUMMARY

- DURING A 24-HR 100-YEAR STORM, THE TWO 75' x 75' x 4' MODUTANKS CAN STORE ALL OF THE IMPACTED WATER FROM THE OPEN EXCAVATIONS (SCENARIO PREVIOUSLY DESCRIBED) AND HAVE A RESERVE CAPACITY OF $19{,}858 \text{ FT}^3$.
- IF THE STORM LASTS TWO FULL DAYS, THE MODUTANKS WILL NOT BE ABLE TO STORE TWO DAYS' WORTH OF IMPACTED WATER AND DIRECT CATCHMENT INTO THE TANKS.

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FILE 11896A

PROJECT EXELON TOWER + TF GARAGE

MADE BY AEP DATE 8/7/13

CHECKED BY GS DATE 8/8/13

SUBJECT CHECK MODUTANK CAPACITY FOR 2-DAY STORM

- IF THE STORM LASTS TWO FULL DAYS, HOW MANY FT^2 OF EXCAVATION AREA CAN BE LEFT OPEN?

  - VOLUME OF EMPTY TANKS $= 2 \times (75' \times 75' \times 4') = 45{,}000\ FT^3$

  - VOLUME OF DIRECT RAINFALL FALLING INTO TANKS OVER 48 HRS $= (2 \times 7.1''/24\ HRS) \times (2\ DAYS) = 28.4''$

    $\Rightarrow (75' \times 75') \times 2 \times \frac{14.2''}{12} = 13{,}312.5\ FT^3$

  - "EFFECTIVE" VOLUME IN TANKS AFTER 2 DAYS OF RAIN $= 45{,}000\ FT^3 - 13{,}312.5\ FT^3$

    $= 31{,}686\ FT^3$

  - IF IT RAINS 14.2" ACROSS THE SITE OVER TWO DAYS, THE MAXIMUM AMT OF OPEN EXCAVATION AREA $= \dfrac{31{,}686\ FT^3}{\left(\frac{14.2''}{12}\right)} = \underline{\underline{26{,}777\ FT^2}}$

    * COMPARE TO 31,244 FT^2 IN SCENARIO DESCRIBED.

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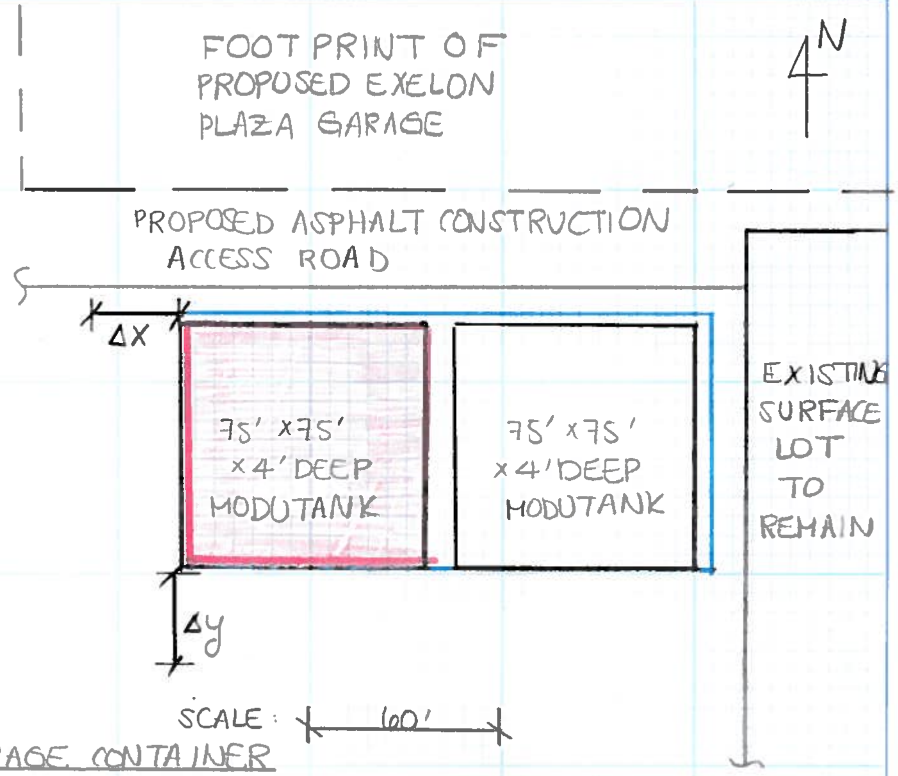
PROJECT EXELON TOWER + TF GARAGE

MADE BY AEP DATE 7/31/13
CHECKED BY GS DATE 8/06/13

SUBJECT CONTAINMENT BERM DESIGN

- DETERMINE EXTENTS AND VOLUME OF PROPOSED ASPHALT CONTAINMENT BERM
- BERM VOLUME MUST CONTAIN VOLUME OF ONE 75'×75'×4' MODUTANK IN THE EVENT OF A TANK FAILURE



VOLUME OF SINGLE STORAGE CONTAINER

$75' \times 75' \times 4' = 22{,}500\ FT^3$ ≡ VOLUME REQUIRED IN CONTAINMENT BERM

VOLUME OF PROPOSED CONTAINMENT BERM

NOTES:

(1) CONTAINMENT BERM IS CONSTRAINED BY PROPOSED ASPHALT CONSTRUCTION ACCESS ROAD TO THE NORTH AND THE EXISTING SURFACE LOT TO THE EAST.

(2) BERM HEIGHT OF 18" WAS ASSUMED. A 4" "SPILL HEIGHT" WAS DESIGNED FOR.

ASSUMPTIONS:

(1) CONTAINMENT BERM EXTENDS 2' BEYOND NORTHERN EDGE OF MODUTANKS AND 5' BEYOND EASTERN EDGE OF MODUTANKS.

(2) AVAILABLE HEIGHT OF BERM IS 14" TO ACCOUNT FOR 4" OF SPILL HEIGHT.

GIVEN THESE CONSTRAINTS, EXISTING BERM VOLUME IS:

$L = 75' + 10' + 5' = 90'$

$W = 2' + 75' = 77'$

$D = 14''/12 = 1.167'$

$V_{EX} = (90') \times (77') \times (1.167') = 8087\ FT^3$

$V_{REQ} = 22{,}500\ FT^3 \rightarrow$ ADD'L VOLUME $\equiv V_{AD} = 22{,}500 - 8087 = 14{,}413\ FT^3$

MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896A

PROJECT EXELON TOWER + TF GARAGE

MADE BY AEP DATE 7/31/13

CHECKED BY GS DATE 8/06/13

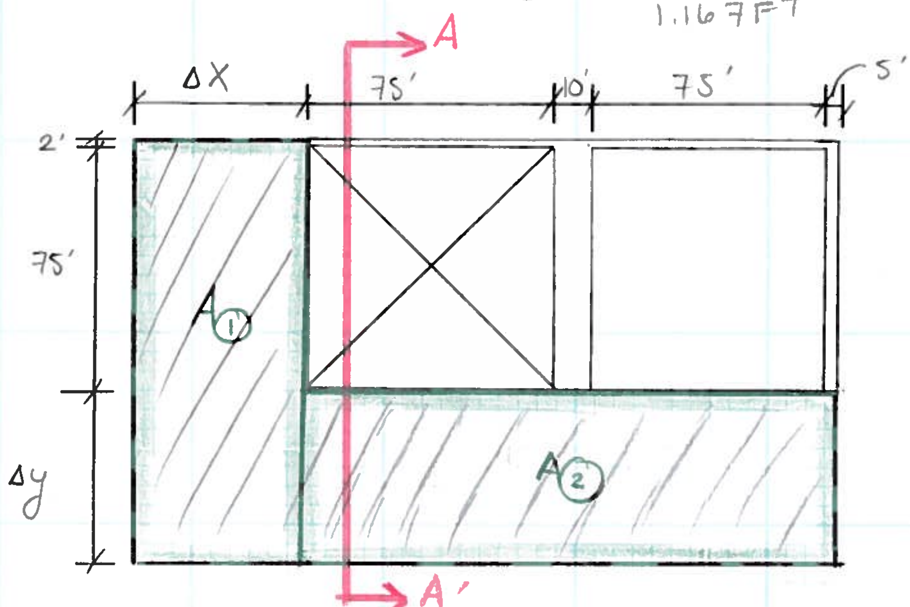
SUBJECT CONTAINMENT BERM DESIGN

ADDITIONAL VOLUME REQUIRED:

- DEPTH IS FIXED AT 14"

$\Rightarrow V_{AD} = 14{,}413 \text{ FT}^3$

ADDITIONAL AREA REQ'D $= A_{AD} = \dfrac{14{,}413 \text{ FT}^3}{1.167 \text{ FT}} = 12{,}351 \text{ FT}^2$



- DETERMINE VALUES OF ΔX AND ΔY THAT WILL PROVIDE $A_{AD} = 12{,}351 \text{ FT}^2$

- ASSUME $\Delta X = \Delta y$, THEREFORE

$$A_{1} = (\Delta X) \times (2' + 75' + \cancel{\Delta y}^{\,\Delta X}) = (\Delta X) \times (2' + 75' + \Delta X)$$

$$= 2\Delta X + 75\Delta X + \Delta X^2 = 77\Delta X + \Delta X^2$$

$$A_{2} = (\cancel{\Delta y}^{\,\Delta X}) \times (75' + 10' + 75' + 5') = \Delta X \times 165$$

AND, $A_{1} + A_{2} = A_{AD} = 12{,}351 \text{ FT}^2$

$$77\Delta X + \Delta X^2 + 165\Delta X = \Delta X^2 + 242\Delta X = 12{,}351 \text{ FT}^2$$

$$\Rightarrow \Delta X^2 + 242\Delta X - 12{,}351 \text{ FT}^2 = 0$$

$$\Delta X = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$a = 1$

$b = 242$

$c = -12{,}351$

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PROJECT EXELON TOWER + TF GARAGE

MADE BY AEP DATE 7/31/13
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SUBJECT CONTAINMENT BERM DESIGN

$$\Delta x = \frac{-242 \pm \sqrt{(242)^2 - 4(1)(-12,351)}}{2(1)}$$

$x = 43.3$

$x = -285.3$

$\therefore \Delta x = \Delta y = 43'$

Check: $A_{①} = (43') \times (2' + 75' + 43') = 5,160 \text{ FT}^2$

$A_{②} = (43') \times (75' + 10' + 75' + 5') = 7,095 \text{ FT}^2$

$A_{EX} = (75' + 10' + 75' + 5') \times (2' + 75') - (75' \times 75') = 7,080 \text{ FT}^2$

$\Rightarrow A_{①} + A_{②} + A_{EX} = 19,335 \text{ FT}^2 \times 1.167 \text{ FT} = \underline{\underline{22,564 \text{ FT}^3}}$

$V_{REQ} = 22,500 \text{ FT}^3$

$$F.S. = \frac{22,564 \text{ FT}^3}{22,500 \text{ FT}^3} = 1.00 \ \underline{OK}$$

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SHEET 4 OF 5

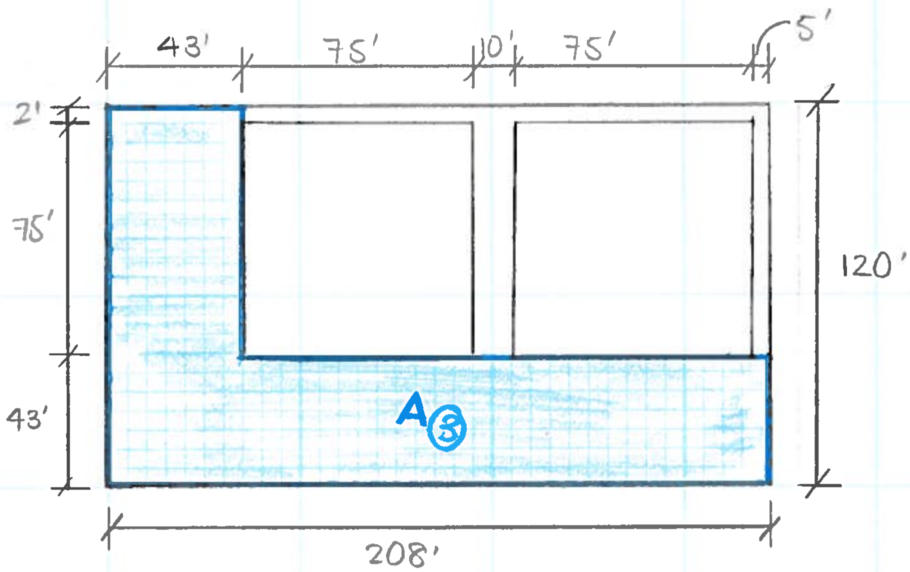
FILE 11896A

MADE BY AEP DATE 8/7/13

CHECKED BY GS DATE 8/7/13

PROJECT EXELON TOWER + TF GARAGE

SUBJECT CHECK CONTAINMENT BERM DESIGN FOR 100-YEAR STORM



IF A 100-YEAR STORM OCCURS WHILE ONE 75' x 75' x 4' TANK LEAKS...

- TANKS ARE ALREADY SIZED TO HANDLE 100-YEAR STORM DIRECT CATCHMENT OF 7.1"/24 HRS AND IMPACTED STORM WATER FROM SITE

$A_{③} \cong (208' \times 120') - 2(75' \times 75') = 13710 \text{ FT}^2$

- RAINS 7.1"/24 HRS $\Rightarrow V_{③} = (13710 \text{ FT}^2) \times (7.1/12) = 8112 \text{ FT}^3$
- THIS VOLUME WILL BE DISTRIBUTED OVER THE ENTIRE AREA OF THE CONTAINMENT BERM...

$$\frac{8,112 \text{ FT}^3}{(208' \times 120')} = 0.325 \text{ FT} = 3.9''$$

$\therefore$ AN ADDITIONAL 3.9" OF DEPTH NEEDS TO BE ADDED TO THE CONTAINMENT BERM DESIGN PLUS AN ADDITIONAL 4" OF "SPILL HEIGHT"

$\rightarrow$ NEW BERM DIMENSIONS :

$208' \times 120' \times (14'' + 3.9'' + 4'')$

$\Rightarrow \underline{\underline{208' \times 120' \times 22''}}$

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FILE 11896A

MADE BY AEP DATE 7/31/13

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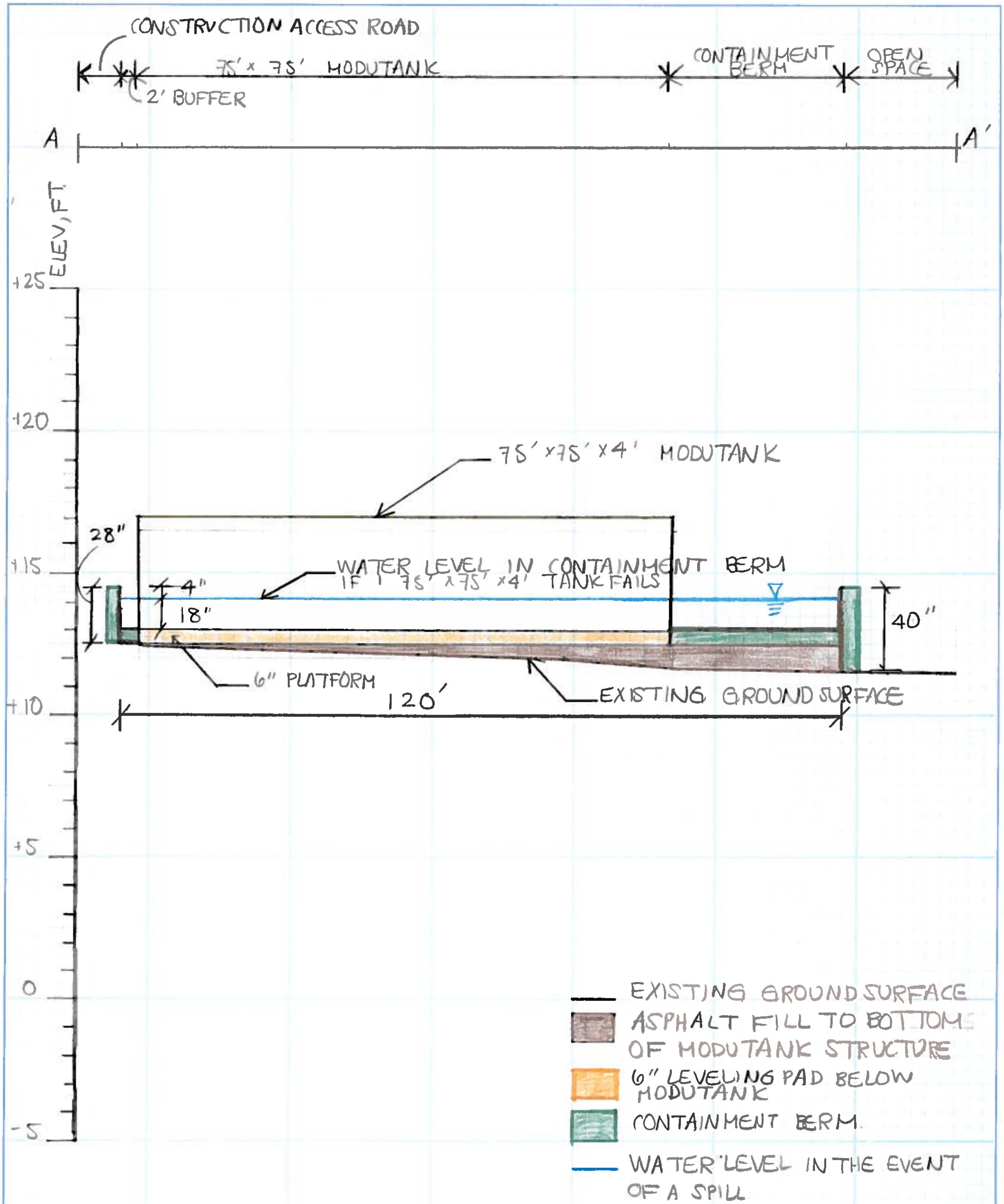
PROJECT EXELON TOWER + TF GARAGE

SUBJECT CONTAINMENT BERM DESIGN



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MEMORANDUM

Date: August 6, 2013
To: Office
From: Adam M. Dyer
Re: EE Memo 3 – Diverted Flow in Drainage Net from Foundation Construction
Exelon Tower, Trading Floor Garage & Plaza Garage, Baltimore, MD
File: 11896A-40

This memorandum summarizes the analysis of impedance to flow and changes in flow direction within the drainage net resulting from construction of foundations for the Exelon Tower, Trading Floor Garage and Plaza Garage development, and utilities supporting the development.

Exhibits

Calculation Set 1 Percent Obstruction to Flow within Drainage Net
Calculation Set 2 Area without Drainage Net
Calculation Set 3 Assessment of Infiltration Galleries

Sketch 1 Proposed Valley Drain and Infiltration Gallery Design Assessment

Available Information

1. Drawing DDP F1.60 – Development Cap, dated June 14, 2013
2. Drawing DDP F1.21 – Multi Media Cap Drainage Plan
3. Drawing DDP F1.25 – Sheet Pile Wall Typical Details
4. Drawing DDP F1.32 – Utility Crossing Plan and Sections

References

1. "Corrective Measures Implementation Construction Completion Report, Phase I: Soil-Bentonite Hydraulic Barrier Wall, Phase II: Final Remedial Construction" prepared by Black and Veatch, Volumes I and II, February 2000.
2. "Maryland Stormwater Design Manual, Appendix D.13", Maryland Department of the Environment (MDE), 2009.

Multimedia Cap

The Corrective Measures Implementation Report (CMI Report) by Black and Veatch details the construction and layering of the multimedia cap (MMC). The MMC includes a synthetic drainage net on the geomembrane. The MMC was constructed such that water that infiltrates the soil cover will flow away from the center of the cap through the drainage net and will not pond on the membrane. A contour of the surface of the geomembrane layer is presented in Ref. 1. The water flowing through the drainage net is discharged into the embankment along the waterside perimeter, and is collected in a toe drain at the land side perimeter. The toe drain, which is outboard of the soil-bentonite barrier, conveys water to the embankment where it is allowed to permeate into the porous embankment fill. Since construction of the MMC the site has been largely unused, except for temporary parking. It is presumed that settlement has not altered the slope of the drainage net and ponding does not occur.

The Surface Soil Monitoring Plan (SSMP) utilizes water in the drainage net to monitor performance of the MMC by testing the quality of representative samples of drainage net water. Drainage net water is sampled at four locations, identified as SSP1, SSP2, SSP3, and SSP4. At each sampling location the drainage net water crosses over a bucket where it enters the embankment; samples are taken from the bucket yearly and tested for total chromium and cyanide. At SSP1 and SSP4, the sampling bucket is at the location where the land side toe drain discharges to the embankment. At SSP2 and SSP3 a small section of the geomembrane is funneled to the sampling bucket.

Building Foundations

Development structures will be supported on high capacity piles which penetrate the geomembrane. Each penetration will be sealed using a mechanical clamp and gasket system. Many pile caps extend below the elevation of the surrounding geomembrane. A geomembrane dam will be placed around each pile cap to isolate drainage net water from the pile cap excavation. This dam will be left in place after pile cap construction is completed.

Utility Installation

A 30" gravity storm drain will be constructed a few feet below the elevation of the membrane on Wills St. and pass over the barrier, at about Elev. +4, at the Dock St. intersection. The MMC synthetic layers will be lowered below this pipe. The storm drain is at the same elevation as the toe drain, so that drainage net water collected in the Wills St. toe drain is isolated from sampling location SSP4. The water that flows in the drainage net in this area will follow the slope of the storm drain and will outlet off cap into the gravel bedding for the storm drain along Dock St.

Dock St. Platform

The development plan uses fill to raise street grades at Dock St. and Wills St., and utilizes these streets as utility corridors. HMS vaults V11, V12, and MJ1 and the HMS conveyance lines between these structures, and a new MMC will be supported on piles to prevent long term settlement under the raised grades. The pile-supported mat (Dock St. platform) is higher than the existing drainage net at the Dock St. perimeter.

Revised Drainage Net Discharge Plan

Drainage net water is obstructed from the existing toe drain along Dock St. and the toe drain is obstructed by the new 30 inch storm drain at the Wills St. intersection with Dock St. The proposed design to accommodate this revision is summarized in Sketch 1 "Proposed Valley Drain and Infiltration Gallery Design Assessment."

A new drain will be constructed on the MMC at the low point in the geomembrane (Valley Drain) south of the Dock St. platform. The Valley Drain to convey drainage net water to the embankment. Referring to Sketch 2, drainage net flow in Area A1, covering approximately 25% of the development area (that portion of the development area west of the geomembrane divide), will discharge to a new sampling location SSP4A. Area A2, covering approximately 65% of development area, will flow to the existing toe drain in Dock St. (east Valley Drain) for discharge through the relocated SSP4. Area A3, along Wills St. east of the proposed geomembrane dam and covering approximately 7.5% of the development area, is proposed to be discharged east of the barrier by adapting the existing toe drain into an infiltration gallery (the toe drain will be subdivided with seepage plugs into 50 ft long segments, each with an infiltration point). Area A4, covering 2.2% of the development area, will be lost to the stone bedding below the new storm drain pipe after the MMC is lowered below the pipe.

The quantity of storm water infiltration anticipated is greatly reduced after the development structures (roofs) and streets (curb, gutter, and storm drains) remove storm water from the MMC drainage layer. The revised toe drain provides for of 90% of the drainage net area below the development to pass through a sampling point (SSP4 and SSP4A), allowing the samples to be representative for monitoring the development influence.

Obstruction to Drainage Net Below Development Structures Analysis

Pile cap construction will isolate the pile cap and piles from the drainage net using a geomembrane dam at the perimeter of each excavation. Drainage net capacity to carry water between these flow obstructions is reviewed in this section. This analysis was performed on pile foundations known as of June 14, 2013. Pile cap design revisions since that time are not significant to the findings of this assessment.

Impedance to flow within the drainage net was quantified by computing the percentage of drainage net removed and not replaced. After development pile caps are completed 87.5% of the site will experience reduced infiltration as a result of the development structures (roofs) and streets (curb, gutter, and storm drains). Only 14.7% of the drainage net area has been obstructed by pile cap construction. Therefore, the MMC drainage layer should be capable of managing the anticipated storm water infiltration.

Drainage net flow capacity becomes restricted at overburden stresses above 2,000 lb/sq.ft. which corresponds to an area fill height of 16 ft over the drainage net. Load applied on the drainage net includes fill to proposed grade in street locations. Proposed fill heights do not exceed 16 ft.

Analysis of Wills St. Infiltration Gallery

The geomembrane dam isolating Wills St. from the drainage net below the development buildings reduces the intake area required for infiltration along Wills St. Calculation Set 3, attached, addresses the construction condition assuming the development structures are not complete and a 25- year and 100 year storm event occur. The infiltration assessment covers one 50 foot long segment of the former toe drain with a 5 foot long infiltration point. A 40 ft wide area of cover soil contributes to this infiltration point. Assuming an infiltration coefficient of 0.2, 240 ft^3/24 hrs of water will infiltrate the drainage net during the 100 year storm. The rate of discharge to the ground through the infiltration point is computed to be only 25 ft^3/24 hrs. Water which reaches the drainage net above that infiltration rate will flow down Wills St. to the Dock St. intersection where it will disappear into the gravel bedding below the storm sewer. This rate is sufficient for the reduced infiltration conditions anticipated after the development structures are in place. However, ground saturation above the geomembrane is possible in the 100 year storm after 24 hrs. Additional rainfall will run off. Saturated conditions will dissipate with time as storage above the membrane is discharge to the ground at the infiltration point. Active use of construction vehicles may be interrupted in this area until the water table drops.

Summary

MMC drainage requires revision in order to accommodate development and to provide the pile support improvement to the MMC and HMS systems below Dock St. in the development area. The MMC geomembrane cannot discharge to the existing toe drain for reasons stated above. Development revisions proposed are acceptable because:

- The risk of infiltration to the HMS pumps is greatly reduced because development roof and street drainage will remove direct storm water from 87.5% of the development area.
- Only 14.7% of the drainage net area is obstructed by pile cap construction.
- Drainage net flow from 90% of the drainage net area will pass through sampling points SSP4 or SSP4A (new) so that the drainage net water may continue to be used to evaluate the MMC performance after development foundations are in place.

By: ______________________________
Adam M. Dyer

AMD\PWD\11896A-40\ Flow in Drainage Net

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SHEET NO. 1 OF 2
FILE 11896A-40

FORM 3

FOR EXELON TOWER & TF GARAGE

MADE BY AMD DATE 6/13/13
CHECKED BY DJG DATE 6/17/13

SUBJECT % OBSTRUCTIONS TO FLOW WITHIN DRAINAGE NET.

PURPOSE: ASSESS THE TOTAL AREA OF REMOVED DRAINAGE NET (DN) FROM PILE CAP CONSTRUCTION.

REFERENCES:

1. DDP FOUNDATION DRAWING DDP-F1.60 - DEVELOPMENT CAP
2. "AREA WITHOUT DRAINAGE NET"
3. SKETCH 1 - FLOW WITHIN DRAINAGE NET.

ASSUMPTIONS:

1. DRAINAGE NET IS REMOVED WITHOUT REPLACEMENT AT PILE CAPS AS SHOWN ON REF 2.

CALCULATIONS:

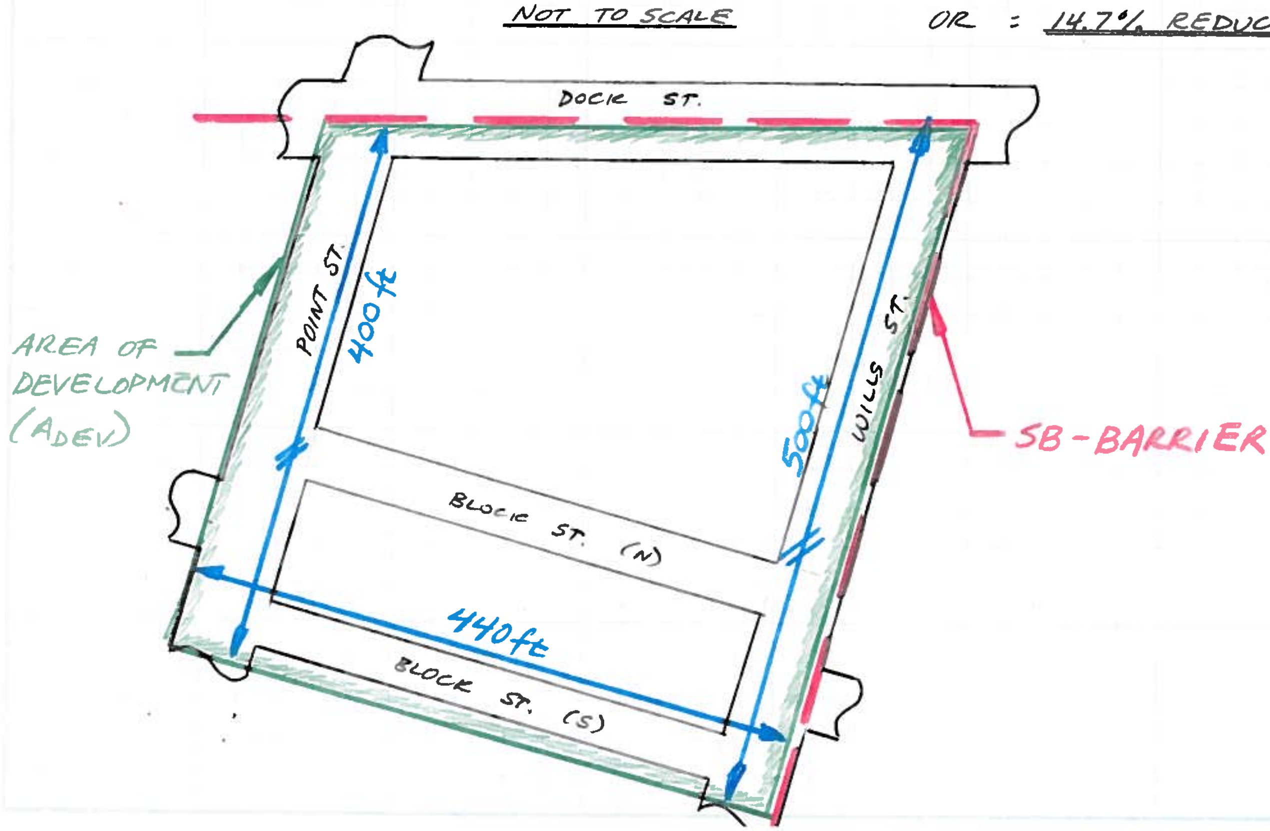
AREA TO BE DEVELOPED: $A_{DEV} = (400+500)/2 \cdot 440\,sf = \underline{198,000\ sf}$

AREA OF REMOVED DN: $A_{DNR} = \underline{29254\,sf}$ (FROM REF 2)

% AREA REMAINING: $(A_{DEV} - A_{DNR}) / A_{DEV} = \underline{85.3\ \%}$

OR : 14.7% REDUCTION

NOT TO SCALE



FORM 3

FOR EXELON TOWER & TF GARAGE

MADE BY AMD DATE 6/17/13

CHECKED BY ______ DATE ______

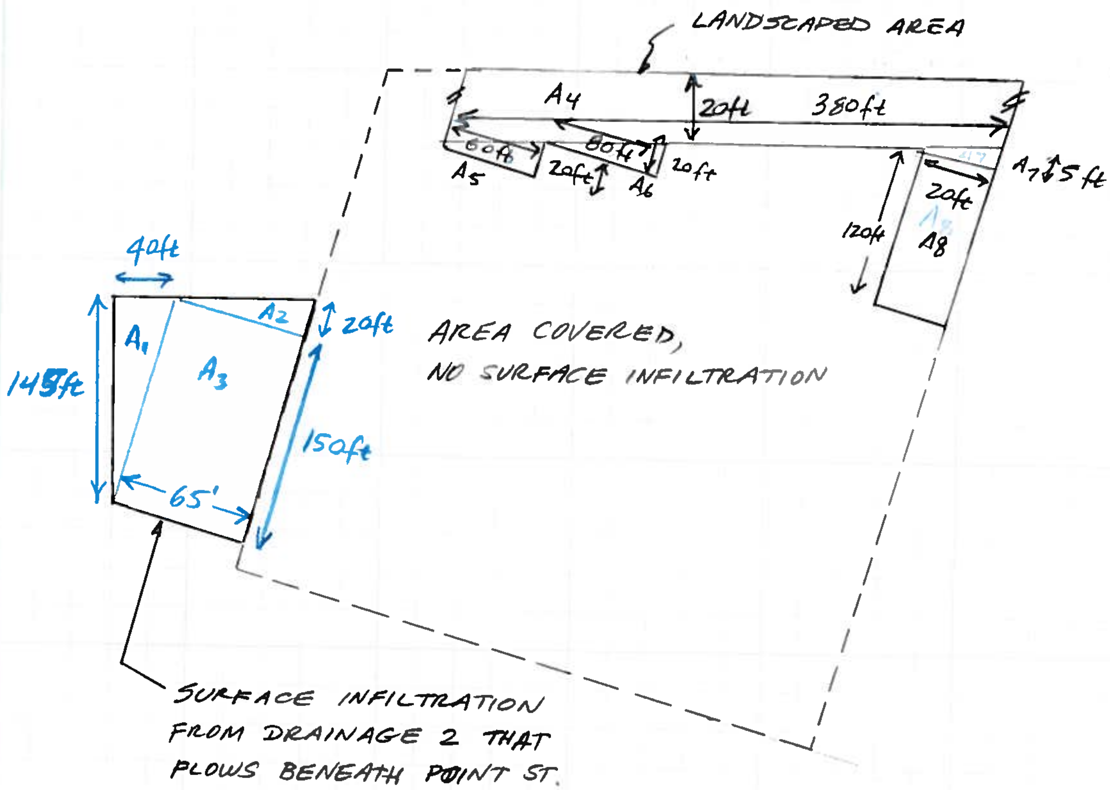
SUBJECT % OBSTRUCTION TO FLOW WITHIN DRAINAGE NET

A_{DEV} = 198,000 sf

A_{DNR} = 29,254 sf

A_{EXP} = AREAS EXPOSED TO SURFACE INFILTRATION.

NOT TO SCALE



$A_{EXP} = \sum_{i=1}^{8} A_i \Rightarrow$

A_1 =	40·145/2 sf	= 2900 sf	DRAINAGE 3 13,300 sf
A_2 =	20·65/2 sf	= 650	
A_3 =	65·150 sf	= 9750	
A_4 =	20·380 sf	= 7600	DRAINAGE 2 11,450 sf
A_5 =	20·60/2 sf	= 600	
A_6 =	20·80/2 sf	= 800	
A_7 =	5·20/2 sf	= 50	
A_8 =	20·120 sf	= 2400	
		24,750 sf	

$$\frac{A_{DEV} - A_{EXP}}{A_{DEV}} = \frac{198,000 - 24,750 \text{ sf}}{198,000} = 87.5\% \text{ LOAD REDUCTION} = LR$$

∴ LOAD = 12.5% OF PREVIOUS

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File No.: 11896A-40

Made by: AMD Date: 6/17/13

FOR: Exelon Tower and TF Garage Engineering Evaluation

Checked by: DJG Date: 6/17/13

SUBJECT: Calc 2: Areas without Drainage Net

DRAFT

Pile Cap	Number of Piles	Excavation Subgrade Elevation	Depth of Excavation Subgrade Below MMC	Pile Cap Edge to Drainage Dam, B (ft)	Length of Pile Cap (ft)	Width of Pile Cap (ft)	Area Without Drainage Net (ft^2)
A-7	6	10.5	0.0	2.0	12.5	8	198
A-6	7	10.5	0.0	2.0	16.5	10	287
A-5	7	10.5	0.0	2.0	16.5	10	287
A-4	7	10.5	0.0	2.0	16.5	10	287
A-3	7	10.5	0.0	2.0	16.5	10	287
A-2	6	10.5	0.3	2.5	12.5	8	224
A-1	5	10.5	0.5	2.8	10	10	240
B-1	6	10.5	0.7	3.1	12.5	8	262
B-2	5	10.5	0.4	2.6	10	10	231
C-1	5	10.5	1.0	3.5	10	10	289
C-2	4	10.5	0.8	3.2	8	8	207
C-5	7	10.5	0.0	2.0	16.5	10	287
B.1-7	5	10.5	0.0	2.0	10	10	196
C-7	5	9.5	0.0	2.0	10	10	196
D-7.8	6	9.5	0.0	2.0	12.5	8	198
D-6	9	9.5	0.0	2.0	12.5	12.5	272
D-5	9	9.5	0.8	3.2	12.5	12.5	357
D-4	8	9.5	1.5	4.3	16.5	10	463
D-3.1	9	10.5	0.8	3.2	12.5	12.5	357
D-2	7	10.5	1.2	3.8	16.5	10	424
D-1	5	10.5	1.3	4.0	10	10	320
B/C-7.8	3	10.5	0.0	2.0	8	7.5	138
C-7.8	6	9.5	0.0	2.0	12.5	8	198
D-7	8	9.5	0.0	2.0	16.5	10	287
E-7.1	4	9.5	0.0	2.0	8	8	144
E-8	3	9.5	0.0	2.0	8	7.5	138
E-10	2	9.5	0.0	2.0	8	3.5	90
E-6.1	4	9.5	0.3	2.5	8	8	166
E-5.1	4	9.5	1.3	4.0	8	8	253
E-4.1	4	9.5	1.7	4.6	8	8	292
E-3.1	4	10.5	0.9	3.4	8	8	216
E-2.1	4	10.5	1.2	3.8	8	8	243
E-1.2	3	10.5	1.3	4.0	8	7.5	245
F-1.2	4	10.5	1.0	3.5	8	8	225
F-2.1	5	10.5	0.9	3.4	10	10	279
F-3.1	6	10.5	0.6	2.9	12.5	8	253
F-4.1	6	9.5	1.3	4.0	12.5	8	324
F-5.1	6	9.5	1.0	3.5	12.5	8	293
F-6.1	6	4.8	5.2	9.7	12.5	8	877
F-7.1	6	4.8	4.4	8.5	12.5	8	740
F-7.8	7	4.8	4.0	7.9	16.5	10	836
F-8	4	4.8	3.8	7.6	8	8	541
F-10	5	6.5	1.2	3.8	10	10	310
G-10	3	6.5	1.3	4.0	8	7.5	245

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G-8.9	7	6.5	1.8	4.7	16.5	10	502
G-8	4	6.5	2.5	5.8	8	8	380
G-7.1	6	6.5	2.8	6.2	12.5	8	508
G-4.1	7	9.5	0.7	3.1	16.5	10	364
G-3.1	7	10.5	0.0	2.0	16.5	10	287
G-2.1	6	10.5	0.3	2.5	12.5	8	224
G-1.2	4	10.5	0.5	2.8	8	8	182
G.9-1.2	3	10.5	0.2	2.3	8	7.5	152
G.9-2.1	3	10.5	0.1	2.2	8	7.5	145
G.9-3.1	3	10.5	0.0	2.0	8	7.5	138
G.9-6.0	9	9.5	0.2	2.3	12.5	12.5	292
G.7-9	3	8.5	0.1	2.2	8	7.5	145
G.9-9	9	8.5	0.3	2.5	12.5	12.5	303
G.7-10	2	8.5	0.0	2.0	8	3.5	90
Shear Wall*		7.5	2.5	5.8	174	55	12336

* - Dimensions preliminary, awaiting final design loads **Total: 29254**

DRAFT

Pile Caps dimensions

# of piles	Comments	Dim 1 (ft)	Dim 2 (ft)
2		8.0	3.5
3	Triangular	8.0	7.5
4		8.0	8.0
5		10.0	10.0
6		12.5	8.0
7		16.5	10.0
8		16.5	10.0
9		12.5	12.5

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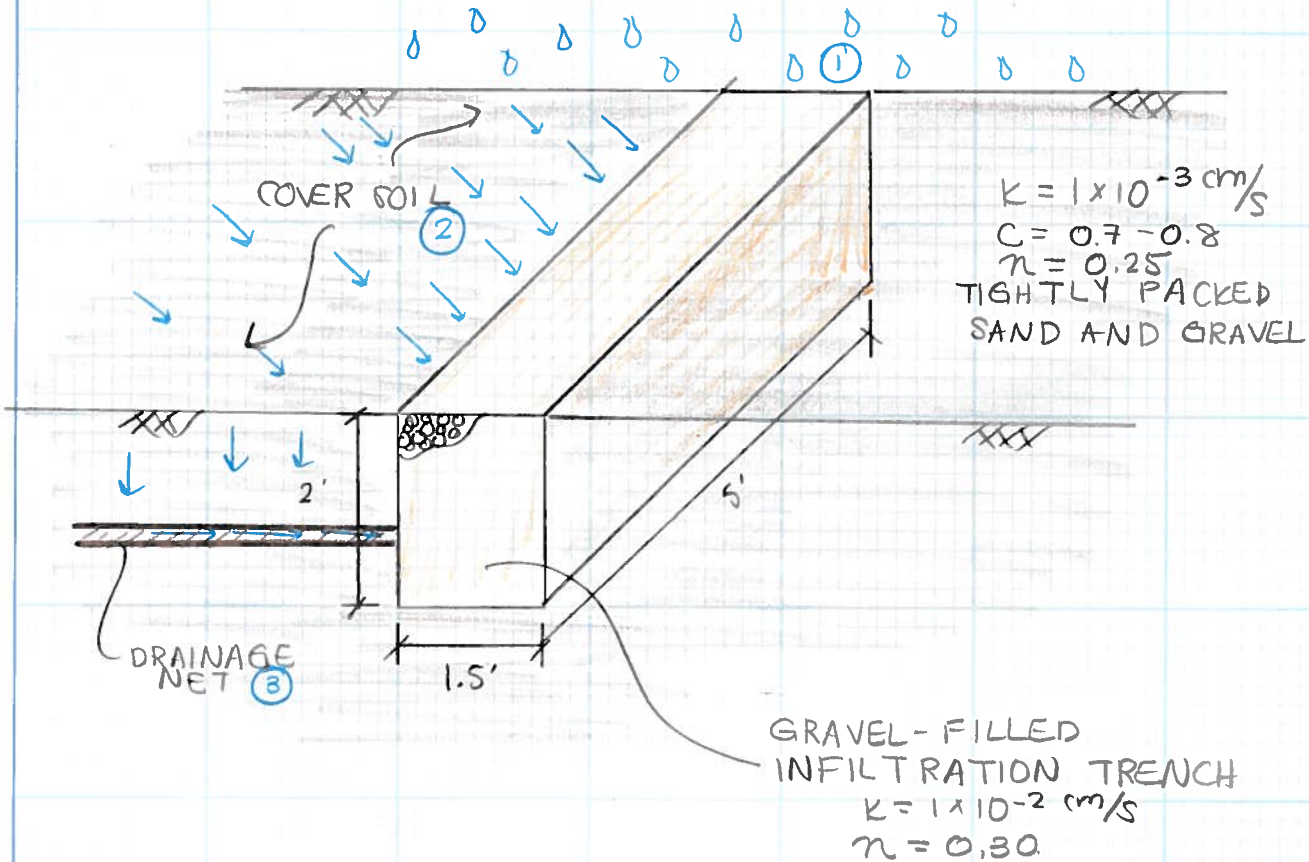
PROJECT EXELON TOWER + TF GARAGE

MADE BY AEP DATE 7/31/13
CHECKED BY AMD DATE 7/31/13

SUBJECT CALCULATION SET 3 - ASSESSMENT OF INFILTRATION GALLERIES

GIVEN: 18" × 5' × 2' INFILTRATION TRENCH FILLED WITH TIGHTLY PACKED GRAVEL ($K = 1 \times 10^{-3}$ cm/s) AND A 100-YEAR STORM, HOW LONG WILL IT TAKE FOR WATER TO INFILTRATE THROUGH THE SUBSOIL?



SOURCES OF WATER

1. WATER FALLING DIRECTLY ON TRENCH (DIRECT CATCHMENT) DURING STORM

VOLUME OF TRENCH ≡ $V_T = 1.5' \times 5' \times 2' = 15 \text{ FT}^3$

TOTAL PRECIPITATION OVER 24 HRS DURING 100-YEAR STORM ≡ $P = 7.1''/24 \text{ HRS} = 0.592 \text{ FT}/24\text{HRS}$

PLAN AREA OF TRENCH ≡ $A_T = 1.5' \times 5' = 7.5 \text{ FT}^2$

VOLUME OF STORM WATER RESULTING FROM DIRECT CATCHMENT ≡ $W_{DC} = (7.5 \text{ FT}^2)(0.592 \text{ FT}/24\text{HRS})$

$= 4.44 \text{ FT}^3/24 \text{ HRS}$

$W_{DC} = 33.2 \text{ GAL}/24 \text{ HRS}$

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SUBJECT CALC SET 3 - ASSESSMENT OF INFILTRATION GALLERIES

(2) WATER FLOWING OVER GROUND SURFACE AS SURFACE RUNOFF TOWARD TRENCH

DRAINAGE BASIN AREA $\equiv A_3 = 2000$ FT^2
RUNOFF COEFFICIENT $\equiv C = 0.80$
BY THE RATIONAL METHOD; $Q_p = CIA$

$Q_p \equiv$ PEAK DISCHARGE
$C \equiv$ RUNOFF COEFFICIENT $= 0.80$
$I \equiv$ RAINFALL INTENSITY $= 7.1$ IN/24 HRS
$A \equiv$ DRAINAGE AREA $= 2{,}000$ FT^2

VOLUME OF WATER RESULTING FROM SURFACE RUNOFF $= W_{SR} = (0.80)(2000\text{FT}^2)(0.592\ \text{FT}/24\text{HRS})$

$W_{SR} = 947.2\ \text{FT}^3/24\ \text{HRS}$

$W_{SR} = 7086\ \text{GAL}/24\ \text{HRS}$

(3) WATER INFILTRATES THROUGH COVER SOIL AND IS CARRIED BY DRAINAGE NET INTO INFILTRATION TRENCH

ASSUMPTIONS:

- WITHIN THE DRAINAGE BASIN AREA (A_3), THE TOTAL VOLUME OF STORM WATER WILL FLOW BY EITHER SURFACE RUNOFF OR INFILTRATION.

THE TOTAL VOLUME OF WATER IN THE DRAINAGE BASIN DURING A 100-YEAR STORM EVENT IS:

$W_{TOT} = A \times I = 2000\ \text{FT}^2 \times 0.592\ \text{FT}/24\text{HRS} = 1184\ \text{FT}^3/24\text{HRS}$

THEREFORE, THE VOLUME OF WATER INFILTRATING INTO DRAINAGE NET IS:

$W_I = W_{TOT} - W_{SR} = 1184\ \text{FT}^3/24\ \text{HRS} - 947.2\ \text{FT}^3/24\ \text{HRS}$

$W_I = 236.8\ \text{FT}^3/24\ \text{HRS}$

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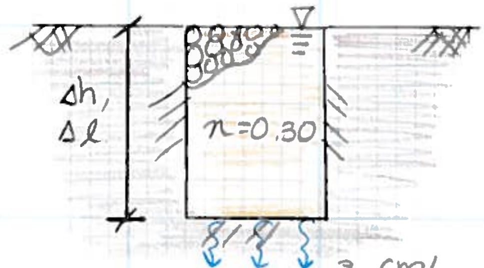
SUBJECT CALC SET 3 - ASSESSMENT OF INFILTRATION GALLERIES

STORAGE CAPACITY OF INFILTRATION TRENCH AND UNDERLYING SOIL

INFILTRATION RATE OF UNDERLYING SOIL:



DARCY'S LAW:

$Q = kiA$

$k \equiv$ HYDRAULIC CONDUCTIVITY

$i \equiv$ HYDRAULIC HEAD $= \Delta h / \Delta \ell$

$A \equiv$ AREA GW FLOWS THROUGH (PLAN AREA OF TRENCH)

$k = 1 \times 10^{-3}$ cm/s $= 3.3 \times 10^{-5}$ ft/s

$i = \Delta h / \Delta \ell = 1.0$

$A = (5')(1.5') = 7.5 FT^2$

$\therefore Q = (3.3 \times 10^{-5}$ ft/s$)(1.0)(7.5 FT^2)$

$= 2.5 \times 10^{-4} FT^3/s$

$Q = 0.9 FT^3/HR$

OVER 24 HOURS, $Q = 0.9 FT^3/HR \times 24$ HRS

$Q = 21.6 FT^3/24$ HRS

AVAILABLE STORAGE OF INFILTRATION TRENCH

FROM WEIGHT-VOLUME RELATIONSHIPS:

POROSITY $\equiv n = \frac{V_v}{V}$ $V_v \equiv$ VOLUME OF VOIDS; $V \equiv$ TOTAL VOLUME

TOTAL VOLUME OF INFILTRATION TRENCH $\equiv V = 15 FT^3$

AVAILABLE STORAGE $\equiv V_v = 0.30 \times 15 FT^3 = 4.5 FT^3$

$V_v = 4.5 FT^3$

TOTAL STORAGE IN INFILTRATION TRENCH OVER 24 HRS.

$S_{TOT} = Q + V_v = 21.6 FT^3/24 HRS + 4.5 FT^3 = 26.1 FT^3/24 HRS$

SHEET 4 OF 5

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SUBJECT CALC SET 3 - ASSESSMENT OF INFILTRATION GALLERIES

STORAGE CAPACITY OF DRAINAGE BASIN

THE 40' x 50' DRAINAGE BASIN HAS 2' OF PERMEABLE SOIL OVERLYING DRAINAGE NET THAT CAN ALSO STORE STORM WATER DURING A STORM EVENT

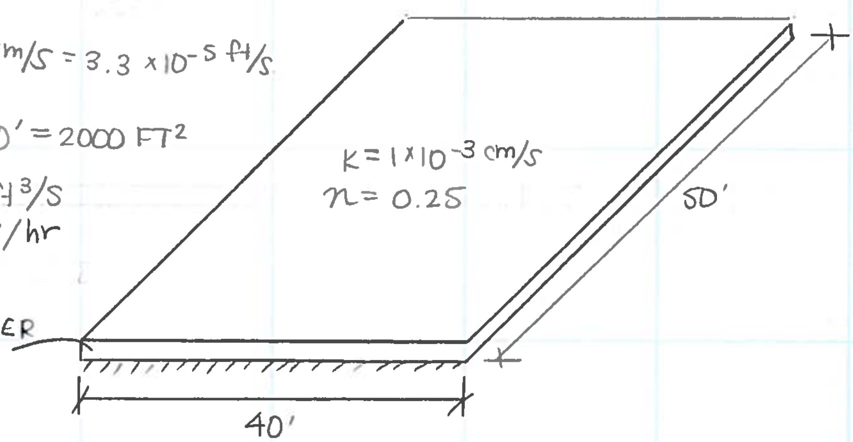
$$n = \frac{V_v}{V}$$

$$V = 40' \times 50' \times 2' = 4{,}000 \text{ FT}^2$$
$$n = 0.25$$

$$0.25 = \frac{V_v}{4{,}000 \text{ FT}^3} \Rightarrow S_{DB} = \begin{array}{l}\text{AVAILABLE}\\\text{STORAGE}\\\text{IN DRAINAGE}\\\text{BASIN}\end{array} = V_V = 1000 \text{ FT}^3$$

INFILTRATION RATE OF DRAINAGE BASIN

$$Q = kiA$$
$$k = 1 \times 10^{-3} \text{ cm/s} = 3.3 \times 10^{-5} \text{ ft/s}$$
$$i = 1.0$$
$$A = 40' \times 50' = 2000 \text{ FT}^2$$
$$\Rightarrow Q = 0.07 \text{ ft}^3/\text{s}$$
$$= 252 \text{ ft}^3/\text{hr}$$



TOTAL VOLUME OF WATER TO BE STORED

SURFACE WATER WILL FLOW INTO STORM DRAINS, THEREFORE ONLY WATER FROM INFILTRATION AND DIRECT CATCHMENT WILL NEED TO BE STORED.

$$W_{TOT} = W_I + W_{DC} = 236.8 \text{ FT}^3/24 \text{ HRS} + 4.44 \text{ FT}^3/24 \text{ HRS}$$
$$= 241.24 \text{ FT}^3/24 \text{ HRS}$$

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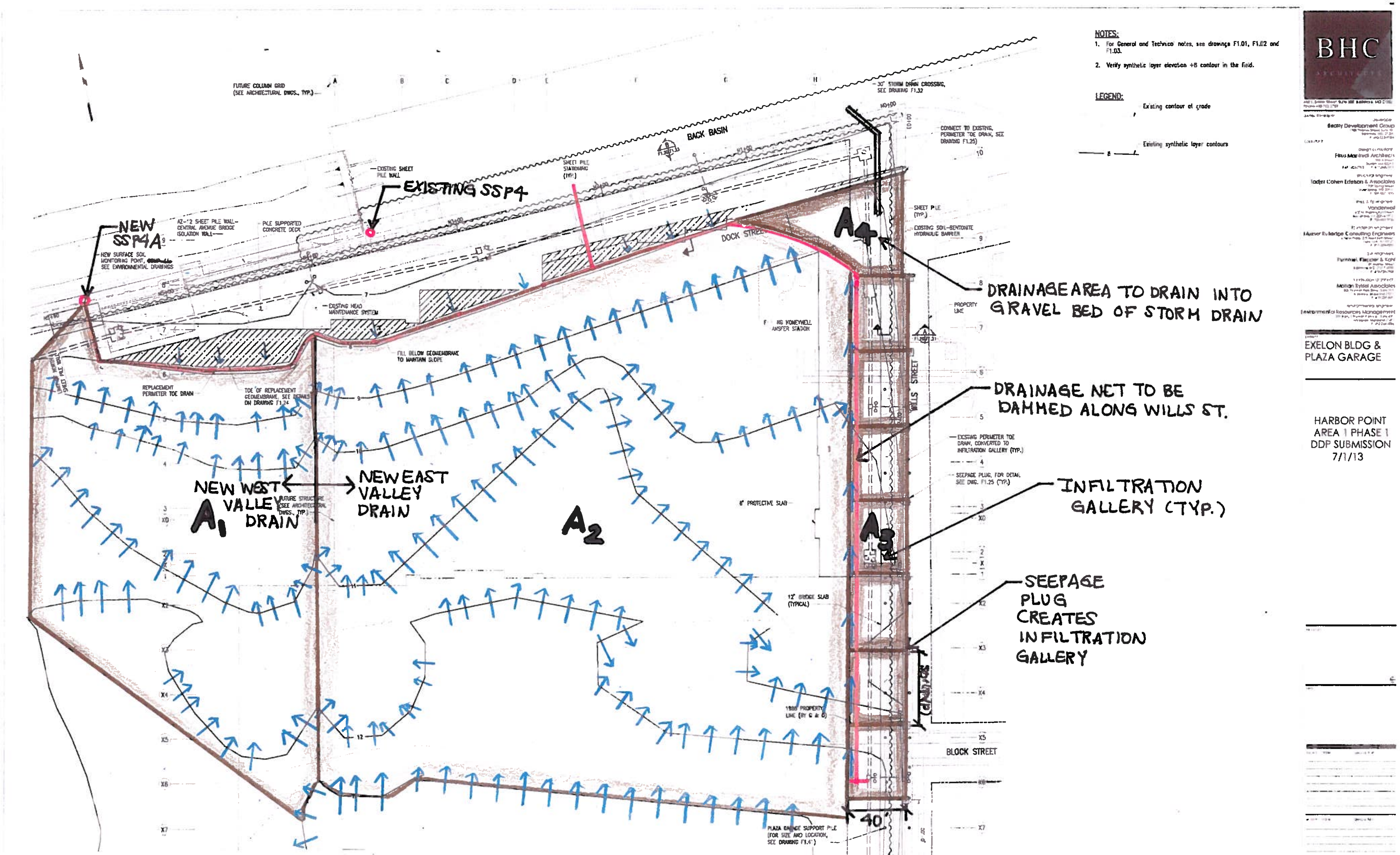
PROJECT EXELON TOWER AND TF GARAGE

SUBJECT CALC SET 3 - ASSESSMENT OF INFILTRATION GALLERIES

DISCUSSION

A 50' x 40' DRAINAGE BASIN PRODUCES 236.8 FT^3/24 HRS OF INFILTRATION DURING A 100-YEAR STORM EVENT. A 1.5' x 2' x 5' INFILTRATION TRENCH HAS A STORAGE CAPACITY OF 4.5 FT^3 AND WILL DRAIN INTO THE UNDERLYING SOIL AT A RATE OF 0.9 FT^3/HR OR 21.6 FT^3/24 HRS. THE DRAINAGE BASIN HAS A STORAGE CAPACITY OF 1000 FT^3, HOWEVER WATER IN THE BASIN WILL TEND TO FLOW TOWARD THE GRAVEL BED UNDERLYING THE STORM DRAIN AT THE INTERSECTION OF WILLS AND DOCK ST.



DRAINAGE BASIN	AREA (FT^2)	% TOTAL AREA
A_1	47,630	25.5
A_2	120,922	64.8
A_3	14,000*	7.5
A_4	4,200	2.2

$\Sigma = 186,752$

*NOTE: SINGLE INFILTRATION GALLERY AREA = 2,000 FT^2 x 7 GALLERIES = 14,000 FT^2

1" = 66.667'
GRAPHIC SCALE

PROGRESS SET NOT FOR CONSTRUCTION

SKETCH 1
PROPOSED VALLEY DRAIN AND INFILTRATION GALLERY DESIGN ASSESSMENT

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MEMORANDUM

Date: August 6, 2013
To: Office
From: Adam M. Dyer and Gina Schoregge
Re: EE Memo 4 – Hydraulic Conductivity of Sheet Pile Barrier
Exelon Building & Plaza Garage, Baltimore, MD
File: 11896A-40

This memorandum summarizes an analysis of the effectiveness of the planned sheet pile barrier within existing soil-bentonite barrier.

Exhibits

Plate 1	Observed Vibration Attenuation during Pile Load Test Program
Plate 2	Equivalent Hydraulic Conductivity Calculation
Plate 3	Verification of Verticality
Attachment 1	Skyline Steel Data Sheets
Attachment 2	SWELLSEAL WA – Technical Information Sheet

Available Information

1. Drawing DDP F1.02 – Structural/Foundation/Sheet Pile Notes, dated July 15, 2013
2. Drawing DDP F1.20 – Sheet Pile Plan, dated July 15, 2013
3. Drawing DDP F1.22, 23 – Sheet Pile Sequence, dated July 15, 2013
4. Drawing DDP F1.24, 25 – Sheet Pile Details, dated July 15, 2013
5. Drawing DDP F1.40 – Foundation Plan, dated July 15, 2013

References

1. "Construction Dewatering and Groundwater Control New Methods and Applications" by J. Patrick Powers, Arthur B. Corwin, Paul C. Shmall, and Walter E. Kaeck, 3rd Edition. Wiley, Hoboken, New Jersey, 2007.
2. "Geoenvironmental Engineering" by Hari D. Sharma and Krishna R. Reddy. Wiley, 2004.
3. "An Introduction to Geotechnical Engineering" by Robert D. Holtz and William D. Kovacs, Prentice Hall, Upper Saddle River, New Jersey, 1981.

Soil-Bentonite Barrier

During construction of the Soil-Bentonite Barrier (SB Barrier), samples of slurry were analyzed for as-built permeability. It was found that the as-built permeability was on the order of 1E-09cm/sec or less, well below the performance criteria of 1E-07cm/sec. This construction has been theorized to develop areas of relieved stress caused by settlement-induced arches which results in low confining stress and provide a path for transmittal of water across the barrier.

The development contract requires future access for repair of the SB Barrier and prohibits imparting vibrations greater than 2 in/sec peak particle velocity in close proximity to the SB barrier. To date, monitoring of the head maintenance system has shown that the SB Barrier has performed as originally constructed.

Building Foundations

As described in the Design Development Plan (DDP), pile foundations will be installed within the SB Barrier 30-foot disturbance restriction. The pile load test program performed in May and June, 2013 measured vibrations associated with pile driving approaching the 2 in/sec peak particle velocity limit, (Plate 1). The Exelon Project has elected to augment the SB barrier with a sheet pile barrier as a pre-emptive repair to allow pile driving in close proximity to the barrier and construction of structures over the barrier alignment.

Sheet Pile Barrier

The sheet pile barrier will consist of continuous AZ 12-770 interlocking steel sheet piles with sealed interlocks. Half of the Interlocks will be sealed by a continuous weld the length of the sheet pile. Half of the interlocks will be sealed with a continuous bead of DeNeef hydrophilic Swellseal (dry method). After installing sheets below the water table, the Swellseal material will expand within the interlock and perform as a compressed gasket to restrict seepage through the interlocks. Sheet piles will be installed using a vibratory hammer.

Sheet Pile installation may result in settlement of the SB backfill as a result of densification. Sheet pile insertion should break any stress arches which may be present.

Corrosion of Sheet Piles

Average corrosion rates for steel sheet piling in marine environments, as provided by Eurocode 3, are listed below:

Table 4-2: Loss of thickness [mm] due to corrosion for piles and sheet piles in fresh water or in sea water

Required design working life	5 years	25 years	50 years	75 years	100 years
Common fresh water (river, ship canal, ...) in the zone of high attack (water line)	0,15	0,55	0,90	1,15	1,40
Very polluted fresh water (sewage, industrial effluent, ...) in the zone of high attack (water line)	0,30	1,30	2,30	3,30	4,30
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0,55	1,90	3,75	5,60	7,50
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone	0,25	0,90	1,75	2,60	3,50

Notes:

1) The highest corrosion rate is usually found in the splash zone or at the low water level in tidal waters. However, in most cases, the highest bending stresses occur in the permanent immersion zone, see Figure 4-1.

2) The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.

Sea Water
Use 25 year corrosion rate for extrapolation: 0.9mm/25years = 0.036mm/year
AZ12-770 Sheeting Minimum Thickness: 8.5mm
Total thickness lost: 8.5mm/0.036mm/yr = 236 years

Fresh Water
Use 25 year corrosion rate for extrapolation: 0.55mm/25years = 0.022mm/year
AZ12-770 Sheeting Minimum Thickness: 8.5mm
Total thickness lost: 8.5mm/0.022mm/yr = 386 years

The site ground water contains 9000 ppm brackish water which is about 1/3 the salt content of sea water at 35000 ppm. Using sea water corrosion rates of 0.036mm/year is too conservative. The total loss of thickness due to corrosion in sea water is 236 years. In fresh water it would take about 386 years. To consider the brackish water, use the average of these two: life span is 311 years.

Verticality of Sheet Piles

The verticality of sheet piles with the required construction tolerances was assessed by geometrically determining if sheet pile exited the wall. As stated on Drawing DDP F1.02, the front edge of the sheet pile must be within 3 inches of the center line of the SB-Barrier and within 1% of plumb. Two cases were examined as shown below in Figure 1. Case 1 interpreted the depth at which the toe of the sheet pile would exit the wall if the sheet pile was installed at its' inboard limit and Case 2 interpreted the sheet pile at its' outboard limit.

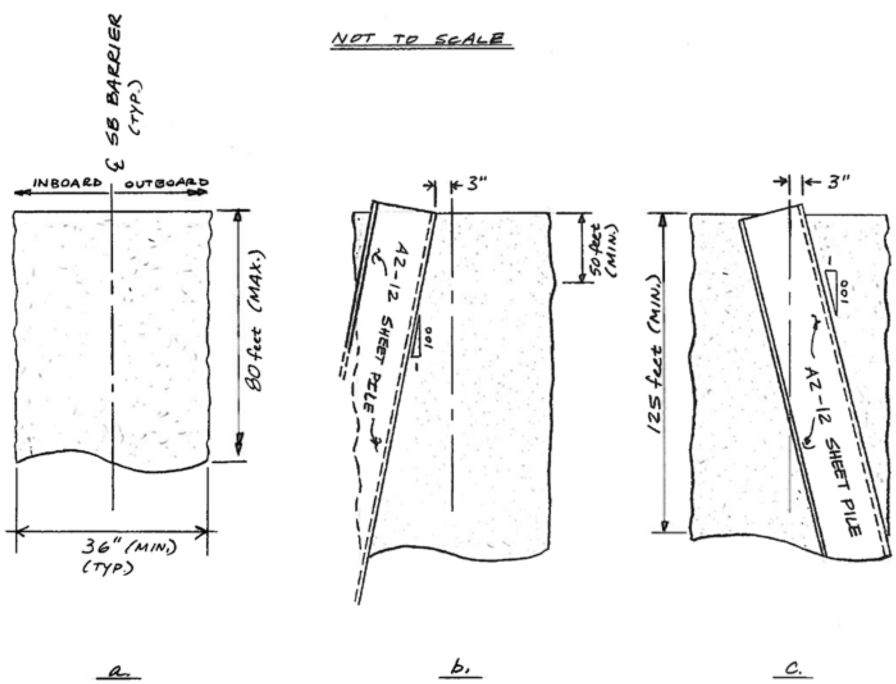


Figure 1 – Assessment of Verticality of Sheet Pile Wall: (a) Existing SB Barrier; (b) Sheet Pile Installed at Inboard Limits; (c) Sheet Pile Installed at Outboard Limits

For Case 1, the sheet pile would exit the wall at a depth of 50 feet. For Case 2, the sheet pile would exit the wall at a depth of 125 feet, for calculations see Plate 3.

Equivalent Hydraulic Conductivity

Analysis

The effectiveness of the sheet pile wall installation was assessed by determining an equivalent hydraulic conductivity, $k_{SH,AVG}$, of the sheet pile wall. The wall $k_{SH,AVG}$ was derived by analyzing the geometric average of equivalent hydraulic conductivity for each material within the system. The system was analyzed with a parametric study of the hydraulic conductivity of Swellseal filled joints, SB-Barrier backfill permeability, and as a function of the width of possible construction gaps, d (Plate 2). A summary of $k_{SH,AVG}$ for no gaps is provided below in Table 1. For the purposes of this assessment the effective permeability of steel was taken as, k_{ST} = 1E-12cm/sec. The equivalent hydraulic conductivity was computed as shown below in Equation 1.

$$k_{SH,AVG} = \frac{k_{Gap} * d + k_{St} * n * (w - t_{Jt}) + k_{Jt} * n * t_{Jt}}{d + n * w}$$

Equation 1 – Geometric Average for Equivalent Hydraulic Conductivity of Sheet Pile Wall

The system was modeled for five scenarios, as described below:

1. k_{SB} = 5e-9 cm/sec, as measured during construction
2. k_{Gap} = 5e-9, k_{Jt} = 1e-5 cm/sec
3. k_{Gap} = 5e-9, k_{Jt} = 1e-6 cm/sec
4. k_{Gap} = 5e-9, k_{Jt} = 1e-7 cm/sec
5. k_{Gap} = 5e-9, k_{Jt} = 1e-9 cm/sec

Results

Table 1 – $k_{SH,AVG}$ for each scenario with a gap of 0in

Wall Modification		Estimated $k_{SH,AVG}$ (cm/sec)	Estimated Fraction of Present Day Barrier Seepage
1	None	5e-09	1.0
2	Swellseal provides k_{Jt} = 1e-05cm/sec	4.12e-08	8.24
3	Swellseal provides k_{Jt} = 1e-06cm/sec	4.13e-09	0.826
4	Swellseal provides k_{Jt} = 1e-07cm/sec	4.13e-10	0.0826
5	Swellseal provides k_{Jt} = 1e-09cm/sec	5.12e-12	0.0001

Discussion

Corrosion Protection
The thickness of the steel sheets provides sufficient corrosion protection for a life span of over 200 years.

Verticality of Sheet Piles
For sheets installed at the construction tolerance battered outboard, Case 1 (Figure 1b), the sheet pile will exit the wall at a minimum depth of 50 feet. This is above the maximum depth of the installed sheets as shown on Drawing DDP F1.20 and will exit the wall on the inboard side. Anticipated soils at this depth will be very dense and will encounter hard driving; easy driving within the soft soil of the SB Barrier will prevent significant deviation outside of barrier.

For sheets installed at the construction tolerance battered inboard, Case 2 (Figure 1c), the sheet pile will exit the wall at a minimum depth of 125 feet. This is well below the maximum depth of installed sheets as shown on Drawing DDP F1.20 and therefore will remain inside the wall.

Equivalent Hydraulic Conductivity
The parametric study shows that the equivalent hydraulic conductivity is heavily dependent on the current state of the SB-Barrier and the capability of the Swellseal to act as a gasket. It should be noted that any gaps in sheeting would result in an ineffective wall. Quality control measures during sheeting installation with respect to the equivalent hydraulic conductivity of the wall should include the following:

1. Interlocks in good condition and free to join to adjacent sheets;
2. Interlock welds are applied to the full length of the sheet and have no gaps;
3. Application of DeNeef Swellseal is applied uniformly using the dry method;
4. Allow no gaps in sheeting during installation;

By: ______________________________
Adam M. Dyer

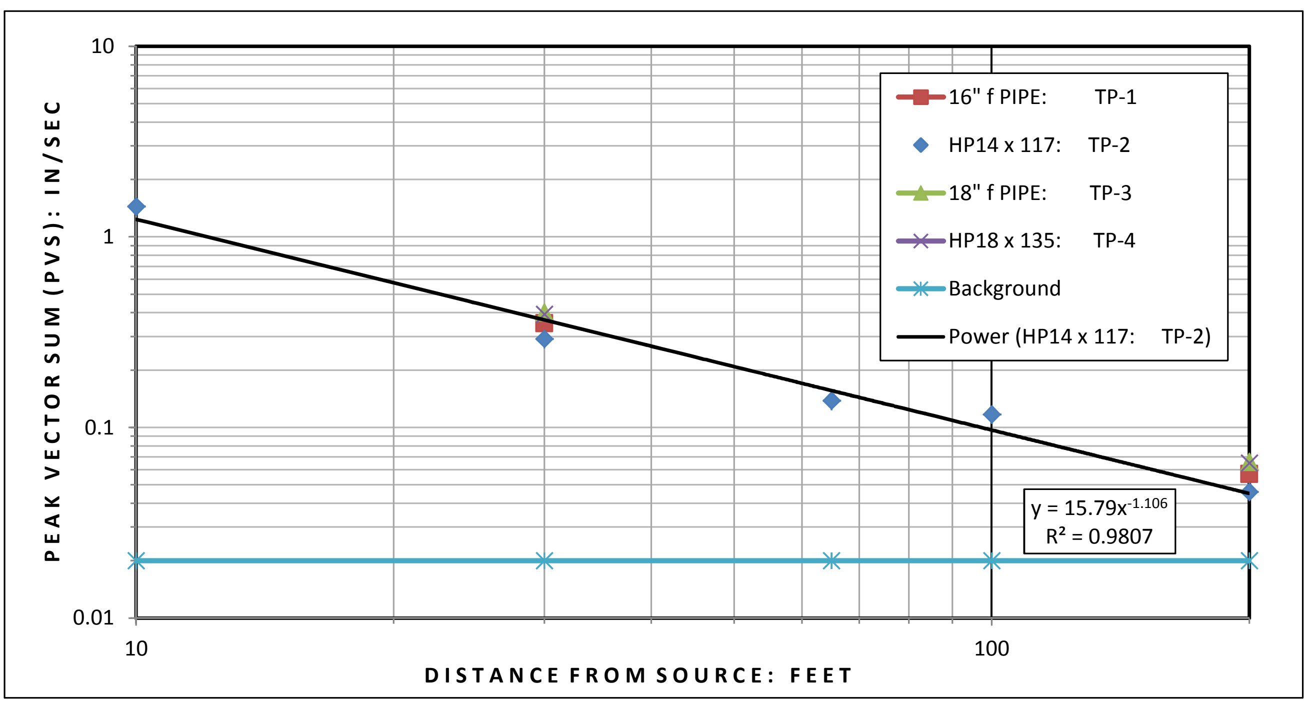
By: ______________________________
Gina Schoregge

SUBJECT: COMPARISON OF MAXIMUM OBSERVED PVS (in/sec) IN THE EAST-WEST DIRECTION BY TP-1 THRU 4

TABLE 1: SHALLOW DRIVING (LESS THAN 55FT BGS):

	DISTANCE FROM SOURCE, FEET	MAX RECORDED PVS (in/sec) BY PILE TYPE				
		16" ϕ PIPE: TP-1	18" ϕ PIPE: TP-3	HP14 x 117: TP-2	HP18 x 135: TP-4	BACKGROUND[1]
UNIT S3	10	NO DATA		1.440	NO DATA	0.05 TO 0.06[2]
	30	0.352	0.401	0.291	0.392	0.02 TO 0.04[2]
	RE-STRIKE	0.481				
	65	NO DATA		0.138	NO DATA	0.06 TO 0.07[3]
	100			0.117		~0.04
UNIT S4	200	0.057	0.066	0.046	0.065	0.02 TO 0.05
	RE-STRIKE	0.065	0.068	0.035	0.046	

CHART 1: SHALLOW DRIVING (LESS THAN 55FT BGS):



MUESER RUTLEDGE CONSULTING ENGINEERS

File No.: 11896A-40

Made by: AMD   Date: 6/18/13

FOR: Exelon Tower and TF Garage Engineering Evaluation   Checked by: Date:

SUBJECT: Equivalent Hydraulic Conductivity After Installation of Sheets in Soil Bentonite Barrier

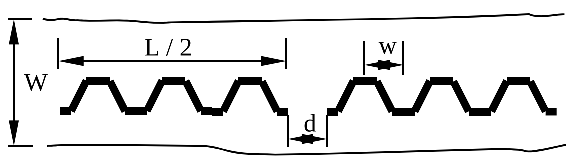
References:

1. Geoenvironmental Engineering by Hari D. Sharma and Krishna R. Reddy
2. Skyline Steel Data sheets

Assumptions:

1. Sheet used is an AZ 12-770; $t_f = t_w = 0.335$ in; w = 30.31 t_f = 0.335 in t_{Jt} = 0.125 in
2. Steel hydraulic conductivity k_{ST} = 1e-12 cm/sec;   w = 30.31 in L_{min} = 250 ft
3. Width of Soil Bentonite Barrier (SB), W = 36 in   W = 36 in   ∴ n = 99
4. Gap between sheets = d (in)
5. Alternate weld/swellseal every sheet at joints, where joint space t_{Jt} = 0.125 in
6. Length between allowed gaps, L ~ 250 feet (where n = #sheets)
7. A geometric average of hydraulic conductivity provides a reasonable estimate of the system k

Wall Diagram:



Calculations:

From Ref. 1, it can be shown that the equivalent hydraulic conductivity across the sheeting (k_{SH}) and across the wall (k_{AVG}):

$$k_{SH,AVG} = \frac{k_{Gap} * d + k_{St} * n * (w - t_{Jt}) + k_{Jt} * n * t_{Jt}}{d + n * w}$$

Scenarios:

1. $k_{SB} = k_{Gap}$ = 5e-9, k_{Jt} = N/A
2. $k_{SB} = k_{Gap}$ = 5e-9, k_{Jt} = 1e-5 (cm/sec)
3. $k_{SB} = k_{Gap}$ = 5e-9, k_{Jt} = 1e-6 (cm/sec)
4. $k_{SB} = k_{Gap}$ = 5e-9, k_{Jt} = 1e-7 (cm/sec)
5. $k_{SB} = k_{Gap}$ = 5e-9, k_{Jt} = 1e-9 (cm/sec)

For various gaps between sheeting panels the k_{AVG} is:

	Equivalent Hydraulic Conductivity, $k_{SH,AVG}$ (cm/sec)				
d (in)	1	2	3	4	5
0.00	5.00E-09	4.12E-08	4.13E-09	4.13E-10	5.12E-12
0.25	5.00E-09	4.12E-08	4.13E-09	4.14E-10	5.54E-12
0.50	5.00E-09	4.12E-08	4.13E-09	4.14E-10	5.95E-12
0.75	5.00E-09	4.12E-08	4.13E-09	4.15E-10	6.37E-12
1.00	5.00E-09	4.12E-08	4.13E-09	4.15E-10	6.78E-12
1.25	5.00E-09	4.12E-08	4.13E-09	4.15E-10	7.20E-12
1.50	5.00E-09	4.12E-08	4.13E-09	4.16E-10	7.62E-12
1.75	5.00E-09	4.12E-08	4.13E-09	4.16E-10	8.03E-12
2.00	5.00E-09	4.12E-08	4.13E-09	4.16E-10	8.45E-12
2.25	5.00E-09	4.12E-08	4.13E-09	4.17E-10	8.86E-12
2.50	5.00E-09	4.12E-08	4.13E-09	4.17E-10	9.28E-12
2.75	5.00E-09	4.12E-08	4.13E-09	4.18E-10	9.69E-12
3.00	5.00E-09	4.12E-08	4.13E-09	4.18E-10	1.01E-11
3.25	5.00E-09	4.12E-08	4.13E-09	4.18E-10	1.05E-11
3.50	5.00E-09	4.12E-08	4.13E-09	4.19E-10	1.09E-11
3.75	5.00E-09	4.12E-08	4.13E-09	4.19E-10	1.14E-11

MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896A -40

MADE BY AMD DATE 7/12/2013

CHECKED BY GS DATE 7/15/13

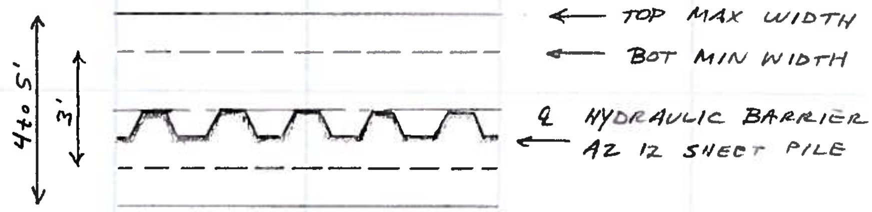
PROJECT EXELON TOWER & TF GARAGE

SUBJECT VERTICALITY OF SHEET PILE WALL

PURPOSE: ASSESSMENT OF SHEET PILE VERTICALITY AND LOCATION CONSTRUCTION TOLERANCES.

REFERENCES: 1. DRAWINGS: DDP F1.01, 02, 20
2. CONSTRUCTION COMPLETION REPORT BY BLACK & VEATCH, DATED FEBRUARY 2000.

SHEET PILE WALL SCHEMATIC:



WIDTH OF TRENCH IS NO LESS THAN 3ft WIDE AS SHOWN BY "HEXOMETER" READINGS DESCRIBED IN REFERENCE 2.

QUALITATIVE ASSESSMENT

DURING DRIVING SHEET WILL FOLLOW LEAST RESISTANT PATH.
SB BARRIER BACKFILL IS SIGNIFICANTLY WEAKER THAN ADJACENT SOILS

∴ POTENTIAL FOR SHEET TO "KICK" TO ONE SIDE IS MINIMAL.,
DURING DRIVING SHEET WILL BE CONTINUOUSLY OBSERVED AND VERTICALITY CHECKED.

CONSTRUCTION TOLERANCES: (SEE DWG DDP F1.20)

HORIZONTAL: WITHIN 3in OF PLAN LOCATION.

VERTICALITY: WITHIN 1% OF PLUMB

MUESER RUTLEDGE CONSULTING ENGINEERS

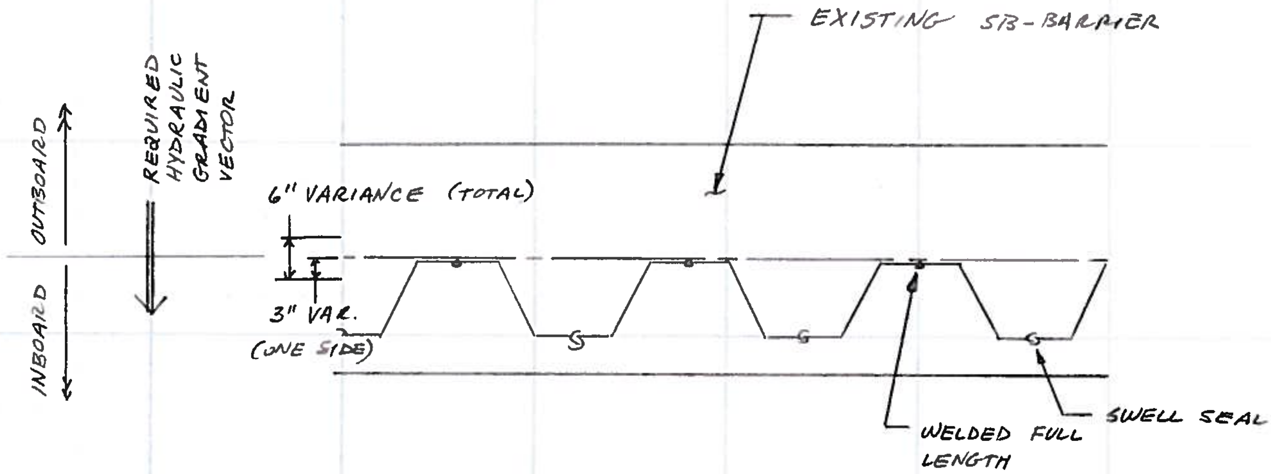
FILE 11896A-40

MADE BY AMD DATE 7/12/13

CHECKED BY GS DATE 7/15/13

PROJECT EXELON TOWER & TF GARAGE

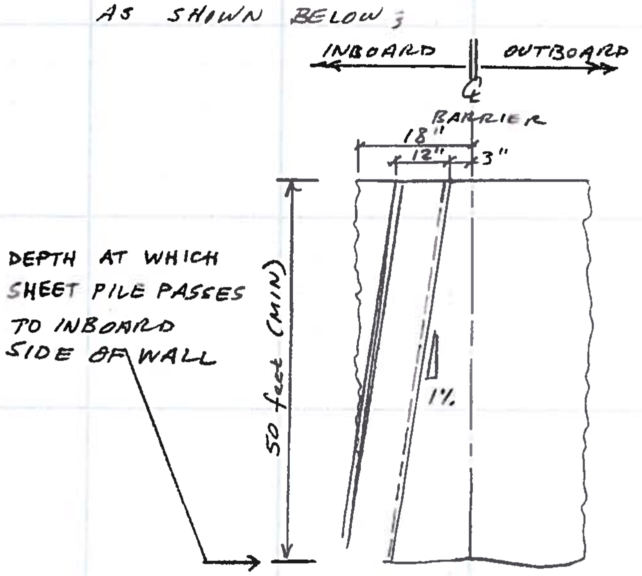
SUBJECT VERTICALITY OF SHEET PILE WALL



ASSESSMENT OF VERTICALITY

INBOARD

1. IF SHEETS ARE AT VERTICAL TOLERANCE AND HORIZONTAL TOLERANCE AS SHOWN BELOW;



$1"/100" = 1\%$ (H/V)

FOR $6"/x = 1\%$

$\therefore x = 600"$ VERTICAL

$= 50$ ft

SHEET PILES PASSING THROUGH THE WALL INBOARD DOES NOT CREATE PREFERENTIAL PATH THROUGH TO OUTSIDE OF WALL.

MUESER RUTLEDGE CONSULTING ENGINEERS

MADE BY AMD DATE 7/26/13
CHECKED BY GS DATE 7/15/13

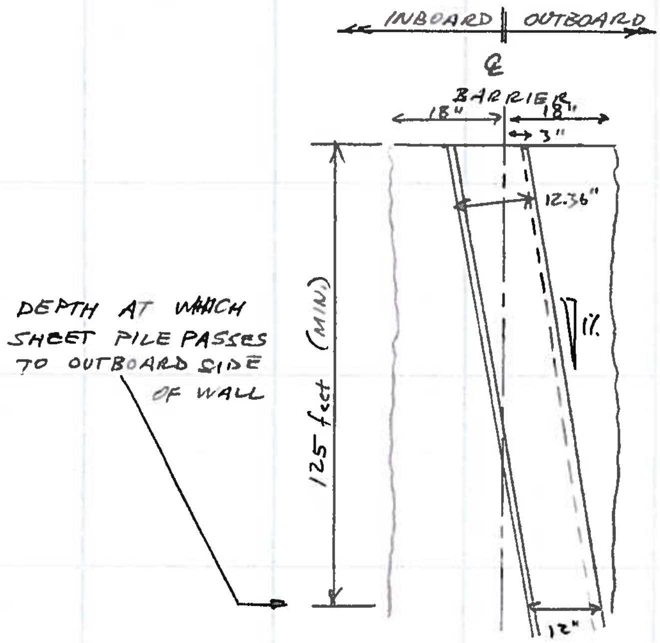
PROJECT EXELON TOWER & TF GARAGE

SUBJECT VERTICALITY OF SHEET PILE WALL.

ASSESSMENT OF VERTICALITY (CONT'D)

OUTBOARD

2. IF SHEETS ARE AT VERTICAL AND HORIZONTAL TOLERANCE AS SHOWN BELOW



1" / 100" = 1% (H/V)

FOR 15" / x = 1%

∴ x = 1500" VERTICAL

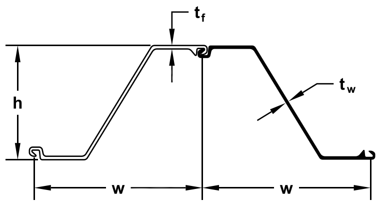
= 125 FEET

MAXIMUM HEIGHT OF WALL ~ 80 feet (SEE DWG. DDP F1.20)

∴ SHEET PILE WILL NOT PASS OUTBOARD OF BARRIER.

AZ

AZ Hot Rolled Steel Sheet Pile



			THICKNESS			WEIGHT		SECTION MODULUS			COATING AREA	
SECTION	Width (w) in (mm)	Height (h) in (mm)	Flange (t_f) in (mm)	Web (t_w) in (mm)	Cross Sectional Area in²/ft (cm²/m)	Pile lb/ft (kg/m)	Wall lb/ft² (kg/m²)	Elastic in³/ft (cm³/m)	Plastic in³/ft (cm³/m)	Moment of Inertia in⁴/ft (cm⁴/m)	Both Sides ft²/ft of single (m²/m)	Wall Surface ft²/ft² (m²/m²)
AZ 12-700	27.56 700	12.36 314	0.335 8.5	0.335 8.5	5.82 123.2	45.49 67.7	19.81 96.7	22.4 1205	26.3 1415	138.3 18880	5.61 1.71	1.22 1.22
AZ 13-700	27.56 700	12.40 315	0.375 9.5	0.375 9.5	6.36 134.7	49.72 74.0	21.65 105.7	24.3 1305	28.6 1540	150.4 20540	5.61 1.71	1.22 1.22
AZ 13-700-10/10	27.56 700	12.42 316	0.394 10.0	0.394 10.0	6.63 140.4	51.85 77.2	22.58 110.2	25.2 1355	29.8 1600	156.5 21370	5.61 1.71	1.22 1.22
AZ 14-700	27.56 700	12.44 316	0.413 10.5	0.413 10.5	6.90 146.1	53.96 80.3	23.50 114.7	26.1 1405	31.0 1665	162.5 22190	5.61 1.71	1.22 1.22
AZ 12-770	30.31 770	13.52 343.5	0.335 8.50	0.335 8.50	5.67 120.1	48.78 72.60	19.31 94.30	23.2 1245	27.5 1480	156.9 21430	6.10 1.86	1.20 1.20
AZ 13-770	30.31 770	13.54 344.0	0.354 9.00	0.354 9.00	5.94 125.8	51.14 76.10	20.24 98.80	24.2 1300	28.8 1546	163.7 22360	6.10 1.86	1.20 1.20
AZ 14-770	30.31 770	13.56 344.5	0.375 9.50	0.375 9.50	6.21 131.5	53.42 79.50	21.14 103.20	25.2 1355	30.0 1611	170.6 23300	6.10 1.86	1.20 1.20
AZ 14-770-10/10	30.31 770	13.58 345	0.394 10.0	0.394 10.0	6.48 137.2	55.71 82.9	22.06 107.7	26.1 1405	31.2 1677	177.5 24240	6.07 1.85	1.20 1.20
AZ 18	24.80 630	14.96 380.0	0.375 9.50	0.375 9.50	7.11 150.4	49.99 74.40	24.19 118.10	33.5 1800	39.1 2104	250.4 34200	5.64 1.72	1.35 1.35
AZ 17-700	27.56 700	16.52 419.5	0.335 8.50	0.335 8.50	6.28 133.0	49.12 73.10	21.38 104.40	32.2 1730	37.7 2027	265.3 36230	6.10 1.86	1.33 1.33
AZ 18-700	27.56 700	16.54 420.0	0.354 9.00	0.354 9.00	6.58 139.2	51.41 76.50	22.39 109.30	33.5 1800	39.4 2116	276.8 37800	6.10 1.86	1.33 1.33
AZ 19-700	27.56 700	16.56 420.5	0.375 9.50	0.375 9.50	6.88 145.6	53.76 80.00	23.41 114.30	34.8 1870	41.0 2206	288.4 39380	6.10 1.86	1.33 1.33
AZ 20-700	27.56 700	16.58 421	0.394 10.0	0.394 10.0	7.18 152.0	56.11 83.5	24.43 119.3	36.2 1945	42.7 2296	299.9 40960	6.10 1.86	1.33 1.33
AZ 26	24.80 630	16.81 427.0	0.512 13.00	0.480 12.20	9.35 198.0	65.72 97.80	31.79 155.20	48.4 2600	56.9 3059	406.5 55510	5.91 1.80	1.41 1.41
AZ 24-700	27.56 700	18.07 459.0	0.441 11.20	0.441 11.20	8.23 174.1	64.30 95.70	28.00 136.70	45.2 2430	53.5 2867	408.8 55820	6.33 1.93	1.38 1.38
AZ 26-700	27.56 700	18.11 460.0	0.480 12.20	0.480 12.20	8.84 187.2	69.12 102.90	30.10 146.90	48.4 2600	57.1 3070	437.3 59720	6.33 1.93	1.38 1.38
AZ 28-700	27.56 700	18.15 461.0	0.520 13.20	0.520 13.20	9.46 200.2	73.93 110.00	32.19 157.20	51.3 2760	60.9 3273	465.9 63620	6.33 1.93	1.38 1.38
AZ 24-700N	27.56 700	18.07 459.0	0.492 12.5	0.354 9.0	7.71 163.3	60.28 89.7	26.26 128.2	45.3 2435	52.3 2810	409.3 55890	6.30 1.92	1.37 1.37
AZ 26-700N	27.56 700	18.11 460	0.531 13.5	0.394 10.0	8.33 176.4	65.11 96.9	28.37 138.5	48.4 2600	56.1 3015	437.8 59790	6.30 1.92	1.37 1.37
AZ 28-700N	27.56 700	18.15 461	0.571 14.5	0.433 11.0	8.95 189.5	69.95 104.1	30.46 148.7	51.4 2765	59.9 3220	466.5 63700	6.30 1.92	1.37 1.37
AZ 36-700N	27.56 700	19.65 499.0	0.591 15.00	0.441 11.20	10.20 216.0	79.70 118.60	34.61 169.00	66.8 3590	76.5 4110	656.2 89610	6.76 2.06	1.47 1.47
AZ 38-700N	27.56 700	19.69 500.0	0.630 16.00	0.480 12.20	10.87 230.0	84.94 126.40	37.07 181.00	70.6 3795	81.1 4360	694.5 94840	6.76 2.06	1.47 1.47
AZ 40-700N	27.56 700	19.72 501.0	0.669 17.00	0.520 13.20	11.53 244.0	90.18 134.20	39.32 192.00	74.3 3995	85.7 4605	732.9 100080	6.76 2.06	1.47 1.47
AZ 42-700N	27.56 700	19.65 499.0	0.709 18.00	0.551 14.00	12.22 259.0	95.49 142.1	41.57 203.00	78.2 4205	90.3 4855	766.0 104930	6.76 2.06	1.47 1.47
AZ 44-700N	27.56 700	19.69 500.0	0.748 19.00	0.591 15.00	12.89 273.0	100.73 149.9	43.83 214.00	81.9 4405	94.9 5105	804.1 110150	6.76 2.06	1.47 1.47
AZ 46-700N	27.56 700	19.72 501.0	0.787 20.00	0.630 16.00	13.55 287.0	105.97 157.7	46.08 225.00	85.7 4605	99.5 5350	842.2 115370	6.76 2.06	1.47 1.47
AZ 46	22.83 580	18.94 481.0	0.709 18.00	0.551 14.00	13.76 291.2	89.10 132.60	46.82 228.60	85.5 4595	98.5 5295	808.8 110450	6.23 1.90	1.63 1.63
AZ 48	22.83 580	18.98 482.0	0.748 19.00	0.591 15.00	14.48 306.5	93.81 139.60	49.28 240.60	89.3 4800	103.3 5553	847.1 115670	6.23 1.90	1.63 1.63
AZ 50	22.83 580	19.02 483.0	0.787 20.00	0.630 16.00	15.22 322.2	98.58 146.70	51.80 252.9	93.3 5015	108.2 5816	886.5 121060	6.23 1.90	1.63 1.63

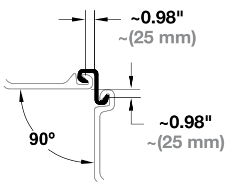
AZ

AZ Hot Rolled Steel Sheet Pile

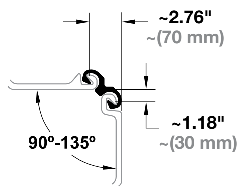
Available Steel Grades											
AMERICAN			CANADIAN			EUROPEAN			AMLoCor**		
ASTM	YIELD STRENGTH		CSA G40.21	YIELD STRENGTH		EN 10248	YIELD STRENGTH			YIELD STRENGTH	
	(ksi)	(MPa)		(ksi)	(MPa)		(ksi)	(MPa)		(ksi)	(MPa)
A 328	39	270	Grade 260 W	38	260	S 240 GP	35	240	Blue 320	46	320
A 572 Gr. 42	42	290	Grade 300 W	43	300	S 270 GP	39	270	Blue 355	51	355
A 572 Gr. 50	50	345	Grade 350 W	51	355	S 320 GP	46	320	Blue 390	57	390
A 572 Gr. 55	55	380	Grade 400 W	58	400	S 355 GP	51	355			
A 572 Gr. 60	60	415				S 390 GP	57	390			
A 572 Gr. 65	65	450				S 430 GP	62	430			
A 690	50	345				S 460 AP	67	460			
A 690*	57	390									

*Not available for AZ 36-700N and larger. ** Corrosion resistant steel, check for availability

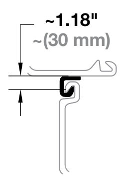
Corner Piles



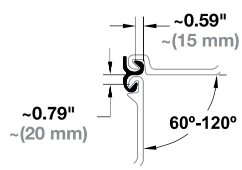
C 14
Grade: S 355 GP
Weight: 9.68 lb/ft
(14.4 kg/m)



Omega 18
Grade: S 430 GP
Weight: 12.10 lb/ft
(18.0 kg/m)



C 9
Grade: S 355 GP
Weight: 6.25 lb/ft
(9.3 kg/m)

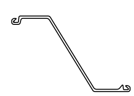


Delta 13
Grade: S 355 GP
Weight: 8.73 lb/ft
(13.0 kg/m)

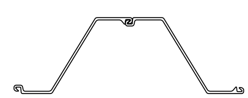
Delivery Conditions & Tolerances

	ASTM A 6		EN 10248	
Mass	± 2.5%		± 5%	
Length	+ 5 inches	– 0 inches	± 200 mm	
Height			± 7 mm	
Thickness			≤ 8.5 mm	± 0.5 mm
			> 8.5 mm	± 6%
Width			± 2%	
Double Pile Width			± 3%	
Straightness			0.2% of the length	
Ends out of Square			2% of the width	

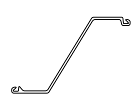
Delivery Forms



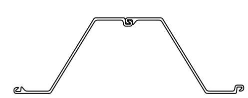
Single Pile
Position A



Double Pile
Form I standard



Single Pile
Position B



Double Pile
Form II on request

Maximum Rolled Lengths*

AZ	101.7 feet	(31.0 m)
C 9	59.1 feet	(18.0 m)
C 14	59.1 feet	(18.0 m)
Delta 13	55.8 feet	(17.0 m)
Omega 18	52.0 feet	(16.0 m)

* Longer lengths may be possible upon request.

SWELLSEAL®

Waterstops for Sheet Piles

Waterproofing the WORLD

de neef®
CONSTRUCTION CHEMICALS, INC.

THE NEED TO SEAL SHEET PILES

The Problem

As the use of sheet piling in wet environments increases, so does the need to create a safe, dry work area after excavation. The high cost of dewatering and treatment, as well as increased concerns for worker safety and potential damage to the surrounding eco-system pose a challenge to both the designer and contractor.

The Solution

SWELLSEAL® WA, hydrophilic polyurethane, offers a safe clean method of sealing sheet piling without the use of hazardous chemicals. Formulated to swell upon contact with water, hydrophilic polyurethanes can expand to any shape to form a seal against water leaking through the interlocks and penetrations in sheet piles.

Swellseal® WA applied with caulking gun

SWELLSEAL® WA

SWELLSEAL® WA is a single component hydrophilic polyurethane that can be applied in wet or dry environments. Upon contact with ground water, it can swell 2 or more times its original volume. When applied to the interlocks of sheet piling, it can swell to seal a leaking interlock in the sheet

Swellseal® WA after driving sheet piles

SWELLSEAL® WA Advantages:

- Easy to install gunnable paste
- No cure time required prior to driving sheets
- Can be applied to wet or dry surface
- Can be applied at cold temperatures
- Can wet and dry cycle repeatedly
- Can be applied to rough surfaces

SWELLSEAL® WA PRODUCT PROPERTIES

UNCURED		
Solids	100%	
Viscosity	Paste	
Density	1.45	ASTM D-3574-95
Flash point	>266° F	ASTM D-93
CURED		
Elongation at break	625%	ASTM D-3574-95
Tensile Strength	Approximately 312 psi	ASTM D-412

Withstands head pressures in excess of 330 ft.

SWELLSEAL® WA Properties:

- Single component hydrophilic polyurethane
- 200% Expansion in water
- Withstands pressures in excess of 330 ft. of head pressure
- Good chemical resistance
- Tenacious bond to wet and dry surfaces
- Conforms to the shape of the interlock
- Does not hinder the removal of sheet piles

Tieback sealed with ***HYDRO ACTIVE® CUT***

REPAIR

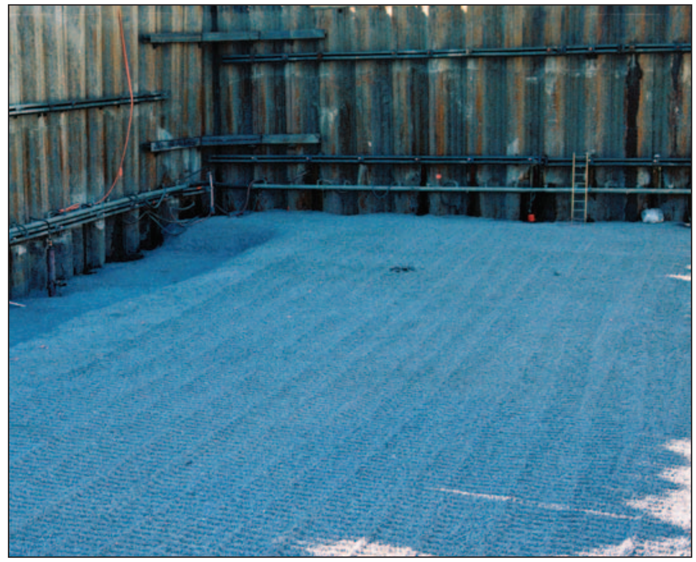
Properties and Advantages:

Leaks that appear after sealing sheets can be repaired with ***HYDRO ACTIVE® CUT***. Applied in liquid form by injection or saturation methods. ***HYDRO ACTIVE® CUT*** swells up to 20 times its original volume to cut off flowing water and seal active leaks.

Ideal Repair Applications

- Tiebacks
- Pipe penetrations
- Flowing water leaks

INSTALLATIONS



PACKAGING

SWELLSEAL® WA

- 10.5 ounce Tubes
- 20 ounce Sausage

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14 Penn Plaza · 225 West 34th Street · New York, NY 10122
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www.mrce.com

MEMORANDUM

Date: July 12, 2013
To: Office
From: Srinivas Yenamandra
Re: EE Memo 5 – Spill Control Volume of New Loading Dock
Exelon Building & Plaza Garage, Baltimore, MD
File: 11896A-Task 40

The proposed Exelon Trading Floor and Parking Garage (TF Garage) structure will occupy a portion of the space currently occupied by the Honeywell Transfer Station (HTS). Partial demolition of the east and west sides of the existing HTS structure (limits of demolition are shown on drawings) is required. The groundwater storage tank room (at north center), the adjacent mechanicals room to the south, and all head maintenance system components are to remain functional throughout the construction period.

Exhibits:
We have attached the following to illustrate our evaluation:

Calculation 1 - Spill Control Volumes
Sketch 1 – New Loading Dock Geometry

Existing Structural Foundations:
The foundations consist of shallow strip footings, shallow isolated column footings and slabs on grade, all of which are founded above the multimedia cap synthetic layers. All demolition work will be performed above the multimedia cap and the synthetic layers will not be exposed. The bottom of existing footing elevations are approximately Elev. +11 and the elevation of synthetic layers vary from Elev. 8 to Elev. 10. The synthetic layers in this area of the site are protected by a concrete mud mat overlain by structural backfill.

Pile Driving Adjacent to Existing Groundwater Storage Tanks and Equipment:
The proposed structure is founded on pile foundations. Prior to pile installation the MMC in the pile cap area will be excavated and the synthetic layers removed for obstruction demolition. No storage tank will hold more than ¼ of its capacity during pile driving. After pile installation the synthetic layers will be repaired. The process of cutting and repair of synthetic layers is described in detail elsewhere.

New Loading Dock:
The new loading dock slab will be constructed after completion of demolition of the existing loading dock and after installation of new piles and pile caps adjacent to the HTS. The new loading dock will be

constructed to provide secondary containment for 5,950 gal, which is greater than the capacity of the transport tank truck (5,000 gal).

The new loading dock will be a structural concrete slab (approximately 57 feet long x 15 feet wide) supported on the TF Garage pile caps and grade beams in this area. The slab will be 12 inches thick at the interface with sump pit and 15 inches deep at the perimeter providing a slope towards the sump pit to facilitate flow of potential spillage into the sump pit.

A collection sump pit 45 feet long x 6 feet wide x 2.5 feet deep will be constructed at the east side and below the loading dock. The new sump pit dimensions are shown on attached Sketch 1. The sump pit provides 5050 gallons of storage. The sloped slabs and drainage trough provide additional storage for 900 gallons.

The top of the loading dock slab slopes up from Elev.+13 at the sump pit to Elev. +13.25 at the perimeter on all four sides. The loading dock is enclosed on the east, west and south ends by walls that connect to adjacent floor slabs. On the North end the loading dock slab connects to the street. The walls on the three sides and the sloped slab in addition to the sump pit will control potential spill during transfer of groundwater from the tanks.

The sump pit and drainage trough will be covered with a metal grating (similar to the one used at the loading dock to be demolished) at the center of the pit and the rest of the sump pit will be covered by the loading dock structural slab. The sump pit base slab, the sump pit walls and the loading dock slab will be constructed in one pour (monolithic) to eliminate joints. In addition, the concrete for the slabs and walls will contain fiber reinforcement. The fiber will be Virgin Nylon Type monofilament, white color, ¾" long (uniform size) as was used in the construction of the existing loading dock, to minimize cracking.

Blast furnace slag, scrubber house fly ash or silica fume will be used in lieu of cement in the concrete used for the construction. The hardened concrete will be coated with a corrosion inhibitor such as Silane Sealer or approved equal.

As substantiated by Calculation 1, the total volume available for spill containment, including available volume above loading dock slab and sump pit, is more than adequate for the design spill of 5000 gallons.

By: ______________________________
Srinivas Yenamandra

MUESER RUTLEDGE CONSULTING ENGINEERS

File 11896

Made By FL Date 06/13/13

FOR Exelon Development

Checked By SY Date 6/13/2013

SUBJECT: **Spill Control Zone Volumes**

Considering that the full load of a standard truck of 5000 gallons will be contained in the sum pit, and allowing additonal volume capacity given the slab sope and the collecting trench, we have:

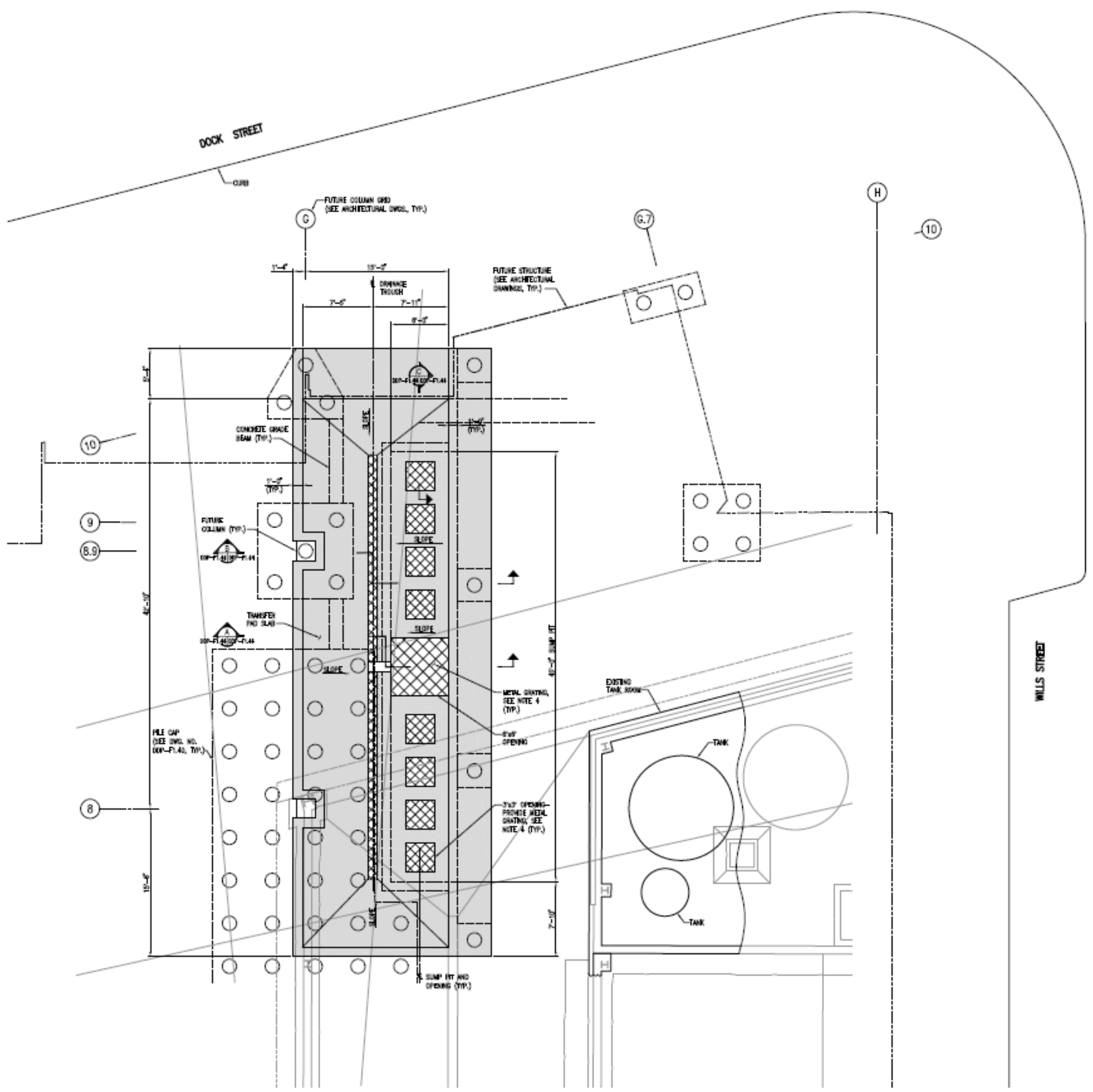
Sum Pit Volume: $V_p := 6ft \cdot 45ft \cdot 2.5ft$ $V_p = 675.0\ ft^3$ $\rightarrow$ $V_p = 5049.4\ gal$

<u>Additional Control Zone Volume</u>

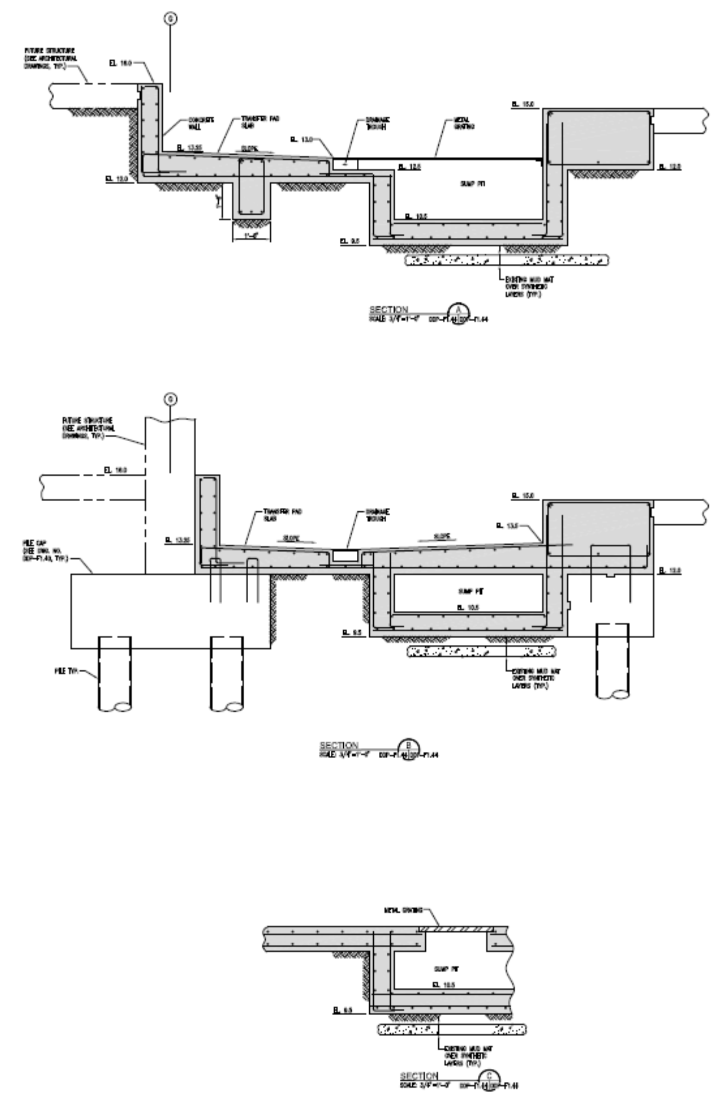
Slab Slope $V_{sl} := 51.59ft \cdot 15.33ft \cdot 0.5 \cdot (15in - 12in)$ $V_{sl} = 98.9\ ft^3$ $\rightarrow$ $V_{sl} = 739.5\ gal$

Center Trench $V_{tr} := 6in \cdot 12in \cdot 45ft$ $V_{tr} = 22.5\ ft^3$ $\rightarrow$ $V_{tr} = 168.3\ gal$

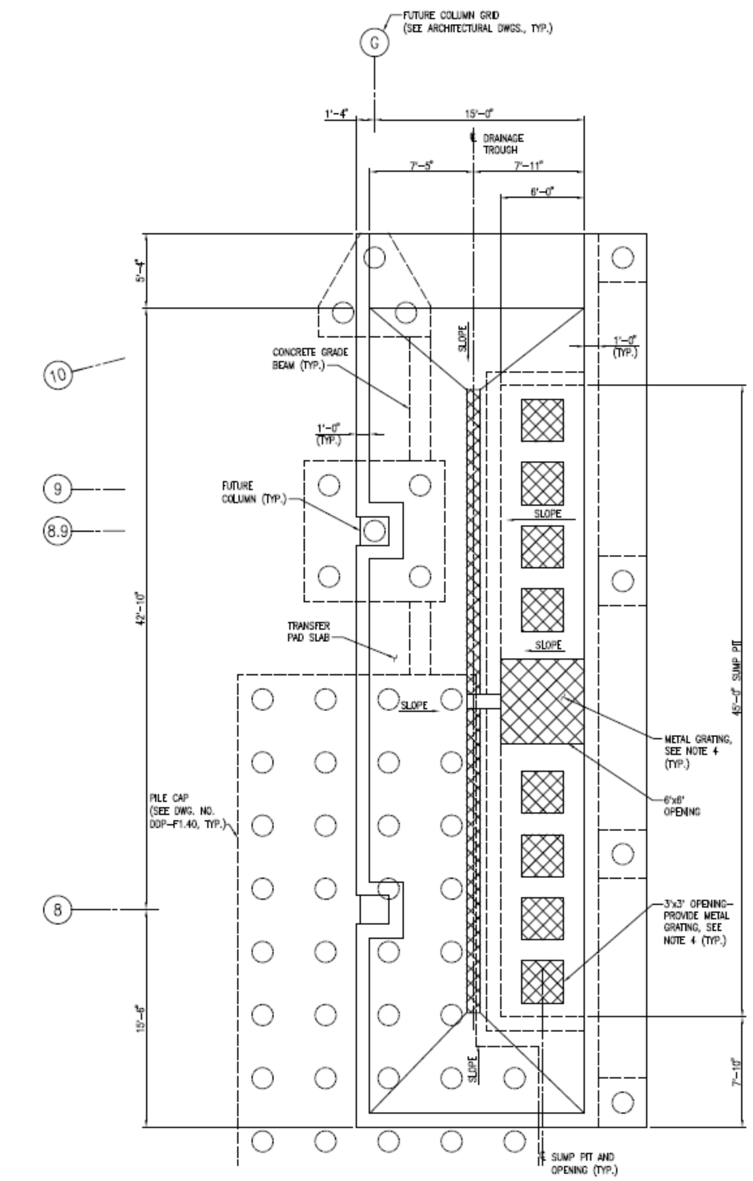
Total Volume available $V_t := V_p + V_{sl} + V_{tr}$ $V_t = 796.4\ ft^3$ $\rightarrow$ $V_t = 5957.2\ gal$



Transfer station location



Sections and Dimensions



Plan dimensions

MUESER RUTLEDGE CONSULTING ENGINEERS			
PROJECT:	EXELON DEVELOPMENT		
CLIENT:	HDP	JOB #:	11896
SKETCH #:	SK-1 (PRELIMINARY SKETCH)		
SKETCH TITLE:	Honeywell Transfer Station		
ISSUED BY:	FL	CHECKED BY:	SY
DATE:	June 14, 2013		

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Consulting Engineers
14 Penn Plaza · 225 West 34th Street · New York, NY 10122
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MEMORANDUM

Date: July 31, 2013
To: Office
From: Daniel George and Felipe Lorca
Re: EE Memo 6 – Slab-on-Grade Development Cap at Central Plaza Garage
Exelon Tower, Trading Floor Garage & Plaza Garage, Baltimore, MD
File: 11896A

Plaza Garage grades call for replacement of the soil cover (min. 30" thickness) with a concrete slab-on-grade, underlain by sufficient Cover Soil to obtain the desired top of slab elevation. The finished slab will be exposed to the environment and will support automobile parking. Styrofoam insulation will be placed below the slab to provide equal or better thermal protection of the MMC synthetic layers. The concrete slab will spread vehicle loads to protect the synthetic layers.

Exhibits
We have attached the following to illustrate our analyses:

Attachment 1 Vulcan 810 Intruder

Calculation 1 Thickness of Thermal Insulation at Plaza Garage
Calculation 2 Vehicular Load Spreading on Slab-on-Grade

References
1. Honeywell Baltimore Works Site. Conceptual Development Plan: Exelon Tower, Trading Floor/Garage and Central Plaza Garage. Honeywell International, Inc: August 29, 2012.
2. Black and Veatch Construction Completion Report for AlliedSignal, Volume I (February 2000)
3. United States American Concrete Institute (ACI). Guide to Thermal Properties of Concrete and Masonry Systems: ACI 122R-02. American Concrete Institute, 2002.
4. ASHRAE Handbook, 1993 Fundamentals with the Permission of the American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc. (ASHRAE), pp. B-9. 1791 Tullie Circle NE, Atlanta, GA 30329.
5. EPRI Soil and Rock Classification for the Design of Ground-Coupled Heat Pump Systems – Field Manual - Cu-6600, Table 3-1.
6. Dow Styrofoam UtilityFitTM XPS 15PSI Extruded Polystyrene Insulation: Product Information. © The Dow Chemical Company. http://msdssearch.dow.com/PublishedLiteratureDOWCOM/dh_007e/0901b8038007ea90.pdf?filepath=styrofoam/pdfs/noreg/179-07944.pdf&fromPage=GetDoc Accessed on 6/11/2013.

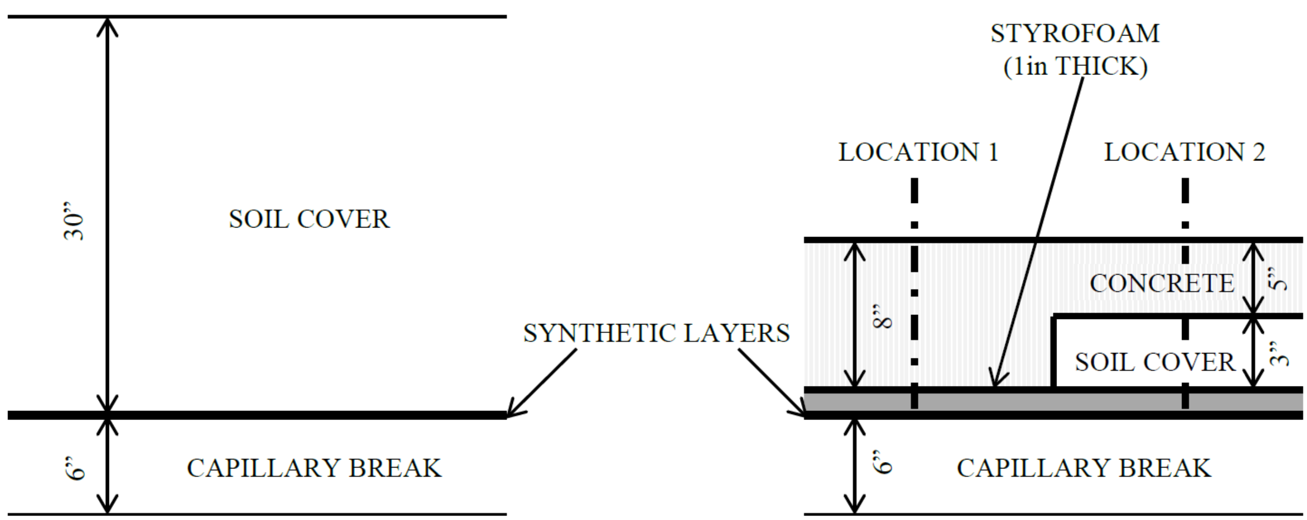
7. Holtz, Robert D., and Kovacs, William D. An Introduction to Geotechnical Engineering. p. 342-343. © 1981 Prentice Hall, Upper Saddle River, NJ.
8. American Association of State Highway and Transportation Officials. *AASHTO LRFD Bridge Design Specifications*. p. 3-24 to 3-25, 3-31 © AASHTO 2012, Washington, D.C.

Thermal Protection Analysis and Assumptions

Thermal Resistance (R-Value) is a measure of the ability of a homogeneous material of unit thickness to resist a temperature difference of one degree Fahrenheit across a unit area (Ref. 3). R-Values are expressed in terms of (ft2*h*°F) / Btu. The assumed R-Values for Cover Soil, Styrofoam, or concrete are (Ref. 4, 5, 6):

- Concrete: R_{conc} = 0.10 per inch
- Cover Soil (sand and gravel): R_{soil} = 0.189 per inch
- Styrofoam: R_{foam} = 5.0 per inch

Existing and future conditions analyzed are shown in Figures 1a and 1b. Thermal resistance analysis was performed for 30” minimum soil cover (assumed sand and gravel) (Figure 1a) and two future cases as shown in Figure 1b. Steel reinforcement was neglected for this analysis, the concrete slab was assumed to be normal weight concrete (150 pcf). Additional soil cover will be left below the Styrofoam, though no additional soil cover was assumed for this analysis.



1a **1b**

Figure 1a and 1b – (a) Existing Conditions, (b) Future Plaza Slab-on-Grade

Findings

The controlling factor to thermal performance is the thickness of Styrofoam used, as its R-Value is high compared to that of soil cover or concrete. The existing 30” of soil cover provides an overall R-Value of 5.67. Both future conditions were analyzed by adding the resistance of each material, assuming the heat has only one path through each system. Analysis performed at Location 1 in Figure 1b at the future Plaza Garage slab haunch resulted in an overall R-Value of 5.80. Similar analysis at Location 2 in Figure 1b through the Plaza Garage slab-on-grade resulted in an overall R-Value of 6.07 (See Table 1). Supporting calculations are provided in Calculation 1.

		EXISTING CONDITIONS		**LOCATION 1**		**LOCATION 2**	
R-Value Parameter / Material	Unit R-Value	Layer Thickness	Equivalent R-Value	Layer Thickness	Equivalent R-Value	Layer Thickness	Equivalent R-Value
	$\frac{ft^2 * h * {}^{\circ}F}{Btu * in}$	Inch	$\frac{ft^2 * h * {}^{\circ}F}{Btu * in}$	Inch	$\frac{ft^2 * h * {}^{\circ}F}{Btu * in}$	Inch	$\frac{ft^2 * h * {}^{\circ}F}{Btu * in}$
Concrete (Ref 4)	0.10	0	0	8	0.8	5	0.5
Cover Soil (Sand and Gravel) (Ref 5)	0.189	30	5.67	0	0	3	0.507
Styrofoam (Ref 6)	5.0	0	0	1	5	1	5
TOTAL:			**5.67**		**5.80**		**6.07**

Table 1 – R-Value Summary

Load Spread Analysis

The bearing stress on the Drainage Net at Locations 1a and 1b was analyzed for the most extreme load conditions beneath the Design Truck, Wheel Loader, and Tow Truck. As discussed in EE Memo 7, bearing stress on the MMC synthetic layers should not exceed 2 ksf, as any higher stress will compromise the flow of the Drainage Net.
The 5-inch thick concrete slab on grade will include steel reinforcing bars, intended to distribute wheel loads even with cracking, facilitating its rehabilitation under a regular repairing cycle.

Design Truck and Wheel Loader

The Design Truck and Wheel Loader were evaluated for bearing stresses to determine if they can be allowed to drive on the finished Plaza Garage Slab (while construction is on-going). They have contact areas with the ground of 8” x 16” and 19.2” x 12.7”, respectively for a single wheel. Applied static plus dynamic loads are 26.6 kips for the Design Truck under a dual wheel and 20.4 kips for the Wheel Loader under a single wheel. Assuming concrete spreads load at a 1:1 ratio and soil spreads load at a 2:1 ratio (Ref. 7), it was determined that neither the Design Truck, nor the Wheel Loader should be permitted to drive on the finished Plaza Garage Slab (See Calculation 2 and Table 2).

Tow Truck

An extreme expected loading condition within the future Plaza Garage was assumed to be the rear axle of a tow truck under static plus dynamic loading while pulling a vehicle, given that emergency vehicle dimensions are bigger than the allowable clearance at the garage. The “Tow Truck” (see Attachment 1) has a maximum operating weight (which includes vehicle and cargo) of 14,500 lbs, with the rear axle supporting 10,000 lbs. The towing hydraulic system has a lift capacity of 4000 lbs. With inclusion of dynamic applied load and lift capacity, the maximum applied load on the rear axle is 18,620 lbs, for a wheel load of 4,655 lbs (four wheels support rear axle). Under this load and using a dual wheel contact area of 15.64” x 12.7” (Calculation 2), it was determined that the Tow Truck will impose bearing

pressures on the MMC synthetic layers of 1.47 ksf and 1.82 ksf at Locations 1 and 2, respectively, each less than 2 ksf (Table 2), not causing undue harm to the MMC synthetic layers.
Under similar loading conditions regarding contact areas, a load of 10.25 kips was calculated as the maximum dynamic impact load for a dual wheel condition, similar to the Tow Truck, which should be permitted to drive on the finished Plaza Garage Slab.

Location	Limit (*ksf*)	Design Truck (*ksf*)	Wheel Loader (*ksf*)	Tow Truck (*ksf*)
Haunch (1)	**2.0**	2.99	2.9	1.47
Slab-on-Grade (2)	**2.0**	3.57	3.54	1.82

Table 2 – Active Vehicle Load Spreading; Bearing Stress at Drainage Net

Conclusions

- The future Plaza Garage will provide sufficient resistance to thermal changes of expansion and contraction and protect the MMC's synthetic layers with 1" Styrofoam insulation.
- Neither the Design Truck, nor the Wheel Loader should be allowed to drive on the slab for the Plaza Garage, based on the load imposed over the MMC synthetic layers.
- Vehicles driving on the Plaza Garage Slab should be limited in weight to no more than that of an active vehicle Tow Truck.

By: ______________________
Daniel George

By: ______________________
Felipe Lorca

810 Intruder

Attachment 1

The Superior Solution to Auto Load Wheel Lifts

The Vulcan 810 Intruder has been specifically designed to fill the needs of private impounders and professional repossessors. The low-profile boom and low-mount planetary winch provide a sleek appearance and superior visibility. The modular design body is adjustable from 88 inch to 94 inch, eliminating the need for fender flares, and includes spacious driver and passenger side tool compartments to provide ample storage for your additional equipment. The proven hydraulic auto load wheel lift system provides for quick and easy operation, even when hooking up parallel parked vehicles. Contact your local Vulcan Distributor for more information on the sleek and stylish Vulcan 810 Intruder.

Innovative. Durable.
Brutally Tough.

Innovative. Durable. Brutally Tough.

Specifications

BOOM

Recovery Boom (at boom end swivel) .... 8,000 lbs.
Maximum Lift Angle .... 21°
Winch (Planetary) .... 8,000 lbs.
Cable .... 3/8" x 100'

UNDERLIFT

Lift Capacity Extended .... 4,000 lbs.
Tow Rating .... 7,500 lbs.
Maximum Reach .... 73"
Optional Power Tilt .... 30° Arc

CHASSIS RECOMMENDATIONS

Minimum C.A. (Cab to Axle) .... 60"
Maximum C.A. (With Tunnel Tool Box) .... 84"
Suggested GVWR .... 14,500 lbs.

Standard Features

- 60" C.A. Steel Modular Body
- Adjustable Body Width, 88" or 94"
- Auto Load Wheel Lift System
- 180° Pivoting Crossbar
- Self Centering Crossbar
- Tailboard Safety Chain Pockets
- Safety Chains
- Safety Chain Pocket Guards
- Wheel Lift Ratchets & Straps
- Driver & Passenger Side Tool Compartments
- In Cab Wheel Lift & Winch Controls
- 8,000 lb. Planetary Winch with 100' of 3/8" Cable
- Junction Box and Wiring Harness
- Mud Flaps
- Federal Standard 108 Lighting
- Tire Spacers (Flat Tires)

Optional Fea-

- Aluminum or Composite Modular Body
- Power Tilt
- Power Tilt with Adjustable limiter switches on elevation & tilt functions
- Steel Light Pylon
- Aluminum Tubular Light Pylon
- 24" Tunnel Tool Box (Steel, Aluminum or Composite)
- Trailer Hitch Attachment
- Motorcycle Attachment
- Dress Up Packages
- Emergency Lighting
- Dollies & Mounting Options
- Clutch Pump or Pump/PTO

The Optional Power Tilt feature on the Intruder allows the wheel lift to operate through a 30-degree arc, which provides easier hook-ups on driveways or sloped roads.

All ratings are based on structural factors only, not vehicle capacities or capabilities. Specifications shown are approximations and may vary depending on chassis selected. Miller Industries Towing Equipment Inc. reserves the right to change or modify product and or specifications without notice or obligation. Some equipment shown is optional.

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		File:	11896A
	Made By: DJG	Date:	6/7/2013
FOR: EXELON	Checked By: FL	Date:	7/25/2013

SUBJECT: Calculation 1: Thickness of Thermal Insulation at Plaza Garage

Thermal protection of synthetic layers is currently provided by a minimum of 30" of soil cover. Soil cover is assumed composed of sand and gravel. Analysis below compares thermal resistance of existing soil cover with future Plaza Garage at Locations 1 and 2. Future Plaza Garage at Location 1 (see Figure 1b) encounters an 8" concrete haunch (t_{haunch}), underlain by molded polystyrene (Styrofoam) (t_{sty}). Future Plaza Garage at Location 2 (see Figure 1b) encounters a 4" concrete slab on grade (t_{conc}) underlain by a minimum of 4" soil cover (t_{soil}) and Styrofoam (t_{sty}).

EXISTING MMC:

$$R_{soil} = k_{soil}^{-1} * \frac{1\ ft}{12\ in}$$

Thermal Resistance of Sand and Gravel Per Inch Thickness (Ref. 5)

Where:

$$k_{soil} = 0.44 \frac{Btu}{ft * h * °F}$$

Thermal Conductivity of Sand and Gravel

$$R_{soil} = \frac{1}{k_{soil} * 12\ in} = 0.189 \frac{ft^2 * h * °F}{Btu * in}$$

Thermal Resistance of Sand and Gravel per Inch

$$R_{soil} * 30\text{ in. Cover Soil} = 5.67 \frac{ft^2 * h * °F}{Btu * in}$$

Thermal Resistance of Minimum Cover Soil

PLAZA GARAGE SLAB:

Component Thermal Resistance:

$$R_{haunch} = 0.10 \frac{ft^2 * h * °F}{Btu * in}$$

Thermal Resistance of Haunch (concrete) Per Inch Thickness (Ref. 4)

$$R_{conc} = 0.10 \frac{ft^2 * h * °F}{Btu * in}$$

Thermal Resistance of Concrete Per Inch Thickness (Ref. 4)

$$R_{soil} = 0.189 \frac{ft^2 * h * °F}{Btu * in}$$

Thermal Resistance of Sand and Gravel Per Inch Thickness

$$R_{sty} = 5.0 \frac{ft^2 * h * °F}{Btu * in}$$

Thermal Resistance of Styrofoam Per Inch Thickness (Ref. 6)

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FOR: EXELON | Made By: DJG | Date: 6/7/2013

Checked By: FL | Date: 7/25/2013

SUBJECT: Calculation 1: Thickness of Thermal Insulation at Plaza Garage

Total Thermal Resistance at Location 1:

$$R_t = R_{haunch}*t_{haunch} + R_{sty}*t_{sty} = (0.10)*(8\text{ in}) + (5.0)*(1\text{ in}) = 5.80\ \frac{ft^2 * h * °F}{Btu}$$

Total Thermal Resistance at Location 2:

$$R_t = R_{conc}*t_{conc} + R_{soil}*t_{soil} + R_{sty}*t_{sty} = (0.10)*(5\text{ in}) + (0.189)*(3\text{ in}) + (5.0)*(1\text{ in}) = 6.07\ \frac{ft^2 * h * °F}{Btu}$$

Location 1	**5.80 > 5.67**
Location 2	**6.07 > 5.67**

Analysis at both Locations 1 and 2 shows the future Plaza Garage will provide sufficient resistance to thermal changes of expansion and contraction and protect the MMC's synthetic layers with 1" Styrofoam insulation.

MUESER RUTLEDGE CONSULTING ENGINEERS File: 11896A

Made By: DJG Date: 6/28/2013

FOR: Exelon Checked By: FL Date: 7/25/2013

SUBJECT: Calculation 2: Vehicular Load Spreading on Slab-on-Grade

Determine if Design Truck, Wheel Loader, and/or Tow Truck are allowed to drive on Plaza Garage Slab-on-Grade (See EE Memo 7 for calculation of Static and Dynamic Loads, wheel/axle layout and Contact Areas):

$\sigma_{MMC} := 2\text{ksf}$ Maximum Allowable Bearing Pressure on MMC Synthetic Layers

Location 1 (See Figure 1b): 8" Concrete, 0" Cover Soil, 1" Styrofoam = 9" depth to MMC synthetic layers.

Location 2 (See Figure 1b): 5" Concrete, 3" min Cover Soil, 1" Styrofoam = 9" depth to MMC synthetic layers.

Design Truck:

$w_{DT} := 24\text{in}$ $l_{DT} := 16\text{in}$ Dimensions of Contact with Slab of a Dual Wheel (8" x 16" each, 8" apart)

$A_{DT} := w_{DT} \cdot l_{DT}$ $A_{DT} = 2.67\,\text{ft}^2$ Contact Area of a DualWheel

$P_{DT} := 1.33 \cdot 20\text{kip}$ $P_{DT} = 26.6\,\text{kip}$ Maximum Applied Static plus Dynamic Load per Wheel

Wheel Loader:

$w_{WL} := 1.60\text{ft}$ $l_{WL} := 1.06\text{ft}$ Dimensions of Contact with Slab of a Single Wheel (19.2" x 12.7")

$A_{WL} := w_{WL} \cdot l_{WL}$ $A_{WL} = 1.7\,\text{ft}^2$ Contact Area of a Single Wheel

$P_{WL} := 20.38\text{kip}$ Maximum Applied Static plus Dynamic Load per Wheel

Assume a 45 degree, 60 degree, and 90 degree load spreading through concrete slab, Cover Soil, and 1" Styrofoam, respectively (Ref. 7).

Load Contact Areas - Design Truck:

Location 1:

$A_{c1DT} := A_{DT}$ $A_{c1DT} = 2.67\,\text{ft}^2$ Contact Area of a Dual Wheel on Slab

$A_{sty1DT} := (w_{DT} + 2 \cdot 8\text{in}) \cdot (l_{DT} + 2 \cdot 8\text{in})$ Contact Area of a Dual Wheel on Styrofoam

$A_{sty1DT} = 8.89\,\text{ft}^2$ Contact Area of a Dual Wheel on MMC Synthetic Layers

MUESER RUTLEDGE CONSULTING ENGINEERS File: 11896A

Made By: DJG Date: 6/28/2013

FOR: Exelon Checked By: FL Date: 7/25/2013

SUBJECT: Calculation 2: Vehicular Load Spreading on Slab-on-Grade

Load Contact Areas - Design Truck (cont'd):

Location 2:

$A_{c2DT} := A_{DT}$ $A_{c2DT} = 2.67\,ft^2$ Contact Area of a Dual Wheel on Slab

$A_{cs2DT} := (w_{DT} + 2 \cdot 5in) \cdot (l_{DT} + 2 \cdot 5in)$ $A_{cs2DT} = 6.14\,ft^2$ Contact Area of a Dual Wheel on Cover Soil

$A_{sty2DT} := (w_{DT} + 2 \cdot 5in + 2 \cdot 1.5in) \cdot (l_{DT} + 2 \cdot 5in + 2 \cdot 1.5in)$ Contact Area of a Dual Wheel on Styrofoam

$A_{sty2DT} = 7.45\,ft^2$ Contact Area of a Dual Wheel on MMC Synthetic Layers

Load Contact Areas - Wheel Loader:

Location 1:

$A_{c1WL} := A_{WL}$ $A_{c1WL} = 1.7\,ft^2$ Contact Area of a Single Wheel on Slab

$A_{sty1WL} := (w_{WL} + 2 \cdot 8in) \cdot (l_{WL} + 2 \cdot 8in)$ Contact Area of a Single Wheel on Styrofoam

$A_{sty1WL} = 7.02\,ft^2$ Contact Area of a Single Wheel on MMC Synthetic Layers

Location 2:

$A_{c2WL} := A_{WL}$ $A_{c1WL} = 1.7\,ft^2$ Contact Area of a Single Wheel on Slab

$A_{cs2WL} := (w_{WL} + 2 \cdot 5in) \cdot (l_{WL} + 2 \cdot 5in)$ $A_{cs2WL} = 4.61\,ft^2$ Contact Area of a Single Wheel on Cover Soil

$A_{sty2WL} := (w_{WL} + 2 \cdot 5in + 2 \cdot 1.5in) \cdot (l_{WL} + 2 \cdot 5in + 2 \cdot 1.5in)$ Contact Area of a Single Wheel on Styrofoam

$A_{sty2WL} = 5.75\,ft^2$ Contact Area of a Single Wheel on MMC Synthetic Layers

SUBJECT: Calculation 2: Vehicular Load Spreading on Slab-on-Grade

Bearing Pressures at MMC Synthetic Layers - Design Truck:

Location 1:

$P_{DT} = 26.6\,kip$

$\sigma_{1DT} := \frac{P_{DT}}{A_{sty1DT}}$ $\sigma_{1DT} = 2.99\,ksf$ $2.99ksf > 2ksf$

Therefore, Design Truck not allowed at Location 1 - Bearing pressure exceeds 2 ksf at MMC Synthetic Layers.

Location 2:

$P_{DT} = 26.6\,kip$

$\sigma_{2DT} := \frac{P_{DT}}{A_{sty2DT}}$ $\sigma_{2DT} = 3.57\,ksf$ $3.57ksf > 2ksf$

Therefore, Design Truck not allowed at Location 2 - Bearing pressure exceeds 2 ksf at MMC Synthetic Layers.

Bearing Pressures at MMC Synthetic Layers - Wheel Loader:

Location 1:

$P_{WL} = 20.38\,kip$

$\sigma_{1WL} := \frac{P_{WL}}{A_{sty1WL}}$ $\sigma_{1WL} = 2.9\,ksf$ $2.9ksf > 2ksf$

Therefore, Wheel Loader not allowed at Location 1 - Bearing pressure exceeds 2 ksf at MMC Synthetic Layers.

Location 2:

$P_{WL} = 20.38\,kip$

$\sigma_{2WL} := \frac{P_{WL}}{A_{sty2WL}}$ $\sigma_{2WL} = 3.54\,ksf$ $3.54ksf > 2ksf$

Therefore, Wheel Loader not allowed at Location 2 - Bearing pressure exceeds 2 ksf at MMC Synthetic Layers.

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File: 11896A

Made By: DJG Date: 6/28/2013

FOR: Exelon Checked By: FL Date: 7/25/2013

SUBJECT: Calculation 2: Vehicular Load Spreading on Slab-on-Grade

Tow Truck - See EE Memo 7 text for wheel/axle layout:

$W_o := 14500\text{lbf}$	Tow Truck Operating Weight
$W_f := 4500\text{lbf}$	Front Axle Weight
$W_r := 10000\text{lbf}$	Rear Axle Weight
$W_p := 4000\text{lbf}$	Maximum Lift Capacity - Extended
$W_{rear} := W_r + W_p$	
$W_{rear} = 14\,\text{kip}$	Maximum Static Load on Rear Axle

Dynamic Applied Stress Calculation - Tow Truck (Ref. 8):

$D_E := 0$	Embedment Depth of Applied Load
$IM := 33\cdot(1 - 0.125\cdot D_E)$	Dynamic Load Allowance for Drainage Net (Additional Percentage of Static Response Applied at Grade)
$IM = 33$	
$W_{dTT} := \dfrac{IM}{100}\cdot W_{rear}$	
$W_{dTT} = 4.62\,\text{kip}$	Additional Allowable Dynamic Load
$W_{TT} := W_{rear} + W_{dTT}$	
$W_{TT} = 18.62\,\text{kip}$	Static plus Dynamic Applied Load at Grade from the Tow Truck
$P_{TT} := \dfrac{W_{TT}}{4}$ $\quad P_{TT} = 4.66\,\text{kip}$	Maximum Load per Wheel on Dual Wheel Rear Axle (4 wheels total)
$w_{TT} := \dfrac{P_{TT}}{0.8\dfrac{\text{kip}}{\text{in}}}$	Width of Contact Area of Wheel (Ref. 8)
$w_{TT} = 0.485\,\text{ft}$	
$\gamma := 1.50$	Load Factor (Ref. 8)
$l_{TT} := 6.4\gamma\cdot\left(1\text{in} + \dfrac{IM\cdot 1\text{in}}{100}\right)$	
$l_{TT} = 1.06\,\text{ft}$	Length of Contact Area of Wheel (Ref. 8)

Dynamic Applied Stress Calculation - Tow Truck (cont'd):

$A_{TT} := (2w_{TT} + 4in) \cdot l_{TT}$ $A_{TT} = 1.39\ ft^2$ Contact Area of a Dual Wheel, Considering 4" of Separation Between Wheels

$P_{TT2} := 2 \cdot P_{TT}$ $P_{TT2} = 9.31\ kip$ Maximum Applied Load

Load Contact Areas - Tow Truck:

Location 1:

$A_{c1TT} := A_{TT}$ $A_{c1TT} = 1.39\ ft^2$ Contact Area of a Single Wheel on Slab

$A_{sty1TT} := (2w_{TT} + 4in + 2 \cdot 8in) \cdot (l_{TT} + 2 \cdot 8in)$ Contact Area of a Single Wheel on Styrofoam

$A_{sty1TT} = 6.32\ ft^2$ Contact Area of a Single Wheel on MMC Synthetic Layers

Location 2:

$A_{c2TT} := A_{TT}$ $A_{c2TT} = 1.39\ ft^2$ Contact Area of a Single Wheel on Slab

$A_{cs2TT} := (2w_{TT} + 4in + 2 \cdot 5in) \cdot (l_{TT} + 2 \cdot 5in)$ Contact Area of a Single Wheel on Cover Soil

$A_{cs2TT} = 4.05\ ft^2$

$A_{sty2TT} := (2w_{TT} + 4in + 2 \cdot 5in + 2 \cdot 1.5in) \cdot (l_{TT} + 2 \cdot 5in + 2 \cdot 1.5in)$ Contact Area of a Single Wheel on Styrofoam

$A_{sty2TT} = 5.12\ ft^2$ Contact Area of a Single Wheel on MMC Synthetic Layers

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File: 11896A

Made By: DJG Date: 6/28/2013

FOR: Exelon Checked By: FL Date: 7/25/2013

SUBJECT: Calculation 2: Vehicular Load Spreading on Slab-on-Grade

Bearing Pressures at MMC Synthetic Layers - Tow Truck:

Location 1:

$P_{TT2} = 9.31\,\text{kip}$

$\sigma_{1TT} := \dfrac{P_{TT2}}{A_{sty1TT}}$ $\sigma_{1TT} = 1.47\,\text{ksf}$ $1.47\text{ksf} < 2\text{ksf}$

Therefore, Tow Truck is allowed at Location 1 - Bearing pressure is less than 2 ksf at MMC Synthetic Layers.

Location 2:

$P_{TT2} = 9.31\,\text{kip}$

$\sigma_{2TT} := \dfrac{P_{TT2}}{A_{sty2TT}}$ $\sigma_{2TT} = 1.82\,\text{ksf}$ $1.82\text{ksf} < 2\text{ksf}$

Therefore, Tow Truck is allowed at Location 2 - Bearing pressure is less than 2 ksf at MMC Synthetic Layers.

The Maximum Allowable Load over the slab, if considering similar loading areas to the Tow Truck will be:

Location 1: $P_{max1} := 2\text{ksf} \cdot A_{sty1TT}$ $P_{max1} = 12.64\,\text{kip}$

Location 2: $P_{max2} := 2\text{ksf} \cdot A_{sty2TT}$ $P_{max2} = 10.25\,\text{kip}$

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MEMORANDUM

Date:	July 16, 2013
To:	Office
From:	Daniel George and Adam M. Dyer
Re:	EE Memo 7 – Construction Vehicle Load Spreading Analysis and Road Layout Exelon Tower, Trading Floor Garage & Plaza Garage, Baltimore, MD
File:	11896A

MRCE has reviewed available information for the Harbor Point Development project and static and dynamic construction loads at the Multimedia Cap (MMC) synthetic layers. The purpose of this evaluation is to determine if these loads cause instability or excessive pressure at the synthetic layers, or if additional fill or other protection is needed to protect the MMC synthetic layers.

Exhibits

We have attached the following to illustrate our analyses:

Attachment 1 — Drawing No. I-1 - “Criteria for Interim Use Harbor Point Site Area 1 West of Wills St.” Dated: September 10, 2003.

Attachment 2 — WINSTRESS Runs – Existing Conditions:
- Static Load Spreading of Design Truck
- Static & Dynamic Load Spreading of Design Truck
- Static Load Spreading of Wheel Loader
- Static & Dynamic Load Spreading of Wheel Loader
- Static Load Spreading of 16,380 Gallon Double-Wall Tank
- Static Load Spreading of 25 Yard Roll-off Box with Aluminum Hard Top

Attachment 3 — JCB Wheel Loader 457 ZX

Attachment 4 — Adler 16,380 Gallon Double Wall Tank

Attachment 5 — Adler 25 Yard Roll-off Box with Aluminum Hard Top

Attachment 6 — Drawing No. DDP-F1.08 – “Construction Access Roads” Dated: June 26, 2013

Attachment 7 — WINSTRESS Runs – Asphalt:
- Static Load Spreading of Design Truck
- Static & Dynamic Load Spreading of Design Truck
- Static Load Spreading of Wheel Loader
- Static & Dynamic Load Spreading of Wheel Loader

Calculation 1 — Static, Dynamic, and Soil Load Application Calculations

Calculation 2 — Water and Soil Containers Applied Load Calculations

Calculation 3 — MMC Bearing Capacity under Design Truck

Calculation 4 — Load on Drainage Net from Modu-Tanks

References

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Multimedia Cap and Underlying Materials

The soil cover present at Area 1 is 30" above the MMC synthetic layers. This thickness of soil was assumed to exist across the site. The top 6" is a crushed stone (CR-6) and the underlying materials are sand and gravel aggregates (Cover Soil). The Geomembrane is protected by a Drainage Net and Cover Geotextile above, and by a GCL and Cushion Geotextile below. The synthetic layers are underlain with compacted crushed stone and controlled fill. The primary concern of the operation of construction access roads is the transmission of construction loads through the soil cover, crushing the MMC synthetic layers, thereby reducing water transmissivity of the Drainage Net. Additional concerns include the bearing capacity of soil cover, and road serviceability and rutting due to frequent construction vehicle use.

Previous Evaluation

In 2003, MRCE provided Interim Use Notes for Site Development of Harbor Point Area 1, which restricted the allowable applied bearing stress at the MMC synthetic layers to 2 ksf (Attachment 1). Laboratory compression test data for the Drainage Net indicates its ability to convey water is compromised above a bearing stress of 2 ksf (Ref. 1).

MRCE's Interim Use Notes limited vehicles to a fully loaded 15 cubic yard (cy) concrete truck (will be referred as the "Design Truck"); highway permitted HS-20 trucks weigh less than that maximum (Ref. 3). This allowance was based on the distribution of wheel loads to stresses below 2 ksf at the 30" depth of the synthetic layers.

Load Spreading Analysis

Calculations of bearing stress at the Drainage Net were performed using WINSTRESS Version 1.0, released in September 2001 by Prototype Engineering, Inc. WINSTRESS is an elastic stress analysis program which applies surface loads on a semi-infinite mass. Output from this program is similar to an application of the 2:1 method of load approximation with depth (Ref. 4).

Bearing Stress at MMC Synthetic Layers

Design Truck

The Design Truck has contact with the ground with one single wheel 20-kip axle, 14' from two dual wheel 40-kip axles spaced 4.5 feet apart, for a total fully loaded weight of 100 kips (Ref. 2). Each wheel has a contact area with the ground of 128 in^2, for a contact pressure under static load of 78 psi (11.25 ksf). Dynamic loading adds an additional 33% of static loading for a total of 103 psi (14.96 ksf) (Calculation 1). The bearing stress felt at the Drainage Net under static and static plus dynamic loading is 1.15 and 1.53 ksf, less than the limit of 2 ksf (using WINSTRESS – Attachment 2).

Wheel Loader

The Wheel Loader (JCB Wheel Loader 457 ZX- Attachment 3) will subject the MMC synthetic layers to heavy loads when unloading delivery vehicles and at soil stockpile areas. The Wheel Loader has contact with the MMC with a two – two single wheel rubber tire axles. When combined with a maximum payload of 12 kips, the front axle carries 30.6 kips. These wheels each have a static contact pressure of 62.7 psi (9.02 ksf). With an additional dynamic load of 33%, contact pressure increases to 83.3 psi (12.0 ksf). The bearing stress at the Drainage Net under these loads is 1.05 and 1.39 ksf, each less than 2 ksf (Attachment 2).

Clean Soil Stockpile Area

A typical earth fill weighs 125 pcf. Approximately 16 feet of earth fill will apply 2 kips per square foot (ksf). Given the 30” of soil cover now in place, earth fill should be limited to 13.5 ft. The maximum earth fill load is at Wills Street, south of the Dock Street. intersection. Fill in this area is less than 10 feet thick. Soil stockpiles placed on the MMC should be limited to no more than 12 feet.

Track Cranes

Large track cranes will be used for pile driving. The toe pressure of the crane tracks under load must be spread by timber mats to an area load which will introduce no more than 2 ksf stress at the synthetic layers. Toe pressure and mat sizes must be determined before track cranes operate on the site.

Stormwater Storage Modu-Tanks

As described in EE Memo #2, stormwater pumped from excavations will be stored in Modu-tanks roughly 4 feet deep and 75 feet square capable of storing up to 150,000 gallons of impacted water. The Modu-tanks will have an approximately uniform bearing pressure at the drainage net of approximately 0.113 tsf which is less than the 1 tsf allowable, as shown on Calculation 4.

Water and Soil Container Load Spreading

Water will be temporarily stored in a 16,380 Gallon Double-Wall Tanks, which have contact with the ground by four 4" wide skids in both transverse and longitudinal directions (Attachment 4), with a fully loaded capacity of 175,000 lbs (Calculation 2). The bearing pressure was assumed to be uniform along the skids. The skids have a contact area with the ground of 6464 in^2, for a contact pressure of 27.1 psi (3.90 ksf). The tanks will remain in place and are emptied and lifted to a single axle for moving.

Contaminated soil may be stored in 25 Yard Roll-off Box with Aluminum Hard Top, which has contact with the ground by four 8" x 10" wheels and two 2" wide, 22' long skids (Attachment 5). The approximate weight at capacity is 90,000 lbs (Calculation 2). The assumption was made that load will be distributed evenly by the skids and wheels. The skids and wheels have a contact area with the ground of 1200 in^2, for a contact pressure of 75 psi (10.80 ksf).

The stress felt at the Drainage Net from the bearing pressure of the water tank and soil box are 0.74 and 0.53 ksf, respectively. These loads are less than that of the Design Truck. Each of these stresses is less than the limiting value of 2 ksf. The container exerts a high bearing stress on the MMC surface when the container is hoisted onto the truck carriage. The CR-6 surface may rut under these high bearing pressures. Ruts should be regarded and the MMC surface should be compacted to repair ruts. Asphalt, concrete pavement, or mats should be used where loaded containers are stored and frequently transferred to/from the truck carriage. Both containers should be located where settlement of compressible strata is not a concern.

Bearing Capacity at MMC Synthetic Layers

A bearing capacity analysis was performed of the Design Truck's wheel load (static plus dynamic) (Calculation 3), considered more critical than the Wheel Loader. The cover soil has a safety factor of 8.3 against bearing capacity failure at the depth of the MMC synthetic layers. The MMC provides a stable environment for supporting the synthetic layers under the planned construction equipment loads.

Construction Road Layout

A layout of construction access roads has been generated to provide a materials delivery loop and stabilized access to all future pile locations (Attachment 6). Construction roads should have a minimum turn radius of 48 feet for truck turns (Ref. 3, 5).

Construction vehicles will access the site through an existing gate at the intersection of Dock Street and Caroline Street and travel along a two lane (30' total width), two way primary construction road to the west end of the site. Deliveries should be made to a materials laydown and soil stockpile area located west of the Exelon tower on Area 1. Concrete barriers should be used to prevent vehicle damage to existing site infrastructure.

Vehicle speeds should be limited to 15 miles per hour to limit dynamic load application to the MMC synthetic layers.

The concrete bridge slab over the perimeter barrier will be placed along the Dock Street alignment, and some of Wills Street after the sheet pile is inserted to augment the barrier. The bridge slab should be designed to carry the Design Truck where it lies below the construction road alignment.

Construction Road Pavement Design

Equivalent Single Axle Loads

Major concerns for a construction road are serviceability and protection against rutting and erosion, in addition to wheel loads (Ref. 6). If an 18-kip single axle is used as a basis for construction road design, the estimated number of equivalent single axle loads (ESAL's) that will pass along this route is 10 per hour, considering all types of construction and personal vehicles. Assuming a site work schedule of 10 hour work days, 6 days per week, and 52 weeks per year, 31,200 ESAL's can be expected to pass along a section of construction road each year. The construction road can be considered a low-volume industrial road (Ref. 7).

Asphalt Construction Access Roads

In order to mitigate dust and reduce maintenance from the frequent passage of construction vehicles, asphalt should be used as a wearing surface for construction roads. Due to the presence of CR-6 as a good existing subgrade (CBR> 20), a compacted 5" minimum of asphalt should be used. The asphalt should be comprised of single lifts of compacted 2" minimum of 12.5 MM (0.5 in) Superpave as surface course and compacted 3" minimum of 19 MM (0.75 in) Superpave as base course, separated by tack coat. MM refers to the maximum size aggregate that can be used. The road should be crowned with a minimum slope of 1.5% per foot and toward the perimeter of the site, limiting sheet flow run-on from flowing into the site. Hot mix asphalt shall be designed, mixed, and constructed in accordance with Maryland State Highway Administration Standard Specifications for Construction and Materials. No stipulations for drainage are recommended, but may be required should ponding become an issue (See EE Memo 2 – Storm Water Storage Demand).

With the addition of 5" asphalt, bearing stress at the MMC synthetic layers due to static and static plus dynamic loading drops, as shown in Tables 1 and 2 and in Attachment 7.

Bearing Stress at Drainage Net (ksf)	**Limit**	**Static**	**Static + Dynamic**
Existing Conditions (30" Soil Cover)	**2.0**	1.15	1.53
30" Soil Cover plus 5" Asphalt	**2.0**	0.99	1.30

Table 1 – Bearing Stress at Drainage Net under Design Truck with and without Asphalt

Bearing Stress at Drainage Net (ksf)	**Limit**	**Static**	**Static + Dynamic**
Existing Conditions (30" Soil Cover)	**2.0**	1.05	1.39
30" Soil Cover plus 5" Asphalt	**2.0**	0.86	1.12

Table 2 – Bearing Stress at Drainage Net under Wheel Loader with and without Asphalt

Conclusions:

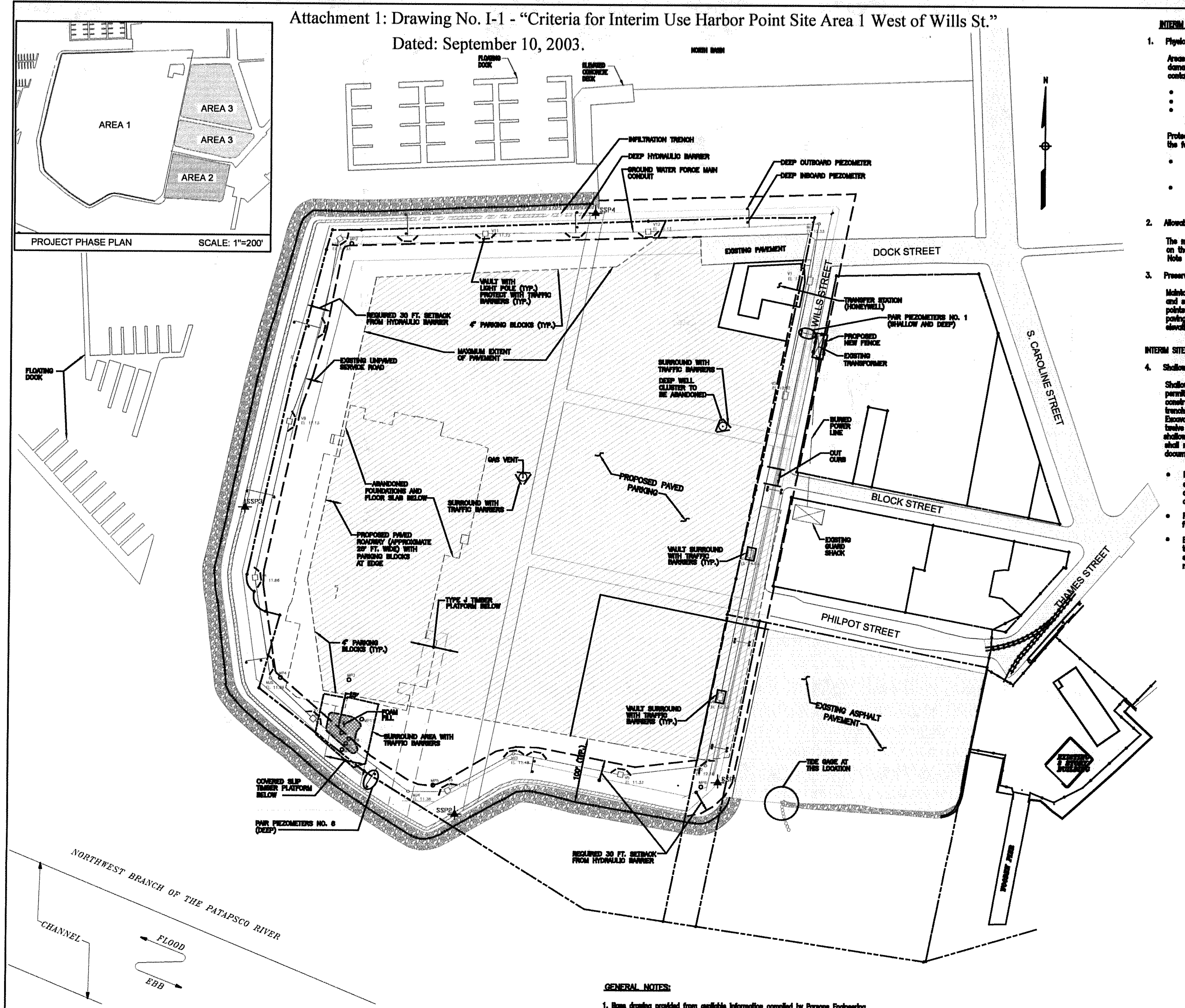
- The Drainage Net's flow capacity is compromised above a bearing stress of 2 ksf.
- All construction access roads should be composed of 5" asphalt to support concentrated loads from construction vehicles.
- Clean soil stockpiles should be limited to no higher than 13.5 feet above existing grade.
- Bearing stress applied by track cranes at the MMC synthetic layers should be limited to 2 ksf.
- Water and soil containers should be located on asphalt, concrete pad, or mats where they may be lifted up or removed.

By: Daniel J. George

By: Adam M. Dyer

DJG:PWD\11896A-40\Construction Vehicle Load Spreading Analysis and Road Layout

Attachment 1: Drawing No. I-1 - "Criteria for Interim Use Harbor Point Site Area 1 West of Wills St."
Dated: September 10, 2003.



INTERIM USE NOTES FOR PROTECTION OF EXISTING SITE INFRASTRUCTURE

1. Physical Protection:

   Areas and items below are to be physically protected against damage. Protection will be by traffic barriers, concrete block, self-contained free-standing planters, or similar items:

   - HMS vaults which extend above ground surface
   - Gas Vent
   - Lightweight geofoam fill area over Covered Slip outboard of former Building 23.

   Protection by means other than physical barriers are required for the following:

   - Covers to wells and equipment at all locations, shall be locked or tightly bolted closed to protect against tampering.
   - Place 4" parking blocks along edge of pavement to keep vehicular traffic on paved surface. Emergency vehicle access shall not be impeded.

2. Allowable Applied Load Intensity

   The maximum allowable applied load intensity (static or dynamic) on the multimedia cap is 2,000 psf. For traffic limitation, see Note 8.

3. Preservation Of Site Sampling And Monitoring Points

   Maintain availability of site monitoring locations, sampling locations and survey points. Interim use construction and paving over survey points is prohibited, unless replaced in kind or extended through the paving, as approved by Honeywell. Survey monitoring point elevations immediately before and after replacement.

INTERIM SITE IMPROVEMENTS

4. Shallow Trenching and Excavations

   Shallow trenches or local excavations within the cover soil are permitted for installing drainage chases, water lines, utilities, or constructing slabs flush with the pavement surface, etc. All trenching and excavation shall be approved by Honeywell. Excavations over the Multimedia Cap shall be limited in depth to twelve inches below the bottom of the crushed stone cover, or shallower if the visual barrier is encountered. The visual barrier shall not be penetrated. All excavations shall be inspected, and documented in depth and extent and transmitted to Honeywell.

   - Excavations are to be performed by smooth bucket equipment (no plate welded across teeth), under the direct observation of a person approved by Honeywell or designated representative.
   - Remove the stone cover down to the geotextile. Cut and fold back the geotextile. Replace geotextile upon completion.
   - Excavation spoils shall be stockpiled on heavy duty plastic tarps, and removed from the site, or returned to the excavation and compacted to 95% Standard Proctor maximum density.

5. Paving Site Areas

   Parking areas and access roads shall be paved. Concrete and asphalt pavement may be placed directly on the crushed stone surface. The paved areas shall be designed to provide storm drainage by sheet flow off cap without causing erosion of cover materials. Crushed stone or concrete water flow diffusers shall be located in area of erosion as needed. Manholes, other flush mounted access openings, settlement plates and equipment shall be raised with the asphalt pavement as approved by Honeywell. Surveys shall be performed before and after raising settlement plates and resets shall be indicated in the O & M records. All well covers in paved areas shall be designed to accommodate the design traffic load and exclude surface water.

6. Lights

   Light poles may be installed on the cap provided the limitations on excavation and bearing intensity are met, and the light poles are stable. Light poles may be fixed to concrete blocks, or flush mounted slabs as desired. Foundation designs for poles shall be approved by Honeywell.

7. Waterfront Improvements

   Not applicable for interim use.

USE LIMITATIONS

8. Vehicles shall typically operate on paved areas. Occasional use on unpaved areas is permitted. Vehicles are limited to a fully loaded 15 cubic yard capacity concrete truck. Highway permitted HS 20 trucks weigh less than that maximum, and are therefore permitted. Concrete trucks shall not clean out or wash down equipment on Area 1, 2, or 3 Cap. Crane pads shall be supported on mats to limit loads in accordance with Note 2.

9. Plants, bushes, trees, etc. shall be set in planters with sides and bottoms impenetrable to roots.

10. Permanent and semi-permanent generators or other motorized equipment requiring refueling shall be placed on pre-cast or cast-in-place concrete pads with a perimeter lip to retain spilled fuel. During refueling, temporary equipment on asphalt shall be surrounded by a sorbent coil appropriate for the type of fuel used. Temporary equipment requiring refueling on unpaved areas shall be placed on a temporary petroleum resistant plastic or concrete containment with perimeter lip, during refueling. Should a spill of any measurable quantity occur, the spill shall be fully cleaned using sorbent towels and other measures as necessary. Honeywell shall be notified of spills. Used sorbent materials shall be disposed of properly. Fuel containers shall be stored on paved areas with containment berms. LPG bottles have no storage or use limitations.

11. Stakes or spikes which will be driven into the ground shall be no longer than 12 inches. Fence posts shall not be driven into the ground, but must be supported by concrete block bases or other

12. Periodic inspections shall be made by Honeywell or designated representative to verify compliance with these criteria.

LEGEND

- MULTIMEDIA CAP
- DOUBLE - LAYER SYNTHETIC DRAINAGE COMPOSITE
- INFILTRATION TRENCH
- METHANE GAS VENT
- SETTLEMENT PLATE
- TOE DRAIN COLLECTION PIPING
- RIP-RAP
- SOIL-BENTONITE BARRIER
- SETBACK FROM SOILBENTONITE AND HMS COMPONENTS
- PIEZOMETER SYSTEM
- MANHOLE JUNCTION
- VAULT JUNCTION WITH PUMPING WELL
- APPROX. LIMIT OF BUILDINGS
- CAP COVER SOIL SAMPLING LOCATION
- DRAINAGE LAYER SAMPLING LOCATION
- 4" PARKING BLOCKS
- EXISTING UNPAVED SERVICE ROAD
- PROPOSED PAVED PARKING/ROAD WAY
- APPROX. LIMIT OF AREAS OF KNOWN OR POTENTIAL SETTLEMENT CONCERN
- APPROX. LIMIT OF AREAS OF DREDGED CHANNEL
- PROPERTY BOUNDARY
- FENCE LINE

GENERAL NOTES:

1. Base drawing provided from available information compiled by Parsons Engineering Science, Inc., acting for Honeywell International Inc.
2. Site use areas shown provide clearance for minor discrepancies in actual locations of buried utilities. Locations of visible utilities shall be confirmed for protection before site work commences.

GRAPHIC SCALE

80' 40' 0 80' 160'

HARBOR POINT
(FORMER HONEYWELL BALTIMORE WORKS)
BALTIMORE MARYLAND

STRUEVER BROTHERS
ECCLES & ROUSE, INC.
BALTIMORE MARYLAND

MUESER RUTLEDGE CONSULTING ENGINEERS
14 PENN PLAZA – 225 W. 34TH STREET, NY, NY 10122

SCALE	MADE BY I.Y.	DATE 9-10-03	FILE NO.
GRAPHIC	CH'KD BY P.W.D.	DATE 9-10-03	9487

CRITERIA FOR INTERIM USE
HARBOR POINT SITE AREA 1
WEST OF WILLS ST.

DRAWING NO. I-1

Attachment 2

Static Load Spreading of Design Truck
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon　　　　　　　Project Number : 11896A
Client : 15 yd3 Concrete Truck　　　Project Manager: GS
Date : 6/24/2013　　　　　　　　Computed by : DJG

Footing #	Corner Point P1		Corner Point P2		Load
	X1(ft)	Y1(ft)	X2(ft)	Y2(ft)	(Ksf)
1	0.00	0.00	0.66	1.33	11.250
2	1.33	0.00	2.00	1.33	11.250
3	6.00	0.00	6.66	1.33	11.250
4	7.33	0.00	8.00	1.33	11.250
5	0.00	4.50	0.66	5.83	11.250
6	1.33	4.50	2.00	5.83	11.250
7	6.00	4.50	6.66	5.83	11.250
8	7.33	4.50	8.00	5.83	11.250

INCREMENT OF STRESS FOR
X = 0.33(ft)　Y = 0.66(ft)　Z = 2.50(ft)

Vert. Dsz
(Ksf)

1.15

Static and Dynamic Load Spreading of Design Truck
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : 15 yd3 Concrete Truck
Date : 6/24/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1		Corner Point P2		Load
	X1(ft)	Y1(ft)	X2(ft)	Y2(ft)	(Ksf)
1	0.00	0.00	0.66	1.33	14.960
2	1.33	0.00	2.00	1.33	14.960
3	6.00	0.00	6.66	1.33	14.960
4	7.33	0.00	8.00	1.33	14.960
5	0.00	4.50	0.66	5.83	14.960
6	1.33	4.50	2.00	5.83	14.960
7	6.00	4.50	6.66	5.83	14.960
8	7.33	4.50	8.00	5.83	14.960

INCREMENT OF STRESS FOR
X = 0.33(ft) Y = 0.66(ft) Z = 2.50(ft)

Vert. Dsz
(Ksf)

1.53

Static Load Spreading of Wheel Loader
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : Wheel Loader
Date : 6/27/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1 X1(ft)	Corner Point P1 Y1(ft)	Corner Point P2 X2(ft)	Corner Point P2 Y2(ft)	Load (Ksf)
1	0.00	0.00	1.60	1.06	9.020
2	0.00	10.83	1.60	11.89	9.020
3	6.83	10.83	8.43	11.89	9.020
4	6.83	0.00	8.43	1.06	9.020

INCREMENT OF STRESS FOR
X = 0.80(ft) Y = 0.53(ft) Z = 2.50(ft)

Vert. Dsz
(Ksf)

1.05

Static and Dynamic Load Spreading of Wheel Loader
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : Wheel Loader
Date : 6/27/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1 X1(ft)	Corner Point P1 Y1(ft)	Corner Point P2 X2(ft)	Corner Point P2 Y2(ft)	Load (Ksf)
1	0.00	0.00	1.60	1.06	12.000
2	0.00	10.83	1.60	11.89	12.000
3	6.83	10.83	8.43	11.89	12.000
4	6.83	0.00	8.43	1.06	12.000

INCREMENT OF STRESS FOR
X = 0.80(ft) Y = 0.53(ft) Z = 2.50(ft)

Vert. Dsz
(Ksf)

1.39

16,380 Gallon Double-Wall Tank
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : 16380 Gallon Tank
Date : 6/24/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1 X1(ft)	Corner Point P1 Y1(ft)	Corner Point P2 X2(ft)	Corner Point P2 Y2(ft)	Load (Ksf)
1	0.00	0.00	0.33	27.33	3.900
2	2.00	0.00	2.33	27.33	3.900
3	6.00	0.00	6.33	27.33	3.900
4	8.00	0.00	8.33	27.33	3.900
5	0.33	0.00	2.00	0.33	3.900
6	0.33	9.00	2.00	9.33	3.900
7	0.33	18.00	2.00	18.33	3.900
8	0.33	27.00	2.00	27.33	3.900
9	2.33	0.00	6.00	0.33	3.900
10	2.33	9.00	6.00	9.33	3.900
11	2.33	18.00	6.00	18.33	3.900
12	2.33	27.00	6.00	27.33	3.900
13	6.33	0.00	8.00	0.33	3.900
14	6.33	9.00	8.00	9.33	3.900
15	6.33	18.00	8.00	18.33	3.900
16	6.33	27.00	8.00	27.33	3.900

INCREMENT OF STRESS FOR
X = 2.17(ft) Y = 9.17(ft) Z = 2.50(ft)

Vert. Dsz
(Ksf)

0.74

25 Yard Roll-off Box with Aluminum Hard Top
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : 25 yd Roll-off Box
Date : 6/24/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1		Corner Point P2		Load
	X1(ft)	Y1(ft)	X2(ft)	Y2(ft)	(Ksf)
1	0.00	0.34	0.50	0.84	10.800
2	0.00	19.42	0.50	19.92	10.800
3	7.05	0.34	7.55	0.84	10.800
4	7.05	19.42	7.55	19.92	10.800
5	2.00	0.00	2.17	22.00	10.800
6	5.38	0.00	5.55	22.00	10.800

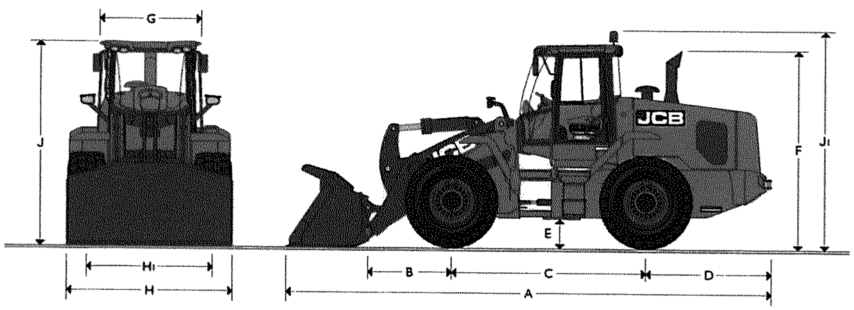
INCREMENT OF STRESS FOR
X = 2.08(ft) Y = 11.00(ft) Z = 2.50(ft)

Vert. Dsz
(Ksf)

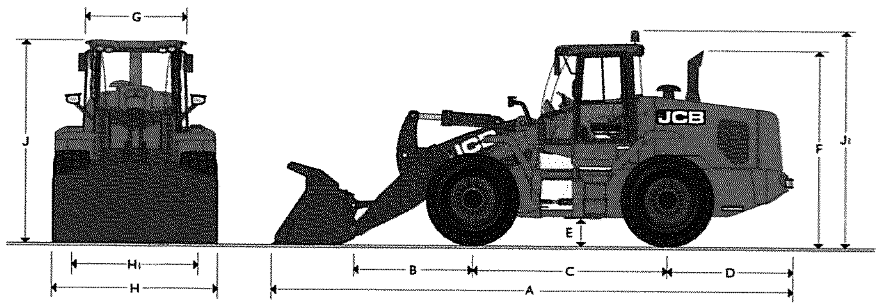
0.53

Attachment 3

STATIC DIMENSIONS – Standard height arm



STATIC DIMENSIONS – High lift arm



STATIC DIMENSIONS – Standard height arm

			ft-in (mm)
A	Overall length with standard bucket		26-2 (7964)
B	Axle to pivot pin		5-4 (1622)
C	Wheel base		10-10 (3300)
D	Axle to counterweight face		6-6 (1974)
E	Minimum ground clearance		1-7 (470)
F	Height over exhaust		10-11 (3318)
G	Width over cab		4-7 (1400)
H	Width over tires		8-10 (2702)
Hı	Wheel track		6-10 (2100)
J	Height over cab		11-1 (3370)
Jı	Overall height (to top of fixed beacon)		12-2 (3714)
	Pin height (maximum)		13-5 (4107)
	Overall operating height		18-3 (5571)
	Front axle weight	lb (kg)	17,921 (8129)
	Rear axle weight	lb (kg)	24,368 (11,053)
	Total weight	lb (kg)	42,289 (19,182)
	Inside radius		10-5 (3182)
	Maximum radius		21-6 (6554)
	Articulation angle	degrees	±40°

Data based on machine equipped with a 4.3yd³ bucket with bolt-on toeplates and 23.5 R25 Michelin XHA (L3) radial tires.

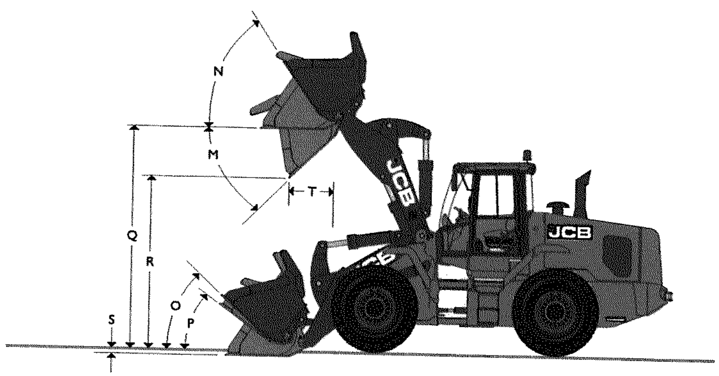
STATIC DIMENSIONS – High lift arm

Attachment 3

			ft-in (mm)
A	Overall length with standard bucket		28-0 (8524)
B	Axle to pivot pin		7-2 (2182)
C	Wheel Base		10-10 (3300)
D	Axle to counterweight face		6-6 (1974)
E	Minimum ground clearance		1-7 (470)
F	Height over exhaust		10-11 (3318)
G	Width over cab		4-7 (1400)
H	Width over tires		8-10 (2702)
Hı	Wheel track		6-10 (2100)
J	Height over cab		11-1 (3370)
Jı	Overall height (to top of fixed beacon)		12-2 (3714)
	Pin height (maximum)		15-4 (4677)
	Overall operating height		20-2 (6140)
	Front axle weight	lb (kg)	18,576 (8,426)
	Rear axle weight	lb (kg)	24,619 (11,167)
	Total weight	lb (kg)	43,195 (19,593)
	Inside radius		10-5 (3182)
	Maximum radius over shovel		22-2 (6770)
	Articulation angle	degrees	±40°

Data based on machine equipped with a 4.3yd³ bucket with bolt-on toeplates and 23.5 R25 Michelin XHA (L3) radial tires.

LOADER DIMENSIONS – Standard height arm



CHANGES TO OPERATING PERFORMANCE AND DIMENSIONS

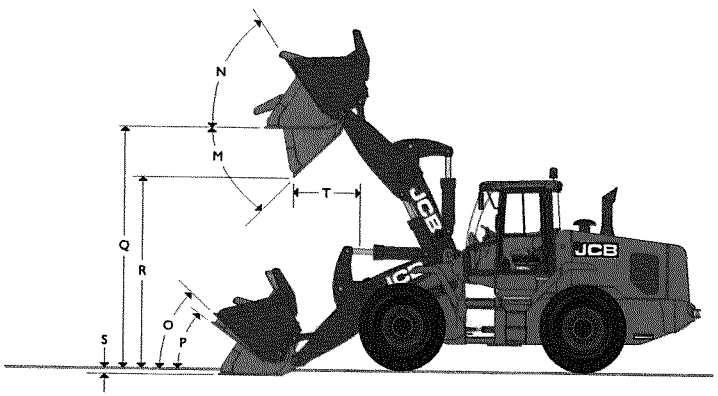
Tire size	Manufacturer	Type	Rating	Op. weight lb (kg)	Tipping loads Straight lb (kg)	Tipping loads Full turn lb (kg)	Dimensions Vertical in (mm)	Dimensions Width in (mm)
23.5R25 (radial)	Michelin	XTLA	L2	-220 (-100)	-156 (-71)	-134 (-61)	-0.08 (-2)	0
23.5R25 (radial)	Goodyear	TL-3A+	L3	714 (324)	506 (230)	433 (196)	0.75 (19)	0
23.5R25 (radial)	Goodyear	RT-3B	L3	388 (176)	275 (125)	235 (107)	0.39 (10)	0
23.5–25 (crossply)	Goodyear	HRL-3A	L3	-220 (-100)	-156 (-71)	-134 (-61)	0.59 (15)	0
23.5–25 (crossply)	Earthmover	20ply	L3	-335 (-152)	-237 (-108)	-203 (-92)	0.24 (6)	0
23.5R25 (radial)	Earthmover		L3	0	0	0	0.16 (4)	0
23.5R25 (radial)	Goodyear	GP-4B	L4	838 (380)	593 (269)	508 (230)	1.38 (35)	0
23.5R25 (radial)	Michelin	XLDD2A	L5	1261 (572)	893 (405)	764 (347)	1.42 (36)	0
23.5R25 (radial)	Michelin	XMINED2	L5	1781 (808)	1262 (572)	1079 (490)	1.42 (36)	0
23.5R25 (radial)	Goodyear	RL-5K	L5	1552 (704)	1099 (499)	941 (427)	1.42 (36)	0
23.5-25 (solid cushion)*	SG Revolution	SE	-	6887 (3124)	1030 (467)	882 (400)	1.18 (30)	0
23.5-25 (solid cushion)*	SG Revolution	DWL	-	6887 (3124)	1030 (467)	882 (400)	1.18 (30)	0
Deduct optional extra counterweight		–	–	-1764 (-800)	-3407 (-1546)	-2812 (-1275)	0	0

*Optional extra counterweights is not available when solid tires are fitted.

Assumes the fitment of Michelin 23.5R25 XHA (L3) tires.

		Unit													
	Bucket mounting		Direct	Direct	Direct	Direct	Direct	Direct	Direct	Quickhitch	Quickhitch	Quickhitch	Quickhitch	Quickhitch	Quickhitch
	Bucket type		General Purpose	General Purpose	Penetration	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose
	Bucket equipment		Tipped teeth	Tipped teeth	Tipped teeth	Reversible toeplate	Reversible toeplate	Tipped teeth & toeplate segments	Tipped teeth & toeplate segments	Tipped teeth	Tipped teeth	Reversible toeplate	Reversible toeplate	Tipped teeth & toeplate segments	Tipped teeth & toeplate segments
	Bucket capacity (SAE heaped)	yd^3 (m^3)	4.1 (3.1)	4.3 (3.3)	4.1 (3.1)	4.3 (3.3)	4.6 (3.5)	4.3 (3.3)	4.6 (3.5)	4.1 (3.1)	4.3 (3.3)	4.3 (3.3)	4.6 (3.5)	4.3 (3.3)	4.6 (3.5)
	Bucket capacity (struck)	yd^3 (m^3)	3.651 (2.791)	3.912 (2.991)	3.651 (2.791)	3.836 (2.933)	4.103 (3.137)	3.836 (2.933)	4.103 (3.137)	3.266 (2.497)	3.515 (2.687)	3.464 (2.648)	3.720 (2.844)	3.464 (2.648)	3.720 (2.844)
	Bucket width	ft-in (mm)	9-4 (2837)	9-4 (2837)	9-3 (2811)	9-2 (2800)	9-2 (2800)	9-2 (2800)	9-2 (2800)	9-4 (2837)	9-4 (2837)	9-4 (2837)	9-4 (2837)	9-4 (2837)	9-4 (2837)
	Bucket weight with wearparts	lb (kg)	3532 (1602)	3627 (1645)	3554 (1612)	3797 (1722)	3892 (1765)	3797 (1722)	3892 (1765)	3043 (1380)	3122 (1416)	3296 (1495)	3376 (1531)	3296 (1495)	3376 (1531)
	Maximum material density	lb/yd^3 (kg/m^3)	3594 (2132)	3352 (1989)	3589 (2129)	3343 (1983)	3129 (1856)	3343 (1983)	3129 (1856)	3263 (1936)	3044 (1806)	3035 (1801)	2840 (1685)	3035 (1801)	2840 (1685)
	Tipping load straight	lb (kg)	38,342 (17,392)	38,103 (17,284)	38,292 (17,369)	38,048 (17,259)	37,809 (17,150)	38,048 (17,259)	37,809 (17,150)	35,233 (15,982)	35,017 (15,884)	34,965 (15,860)	34,748 (15,762)	34,965 (15,860)	34,748 (15,762)
	Tipping load full turn	lb (kg)	31,956 (14,494)	31,741 (14,397)	31,908 (14,473)	31,671 (14,365)	31,455 (14,267)	31,671 (14,365)	31,455 (14,267)	29,275 (13,278)	29,079 (13,190)	29,015 (13,161)	28,817 (13,071)	29,015 (13,161)	28,817 (13,071)
	Payload at 50% FTTL	lb (kg)	15,978 (7247)	15,871 (7199)	15,954 (7237)	15,836 (7183)	15,728 (7134)	15,836 (7183)	15,728 (7134)	14,638 (6639)	14,540 (6595)	13,102 (5943)	13,003 (5898)	13,102 (5943)	13,003 (5898)
	Maximum break out force	lbf (kN)	38,666 (172)	37,092 (165)	38,666 (172)	36,193 (161)	34,619 (154)	36,193 (161)	34,619 (154)	34,394 (153)	33,046 (147)	32,146 (143)	30,798 (137)	32,146 (143)	30,798 (137)
M	Dump angle maximum	degrees	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°
N	Roll back angle at full height	degrees	67°	67°	67°	67°	67°	67°	67°	67°	67°	67°	67°	67°	67°
O	Roll back at carry	degrees	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°
P	Roll back at ground level	degrees	39°	39°	39°	39°	39°	39°	39°	39°	39°	39°	39°	39°	39°
Q	Load over height	ft-in (mm)	12-6 (3822)	12-6 (3822)	12-3 (3856)	12-6 (3831)	12-6 (3831)	12-6 (3822)	12-6 (3822)	12-6 (3822)	12-2 (3702)	12-6 (3822)	12-2 (3702)	12-6 (3822)	12-2 (3702)
R	Dump height (45° dump)	ft-in (mm)	9-0 (2741)	8-10 (2699)	9-1 (2765)	9-6 (2887)	9-4 (2845)	9-0 (2741)	8-10 (2699)	8-7 (2621)	8-5 (2559)	9-1 (2767)	8-11 (2725)	8-7 (2621)	8-5 (2559)
S	Dig depth	ft-in (mm)	0-3 (74)	0-3 (74)	0-3 (74)	0-4 (91)	0-4 (91)	0-4 (109)	0-4 (109)	0-3 (74)	0-3 (74)	0-4 (91)	0-4 (91)	0-4 (91)	0-4 (91)
T	Reach at dump height	ft-in (mm)	3-11 (1183)	3-9 (1135)	4-0 (1207)	3-7 (1085)	3-5 (1039)	3-11 (1183)	3-9 (1135)	4-3 (1301)	4-1 (1255)	3-11 (1205)	3-10 (1159)	4-3 (1301)	4-1 (1255)
	Reach maximum (45° dump)	ft-in (mm)	7-0 (2140)	7-2 (2182)	7-1 (2164)	6-8 (2032)	6-10 (2074)	7-0 (2140)	7-2 (2182)	7-5 (2260)	7-7 (2302)	7-1 (2152)	7-2 (2194)	7-5 (2260)	7-7 (2302)
	Operating weight (includes 176lb operator and full fuel tank)	lb (kg)	43,945 (19,933)	44,053 (19,982)	43,967 (19,943)	44,210 (20,053)	44,318 (20,102)	44,210 (20,053)	44,318 (20,102)	44,659 (20,257)	44,767 (20,306)	44,924 (20,377)	45,032 (20,426)	44,924 (20,377)	45,032 (20,426)

LOADER DIMENSIONS – High lift arm



CHANGES TO OPERATING PERFORMANCE AND DIMENSIONS

Tire size	Manufacturer	Type	Rating	Op. weight lb (kg)	Tipping loads Straight lb (kg)	Tipping loads Full turn lb (kg)	Dimensions Vertical in (mm)	Dimensions Width in (mm)
23.5R25 (radial)	Michelin	XLTA	L2	-220 (-100)	-129 (-58)	-110 (-50)	-0.08 (-2)	0
23.5R25 (radial)	Goodyear	TL-3A+	L3	714 (324)	417 (189)	357 (162)	0.75 (19)	0
23.5R25 (radial)	Goodyear	RT-3B	L3	388 (176)	227 (103)	194 (88)	0.39 (10)	0
23.5–25 (crossply)	Goodyear	HRL-3A	L3	-220 (-100)	-129 (-58)	-110 (-50)	0.59 (15)	0
23.5–25 (crossply)	Earthmover	20ply	L3	-335 (-152)	-196 (-89)	-167 (-76)	0.24 (6)	0
23.5R25 (radial)	Earthmover		L3	0	0	0	0.16 (4)	0
23.5R25 (radial)	Goodyear	GP-4B	L4	838 (380)	489 (222)	418 (190)	1.38 (35)	0
23.5R25 (radial)	Michelin	XLDD2A	L5	1261 (572)	736 (334)	630 (286)	1.42 (36)	0
23.5R25 (radial)	Michelin	XMINED2	L5	1781 (808)	1040 (472)	890 (404)	1.42 (36)	0
23.5R25 (radial)	Goodyear	RL-5K	L5	1552 (704)	906 (411)	775 (352)	1.42 (36)	0
23.5-25 (solid cushion)*	SG Revolution	SE	-	6887 (3124)	4021 (1824)	3440 (1560)	1.18 (30)	0
23.5-25 (solid cushion)*	SG Revolution	DWL	-	6887 (3124)	4021 (1824)	3440 (1560)	1.18 (30)	0
Deduct optional extra counterweight		–	–	-1764 (-800)	-2808 (-1274)	-2317 (-1051)	0	0

*Optional extra counterweights is not available when solid tires are fitted.

Assumes the fitment of Michelin 23.5R25 XHA (L3) tires.

	Bucket mounting		Direct	Direct	Direct	Direct	Direct	Direct	Direct	Quickhitch	Quickhitch	Quickhitch	Quickhitch	Quickhitch	Quickhitch
	Bucket type		General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose	General Purpose
	Bucket equipment		Tipped teeth	Tipped teeth	Tipped teeth	Reversible t/plate	Reversible t/plate & t/plate segments	Reversible t/plate & t/plate segments	Reversible t/plate	Tipped teeth	Tipped teeth	Reversible t/plate	Reversible t/plate & t/plate segments	Reversible t/plate & t/plate segments	Reversible t/plate & t/plate segments
	Bucket capacity (SAE heaped)	yd^3 (m^3)	3.7 (2.8)	4.1 (3.1)	4.3 (3.3)	4.3 (3.3)	4.6 (3.5)	4.3 (3.3)	4.6 (3.5)	4.1 (3.1)	4.3 (3.3)	4.3 (3.3)	4.6 (3.5)	4.3 (3.3)	4.6 (3.5)
	Bucket capacity (struck)	yd^3 (m^3)	3.266 (2.497)	3.651 (2.791)	3.912 (2.991)	3.836 (2.933)	4.103 (3.137)	3.836 (2.933)	4.103 (3.137)	3.266 (2.497)	3.515 (2.687)	3.464 (2.648)	3.720 (2.844)	3.464 (2.648)	3.720 (2.844)
	Bucket width	ft-in (mm)	9-4 (2837)	9-4 (2837)	9-4 (2837)	9-2 (2800)	9-2 (2800)	9-2 (2800)	9-2 (2800)	9-4 (2837)	9-4 (2837)	9-4 (2837)	9-4 (2837)	9-4 (2837)	9-4 (2837)
	Bucket weight with wearparts	lb (kg)	3371 (1529)	3532 (1602)	3627 (1645)	3797 (1722)	3892 (1765)	3797 (1722)	3892 (1765)	3043 (1380)	3122 (1416)	3296 (1495)	3376 (1531)	3296 (1495)	3376 (1531)
	Maximum material density	lb/yd^3 (kg/m^3)	2983 (1770)	2681 (1591)	2500 (1483)	2493 (1479)	2333 (1384)	2493 (1479)	2333 (1384)	2455 (1457)	2290 (1358)	2284 (1355)	2138 (1269)	2284 (1355)	2138 (1269)
	Tipping load straight	lb (kg)	29,210 (13,250)	29,080 (13,191)	28,898 (13,108)	28,857 (13,089)	28,679 (13,009)	28,857 (13,089)	28,679 (13,009)	26,978 (12,237)	26,812 (12,162)	26,775 (12,145)	26,611 (12,071)	26,775 (12,145)	26,611 (12,071)
	Tipping load full turn	lb (kg)	24,164 (10,961)	24,057 (10,912)	23,897 (10,840)	23,845 (10,816)	23,683 (10,743)	23,845 (10,816)	23,683 (10,743)	22,230 (10,084)	22,085 (10,017)	22,037 (9996)	21,889 (9929)	22,037 (9996)	21,889 (9929)
	Payload at 50% FTTL	lb (kg)	12,082 (5481)	12,029 (5456)	11,949 (5420)	11,923 (5408)	11,842 (5372)	11,923 (5408)	11,842 (5372)	11,115 (5042)	11,043 (5009)	11,019 (4998)	10,945 (4965)	11,019 (4998)	10,945 (4965)
	Maximum break out force	lbf (kN)	36,867 (164)	33,945 (151)	32,596 (145)	31,922 (142)	30,573 (136)	31,922 (142)	30,573 (136)	30,123 (134)	28,999 (129)	28,325 (126)	27,201 (121)	28,325 (126)	27,201 (121)
M	Dump angle maximum	degrees	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°	45°
N	Roll back angle at full height	degrees	53°	53°	53°	53°	53°	53°	53°	53°	53°	53°	53°	53°	53°
O	Roll back at carry	degrees	52°	52°	52°	52°	52°	52°	52°	52°	52°	52°	52°	52°	52°
P	Roll back at ground level	degrees	44°	44°	44°	44°	44°	44°	44°	44°	44°	44°	44°	44°	44°
Q	Load over height	ft-in (mm)	14-5 (4393)	14-5 (4393)	14-5 (4393)	14-5 (4402)	14-5 (4402)	14-5 (4393)	14-5 (4393)	14-5 (4393)	14-0 (4273)	14-5 (4393)	14-1 (4282)	14-5 (4393)	14-0 (4273)
R	Dump height (45° dump)	ft-in (mm)	11-1 (3376)	10-10 (3312)	10-9 (3270)	11-4 (3458)	11-2 (3416)	10-10 (3312)	10-9 (3270)	10-6 (3192)	10-3 (3130)	10-11 (3338)	10-10 (3296)	10-6 (3192)	10-3 (3130)
S	Dig depth	ft-in (mm)	0-3 (75)	0-3 (75)	0-3 (75)	0-4 (101)	0-4 (101)	0-4 (101)	0-4 (101)	0-3 (75)	0-3 (75)	0-4 (101)	0-4 (101)	0-4 (101)	0-4 (101)
T	Reach at dump height	ft-in (mm)	3-7 (1099)	4-2 (1259)	4-0 (1213)	3-10 (1162)	3-8 (1117)	4-2 (1259)	4-0 (1213)	4-6 (1379)	4-5 (1333)	4-3 (1283)	4-1 (1237)	4-6 (1379)	4-5 (1333)
	Reach maximum (45° dump)	ft-in (mm)	8-5 (2553)	8-7 (2617)	8-9 (2659)	8-3 (2509)	8-4 (2551)	8-7 (2617)	8-9 (2659)	9-0 (2737)	9-1 (2779)	8-8 (2629)	8-9 (2617)	9-0 (2737)	9-1 (2779)
	Operating weight (includes 176lb operator and full fuel tank)	lb (kg)	44,690 (20,271)	44,851 (20,344)	44,959 (20,393)	45,116 (20,464)	45,224 (20,513)	45,116 (20,464)	45,224 (20,513)	45,563 (20,667)	45,673 (20,717)	45,830 (20,788)	45,938 (20,837)	45,830 (20,788)	45,938 (20,837)

LOADER

Heavy duty three cylinder geometry provides high breakout forces with excellent loading characteristics. The pin, bush and sealing design on all pivot points provide extended maintenance intervals.

ENGINE

6-cylinder variable geometry turbo-charged and charge air cooled 8.9l diesel engine. High pressure common rail fuel injection, cooled exhaust gas recirculation and a diesel particulate filter combine to reduce emissions and optimise fuel efficiency. Selectable Power or Economy modes.

Manufacturer		Cummins
Model		QSL9
Displacement	in³ (ltr)	543 (8.9)
Bore	in (mm)	4.49 (114)
Stroke	in (mm)	5.69 (145)
Aspiration		Variable Geometry Turbocharger
No. of Cylinders		6
Max. Gross Power to SAE J1995/ISO 14396	hp (kW) @ 1800rpm	250 (186)
Rated Gross Power to SAE J1995/ISO 14396	hp (kW) @ 2200rpm	250 (186)
Net Power to SAE J1349	hp (kW) @ 2100rpm	247 (184)
Gross Torque at 1400rpm	lbf-ft (Nm) @1500rpm	800 (1085)
Economy Working Range	rpm	800 - 1800
Torque Rise	%	34.1
Valves per Cylinder		4
Wet Weight	lbs (kg)	1560 (708)
Air Cleaner		Cyclonic pre filter with scavenge system
Fan Drive Type		Hydraulic
Emissions		US EPA Tier 4i, EU Stage IIIB

TRANSMISSION

4 wheel drive, automatic 4 speed transmission. "Power-Inch" intelligent clutch cut off technology as standard . Optional 5 speed transmission with auto-locking torque converter available for even more speed and efficiency.

Type		4 speed non-lock up converter	5 speed with lock up torque converter
Make		ZF	ZF
Model		4WG210 (standard)	5WG210 with lock-up (option)
Forward speed 1	mph (kph)	4.3 (7.0)	4.4 (7.1)
Forward speed 2	mph (kph)	8.5 (13.7)	7.8 (12.6)
Forward speed 3	mph (kph)	16.2 (26.1)	11.9 (19.1)
Forward speed 4	mph (kph)	25.8 (41.5)	18.1 (29.1)
Forward speed 5	mph (kph)		26.6 (42.7)
Reverse 1	mph (kph)	4.6 (7.3)	4.7 (7.5)
Reverse 2	mph (kph)	9.0 (14.4)	8.3 (13.3)
Reverse 3	mph (kph)	17.0 (27.4)	19.0 (30.6)

AXLES

3 axles options available; Torque proportioning differentials, Limited slip differentials or Open differentials with automatic differential locking. All axle options feature wheel speed braking for lower heat build up and longer service life.

Type	Open Differential	Limited Slip Differential	Open Differential with auto-locking front
Make and Model	ZF MT-L 3095 MK 2 (front and rear)	ZF MT-L 3095 MK 2 (front and rear)	ZF MT-L 3095 MK 2 (front and rear)
Overall Axle ratio	23.334:1	23.334:1	23.334:1
Rear Axle Oscillation	±12.5°	±12.5°	±12.5°

ELECTRICAL SYSTEM

24 volt negative ground system, 70 Amp alternator with 2 x 110 Amp hour low maintenance batteries. Isolator located in rear of machine. Ignition key start/stop and pre-heat cold start. Primary fuse box. Other electrical equipment includes quartz halogen, twin filament working lights, front/rear wash/wipe, heated rear screen, full roading lights, clock, gauge and warning light monitoring. Connectors to IP67 standard.

System voltage	Volt	24
Alternator output	Amp hour	70
Battery capacity	Amp hour	2 x 110

STEERING

Priority steer hydraulic system with emergency steering. Piston pump meters flow through steer valve to provide smooth low effort response. Steering angle ± 40°. Steering cylinders fitted with end rod damping to provide cushioned steering at full articulation. Adjustable steering column.

BRAKES

Hydraulic power braking on all wheels, operating pressure 1160psi (80 bar). Dual circuit with accumulator back-up provide maximum safety under all conditions. Hub mounted, oil immersed, multi-plate disc brakes with sintered linings reduce heat build up. Wheel speed braking improves performance and reduce wear. Parking brake, electro-hydraulic disc type operating on transmission output shaft.

SERVICE FILL CAPACITIES

	gal (liters)
Hydraulic system	35.7 (135)
Fuel system	81.6 (309)
Engine oil (includes filter)	5.0 (19)
Engine coolant	10.6 (40)
Axles	9.0 (34)
Transmission	10.8 (41)

CAB

Resiliently mounted ROPS/FOPS structure (tested in accordance with EN3471:2008/EN3449: 2008 (Level 2). Entry/exit is via a large rear hinged door, grab handles giving 3 points of contact and and anti-slip inclined steps. Forward visibility through a curved, laminated windscreen with lower glazed quarter panels, two interior mirror and heated exterior mirrors. Instrumentation analogue/digital display gauges along with full color LCD screen including selectable machine and operator menus along with service and diagnostic screens. Heating/ventilation provides balanced and filtered air distribution throughout the cab via a powerful 27,300 BTU capacity heater, with air conditioning and climate control system as options. Provision of speakers and antenna for radio fitment (radio/CD not included). The cab environment is positively pressurised preventing the ingress of dust including in-cab recirculation filter. Fabric mechanical suspension seat as standard with various options including vinyl material, air suspension, heating and deluxe Grammer Actimo XXL air suspension seat with headrest, twin armrests, lumbar support, backrest extension, heating and full adjustment. Coat hook, cup holder and additional storage space. Fuse box positioned at rear for access to fuses, relays and diagnostic connectors.

TIRES

A variety of tire options are available including:
23.5R25 XTLA (L2), 23.5R25 XHA (L3), 23.5R25 TL-3A+ (L3), 23.5R25 RT-3B (L3), 23.5x25x20 ply HRL (L3), 23.5x25x20 ply (L3), 23.5R25 JCB (L3), 23.5R25 XMINE (L5), 23.5R25 XLDD2 (L5), 23.5R25 RL-5K (L5), 23.5R25 DWL (Solid Cushion), 23.5R25 SE (Solid Cushion)

ATTACHMENTS

An extensive range of attachments are available to fit directly or via the JCB quickhitch mounting.

LOADER HYDRAULICS

Twin variable displacement piston pumps feed a "load sensing" system providing a fuel efficient and responsive distribution of power as required. Main services are servo actuated from a single lever (joystick) loader control. Auxiliary circuits controlled via additional lever or joystick mounted electrical buttons. Accumulator back-up is available to control loader in the event of loss of pump pressure.

Pump type		Twin variable displacement piston pumps
Pump 1 max. flow	gal/min (l/min)	43 (163)
Pump 1 max. pressure	PSI (bar)	3625 (250)
Pump 2 max. flow	gal/min (l/min)	43 (163)
Pump 2 max. pressure	PSI (bar)	2320 (160)
Hydraulic cycle times at full engine revs		**seconds**
Arms raise (full bucket)		5.8
Bucket dump (full bucket)		1.2
Arms lower (empty bucket)		4.1
Total cycle		11.1

Ram dimensions		**Bore**	**Rod**	**Closed centers**	**Stroke**
Bucket ram x2	in (mm)	7.1 (180)	3.0 (90)	42.5 (1080)	22.4 (570)
Lift ram x2	in (mm)	6.3 (160)	3.1 (80)	50.8 (1290)	29.3 (744)
Steer ram x2	in (mm)	3.5 (90)	2.0 (50)	24.4 (621)	12.3 (312)

STANDARD EQUIPMENT

Loader: Bucket reset mechanism (selectable), loader arm kickout mechanism (selectable), loader control isolator, single lever or multi lever servo control, high breakout forces with excellent loading characteristics, safety strut.

Engine: Air cleaner – cyclonic pre filter with scavenge system. Variable geometry turbocharger, cooled exhaust gas recirculation, diesel particulate filter, isolated cooling package with hydraulically driven cooling fan. Selectable ECO mode (217hp)

Transmission: Single lever shift control, neutral start, 'Power-Inch' Intelligent clutch cut off on footbrake (selectable), direction changes and kickdown on gear selector and loader control lever.

Axles: Epicyclic wheel hub reduction, fixed front, oscillating rear.

Brakes: Mulit-plate wet disc brakes, sintered brake pads, dual circuit hydraulic power, wheel speed braking. Parking disc brake on transmission output shaft.

Hydraulics: Twin piston pumps with priority steer, emergency steer back-up, 2 spool loader circuit with accumulator support, 3rd spool auxiliary hydraulic circuit, 4th spool optional.

Steering: Adjustable steering column, "soft feel" steering wheel, 5 turns lock to lock, resilient stops on max lock.

Cab: ROPS/FOPS safety structure, interior light, center mounted master warning light. Electronic monitoring panel with full color LCD display. Two speed intermittent front windscreen wipe/wash and self park, single speed rear windscreen wipe/wash and self park. 3 speed heater/demisting with replaceable air filter, RH opening windows, sun visor, internal rear view mirror, heated external mirrors, adjustable suspension seat with belt and headrest, operator storage, laminated windscreen, heated rear screen, loader control isolator, horn, adjustable armrest.

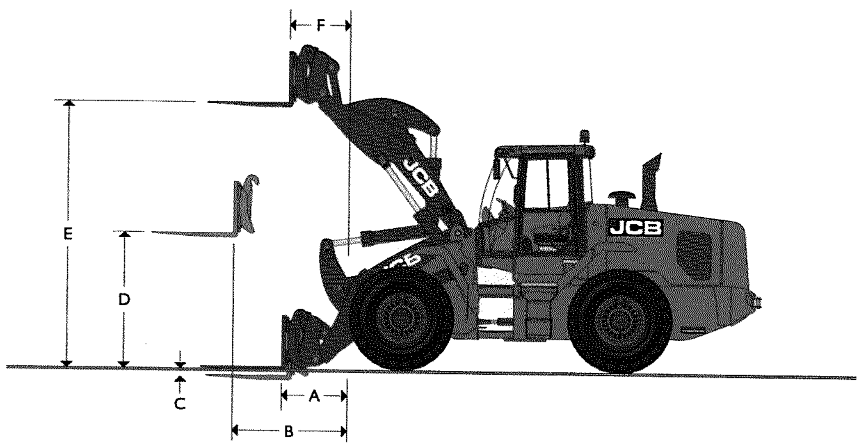
Electrical: Road lights front and rear, parking lights, front and rear working lights, reverse alarm and light, rear fog light, battery isolator, radio wiring and speakers, 70 amp alternator, rotating beacon.

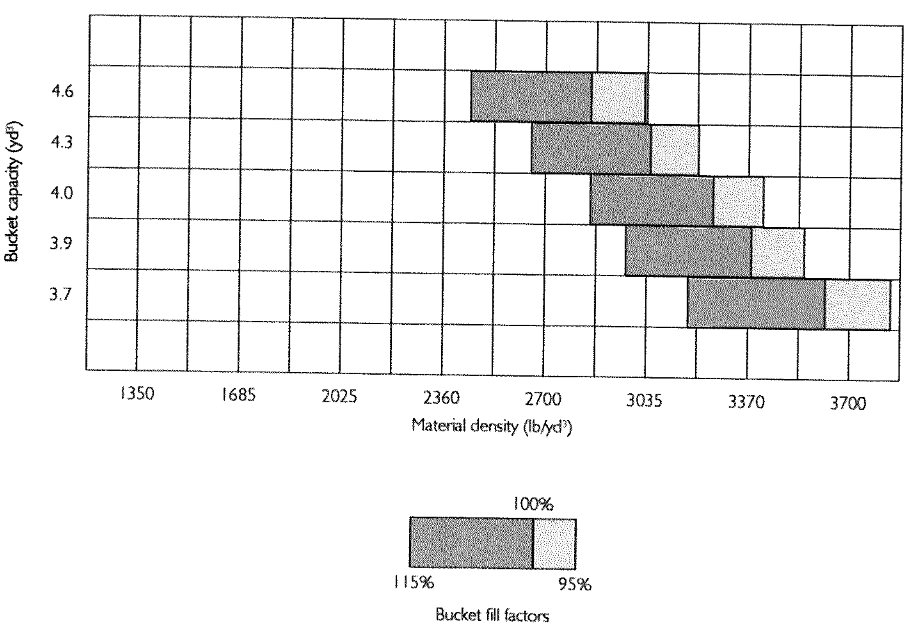
Bodywork: Front and rear fenders, side and rear access panels, mesh air intake screens, flexible bottom step, full width rear counterweight, recovery hitch, lifting lugs, belly guards.

OPTIONAL EQUIPMENT

Loader: High lift loader end, Smoothride system (SRS), hydraulic quickhitch with in-cab pin isolation, replaceable bucket wear parts.
Engine: Widecore radiator, epoxy coated radiator / coolers, automatically reversing cooling fan, engine block heater
Transmission: 5 speed transmission with Lock-up torque converter, transmission cooler bypass
Axles: Limited slip differentials front and rear, Open differential with automatic differential locking - 100% (front axle only)
Hydraulics: ARV kit, 4th hydraulic spool
Cab: Canopy cab, wastemaster cab, air conditioning, Climate control, joystick or multi-lever hydraulic controls, auxiliary hydraulic control on separate lever or joystick mounted (proportional), 24V to 12V in cab converter, cab screen guards, heated air suspension seat, Grammer Actimo XXL seat, front and rear blinds, P3 cab air filter, Carbon cab air filter
Electrical: Reversing camera (color), additional front and rear work lights, sealed electrics, non-heated mirrors
Bodywork: Full rear fenders, light guards, number plate light kit, white noise reverse alarm, smart reverse alarm.
Miscellaneous options: Automatic greasing system, Biodegradable hydraulic oil, fire extinguisher, grease gun and cartridge
Wastemaster package: Includes front and rear light guards, widecore radiator, carbon cab air filter, front screen guard, full belly guarding, Wastemaster decal.

457 HT – LOADER DIMENSIONS – FORK FRAME WITH FORKS





LOADER DIMENSIONS – FORK FRAME WITH FORKS

	Assumes the fitment of Michelin 23.5R25 XHA (L3) tires		Standard arm	High lift arm
	Fork carriage width	ft-in (mm)	4-11 (1500)	4-11 (1500)
	Length of tines	ft-in (mm)	4-0 (1220)	4-0 (1220)
A	Reach at ground level	ft-in (mm)	3-7 (1084)	5-5 (1644)
B	Reach at arms horizontal	ft-in (mm)	5-7 (1695)	7-2 (2172)
C	Below ground level	ft-in (mm)	0-1 (16)	0-1 (16)
D	Arms, horizontal height	ft-in (mm)	6-6 (1975)	6-6 (1975)
E	Arms, maximum height	ft-in (mm)	13-1 (3997)	15-0 (4567)
F	Reach at maximum height	ft-in (mm)	2-5 (735)	2-8 (813)
	Payload*	lb (kg)	17,951 (8142)	13,391 (6074)
	Tipping load straight	lb (kg)	26,900 (12,202)	20,228 (9175)
	Tipping load full turn (40°)	lb (kg)	22,439 (10,178)	16,741 (7594)
	Attachment weight	lb (kg)	1301 (590)	1301 (590)

*At the center-of-gravity distance 24in (600mm). Based on 80% of full turn tipping load as defined by ISO 8313. Manual fork spacings at 2in (50mm) increments. Class 4A Fork section 6in x 2.4in (150mm x 60mm).

BUCKET SELECTOR

Material	Loose density lb/yd³	Loose density kg/m³	Fill factor %
Snow (fresh)	337	200	110
Peat (dry)	674	400	100
Sugar beet	894	530	100
Coke (loose)	961	570	85
Barley	1012	600	85
Petroleum coke	1146	680	85
Wheat	1231	730	85
Coal bitumous	1290	765	100
Fertilizer (mixed)	1737	1030	85
Coal anthracite	1764	1046	100
Earth (dry) (loose)	1939	1150	100
Nitrate fertilizer	2180	1250	85
Sodium chloride (dry) (salt)	2192	1300	85
Cement Portland	2428	1440	100
Limestone (crushed)	2580	1530	100
Sand (dry)	2613	1550	100
Asphalt	2698	1600	100
Gravel (dry)	2782	1650	85
Clay (wet)	2832	1680	110
Sand (wet)	3187	1890	110
Fire clay	3507	2080	100
Copper (concentrate)	3878	2300	85
Slate	4721	2800	100
Magnetite	5402	3204	100

A GLOBAL COMMITMENT TO QUALITY

JCB's total commitment to its products and customers has helped it grow from a one-man business into one of the world's largest manufacturers of backhoe loaders, crawler excavators, wheeled excavators, telescopic handlers, wheeled loaders, dump trucks, rough terrain fork lifts, industrial fork lifts, mini/midi excavators, skid steer loaders and tractors.

By making constant and massive investments in the latest production technology, the JCB factories have become some of the most advanced in the world.

By leading the field in innovative research and design, extensive testing and stringent quality control, JCB machines have become renowned all over the world for performance, value and reliability.

And with an extensive dealer sales and service network in over 150 countries, we aim to deliver the best customer support in the industry.

Through setting the standards by which others are judged, JCB has become one of the world's most impressive success stories.

JCB Headquarters Savannah, 2000 Bamford Blvd., Savannah, GA 31322. Tel: 912.447.2000. Fax: 912.447.2299. www.jcb.com

DWUSA 3243 05/13

Attachment 4

Easy-to-clean, smooth-wall interior

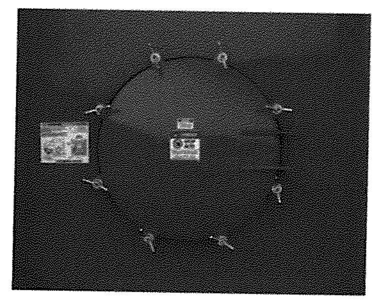
16,380 Gallon
Double-Wall Tank

Capacity: 16,380 gal (390 bbl)
Height: 9' 8"
Width: 8' 6"
Length: 46'
Tare Weight: 38,000 lbs

All sizes are approximate

At Adler Tank Rentals, we are committed to providing safe and reliable containment solutions for all types of applications where performance matters.

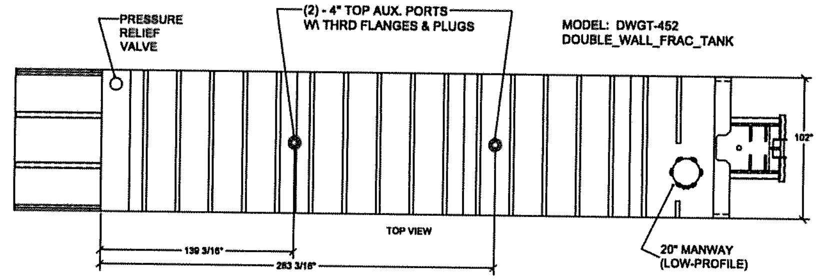
Providing maximum protection against potentially hazardous spill risk and environmental contamination, the 16,380 Gallon Double-Wall Tank ensures full secondary containment of both hazardous vapors and the tank's liquid contents.

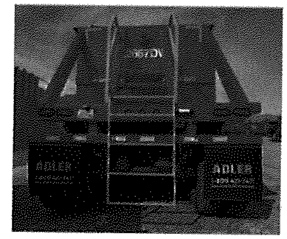


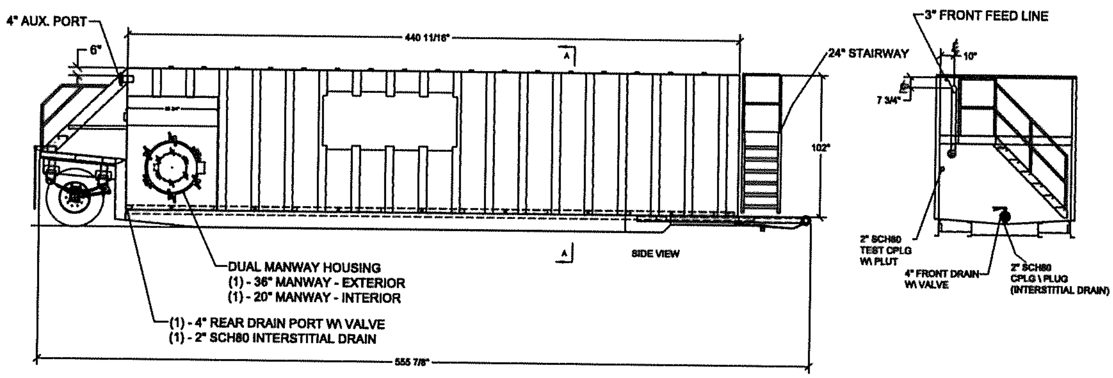
Mechanical Features

- Epoxy-coated interior
- 3" fill line
- Two (2) standard 20" side-hinged manways
- Two (2) 4" valved floor-level fill/drain ports valves for low point drain out
- 36" manway access to interstitial space
- 4" vent with 1 lb pressure/ 4 oz vacuum pressure relief valve
- Sloped and V bottom for quicker drain out and easier cleaning
- Easy-to-clean design with smooth-wall interior, no corrugations and no internal rods
- Two (2) 4" threaded and plugged auxiliary ports on roof
- Front-mounted ladderwell for top access
- Fixed rear axle for increased maneuverability
- Nose rail cut-out for easy access when installing hose and fittings on the front/bottom of tank
- 100% secondary containment; literally a tank built within a tank for storage of risk-potential materials in environmentally sensitive areas
- One (1) 2" interstitial space drain below 4" total drain

16,380 Gallon Double-Wall Tank







Tank configurations may vary in selected markets

Safety Features

- Non-slip step materials on ladderwells and catwalks
- "Safety yellow" rails and catwalks for high visibility
- Safe operation reminder decals

Options

- Bare steel interior
- Steam coils
- Audible alarms, strobes and level gauges (digital and mechanical)

Comprehensive Service

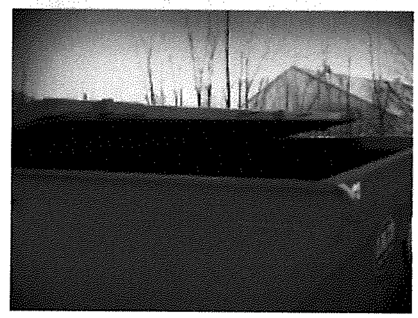
Adler Tank Rentals provides containment solutions for hazardous and non-hazardous liquids and solids. We offer 24-hour emergency service, expert planning assistance, transportation, repair and cleaning services. All of our rental equipment is serviced by experienced Adler technicians and tested to exceed even the most stringent industry standards.

ADLER TANK RENTALS

800-421-7471 www.adlertankrentals.com

Attachment 5
Attachment 5

25 YARD ROLL-OFF BOX WITH ALUMINUM HARD TOP



In Select Markets

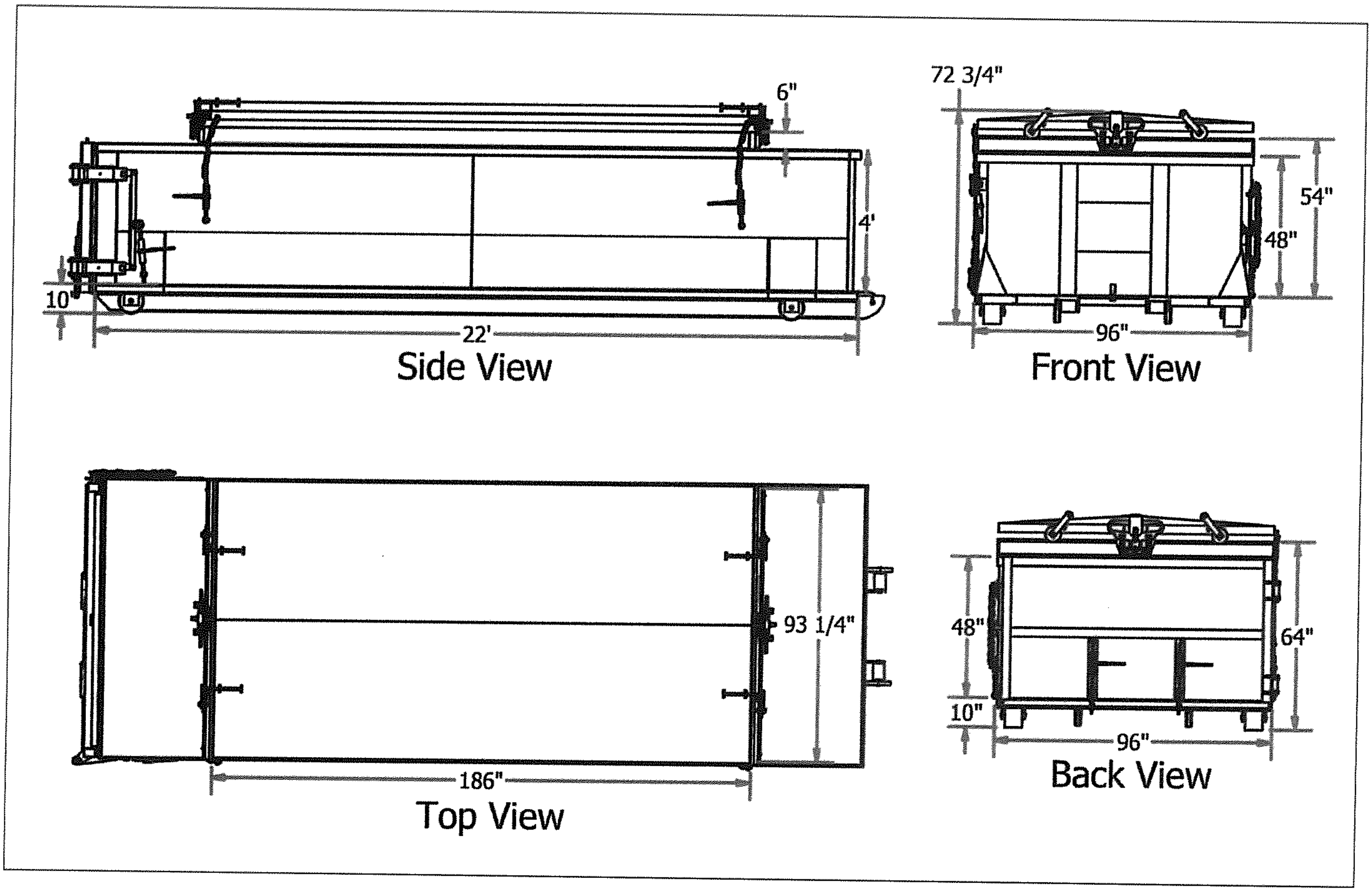
Capacity: 25 yd
Height: 6′
Width: 8′
Length: 23′

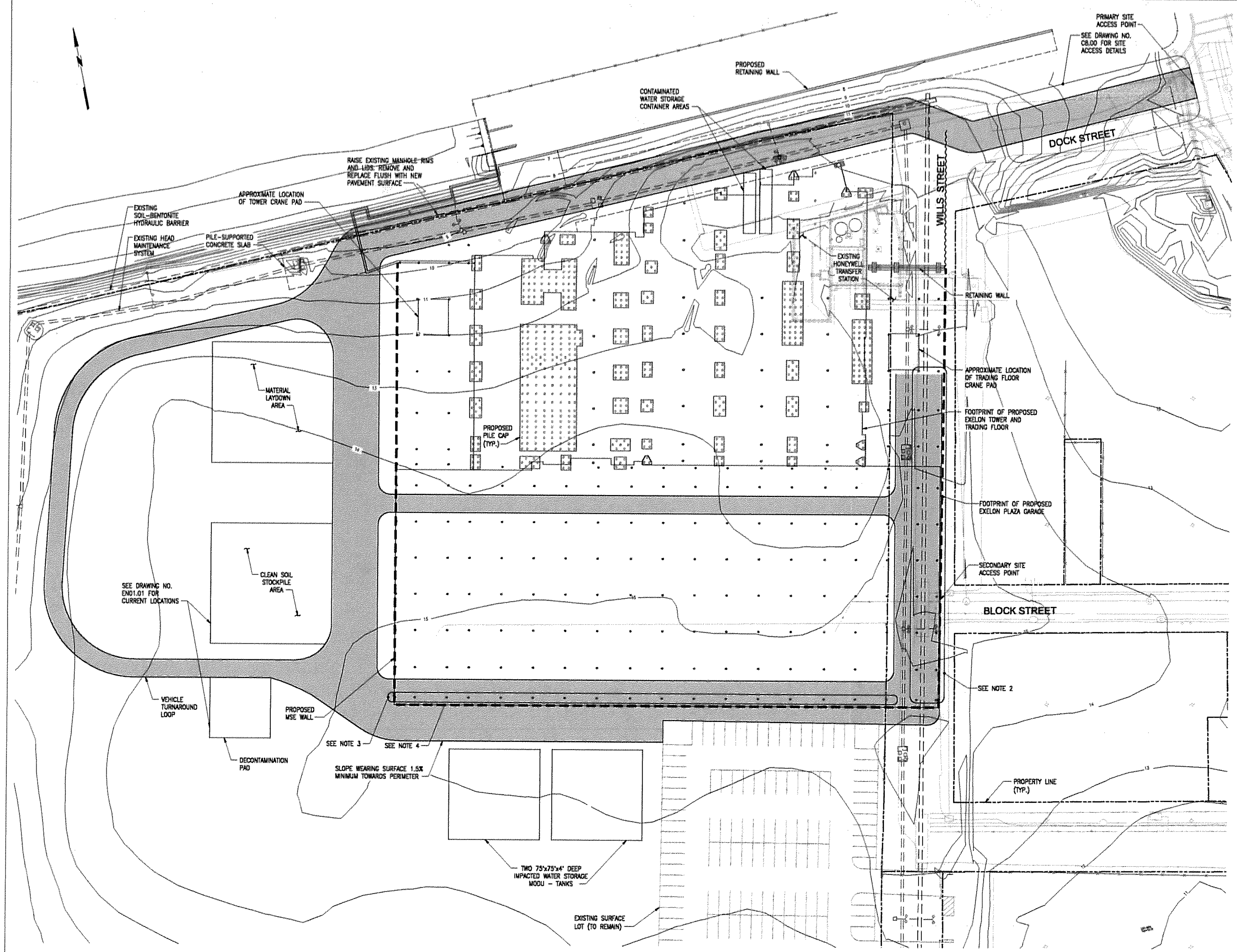
All sizes are approximate

Mechanical features:

- Rolling aluminum lid equipped with ratcheting binders to lock in place
- Plastic liners available upon request
- Compatible with standard roll-off frame truck

25 Yard Roll-Off Box With Aluminum Hard Top





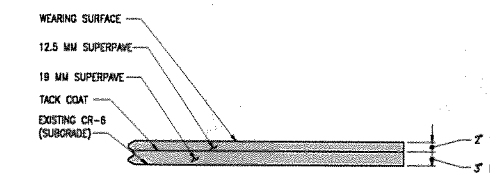
NOTES:

1. For General and Technical notes, see Drawings DDP-F1.01 and DDP-F1.02.
2. Piles within limits of proposed construction roads shall be driven, cut below asphalt sub-base, and protected by an interim cap of Cover Soil before road is paved. Piles should be filled with concrete after road use is no longer needed.
3. Construction access road in this area should be demolished prior to pile installation.
4. Construction access road in this area should be demolished up to 6' minimum from MSE wall.

LEGEND:

13 - As-Built elevation contour at grade

- Proposed Asphalt Construction Access Road



DETAIL 1: TYPICAL ASPHALT SECTION
SCALE: 1:1

GRAPHIC SCALE

PROGRESS SET NOT FOR CONSTRUCTION

BHC ARCHITECTS

1601 S. Exeter Street, Suite 200, Baltimore, MD 21202
Phone 410 752 2759

owner/developer

developer:
Beatty Development Group

consultants

design consultant:
Elkus Manfredi Architects

structural engineer:
Tadjer Cohen Edelson & Associates

mep & fp engineer:
Vanderweil

foundation engineers:
Mueser Rutledge Consulting Engineers

civil engineers:
Rummel, Klepper & Kahl

landscape architect:
Mahan Rykiel Associates

environmental engineer:
Environmental Resources Management

project
EXELON BLDG & PLAZA GARAGE

HARBOR POINT
AREA 1 PHASE 1
DDP SUBMISSION
7/1/13

key plan

seal

issued date description

revision date description

title
CONSTRUCTION ACCESS ROADS PLAN

date:	07/01/13
drawn by:	E.C.
checked by:	D.J.G.
scale:	1:30
project number:	MPCS-11896A
sheet number:	

DDP-F1.08

Attachment 7

Static Load Spreading of Design Truck with Asphalt
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name:	Exelon	Project Number :	11896A
Client	: 15 yd3 Concrete Truck	Project Manager:	GS
Date	: 6/24/2013	Computed by :	DJG

Footing #	Corner Point P1 X1(ft)	Corner Point P1 Y1(ft)	Corner Point P2 X2(ft)	Corner Point P2 Y2(ft)	Load (Ksf)
1	0.00	0.00	0.66	1.33	11.250
2	1.33	0.00	2.00	1.33	11.250
3	6.00	0.00	6.66	1.33	11.250
4	7.33	0.00	8.00	1.33	11.250
5	0.00	4.50	0.66	5.83	11.250
6	1.33	4.50	2.00	5.83	11.250
7	6.00	4.50	6.66	5.83	11.250
8	7.33	4.50	8.00	5.83	11.250

INCREMENT OF STRESS FOR
X = 0.33(ft) Y = 0.66(ft) Z = 2.92(ft)

Vert. Dsz
(Ksf)

0.93

Vert. Dsz + Asphalt Weight = 0.93 + (145pcf)*(0.42ft) = 0.99 ksf

Static and Dynamic Load Spreading of Design Truck with Asphalt
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : 15 yd3 Concrete Truck
Date : 6/24/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1		Corner Point P2		Load
	X1(ft)	Y1(ft)	X2(ft)	Y2(ft)	(Ksf)
1	0.00	0.00	0.66	1.33	14.960
2	1.33	0.00	2.00	1.33	14.960
3	6.00	0.00	6.66	1.33	14.960
4	7.33	0.00	8.00	1.33	14.960
5	0.00	4.50	0.66	5.83	14.960
6	1.33	4.50	2.00	5.83	14.960
7	6.00	4.50	6.66	5.83	14.960
8	7.33	4.50	8.00	5.83	14.960

INCREMENT OF STRESS FOR
X = 0.33(ft) Y = 0.66(ft) Z = 2.92(ft)

Vert. Dsz
(Ksf)

1.24

Vert. Dsz + Asphalt Weight = 1.24 + (145pcf)*(0.42ft) = 1.30 ksf

Static Load Spreading of Wheel Loader with Asphalt
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : Wheel Loader
Date : 6/27/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1		Corner Point P2		Load
	X1(ft)	Y1(ft)	X2(ft)	Y2(ft)	(Ksf)
1	0.00	0.00	1.60	1.06	9.020
2	0.00	10.83	1.60	11.89	9.020
3	6.83	10.83	8.43	11.89	9.020
4	6.83	0.00	8.43	1.06	9.020

INCREMENT OF STRESS FOR
X = 0.80(ft) Y = 0.53(ft) Z = 2.92(ft)

Vert. Dsz
(Ksf)

0.80

Vert. Dsz + Asphalt Weight = 0.80 + (145pcf)*(0.42ft) = 0.86 ksf

Static and Dynamic Load Spreading of Wheel Loader with Asphalt
RECTANGULAR LOADS
UNIFORM VERTICAL

Project Name: Exelon
Client : Wheel Loader
Date : 6/27/2013

Project Number : 11896A
Project Manager: GS
Computed by : DJG

Footing #	Corner Point P1 X1(ft)	Corner Point P1 Y1(ft)	Corner Point P2 X2(ft)	Corner Point P2 Y2(ft)	Load (Ksf)
1	0.00	0.00	1.60	1.06	12.000
2	0.00	10.83	1.60	11.89	12.000
3	6.83	10.83	8.43	11.89	12.000
4	6.83	0.00	8.43	1.06	12.000

INCREMENT OF STRESS FOR
X = 0.80(ft) Y = 0.53(ft) Z = 2.92(ft)

Vert. Dsz
(Ksf)

1.06

Vert. Dsz + Asphalt Weight = 1.06 + (145pcf)*(0.42ft) = 1.12 ksf

MUESER RUTLEDGE CONSULTING ENGINEERS File: 11896A

Made By: DJG Date: 6/24/2013

FOR: Exelon Checked By: AMD Date: 6/28/2013

SUBJECT: Calculation 1: Static, Dynamic, and Asphalt Load Application Calculations

Static Applied Stress Calculation - Design Truck (See Ref. 2 for axle/wheel layout):

$w := 0.667\,ft$ $\quad l := 1.333\,ft$ — Dimensions of Contact with Ground of a Single Wheel (8" x 16")

$A := w \cdot l$ $\quad A = 0.89\,ft^2$ — Contact Area of a Single Wheel

$P := 10\,kip$ — Applied Load per Wheel

$\sigma_s := \frac{P}{A}$ $\quad \sigma_s = 11.25\,ksf$ — Bearing Stress at Grade per Wheel

Dynamic Applied Stress Calculation - Design Truck (Ref. 3):

$D_E := 0$ — Embedment Depth of Applied Load

$IM := 33 \cdot (1 - 0.125 \cdot D_E)$

$IM = 33$

Dynamic Load Allowance for Drainage Net
(Additional Percentage of Static Response Applied at Grade)

$\sigma_d := \frac{IM}{100} \cdot \sigma_s$

$\sigma_d = 3.71\,ksf$

Additional Allowable Dynamic Load

$\sigma_T := \sigma_s + \sigma_d$

$\sigma_T = 14.96\,ksf$

Static plus Dynamic Applied Load at Grade
from the Design Truck

Asphalt Applied Stress Calculation:

$\gamma_{asp} := 145\,pcf$ — Assumed Unit Weight of Asphalt

$D_{asp} := 5\,in$ — Recommended Height for Asphalt for Construction Roads (as per Ref. 7)

$\sigma_{asp} := \gamma_{asp} \cdot D_{asp}$

$\sigma_{asp} = 0.06\,ksf$ — Additional CR-6 Applied Stress due to Construction Roads

MUESER RUTLEDGE CONSULTING ENGINEERS File: 11896A

Made By: DJG Date: 6/24/2013

FOR: Exelon Checked By: AMD Date: 6/28/2013

SUBJECT: Calculation 1: Static, Dynamic, and Asphalt Load Application Calculations

Static Applied Stress Calculation - Wheel Loader (See Attachment 3):

$W_o := 43195lb$		Wheel Loader Operating Weight
$W_f := 18576lb$		Front Axle Weight
$W_r := 24619lb$		Rear Axle Weight
$W_p := 12082lb$		Payload
$W_{front} := W_f + W_p$		
$W_{front} = 30658\ lb$		Maximum Load on Front Axle
$P := \dfrac{W_{front}}{2}$	$P = 15329\ lb$	Maximum Load per Wheel on Front Axle
$w := \dfrac{P}{0.8}$	$w = 1.597\ ft$	Width of Contact Area of Wheel (Ref. 3)
$\gamma := 1.50$		Load Factor (Ref. 3)
$l := 6.4\gamma \cdot \left(1in + \dfrac{IM}{100}\right)$		
$l = 1.06\ ft$		Length of Contact Area of Wheel (Ref. 3)
$A := w \cdot l$	$A = 1.699\ ft^2$	Contact Area of a Single Wheel
$P = 15329\ lb$		Applied Load per Wheel
$\sigma_s := \dfrac{P}{A}$	$\sigma_s := 9.02ksf$	Bearing Stress at Grade per Wheel

MUESER RUTLEDGE CONSULTING ENGINEERS File: 11896A

Made By: DJG Date: 6/24/2013

FOR: Exelon Checked By: AMD Date: 6/28/2013

SUBJECT: Calculation 1: Static, Dynamic, and Asphalt Load Application Calculations

Dynamic Applied Stress Calculation - Wheel Loader (Ref. 3):

$D_E := 0$ Embedment Depth of Applied Load

$IM := 33 \cdot (1 - 0.125 \cdot D_E)$

$IM = 33$ Dynamic Load Allowance for Drainage Net
(Additional Percentage of Static Response Applied at Grade)

$\sigma_d := \frac{IM}{100} \cdot \sigma_s$

$\sigma_d = 2.98\,\text{ksf}$ Additional Allowable Dynamic Load

$\sigma_T := \sigma_s + \sigma_d$

$\sigma_T = 12\,\text{ksf}$ Static plus Dynamic Applied Load at Grade
from the Wheel Loader

SUBJECT CALCULATION 2: WATER AND SOIL CONTAINERS APPLIED LOAD CALCULATIONS

1. 16380 GALLON DOUBLE-WALL TANK (SEE ATTACHMENT 4)

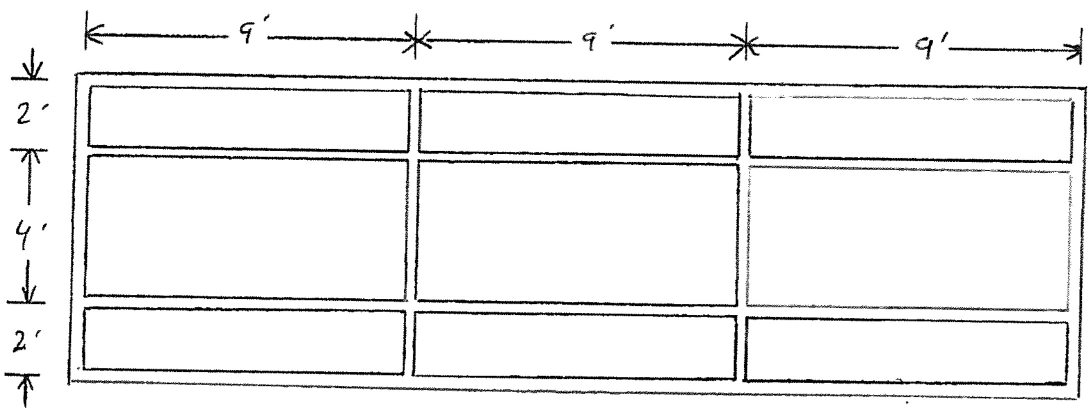
LOADS

- TANK BEARS ON FRAMEWORK OF 4" WIDE STEEL SKIDS (SHOWN BELOW) AND IS ASSUMED FILLED TO CAPACITY WITH WATER.
- TARE WEIGHT: 38000 lbs
- PAYLOAD

$$16380 \text{ gal}\left(\frac{8.35 \text{ lb}}{1 \text{ gal}}\right) = 136773 \text{ lbs}$$

- TOTAL MAXIMUM WEIGHT = 38000 + 136773 = 174773 lbs ≈ 175000 lbs

LAYOUT



GROUND CONTACT AREA OF DOUBLE-WALL TANK

PLAN VIEW

AREA

LONGITUDINAL: $(9'+9'+9')(4'')(4 \text{ SKIDS})(12 \text{in}/1 \text{ft}) = 5184 \text{ in}^2$

TRANSVERSE: $[(2'+4'+2')(4'')(12 \text{in}/1 \text{ft}) - 4 \text{ SKIDS}(4'' \times 4'')] 4 \text{ SKIDS}$
$= 1280 \text{ in}^2$

TOTAL $= 5184 + 1280 = 6464 \text{ in}^2$

BEARING STRESS $= \dfrac{175000 \text{ lb}}{6464 \text{ in}^2} = 27.1 \text{ psi} = 3.90 \text{ ksf}$ (ASSUMED UNIFORM)

→ ACCORDING TO WINSTRESS, MAXIMUM BEARING STRESS AT DRAINAGE NET IS 0.74 KSF < 2.0 KSF. ∴ NO REINFORCEMENT REQUIRED.

SUBJECT CALCULATION 2: WATER AND SOIL CONTAINERS APPLIED LOAD CALCULATIONS

2. 25 YARD ROLL-OFF BOX WITH ALUMINUM HARD-TOP (SEE ATTACHMENT 5)

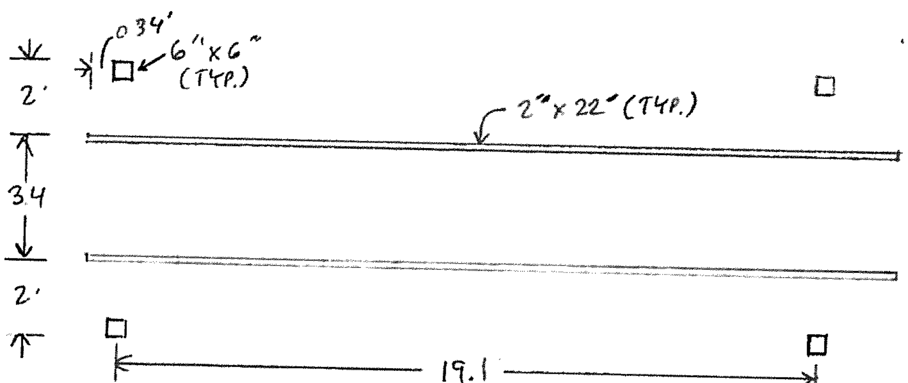
LOADS

- BOX BEARS ON FOUR 8"x10" WHEELS (ASSUMED 6"x10" CONTACT AREA) AND ASSUMED TO ALSO BEAR ON TWO 2" WIDE, 22' LONG SKIDS.
- BOX IS ASSUMED TO BE FILLED TO CAPACITY WITH SOIL, DEBRIS, WATER, CONCRETE FRAGMENTS, VOID SPACE, ETC ~ 125 PCF.
- TARE WEIGHT = 5000 lbs
- PAYLOAD:

$$(\text{CAPACITY})(\text{UNIT WEIGHT OF CONTENTS}) = (25\,yd^3)(125\,pcf)(27\,ft^3/yd^3) = 84375\text{ lbs}$$

- TOTAL MAXIMUM WEIGHT = 5000 + 84375 = 89375 ≈ 90000 lbs

LAYOUT



GROUND CONTACT AREA OF ROLL-OFF BOX

PLAN VIEW

AREA

WHEELS: $(6" \times 6")(4\text{ WHEELS}) = 144\text{ in}^2$

SKIDS: $(22')(12\text{in}/1\text{Ft})(2")(2\text{ SKIDS}) = 1056\text{ in}^2$

TOTAL: $144 + 1056 = 1200\text{ in}^2$

BEARING STRESS: $\dfrac{90000\text{ lbs}}{1200\text{ in}^2} = 75\text{ psi} = 10.80\text{ ksf}$

→ ACCORDING TO WINSTRESS, MAXIMUM BEARING STRESS AT DRAINAGE NET IS 0.53 KSF < 2.0KSF ∴ NO REINFORCEMENT REQUIRED.

MUESER RUTLEDGE CONSULTING ENGINEERS
CALCULATION #4

SHEET 1 OF 1
FILE : 118964-40
MADE BY: AMD 7/15/1[illegible]
CHK'D BY: DJG 7/16/1[illegible]

PROJECT: EXELON TOWER & TF GARAGE

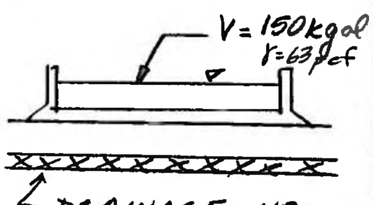
SUBJECT: LOAD ON DRAINAGE NET FROM MODU-TANKS

Modutank @ Site will be similar in construction to image @ center.

SIZE : 75 x 75 ft

CAPACITY : 150K gal

LIQUID STORAGE FOR CONTAMINATED WATER : $\gamma = 8.42 \text{ lbs/gal}$



$q_{ALL} = 1 \text{ tsf}$

$$q_{APP} = \frac{150\text{kgal} \cdot 8.42 \text{ lb/gal}}{75^2 \text{ sf}}$$

$$= \frac{2000 \cdot 8.42 \text{ lb}}{75 \text{ sf}}$$

$$= 225 \text{ psf} = 0.113 \text{ tsf}$$

$0.113 \text{ tsf} < 1.0 \text{ tsf}$ ∴ q_{APP} IS OK.

Mueser Rutledge
Consulting Engineers
14 Penn Plaza · 225 West 34th Street · New York, NY 10122
Tel: (917) 339-9300 · Fax: (917) 339-9400
www.mrce.com

MEMORANDUM

Date: August 5, 2013
To: Office
From: Matthew Goff
Re: EE Memo 9 – Pile Supported MMC & HMS above Dock Street Bulkhead
Exelon Building & Plaza Garage, Baltimore, MD
File: 11896A-40

This memorandum summarizes the design and analysis of the pile supported platform, which supports the HMS and MMC along Dock Street.

Exhibits
Sketch 1 Connection of Concrete Slab to Existing Vault
Sketch 2 Retaining Wall Cross Section

Available Information
1. Drawing DDP F1.40 – Foundation Plan
2. Drawing DDP F1.42 – Foundation Partial Plan
3. Drawing DDP F1.52 – Foundation Details and Sections
4. Drawing 1000C – General Plan
5. Drawing 1001C – Bulkhead Type A Plans and Sections
6. Drawing 1002C – Bulkhead Types B and C Plans and Sections

Pile-Supported MMC & HMS
The multimedia cap (MMC) and head maintenance system (HMS) components are supported by a structural system consisting of a two-way concrete slab supported on steel pipe piles. The purpose of the structure is to support the MMC and HMS, and to prevent settlement of the street and utilities caused by potential deterioration of the bulkhead and the proposed raised grades along Dock St. The limits of the pile-supported Dock St. platform extend from the sheet pile barrier wall along Wills St. at MJ1, to the west side of Vault V-11, shown on Drawings DDP-F1.40 and DDP-F1.42.

The pile supported platform is proposed both due to the presence of an existing timber bulkhead located below existing grade along Dock St. and the presence of compressible clay west of Vault V-12. The estimated settlement under development fill is addressed in EE Memo 1. The timber frame of the existing bulkhead consists of a timber headwall, which is supported by timber tiebacks anchored to timber deadmen and timber piles. The headwall, granite block headwall, and deadmen are oriented in the east-west direction and the tiebacks are oriented in the north-south direction. The existing timber tiebacks and deadmen are located at approx. Elev. +1 to Elev. 0. The existing timber bulkhead is presumed to be in poor condition and further deterioration could lead to settlement of overlying structures. The location of the existing timber bulkhead is based on a 1989 survey performed by

Greenhorne and O'Mara and is shown on Drawing Nos. 1000C, 1001C, and 1002C. The existing timber deadmen below the pile-supported slab are also shown on Drawing DDP-F1.42.

In addition to the structural system, the pile-supported MMC also consists of a protective 6" concrete slab over synthetic layers that extend across the top of the structural slab. At the existing soil-bentonite barrier wall, the new "sheet pile barrier" is extended into the concrete slab to support the platform and to create a seal between the platform and the barrier. To the south of the pile-supported concrete slab, the synthetic layers at the top of the structural slab (Elev. +8.5) are sealed to synthetic layers of the existing MMC (Elev. +8) (Valley Drain). The process of connecting the two sets of synthetic layers is shown on Drawings DDP-F1.21 through DDP-F1.24.

Design of Structural System

The structural system is designed to support traffic loading, the HMS vaults, the protective slab, the concrete retaining walls, and the soil above the structural slab. The vehicle live load is assumed to be a uniform distributed load of 250 psf. This design live load is taken from Table 4-1 "Minimum Uniformly Distributed Live Loads" of ASCE 7-05 for sidewalks and vehicle driveways subject to trucking. The proposed roadway elevation above the pile-supported slab ranges from approx. Elev. +14 at Wills St. and Dock St. to approx. Elev. +19 at Dock St. and Point St.

Two design sections were chosen for the pile-supported concrete slab design. Design Section 1 (DS-1) has a proposed street elevation of Elev. +19 and Design Section 2 (DS-2) has a proposed elevation of Elev. +15. DS-1 is used for design of the pile-supported slab to the west of column line C and DS-2 is used to the east of column line C. The structural elements of the pile-supported slab were designed for the retained and supported soil from these two design sections. These structural elements consist of the two-way concrete slab, concrete retaining wall, and steel pipe piles.

The structural concrete slab is 18" thick with a top elevation of Elev. +8.5. It is designed as a two-way slab that spans between steel pipe piles in both the north-south and east-west directions. Sections are shown on Drawing DDP-F1.53.

In addition to supporting the roadway loading and soil weight, the structural slab supports the HMS components. The caisson HMS pipes are supported on hanger rods embedded into the slab. Refer to DDP-EN1.01 for additional information on the HMS hanger supports.

The two-way slab (without girders) should largely be constructed above the MMC synthetic layers. During construction, it is likely that obstructions (primarily elements of the existing timber bulkhead) may be encountered while installing the steel pipe piles. With the two-way slab, the pipe piles can be relocated two feet in any direction to avoid obstructions if the location of adjacent pipe piles is not altered.

In addition to supporting the soil and vehicle loading, the two-way slab is also designed to support vaults V-11 and V-12 and the manhole at the intersection of Dock St. and Wills St. The vaults and manhole are connected with dowels to the two-way slab along all four sides of the structure. The typical connection between the vaults and two-way concrete slab is shown on Sketch 1.

In the area of DS-1 near the intersection of Dock St. and Point St., the piles and structural slab also support the concrete retaining wall. The retaining wall runs along the northern edge of the pile-supported slab, and then turns south at Point St. and extends over the top of the structural slab. The retaining wall then turns east along the southern edge of the pile-supported slab and follows the face of the Exelon buildings. The location of the retaining walls is shown on Drawings DDP-F1.40 and DDP-F1.42. A section through the western retaining wall looking north is shown on Sketch 2.

The retaining wall along the face of the building to the south extends upward from the pile-supported structural slab to the base slab of the building. This wall retains soil from above the pile-supported slab to below the building slab to the south. The wall extends along the face of the building up to the point where proposed grade and existing grade at the face of the building are the same.

The cantilever retaining walls are designed to laterally support the soil fill under the proposed roadway and vehicle surcharge. The top of the wall extends to the elevation of proposed grade. At its tallest section, the wall extends from the top of structural slab at Elev. +8.5 to proposed grade at Elev. +19. The wall dimensions taper from 2'-0" at the bottom to 1'-6" at the top. The base moment and shear from the lateral pressure on the wall are transferred into the two-way slab below the wall. The two-way slab distributes the lateral and vertical load to the piles.

Steel pipe piles support the two-way concrete slab. The pipe piles are 16" in diameter and provide adequate capacity for the loading of both design sections. In order to reduce the number of pipe piles and the size of the concrete slab, the sheet pile wall in the S-B barrier wall was designed as an additional support for the slab. Utilizing the sheet pile wall as a support location eliminates a row of pipe piles.

The north-south spacing and location of the steel pipe piles have been specifically selected to avoid conflict with the existing timber bulkhead and damage to the existing HMS. Pile locations may need to be shifted east-west to avoid timber tiebacks which are at approximately 8-ft spacing. To prevent excessive pile driving damage to the existing HMS conduits, a clearance of 3' is maintained from the outside edge of the HMS conduits to the rows of pipe piles.

The locations of the existing timber bulkhead were ascertained from the 1989 Greenhorne and O'Mara survey. The timber headwall and deadmen locations of Bulkhead Type A and Bulkhead Types B and C have been taken from this survey and are shown on Drawing DDP-F1.42. However, the exact locations of the timber tiebacks are not known from the 1989 survey information. The tiebacks are shown to be spaced at 8' +/-. To avoid conflict with the existing timber tiebacks and deadmen, the pipe piles have been placed in the open bays between the rows of timber deadmen and spaced at intervals of 8' and 16' on center. Once the location of an existing timber tieback is determined by probing, this spacing and arrangement should allow for the pipe piles to be installed in these open bays with minimal obstructions encountered.

By: Matthew Goff
Matthew Goff

SHEET____ OF____

MUESER RUTLEDGE CONSULTING ENGINEERS

FILE 11896

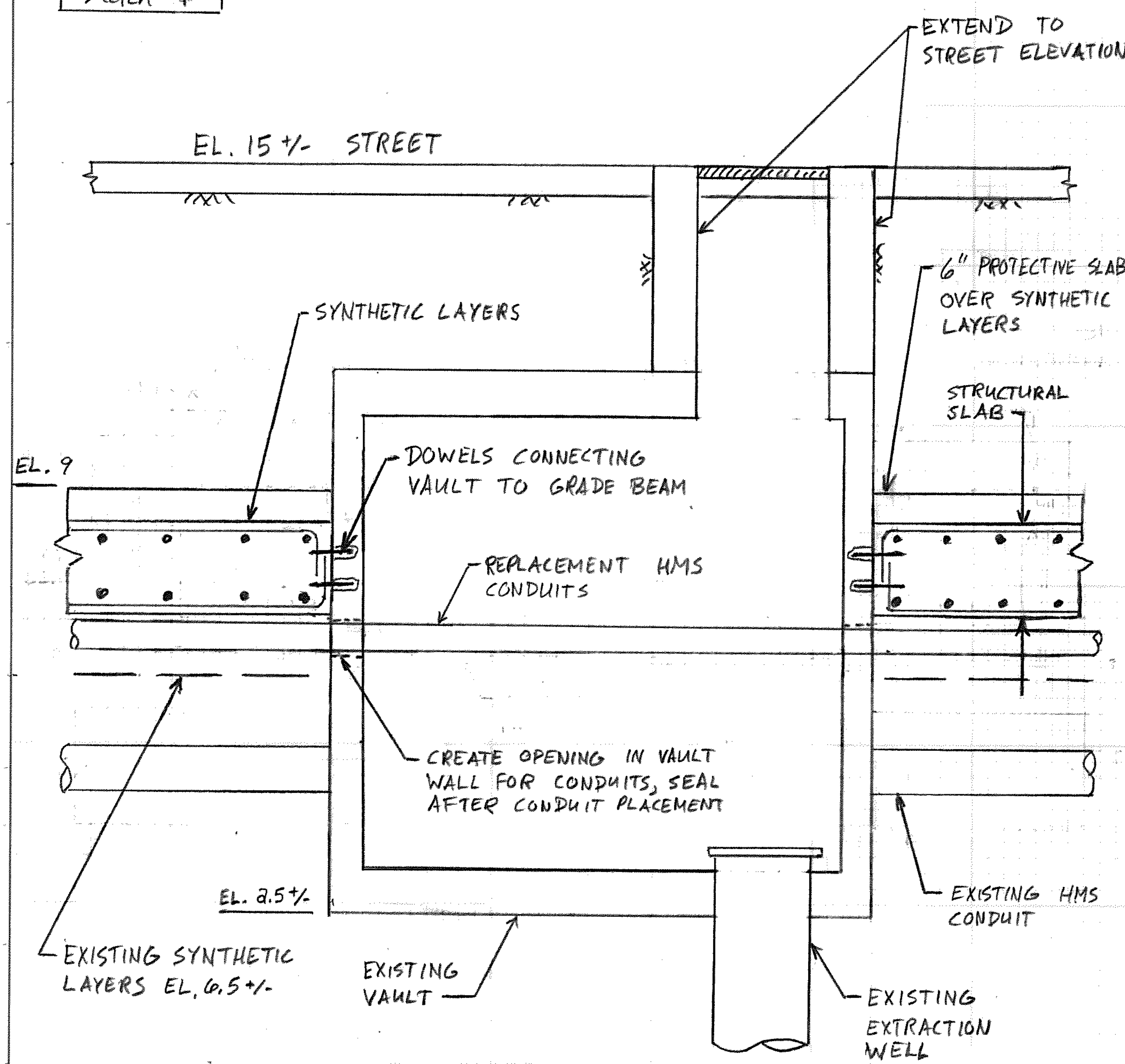
PROJECT Exelon - Dock Street

MADE BY MSG DATE 7/15/13

CHECKED BY SY DATE 7/15/13

SUBJECT Connection of Concrete 2-Way Slab to Existing Vault Concept

Sketch 1



SECTION THROUGH EXISTING VAULT

MUESER RUTLEDGE CONSULTING ENGINEERS

SHEET ____ OF ____

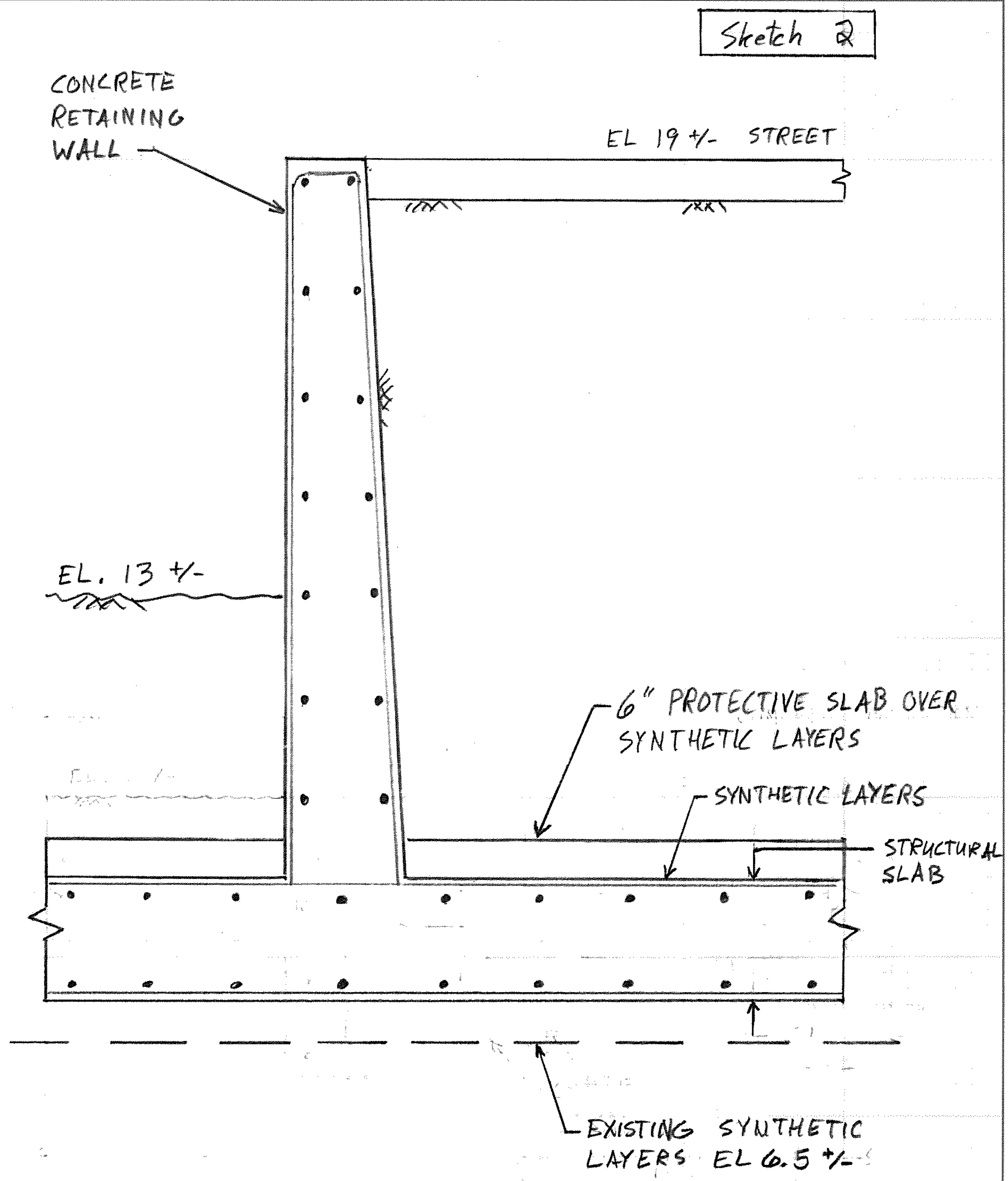
FILE 11896

MADE BY MSG DATE 7/15/13

CHECKED BY SY DATE 7/15/13

PROJECT Exelon - North South Retaining Wall
West of Point St.

SUBJECT Retaining Wall Cross Section — Concept



SECTION THROUGH RETAINING WALL
LOOKING NORTH

Memorandum

Environmental
Resources
Management

200 Harry S. Truman
Parkway, Suite 400
Annapolis, MD 21401
(410) 266-0006
(410) 266-8912 (fax)

ERM

To: Adam Dyer
Geotechnical Engineer

Company: Mueser Rutledge Consulting Engineers

From: Spencer Pierini

File number: 0166008

Date: June 18, 2013

Subject: Engineering Evaluation Memorandum No. 8

REPLACE GAS FIRED UNIT HEATER WITH ELECTIC HEATERS:

Gas fired unit heaters UHG-201,201&203 will be replaced by equivalent electric powered units to maintain the thermal conditions within the tank room. The three existing gas fired heaters consist of two units that are rated at 45,600 BTUH and one at 33,200 BTUH. Replacement electric powered unit heaters shall be sized as follows: two (2) at 15kW and one (1) at 7.5kW. Each unit heater shall have an integral adjustable thermostat and disconnect switch. Contractor shall source electrical power from the adjacent electric room and install the power feed in accordance with NEC. The cut sheets for the proposed heaters are attached.

INSTALL FAN TEMPORARILY TO MAINTAIN POSITIVE PRESSURE:

A filtered air supply fan shall be installed in the electric room to filter the air delivered to the room to eliminate the potential for dust intrusion from construction activities and positively pressurize the room. The fan filter unit is sized at 1750 CFM and intended to operate continuously. The fan filter shall be ceiling hung on vibration isolators and positioned such that the filter section is accessible for filter changes. Contractor shall source electrical power from the adjacent electric room and install the power feed in accordance with NEC and provide a disconnect switch at the unit. The cut sheets for the proposed fan are attached.

INSTALL PERMANENT EXHAUST FAN AND LOUVERS:

The existing Exhaust Fans EF-201, and EF-202 that are rated for 1,850 cfm each (3,700 cfm total), will be replaced with a single exhaust fan with acoustical louver capable of 3,700 cfm as detailed on sheet M4.07. The exhaust fan motor will have a nominal rating of 208 volts, 3 phase, 60 HZ.

A new intake louver will also be installed to replace the existing intake louver L-201. The new intake louver will be sized to accommodate the proposed 3,700 cfm exhaust fan. The electrical/mechanical, and storage room along with the new office space will be supplied with conditioned air system with air return. The cut sheets for the proposed exhaust fan and acoustical louver will be provided by the MEP Contractor. All existing exhaust fans and intake louvers will be demolished and restored in accordance with architectural plans.

PUMP SIZE FOR SUMP PUMP:

The existing pump shall be relocated to the new sump at the new loading dock area. The existing submersible centrifugal pump has 2-inch discharge and is driven by 0.5 HP, submersible motor with a nominal rating 208 volts, 3-phase, 60 HZ, 3,500 RPM. The existing pump has the capacity to deliver 40 GPM flow at 30 feet of total dynamic head.

The pump at the new sump will be installed at the same elevation as it is in the existing sump (existing sump floor elevation 11 feet and new sump floor elevation approximately 10.5 feet). The discharge at the tank will be at the same elevation. Therefore, the elevation head will not change. The frictional head loss in piping will be less than existing because of reduced pipe length. The pipe size and material will be similar to existing (2-inch rigid PVC). The total dynamic head would be slightly less than existing because of less frictional head loss. Thus, the existing pump is sufficiently sized to transfer sump water into the tank inside tank room.