Surface Flood-induced Groundwater Seepage Modeling for Coastal Flood Hazard Areas

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Abstract: Under a newly raised design flood elevation (DFE) for resilience design, many existing buildings need significant structural improvements to meet the structural capacity requirements and comply with the current design standards. The proposed structural improvements are often times not only exorbitantly costly but also face constructability challenges due to site restrains. This paper presents the methodology and results of a numerical transient seepage analysis, which could allow architects and structural engineers to design for a refined, more realistic lateral pressure profile for foundation elements and uplift pressure for interior slabs. The paper summarizes a case study performed at a number of public housing sites in Manhattan, in the close vicinity of the East River. SEEP/W (GeoStudio), a 2-dimensional finite element seepage analysis module, was used to simulate time-dependent, groundwater seepage response primarily due to surface water infiltration. The DFEs were based on the 1-percent annual exceedance probability flood, a required freeboard and a projected future sea level rise. The transient simulations adopted a stage hydrograph (i.e., water level versus time), whose shape closely resembles the observed water levels in the New York Harbor during Hurricane Sandy in 2012. Transient groundwater seepage simulations were performed to evaluate the temporal response in the vicinity of the foundation under the design flood hydrograph.

I INTRODUCTION

Hurricane Sandy 2012 started out as a classic late-season hurricane. After multiple landfalls in the Caribbean, the cyclone quickly weakened and went through a complex evolution in the Bahamas. The system re-strengthened into a hurricane while it moved northeastward, parallel to the coast of the southeastern United States. Sandy made landfall as a post-tropical cyclone in Southern New Jersey with 70-kt maximum sustained winds on October 29, 2012. Because of its tremendous size, Sandy resulted in a catastrophic storm surge along the New Jersey and New York coastlines. Preliminary U.S. damage estimates were near $50 billion (Blake et al., 2013). The financial loss for the New York City alone was estimated to be $19 billion (Greenhalgh, 2013). Sandy was considered the second-costliest cyclone to hit the United States since 1900.

As Sandy travelled northward along its track, it caused water levels to rise along the entire east coast of the United States from Florida to Maine. The highest storm surges and most severe inland inundation occurred in New Jersey, New York, and Connecticut, especially in and around the New York City metropolitan area. Figure 1 presents a high resolution storm surge inundation map for the Lower Manhattan and its neighboring boroughs. The source ArcGIS data is available online and was compiled and published by Federal Emergency Management Agency (FEMA) Modeling Task Force (MOTF) (FEMA, 2014).

The highest stillwater level during Sandy was recorded at a National Oceanic and Atmospheric Administration (NOAA) / National Ocean Service (NOS) station at the Battery Park in Manhattan, New York, reaching a record high of elevation 11.3 feet, North American Vertical Datum 1988 (NAVD88) (Blake et al., 2013). The location of the NOS tide gage is shown on Figure 1. This observed water level was approximately 9.4 feet above the normal astronomical tide. Figure 2 presents the time series of the astronomical tides, observed water levels and storm surge heights between October 28 and 31, 2012 at the Battery from NOAA's Tides and Currents website.

Storm surge is defined as an abnormal rise in water level above normal, astronomical tides, mainly due to wind stresses and pressure differentials from a storm and commonly expressed as:

$$\eta = H - h$$

where

η = storm surge height in feet
H = observed water level in feet (referenced to a given vertical datum)
h = astronomical tide in feet (referenced to a given vertical datum)

For the northeast region of the Unite States, storm surge events were mostly caused by hurricanes and extratropical storms, a representative form of which are "Nor'easters" with characteristic, strong winds predominantly from the northeast direction.

Figure 1: FEMA MOTF's High Resolution Storm Surge Inundation Map for New York City

After Hurricane Sandy, region-wide recovery and reconstruction efforts were carried out as a collaborative effort of Federal, State and local governments. One of the key strategies of post-Sandy rebuilding programs is to build with improved resilience to better prepare communities to withstand future storms and other risks posed by a changing climate. In the meantime, FEMA, leading and managing the National Flood Insurance Program (NFIP), rolled out a set of new, updated Flood Insurance Rate Maps (FIRM) for the New York metro area between 2013 and 2015, based on the FEMA Region 2 study, which was initiated in 2009 to update the 25-year old FIRMs in the region with the best available source data and methodology. Figure 3 presents a comparison of the published stillwater elevations near the Battery Park between the 2007 and 2013 Flood Insurance Studies (FIS) (FEMA, 2007; FEMA, 2013). The newly revised (preliminary) 2013 FIS proposed a higher stillwater flood frequency curve for this area than the previous 2007 version. Based on Figure 3, the peak stillwater level at the Battery generated by Sandy was a 1-in-100 year event (annual exceedance probability of 0.01) according to the 2013 FIRM and a 1-in-500-year event according to the 2007 FIRM. This is a significant difference in terms of the estimated return period of this storm surge event.

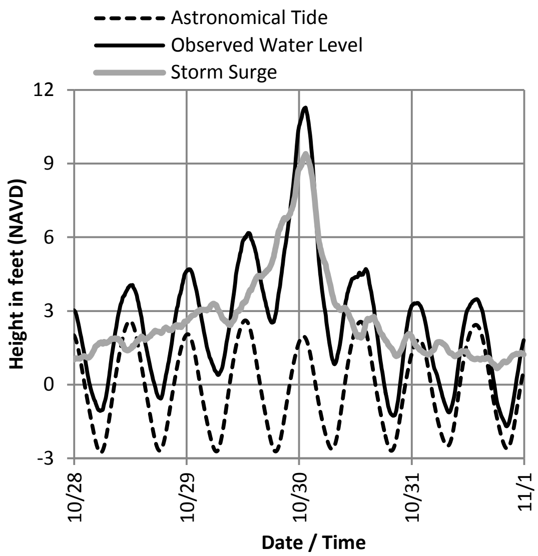

Figure 2: Water Levels at Station 8518750, The Battery, New York during Hurricane Sandy in 2012

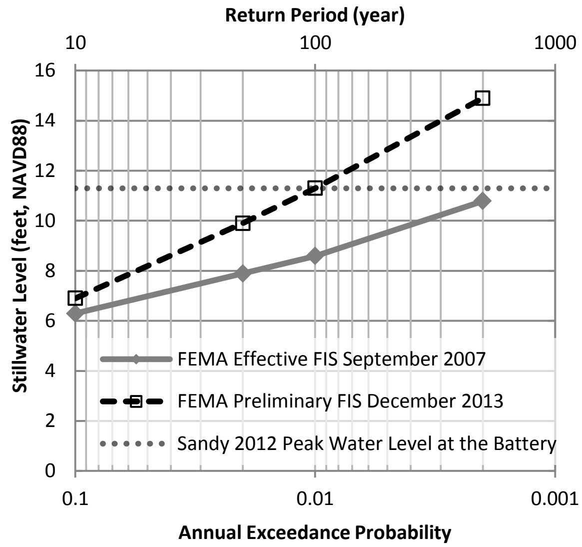

Figure 3: Comparison of Stillwater Elevations from Effective and Preliminary FEMA FISs

In 2014, the American Society of Civil Engineers (ASCE) published “Flood Resistant Design and Construction” (ASCE 24-14), which is a referenced standard in the 2015 International Building Code® (IBC). Building and structures within the scope of the IBC to be constructed in flood hazard areas must be designed in accordance with ASCE 24-14. Cities/towns around New York and New Jersey also issued revised local municipal codes with resilient building design requirements after Hurricane Sandy. These resilient design guidance documents generally adopt a new, higher design flood elevation (DFE) for both new construction and improvement programs for existing structures. The DFE is often defined as the 1-percent flood level (base flood elevation, i.e., BFE) plus a specified freeboard:

$$DFE = BFE + Freeboard$$

BFE is defined as a flood level, including wave effects, that has a 1-percent chance of being equaled or exceeded in any given year. The freeboard value, which is normally 1 or 2 feet, varies depending on the flood design class for a given building (ASCE, 2014).

A sample hydrostatic pressure profile, including lateral pressure and uplift, is shown in Figure 4, per FEMA and ASCE’s design requirements (ASCE, 2010; FEMA, 2014). Figure 4 applies to buildings where no water inflow is allowed for the interior space. For existing structures, it is often a challenge to meet the requirements imposed by a raised DFE. When an updated FIRM increases the 1% annual exceedance probability (AEP) flood elevation (i.e., BFE), it expands the boundary of the SFHA in certain areas. Some structures that were previously outside FEMA's Special Flood Hazard Area (SFHA) will be newly located within the expanded SFHA and subject to the NFIP regulations. During the post-Sandy recovery and rebuilding process, architects and engineers realized that structural design for full hydrostatic pressure under the new DFE significantly exceeded the allowable structural capacity of existing foundation walls and basement slabs. Other issues such as erosion of in-situ soils around foundations and interior flooding also became critical. Substantial structural improvement options are often needed to bring these existing buildings into compliance with the regulatory guidance and make them capable of withstanding the new, raised DFE. These improvements are not only costly but can also be infeasible due to constructability challenges. Architects and engineers try to look for innovative solutions that can help these existing buildings improve resilience.

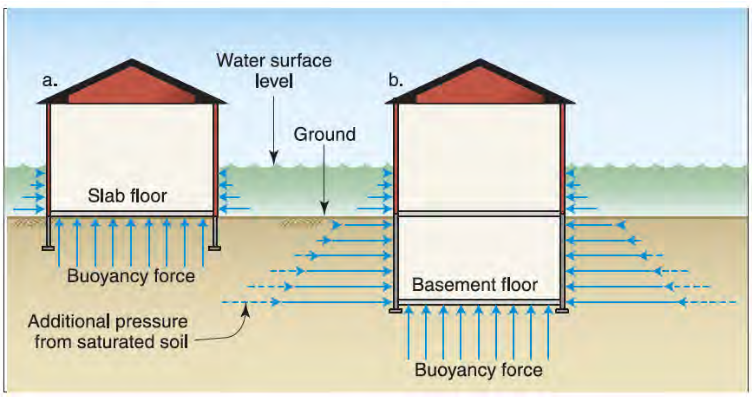

Figure 2-6. Hydrostatic pressure acts on walls and concrete slab floors. The weight of saturated soils adds to the pressure on basement walls. Figure 2-6a shows a home with a concrete slab floor. Figure 2-6b shows a home with a basement.

Figure 4: Hydrostatic Pressure for Structures - Reproduced from Figure 2-6 of FEMA P-312

II CASE STUDY - INPUT

The case study presented in this paper is an analysis performed for a public housing site as part of the post-Sandy capital improvement programs around the New York metro area. The required DFE value imposes a large hydrostatic force on both the vertical foundation elements and ground-level slabs, assuming hydrostatic conditions under the hypothetical design flood. Based on preliminary structural analysis results, the buildings needed significant structural improvement to be able to withstand the assumed full hydrostatic force to comply with the design standards. The two key objectives of the study are (a) to calculate more realistic water pressures on the building structural elements using a transient seepage analysis approach; and (b) to demonstrate and compare effectiveness of various improvement options for lowering the hydrostatic pressure using the transient method.

II.1 Site Location

The site is located in the East Village along the East River, a tidal strait between the New York Harbor and the Long Island Sound. This paper generalizes the analyses performed for a number of different sites in the same neighborhood. Figure 5 shows the approximate site location. The public housing complex consists of multiple high rise buildings that were originally designed and constructed in the 1940s. The general site grade is at a typical elevation of 9 feet, NAVD88. The site was flooded during Hurricane Sandy, due to the high storm surge, with up to approximately 2 feet of standing water on the ground. A number of buildings were flooded in the interior space, especially at the subgrade level. The interior flooding was caused by flood water entering from various openings such as windows and doors.

Figure 5: Case Study Site Location

II.2 Design Flood Elevations

The project site is currently mapped by the latest FEMA FIRM as SFHA with flood zone designations of AE12, which indicate a base flood elevation of 12 feet, NAVD88. Based on ASCE 24-14, these apartment buildings belong to Class 3 for flood designs. Class 3 is defined as "Buildings and structures that pose a high risk to the public or significant disruption to the community …" (ASCE, 2014). An additional one-foot of freeboard (beyond the FEMA BFE) is required by ASCE 24-14 for flood resistant design. In addition, to be eligible for resiliency grant funding opportunities, a projected sea level rise (SLR) needs to be included. For this project, the Sea Level Change Curve Calculator, an online tool developed and hosted by the U.S. Army Corps of Engineers (USACE) was used to estimate the projected sea level change for the assumed design life of the site (USACE, 2016).

Figure 6 depicts the relative sea level changes at the Battery from 2016 through 2100 by the Calculator. The "USACE Low" scenario represents the sea level trend based on historical observed data. The Calculator predicts a potential sea level rise of up to nearly 5 feet by the end of the year 2100 at this location, under the high scenario. The design team selected the intermediate scenario and used a total sea level rise of 1.75 feet for the resiliency design of the property (solid line in

Figure 6). Therefore, the design flood elevation was determined to be:

DFE = BFE + Freeboard + SLR = 12' NAVD88 + 1' + 1.75' = 14.75' NAVD88

This transient analysis used a stage hydrograph instead of a constant water level to better represent the hypothetical flood being analyzed. A stage hydrograph is a time series of flood elevations. The hydrograph used for this study consisted of a rising segment for 5 hours, followed by a peak duration of 2 hours at the DFE and a descending segment for 5 hours. The total simulation period was 12 hours. The Sandy observed water levels were shown as a comparison in Figure 7. The hydrograph adopted was considered to be conservative and representative of a severe tropical cyclone for the New York City metro area. Based on the design hydrograph, it was estimated that the site would be flooded for a duration of approximately 6 hours (Figure 7).

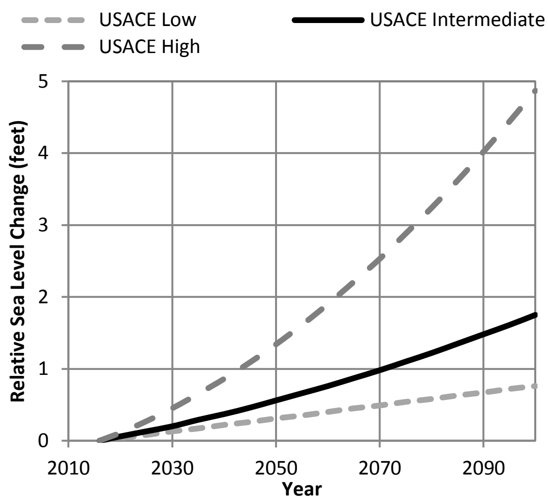

Figure 6: Relative Sea Level Change Projection (2016 to 2000) at The Battery, New York

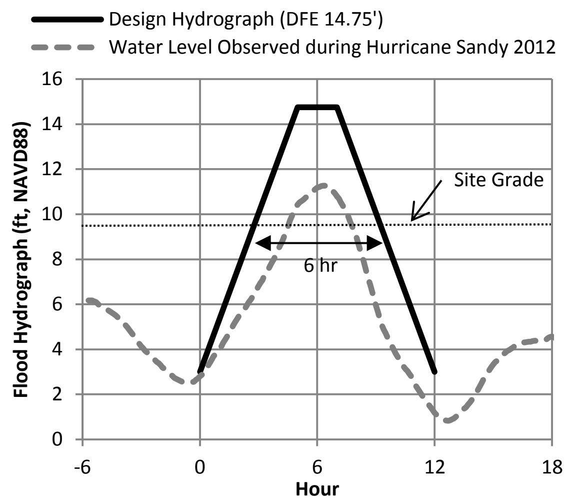

Figure 7: Input Flood Stage Hydrograph

II.3 Critical Design Section

A number of cross sections through the existing buildings were considered and one critical location was selected for the groundwater seepage modeling. A schematic of the analysis cross section is shown in Figure 8 (not to scale). Note that two "conceptual" improvement options are sketched in the same figure. One of the concepts is a horizontal hardscape at the ground surface and the other is a vertical seepage cutoff wall in front of the exterior building elements. Parameters used were generalized and are approximate, considered to be adequate due to the generic nature of this study.

II.4 Input Hydraulic Conductivity Values

Soil hydraulic conductivity values were estimated based on archived boring logs from the 1940s. The subsurface stratigraphy based on the boring logs at this site is mostly an approximately 20-foot deep fill layer overlying a thick sand deposit. Conservative hydraulic conductivity values were assumed for this conceptual study. Typical values for concrete (or equivalent impermeable barrier) and steel sheeting were used. The input geotechnical parameters are summarized in
Table 1.

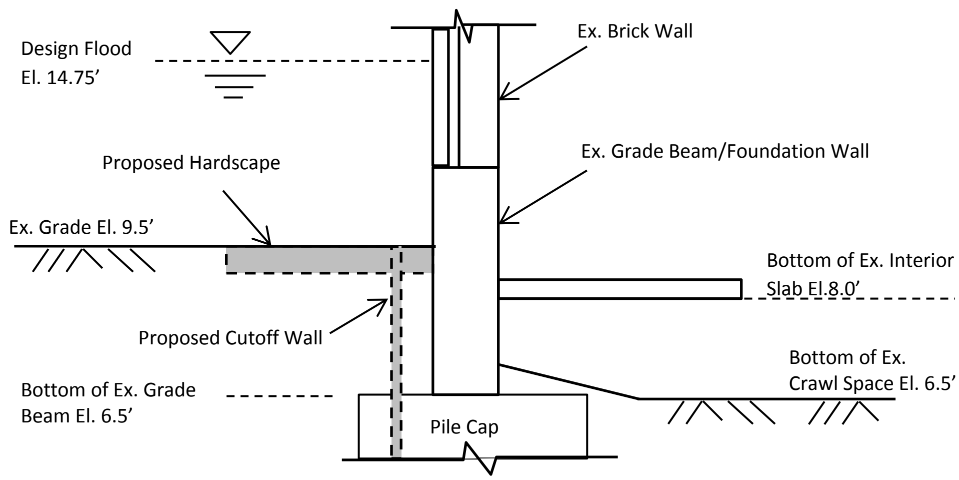

Figure 8: Selected Critical Cross Section of Existing Buildings with Proposed Improvements

Material Name	Depth	Hydraulic Conductivity	
		(cm/sec)	(ft/sec)
Fill	0 to 20 feet	1.E-1	3.3E-03
Sand	20 feet and below	1.E-2	3.3E-04
Concrete	--	1.E-8	3.3E-10
Steel Sheeting	--	1.E-9	3.3E-11

Table 1: Generalized Soil Stratigraphy and Input Hydraulic Conductivity Values

III CASE STUDY – ANALYSIS AND RESULTS

The seepage analysis used a 2-dimensional seepage model, SEEP/W, developed by GeoStudio. Transient seepage modeling was performed using the stage hydrograph (Figure 7) as a prescribed head boundary on the ground surface (i.e., the exterior side of the modeled section). The initial conditions were assumed to be a steady-state condition with a prevailing groundwater at Elevation 3 feet. This groundwater table was assumed to be slightly higher than the observed water table in the area to account for future sea level change.

III.1 Simulated Scenarios with Conceptual Improvement Options

Four SEEP/W simulations were performed for this study, as described in Table 2 below. The horizontal hardscape (or any equivalent impervious barrier) option was primarily aimed to reduce ground surface infiltration around the building. Widths of horizontal hardscape were varied as a sensitivity parameter for Cases 2 and 3. Case 4 represents a scenario where a vertical cutoff wall (e.g., steel sheeting) is installed upstream (to the seepage flow) of the grade beam. The cutoff wall is expected to increase the flow length, reduce the hydraulic gradient and thus reduce the uplift pressure and exit gradient around the existing foundation.

The archived boring logs from the 1940s indicate that the prevailing groundwater table around the site varied approximately between 7 to 10 feet below the existing grade, which was equivalent to Elevation 2' to -1' NAVD88. The recorded groundwater levels seem to indicate that the groundwater table at the project site is tidally influenced and varies within the normal tidal range (Elevation 2.3' to -2.8', NAVD88). A typical SEEP/W model is shown in in Figure 9. During the steady-state phase, both sides of the vertical boundaries were assigned with the fixed groundwater table at Elevation 3 feet, which provided the initial conditions for the subsequent transient analysis (Figure 9(a)). This value is slightly more conservative than the Mean Higher-High Water (MHHW) at the Battery. During the transient phase, the exterior side of the vertical boundary was released and a flood boundary was assigned at the ground surface using a stage hydrograph shown in Figure 9(b). Because there is a crawl space below the ground floor, factor of safety against piping (FOS_{piping}) near the exterior wall was also calculated, by assigning a no flux boundary with the "potential seepage face review" option along the soil surface of the crawl space. FOS_{piping} can be estimated using the following equation:

$$FOS_{piping} = i_{critical} \ / \ i_{exit}$$

where

i_{exit} = exit gradient (to be determined by SEEP/W);
$i_{critical} = \gamma' \ / \ \gamma_w$, critical gradient that can initiates piping failure;
γ' = effective unit weight of soil;
γ_w = unit weight of water;

In this study, assuming a saturated unit weight of 120 pounds per cubic feet (pcf) and a typical unit weight of 64 pcf for brackish water for the fill material, the critical gradient was determined as:

$$i_{critical} = \frac{120pcf - 64pcf}{64pcf} = 0.875$$

Case ID	Descriptions
1	Existing conditions (no improvement; permeable ground)
2	6 feet wide hardscape
3	12 feet wide hardscape
4	Cutoff wall extending 6 feet below bottom of grade beam

Table 2: Conceptual Design Improvements for SEEP/W Simulations

III.2 Hydrostatic Conditions

Current FEMA and ASCE design standards recommend the use of hydrostatic pressure for flood resistant design. Because this study site is not located within Coastal A zones, dynamic pressure was not considered. The top of the hydrostatic pressure profile is the DFE of 14.75' NAVD88. Hydrostatic uplift pressure on the ground floor slab (at Elevation 8' NAVD88) can be calculated as (refer to Figure 4):

$$p = \Delta H \cdot \gamma_w = (14.75' - 8.0') \cdot 64\, pcf = 432\, pounds\ per\ square\ feet\ (psf)$$

The above calculated value of 432 psf significantly exceeded the maximum allowable pressure of 60 psf of the ground floor slab from the original design. Similarly, the lateral hydrostatic pressure at the bottom of the grade beam at Elevation 6.5' NAVD88 was calculated to be 528 psf. Substantial structural upgrade including a buttress against the exterior wall and a new structural slab were required to withstand the assumed hydrostatic condition.

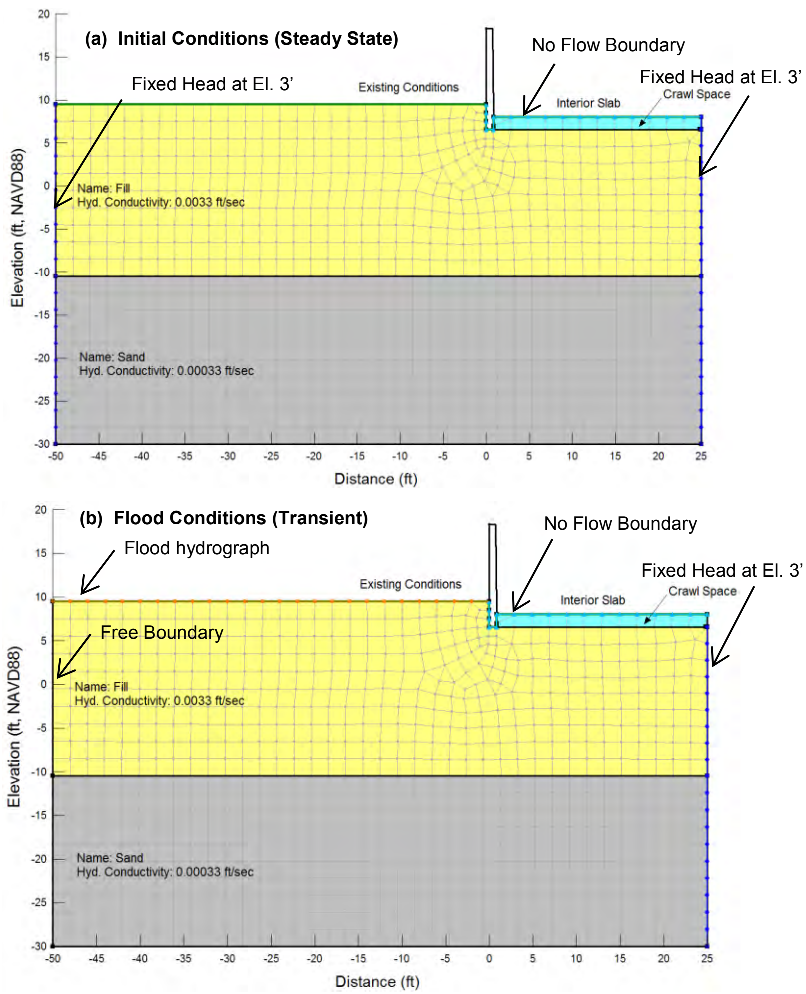

Figure 9: Typical SEEP/W Model - Finite Element Mesh (Existing Conditions)

III.3 SEEP/W Modeled Results

Table 3 summarizes the numerically simulated results for Cases 1 through 4. The maximum uplift pressure was extracted at the bottom of the slab and the minimum FOS_{piping} was calculated at the soil surface of the crawl space.

Figure 10 presents the temporal response of the uplift pressure against the bottom of the slab for Cases 1 through 4. The time series followed a similar bell-shaped curve as the input hydrograph. SEEP/W computed total head contour plots for Cases 3 and 4 are show in Figure 11 and Figure 12 as examples. The contour plots present the peak pressure and flow vectors at Hour 7 around and under the modeled building. The horizontal hardscape with a width around 12 feet is able to reduce the uplift (i.e., buoyancy) pressure to its maximum allowable value. The vertical cutoff wall is a relatively more efficient way to reduce the seepage flow in this transient analysis. By extending the cutoff 6 feet below the bottom of the grade beam, there is nearly no uplift on the first floor slab under the hypothetical flood.

Case ID	Descriptions	Maximum Pressure (psf) at Bottom of Grade Beam	Maximum Uplift (psf) at Bottom of Grade Beam	SEEP/W Calculated Exit Gradient	Minimum FOS_{piping} at Crawl Space
Hydrostatic	- -	528	432	- -	- -
1	Existing conditions	310	180	0.9	1.0
2	6-ft hardscape	200	120	0.6	1.4
3	12-ft hardscape	140	60	0.4	2.2
4	Cutoff 6-ft below grade beam	n/a	<10	0.2	>4

("- -" denotes not evaluated and "n/a" denotes not applicable)

Table 3: SEEP/W Simulated Results with Conceptual Design Improvements

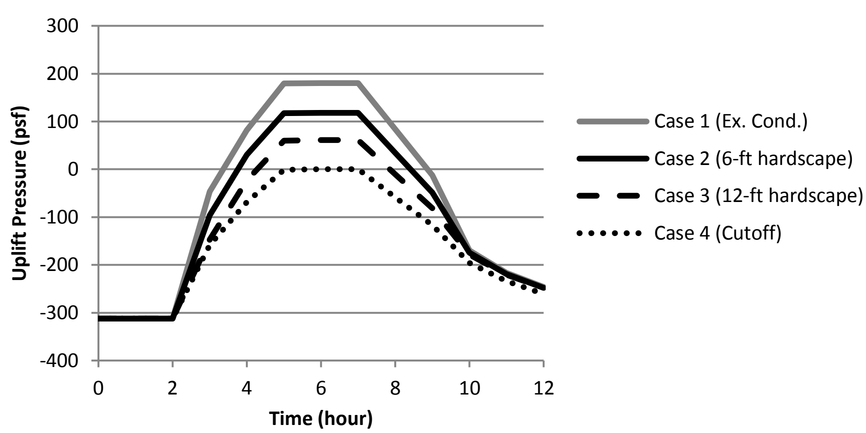

Figure 10: Time Series of Uplift Pressure against Bottom of Slab

<u>**III.4 Discussions**</u>

This analysis used a "half-space" model (i.e., only one side of the modeled cross section receives surface infiltration due to the incoming flood) and adopted a constant head boundary condition on the interior side during the flood event. This implies that the groundwater table around the site is assumed to remain unchanged during the relatively short-lived coastal flood (Figure 7). The authors believe this is a reasonable assumption for this 2-dimsional seepage analysis. Limited sensitivity test results indicate that when a “full-space” model is used and the constant head boundary was placed farther away from the building, there is understandably a slight increase of the computed lateral pressure and uplift. For this analysis, because of the generic nature of the study and adoption of conservative soil hydraulic conductivity values, the simulation results are still considered reasonably conservative.

The numerical analysis results indicate that the vertical cutoff wall (potentially using steel sheeting) appears to be an effective way to reduce seepage and uplift. However, this option faces serious constructability challenges in this tight, urban setting with numerous existing underground utilities around the site. The horizontal hardscape concept is able to tie some existing impervious ground surface features such as sidewalks with additional improvement to effectively slow down the infiltration process and reduce seepage pressure around the foundation and under the ground floor slab. This concept was later adopted by the design team and incorporated for resiliency design/improvement grant applications.

Key assumptions of this study include:

- The analysis adopted a steady-state, horizontal groundwater table at Elevation 3’, NAVD88, at the site as the initial conditions when the storm surge flood occurs. This assumed water level is higher than the current MHHW at the Battery. However, the actual groundwater table can be higher if there is a significant precipitation event, which could potentially raise the groundwater level prior to the design flood.

- The analysis used a constant head boundary for the transient analysis on the interior side of the 2D model. The authors assumed that the hypothetical flood was relatively short-lived such that the groundwater table during the transient phase did not rise in response to the coastal flooding.

- The analysis used a stage hydrograph with a peak elevation lasting for 2 hours. This is more typical of a tropical cyclone-induced storm surge event around New York City. Flood hydrographs associated with Nor’easters assume a wider, bell-shaped curve. The computed results presented in this paper could potentially increase if the flooding lasts longer.

V CONCLUSIONS

This paper presents an example transient numerical seepage analysis for public housing complexes in Manhattan, New York, subject to coastal storm surge-related flooding. The seepage flow was induced by surface infiltration during a hypothetical design flood. A stage hydrograph was used to model the flood. The analysis methodology is also applicable to sites that are subject to short-duration riverine flooding.

The results indicate that transient, numerical analyses allow the use of reduced hydrostatic pressure under a building for structural design purposes, when the full, hydrostatic pressure significantly exceeds the allowable capacity of an existing structure. Numerical simulations also allow evaluations of effectiveness of different improvement concepts. The analysis results successfully aided the owner and design team in the development of conceptual resiliency design packages as part of FEMA funding applications.

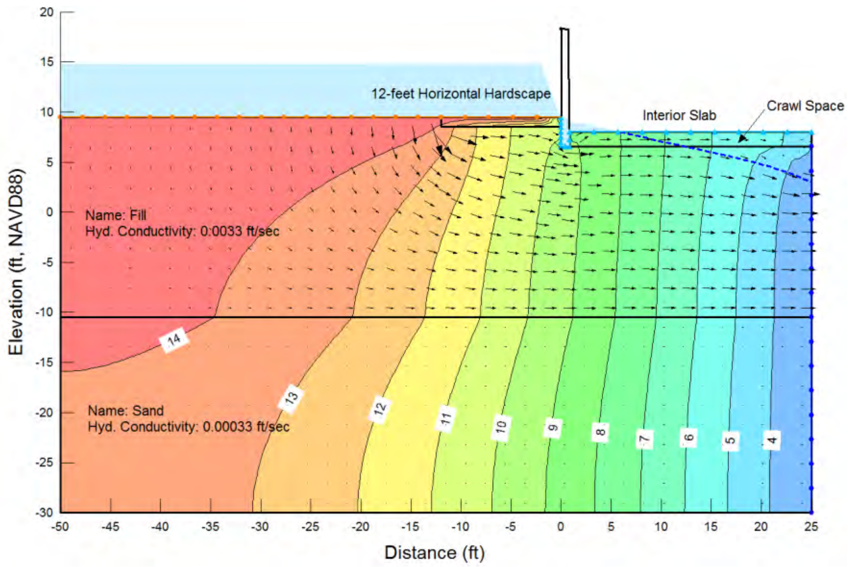

Figure 11: Total Head Contours at Hour 7 – Case 3

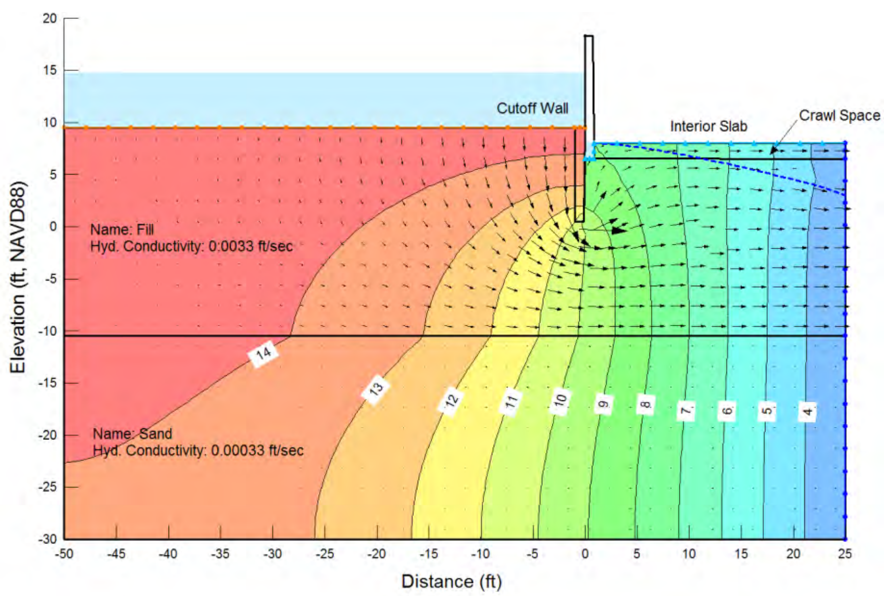

Figure 12: Total Head Contours at Hour 7 – Case 4

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