ABSTRACT

The Eemskanaal levee, part of the Dutch regional flood defenses located in Groningen, is planned to be upgraded in order to meet the requirements of the latest flood protection standards. Due to gas-extraction-induced earthquakes in the area, the need for additional measures to mitigate earthquake-related demands on the levee and the retaining structures was to be evaluated. This paper presents the dynamic numerical analyses performed to evaluate the dynamic levee performance. The objective of the analyses was to provide reliable estimates of earthquake-induced levee deformations and demands on the retaining structures for the design shaking levels. Nonlinear constitutive models were used to model the soil behavior including the strength degradation of the soft fine-grained deposits underlying the levee under large shear strains. Soil-structure interaction phenomena were explicitly modeled through the simulation of the retaining structures in the 2D models. Analyses results indicated that a failure surface develops within the soft fine-grained Holocene deposits leading to soil movement towards the landside. However, the estimated levee crest settlements are lower than the maximum allowable crest settlement to prevent flooding and the maximum bending moments that develop on the sheet pile walls remain below the moment capacity of the walls.

Keywords: dynamic levee performance; induced seismicity; strain softening; numerical analyses

1. INTRODUCTION

Gas-extraction-induced earthquakes in the Groningen area have led to increased seismic demands in an area of low tectonic activity creating the need for dynamic performance assessment of existing structures that have not been designed to undertake earthquake loads. The Eemskanaal levee located between the cities of Groningen and Delfzijl (Figure 1) forms part of the so-called regional flood defenses and contains retaining structures such as freestanding or anchored sheet pile walls. As part of the levee reconstruction program to meet the more recent requirements against flooding, earthquake effects needed to also be considered.

Soft fine-grained Holocene deposits underlie most of the Eemskanaal levee and the cyclic softening potential of these deposits under earthquake loading has raised the issue of dynamic levee instability. Since the levees form part of the regional flood defenses, the levee crest settlement under earthquake loading was a key factor in assessing whether additional measures were required to mitigate earthquake-induced deformations. Existing simplified techniques widely used by practitioners in assessing dynamic levee stability and displacements (Newmark 1965, Sarma 1975, Makdisi and Seed

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1978, Lee 1974, Serff et al. 1976, Seed et al. 1973, Seed 1979, Swaisgood 2003, Bray and Travasarou 2007) do not account for the presence of retaining structures nor strength degradation of the soils underlying the levee and therefore could not be used to provide reliable estimates of levee settlements or seismic demands on the retaining structures. To overcome the limitations of existing simplified methods and provide a robust and reliable answer to the question of whether additional measures were required to mitigate earthquake induced deformations dynamic numerical analyses were performed at selected locations to evaluate dynamic levee performance. The objective of the analyses was to provide reliable estimates of earthquake-induced levee deformations and demands on the retaining structures for the design shaking levels.

Figure 1. The Province of Groningen and the Eemskanaal between the city of Groningen and Delfzijl.

2. SITE CONDITIONS

This paper focuses on the numerical evaluation of two typical cross sections of the Eemskanaal levee containing: A) a freestanding sheet pile wall (Figure 2) and B) an anchored sheet pile wall (Figure 3), respectively. The site geology of the Eemskanaal area includes Holocene tidal deposits (Naaldwijk Formation), consisting of clay and peat, extending from the surface down to an elevation varying from -5 to -10 m N.A.P (Normal Amsterdam Level, reference plane for height in the Netherlands), overlying Pleistocene dense sands (Boxtel Formation) and hard clays (Drente and Peelo Formations). As shown by CPT soundings, field and laboratory data, the Holocene deposits are characterized by low undrained shear strength (Figures 2 and 3); significantly lower than the older Pleistocene formations.

Strength parameters were obtained by interpretation of available CPT and laboratory data. Undrained shear strength was calculated from CPT tip resistance using a cone bearing resistance factor, $N_{tk}$, equal to: i) 17 for Holocene deposits and ii) 15 for Pleistocene fine-grained layers. It is worth noticing that the peat layers encountered in this area, called “veen” in Dutch, are stiffer than the soft Holocene clays, perhaps contrary to what would be “expected” for typical peat material. Sensitivity is defined as the ratio between the peak static undrained shear strength over the residual strength and it is used to describe quantitatively the softening behavior of fine-grained materials. Clay sensitivity was estimated from UU tests on undisturbed and remolded samples obtaining values ranging from 2 to 6.

Shear wave velocity was estimated as the average of published correlations with CPT data. For fine-grained layers the average of the predictions using Mayne and Rix 1995, Andrus et al. 2007, and Robertson 2009 was used, while for sands the average of the predictions using Rix and Stokoe 1991, Andrus et al. 2007, and Robertson 2009 was used.
Figure 2. Geometry and stratigraphy of cross section A containing a freestanding sheet pile wall (top). Distribution of shear wave velocity, $V_s$ and undrained shear strength versus elevation (bottom).

Figure 3. Geometry and stratigraphy of cross section B containing a freestanding sheet pile wall (top). Distribution of shear wave velocity, $V_s$ and undrained shear strength versus elevation (bottom).

The idealized shear wave velocity, $V_s$, and undrained shear strength, $S_u$, profiles used for the numerical analyses of cross sections A and B are depicted in Figures 2 and 3, respectively, together
with interpreted values from CPT data. Both cross sections include layers within the Holocene deposits with extremely low Su values, less than 10 kPa (i.e. elevations -2 to -4 m NAP in cross section A and elevations -3 to -8 m NAP in cross section B), being prone to potential development of large shear deformations under seismic loading.

3. MODELING APPROACH

3.1 Overview

Two dimensional dynamic analyses were performed using the finite difference code FLAC (Itasca, 2011). FLAC2D is a two-dimensional explicit finite difference program that has the ability to model structures (including sheet pile walls and anchors) and capture soil-structure interaction related phenomena.

Centrifuge experiments were used to confirm that the numerical modelling procedure applied to the current project is able to capture the fundamental behaviors associated with the problem being evaluated, including non-linear soil response, dynamic soil-structure interaction and associated failure mechanisms/deformations and forces.

The FLAC2D analyses were intended to realistically model the time-dependent, nonlinear hysteretic behavior of fine-grained materials that were identified during site characterization. The goal of these analyses was to provide estimates of levee displacements and structural demands at relatively vulnerable sections under the design event. The finite difference models were first brought to equilibrium under gravity loading. The earthquake time histories were then introduced at the base of the models. Free-field boundary conditions were applied to the two lateral sides of the model (Itasca, 2011).

3.2 Soil Constitutive Models and Model Calibration

Fine-grained soil deposits were modeled with the Mohr-Coulomb failure criterion in combination with UBCHYST hysteretic model (Naesgaard and Byrne, 2011). The model is able to describe the reduction of secant modulus with shear strain and the accumulation of permanent shear strain resulting from ratcheting when the soil is loaded with a static shear bias. The UBCHYST model has been used by several researchers for seismic analyses of retaining structures (e.g. Candia, 2013; Mikola and Sitar, 2013; Wagner and Sitar, 2016). In the absence of site-specific strain-controlled cyclic tests, the model parameters were fit to approximate target shear modulus reduction curves from Darendeli (2001). For clays, the peak undrained shear strength was increased by 20% due to dynamic loading conditions. In addition, strain softening was assigned to soft Holocene soft deposits assuming a sensitivity value of 5. The envelope undrained shear strength for Holocene clays including peak strength and strain softening is depicted on Figure 4. Figure 5 illustrates characteristic monotonic shear stress-strain curves and loops obtained from cyclic loading under constant shear strain amplitude and earthquake loading with initial static bias. The modelled unloading-reloading behavior for non-liquefiable layers follows the Masing rule.

![Figure 4. Envelope of undrained shear strength assigned to Holocene deposits including the dynamic peak](image-url)
strength and the softening at large deformations.

Figure 5. Calibration procedure and characteristic monotonic shear stress-strain curves and loops obtained from cyclic loading under constant shear strain amplitude and earthquake loading with initial static bias, using UBCHYST model (Naesgaard and Byrne, 2011).

The dense Pleistocene coarse-grained soil deposits (i.e. Boxtel sand) were modeled with the PM4Sand (Boulanger and Ziotopoulou, 2013; Ziotopoulou and Boulanger, 2013) constitutive model to simulate potential pore pressure development within these layers where in some cases the retaining structures were founded (even if liquefaction is not triggered under the design seismic demand). A relative density of 64 to 70% was used for the Boxtel sands based on interpreted CPT data.

3.3 Modeling of Retaining Structures

Sheet Pile Walls. The sheet pile walls were modeled using beam elements which are defined by their material and geometric properties (Itasca, 2011). The beam elements were assumed to behave as linearly elastic and the calculated maximum bending moments and shear forces were checked against the elastic moment capacity and shear strength of the section used. Stiffness was defined by the elastic (Young’s) modulus E, and the geometric properties (i.e. length L, cross-section area A, moment of inertia I). The soil-beam interaction was modelled through interface elements that describe the interaction in both normal and tangential directions. In the tangential direction a Coulomb-type frictional behavior (slider) was used assuming a capacity associated with 2/3 of the soil friction angle. In the normal direction, where soil and structure are in contact, there is full transmission of the forces. Separation between soil and structure is allowed in which case no force is transmitted from one medium to the other.

Anchors. The ungrouted part of the anchors was modelled using rockbolt elements (Itasca, 2011). These elements interact with the surrounding soil through springs (similar to pile elements). They can therefore deform due to soil displacement/settlement. A spacing parameter was defined so that the rockbolt is not considered to be continuous in the out-of-plane direction. The element has both axial and bending stiffness and the failure is defined in terms of a tensile failure strain limit. The total plastic tensile strain derives from the axial plastic strain and the bending strain. If the total plastic strain exceeds the limit, the forces and the bending moments in the rockbolt are set to zero and the
rockbolt is assumed to have failed. The grouted part of the anchor was modeled with pile elements. The nonlinear behavior of the interaction between the grout body and the surrounding soil is described by an elastic-perfectly plastic law defined by a maximum grout shear strength and a stiffness.

3.4 Validation

Candia and Sitar (2013) conducted a series of experiments at the centrifuge facilities of UC Davis in order to investigate the dynamic response of rigid and flexible retaining walls with cohesive backfills. Figure 6 illustrates the 2D numerical model developed in prototype scale using the finite difference code FLAC2D Version 7.0 (Itasca 2011). The base of the numerical model was excited with a simulation of a ground motion recorded in an earthquake (KocaliYPT060-1 motion). UBCYST model was used for modeling the soil behavior. The comparison between numerical and experimental results is focused on the response of the flexible cantilever wall, considered more relevant to the flexible sheet pile walls in Eeskeal levee, and the free-field behaviour of the cohesive backfill. Figure 7 depicts the comparison between experimentally and numerically obtained time histories of the acceleration in the free-field at the surface (location A23) and the bending moments developed on the cantilever wall. In general, the numerical analyses appear to reasonably reproduce the experimental behaviour.

Figure 6. Numerical simulation of the centrifuge experiment by Candia and Sitar, 2013 with the cantilever wall and the cohesive backfill soil (top). The input motion applied at the base of the model (bottom).

Figure 7. Comparison between experimental and numerical results in terms of time histories of: i) the acceleration in the free-field at the location A23 in Figure 6 (top) and the bending moments developed on the cantilever wall (bottom).
4. NUMERICAL EVALUATIONS OF DYNAMIC LEVEE PERFORMANCE

4.1 Model Geometry

Numerical analyses results are presented for two cross sections (A and B) as discussed above. The finite difference meshes of the two numerical models are shown on Figure 8. Cross section A (upper illustration) includes soft Holocene clay and “veen” layers underling the levee down to about El. -5 m NAP. Boxtel sand, Drente and Peelo clays underlie the Holocene deposits. The base of the model is at El. 22 m NAP. The levee crest elevation is at El. +1.8 m NAP. The capacity of freestanding sheet pile canal wall (type L603) is 135 kNm/m accounting for corrosion effects and its base is at El. -10.5 m NAP within the Peelo formation.

![Cross section A and Cross section B](image.png)

Figure 8. Numerical models of cross sections: A with freestanding sheet pile wall (top) and B with anchored sheet pile wall (bottom).

In Cross section B (lower illustration), the levee crest elevation is at El. +2 m NAP. Soft Holocene clay and “veen” layers underlie the levee up to about El. -8 m NAP. Pleistocene Boxtel sand and Peelo clays underlie the Holocene deposits. The base of the model is at El. 26 m NAP. The anchored sheet pile wall (type AZ12-700) base is at El. -6 m NAP within the Holocene deposits. The moment capacity of the canal wall, considering corrosion effects, is 154 kNm/m. The angle of the anchor forms a 30-degrees angle to the horizontal. The grout body starts at El. -9 m NAP and ends at El. -11.0 m NAP (length = 4 m), with a diameter of 0.2 m and a perimeter equal to 0.63 m. The yield capacity of the anchor (type R32-360) is 172 kN considering corrosion effects. A spacing equal to 4.2 m has been assumed.

4.2 Development of Input Time Histories

The v4 Ground Motion Model (Bommer et al., 2017) was used to define seismic ground motion demands for the evaluation of the levee. A probabilistic assessment of seismic performance of the Eemskanaal levee was undertaken using First Order Reliability Method and numerical analyses results from representative levee sections (Jongejan et al 2017). Design criteria including return periods, and partial material factors for the project were derived as part of this study. The design return periods varied between 400 and 1100 years along the Eemskanaal, depending on the required reliability of the section and the location relative to the earthquake sources.

Input time histories were required for the 2D numerical analyses at El. -25 m NAP, however the seismic demand from the v4 GMM is defined at the ground surface. In order to develop input time histories for the 2D numerical evaluations site response analyses were performed to propagate the time histories from bedrock level (i.e. base of North Sea formation (NS_B) at depth of about 800 m) to the surface.
As a first step acceleration time histories were selected and spectrally matched to the bedrock response spectra per v4 GMM (Bommer et al., 2017) for a range of return periods. The normalized bedrock design spectra for all selected locations and return periods practically have the same shape allowing for the development of a unique normalized target spectrum. Eleven recorded ground motion time histories were selected and spectrally matched to the normalized bedrock design spectrum. The selection process of recorded ground motions was based on site-specific parameters such as magnitude, site-to-source distance, frequency content (i.e., spectral shape), peak intensity measures, and ground motion duration ($D_{5.75}$). Fault rupture mechanism, and site class were also taken into consideration in the selection process. Ground motion modification was performed in the time domain by introducing wavelets in the seed time history to adjust both its amplitude and frequency content. The time-domain spectral matching was accomplished using the computer code RSPMATCH written by Abrahamson (2003), which generally follows the algorithm set forth by Lilhanand and Tseng (1988). An example of spectral matching using the TCU127 record form an aftershock of the 1999 Chi-Chi earthquake in Taiwan, is shown in Figure 9.

Equivalent linear site response analyses were performed using the computer code STRATA (Kottke et al., 2013) to propagate the spectrally matched horizontal ground motions from 800 m depth up to the surface using a frequency-domain solution. Ground motions were extracted at an elevation of about -25 m NAP within the Peelo formation as “within” motions. These motions, plotted in Figure 10 for cross sections A and B, were then applied to the rigid base of the 2D finite difference model (at El. -25m NAP).

Figure 9. Spectral matching of a TCU-137 motion from an aftershock of 1999 Chi-Chi Earthquake in Taiwan to the bedrock design spectrum.
4.3 Results

Representative results from the two dimensional dynamic analyses for both cross sections are shown on Figures 11-14. Figure 11 presents total displacement vectors of the soil and the structural elements (a,b) and shear strains within the soil (c,d) at the end of shaking, for both cross sections. As shown on this figure, soil movement occurs mainly toward the landside (where no retaining structure is present) and the displacement vectors are limited within the Holocene soft deposits. A shear zone develops starting behind the upper part of the canal wall, passing though the Holocene layers and emerging close to the stream at the landside. The latter is more evident in case of cross section A (Figure 11c) where the Holocene deposits are shallower.

For cross section B, where the Holocene deposits extend deeper, the main shear zone extends landward, while a secondary shear zone also forms towards the canal-side passing below the canal wall through the same weak Holocene. The soil movement towards the canal-side in this cross section is related to the relatively low wall embedment and its foundation within the soft Holocene deposits. As expected in both cases the pore pressure development within the Pleistocene Boxtel sands is limited corresponding to values of excess pore pressure ratio, Ru, below 30% under the design event.

The soil deformation results in levee crest settlements varying between 6 and 8 cm and horizontal landward displacements on the order of 10 cm (Figure 12). It should be mentioned that the lowest
allowable elevation against flooding is +1.3 m NAP (i.e., 0.5 m below the current levee elevation) based on Jongejan et al. (2017, 2018). The analyses showed that the levees are safe against flooding under the design event.

Demands on the structural elements were also checked against the project design criteria. Figure 13 depicts the maximum bending moment distribution on the canal walls for both cross sections during shaking. The bending moments developed on the canal walls remain low values during shaking, far below the yield capacity, due to the primarily landward soil movement (away from the wall) resulting in reduction in soil pressure on the wall. The developed shear forces were negligible compared to the shear strength of canal sheet pile walls.

Time histories of the axial force and axial strain that develop on the anchor are plotted on Figure 14. It is observed that the anchor axial force exceeds the yield capacity instantaneously, as indicated by the short “plateau” formed at about 5.2 sec (highlighted by the dashed circle on Figure 14). Due to the instantaneous occurrence of this exceedance, the axial strains that develop on the anchor remain much lower than the typical limit at breakage of 5% for this type of anchors, as shown on Figure 14. As the analysis results showed, the embankment soil during the earthquake moves towards the landside and away from the wall. This behavior in conjunction with the tendency of the wall tip to rotate towards the canal, result in the unloading of the wall and a corresponding reduction in the anchor force.
5. CONCLUSIONS

The Eemskanaal levees are planned to be upgraded meet the requirements of the latest flood protection standards. The performance of the new levee designs needed to also be checked against earthquake loads due to induced seismicity in order to evaluate whether additional measures are needed. Dynamic numerical analyses were performed to evaluate the performance of the Eemskanaal levee. Nonlinear constitutive models were used to model the soil behavior including strength degradation of the soft fine-grained deposits underlying the levee under large shear strains. Soil-structure interaction phenomena were explicitly modeled through the simulation of the retaining structures in 2D models. Analyses results indicate that a failure surface develops within the soft fine-grained Holocene deposits leading to soil movement towards the landside (i.e. where no retaining structure is present). Despite the observed soil movement, the estimated levee crest settlements are lower than the maximum allowable crest settlement to prevent flooding. Additionally, the maximum bending moments that develop on the sheet pile walls at the canal side remain below the moment capacity of the walls. The numerical analyses results provided a robust and reliable basis for the project stakeholders to finalize their earthquake design decisions.

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7. REFERENCES


