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Model Solutions for Performance-Based Seismic Analysis of an Anchored Sheet Pile Quay Wall

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***Abstract*—**Conventional seismic designs of quay walls in ports are mostly based on pseudo-static analysis. A more advanced alternative is the Performance-Based Design (PBD) method, which evaluates permanent deformations and amounts of (repairable) damage under seismic loading. The aim of this study is to investigate the suitability of this method for anchored sheet pile quay walls that were not purposely designed for seismic loads. A research methodology is developed in which pseudo-static, permanent-displacement and finite element analysis are employed, calibrated with an experimental reference case that considers a typical anchored sheet pile wall. A reduction factor that accounts for deformation behaviour is determined for pseudo-static analysis. A model to apply traditional permanent displacement analysis on anchored sheet pile walls is proposed. Dynamic analysis is successfully carried out. From the research it is concluded that PBD evaluation can effectively be used for seismic analysis and design of this type of structure.

***Keywords*—**Anchored sheet pile quay wall, simplified dynamic analysis, performance-based design, pseudo-static analysis

I. Introduction

PORTS are civil works which have a major societal and economic importance. Quay structures are infrastructural elements of primary significance for the functioning of a port system. The ability to economically design quay structures with sufficient seismic resistance is therefore of great importance in areas that are prone to earthquakes [1].

Conventional seismic designs are force-based i.e. that structures are designed to have sufficient capacity to withstand a pseudo-static seismic design force. This methodology is associated with no insight in the performance of the structure when exceeding the pseudo-static limit equilibrium state and uneconomic design due to the demand that the structure can resist a high seismic design force without deformation. A more advanced alternative is Performance-Based Design (PBD) methodology. In this methodology the key design parameters for the seismic performance of structures are stress states and deformations of soil and structure, rather than just a seismic design force. Furthermore, it recognizes that certain amounts of permanent deformations associated with different degrees of (repairable) damage are allowable [1], [2].

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Seismic analysis is required to evaluate the key design parameters for the seismic performance of soil and structure. The alternation of attenuating seismic waves by local soil deposits is determined with site-response analysis. For evaluation of the response of structures to seismic loading there are three levels of seismic analysis available, i.e. simplified, simplified dynamic and dynamic analysis. With simplified (e.g. pseudo-static) analysis the limit elastic force equilibrium and a threshold for displacement can be computed. In simplified dynamic (e.g. Newmark) analysis a failure mechanism is assumed from which the amount of displacement and stress/strain is determined. Dynamic (e.g. Finite Element) analysis can evaluate displacements, stress/strain and corresponding failure mechanism(s) without assumption on beforehand [2].

II. Problem Definition

The present study is embedded in the topic of performance-based seismic design of quay structures. Typical quay types are gravity-based quay walls, sheet pile quay walls and pile-deck structures. The observed trend in seismic quay design is that gravity and sheet pile structures (i.e. soil retaining walls) are associated with areas with zero to low seismicity while pile-deck structures are generally the preferred solution in areas with higher seismicity. Seismic load values up to which the quay structure types are typically applied in design are shown in Fig. 1. The typical seismic load values are expressed in terms of peak ground acceleration (PGA).

The observed trend can be explained by more favourable seismic performance (i.e. more deformation capacity) of pile-deck structures compared to retaining walls. In line with this trend it is found that PBD methodology is developed to significant lesser extent for retaining walls (especially anchored sheet pile walls) than for pile-deck structures. Guidelines for design of sheet pile quay walls are currently predominantly based on pseudo-static design [4]-[9].

Experience has shown that it is desirable to consider sheet pile quay walls in a more realistic and economic way in seismic analysis. In new seismic quay design anchored sheet pile walls are easily excluded for higher seismic demands although being efficient structures. In addition, it can possibly occur that unnecessary negative advice on the seismic safety of existing sheet pile walls (not purposely designed for seismic loads) is given. These are clear incentives for the present research to focus on investigating the applicability of PBD methodology on anchored sheet pile quay walls.

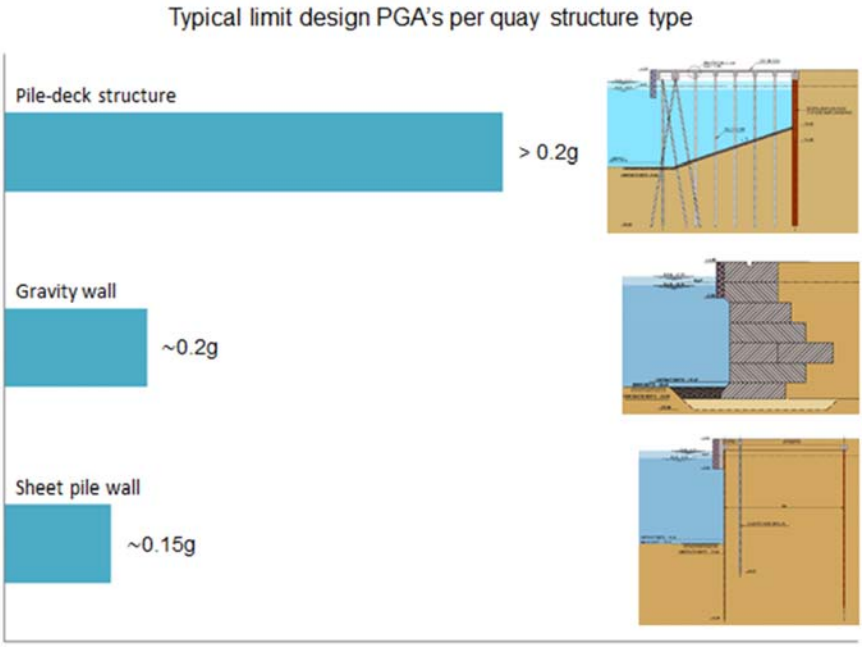

Fig. 1 Observed trend in seismic quay design [3]

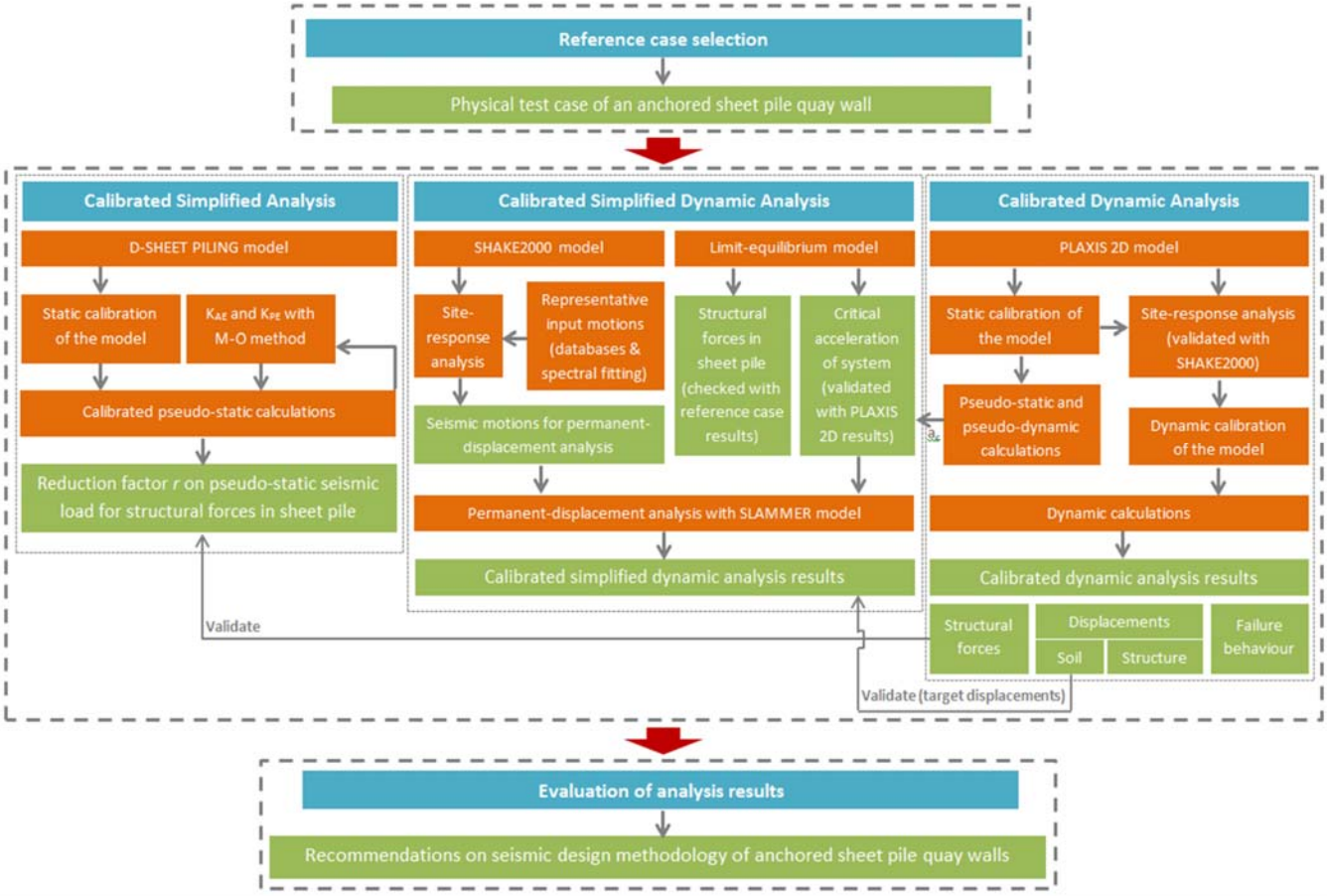

Fig. 2 Flowchart of the research methodology

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III. Research Description

For the present study a research methodology is developed. The flowchart of the research methodology is presented in Fig. 2. It basically consists of four steps.

Step 1: Selection of experimental reference case: A reference case is selected from literature to calibrate the seismic analysis with experiment measurements.

Step 2: Calibrated simplified analysis: The aim is to improve pseudo-static analysis by accounting for deformation behaviour during seismic loading.

Step 3: Calibrated simplified dynamic analysis: The goal is to investigate the applicability of permanent-displacement (Newmark) analysis on anchored sheet pile quay walls

Step 4: Calibrated dynamic analysis: This step is applied to validate findings from the simplified and simplified dynamic analysis models and to simulate reference case failure behaviour.

IV. Reference Case

The reference case is taken from [10]. This conference paper reports on sequential centrifugal shake-table experiments that are performed on a scale model (1:30) of an anchored sheet pile quay in the field. The layout of the test model is presented in Fig. 3. It shows the sheet pile wall with batter pile anchor, the field and (scale model) dimensions, the water level and the measurement devices. The soil in the model is homogeneous coarse silica sand (D_{50} = 1.2 mm) that is significantly compacted to a relative density of 80%. Due to this soil condition liquefaction is prevented. Because the testing is performed on a scale model (1:30), the entire experiment is carried out under a centrifugal gravity of 30g.

Fig. 4 presents the testing procedure that is followed in [10]. The procedure starts with a static experiment (CASE-000) in which the initial stress situation corresponding to deepening of the quay (from -7.5 m to -9.5 m below water level) is simulated. The subsequent seismic (shake-table) testing consists of successively introducing four seismic events to the scale model (CASE-100, CASE-200, CASE-300 and CASE-600). The input signals corresponding to the seismic events have maximum acceleration amplitudes of 0.1g, 0.2g, 0.3g and 0.6g respectively (field values).

In [10], an artificially created signal is used for CASE-100 and a signal recorded during a real earthquake for CASE-200 to CASE-600. The recorded signal, obtained at the test site of the quay wall under consideration, is scaled up to the 0.2g, 0.3g and 0.6g PGA values for the testing. For the present study the artificial signal could be reproduced, while for the recorded signal six representative records from earthquake databases were selected and processed. Fig. 5 shows an example of signals applied in the present research.

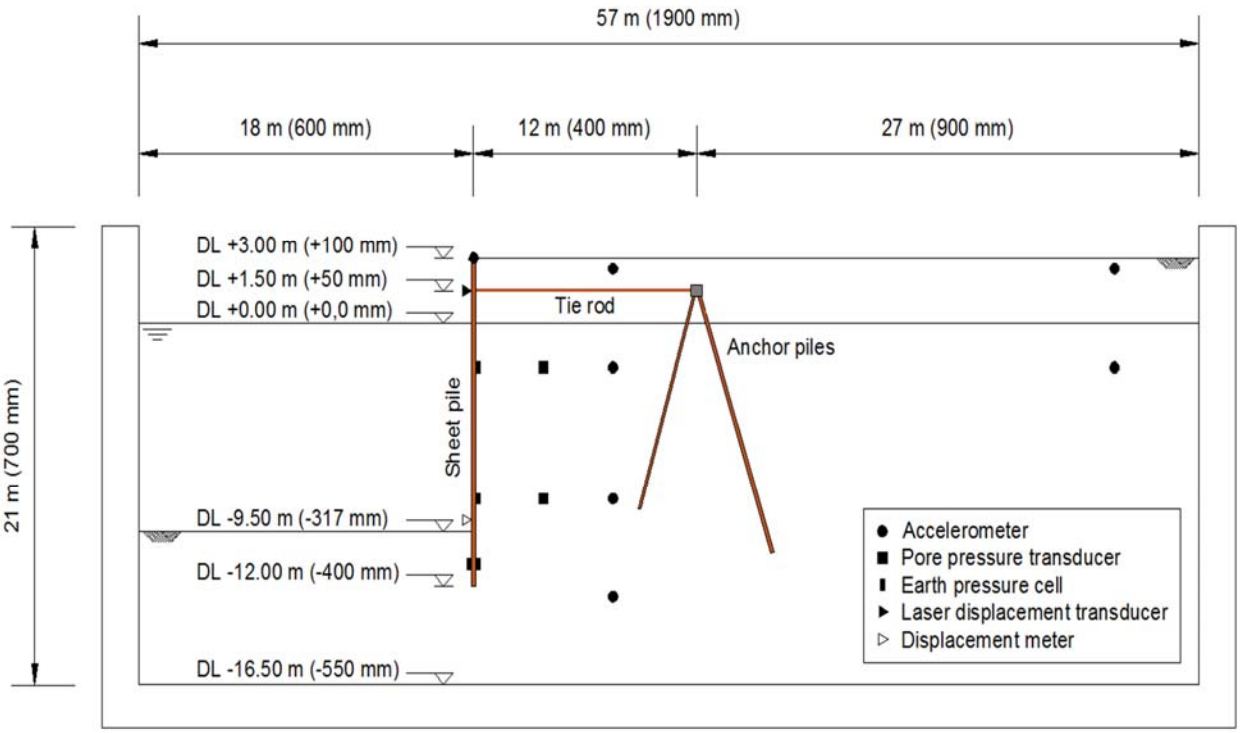

Fig. 3 Reference case test model for centrifugal shake-table experiments, adapted from [10]

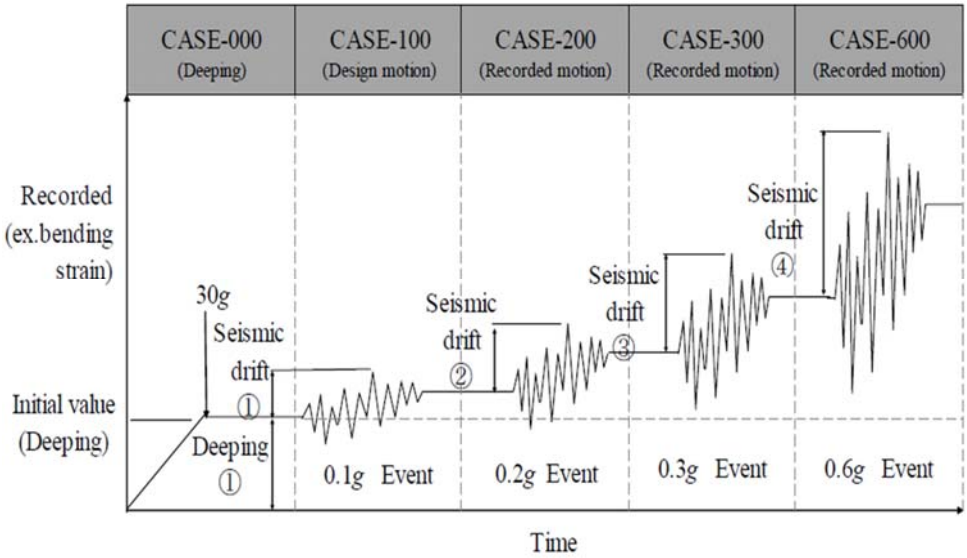

Fig. 4 Reference case testing procedure, as reported in [10]

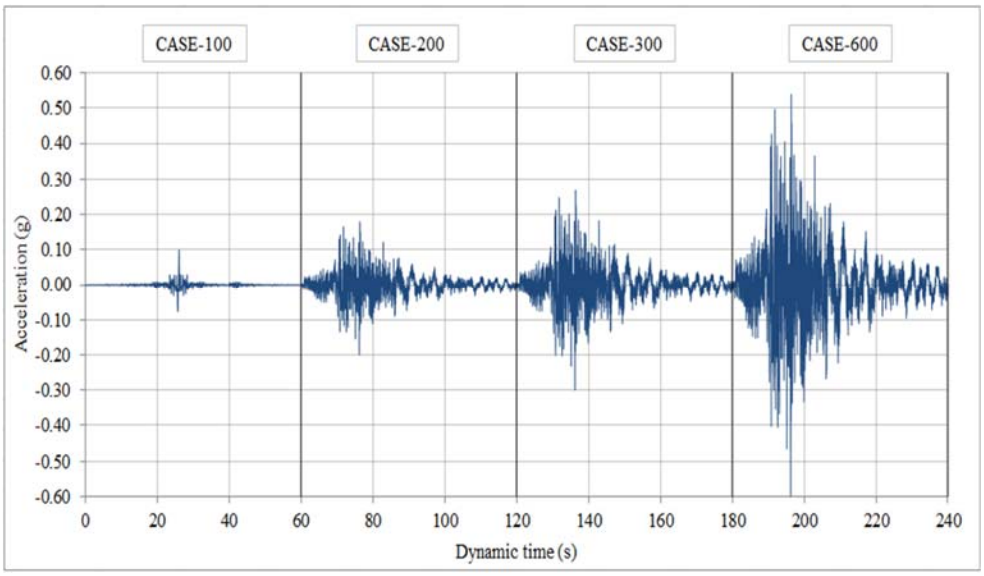

Fig. 5 Example of accelerograms applied in present research

The experiment results for the sequential load cases that are reported in [10] consist of bending moments in the sheet pile wall, tie rod forces and displacements at tie rod level and sea bed level. The results (field values) are presented in Table I.

TABLE I
REFERENCE CASE TEST RESULTS, ADAPTED FROM [10]

CASE	M_{max} (kNm)	F_{anchor} (kN)	u_{anchor} (mm)	u_{seabed} (mm)
000	207	108	14	6
100	301	182	20	9
200	547	261	66	41
300	449	262	93	64
600	728	308	232	195

From the results, it is noted that after CASE-300 the bending moments in the sheet pile structure have decreased and the displacements significantly increased with respect to CASE-200. This suggests passive soil failure in front of the quay wall after which the wall has moved. In the bending moment lines of CASE-300 and CASE-600 it can be seen that the point of contra-flexure (located at the seabed) has moved up approximately one meter, which again suggest passive wedge failure during CASE-300 at which the soil is pushed upwards. Considering the relatively small penetration depth of the sheet pile it is also likely that the failure mechanism with push-up of the passive soil wedge will occur. During CASE-600 passive soil resistance apparently redevelops and bending moments increase.

The bending moment measurements (field values) are shown in Fig. 6.

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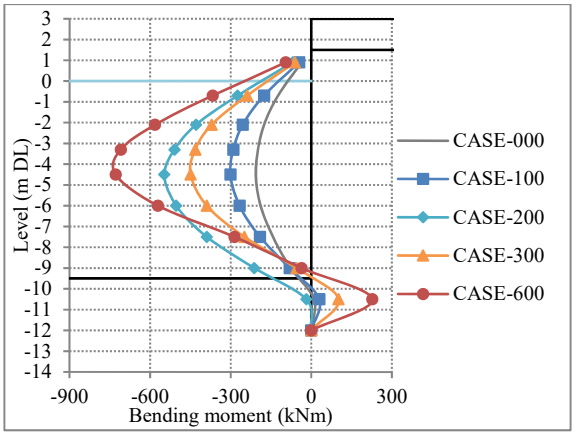

Fig. 6 Reference case bending moment results, adapted from [10]

V. Simplified Analysis

The objective of the simplified analysis, calibrated with the reference case findings, is to deduce a reduction factor on pseudo-static load that accounts for deformation behaviour of the anchored sheet pile wall. For this purpose, an elasto-plastic spring model compliant with the reference case geotechnical and structural setup is constructed, after which the model is calibrated with the static reference case bending moment measurements. The model is presented in Fig. 7.

In Fig. 7, the inclusion of pseudo-static loads in the elasto-plastic spring model is schematized. The pseudo-static input parameters for the model are the dynamic soil pressures coefficients (KAE and KPE) and the hydrodynamic pressure (pw) on the wall. These are calculated with the Mononobe-Okabe equations (1), (2) and Westergaard solution (3) respectively [8], [9]. For the full nomenclature reference is made to Appendix A.

$$K_{AE} = \frac{\cos^2(\varphi-\theta-\psi)}{\cos\psi\cos^2\theta\,\cos(\delta+\theta+\psi)\left[1+\sqrt{\frac{\sin(\delta+\varphi)\sin(\varphi-\beta-\psi)}{\cos(\delta+\theta+\psi)\cos(\beta-\theta)}}\right]^2} \quad (1)$$

$$K_{PE} = \frac{\cos^2(\varphi+\theta-\psi)}{\cos\psi\cos^2\theta\,\cos(\delta-\theta+\psi)\left[1-\sqrt{\frac{\sin(\delta+\varphi)\sin(\varphi+\beta-\psi)}{\cos(\delta-\theta+\psi)\cos(\beta-\theta)}}\right]^2} \quad (2)$$

$$p_w = \frac{7}{8} k_h \gamma_w \sqrt{z_w H_{wall}} \quad (3)$$

Seismic input in (1), (2), and (3) are the horizontal seismic coefficients k_h corresponding to the seismic shake events (0.1g, 0.2g, 0.3g and 0.6g). In (3), k_h can be entered directly. In (1) and (2), the seismic coefficient is entered via the pseudo-static inclination angle with the vertical ψ (4).

$$\psi = \tan^{-1}\left(\frac{k_{he}}{1-k_v}\right) \quad (4)$$

In the calculations of ψ the vertical seismic coefficient k_v is set to zero, as is customary for sheet pile walls. The horizontal seismic coefficient k_h is translated into an equivalent horizontal seismic coefficient k_{he} to account for saturation of the soil in front and behind the quay wall. The angles θ and β in (1) and (2) are set to zero (no inclination of wall and backfill) in correspondence with the geometry of the present anchored sheet pile wall [8], [9].

The pseudo-static computation results for the forces in the sheet pile are compared with the reference case measurements. After comparison the original k_h-values of the shake events (and by that the K_{AE}-, K_{PE}- and p_w-values) are iteratively reduced by means of a factor r, i.e. $\left(\frac{k_h}{r}\right)$, until a fit is found between reference case measurements and pseudo-static results. The bending moment line fits are presented in Fig. 8.

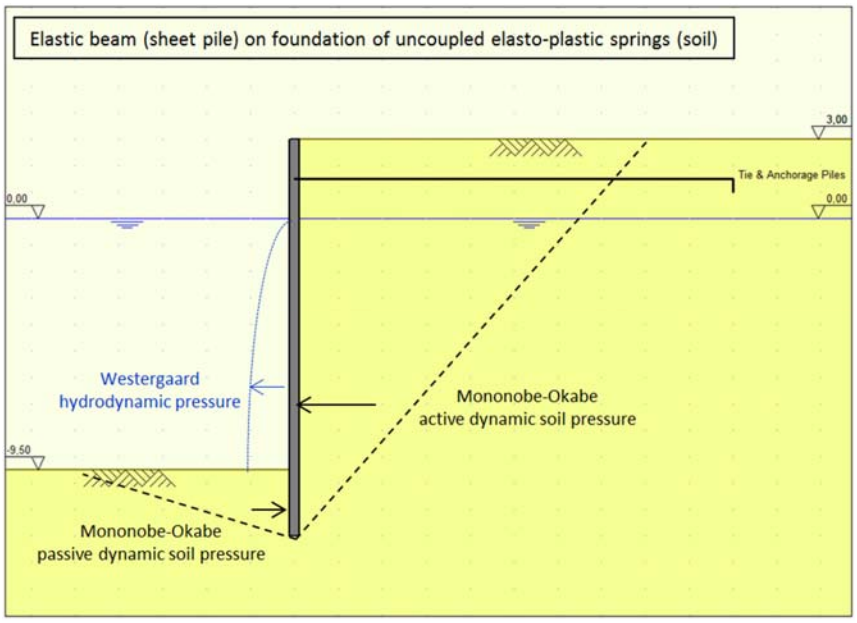

Fig. 7 Calibrated elasto-plastic spring model (D-SHEET PILING) for pseudo-static analysis

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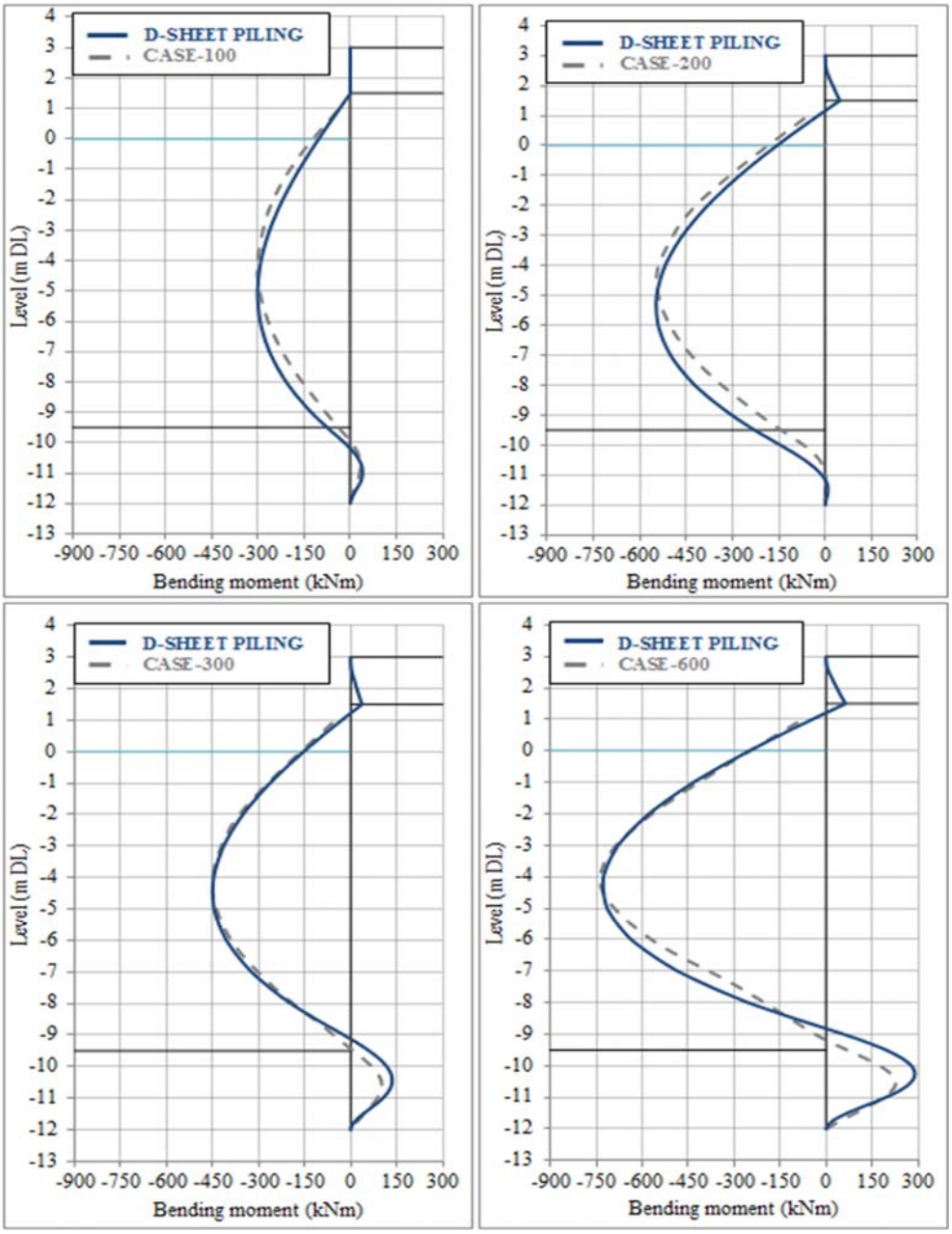

Fig. 8 Bending moment line fits between pseudo-static results and reference case measurements

Through the obtained fits deformation-based reductions on the pseudo-static load for structural forces in the sheet pile are determined. The reductions found for the present reference case are in the range of 45% to 50% (r = 1.82 to r = 2.00), as is presented in Table II.

TABLE II
CALIBRATED SIMPLIFIED ANALYSIS RESULTS

Parameter	CASE-100	CASE-200	CASE-300	CASE-600
Experiment seismic coefficient k_h (-)	0.10	0.20	0.30	0.60
Reduced seismic coefficient k_h (-)	0.05	0.11	0.15	0.32
Pseudo-static reduction factor r (-)	2.00	1.82	2.00	1.86

The results obtained after performing calibrated simplified analysis indicate that in pseudo-static methodology a deformation-based seismic load reduction for structural forces in the sheet pile could be permitted.

VI. SIMPLIFIED DYNAMIC ANALYSIS

The goal of the simplified analysis is to investigate the applicability of permanent-displacement (Newmark-rigid-sliding-block) analysis - originally developed for landslides and embankments [11] - on anchored sheet pile quay walls. In the analysis the assumed failure mechanism is a rigid block that starts to slide after exceedance of a certain critical acceleration. The amount of permanent displacement due to

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sliding can be computed by integrating the exceedance of a seismic accelerogram over the critical acceleration twice. This principle is clarified by Fig. 9 [11].

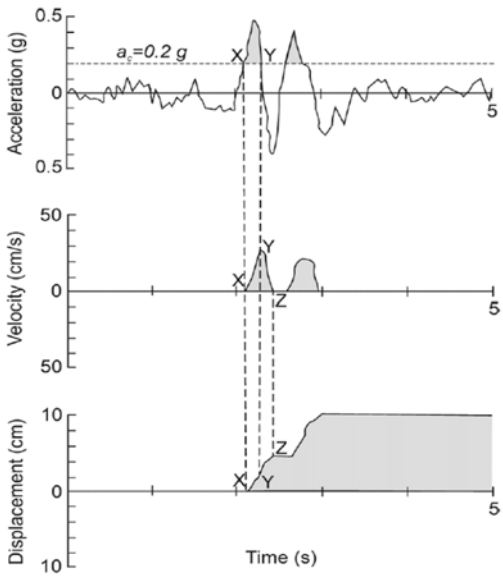

Fig. 9 The principle of permanent-displacement analysis, from [11]

For sliding-block analysis of the anchored sheet pile wall a critical acceleration has to be computed that in combination with the accelerograms of the sequential shake events results in permanent displacements. For obtaining the critical acceleration an analytical limit equilibrium model is developed by the author, based on failure mechanisms of anchored sheet pile walls as proposed in [12] and [13]. The limit equilibrium model is presented in Fig. 10.

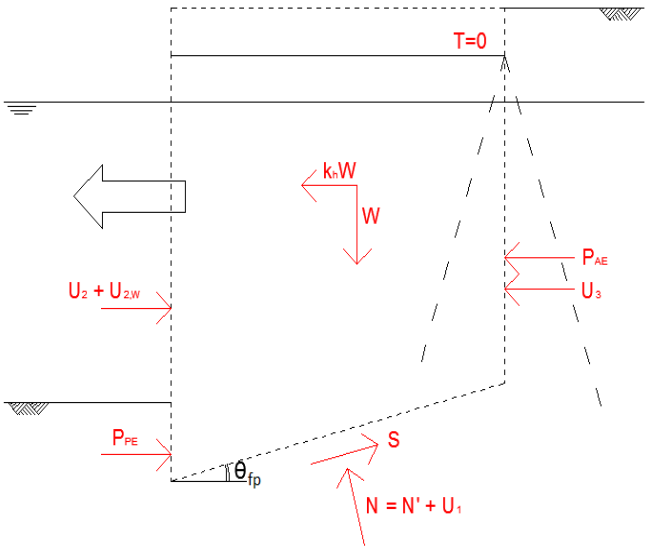

Fig. 10 Developed limit-equilibrium model for computing the critical acceleration of the anchored sheet pile wall

The model assumes that the entire soil-structure system starts to slide as a rigid body when exceeding the limit-equilibrium of forces that act on it. This model incorporates the simplifications that the batter pile anchor is schematized as an average vertical anchor pile, that the failure planes are not curved and that the friction between wall and soil is not considered (i.e. δ=0). Limit-equilibrium is reached in case of limit anchor stability (anchor situated within seismic failure wedge). From the equilibrium of forces on the rigid body the critical horizontal seismic coefficient k_{cr} at which the body starts to slide can be computed with (5). For the nomenclature of (5) reference is made to Appendix B.

$$k_{cr} = \frac{P_{PE}+U_2+U_{2,W}+S\cos\theta-U_1\sin\theta-N'\sin\theta-P_{AE}-U_3}{U_1\cos\theta+N'\cos\theta+S\sin\theta} \quad (5)$$

The ability of the limit-equilibrium model to compute the critical acceleration is validated with finite element calculations with a PLAXIS 2D model that is compliant with the reference case geotechnical and structural setup and calibrated with the static reference case measurement results. In the PLAXIS 2D model a pseudo-static horizontal acceleration is iteratively introduced at which the model develops the critical failure plane that causes soil failure. Such a PLAXIS 2D result is shown in Fig. 11.

Fig. 11 Failure plane at critical acceleration as computed with PLAXIS 2D

Comparison of the results of both models for two different soil setups (relative density 80% and 100%) shows that the limit-equilibrium and PLAXIS 2D results are in good agreement, as can be seen from Table III.

TABLE III
CRITICAL ACCELERATION RESULTS

Critical acceleration			
Soil material setup	Limit-equilibrium model	PLAXIS 2D model	Ratio (LE / P2D)
RD80%	0.22g	0.21g	1.05
RD100%	0.29g	0.27g	1.07

In addition to a difference in the order of only 5%, it can be seen that the results of the two models are also in good agreement with the experimental findings in the reference case where failure was observed between the 0.2g and 0.3g shake event.

Besides validating the ability of the limit-equilibrium model to compute the critical acceleration of the soil-structure system it is also checked whether the model is capable of computing structural forces in the sheet pile wall at the critical state. For this purpose, it is attempted to compute the shear forces (and from that the bending moments) in the sheet pile wall by dividing the sliding mass of the limit-equilibrium model into slices. For each of these slices equilibrium of forces

(equivalent to the forces in the original limit-equilibrium model) is derived. From these force-equilibria the resulting horizontal (shear) forces in the sheet pile wall are computed. This approach is clarified by Fig. 12.

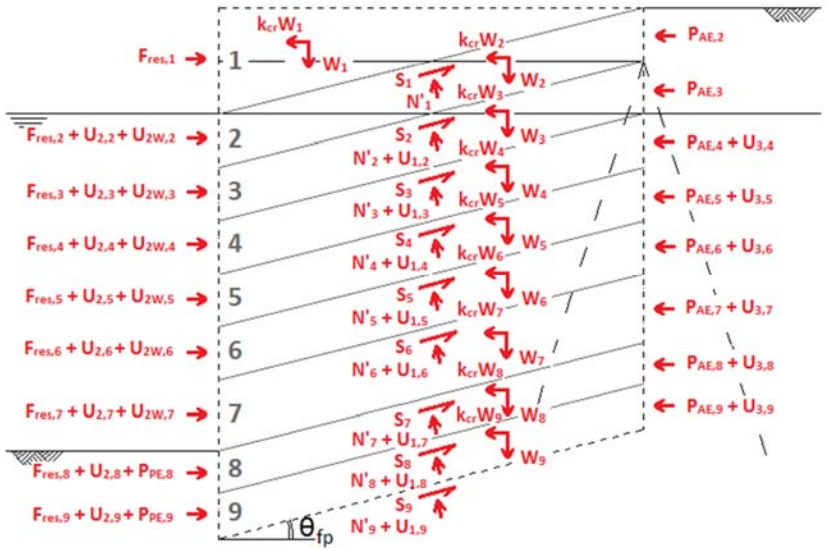

Fig. 12 Force-equilibria of single slices for computing structural forces in the sheet pile wall

The result for the critical acceleration of 0.29g (RD100% setup) is compared with the reference case measurements for the 0.3g shake event. It can be seen from Fig. 13 that the results are in reasonable agreement.

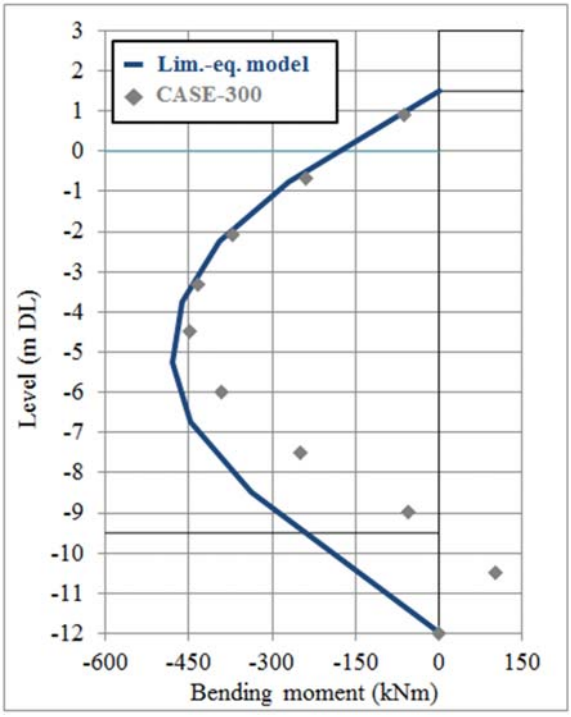

Fig. 13 Bending moment line computed with the limit-equilibrium model compared with the reference case measurements

Differences in results are explained by the erroneous assumption that is embedded in the way of computing the force-equilibria where it is assumed in the schematization of Fig. 12 that the slices of the sliding mass are rigid bodies which transfer the forces one-to-one onto the sheet pile wall. In reality these soil bodies deform internally through which transferred forces onto the wall are changed. The deviating bending moment shape beneath sea bed level is explained by the fact that for the critical acceleration the passive soil wedge starts sliding simultaneously with the rigid body and hence no passive soil pressure is exerted on the wall. For the CASE-300 measurements it is expected from the reference case behaviour that passive soil resistance has redeveloped again after passive soil failure.

The final step in the calibrated simplified dynamic analysis is to combine the computed critical accelerations with the accelerograms that are representative for the reference case shake events in order to calculate the permanent-displacements by double integration. In addition permanent displacements are computed with empirical regression equations from literature [14]-[18]. The permanent-displacement results are validated with dynamic PLAXIS 2D calculation results (see Fig. 14) and in addition compared with reference case displacement measurements.

It is found that the permanent-displacement results are in poor agreement with the PLAXIS 2D and reference case displacements. Only few permanent displacement results were within a range of 50% and all were too low. A possible explanation is that in the current analysis a constant critical acceleration is assumed while in reality this critical acceleration can be expected to be strain-dependent (decreasing with increasing strain) [11].

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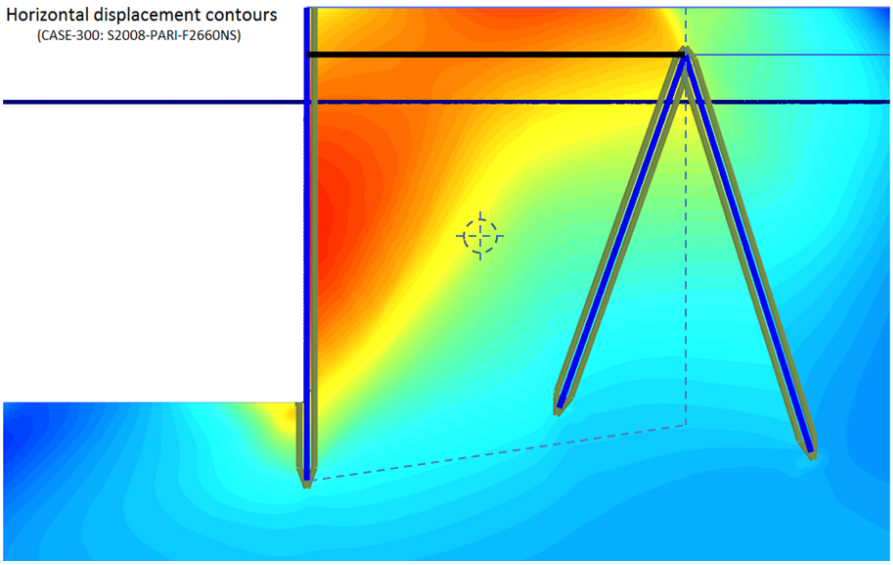

Fig. 14 PLAXIS 2D horizontal dynamic displacement contours plotted over the limit-equilibrium model layout

VII. Dynamic Analysis

The aim of the dynamic analysis is to validate findings from simplified and simplified dynamic analysis and to simulate the failure behaviour of the reference case experiment. The dynamic analysis is performed with the PLAXIS 2D model described in Section VI.

The PLAXIS 2D model is compliant with the (field) structural properties and dimensions of the reference case model (Fig. 15). To mitigate boundary effects in the relatively small geometry viscous boundary conditions are introduced to the vertical boundaries of the PLAXIS 2D model. The applied soil material model is the Hardening Soil small-strain stiffness model (HSsmall) because of the favourable dynamic damping characteristics [19]. Its parameters are determined with [20], and calibrated with the reference case static measurements.

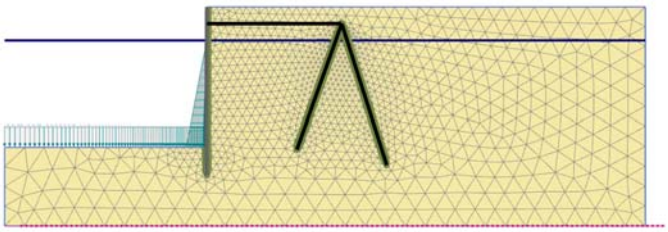

Fig. 15 PLAXIS 2D model of the reference case

The dynamic performance of the PLAXIS 2D model is validated by comparing 1D site-response analysis output of the PLAXIS 2D model with the analytical equivalent-linear SHAKE2000 model. The site-response analysis results of both models are in proper agreement after adding a Rayleigh damping ratio of 5% to the HSsmall model in addition to its hysteretic and plasticity damping characteristics. No Rayleigh damping is added to the sheet pile wall as the frequencies of the predominant modes of the structure are outside the predominant frequency range of the seismic load. Furthermore, soil characteristics have a much stronger influence on the structural forces in the anchored sheet pile wall than the characteristics of the structural elements itself.

The sequential (seismic) loading in the PLAXIS 2D model follows the same phasing as the reference case experiment, i.e. initial phase, static phase (seabed at DL -7.5 m), static phase (seabed at DL -9.5 m), CASE-100, CASE-200, CASE-300 and CASE-600. For CASE-100 the reconstructed artificial signal is used. For CASE-200 to CASE-600 the six representative signals that were selected for permanent-displacement analysis are applied.

In Fig. 16, the PLAXIS 2D dynamic bending moment results in comparison to the reference case measurements are presented. The figure shows that the PLAXIS 2D result is in good agreement with the experiment for CASE-100. For CASE-200 to CASE-600 it is concluded that the PLAXIS 2D results are in a reasonable range of the experiment measurements but the bending moment shapes deviate. In addition, it is noted that the results of the six representative records are in good agreement with each other.

The (average) displacements of the sheet pile wall for the sequential shake events as computed by PLAXIS 2D, are plotted and compared with the reference case measurements in Fig. 17. It is shown that the amount of sliding displacement is in general properly determined by PLAXIS 2D but clearly overestimated for the 0.6 shake event. The type of failure behaviour (i.e. sliding with a larger displacement at anchor level than at seabed level) matches with the reference case.

Taking the bending moment and displacement results of CASE-200, CASE-300 and CASE-600 into account it appears that PLAXIS 2D does not simulate the exact failure behaviour of the reference case. Passive soil resistance is not sufficiently redeveloped in the PLAXIS 2D model after sliding commences, while in the reference case this is expected to happen. This can especially be seen from the difference between PLAXIS 2D and reference case in bending moment line shapes and the larger displacements calculated by PLAXIS 2D for the 0.6g shake event.

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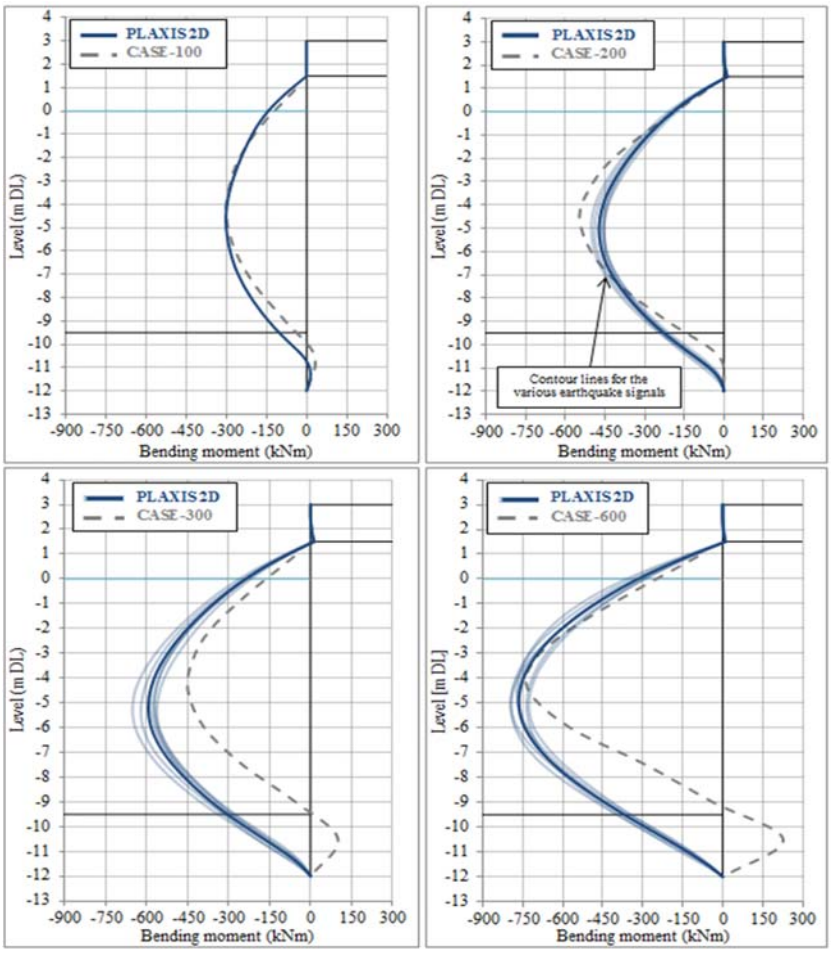

Fig. 16 PLAXIS 2D dynamic bending moment results in comparison to reference case measurements

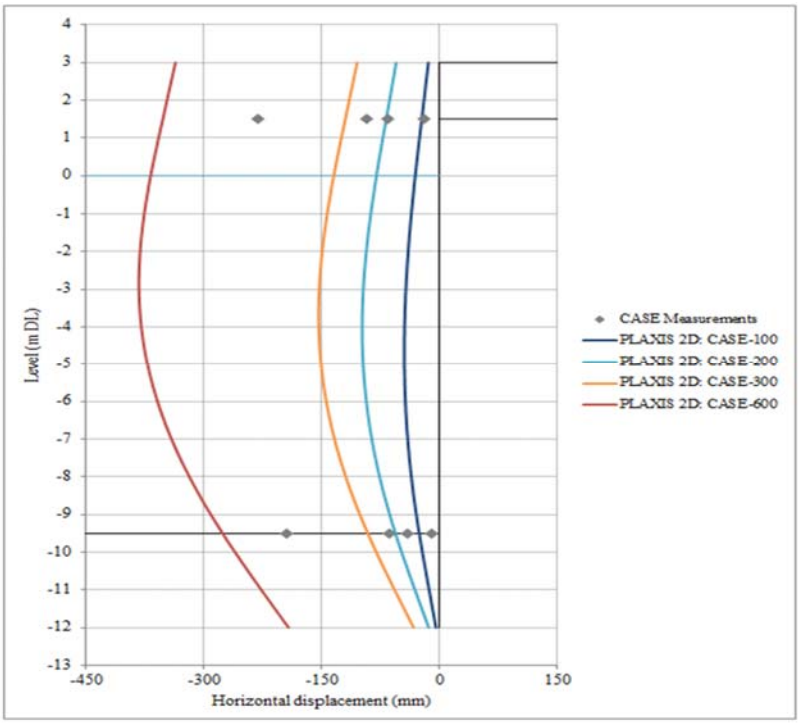

Fig. 17 PLAXIS 2D dynamic displacement results in comparison to reference case measurements

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VIII. Evaluation of Results

For simplified, i.e. pseudo-static, methodology a seismic load reduction is derived for structural forces in the sheet pile wall. This reduction has to account for deformation capacity. The reduction is specified by the reduction factor r which is applied via the Mononobe-Okabe and Westergaard equations. For the reference case it is concluded that a reduction in the range of 45% to 50% (r = 1.82 to r = 2.00) is adequate. These results follow from a careful calibration with the reference case and a reasonable validation with PLAXIS 2D.

In order to make traditional Newmark-sliding-block / permanent-displacement analysis more suitable for an anchored sheet pile quay wall a simple analytical limit-equilibrium model for the reference case is developed. From comparison with the reference case and validation with PLAXIS 2D it is concluded that the limit-equilibrium model is able to properly estimate the critical acceleration of the anchored sheet pile quay wall. In addition, it is concluded from comparison with the reference case that the limit-equilibrium model is able to compute the structural forces in the sheet pile wall for the critical acceleration with reasonable accuracy.

From the permanent-displacement results it is concluded that more research effort is required to determine whether permanent-displacement analysis as a whole is suitable for anchored sheet pile quay walls. It is observed that permanent displacement results do not match with the displacements of the reference case. An explanation can be that permanent-displacement analysis, originally developed for embankments / landslides, is less suitable for anchored sheet pile quay walls. The validity of this explanation is questionable though, as the exact properties of the applied earthquake signals (representative but not equal to the experiment record) can have a significant influence on the permanent-displacement results. Furthermore, it shall be noted that in the current analysis a constant critical acceleration is assumed while in reality the critical acceleration is expected to be strain-dependent and decreasing with increasing strain.

Dynamic analysis is performed to validate simplified and simplified dynamic analysis results. It is concluded that PLAXIS 2D is able to compute the reference case failure behaviour reasonably well. Differences with the experiment behaviour are explained by computational limitations in simulating large soil deformations and soil mass relocation.

Complementary it is concluded that the PLAXIS 2D pseudo-static approach proves to be suitable to determine the critical acceleration of an anchored sheet pile structure.

A final aspect to mention is a reflection on the trend in seismic quay design, as presented in Fig. 1. The question arises whether it can be quantified to what extent the '0.15g-boundary' for anchored sheet pile walls can be crossed, based on literature and the present study results. In Fig. 18 the permanent-displacement values at top of sheet pile corresponding to different degrees of damage as reported by [2] and [21] are graphically combined with both the reference case and PLAXIS 2D top displacements.

From Fig. 18, it can be seen that it would be possible (without increasing structure dimensions) to allow a 0.3g earthquake signal, with only negligible damage on the sheet pile wall. This is associated with only a short disruption of serviceability (e.g. due to damage to cranes on top of the quay) or no disruption of serviceability at all.

The permanent-displacement value at top of sheet pile proposed by [2] is 1.5% of the retaining height. This value is in Fig. 18 included at 190 mm and also corresponds to serviceability performance. Compared to the values given by [21] this seems somewhat optimistic. By interpreting the displacement value given in [2] as a value which is a trade-off between serviceability and a reasonable amount of damage, and by interpreting the displacement value by [21] for 'noticeable wall damage' as a first upper boundary for acceptable damage, it is deduced from Fig. 18 that earthquakes in the range of 0.4g to 0.6g are still associated with repairable damage. Earthquakes that are more severe are expected to result in non-acceptable amounts of damage and eventually collapse.

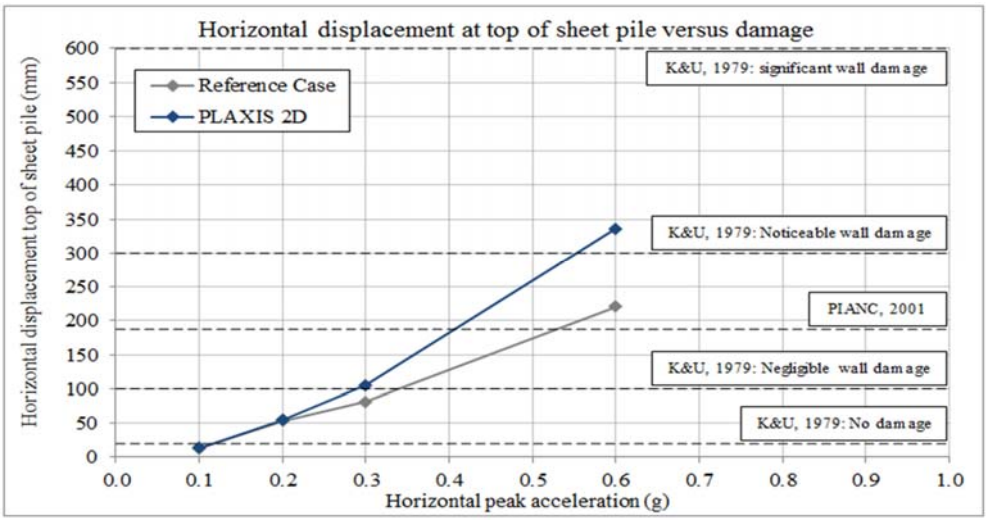

Fig. 18 Findings from literature and the present study: horizontal displacement at top of sheet pile versus damage

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It is emphasized that the quantitative reflections on PBD are based on the characteristics of the present reference case only and are maybe related with the specific failure mechanism of passive soil failure, which limits the maximum possible wall bending. Nonetheless an indication of the seismic performance limits of anchored sheet pile quay walls in quantitative terms is provided. For the anchored sheet pile wall of the reference case it is concluded that it possible to cross the '0.15g seismic design boundary' without substantial increase of dimensions of the structure but with acceptance of a certain amount of (controllable) damage.

IX. Conclusions and Recommendations

Pseudo-static analysis with a reduction on seismic load that accounts for deformation capacity appears not only applicable to gravity walls [6], but also to anchored sheet pile walls. Elements of simplified dynamic analysis appear adequate for calculation of anchored sheet pile walls also. More research on the influence of a strain-dependent critical acceleration is required though. Dynamic analysis successfully validates findings from simplified and simplified dynamic analysis and simulation in general complies with experiment measurements. As a result of the present research findings it is concluded that performance-based design (PBD) evaluation can be effectively used for anchored sheet pile quay walls.

Three main recommendations follow from the present study. First of all, it is recommended to perform further study on the strain-dependent critical acceleration in simplified dynamic analysis. Secondly it is suggested that more seismic test cases of anchored sheet pile walls with varying setups are created. In general, a recommendation is made with respect to field measurement data. It is believed by the author that field measurement data needs to become available for increasing the value of seismic research of anchored sheet pile walls. With this data true verification of research findings can be achieved. To this end it is recommended to install instrumentation on existing quay walls and to make seismic measurement data publicly available.

Appendix

A. Nomenclature Mononobe-Okabe and Westergaard Equations

β inclination angle of the backfill with respect to the horizontal (°)
γ_w unit weight water (kN/m^3)
δ soil wall friction angle (°)
θ inclination angle of the wall interface with respect to the vertical (°)
φ friction angle of the soil (°)
ψ inclination angle of the seismic coefficient k with the vertical (°)
H_{wall} retaining height of sheet pile quay wall (m)
k_h horizontal seismic coefficient (-)
k_{he} equivalent horizontal seismic coefficient (-)
k_v vertical seismic coefficient (-)
K_{AE} dynamic active soil pressure coefficient (-)
K_{PE} dynamic passive soil pressure coefficient (-)
p_w dynamic water pressure (kN/m^2)
z_w depth in water column (m)

B. Nomenclature in (5)

k_h horizontal seismic coefficient (-)
N normal force in failure plane beneath sliding mass, whose effective component is denoted by N' (kN)
P_{AE} dynamic active soil thrust behind vertical failure plane (kN)
P_{PE} dynamic passive soil thrust in front of sheet pile (kN)
S shear force along the failure plane beneath the sliding mass (kN)
T force in anchor tie (kN)
U_1 Hydrostatic force 1, in failure plane beneath sliding mass (kN)
U_2 Hydrostatic force 2, in front of sheet pile (kN)
$U_{2,W}$ Westergaard hydrodynamic force over the water depth in front of sheet pile (kN)
U_3 Hydrostatic force 3, behind the vertical failure plane (kN)
W weight of sliding soil body (kN)
θ_{fp} angle of the failure plane beneath the sliding mass, with respect to the horizontal (°)

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