Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0417

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

A modified modal earthquake analysis of caisson foundations

Une analyse sismique modale modifiée des fondations sur caisson

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ABSTRACT: Earthquake design of structures with caisson foundations requires estimation of the earthquake loads, sufficient bearing capacity and evaluation of their displacements induced by the earthquake event. A simplified modal non-linear analysis (SMNA) procedure, developed for caisson foundations, such as subsea foundations, monopiles and bridge foundations, has previously been established (ISFOG-2015). In view of reducing the computational time and the input data required to describe the soil response and earthquake loading, the details of stresses and deformations in the soil mass were replaced in SMNA by the force and moment resultants acting on the caisson with the resulting displacements.

A modified modal procedure (MMNA) is conceived in order to simplify the required input data and the iteration process to improve its convergence. The MMNA procedure uses a generalized force-displacement- and moment-rotation formulation based on a base hyperbolic relation and a scaling factor depending on the ratio moment/horizontal force. The H-M space is generalized in terms of bearing factors and interaction coefficients. The formulation is validated by specific FE-analyses in Plaxis 3D. The MMNA procedure is compared with results from time domain analyses based on the same geometry and the same seismic input data. The comparison shows that the MMNA procedure gives a somewhat conservative earthquake response compared with the time history analyses. Time history analyses are on the other hand more time consuming in terms of input and computational time.

RÉSUMÉ: Le dimensionnement parasismique des structures fondées sur caisson nécessite l'estimation des charges sismiques, de la capacité portante nécessaire et une évaluation de leurs déplacements en réponse au tremblement de terre. L'analyse simplifiée non linéaire modale (ASNM) a précédemment été développée (ISFOG-2015) pour les fondations sur caissons telles que les fondations sous-marines, les monopieux et les fondations des piles de ponts. Dans le but de réduire le temps de calcul ainsi que le nombre de données d'entrée permettant de décrire la réponse du sol et le chargement sismique, il est possible de remplacer, dans l'ASNM, le détail des contraintes et des déformations dans le sol par la résultante des forces et des moments qui s'exercent sur le caisson en générant des déplacements équivalents.

L'analyse sismique modale modifiée (AMNM) permet de simplifier les données d'entrée et le calcul itératif ainsi que d'améliorer sa convergence. L'AMNM utilise une formulation généralisée force-déplacement et moment-rotation suivant une fonction hyperbolique et un facteur d'échelle fonction du rapport entre le moment et les forces horizontales. L'espace H-M est généralisé en termes de facteurs de capacité portante et de coefficients d'interaction. La formulation est validée par des analyses aux éléments finis avec Plaxis 3D. La procédure MMNA est comparée avec des résultats d'analyses dans le domaine temporel utilisant la même géométrie et les mêmes données sismiques. La comparaison montre que la procédure MMNA donne une réponse sismique assez prudente comparée aux résultats d'analyses dynamiques transitoires. Les analyses dynamiques transitoires sont, par contre, plus longues à mettre en place et à calculer.
Keywords: Earthquake Design, Caisson Foundations, Modal analyses, Soil-structure interaction.

1 INTRODUCTION

The earthquake loads acting on caisson foundations and offshore subsea foundations are traditionally determined either by using the oversimplified assumption that the structure has one degree of freedom, or by dynamic soil-structure interaction analyses in time or frequency domain. A simplified iterative modal analysis procedure was outlined by Athanasiu et al. (2015), which accounted for non-linearity in the soil, the interaction between horizontal and moment loading acting at the caisson, as well as soil damping. The analysis procedure was denoted as Simplified Modal Non-linear Analyses (SMNA), where the global response of the caisson foundation was derived.

The SMNA procedure has been further generalized is terms of; 1) a generalized force-displacement (H- δ) and moment-rotation (M- θ) relationship for combined loading, 2) an updated capacity envelope for combined horizontal and moment loading, 3) an updated iteration procedure to improve its convergence. The revised procedure is denoted as Modified Modal Non-linear Analyses (MMNA).

The revised formulation is validated by FE-analyses in Plaxis 3D (Brinkgreve, 2018). Further, the MMNA procedure is compared with results from time domain analyses with equal geometry and seismic input data from Rørvik & Todem (2018). A parametric study is then presented, where the earthquake response of different caisson geometries is evaluated.

MMNA applies to single caisson foundations, where the structural stiffness is relatively stiff compared to the soil stiffness, which is typical for subsea structures. For other caisson foundations where the structure is flexible compared to the soil stiffness, such as wind turbines and bridge structures, the principles of MMNA can be used, by considering an interaction with a structural model, as shown at the end of the paper.

2 MODIFIED MODAL NON-LINEAR ANALYSIS (MMNA)

2.1 Principles of SMNA and MMNA

The SMNA method is based on a 2 DOF modal analysis procedure, in which the caisson response is modelled by a capacity surface in H-M (horizontal force-moment) space and back-bone curves for H-δ and M-θ with different loading ratios h=M/H.

The iterative procedure is subject to the following conditions; 1) dynamic stiffness should be compatible with the soil caisson response stiffness, 2) the dynamic loading ratio $h_{dyn}=M_{EQ}/H_{EQ}$ is the same as the assumed loading ratio h, 3) δ and θ is the same as the response obtained in the soil/caisson analyses, 4) the same mobilization degree, $f=H/H_{ult}=M/M_{ult}$ is obtained for both the dynamic force and the dynamic moment.

The MMNA procedure is based on the SMNA method, where the following aspects have been further generalized and updated; 1) Generalized H-M yield envelope, 2) Generalized force-displacement and moment-rotation relationships, and 3) Updated iteration process.

The required input data for MMNA procedure may be summarized as follows; 1) response spectra, 2) damping profile in terms of damping ratio vs mobilization degree, 3) Total mass of the caisson/structure/added mass/added soil (M_{tot}), with the corresponding total mass moment of inertia (I_θ), about the centre of gravity (CoG) of the system, 4) parameters required to define the H-M capacity surface, 5) Stiffness parameters for the H-δ and M-θ curves. The formulations are given in the subsequent sections.

2.2 H-M capacity surface

The H-M yield surface is given in terms of normalized bearing capacity factors N_P and N_M, where N_P is the normalized lateral caisson capacity, defined as:

$$N_p = \frac{H_{ult}}{D \cdot L \cdot s_{u,average}} \quad (1)$$

and the normalized moment capacity is given as:

$$N_M = \frac{M_{ult}}{L \cdot H_{ult}} = \frac{M_{ult}}{N_p \cdot D \cdot L^2 \cdot s_{u,average}} \quad (2)$$

For aspect ratios L/D>2, N_P is usually in the range of 7-11 while N_M is usually is the range of 0.20-0.24, Sørlie (2013). Both factors are depending on the shape of the shear strength profile, OCR, if suction behind the caisson shall be accounted for etc.

The reference position for H-M surface is located at the decoupling position (DP), where the interaction between horizontal loading and moments can be neglected, as illustrated in Figure 1.

The ultimate horizontal soil capacity can be determined by specific FE-analyses, by using limit state software, such as DOWEL (Athanasiu 2009) or DISEL (Athanasiu 1999), or be estimated using published literature such Kay & Palix (2011), Brandt & Athanasiu (2016) or Sørlie (2013). The yield envelope is given in equation 3.

$$\left(\frac{H}{f \cdot H_{ult,0}}\right)^{\alpha} + \left(\frac{M}{f \cdot M_{ult,0}}\right)^{\beta} = 1 \quad (3)$$

Where α and β are interaction coefficients.

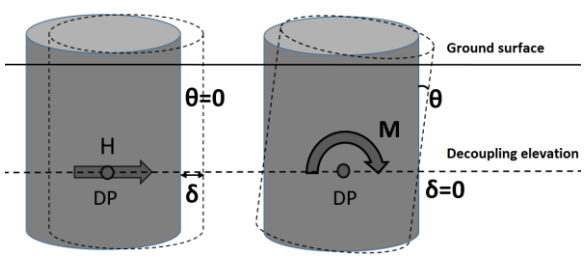

Figure 1. Principle of decoupling elevation/ decoupling position (DP).

2.3 *Generalized H-δ and M-θ relationship for combined loading*

The MMNA is based on a base hyperbola for both force-displacement and moment-rotation relationships, as shown in the equation 4.

$$H = \frac{\delta}{\frac{1}{K_{\delta,initial}} + \frac{\delta}{R_{\delta,ult} \cdot H_{ult,0}}} \quad (4a)$$

$$M = \frac{\theta}{\frac{1}{K_{\theta,initial}} + \frac{\theta}{R_{\theta,ult} \cdot M_{ult,0}}} \quad (4b)$$

Where $K_{\delta,initial}$ and $K_{\theta,initial}$ are the initial stiffness of the H-δ and M-θ curves and R_{ult} is the ratio between the asymptotic value of the hyperbola and the capacity.

For combined loading, a horizontal load will in general give a reduced rotational stiffness, while a moment will cause a reduction in the horizontal stiffness. This interaction effect is included in the generalized formulation by introducing a normalized loading angle, defined by equation 5.

$$\tan(\beta) = \frac{\left(\frac{H}{H_{ult}}\right)}{\left(\frac{M}{M_{ult}}\right)} \quad (5)$$

The generalized formulation may be reformulated by using the mobilization degree rather than H or M, and by using failure displacement and failure rotation, as shown in the equations 6.

$$\delta = \frac{\cos(\beta) \cdot \delta_{ult} \cdot \frac{K_{ult}}{K_0} \cdot f}{1 - \left(1 - \frac{K_{ult}}{K_0}\right) \cdot f} \quad (6a)$$

$$\theta = \frac{\sin(\beta) \cdot \theta_{ult} \cdot \frac{K_{ult}}{K_0} \cdot f}{1 - \left(1 - \frac{K_{ult}}{K_0}\right) \cdot f} \quad (6b)$$

The stiffness ratio K_{ult}/K_0 represents the ratio between the secant stiffness at soil failure and the initial stiffness, while the parameters δ_{ult} and θ_{ult} represent the failure displacement and the failure rotation. In total, 4 independent displacement/stiffness parameters are required; $\frac{K_{ult}}{K_0}$ and δ_{ult} for M=0 and K_{ult}/K_0 and δ_{ult} for H=0.

The $\cos(\beta)$- and $\sin(\beta)$-terms ensure a caisson response where the caisson translates without rotation if a horizontal force is applied at DP, and a caisson response where the caisson rotates without translation at DP if a moment is applied.

As the displacement vector is not associated with the yield surface, the formulation may be regarded as non-associated flow in terms of the elasto-plastic framework.

2.4 Updated iteration procedure

The iteration procedure for MMNA may be summarized as follows:

1) For iteration 0, assume an initial value of the mobilization degree f and a loading ratio h=M/H. Based on loading ratio, calculate H_{ult} and M_{ult} from equation 1 and the normalized loading angle β from equation 3. For other iterations, f and h are calculated from the previous iteration, where f is calculated by equation 7.

$$f_n = \frac{1}{2}(f_{H,n-1} + f_{M,n-1}) \qquad (7)$$

2) Calculate the damping ratio D from f, and determine the response spectra modification factor η from damping ratio. The relationship between D and η may be determined using either Eurocode or ISO, given by equation 8.

$$\eta = \sqrt{\frac{10}{5+D}} \geq 0.55 \text{ (Eurocode)} \qquad (8a)$$

$$\eta = \ln(5)/\ln(D),\ D \leq 10\% \text{ (ISO)} \qquad (8b)$$

3) Calculate the global linearized translational stiffness K_δ and rotational stiffness K_θ based on ratio H/δ and M/θ. The force and moment is calculated based on f and H_{ult} and M_{ult}, while displacement and rotation is calculated based on f and β.

4) the diagonal stiffness matrix at DP is converted to a full stiffness matrix at CoG position, and the two eigen-frequencies and the two eigenmodes are calculated (Chopra 1995). The pseudo-spectral accelerations, $PSa(T_1)$ and $PSa(T_2)$ are then calculated from the response spectra as illustrated in figure 2. The damping modification factor is accounted for by multiplying the spectral accelerations with η.

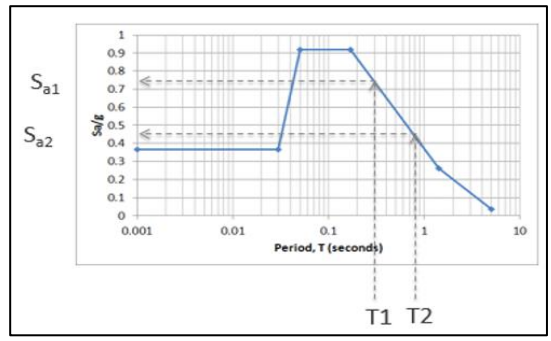

Figure 2. Conversion from eigen-periods to spectral accelerations

5) Calculate the force and the moment for the two modes similar to Athanasiu *et al* (2015). The dynamic forces from the two modes are combined using Square Root of Sum of Squares (SRSS). The loads are first calculated at CoG position, and converted to DP.

6) Calculate the translation and rotation at DP position based on the dynamic forces obtained in step 5 and the secant (linearized) stiffness (K_δ and K_θ) used in the current iteration.

7) Calculate the mobilization degree f_H and f_M based on H-δ- and M-θ-relationship for the current loading ratio h and the obtained δ and θ from step 6. Step 5-7 is illustrated in Figure 3.

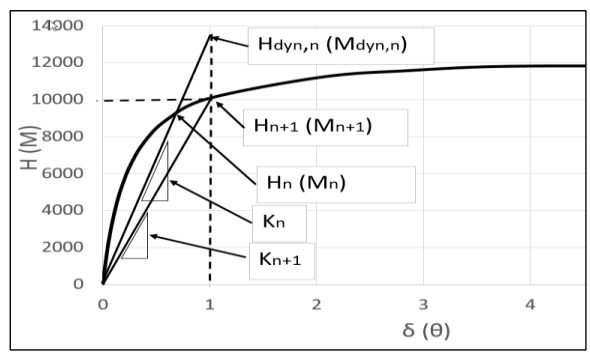

Figure 3. Illustration of iteration procedure in terms of δ vs H or θ vs M

8) Repeat step 1-7 until a convergence is obtained. Usually 3-5 iterations are sufficient.

The iteration procedure fulfils the compatibility requirements presented in section 2.1. The

convergence for an offshore project outside India is shown in Figure 4.

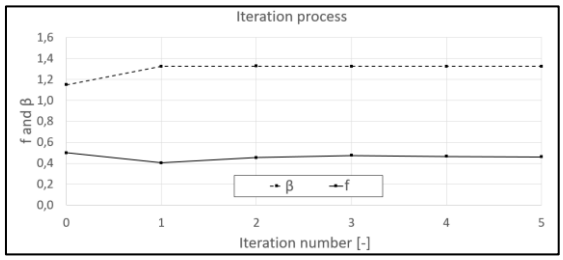

Figure 4. Mobilization degree f and normalized loading angle β vs iteration

3 VALIDATION OF PROCEDURE

3.1 Validation for MMNA formulation by static FE-analyses

Static FE-analyses have been carried out to validate the formulation for H-M capacity and H-δ and M-θ formulation. The soil conditions and the caisson geometry is summarized in table 1.

Table 1. Model parameters for static FE-analyses

Parameter	Value
Caisson aspect ratio L/D	2.2
s_{uDSS}/p_0'	0.35
OCR	1.2 - 1.5
I_p	40%
G_{max}/s_u	1000
G_{50}/s_u	150

The static FE-analyses have been carried out in Plaxis 3D (Brinkgreve 2018). The soil model Hardening Soil small strain (HSss) has been used to represent the soil. Cyclic effects on stiffness and strength have been evaluated based on Andersen (2015), and by evaluating the cycles from specific time histories from site-response analyses. The model is shown in Figure 5. Different loading combinations h=M/H have been considered, which is compared with the MMNA formulation.

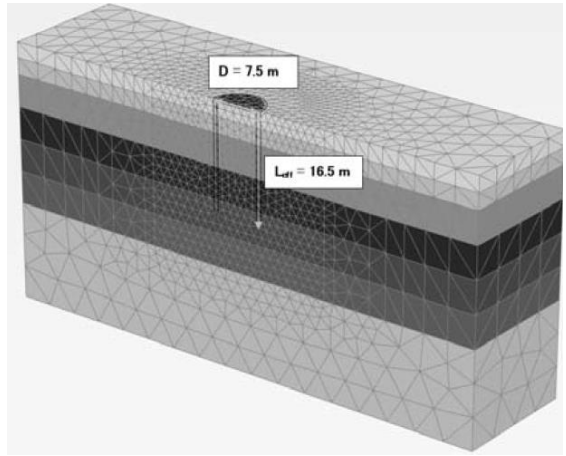

Figure 5. Static FE-model

The resulting H-M capacity surface and the H-δ and M-θ curves from both the MMNA procedure and from the FE-analyses is presented in Figure 6 and Figure 7. It is seen that the MMNA procedure gives an excellent representation of the results from the FE-analyses.

The model parameters used for the MMNA formulation are summarized in table 2.

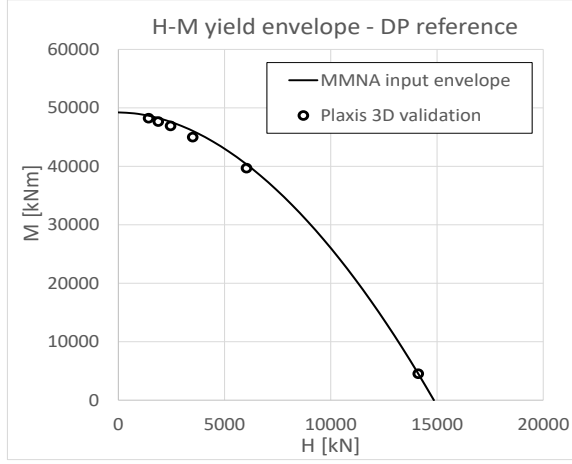

Figure 6. Capacity envelope from MMNA formulation and Plaxis 3D validation

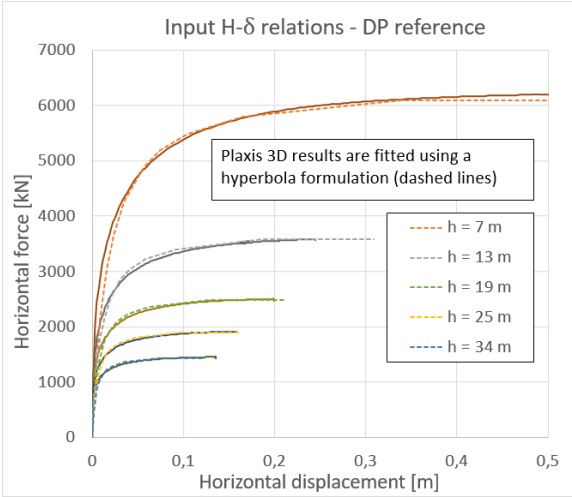

Figure 7. Force vs displacements (at DP) by generalized hyperbola formulation and from Plaxis 3D analyses

Table 2. Model parameters for static FE-analyses

Parameter	Value
N_P	10
N_M	0.20
α	1.9
β	1.0
δ_f/D	10 %
θ_f	5 degrees
$(K_{ult}/K_0)_\delta$	5 %
$(K_{ult}/K_0)_\theta$	6 %

3.2 Validation of MMNA by time-history analyses (THA)

Time domain analyses of a caisson foundation supporting a subsea structure have been carried out by Rørvik & Todem (2018). Time-histories were first derived from a seabed response spectra by performing site-response analyses.

Then, depth specific time histories were applied at the far-end of the soil springs in the soil-structure interaction model. The caisson was modelled as a beam on non-linear Winkler foundation (p-y curves, where the cyclic behaviour followed the Masing rules). The p-y curves were calibrated towards Plaxis 3D analyses.

The results from the analyses were compared with the MMNA procedure and are shown in table 3. It is seen that the modal analyses gives a conservative earthquake response compared to the time domain analyses for this particular case. This is in line with expectations due to following factors: 1) The reaction forces from modal analyses are combined using SRSS method, which usually leads to conservative estimates of the reaction forces, 2) The equivalent damping ratio using p-y curves and the Masing rules may deviate from the damping profile used for modal analysis, 3) The relationship between η and D given by the ISO-code is likely to be conservative, 4) The time-history is based on non-linear analyses, and will in general be different than equivalent linear modal analyses. THA was also compared with SMNA in Athanasiu et al. (2015), the THA gave a slightly lower earthquake response.

Table 3. Earthquake response; comparison between results from time-history analysis (THA) and from MMNA procedure

Parameter	THA/MMNA
Acceleration at ground surface	0.5
Displacement at ground surface	0.9
Caisson rotation	0.5
Moment at ground surface	0.8
Base shear at ground surface	0.6

4 PARAMETRIC STUDY OF CAISSON GEOMETRY

A parametric study has been carried out which considers the effect of the aspect ratio, L/D. The soil conditions for the parametric study are summarized in Table 1. The geometry and weights is the same for all cases, however with a different caisson length. 3 cases have been considered; L/D= 1.5, 2.2 and 3.0. The change in earthquake response as a function of caisson length is summarized in Table 4. Figure 9 – Figure 11 show the base shear at ground surface vs aspect ratio, soil mobilization degree vs aspect ratio and caisson

displacement at ground surface vs aspect ratio, respectively.

It is seen that both the base shear and the base moment at ground surface increases with larger caisson lengths, which is due to a larger caisson stiffness, which gives larger spectral accelerations. The soil mobilization degree and displacements are decreasing with increased caisson lengths, as the caisson capacity and caisson stiffness increases significantly with increased caisson length.

Table 4. Earthquake response for different aspect ratios L/D

Parameter	L/D=1.5	L/D=2.2	L/D=3.0
θ	1.14	0.42	0.22
δ_{seabed}	0.30	0.13	0.08
a_{seabed}	5.55	7.92	9.04
H_{seabed}	2131	2819	3111
M_{seabed}	8164	10162	10870

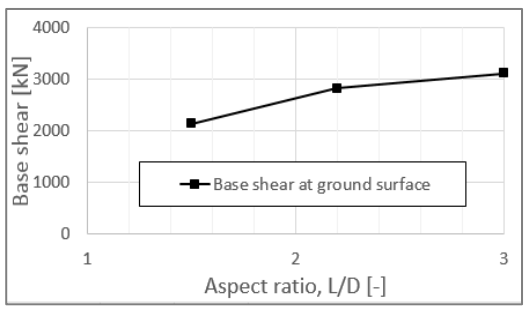

Figure 9. Base shear at ground surface vs aspect ratio L/D

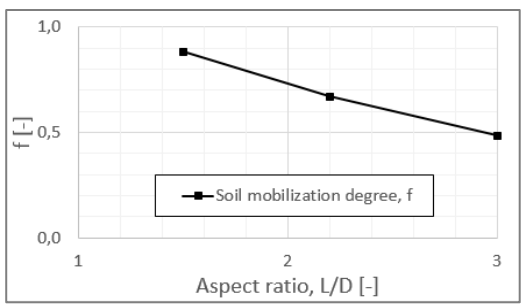

Figure 10. Soil mobilization degree, f vs aspect ratio (L/D)

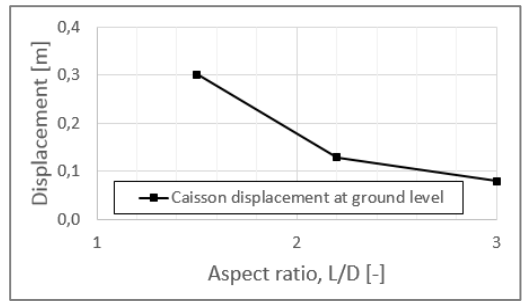

Figure 11. Caisson displacement at ground surface vs aspect ratio

5 APLICATIONS TO FLEXIBLE STRUCTUES

The MMNA procedure is developed for caisson foundations where the structure is considered stiff compared to the soil.

For slender structures supported by caisson foundations, such as wind turbine structures and bridge structures, the effect of the structural stiffness should be considered in an earthquake evaluation. The principles behind the MMNA procedure can be used combined with considering the interaction with a structural model. The analysis procedure for such structures is defined as follows:

1) Establish the H-M capacity surface and the H-δ- and M-θ curves according to section 2.

2) Establish the structural model where both the mass of the caisson and the added soil mass should be included. The horizontal and rotational spring should be applied at the DP elevation.

3) Determine the secant stiffness K_δ and K_θ from step 1-3 given in section 2.4. The soil damping will contribute to the overall modal damping ratio, which may be accounted for in the structural model.

4) Perform the structural analysis, and evaluate the updated reaction forces at the DP position.

5) Repeat step 3-4 above until a sufficient convergence is obtained. Usually, 1-2 iteration is adequate, given a reasonable initial stiffness for iteration 0.

6 CONCLUSIONS

The Simplified Modal Non-linear Analyses (SMNA) procedure (Athanasiu et. al 2015) has been further developed and generalized, and has been denoted as Modified Modal Non-linear Analyses (MMNA). It is a simplified earthquake procedure for caisson foundations where the structure is relatively stiff compared to the soil response. An analysis procedure for flexible structures supported by caisson foundations is also described, where the MMNA procedure is combined with a structural model which enables the procedure to be used for flexible structures.

The MMNA procedure has both been validated by performing static FE-analyses in Plaxis 3D and by comparing the results from MMNA with results from time history analyses. The comparison show that the MMNA procedure gives a somewhat conservative earthquake response compared with the THA, which is in line with expectations. However, THA are in general more time consuming in terms of input and computational time. MMNA procedure is thus attractive for both concept studies and detailed design, while THA can be used for special cases. Further, a parametric study has been presented, where the earthquake response for different aspect ratios of the caisson L/D has been evaluated.

The MMNA procedure has been implemented in an in-house developed software. The procedure has been used in several projects located in different areas and conditions around the world.

7 ACHNOWLEDGMENT

The authors wish to acknowledge the financial support to R&D project provided by the Head of Geotechnical Division, Mr. Leif Olav Bogen. The contribution of colleagues through fruitful discussions is also gratefully acknowledged.

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