Advanced Numerical Analysis of Caisson Breakwater on Friction Materials

Ibsen, Lars Bo; Andersen, A. T.; Madsen, E. B.; Schaarup-Jensen, Kjeld

Published in:
Dynamics of Structures 1993-1997

Publication date:
1998

Document Version
Publisher's PDF, also known as Version of record

C.2 Advanced Numerical Analysis of Caisson Breakwater on Frictional Materials

L.B. Ibsen, Andersen A.T, Madsen E.B, Schaarup-Jensen A.L

Department of Civil Engineering, Aalborg University
DK-9000 Aalborg, Denmark

The Single Hardening Model was introduced and investigated with the purpose of evaluating its capability of predicting the behaviour of frictional materials. The database, resulting from the work in project C.2.2, has been used for the calibration of the model. Besides providing a solid basis for the parameter study, the triaxial tests serve to elucidate the general behaviour of sand. Among other things, the aim of this work has been to investigate the applicability of the Single Hardening Model for solving geotechnical problems of a more complex nature. In this article a caisson breakwater structure, which has been tested in the centrifuge at Delft Geotechnics, is modelled in ABAQUS by means of the verified material model subroutine containing the Single Hardening Model. The response of the caisson is investigated under drained as well as undrained conditions. The results from the numerical analysis are compared to results obtained from the centrifuge test. Furthermore, an analytical bearing capacity analysis in the shape of an upper bound solution is put forward as a reference for the numerically obtained horizontal bearing capacity. Finally, reference numerical calculations based on the simpler Drucker-Prager Model are brought in for comparison. The performed analysis with the Single Hardening Model and have these analysis are compared, with analytical and the centrifuge test. The work is fully reported in (Andersen et all, 1998.a).

1 INTRODUCTION

Traditionally, a general geotechnical problem is divided into two parts which are considered individually. Thus, the bearing capacity of the given structure is calculated by means of the theory of plasticity, with no regard to the deformation taking place. In this bearing capacity analysis, the soil is assumed to behave perfectly plastic. Subsequently, the settlements of the structure in question are considered. In this phase, the response of the soil material beneath the foundation is often assumed to be linear elastic. However, considering the stress-strain curve obtained from e.g. a triaxial test on a soil specimen, it becomes clear that the actual soil response is highly non-linear. This implies that neither the assumption of perfect plasticity, nor the assumption of linear elasticity, fit the experimentally observed behaviour well.

Over the years, the capacity of computers has undergone large developments. Hereby, it has become obvious to bring more complex solution methods into the analysis of geotechnical problems. Thus, when dealing with problems with a complexity, which displays the traditional methods as defective, it is obvious to employ a finite element program as e.g. ABAQUS for the solution. However, in order to obtain reliable results from a finite element analysis, it is essential that the applied constitutive theory is capable of matching the experimentally observed behaviour of the considered material in a satisfactory way. In course of time, several constitutive theories have been proposed in order to provide this fundamental basis for an exact modelling of the response of soil materials. Of these could be mentioned: The Mohr-Coulomb Model, the Drucker-Prager Model, and the Cam Clay Model. However, each of the above mentioned soil models has its limitations as regards to capturing the response of certain soil materials. As an example it could be mentioned that the Cam Clay Model mentioned above is generally considered to be able to capture the response of normally consolidated clay, whereas the constitutive theory defined by this model is insufficient for describing the response of frictional materials.

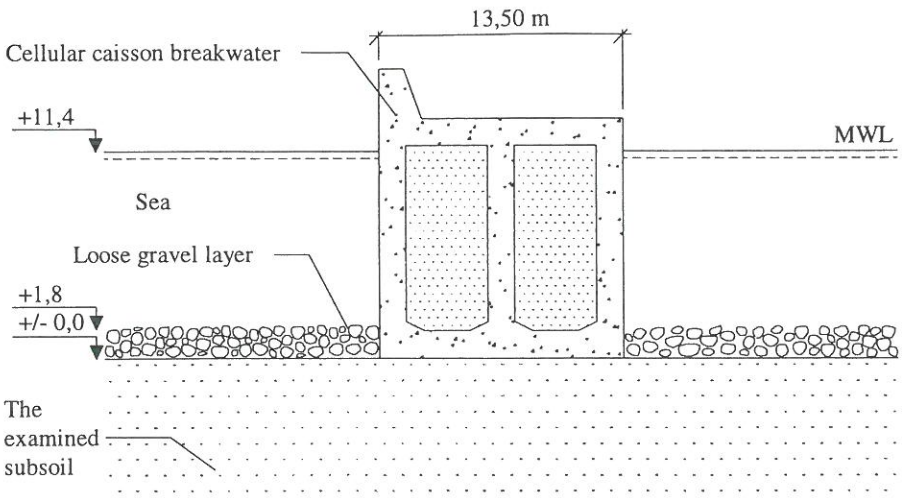

Figure 1. Section with the prototype of the continuous cellular caisson breakwater.

In connection with the present research, the Single Hardening Model developed by Poul V. Lade is of particular interest. The Single Hardening Model has been developed with the purpose of describing the response of frictional materials in a more accurate way. Among other things, the aim of this work has been to investigate the applicability of the Single Hardening Model for solving geotechnical problems of a more complex nature. In this article a caisson breakwater structure which has been tested in the centrifuge at Delft Geotechnics, is modelled in ABAQUS by means of the verified material model subroutine containing the Single Hardening Model. In Figure 1. a section with the prototype of the continuous cellular caisson breakwater modeled is shown.

2. CALIBRATION OF THE MODEL

On the basis of the results from the performed triaxial tests together with results from the database at the Soil Mechanics Laboratory at Aalborg University, it is possible to calibrate the Single Hardening Model for the two different materials in question, Baskarp Sand No 15 and Eastern Scheldt Sand. In this connection, it should be mentioned that calibration of the Single Hardening Model requires determination of twelve parameters. These parameters are listed below:

Elastic parameters: v, λ, M
Failure envelope parameters: η_1, m, a
Plastic potential parameters: ψ_2, μ
Hardening parameters: C, p
Yield surface parameters: h, q

The obtained set of calibration parameters at different void ratios have formed the basis for an investigation of a possible void ratio dependency formulation, as shown in (Andersen et all, 1997).

2.1 *Observed soil behaviour*

Besides providing a solid basis for the parameter study, the triaxial tests have served to elucidate the general behaviour of sand. In this way, it is stated that tested sand supports the characteristic state theory explained in detail elsewhere, (Ibsen L.B, 1998). Furthermore, the behaviour observed during large stress reversals has been subjected to an analysis. Performing un- and reloading cycles it was observed that plastic strains were present, in contrast with the theory of the Single Hardening Model where purely elastic behaviour during a load cycle is assumed. However, performing un- and reloading cycles down to 50 percent of the maximum shear stress, purely elastic behaviour was indeed observed. This supports the presence of the Bauschinger effect and questions the applicability of the Single Hardening Model, if a precise description of large un- and reloading cycles is wished for. Finally, the assumption of isotropic soil behaviour has been investigated, as this is a fundamental assumption made by the Single Hardening Model. Thus, the slopes of the volumetric strain curves from hydrostatic compression tests have been considered. It was found out that some scatter is present, as the slopes differ slightly from the required slope of 1 in 3. The scatter is put down to experimental uncertainties.

2.2 *Parameter calibration*

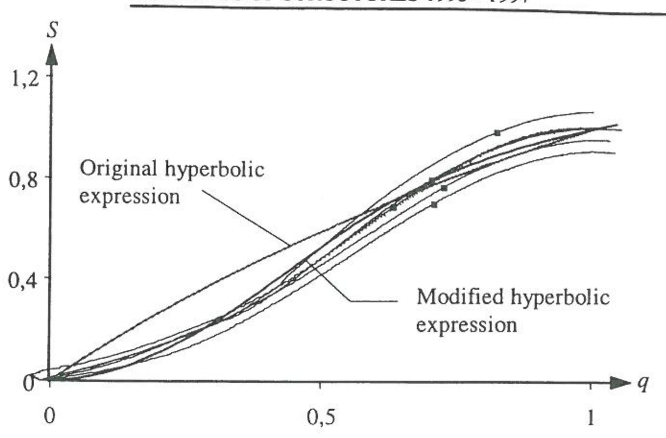

Figure 2. q,S variations for Baskarp Sand No. 15 e=0,85, approximated by the original hyperbolic expression and the modified hyperbolic expression.

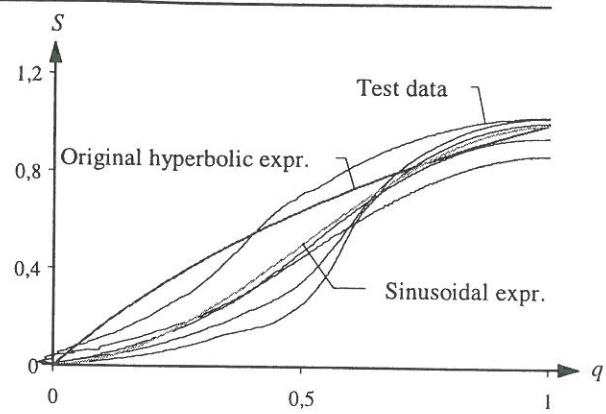

Figure 4. q,S variations for Baskarp Sand No. 15 with e = 0,70, and σ_3' = 80, 160, 320, 640, and 800 kPa. ζ = 0,502 and υ = 0,503.

Parameter calibrations have been carried out in the same manner as introduced in (Andersen et all ,1997). Similar tendencies were observed, as especially the yield surface parameter, *h* and the *q,S* variation are encumbered with some deviation relative to the assumptions of the Single Hardening Model, as shown in Figure 2 and Figure 4. Therefore, initiatives have been taken in order to improve the parameter calibrations in general.

First and foremost, the observed deviation relative to isotropic soil behaviour during hydrostatic compression has been investigated. Thus, a determination of the hardening parameters based on ε_1, only, was carried out. It has turned out that in this way, the variation of the yield surface parameter, *h* with respect to the confining pressure becomes less pronounced. Thus, a clear improvement of the triaxial test predictions was observed and therefore it has been chosen to base the entire parameter determination on this assumption. Still, the suggested *q,S* variation has been seen to capture the experienced variations inadequately, which has given rise to the development of alternative proposals for the *q,S* expression, as shown in Figure 2 and Figure 3. The first attempt resulted in a modified hyperbolic expression calibrated on the basis of the characteristic state, as shown in Figure 2 . However, the modified hyperbolic expression has been seen to be numerically unstable in a general FEM description. As a numerically stable alternative, a sinusoidal expression was developed, whereby a better reconstruction of the *q,S* variation for all materials is possible, as shown in Figure 4. Some deviations are still experienced for Baskarp Sand No. 15 at the smallest void ratios. It has been concluded that the best predictions are obtained using the sinusoidal expression combined with a reduced value of the initial slope of the softening curve.

The remaining model parameters have been investigated as well, in order to estimate the influence of a change of these parameters to the numerical predictions. In that way it has been concluded that the influence of a change of the elastic parameters is insignificant as long as the parameters are chosen

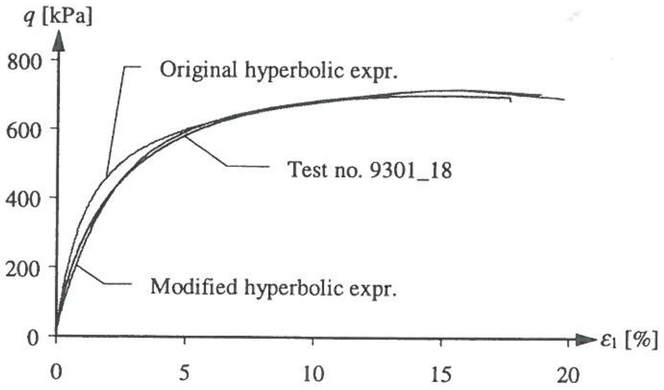

Figure 3. Predicted and experimental stress-strain curves for Baskarp Sand No. 15 with void ratio, e = 0,85, and σ_3' = 320 kPa.

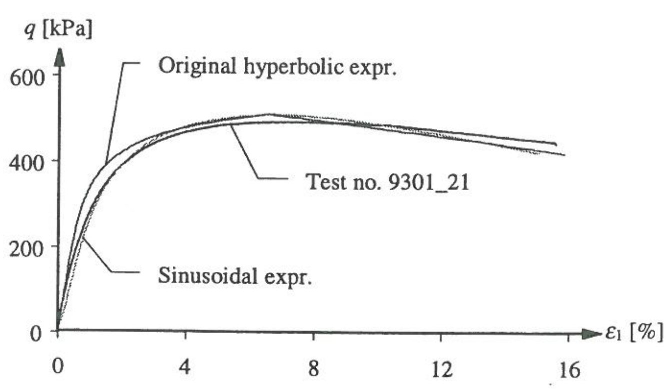

Figure 5. Predicted and experimental stress-strain curves for Baskarp Sand No. 15 with e = 0,70, and σ_3' = 160 kPa.

within a reasonable range. For triaxial test predictions, the same observation is present for the parameter, ψ_1, which governs the shape of the plastic potential- and yield surfaces in the octahedral plane. This parameter is determined on the basis of the failure parameter, m. The remaining plastic potential parameters, ψ_2 and μ have been seen not to influence the stress-strain behaviour. However, evaluating the predicted volumetric strain curves, the change of these parameters is clearly registered. It should be noted that no attempt has been done in order to modify the failure parameters, as these parameters are generally determined on a firm basis.

Finally, a void ratio dependency formulation has been put forward on the basis of the parameters determined from the tests in the database. As additional reference, data determined for Monterey No. 0 Sand, and Sacramento River Sand have been brought in. On the basis of the limited amount of data it is observed that a coherence between the parameters and the void ratio is present, although quite some scatter is present.

Summing up, it is stated that the below listed issues are considered to be essential in order to achieve numerical predictions of the highest quality:

1. In general, the experimental behaviour of Eastern Scheldt Sand is captured the best by the numerical predictions. One of the significant differences between the tests performed on Baskarp Sand No. 15, and the tests on Eastern Scheldt Sand is a smaller amount of preconsolidation during preparation for Eastern Scheldt Sand. As the development of plastic work is essential to the parameter determination, it is believed that it is important to carry out the preparation of the test specimens at the lowest possible vacuum in order to prevent a high rate of pre-consolidation.

2. Furthermore, it is advised to plan the performance of the triaxial tests in such a manner that the parameter calibration is supported by reliable test results. Thus, a hydrostatic compression test to a high level of confining pressure is an absolute must in order to determine reliable hardening parameters. It has moreover been observed that a hydrostatic un- and reloading cycle supports the determination of the elastic parameters quite well in terms of the bulk modulus.

3. In order to estimate reliable data for Poisson's ratio, it is required that the triaxial tests are performed at constant confining pressure.

4. At least three drained triaxial tests must be performed over a wide range of confining pressures. The tests should include un- and reloading cycles after peak failure.

5. The hardening parameters should be estimated on the basis of the assumption of isotropic soil behaviour.

6. Finally, the newly described sinusoidal expression together with an initial slope of the softening curve equal to 0,8 provide significantly better predictions of the experimentally observed behaviour.

Taking these issues into account it is believed that it is generally possible to optimise the Single Hardening Model in order to provide reliable numerical predictions.

3. IMPLEMENTATION

Until this point, the constitutive behaviour defined by the Single Hardening Model has been employed for predictions of triaxial tests, exclusively. For use in a general problem, however, the Single Hardening Model must be linked with a finite element program. Therefore, the constitutive model has been implemented in a subroutine working together with the finite element program ABAQUS.

In order to ensure that reliable results are obtained when using the material model subroutine for actual calculations, the implementation has been verified. During the verification process it was experienced that the implemented version of the model was unable to handle certain situations in a proper manner. Thereby, the necessity of obtaining knowledge regarding the actual implementation arose. In this way, great emphasis has been laid on investigating the material model subroutine in order to solve the problems encountered during the verification process. As a particular interesting issue, the concept of strengthening of a material due to preshearing was investigated. In order to investigate the behaviour of the subroutine in the case of preshearing a triaxial specimen exposed to the following loading scheme has been analysed:

1. Isotropic consolidation to $\sigma_3' = 160$ kPa

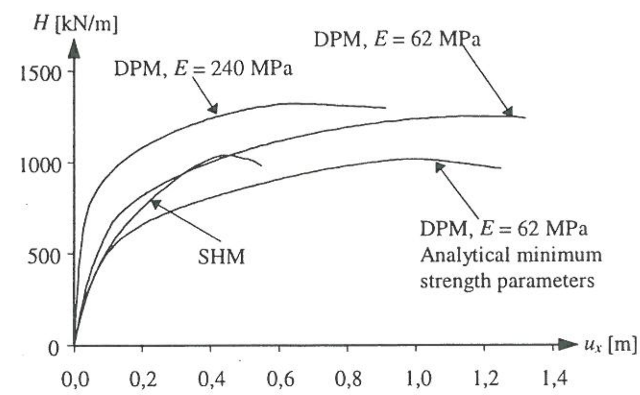

Figure 6. Appearance of the failure- and yield surface in the case of preshearing.

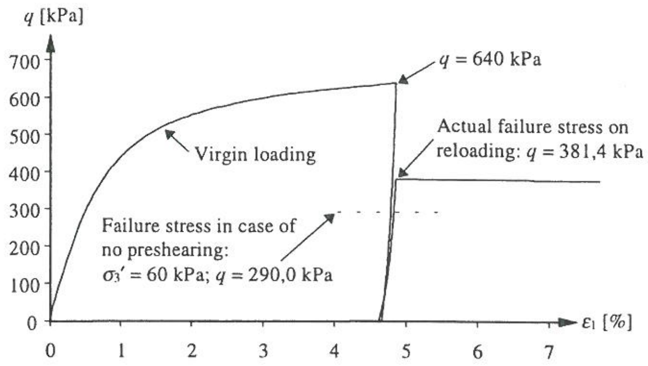

Figure 7. Stress-strain curve for triaxial specimen exposed to preshearing.

2. Triaxial compression to $q = 640$ kPa
3. Unloading to isotropic stress, $\sigma_3' = 160$ kPa
4. Decreasing confining pressure to $\sigma_3' = 60$ kPa
5. Reloading until peak failure is reached

The analysis is based on the parameters corresponding to Eastern Scheldt Sand with $e = 0{,}59$. Inserting the failure envelope parameters for this material into failure criterion, the peak value of deviator stress can be calculated to be: $q_{failure} = 661{,}2$ kPa, meaning that the original failure envelope is not reached during preshearing, as shown in Figure 6. However, the stress of 640 kPa is sufficient to cause the yield surface to be extended beyond the failure envelope, whereby a strengthening of the material when reloading at a lower confining pressure would be expected. In Figure 7 the resulting stress-strain curve from the preshearing analysis is presented.

It appeared that the implemented version of the Single Hardening Model is in fact capable of handling the concept of preshearing, although still some unanswered questions are present.

It has been observed that in the case of preshearing, the consistency condition, which is an essential part of an elasto-plastic formulation is no longer fulfilled. In order to solve this problem, intensive investigations of this particular issue must be carried out in the future.

4. ANALYSIS OF CAISSON BREAKWATER

On the basis of the verified version of the material model subroutine, the response of a gravity caisson breakwater has been subjected to a numerical analysis. As one out of several bases for comparison, the results from a centrifuge test performed at Delft Geotechnics have been brought in. In order to ensure reliable use of the test results, emphasis has been laid on acquiring knowledge regarding the special technique of centrifuge testing. The application of the caisson dead load in the caisson is carried out by accelerating the centrifuge to the chosen level of gravitational acceleration. This acceleration process has been compared with a corresponding numerical analysis, based on the Single Hardening Model. An observed deviation between the experimental and numerical behaviour has been put down to experimental uncertainties, together with an uncertain constitutive description at very low stress levels.

4.1 *Analytical analysis*

The drained bearing capacity due to horizontal loading of the caisson breakwater as predicted by the Single Hardening Model is compared with the bearing capacity obtained from an analytical analysis based on a kinematically admissible rupture figure. Furthermore, an analysis based on the bearing capacity formulation of Terzaghi has been carried out. A clear resemblance between the bearing capacity obtained on the basis of the Single Hardening Model and the analytically calculated bearing capacities has been experienced.

As an additional reference, predictions of the drained response of the caisson breakwater, based on the much simpler linear Drucker-Prager Model have been carried out. The employed version of the Drucker-Prager Model is the linear elastic, perfectly plastic model. It has been experienced that the response of the caisson is highly dependent of the used input parameters, as quite distinct results can be obtained, although the chosen parameters are considered to be relevant for the material in question. In this way, it must be stated that if the linear Drucker-Prager Model is to be used at all, it is essential to be able to provide a reasonable estimate on the stress state in the soil in order to apply the correct strength parameters for the analysis in question. The movements of the caisson predicted by the Single Hardening Model, and the Drucker-Prager Model, respectively, have been seen not to coincide. The predictions by the Drucker-Prager Model are consid-

Figure 8. Predictions of the horizontal loading, performed by the Single Hardening Model and the Drucker-Prager Model.

ered to describe the situation inadequately, due to the assumption of linear elastic, perfectly plastic soil behaviour.

4.2 *Numerical analysis*

A simplified section of the cylindrical container elucidates the model of the caisson breakwater used in the centrifuge test is shown in Figure 9.

The response of the caisson when exposed to load cycles of varying magnitude has been investigated by the Single Hardening Model as well. The dependency of the void ratio has been analysed. In both cases, the predicted response is considered reasonable.

During the centrifuge test, the caisson breakwater is exposed to several impact loads of a large magnitude, in order to simulate a storm situation. On the basis of the scaling factors, the impact loads reveals

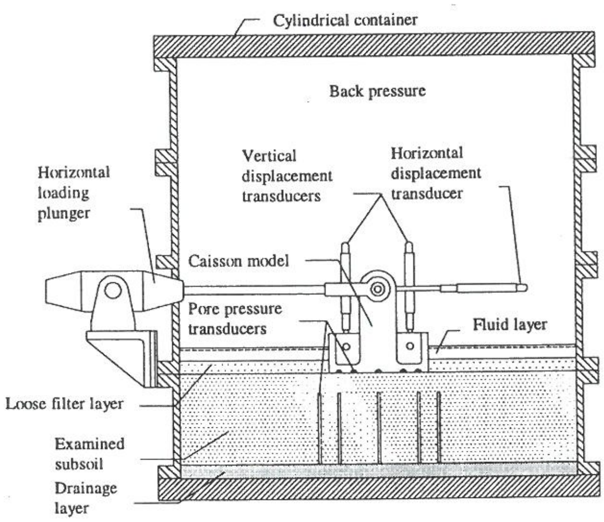

Figure 9. Section of the cylindrical container containing the test set-up of the gravity caisson breakwater in the centrifuge test. Loading connection and transducers are illustrated.

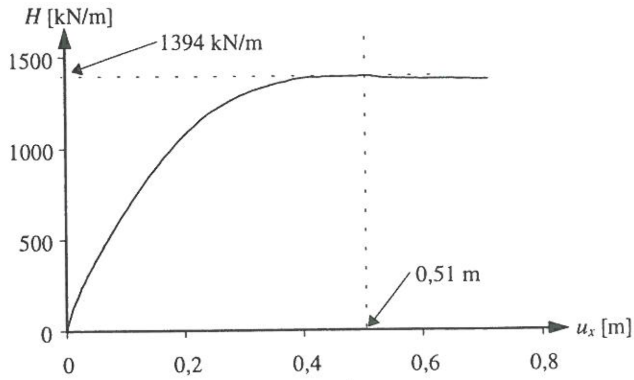

Figure 10. Performance curve for the undrained response of the caisson, predicted by the Single Hardening Model.

a wave impact force of H = 1440 kN/m for the prototype structure.

As an estimate on the response of the caisson in this situation, an undrained numerical analysis including the pore fluid response has been carried out by the Single Hardening Model. The Performance curve for the undrained response of the caisson, predicted by the Single Hardening Model is shown in Figure 10.

The horizontal bearing capacity obtained from this analysis is larger than the corresponding drained prediction, which is due to a negative pore pressure build-up in the soil layer. A nice correspondence between the numerically predicted bearing capacity, and the maximum force experienced in the centrifuge test has been observed.

4.3 *Failure mechanisms*

The failure mechanisms observed from the respective numerical analyses have formed the basis of a visual comparison. In all cases, a fully developed rupture zone, concentrated around the corner of the caisson, opposite to the load application point, is experienced. This behaviour is in correspondence with the rupture figure considered in connection with the analytical upper bound solution. Furthermore, the numerically predicted displacement fields are similar to the displacement field considered in the analytical analysis.

4.4 *Computational efficiency*

Regarding the computational efficiency of the respective numerical analyses, it should be mentioned that large differences have been experienced regarding the time needed for the analyses. In this way, the predictions based on the Single Hardening Model have been observed to require considerably more computational power than the corresponding Drucker-Prager analyses. It should be kept in mind, however, that due to the linear-elastic, perfectly plastic behaviour assumed by the Drucker-Prager Model, the number of calculations within each increment is small compared to the corresponding Single Hardening Model prediction, as this model includes hardening plasticity. Still, it would be preferable if it was possible to increase the computational efficiency of the subroutine containing the Single Hardening Model. Actually, this is considered to be a necessity if it is desired to be able to model a general three dimensional prototype structure. This statement is based on the fact that during the verification of the material model subroutine, the response of a simple shear test has been predicted by means of three dimensional elements. Even though the number of elements of the model is no higher than 400, the required computational time is extremely extensive. For modelling a general three dimensional structure, the required number of elements would be considerably higher. A possible point of effort regarding an improvement of the efficiency of the routine could be a change of the stress updating procedure, which is currently based on the forward Euler scheme.

5. CONCLUSION

Still, it must be emphasised that use of an advanced constitutive model like the Single Hardening Model is bound to require great efforts as regards to the calibration of the model, as well as the computational power needed for the actual analysis. Therefore, the relevance of applying such advanced model for solution of a geotechnical problem must be judged on the basis of the character of the problem considered. Hence, if e.g. only the drained bearing capacity of a given structure is desired, a simple analytical solution is considered to provide sufficiently precise results, meaning that in such cases application of a complex constitutive model is far from necessary. On the other hand, if the actual stress-strain response of the considered material is important to the results, as e.g. when the pore pressure build-up in the soil is essential, the use of a constitutive relation, which is able to capture the actual soil response is indeed relevant. On the basis of the analyses performed throughout this work, it is stated that the Single Hardening Model is at this point ready for application in such types of geotechnical problems.

6. REFERENCES

Andersen A. T, Madsen E.B, Schaarup-Jensen A.L, Ibsen L.B. 1997. *Constitutive Modelling of Baskarp Sand.* Research Report by Aalborg University, Geotechnical Engineering Group. *co-sponsored by* Commission of the European Union Directorate General XII MAST contract MAS3 - CT95 - 0041 (1996 - 1999). 200 pages report

Andersen A. T, Madsen E.B, Schaarup-Jensen A.L, Ibsen L.B, Jacobsen 1998.a. *Advanced Numerical Analysis of Caisson Breakwater Founded on Frictional Materials.* Research Report by Aalborg University, Geotechnical Engineering Group. *co- sponsored by* Commission of the European Union Directorate General XII MAST contract MAS3 - CT95 - 0041 (1996 - 1999). 300 pages report.

Andersen A. T, Madsen E.B, Schaarup-Jensen A.L 1998.b. *DATA REPORT 9701 Part 1. Easter Scheldt Sand, Baskarp Sand No.15.* Data Report by Aalborg University, Geotechnical Engineering Group. *co- sponsored by* Commission of the European Union Directorate General XII MAST contract MAS3 - CT95 - 0041 (1996 - 1999).

Ibsen L.B, 1998. *Dynamic Responce of Course Granular Material to Wave load.* Article in this report.