Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering– Rahman and Jaksa (Eds)

Evaluation of a practice-oriented nonlinear soil constitutive model for site response analyses on liquefiable soil deposits

Évaluation d'un modèle constitutif de sol non linéaire axé sur la pratique pour les analyses de réponse du site sur les dépôts de sol liquéfiables

Pascale C. Rousé
Itasca Chile SpA, Chile

Jenny Ramirez
Geosyntec Consultants, Columbia, Maryland, USA

Augusto Lucarelli
Itasca Consulting Group, Inc. Minneapolis, Minnesota, USA

ABSTRACT: Numerical simulations are becoming increasingly popular among practicing engineers to evaluate soil liquefaction and its destructive effects on the build environment. However, the selection of soil constitutive models and their calibration remain challenging even at element level conditions. In this paper, the results of a centrifuge experiment of a leveled, layered soil profile, including a liquefiable layer, are used to evaluate the predictive capabilities of an advanced, recent-developed, practice-oriented, nonlinear sand model: P2PSand. Simulations were performed in the finite-difference, 3-D, computer program FLAC3D. The soil model parameters were calibrated based on a series of triaxial tests. Results are shown as a direct comparisons of the numerical results with the element test data and centrifuge test results. This effort aims to highlight the capabilities and limitations of the P2PSand model in predicting key liquefaction consequences.

RÉSUMÉ : Les simulations numériques sont de plus en plus populaires parmi les ingénieurs en exercice pour évaluer la liquéfaction des sols et ses effets destructeurs sur l'environnement de construction. Cependant, la sélection des modèles constitutifs des sols et leur calibration restent difficiles, même dans des conditions au niveau des éléments. Dans cet article, les résultats d'une expérience de centrifuge d'un profil de sol nivelé en couches, y compris une couche liquéfiable, sont utilisés pour évaluer les capacités prédictives d'un modèle de sable non linéaire avancé, compatible avec l'état critique, orienté vers la pratique: P2PSand. Les simulations ont été effectuées dans le programme 3D à différences finies FLAC3D. Le paramètres du modèle du sol ont été calibrés sur la base d'une serie d'essais triaxiaux. Le résultats son montrés par comparison directes avec les résultats numériques des élements et les résultats de centirfuge. Cet effort vise à mettre en évidence les capacités et limitations du modèle P2PSand pour un prédiction fiable des conséquences clés de la liquéfaction.

KEYWORDS: Liquefaction, 3D numerical simulation, P2PSand

1 INTRODUCTION

Advanced constitutive models are increasingly used in engineering practice to numerically simulate the behavior of geotechnical structures on liquefiable soils (e.g., PDMY02 by Elgamal et al. (2002); SANISAND by Dafalias and Manzari; (2004); UBCSand by Beaty and Byrne (2011); PM4Sand by Boulanger and Ziotopoulous (2015); among others). In this paper, the recent-developed, practice-oriented, two-surface plastic constitutive model, P2PSand, is used. This model was developed to model sands for earthquake engineering applications such as the parameters are calibrated based on in-situ test data (e.g., relative density, standard penetration test results) within a reasonable calibration effort. This model is based on an existing critical state model developed by Dafalias and Popov (1975, 1976); therefore, it preserves the characteristic that a same set of parameters should capture soil behavior for different densities and confining stresses. The model retains the general three-dimensional formulation used in SANISAND and uses the Lode angle for the effect of the second principal stress. The model uses default parameters that are compatible to the cyclic resistance chart in the semi-empirical procedure but also allows for the user to assign customized parameters.

This paper compares the results of 3D numerical simulations with centrifuge experiments. The numerical simulations were conducted in the three-dimensional non-linear finite-difference program FLAC3D (Itasca, 2019) using the P2PSand soil constitutive model. On the absent of in-situ test data model parameters were calibrated against a series of monotonic and cyclic triaxial tests to evaluate the shortcomings and advantages or this approach. The predictive capabilities of the model in capturing the seismic site response of a layered soil model in term of accelerations, generation and distribution of excess pore pressure and resulting deformations are evaluated.

2 CENTRIFUGE MODEL

This paper uses the results of a centrifuge test performed at University of Colorado Boulder to compare the results obtained with the numerical simulations using P2PSand. This centrifuge test models a layered-leveled soil profile to evaluate liquefaction on free-field conditions and includes three layers, as shown in Figure 1. All dimensions in this figure are shown in prototype scale. The bottom layer is a dense Ottawa sand F65 (D_r≈90%) with a thickness of 12m. This dense sand is overlaid by a looser Ottawa Sand F65 with D_r≈40% and a thickness of 6m. On top of the soil profile, 2m of Monterey sand at a D_r≈90% is present. The initial soil properties of each layer are included in Table 1.

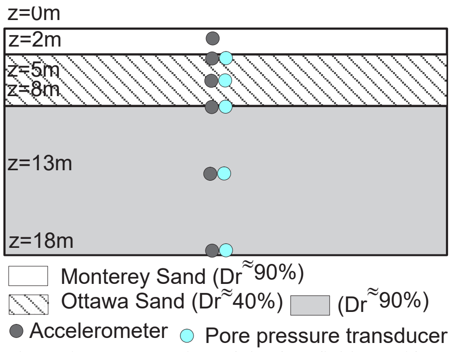

Figure 1. Schematic representation of the free field centrifuge test used in this study, including the instrumentation (after Ramirez et al., 2018)

Table 1. Initial soil properties in the centrifuge test

Layer	void ratio	γ_{sat} (kN/m^3)	K (m/s)
Monterey Sand (D_r≈90%)	0.570	19.81	$5.3\text{x}10^{-4}$
Ottawa sand (D_r≈40%)	0.698	19.05	$1.4\text{x}10^{-4}$
Ottawa sand (D_r≈90%)	0.557	19.89	$1.2\text{x}10^{-4}$

γ_{sat} : Saturated unit weight ; K ; Permeability

Soil specimen was prepared in a flexible-shear-beam container and fully saturated with a solution of a methylcellulose in water (Stewart et al. 1994) under a vacuum with a viscosity of 70 times that of water. Subsequently, the container was spun to 70g of centrifugal acceleration. While the model in fly, a series of 1D horizontal earthquake motions were applied to the base of the model. This paper only focuses on the results of the first significant earthquake, namely Kobe-L, which corresponds to the horizontal component of the 1995 Kobe earthquake registered in Japan. Figure 2 shows the acceleration time history and the acceleration response spectrum (5%-damped) of the Kobe-L motion, with a peak ground acceleration, PGA, of 0.41 gravity (g). Figure 3 shows the accelerations Fast Fourier Transform, where the relevant maximum frequency is smaller than 10 Hz. This frequency (f_{max}) was used to select the maximum allowable element size, h_{max} (where $h_{max}=V_s/(16f_{max})$ using a maximum V_s equal to 80 m/s (Ramirez et al. 2018). As a result, the maximum height of elements for the two top upper layer is 0.5 m while it varies between 1 to 1.3 m for the dense layer Test results were obtained at the boundaries of between layers and at their mid-depths using accelerometers, pore pressure transducers, and linear variable displacement transducers (LVDTs) (See Figure 1).

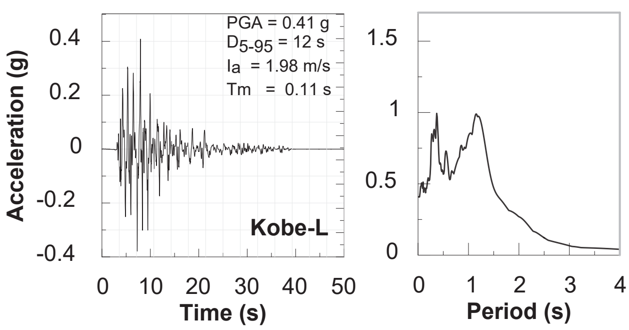

Figure 2. Acceleration time history and response spectrum (5%-damped) of the model input motion (Kobe-L)

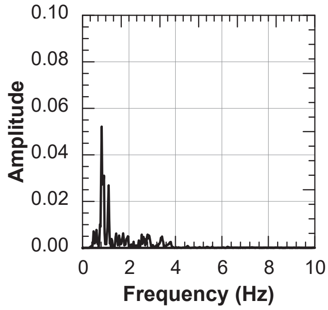

Figure 3: Accelerations Fast Fourier Transform

3 NUMERICAL MODEL

3.1 *P2PSand Constitutive Model*

P2PSand was developed by (Cheng, 2018) and it mainly requires ten material parameters. Three of them are related to elasticity properties, four parameters depend upon the critical state, and the rest are the reference pressure (usually taken as 100kPa) and the minimum and maximum void ratios. The other parameters can be calibrated against monotonic and cyclic tests or can be taken as the default values based on the relative density.

The elastic law is based on the formulation proposed by Dafalias and Manzari (2004). Therefore, the shear modulus is based on the current mean pressure (p) and modified in terms of relative density as indicated in Equation 1.

$$G = G_r P_{atm}\left(\frac{p}{P_{atm}}\right)^{0.5} = g_0(D_r + C_{Dr})P_{atm}\left(\frac{p}{P_{atm}}\right)^{0.5} \tag{1}$$

Where G_r is a density-dependent material parameter as a function of g_0 and C_{Dr}, with default values of 1240 and 0.01, respectively. A high correlation is observed for both laboratory tests and in-situ data for values of D_r between 15 and 85%.

The critical state line is similarly defined by using the relative density as follows:

$$D_{rc} = D_{rc0} + \lambda_r\left(\frac{p}{P_{atm}}\right)^{\xi} \tag{2}$$

where D_{rc0} is the critical state relative density at the reference pressure, λ_r is the slope of the critical state line in the $Dr - logp'$ plot, and ξ is a material parameter usually equal to 0.7 for most sands.

The Lode Angle dependency of P2PSand adopts a generalized function (Van Eekelen, 1980) as:

$$g(\theta, c) = \left[\frac{1+c^{1/z}+(1-c^{1/z})cos3\theta}{2}\right]^{z} \tag{3}$$

where θ is the lode angle, c is the ratio of the triaxial compression strength to the extension strength and z = -0.25.

The bounding surface is defined from Lashkari (2009) as follows:

$$M^b = gM^c(1 + n^b I_r) \tag{4}$$

where $M^c = 6sin\phi_{cs}/(3 - sin\phi_{cs})$ and n^b is a material parameter.

The dilatancy surface has the form of:

$$M^d = gM^c(1 - n^d I_r) \tag{5}$$

where n^d is 4 to 6 times n^b.

An adjusting parameter for cycling loading or non-backbone loading path, K_c, is used to capture the sand characteristic that the dilation/contraction evolution rate is slightly different between a virgin loading and a cycling loading. This parameter should be calibrated last and it is defined as follows.

$$K_c = a_0 + a_1 D_r + a_2 (D_r)^2, \tag{6}$$

where a_0=3.8, a_1= -7.2, and a_2=3.0 are the default values.

3.2 P2PSand Calibration

P2PSand model was developed for doing calibration against in-situ conditions; however, in this paper the calibration was based on strain-controlled drained and undrained monotonic and cyclic triaxial compression tests at different confining pressures and relative densities of 40, 60 ad 90%. This approach was still considered insightful to evaluate the shortcomings and limitations when using laboratory data. Elastic and critical state parameters were obtained from the calibration process from Ramirez et al. (2018) and therefore, only the parameters n^b and n^d were calibrated on this paper. Figure 4 shows the comparison for the drained tests at a confining pressure of 200 kPa and relatives used in the centrifuge tests, 40% and 90%. The deviatoric stress and maximum and rate of volumetric strain is well captured, particularly for the lowest relative density.

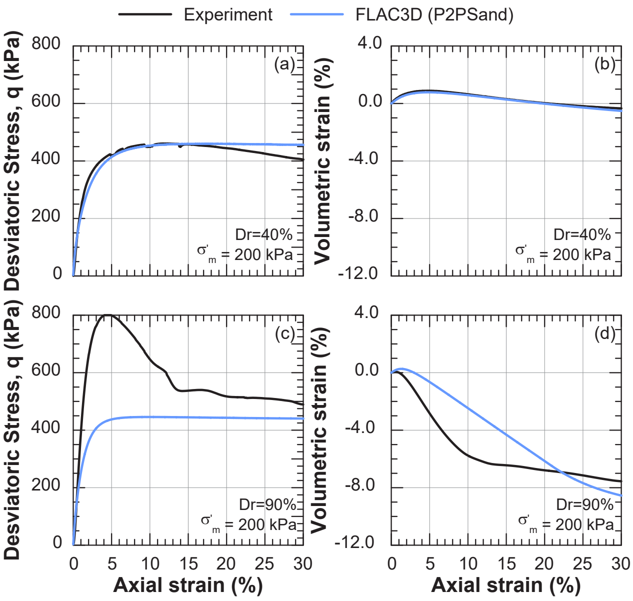

Figure 4. Comparison of measured and simulated monotonic triaxial test data.

The dynamic parameters were calibrated against strain-controlled cyclic triaxial tests at a confining pressure of 100 kPa, relative densities of 40, 60 and 90%, and axial strains between 0.042 and 0.44%. Figure 5 shows the comparison of the numerical simulation of a cyclic triaxial test and its corresponding laboratory test both for a medium dense sand (Dr ≈ 60%). The numerical simulation is able to capture the initial shear strength module and the maximum deviatoric stress. It is noted that liquefaction on the numerical models is achieved faster than observed on the laboratory tests.

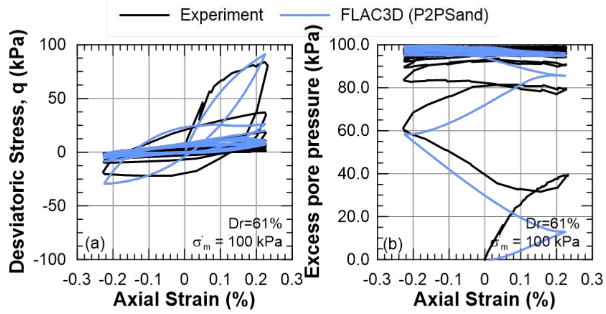

Figure 5. Comparison of measured and simulated cyclic triaxial test data.

Since no laboratory data was available for the calibration of Monterey sand, the same parameters of dense Ottawa Sand were assigned. Because Monterey sand does not control the soil deposit response, this approach was considered appropriate. A single set of calibrated parameters was chosen to have a relatively good match with the element test laboratory data. The parameters used for the element test calibration are shown in Table 2. The calibration of the triaxial tests at a relative density of 90% was performed for a D_r of 85% because P2PSand was developed for materials with a maximum relative density of 85%. This constrain affected the predictions on the bottom dense sand on the site response analysis, as explained in the next section.

Table 2. P2PSand calibrated parameters

Parameter	*Value*	*Reference*
Elasticity, G_r	130	
Poisson's ratio	0.33	
Critical state 1, D_{rc0}	0.142	
Critical state 2, λ_c	0.10	
Critical state 3, ξ	0.70	Ramirez et al. 2018
Critical friction angle, φ_{cv}	31.5	
Maximum void ratio	0.82	
Minimum void ratio	0.53	
Bounding coefficient, η^b	0.01	
Dilatancy coefficient, η^d	0.85	Calibrated in this paper
Cycling factor, K_c (a_0, a_1, a_2)*	7.0	

*Only varied during the site response analysis (See next section)

3.3 Prediction Model

The centrifuge test was simulated as a fully-coupled three dimensional (3D) column using the three-dimensional non-linear finite-difference program FLAC3D (Itasca, 2019). The height of the soil specimen was 18 m and was discretized with ten 1.0-m height elements for dense Ottawa Sand at the bottom of the deposit, twelve 0.5-m height elements for loose liquefiable Ottawa Sand, and four 0.5-m height elements for dense Monterey Sand at the top of the soil specimen. The element heights were equal or less the maximum calculated height. The finite difference zone height was defined so it allows the correct transmission of the seismic waves of the analysis.

Ottawa and Monterey Sand were represented by the P2PSand constitutive model using the parameter shown in Table 2. The hydraulic conductivity for each layer includes the values shown in Table 1. As a sensitivity analysis, the impact of K_c on the site response analysis was evaluated. Therefore, two numerical results are presented: (a) with K_c=7.0, based on calibration, and (b) K_c by default, based on FLAC3D manual recommendations. The rate-plastic-shear and volume was used lower than default values keeping a ratio of 1/3 as 0.15 ad 0.05, respectively.

The bottom boundary was modeled as rigid and the sides were modeled as free field. The seismic record (shown in Figure 2) was applied at the rigid base of the model, and a Maxwell damping, which is independent of frequency (Dawson and Cheng, 2021), of 5.2% was used. The top boundary considers drained conditions while all interior nodes had the capability to accumulate pore water pressures. Monitoring points were established in the soil sample at the top and bottom of liquefiable layer as well as the middle of the bottom dense layer. Excess pore pressure time histories are presented at the boundaries between layers and at their mid-depth.

Figure 6 shows a comparison between the acceleration time histories of both centrifuge and numerical model. The peak ground accelerations are relatively well captured; however, the model is not able to capture spectral accelerations at larger periods. It is possible the selected damping is negatively impacting this match. As shown in Figure 7, excess pore pressures are overall well captured by the numerical model, particularly within the liquefiable layer. However, the pore pressure generation and dissipation rates obtained with the numerical model show notorious differences, especially in the dense Ottawa Sand. Settlements at the upper layer are shown in Figure 8. The difficulties in capturing volumetric strains have been reported when using other models. For instance, Ramirez et al. 2018 reports almost no settlements when modeling the same soil column with two different constitutive model. In this paper, even though experimental settlement could not be fully captured, this constitutive model is able to partially reproduce these volumetric deformations, particularly until the maximum pore pressure is reached.

The impact of using a value of K_c = 7 instead of its default value given by $K_c = a_0 + a_1 D_r + a_2 (D_r)^2$, where a_0 =3.8, a_1= -7.2 and a_2=3.0 is also examined in Figures 6 through 8. A slightly better match is obtained for the acceleration time history and settlements, when using a value of K_c = 7. Little influence of the K_c on the excess pore pressure is observed.

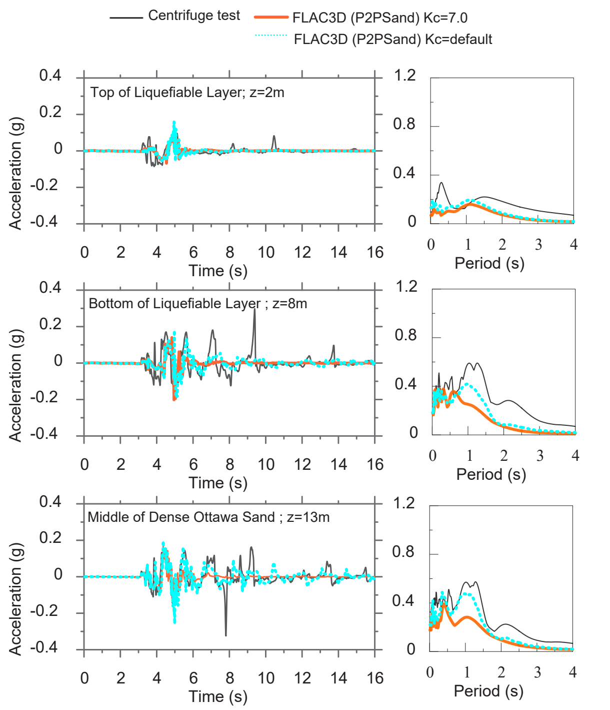

Figure 6. Comparison between the experimental and numerical results in terms of acceleration time histories

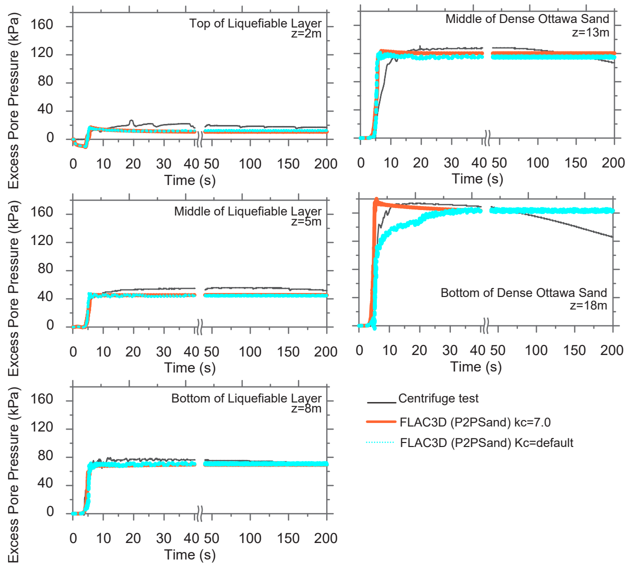

Figure 7. Comparison between the experimental and numerical excess pore pressure at different depths

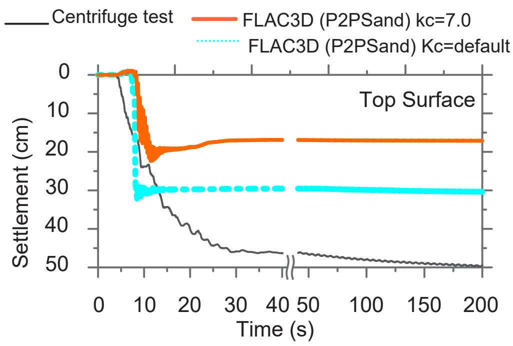

Figure 8. Comparison between the experimental and numerical settlement at the top of the soil column

4 SUMMARY AND CONCLUSIONS

Monotonic and cyclic triaxial tests on Ottawa sand were used to calibrate the in-situ based constitutive model P2PSand. Elastic and critical state parameters were obtained from Ramirez et al. (2018) and the bounding and dilatancy coefficient were calibrated in this study. In addition, the effect of the cyclic factor K_c. was investigated in more details by looking at the impact on the site response prediction.

Because the P2PSand model was developed for doing calibration against in-situ conditions, the comparison with laboratory test data is hardly comparing, especially for larger relative densities. P2P is not suitable to model dense to very dense sand, as the maximum dense that is accepted in the model parameters is 85%.

Usually, the structure of the soil for freshly pluviated material (as modeled in laboratory) is different from the one observed in-situ. From the authors experience, under a modelling point of view, equivalent relative density could be in the order of 20% less than the laboratory relative density, especially for values higher than 75-80%.

Overall P2PSand was able to reproduce partially volumetric settlements, which represents an important aspect when evaluating liquefaction. However, spectral accelerations and pore water pressure generation are not well captured along the column. It is hypothesized that because the model is comprised by a thick dense sand at the bottom that is not able to be reproduced during calibration, accelerations are not well propagated to the surface.

5 ACKNOWLEDGEMENTS

This research was developed from the second author's PhD research at the University of Colorado Boulder.

6 REFERENCES

Beaty, M.H., and Byrne, P.M. 2011. UBCSAND Constitutive model, Version 904aR. Itasca UDM Web Site.

Boulanger, R.W., Montgomery, J., and Ziotopoulou, K. 2015. Nonlinear deformation analyses of liquefaction effects on embankment dams. *Geotechnical and Geological Earthquake Engineering*, 37, 247-283.

Cheng, Z., and Detournay, C. 2021. Formulation, validation and application of a practice-oriented two surface plasticity sand model. Computers and Geotechnics, 132, 1-24.

Dafalias, Y.F., and Manzari, M.T. 2004. Simple plasticity sand model accounting for fabric change effects. *Journal of Engineering Mechanics* 130, 622-634.

Dafalias, Y.F., and Popov, E.P. 1975. A model of nonlinearly hardening materials for complex loading. *Acta Mechanica*, 21, 173-192.

Dafalias, Y.F., and Popov, E.P. 1976. Plastic internal variables formalism of cyclic plasticity. *Journal of Applied Mechanics ASME*, 43, 645-651.

Dawson, E.M. and Cheng, Z. 2021. Maxwell Damping, an Alternative to Rayleigh Damping, Draft paper for Geo-Extreme Conference, Savanah, Georgia, August 15-18.

Elgamal, A., Yankg, Z., and Parra, E. 2002. Computational modeling of cyclic mobility an post-liquefaction site response. *Soil Dynamics and Earthquake Engineering* 22, 259-271.

Kuhlemeyer, T.L., and Lysmer J. 1973. Finite element method accuracy for wave propagation problems. *Journal of Soil Mechanics and Foundations*, Div. ASCE, 99(SM5), 421-427.

Lashkari, A. 2009. On the modeling of state dependency of granular soils. *Computers and Geotechnics*. 36, 1237-1245.

Ramirez, J., Barrero, A.R., Chen, L., Dashtu, S., Ghofrani, A. Taiebat, M., and Arduino, P. 2018. Site response in a layered liquefiable deposit: evaluation of different numerical tools and methodologies with centrifuge experimental results. Journal of Geotechnical and Geoenvironmental Engineering, 144, 2-22.

Van Eekelen, H.A.M. 1980. Isotrpoic yield surfaces in three dimensions for use in soil mechanics. *International Journal of Numerical and Analytical Methods in Geomechanics*, 4, 89-101.