FLOW LIQUEFACTION SIMULATION USING A COMBINED EFFECTIVE STRESS – TOTAL STRESS MODEL

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ABSTRACT
Pore water redistribution and soil mixing can give post-liquefaction strengths much lower than those from undrained laboratory tests. Fully coupled effective stress numerical analyses have given important insight into the process but further development and calibration is needed. In the meantime a combined effective stress – total stress analysis is recommended. In the combined procedure the effective stress analysis is used to determine zones of liquefaction and deformations that occur during strong shaking. Following strong shaking liquefied elements are changed to a total stress model with a specified residual strength that is back-calculated from case histories.

RÉSUMÉ
La redistribution des pressions d’eau interstitielles et le mélange de sols granuleux donnera résistances après-liquefaction beaucoup plus basses que celles des essais de laboratoire non drainés. Des analyses numériques entièrement couplées d’effort effectif ont donné un éclairage important dans le procès mais autres développements et calibrations sont nécessaires. En attendant, une analyse combinée d’effort effectif – effort totale est recommandée. Dans cette procédure combinée l’analyse d’effort effectif est utilisée pour déterminer des zones de liquéfaction et déformations qui se produisent pendant la secousse. Après la secousse les éléments liquéfiés sont changés en modèle total d’effort avec une résistance résiduelle spécifiée qui est retro-calculée à partir des antécédents.

1 INTRODUCTION
Numerous flow and bearing failures have occurred during or following strong earthquake shaking when liquefaction is triggered (Ishihara 1984, Kokusho 2003, Hamada 1992, Seed et al. 1987, Poulos et al. 1985). These flow liquefaction failures are deemed to occur when the static driving stress exceeds the soil shear strength. Shear strength of soil following triggering of liquefaction has been called residual or liquefied strength, and has commonly been assumed to be an undrained strength parameter. Seed (1987) back-calculated these strengths from case histories while others (Poulos et al. 1985) attempted to determine them from laboratory tests. Recent work (Vaid & Eliadorani 1998, Yoshida & Finn 2000, Kokusho 1999, Kokusho 2003, Kulasingam et al. 2004, Sento et al. 2004, Seid-Karbasi & Byrne 2004, Naesgaard et al. 2005; Malvick 2005, and Naesgaard et al. 2006) has shown that the undrained assumption is often not correct and may be unconservative. They have demonstrated that pore water flow and pressure redistribution which occurs during and following earthquake shaking may result in relatively thin zones with very high void ratio, or in the extreme, water inter-layers, below low permeability layers. These high void ratio/water inter-layer zones have very low to near zero shear strength, much lower than that obtained from undrained laboratory element tests. Without a low permeability barrier to retard the escape of groundwater, and/or some form of soil mixing (Naesgaard & Byrne 2005, Vasquez-Herrera & Doby 1989), flow liquefaction generally does not occur, even for loose sands on relatively steep slopes subjected to strong shaking.

Fully coupled effective stress numerical analyses procedures developed at the University of British Columbia have been used to emulate field and centrifuge test case histories and have given important insight into liquefied strength. The difficulties with the fully coupled effective stress analyses are that (1) the results are dependent on stratigraphy details that often are not well known and often on a scale smaller than the size of the smallest practical element in the numerical model, (2) localization occurs below the low permeability barriers and the behaviour becomes element size dependent (Naesgaard et al. 2005, Yang & Elgamal 2002), (3) mixing of layered soils often occurs during liquefaction and quantification of the effects is difficult and (4) the effects of out-of-plane shaking are not considered in the current two-dimensional analyses. Perhaps the coupled effective stress analyses alone will be able to fulfill design requirements with further development and calibration; however, it is currently recommended that an alternative total-stress analysis should also be carried out in conjunction with the coupled effective stress analysis to overcome the shortcomings discussed above. In the proposed effective-stress/total-stress approach the residual strength that is back-calculated from case histories (similar to those originally proposed by H. Seed and colleagues (Seed 1987)) is introduced into the process. In this manner the empirical experience used in current design practice is included in the process.
When typical loose sand, relative density \( D_r = 40\% \), is tested in drained (dry) cyclic simple shear it is initially contractive on loading and unloading. However, on loading to large strains the stress state exceeds the constant volume friction angle \( (\Phi_{cv}) \) and the soil becomes dilative. Dense sand behaves in a similar manner except the dilative response is more pronounced. Both dense and loose sands are contractive on unloading. The net result of cyclic loading is generally a reduction in sample volume.

If the pores are filled with water that is prevented from escaping (undrained condition) then pore pressures will increase when the soil skeleton attempts to contract, and decrease when the soil skeleton attempts to expand (dilate). With repeated cycles the stress path may reach the zero effective stress / zero shear strength origin and true liquefaction occurs. However, with continued monotonic shearing to large strains the soil will attempt to dilate and gain strength. The residual strength will not be reached until, (i) the pore-water cavitates and thus allows the sample to increase in volume and reach the steady state, or (ii) the high mean effective stress generated by dilation suppresses the dilation and the soil reaches its critical state strength, or (iii) the sand grains crush and the soil reaches a critical state of the crushed material. The strength of the sand reached in (i), (ii) or (iii) is generally much higher than the commonly accepted ‘undrained’ residual strengths back-calculated from case histories and is likely higher than the drained strength.

If, in lieu of undrained loading, a small inflow of water (expansion) happens then shear can occur without strength gain. Upon reaching the critical state further inflow will cause total loss of strength (Vaid & Eliadorani 1998, Boulanger & Truman 1996, Sento et al. 2004). This process of void redistribution was originally proposed by Whitman (1985).

Natural and many man-made soils are often layered and have variations in grain-size and permeability. Earthquake shaking and related liquefaction will induce pore water gradients and flow that will result in contraction of the soil skeleton with outflow of pore water in some areas and expansion with inflow of pore water in other areas. Inflow gives soil strengths considerably lower than that obtained assuming undrained conditions. When there is a low permeability layer or barrier, the upward migrating pore water gets trapped beneath the barrier and forms a layer of local expansion with low effective stress. At the limit, an actual water interlayer will develop. In embankments and water-edge slopes, liquefaction within the center portion of embankments or back from a water-edge slope will induce pore pressures equal to the local overburden pressure. When this high pressure water migrates laterally toward the edges of the embankment or toward the water-edge slope (location of lower overburden pressure), expansion and potential for strength reduction will also occur. Since water flow and pressure redistribution takes time there is often a delay between time of earthquake shaking and time of flow failure. There are numerous case histories where soil liquefaction occurred during earthquake shaking but related flow failure did not occur until some time after end of shaking. The classical examples are the Lower San Fernando Dam where the upstream face of the dam failed 20 to 30s after end of earthquake shaking (Seed, 1973, Seed, 1987) and in Niigata where eyewitnesses reported that the girders of the Showa Ohashi Bridge fell a few minutes after the earthquake motion had ceased (Hamada, 1992).

Numerical analyses have shown that the blockage effect on flow caused by the presence of the barrier causes the highest rate of expansion to occur directly beneath the barrier. This is important as this can cause localization and formation of a thin very weak layer at the interface. The expansion is believed to be mainly due to the influx of water rather than from shear induced dilation. If the zone directly under the barrier is thought of as a very thin soil element, with plane boundaries, then an extremely small influx of pore water will cause a large volumetric expansive strain under the barrier and may result in the element going to the critical state at zero effective stress. Further inflow will cause localization and formation of a water film with zero effective stress and zero shear strength. Further shearing does not induce dilation. The strength will remain zero until the water film drains.

With a static shear bias, the expanding zone under the barrier will attempt to fail prior to reaching the critical state and zero strength. This will result in a large shear strain that is a function of inflow volume and soil density (Boulanger and Truman 1996, Sento et al. 2004). However, with continued inflow the layer eventually reaches the critical state at a shear stress corresponding to the static bias and any further inflow will lead to a flow slide.

In real soils the barrier boundary will not be perfectly plane, infinitely thin or of infinite lateral extent but will have undulations, varying normal total stresses, finite grain sizes, varying permeability, etc. (Naesgaard et al. 2005). Earthquake duration and slope geometry may also affect the residual strength. These items will result in the liquefied (residual) shear strength along the interface varying both in time and space with an average value that is greater than zero. This ‘average value’ is the residual shear strength that is believed to be reflected in the low values back-calculated from case histories.

3 SOIL MIXING

Another procedure that may result in high void ratios, excess pore water and liquefaction is soil mixing (Naesgaard & Byrne 2005, Vasquez-Herrera & Doby 1989). Natural water-pluviated soils are often layered. When layers of coarse and fine grained soils are mixed together in an undrained or near-undrained environment than the resulting post-liquefaction shear strength of the mixed soil is much lower than the shear strength of the individual layers. This is illustrated in Fig. 1 and 2.
During the soil liquefaction and post-liquefaction shear process there are numerous occasions during which extensive turbulence and mixing of particles occurs. This has the potential for creating zones with post-liquefaction strengths well below those that would be obtained from undrained laboratory tests on the individual non-mixed layers.

4 RESIDUAL STRENGTHS FROM BACK-ANALYSIS OF FIELD CASE HISTORIES

Field experience during past earthquakes indicates that residual strengths can be much lower than values obtained from undrained laboratory tests on undisturbed samples. This is postulated to be due to the presence of low permeability barriers and soil mixing. Based on back-analysis of field case histories, Seed and Harder (1990), Olson and Stark (2002), Idriss and Boulanger (2007) and others have proposed residual shear strength (S_r) or strength ratio (S_r/\sigma_v') for liquefied soil as a function of either SPT (N_1)_{60} or normalized CPT tip resistance q_{c1}. The recent procedure by Idriss and Boulanger (2007) includes different relationships for cases with and without low permeability barriers (Fig.3) and has been used in the example analysis in this paper.

Back-calculating post-liquefaction ‘residual’ strength from case histories and treating it as an “undrained” soil strength property that is a function of penetration resistance or soil density is a gross simplification with a large scatter in available data (Fig. 3). However at this time this may still be the best strength data available.

Figure 1. ‘A’ is prior to mixing, ‘B’ & ‘C’ are two alternative mixed configurations. ‘A’ has significantly higher residual strength than ‘B’ & ‘C’. From Naesgaard & Byrne (2005).

Figure 2. Post-liquefaction strengths of individual sandy SILT and silty SAND layers are much higher than the post-liquefaction strengths of a mixture of the two soils. From Naesgaard & Byrne (2005).

Figure 3. Residual strength ratio (S_r/\sigma_v') of liquefied soil versus equivalent-clean-sand- corrected SPT (N_1)_{60}, for \sigma_v' < 400 kPa. From Idriss & Boulanger (2007).

5 PROPOSED COMBINED EFFECTIVE STRESS – TOTAL STRESS APPROACH

Fully coupled effective stress numerical analyses can model the triggering of liquefaction, the cyclic mobility deformations that occur during shaking, and emulate the pore pressure migration and trapping of water under low permeability barriers as observed in field and centrifuge test case histories (Byrne et al. 2006). One of the difficulties with the fully coupled effective stress numerical modelling is that localization occurs below the low permeability barriers and the behaviour becomes element size dependent (Yang & Elgamal 2002, Seid-Karbasi & Byrne 2004, Naesgaard et al. 2005). Using the very small elements necessary to correctly simulate the behaviour is not practical due to time and memory constraints. Large elements can be made to behave like small elements by curtailing dilation in the large element at a void ratio that is considerably less than that calculated from critical state theory (Naesgaard et al. 2005). However, it is difficult to know if the coupled effective stress model is correctly capturing the behaviour and further work in calibrating the model is required. As a tool for use in current design practice it is proposed to augment the coupled effective stress analyses with a total stress analysis which uses liquefied (residual) strengths back-calculated from case histories.

The proposed numerical process is as follows:

1. The model is built and brought to static equilibrium.
2. A dynamic coupled effective stress analysis is carried out to the end of strong earthquake shaking.
During the analyses zones where liquefaction has been triggered (pore pressure ratio $R_u (u/\sigma'_v) > 0.70$ to 1.0) are tracked. Following end of strong earthquake shaking a data file is saved and analyses are continued as two separate cases (3a & 3b).

(3a) In one of the analysis branches the coupled effective stress analysis is continued to allow further pore pressure redistribution to occur and to bring insight into the related localization and potential post-liquefaction flow.

(3b) In the other analysis branch, elements within the post-shaking model that have liquefied during the first phase are changed to a total stress constitutive model with a ‘residual’ strength and the analysis is continued until static equilibrium is reached. The proposed post-liquefaction residual strength is that back-calculated from case histories.

(4) The results from both the coupled effective-stress analysis and the combined effective stress/total-stress analysis should be considered for design.

A flow chart illustrating the procedure is given in Fig. 4.

5.1 Coupled Effective Stress Analysis

A coupled effective stress constitutive model (UBCSAND) and analyses procedure for modelling earthquake shaking and liquefaction has been developed at the University of British Columbia. UBCSAND is an elasto-plastic effective stress model with the mechanical behaviour of the sand skeleton and pore water flow fully coupled (Beaty & Byrne 1998; Byrne et al. 2004). The model includes a yield surface related to the developed friction angle, non-associative flow rule, and definitions for loading, unloading, and hardening. Key elastic and plastic parameters used are adjusted so as to give a good match with simple shear laboratory tests as the loading path of this test, including rotation of principal stress axes, closely approximates that which occurs during earthquake loading. The UBCSAND constitutive model is run within the finite difference program FLAC 5.0 (ITASCA 2005). In the FLAC program dynamic analyses are carried out in the time domain with full coupling between groundwater flow and mechanical loading.

The UBCSAND model has been calibrated against simple shear laboratory tests, centrifuge tests with and without impermeable silt barriers (Yang et al. 2004, Phillips et al. 2005, Seid-Karbas et al. 2005, Byrne & Park 2005) and the empirical liquefaction triggering charts (Idriss & Boulanger 2007). The model is able to emulate both the observed drained behaviour of loose sand soils (Contraction when sheared below the $\Phi_{cv}$ and dilative above the $\Phi_{cv}$) and the build-up of pore pressure and soil liquefaction that occurs in undrained simple shear tests. The model is also able to emulate the behaviour of a flow failure when a low permeability barrier was present and no failure when the barrier was absent or when drains were installed through the barrier (Byrne et al. 2006, Naesgaard et al. 2005) and able to emulate the post-shaking failure of the Lower San Fernando Dam (Naesgaard et al. 2006). Atigh & Byrne, 2004, showed that UBCSAND could emulate the behaviour of the triaxial tests with fluid inflow carried out by Vaid & Eliadorni, 1998.

As mentioned, pore pressure redistribution in soil profiles with low permeability barriers can cause localization below the barrier and the analyses results become element size dependent. Having very small elements in the model is not practical due to analysis time constraints. To counter this effect, large elements are made to behave like smaller elements by setting dilation to zero or near zero in elements that are expanding due to water inflow. This change is made at the end-of-strong shaking so the calibrated shear-induced liquefaction triggering behaviour is not affected.

5.2 Post-liquefaction Total Stress Analysis

The pore pressure ratio $R_u$ is monitored during the coupled effective stress analysis and the maximum value in each element is retained. At the end of strong shaking, elements with maximum $R_u$ greater than a value around 0.7 to 1.0 are deemed to have liquefied and are changed to a Mohr Coulomb constitutive model with a friction and dilation angle of zero, and cohesion set equal to the residual strength $S_r$. The $S_r$ is that back-calculated from case histories as proposed by Seed & Harder (1990), Olson & Stark 2002, and Idriss & Boulanger 2007. The shear modulus, bulk modulus, density, porosity and permeability in the liquefied elements are set equal to the

Figure 4. Flow chart showing combined coupled effective stress / total stress analyses procedure.
values present at the end of strong shaking (end of phase 2) in the initial coupled effective stress analysis.

6 LOWER SAN FERNANDO DAM - EXAMPLE ANALYSIS

In 1971 the Lower San Fernando Dam (LSFD) was shaken by a large earthquake with a peak velocity pulse of around 0.6 m/s and peak ground acceleration of approximately 0.5 g (Seed 1973; Seed et al. 1989, Castro et al. 1989, Castro 1995). Approximately 20 to 30 s after the end of the earthquake shaking the upstream face of the dam catastrophically failed leaving only 1.5 m of freeboard and putting a large population at risk. Extensive investigation and analyses of the dam were conducted following the event. From the studies it was concluded that the hydraulic fill soil within lower and central portions of the dam had liquefied and overlying portions of the dam had flowed out riding on the liquefied soil (Fig. 5).

In the current example the liquefiable portions of hydraulic fill soils were given ($N_{100}$) values of 12 while the clayey core of the dam was given undrained shear strength of 18% of the initial vertical overburden pressure (Fig. 6). The UBCSAND constitutive model was used for the potentially liquefiable sandy soil portions of the dam while a Mohr Coulomb model was used for the clayey core and portions of the dam above the water table. The effective stress analyses were fully-coupled (mechanical - pore water flow) and run within the program FLAC.

In the actual dam the spacing of the low permeability layers is much finer than that in the numerical model. Limitations on smallest element size are necessary for analysis time to be in a practical range and therefore the behaviour of finely spaced layers must be emulated with larger elements by introducing a dilation cut-off in any elements expanding beyond their original volume.

6.1 Coupled Effective Stress Analysis of LSFD – Phases 2 & 3a

In the example LSFD analysis the coupled effective stress analysis was carried out using the UBCSAND constitutive model within the program FLAC as described in section 5.1 for all saturated sand elements while a total stress Mohr Coulomb model was used for the clay/silt core portion of the dam.

Initially the whole model was brought to static equilibrium using a Mohr Coulomb model and drained stiffness parameters (phase 1). Then prior to earthquake shaking the sand elements were changed to the UBCSAND model while portions of the clay/silt core were left with a Mohr Coulomb model but given undrained stiffness parameters. The model is then brought to equilibrium in dynamic mode and all displacements reset to zero (end of phase 1). Following this the dynamic analysis with earthquake motion input at the base was run until excessive grid distortion stopped the analysis (about 40 s). This constitutes phase 2 and 3a in Fig. 4. A file was saved at after end of strong shaking (15 s) for use in the phase 3b Total Stress alternate analysis.

![Figure 5. Lower San Fernando Dam which failed into the reservoir approximately 30s after end of earthquake shaking. Black areas are soil that was mixed and deemed to have liquefied. Lower figure is reconstructed profile which shows location of liquefied soil before failure. (Seed et al. 1973).](image-url)
6.2 Post-Liquefaction Total Stress Analysis of LSFD – Phase 3b

Elements which had liquefied (R_u > 0.7) during the strong shaking portion of the analysis (Phase 2) where changed to a Mohr Coulomb model with zero friction angle, zero dilation, and a cohesion set equal to a residual strength (S_r) calculated using the initial pre-shaking effective stress (p') and the S_r/p' ratio recommended by Idriss and Boulanger (2007) (Fig. 3). The dynamic analysis was then continued until stopped by excessive grid distortion (≈ 18 s).

6.3 Results of LSFD example analysis

During strong shaking (phase 2) the upstream shell moved approximately 3 m upstream and the downstream shell moved approximately 1.5 m downstream. Liquefaction occurred both within the upstream and downstream portions of the dam as shown in Fig. 7. Strong shaking ended at approximately 10 s (Fig. 8).

Continuing the coupled effective stress analysis (Phase 3a) resulted in very little movement within the dam until approximately 32 s at which time the upstream shell proceeded to fail in a similar manner to that reported for the actual dam (Fig. 8). Fig. 9 shows displacement and velocity of the dam at 40 s at which time the program stopped due to excessive distortion of some of the elements. The downstream face did not move.
significantly following end of strong shaking as was the case of the actual dam.

In the Phase 3b total stress alternative the dam proceeded to fail once the liquefied elements were changed to a Mohr Coulomb model with residual strength $S_r$ as given by the Idriss & Boulanger (2007) relationship. This is not unexpected as the Lower San Fernando dam was one of the key case histories used in deriving the relationship. Again the dam only failed in the upstream direction as was the case with the actual dam.

7 CONCLUSIONS

Fully coupled effective stress numerical analyses have emulated field and centrifuge test case histories and given important insight into flow liquefaction behaviour. However, there is uncertainty in the procedures and a combined effective stress – total stress analysis carried out in addition to the fully coupled analysis is recommended. In the combined procedure the effective stress analysis is used to determine zones of liquefaction and deformations that occur during strong shaking. Following strong shaking liquefied elements are changed to a total stress model with a specified residual strength. The proposed residual strength is that back-calculated from case histories similar to those originally proposed by H. Seed and colleagues. In this manner the empirical experience used in current design practice is included in the analytical process.

An example back-analysis of the Lower San Fernando Dam demonstrates the proposed analysis procedure. Both the coupled effective stress analysis and the total stress alternative indicate a flow failure of the upstream shell into the reservoir but no flow failure of the downstream slope as was the case in the 1971 failure. The coupled effective stress analysis also indicated that the failure would occur approximately 25 s after the end of strong shaking - similar to what was actually observed.

ACKNOWLEDGEMENTS

The authors acknowledge the support from the British Columbia Ministry of Transportation - UBC Professional Partnership program, the Canadian Council of Professional Engineers, Trow Associates Inc., and the National Scientific and Engineering Research Council Strategic Liquefaction Grant No. NSERC 246394. The writers would like to acknowledge the contribution of Mahmood Seid Karbasi, Sung Sik Park, Michael Beaty, in development of the UBCSAND model and analysis procedures and Pascale Rouse for abstract translation.

REFERENCES


Naesgaard, E., Byrne, P.M., and Seid-Karbasi, M. 2006. Modeling flow liquefaction and pore water redistribution mechanisms, 8th NCEI, San Francisco, April


