Construction modelling of Oroville Dam using the hardening soil model

For LUSAS version:	23.0
For software product(s):	LUSAS Bridge plus or LUSAS Civil&Structural plus
With product option(s):	Geotechnical, Nonlinear

Problem Description

The construction of Oroville dam was modelled using the Duncan-Chang model by Kulhawy and Duncan [1] as described in the LUSAS geotechnical overview example "Construction of the Oroville Dam". In this example the problem will be re-solved using the hardening soil model with parameters matched to the those of the Duncan-Chang material properties

The dam was built in three stages. At its centre is a concrete core block. A cofferdam was built in front of this and finally the main dam itself was constructed. The basic layout is shown in figure 1.

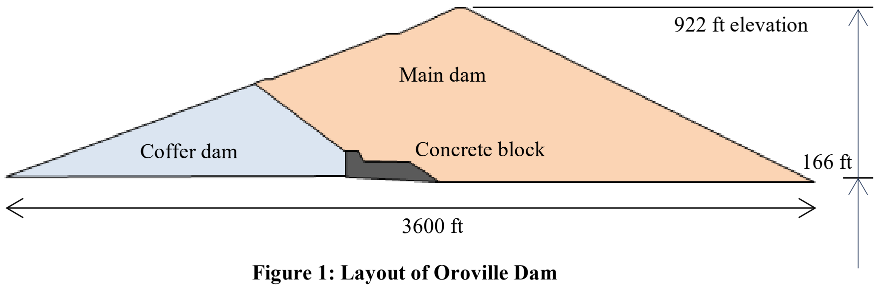

Figure 1: Layout of Oroville Dam

The modelling of the construction of the coffer dam is done by splitting it into six horizontal layers or lifts which are activated sequentially followed by the main dam using a further twelve lifts. In this solution, interface elements have been added between the concrete block and the soil and also between the coffer dam and the main dam. When the soil is placed, there is no friction at the boundary between the materials. After placement the sliding interface is replaced with large stiffness springs which glue the different materials together. The use of sliding interfaces prevents the unrealistic buildup of shear stresses as the loose soil is deposited and as a consequence reduces the minimum failure stiffness required to calculate the Duncan-Chang solution.

Keywords

Plane Strain, Duncan-Chang, hardening soil, hardening soil with strain hardening cap, activation, deactivation, interface elements.

Associated Files

Associated files can be downloaded from the user area of the LUSAS website.

- **construction_of_oroville_dam_using_HS.mdl** contains model with different materials used in the various analyses.
- **material_match.mdl** contains different materials to run drained triaxial and oedometer tests.

Discretisation

The problem is modelled using quadrilateral plane strain elements, QPN8 and IPN6 interface elements (figure 2).

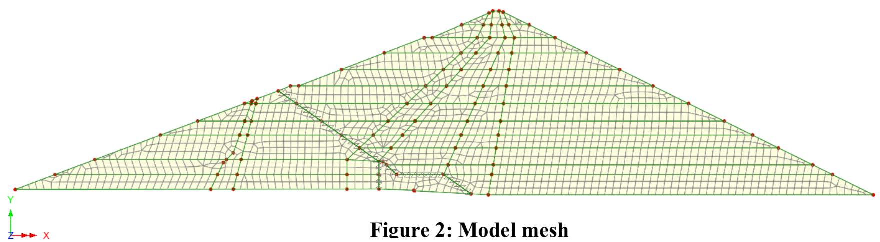

Figure 2: Model mesh

Material Properties

The dam is constructed from four materials used for the shell, the transition, the core and the concrete block as shown in figure 3.

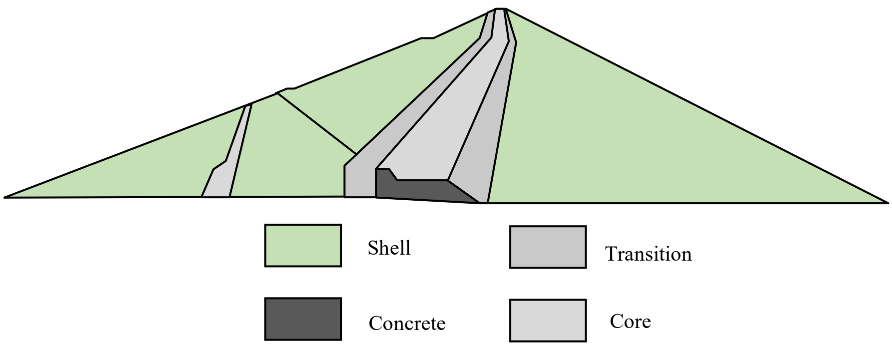

Figure 3: Layout of material layers

The soils are modelled using the Duncan-Chang material model and the hardening soil material model whilst the concrete is treated as a linear elastic material. Properties are given in table 1.

Table 1: Duncan-Chang/ hardening soil material properties

Parameter	Shell	Transition	Core	Concrete
Young's modulus (lbf/ft^2)	-	-	-	835x10^6
Density (lb/ft^3)	4.661	4.661	4.661	5.033
Cohesion (lbf/ft^2)	0	0	2795	-
Friction angle ϕ°	43.5	43.5	25.1	-

Modulus number, K	3780	3350	345	-
Modulus number for loading/unloading, K_{ur}	4160	3685	380	-
Modulus exponent, n	0.19	0.19	0.76	-
Failure ratio, R_f	0.76	0.76	0.88	-
Atmospheric pressure, P_a (lbf/ft^2)	2116	2116	2116	-
K_0	0.312	0.312	0.576	-
Poisson's ratio / parameters	-	-	-	0.15
Poisson's ratio at atmospheric pressure, G	0.43	0.43	0.30	-
*Cell pressure moderation factor, F	0.19	0.19	-0.05	-
*Poisson's ratio tangent factor, D	14.8	14.8	3.83	-
*Young's modulus when soil fails (lbf/ft^2)	80000	70900	7300	-
*Poisson's ratio on soil failure	0.49	0.49	0.49	-
Min.stress to evaluate soil stiffness (lbf/ft^2)	450	450	450	-
Max.stress in tension (lbf/ft^2)	0	0	5960	-

*parameters apply to the Duncan-Chang model only

Loading Conditions

Gravity is the only loading.

Theory

None. The results are compared with the Duncan-Chang solution.

Modelling Hints

The hardening soil model dialog has an option to input properties of the Duncan-Chang material model directly. Both material models share the same assumption regarding the asymptotic behaviour of soil in a drained triaxial test. The Duncan-Chang model is developed into an incremental elastic solution whilst the hardening soil model is defined in an elasto-plastic framework. The Duncan-Chang model developed various methods to model how Poisson's ratio changes during loading, whilst the hardening soil uses a constant Poisson's ratio but models plastic straining. The question is whether the models can produce similar results?

Figure 4 shows the simulated results for a drained triaxial test on the core material at a cell pressure of 4000lbf/ft^2. As expected, the Duncan-Chang (DC) material and the hardening soil (HS) material agree up until failure. The Duncan-Chang material continues to rise after failure as it swaps to the failure stiffness which is 1% of the initial stiffness. The hardening soil simply continues displacing with further plastic straining.

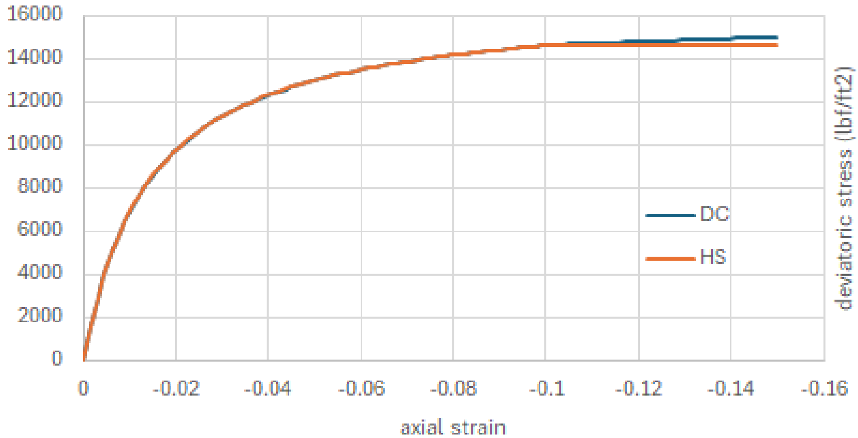

Figure 4: Triaxial (CD) for Core material

In figure 5 the volumetric strains developed in the triaxial test (CD) are compared. Using a constant Poisson's ratio of 0.3 with the hardening soil model produces too little volumetric straining compared to Duncan-Chang, whilst using a Poisson's ratio of 0.05 is much closer. In the analysis using the hardening soil, the Poisson's ratio of 0.05 is adopted.

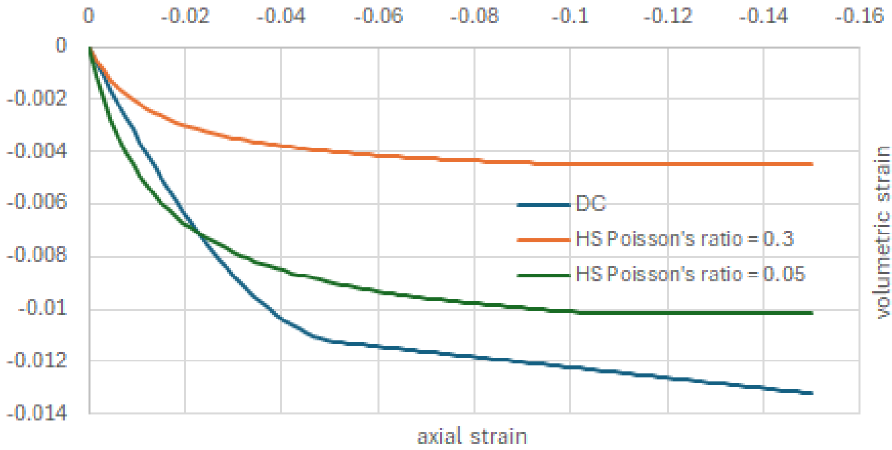

Figure 5: Change in volumetric strain in triaxial (CD) test for Core material

Similar results for triaxial tests (CD) at cell pressures of 4000lbf/ft^2 are shown in figures 6 to 9 for the shell and transition materials. Again, it is possible to find a value of Poisson's ratio that gives a reasonable approximation to the volumetric strains.

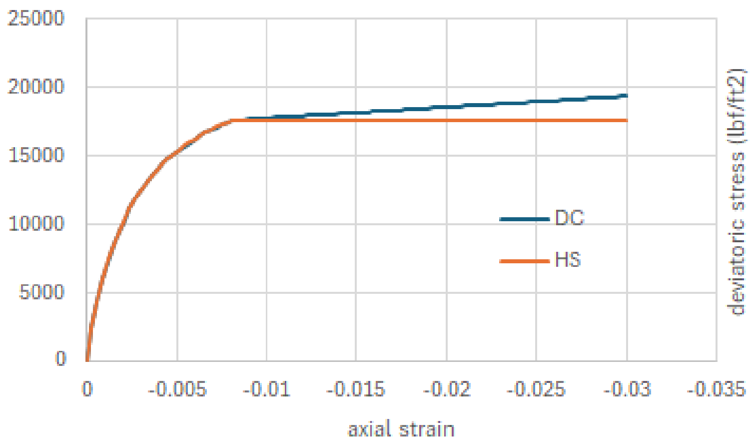

Figure 6: Triaxial (CD) for shell material

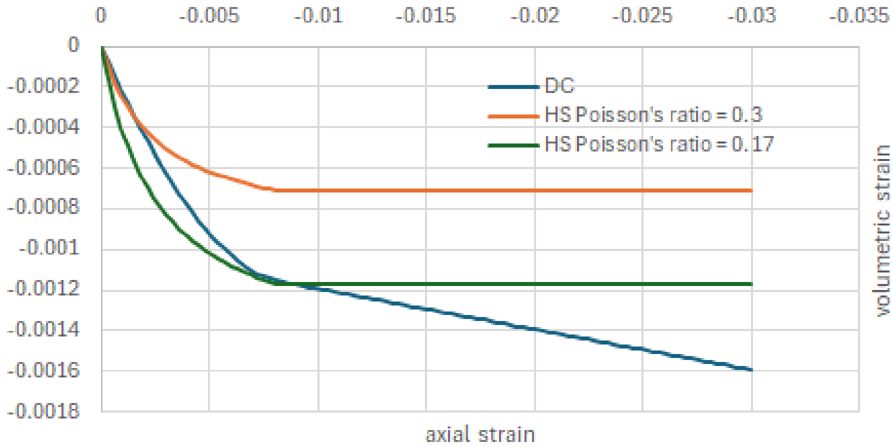

Figure 7: Change in volumetric strain in triaxial (CD) test for shell material

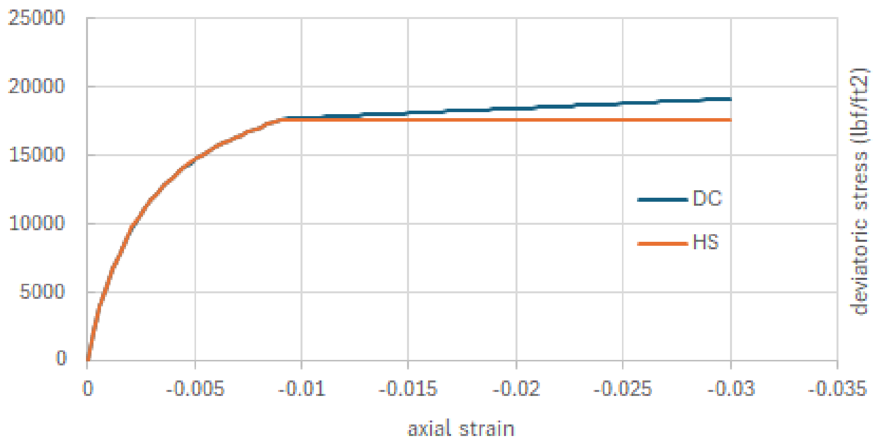

Figure 8: Triaxial (CD) for transition material

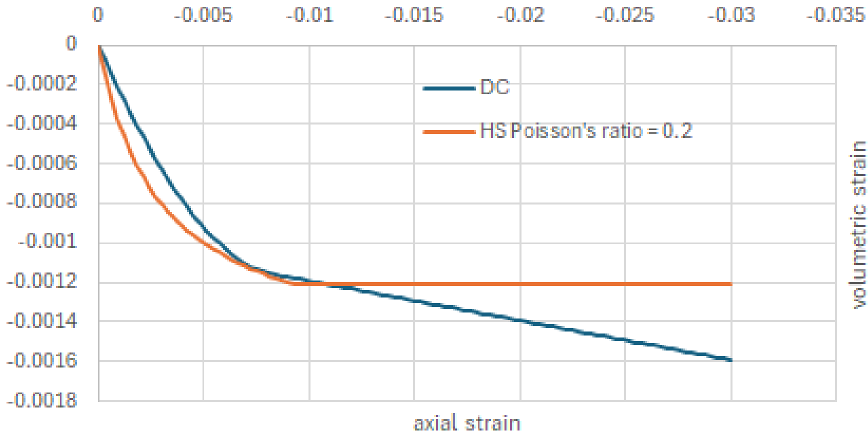

Figure 9: Change in volumetric strain in triaxial (CD) test for shell material

Comparison

The FE model is solved twice, first with Duncan-Chang and then with hardening soil with matched parameters. Each lift is modelled with a target of 10 increments, but the hardening soil may take a few more increments to complete using automatic step sizing. The target number of 'Iterations per increment' on the 'Nonlinear and Transient' dialog is increased from 4 to 10 because each increment is solved in three phases - first the new lift, second the rest of the dam and third both together. The default convergence parameters are adjusted with the change in total displacement norm tightened from 1% to 0.1% and the incremental displacement norm disabled. This is because the incremental displacement norm can result in extra iterations driving down the automatic step size without significantly changing the solution.

Figures 10 and 11 show the final settlement for the Duncan-Chang and the hardening soil models. Both show that the largest settlement occurred in the core of the main dam with the Duncan-Chang model predicting more concentrated settlement higher up the core. The maximum settlement of the Duncan-Chang model was 4' 11'', 3.3 inches more than the hardening soil model. In general, the patterns of settlement are the same.

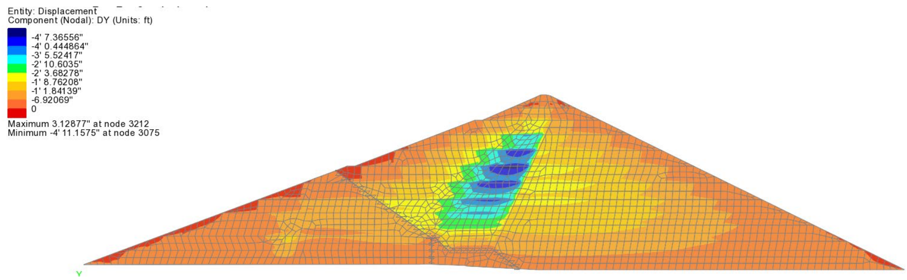

Figure 9: Final settlements for Duncan-Chang model

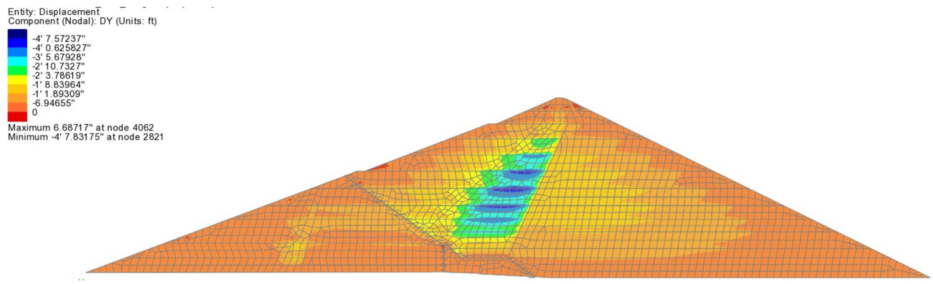

Figure 10: Final settlements for hardening soil model

Solution using cap

This type of problem is dominated by consolidation which is modelled by the cap in the HS model. To evaluate the cap properties an oedometer test is run using the Duncan-Chang model which is then used to define the hardening soil strain hardening cap. Note that the oedometer curve data (vertical strain v vertical stress) obtained from the Duncan-Chang analysis can be pasted directly from the Windows Clipboard into the Hardening Soil dialog. Figure 11 shows the original Duncan-Chang solution for the core material and the matched strain hardening path of the hardening soil model.

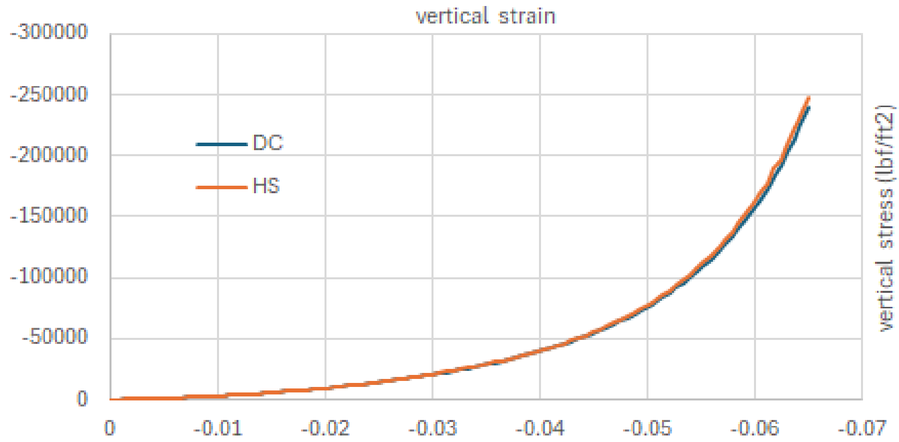

Figure 11: Oedometer solution for Core material

A valid solution can be calculated for a Poisson's ratio of 0.32 and above. In figure 12 the coefficient of lateral earth pressure K_0 is plotted. The hardening soil maintains a value of more or less 0.576 whilst the Duncan-Chang value ranges from 0.36 to 0.64.

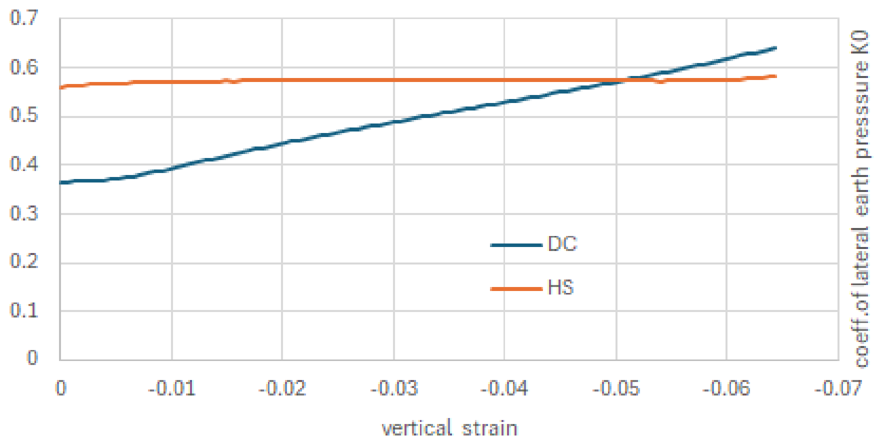

Figure 12: Variation of Core material K_0 with vertical strain

The triaxial (CD) test is re-run to see the effect of the cap. During the test the soil is consolidated at the same time as being sheared leading to contribution of extra plastic strains from the cap. Figure 13 shows that there are only small changes in the deviatoric stress/strain plot and figure 14 shows that we now get greater volumetric strains than produced by the Duncan-Chang model.

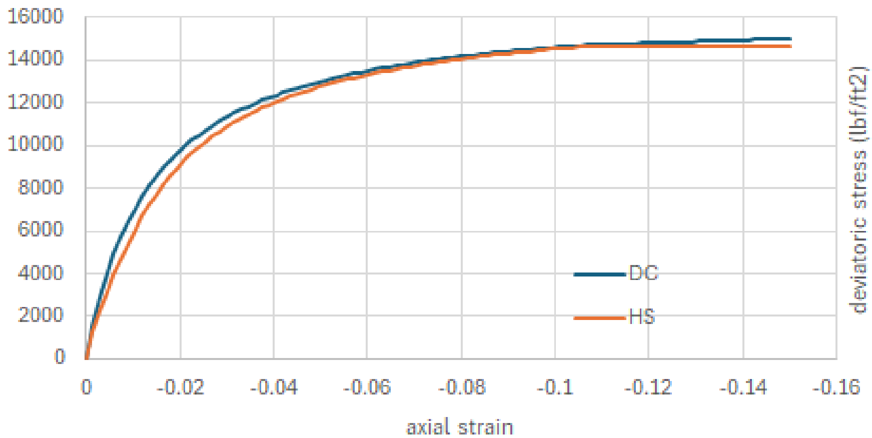

Figure 13: Triaxial CD test (Core) with HS and strain hardening

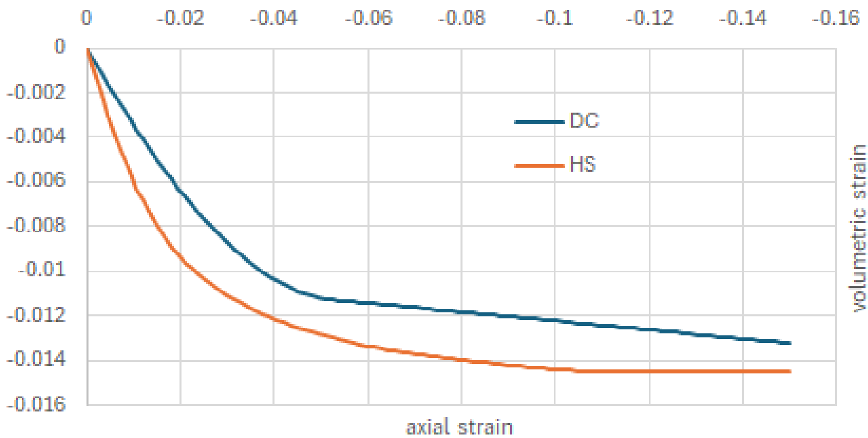

Figure 14: Volumetric change for triaxial test (Core) with HS and strain hardening cap

Repeating the same procedure for the both the shell and transition materials we find that the Duncan-Chang oedometer curve is stiffer than the curve from the hardening soil model without the cap. Unfortunately, it is not possible to fit a strain hardening cap when the elastic stiffness is less than the oedometer stiffness. It is possible to define a minimum elastic stiffness to overcome the problem but in this case the stiffness would be very high and affect the overall solution at low stress, so the shell and transition are used without a cap.

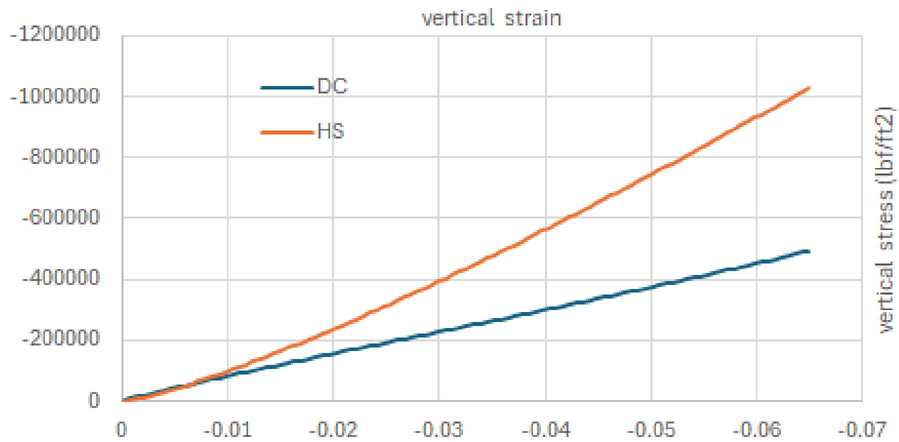

Figure 14: Oedometer solution for transition material

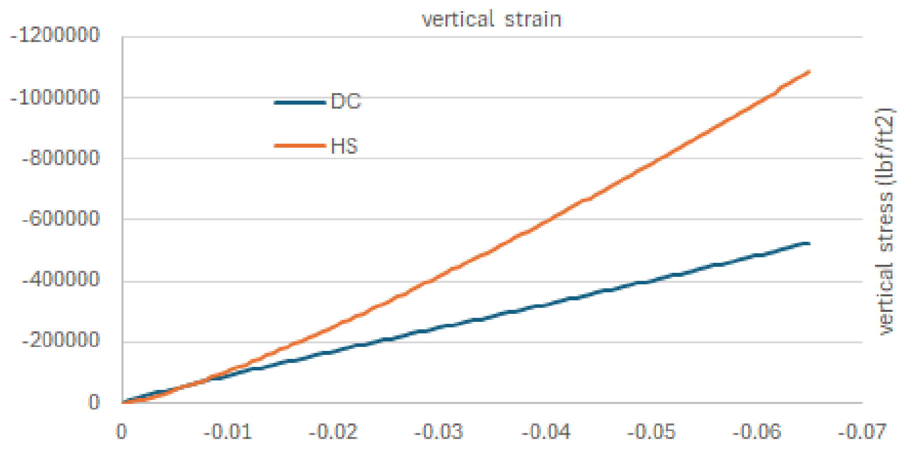

Figure 15: Oedometer solution for shell material

Comparison

Again, the maximum displacement in the Duncan-Chang solution exceeds the hardening soil solution, this time by 3.9 inches. The use of the strain hardening cap results in more concentrated displacement in the core.

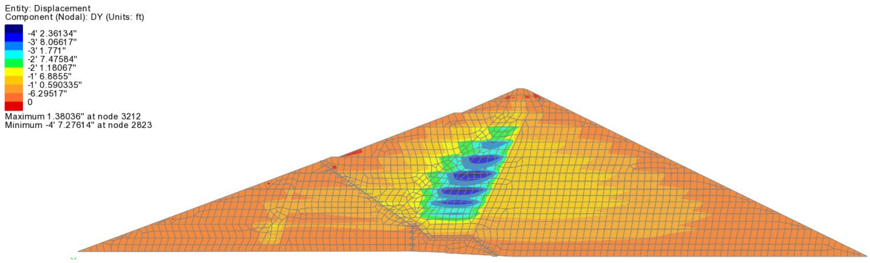

Figure 17: Final settlement for HS solution with strain hardening cap

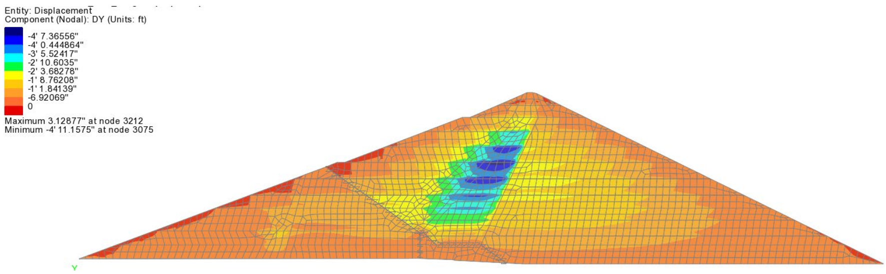

Figure 18: Final settlement for Duncan-Chang solution

References

[1] Kulhawy F.H. and Duncan J.M., Stresses and movements in the Oroville Dam, ASCE J.Soil Mech.Foundations Div., p653-665, 1972.