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The selection of appropriated models to solve the consolidation problem of the zoned earth dam

La sélection des modèles appropriés pour résoudre le problème de consolidation du barrage de terre par la méthode d'élément finie

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ABSTRACT

A finite element formulation to analyze two-dimensional characteristics of zoned earth dams is presented.This formulation is implemented by using a suitable constitutive relation for each material constituting the zoned earth dam earthen structure. Two-dimensional consolidation problems are solved to verify Biot's theory through the use of the developed EMDAM program. A zoned earth dam, namely, Llyn Brianne zoned earth dam built in the United Kingdom is selected to be analyzed. Presentations of numerical algorithm results of the deformation and the excess pore water pressure in the body of Llyn Brianne dam and its foundations during and after construction are made. The developed program (EMDAM) gives acceptable predictions and the results are close to the observations.

RÉSUMÉ

Une formulation d'élément finie pour analyser des caractéristiques à deux dimensions de barrages de terre sont présentées.
Cette formulation est appliquée en utilisant une relation constituante convenable pour chaque matériel constituant le barrage de terre.Les problèmes à deux dimensions de consolidation sont résolu pour vérifier la théorie de Biot à l'aide du programme développé (EMDAM). Le barrage qui est choisi d'etre analysé est nommé Liyn Brianne a été construit a la grande britagne . La présentation de résultats numériques d'algorithme nous donne la deformation et la pression d'eau dans le corp du barrage et ses foundations pendant et après la construction. Le programme développé (EMDAM) donne des prédictions acceptables et les résultats sont proches aux observations.

1 INTRODUCTION

Consolidation plays an important role in many soil mechanics problems. Provide a realistic representation of the stress- strain characteristics for the porous medium and predict the settlement of a finite loaded foundation on a thick compressible stratum represented the major problem to geotechnical engineering problems.

Soils are complex materials exhibiting a wide range of behaviour depending on classification, stress history and characteristics of the disturbing force. Therefore, the selection of a particular constitutive model may not be a random choice, but one which is arrived at by careful weighing of the physical and practical requirements. The selection of appropriate constitutive models may have a significant influence on the numerical results obtained. The constitutive relations of soil mechanics can generally be idealized into two groups; those which depend on elasticity and those which are not. The main difference between elasticity and plasticity is that in elasticity the deformations are recovered upon unloading while in plasticity retain a part of deformation on unloading (Desai and Siriwardane, 1984).

Because the zoned earth dam is constituted from different materials and the analysis of stresses and deformations in it is an exceedingly complex problem, it is necessary to choose a suitable number of constitutive relationships in order to render the problem tractable. A constitutive law or model represents a mathematical model that describes our ideas of the behaviour of a material.

2 LLYN BRIANNE DAM

The Llyn Brianne zoned earth dam as described by Carlyle (1969) reached its maximum height of (90 m from streambed level in 1971).It has a central wide clay core supported by rockfill shoulders with an average downstream slope of 1 vertically to 1.75 horizontally and an upstream slope of 1 to 2. The site is in a gorge cut by the river Towy (Central Wales) in Slaty mudstone and bedrock is close to the surface.

Figure 1. shows the position of the three horizontal plate gauges installed by the Consultants Binnie and Partners, London and the Building Research Establishment (U.K.) to measure deformations in the downstream rockfill. The plate gauges and the field measurements carried out during the construction of the dam are presented by Penman and Charles (1972a). The results for the buried plates are given in terms of the vertical and horizontal displacements of plates, plotted against the vertical height to the fill surface above the plates. In addition, Penman and Charles carried out an interesting and simple, linear-elastic obtained between observed and predicted movements at the plate gauges.

3 MODELING AND MATERIAL PROPERTIES

The dam is analyzed to obtain an estimate of the deformations, which can be caused by the self-weight of the fill during construction using a suitable constitutive relationship for each material used within the dam.Penman and Charles (1972a) and Carlyle (1969) stated that the compression test on intact cores of the bedrock had given a value of Young's modulus of $14*10^6$ kN/m^2 and it was apparent that the strength of the bedrock was not a critical factor in the stability of the dam.

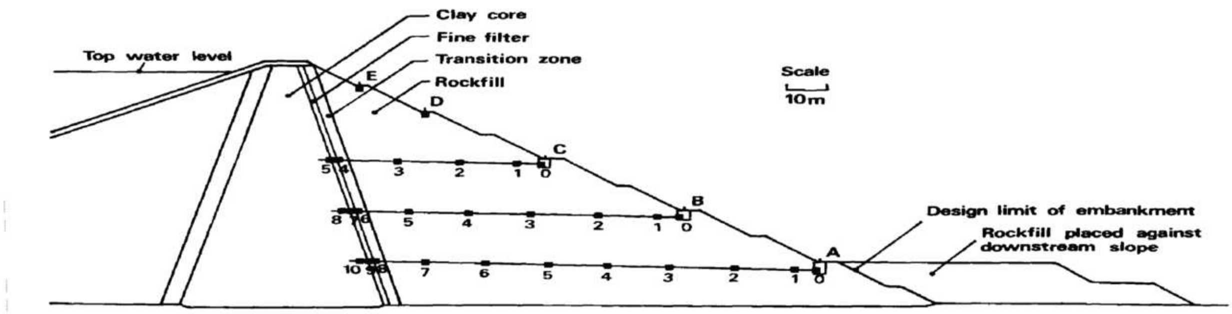


Figure 1. Cross Section of Llyn Brianne Dam Showing Position of Plate Gauges (after Penman and Charles, 1972a).

A line 52m below ground surface is taken to have zero movement, which means that the bedrock below this depth is assumed to be completely rigid (Penman and Charles 1972a). The embankment requires a rockfill filling rate of $13.77*10^4$ m^3/month and an average rate of rise of the dam of 6.096m/month (Carlyle, 1969).

In order to compare the current study results with Penman and Charles ones, the finite element mesh of the dam, including the foundation to a depth of 52 m and extending for distance of 235 m upstream and 280 m downstream from the dam centerline is drawn in Figure. 2 The time of construction is assumed to be 53 days for the first and the second layers, 51 days for the third and fourth layers and 39 days for the rest of the layers. The foundation is divided into five horizontal layers to allow for an increasing stiffness with depth. The embankment section is divided into ten layers within different thicknesses to simulate the construction and the position plate of gauges. Three constitutive models (hyperbolic, bounding surface, and endochronic) and six material types are employed in the analysis of the Llyn Brianne dam. The material properties for the models used for each material type, as shown in Table 1, are collected from various references (Penman and Charles (1972a, 1972b) and Carlyle (1969)). Any input parameter required by the analysis not found directly in the adopted references is either arrived at through appropriate relations to other known parameters, or assumed within reason.

Note: All Dimensions are in Meters.

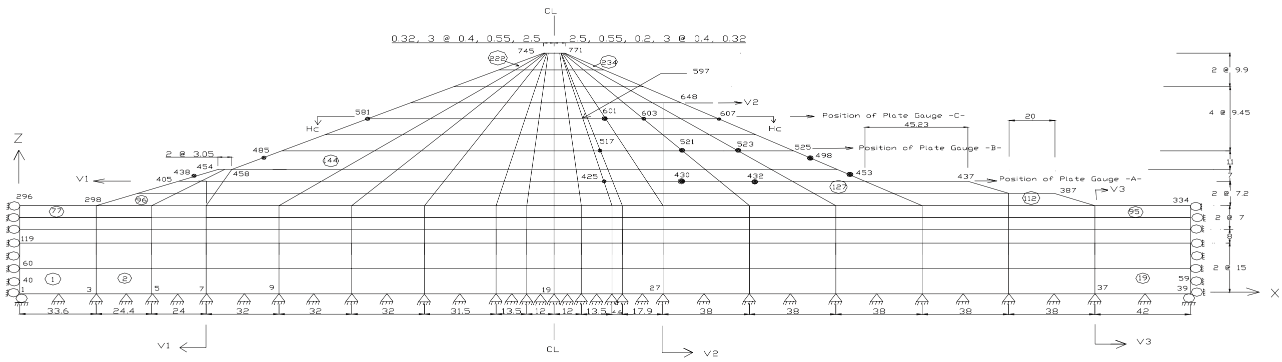


Fig.2:- Finite Element Mesh of Llyn Brianne Dam.

Table 1 Material Properties for the Llyn Brianne Dam (Obtained from Different References)

Hyperbolic Model (Bedrock and Rockfill)	Material Type1	Material Type2	Material Type3	Material Type4
Parameters	Foundation below 22 m (Bedrock)	Foundation 7-22 m (Bedrock)	Foundation (Bedrock)	Rockfill
Unit weight, γ_t ,kN/m^3	22.76	19.62	19.33	21.58
Cohesion, c , kN/m^2	22.5	10	18	7.0
Friction angle, ϕ ,degree	15	32	30	47
Reduction in friction angle, $\Delta\phi$, degree	0	0	0	0
Modulus number (Loading), k_n	3780	3214	3124	400
Modulus number (Unloading-reloading), k_{ur}	5670	4821	4686	600
Failure ratio, R_f	0.75	0.75	0.75	0.84
Bulk modulus number, k_b	3402	1701	847	364
Earth pressure coefficient at rest, ko	0.5	0.5	0.5	0.5
Poisson's ratio, μ	0.15	0.15	0.3	0.5

Table Continue

Horizontal permeability, k_y ,m/sec	$6*10^{-5}$	$6*10^{-5}$	$6.3*10^{-5}$	$3*10^{-3}$
Vertical permeability, $k_{y,}$,m/sec	$6*10^{-5}$	$6*10^{-5}$	$6.3*10^{-5}$	$3*10^{-3}$

Bounding surface Plasticity (Clay Core) Material Type 5		Endochronic (Filter) Material Type 6			
Parameters		Parameters			
Poisson's ratio, μ	0.4	Poisson's ratio, μ	0.3	Maximum void ratio, emax	1.2
Slope of the critical state line, M_{cs}	1.2	Specific gravity, G	2.67	Horizontal permeability, kx , m/sec	$3*10^{-5}$
Atmospheric pressure, Pa ,kN/m^2	101.34	Confining pressure	100	Vertical permeability, ky , m/sec	$3*10^{-5}$
Initial void ratio, e_o	0.9	Compression index, Cc	0.39		
Horizontal permeability, k_x , m/sec	$3*10^{-8}$	Initial void ratio, eo	1.0		
Vertical permeability, k_y ,m/sec	$3*10^{-8}$	Minimum void ratio, e_{min}	0.90		

4 MODELING USED FOR LLYN BRIANNE DAM

4.1 *Hyperbolic model*

The hyperbolic model representing the non-linear elastic model or the variable elastic model developed by Duncan and Chang (1970) is chosen.In their aforementioned work, Duncan and Chang derived the formula for tangent modulus Et. The equation is :-

$$E_t = \left[1 - \frac{R_f (1 - \sin\phi)(\sigma_1 - \sigma_3)}{2\bar{c}\cos\phi + 2\bar{\sigma}_3 \sin\phi}\right]^2 k_n \,\text{Pa}\left(\frac{\sigma_3}{\text{Pa}}\right)^n \quad (1)$$

where c=Cohesion; ϕ= Friction angle; R_f=failure ratio; $(\sigma_1-\sigma_3)_f$=the compressive strength, or stress difference at failure; $(\sigma_1-\sigma_3)_{ult}$= the asymptotic value of stress difference; and Pa = atmospheric pressure.

4.2 *Bounding Surface Model*

The inadequacy of the classical yield surface plasticity formulation to describe realistically the material response under cyclic loading conditions necessitated the development of a new concept (Valanis 1971). The salient feature of the bounding surface approach is that plastic deformation occurs for stress states within the surface at a rate, which depends on the proximity between the current stress point and an "image" stress point on the bounding surface defined by a proper mapping rule (Dafalias, 1981). In this study, this model will be used to describe the behaviour of clay core in the dam. The constitutive relation in inverse form is given by the following equation:

$$\dot{\sigma}_{ij} = D_{ijkl}\,\dot{\varepsilon}_{kl} \quad (2)$$

where σ_{ij} =total stress tensor ,D_{ijkl}= the elasto-plastic tensorial moduli; and ε_{kl}= strain tensor.

4.3 *Endochronic Model*

In 1971, Valanis proposed an alternative theory of viscoplasticity called "Endochronic theory", which is based on irreversible thermodynamics and the concept of intrinsic time which is not the absolute time measured by a clock as in viscoplasticity but a material property too.The endochronic model is chosen to predict the overall response of sands. The choice is made on the basis of its successful application to model sand behaviour and its sound theoretical basis.The parameters that are required for the endochronic model are easy to be calculated from conventional laboratory tests. In addition to that, the theory assumed inelastic change to be caused only by the rearrangement of grains; this model treats the sand as a non-linear elastic-plastic material. For sand, the stress-strain relations are conveniently written in terms of separate deviatoric and volumetric components (Bazant et. al, 1982):

$$de_{ij} = ds_{ij}\ /2G + de_{ij}^{\ p} \quad (3)$$

where eij, e^p_{ij} = the elastic and inelastic deviatoric strain; S_{ij} = the deviatoric components of the stress tensor σ_{ij}; and G = the shear modulus .

5 ANALYSES AND RESULTS

5.1 *Comparisons among Field Observed Results, Former and Current Analyses*

The finite element analysis gives values for displacements at all nodal points. The predicted displacements of the nodal points in comparison to the position of some buried plates have been plotted against the vertical height of the fill surface above each point, together with the measured displacement, to give direct comparison.

In Figure 3 shows vertical displacement results that are predicted by the current analysis and those which were observed for vertical displacements in three plate gauges (A, B and C) which are shown in Figure 1. Plot shows that the vertical displacement results predicted using the current analysis are about 4-20% less than those observed, and for greater heights of fill, the observed vertical displacements usually exceeds the predicted amounts. This may be due to in part of creep movements occurring under constant stress conditions.

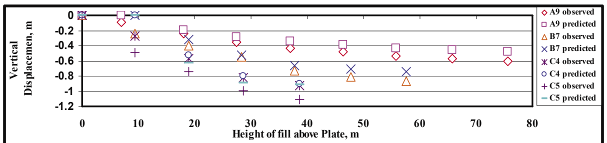


Figure 3. Vertical Displacements at Different Plates During Construction of Llyn Brianne Dam

The comparison between the observed and predicted horizontal displacements are given in Figure 4 The predicted horizontal displacement results are about 7 –17% more than those observed for plate gauges B and C, but at plate gauge A, they are about 9-25% more. This can be caused by the added restriction to horizontal movements afforded by steep sides of the narrow V-shaped valley in which the damwas build. This would offer more resistance than allowed for by the condition of plane strain assumed in the current two-dimensional analysis, an effect particularly important near the bottom of the valley.

The deformation shapes during and after construction are given in Figures 5 and 6. During construction, the maximum movement occurs at centerline of the due to the high concentration of load on this zone. At the end of the staged construction, the maximum displacement in both vertical and horizontal directions occurs at a level, which is equal to 93.85m. This behaviour is due to that during construction, each layer not only applies more loads onto the structure, but also it contributes to its stiffness.

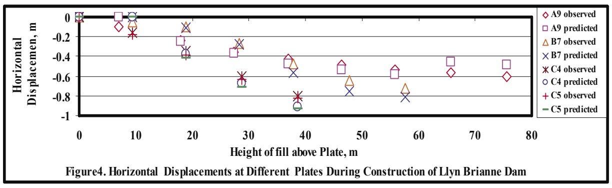


Figure4. Horizontal Displacements at Different Plates During Construction of Llyn Brianne Dam

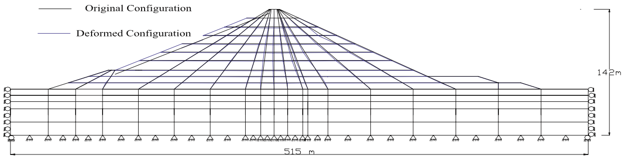


Figure 3. The Deformed Shape of Llyn Brianne Dam During Construction.

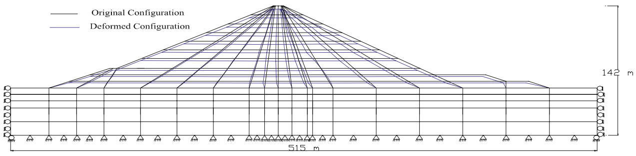
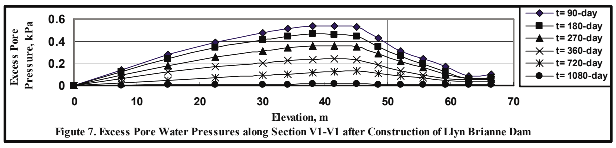


Figure 4. The Deformed Shape of Llyn Brainne Dam after Construction.

5.2 *Excess Pore Water Pressures*

Figure 5 to 8 show the excess pore water pressure dissipation at four vertical sections shown in figure (2) for Llyn Brianne dam. These sections are (V1-V1), centerline, (V2-V2) and (V3-V3). Due to non-homogeneity of the Llyn Brianne earth dam and the differences in the permeability values for the different zones of it, there is no symmetry in excess pore water pressure dissipation in spite of two -way drainage conditions. The analysis gives pore water pressures at centerline of the dam greater than the other two sections. This behaviour is due to the concentration of load at the centerline of the dam and the far distance between this section and the lateral and bottom dissipation zones.



Figute 7. Excess Pore Water Pressures along Section V1-V1 after Construction of Llyn Brianne Dam

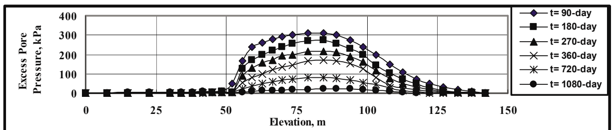


Figure 8. Excess Pore Pressures along Centerline after Construction of Llyn Brianne Dam

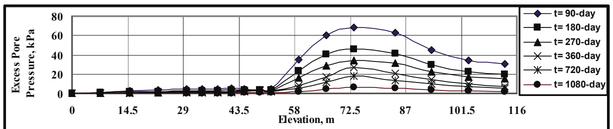


Figure 9. Excess Pore Pressures along Section V2-V2 after Construction Llyn Brainne Dam

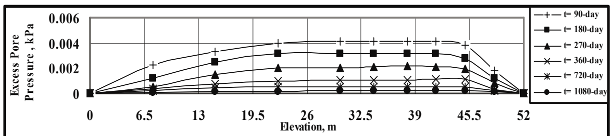


Figure10. Excess Pore Pressures along Section V3-V3 after Construction of Llyn Brianne Dam.

6 CONCLUSIONS

For Llyn Brianne dam at plate gauges A, B, and C, the vertical displacement results predicted using the current analysis are about 4 to 20% less than those observed, and for greater heights of fill, the observed vertical displacements usually exceeds the predicted amounts. This may be due to in part of creep movements occurring under constant stress conditions.

The predicted horizontal displacement results are about 7 to 17% more than those observed for plate gauges B and C, but at plate gauge A, they are about 9 to 25% more. This can be caused by the added restriction to horizontal movements afforded by steep sides of the narrow V-shaped valley in which the dam was built. This would offer more resistance than allowed for by the condition of plane strain assumed in the current two-dimensional analysis: an effect particularly important near the bottom of the valley.

During construction, the maximum movement occurs at centerline of the dam due to the high concentration of load on this zone. At the end of the staged construction, the maximum displacement in both vertical and horizontal directions occurs at a level, which is equal to 93.85m. This behaviour is due to that during constru- ction, each layer not only applies more loads onto the structure, but also it contributes to its stiffness.

Due to non-homogeneity of the Llyn Brianne earth dam and the differences in the permeability values for the different zones of it, there is no symmetry in excess pore water pressure.

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