NUMERICAL SIMULATION OF A STAGE CONSTRUCTED ROCKFILL DAM ON PLASTIC CLAY FOUNDATION

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ABSTRACT

A numerical simulation for predicting behaviour of a stage constructed Earth Core Rock Fill (ECRF) Dam founded on highly plastic clay foundation with coupled stress-strain-pore-water pressure (consolidation) analysis is carried out for a construction stage project in southern India. Load-deformation analysis of the clay core underlain by a plastic concrete diaphragm type cut-off wall is also evaluated to assess the structural integrity of the cut-off wall during construction. The effectiveness of ground improvement using stone columns in clay foundation is analysed and stability analysis is conducted for each construction stage.

Staged construction improves stability and reduces post construction settlement. The objective of the study is to optimize the rate of construction of ECRF dam for different consolidation periods ensuring stability of the dam at all times.

During numerical modeling a 2D plane strain, non-linear, time dependent behaviour of the structure was studied using a commercially available Finite Element Analysis computer code named SIGMA/W of GEO-SLOPE International, Ltd. For geotechnical characterisation of dam foundation, engineering properties were derived from extensive in-situ field investigations and were suitably supplemented with laboratory tests. Suitable comments on installation of appropriate instrumentation to monitor and control the performance of ECRF dam during construction have also been provided.

1. INTRODUCTION

Highly plastic normally consolidated clay foundation tends to have high compressibility and settle on application of large loads. Earth Core Rockfill dams built on such foundation pose stresses that may induce excessive deformations and lateral displacements which can be detrimental to the integrity of the dam body. Such dams are built using design as you construct approach controlling rate of fill placement and utilizing the instrumented response data collected during construction. For staged construction of earth embankments on soft and saturated soil layers, pore water pressure build-up can be very important for stability. A realistic prediction estimation of pore water pressures with aid of coupled stress-strain consolidation analysis proves to be useful in planning a safe construction sequence.

2. DESCRIPTION OF THE PROJECT

An earth-cum rock fill (ECRF) dam is being constructed across Godavari River at Polavaram, Andhra Pradesh, India. The entire dam length is demarcated into three zones namely: Gap-I (of 564 m length), Gap-II (of 1750 m length) and Gap-III (of 140 m length) as shown in Figure 1. Based on the interpretation of geotechnical investigations carried out, Gap-II is subdivided into two regions, as shown in Figure 2, namely:

(a) Region-I: from Chainage 0 m to Chainage 1025 m (primarily comprising of sand and silty sand)

(b) Region-II: from Chainage 1025 m to Chainage 1750 m (primarily comprising of clay of medium to high plasticity with surface sand deposits up to Chainage 1500 m, beyond which rock is available at the river bed).

This paper addresses prediction of behaviour of a stage constructed Earth Core Rock Fill (ECRF) Dam founded on highly plastic clay foundation i.e. Region-II of Gap-II.

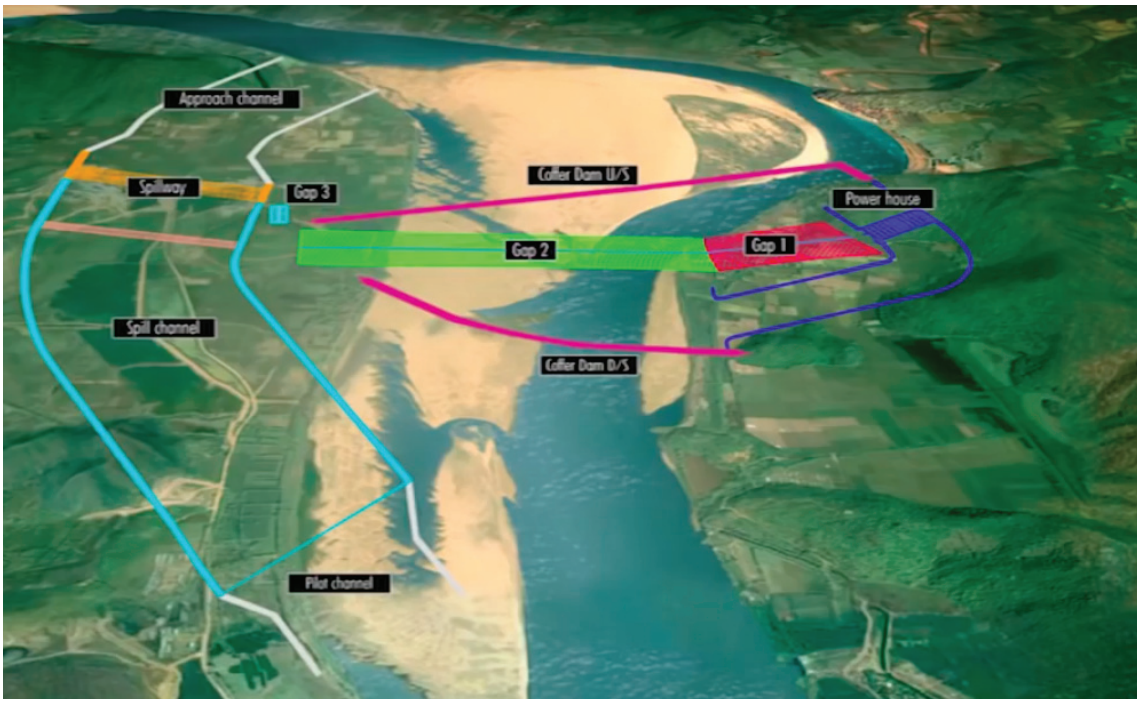

Figure 1 : Plan of Polavaram dam site across Godavari River. Three zones i.e. Gap-I, Gap-II & Gap-III are demarcated as shown.

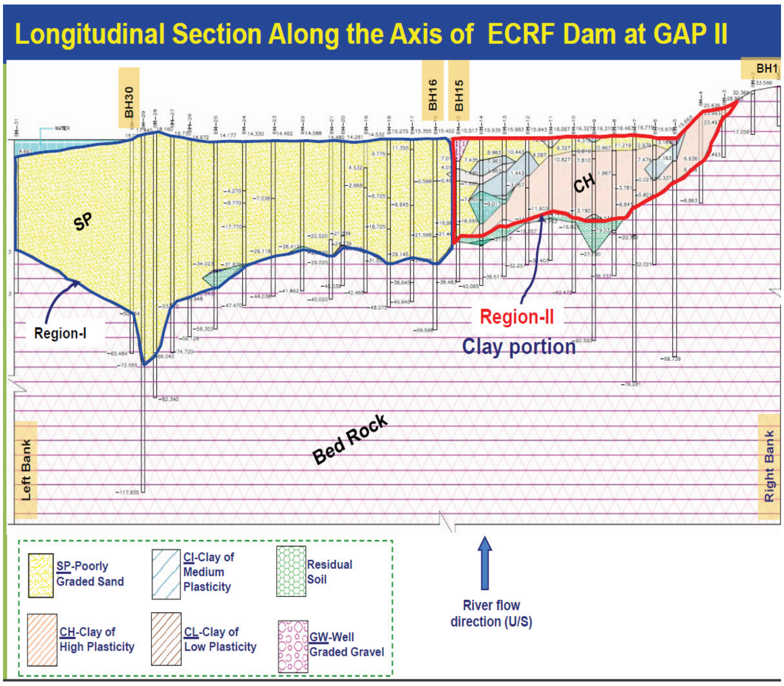

Figure 2 : Longitudinal Section depicting the sub-soil information along the axis of ECRF at Gap-II

3. SUBSURFACE CONDITIONS

For geotechnical characterization of dam foundation, engineering properties were derived from extensive in-situ field investigations and were suitably supplemented with conventional laboratory tests. As a part of the sub-surface exploration at the dam site, several Standard Penetration Tests (SPTs), electric Cone Penetration Tests (ECPTs) and three Cross-Hole Tests (CHTs) were conducted at Gap-II to delineate the sub-soil stratigraphy as well as to reasonably estimate the relevant engineering properties. Laboratory investigations such as grain size distribution curves, Atterberg's limits, consolidation tests and CU triaxial shear tests were conducted on undisturbed and disturbed samples.

As evident in figure 2, in region II, the highly plastic clay layer delineated in the foundation has a clay percentage by weight of up to 50% with an average of 30%. Average plasticity index is about 35% and liquid limit ranges from 40% to 75%. The clay was found to be normally consolidated clay. The constrained modulus D_s is estimated from SPT 'N' values and ECPT 'q_c' values.

3.1 Material properties obtained from geotechnical investigations

The intention of the analysis is to conduct a coupled stress-strain-pore water pressure analysis and the focus is on the consolidation of highly plastic foundation clay. The material models used in finite element analysis are Elastic-Perfectly Plastic with Mohr-Coulomb yield criterion and Modified Cam Clay constitutive model.

The major thrust of the numerical analysis is on the cohesive soil layers in the foundation where the generation and dissipation of excess pore-water pressure and consolidation is going to take place. A modified Cam clay (MCC) model (Roscoe and Burland, 1968) is used for their analysis. The major soil parameters required for description of MCC model are Over-consolidation ratio, ν (Poisson's ratio), e_0 Initial Void ratio, φ' (effective friction angle for calculating M (Slope of critical state line)), λ (Slope of normal consolidation line), κ (Slope of over-consolidation i.e. swelling line) calculated using equations 1 to 3.

$$M = \frac{6 \sin\varphi'}{3 - \sin\varphi'} \tag{1}$$

$$\lambda = \frac{C_c}{2.303}, \tag{2}$$

$$\kappa = \frac{C_s}{2.303} \tag{3}$$

where C_c = Compression Index and C_s = Swelling Index

Table 1 and 2 depict the typical properties of various material used for the numerical model based on the two constitutive models described above.

Large size triaxial shear test (specimen size 381 mm diameter and 813 mm height) and large size oedometer test (specimen size 1000 mm x 600 mm) both prepared at 87% relative density, were carried out to evaluate the properties of the rockfill. Tests have carried out for specimen containing maximum particle size 25 mm, 50 mm and 80 mm and extrapolated to max. particle size 600 mm.

Table 1 : Material properties with Elastic-Perfectly Plastic constitutive model adopted for FE analysis

Material	Material Category	Material Model	Unit Weight γ (kN/m^3)	E-modulus (kPa)	Poisson's Ratio ν	Cohesion (kPa)	Phi φ (degree)	Dilation Angle ψ (degree)	Coef. of vol. compress-ibility m_v (/kPa)	Sat. k_x (m/day)
Embankment Fill Clay Core	Total Stress Parameters	Elastic-Plastic (Total)	20	50,000	0.49	20	18	0.1	-	-
Embankment Fill Plastic Clay	Total Stress Parameters	Elastic-Plastic (Total)	20	25,000	0.49	30	15	0.1	-	-
Embankment Rockfill	Effective-Drained Parameters	Elastic-Plastic (Effective)	21	300,000	0.30	0	40	10	0	-
Foundation Silty Sand	Effective Parameters w/ PWP Change	Elastic-Plastic (w/ PWP change)	16	20,000	0.334	0	30	5	5×10^{-4}	0.864
Weathered Rock	Total Stress Parameters	Linear Elastic (Total)	22	92,000	0.25	-	-	-	-	-
Stone Column	Effective-Drained Parameters	Elastic-Plastic (Effective)	21	30,000	0.30	0	40	10	-	-
Deep Soil Mix	Effective Parameters w/ PWP Change	Elastic-Plastic (w/ PWP change)	18	50,000	0.30	10	35	5	2×10^{-4}	0.0864
Plastic Concrete	Total Stress Parameters	Linear Elastic (Total)	22	92,000	0.25	-	-	-	-	-
Interface of Soil & Cut-off	Total Stress Parameters	Elastic-Plastic (Total)	22	92,000	0.25	10	25	0.1	-	-

Table 2 : Material properties with Modified Cam Clay constitutive model adopted for FE analysis

Material	Material Category	Material Model	Unit Weight γ (kN/m^3)	OC Ratio	Poisson's ratio ν	Initial Void Ratio e_0	Phi φ (degree)	Lambda λ	Kappa κ	Coef. of vol. compressibility m_v (/kPa)	Sat. k_x (m/day)
Foundation Silty Clay (CH) upper	Effective Parameters w/ PWP Change	Soft Clay (MCC w/ PWP Change)	17	1.1	0.334	1	22	0.1736	0.0173	6 x10^{-4}	8.64 x 10^{-3}
Foundation Silty Clay (CH) lower	Effective Parameters w/ PWP Change	Soft Clay (MCC w/ PWP Change)	18	1.1	0.334	1	25	0.1736	0.0173	7 x 10^{-4}	8.64 x 10^{-4}

4. GROUND IMPROVEMENT TECHNIQUES

On assessment of the geotechnical parameters of the subsoil in the Region-II of Gap-II which primarily consists of clay of medium to high plasticity (max. thickness ≈ 34 m) with overlying loose sand lenses of max. thickness ≈ 5.5 m, stone columns (vibro-replacement) appears to be a suitable ground improvement technique. Vibro-compaction method is also recommended to improve the surface sand deposits at Region-II in the top 5.5 m to increase its density as a mitigation measure against liquefaction hazard.

Some design characteristics of the proposed ground improvement using stone columns is presented in Table 3. A 1.5 m thick plastic concrete diaphragm type cutoff wall is placed below the centrally located clay core up to weathered rock, for controlling seepage through foundation. To avoid damage/displacement/any adverse effect on the already constructed diaphragm wall during the installation of stone columns, deep soil mixing is also proposed along 20 m width at dam centre portion (i.e. 10 m on either side from centre line of dam alignment).

Table 3 : Geometric parameters and material characteristics of stone columns adopted for design

Vibro-replacement installation technique	Wet top feed method using well graded crushed gravel material of size from 12 mm to 75 mm.
Granular Blanket thickness	500 mm over the installed stone columns
Stone Column length, L	14 m to 16 m from the founding level
Stone Column diameter, D	900 mm
Stone Column pattern	Square
Stone Column spacing, s	2.0 m to 2.5 m varies from dam centre portion to dam toe portion
Treatment Area	Entire width of dam along alignment + 2 additional rows beyond the dam toe line (except for 10 m on either side of cutoff wall where deep soil mixing is proposed)
Area Replacement Ratio, a_s	10.17% (for s = 2.5 m) to 15.90% (for s = 2.0 m)

Deep soil mix properties are assumed as trial field tests have not been carried out yet. The strength parameters (such as confined compressive strength and axial strain at failure) of plastic concrete are obtained from triaxial tests at a confining pressure of 400 kPa after 28 days of curing. These parameters are shown in the Table 4.

Table 4 : Average strength and stiffness parameters of plastic concrete obtained after 28 days of curing

Parameters	Values
Confined Compressive Strength (CCS)	4.80 MPa
Axial Strain at Failure	5.2%
Initial Elasticity Modulus (from CCS test)	392 MPa
Secant Elasticity Modulus (from CCS test)	92 MPa
Permeability	4.1 x 10^{-10}m/s

5. ANALYSIS OF STAGED-CONSTRUCTED ECRF DAM

A typical cross-section of the earth core rockfill dam used in analysisis shown in Figure 3.

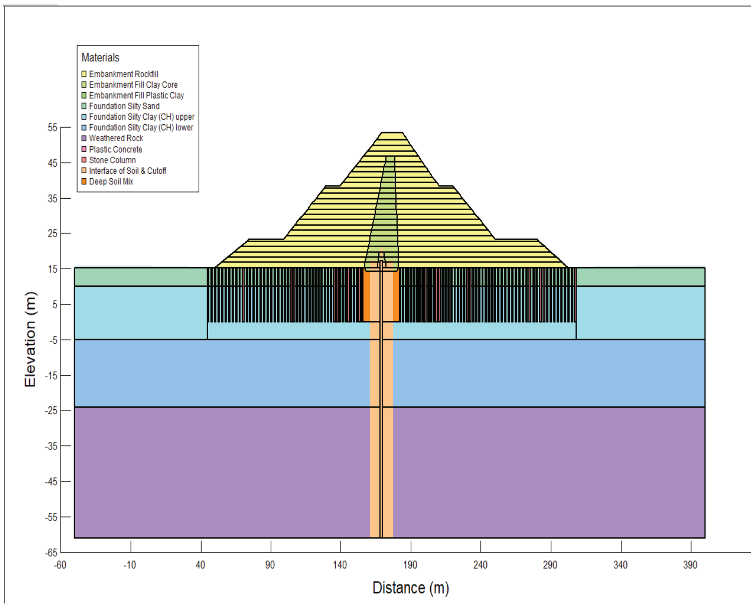

Figure 3 : Typical Cross Section of Earth Core Rockfill dam at Gap-II Region-II

During preliminary design, the total settlement due to consolidation of untreated ground is calculated using conventional Terzaghi's 1-D consolidation theory. This is justified as the width of the dam is very large compared to the thickness of the clay layer (the width of the loading area is greater than three times the thickness of the soft soil). The consolidation settlement is found to be of the order of 3 to 4 m obtained from previous studies (Ramana, 2017; Choudhary et al.2018).

To accelerate the consolidation of the foundation and increase the stability of the dam, ground improvement is proposed to be taken up. Post ground improved the dam is to be constructed in stages to ensure its safety. Staged construction improves stability and reduces post construction settlement. The objective of the study is to optimize the rate of construction of ECRF dam for different consolidation periods ensuring stability of the dam at all times. The goal is to achieve a factor of safety (FoS) of one at all times during construction.

A numerical simulation for predicting behaviour of a stage constructed Earth Core Rock Fill (ECRF) Dam founded on highly plastic clay foundation improved using stone columns and deep soil mixing with coupled stress-strain-pore-water pressure (consolidation) analysis is carried out. Load-deformation analysis of the clay core underlain by a plastic concrete diaphragm type cut-off wall is also evaluated to assess the structural integrity of the cut-off wall during construction. The effectiveness of ground improvement using stone columns in clay foundation is analysed and stability analysis is conducted for each construction stage. During numerical modeling, a 2D plane strain, non-linear, time dependent behaviour of the structure was studied using a commercially available Finite Element Analysis computer code named SIGMA/W of GEO-SLOPE International, Ltd., Canada.

The ECRF dam is simulated to be built in 25 lifts (each being 1.5 m high) and placed in 3 days for a given stretch. The number of days of fill placement is varied from single stage loading to 2 days, 3 days, 6 days and 9 days, to study its effect. In one case a dissipation phase of 3 days is also used assuming constant loading with time for dissipation of pore water pressure. An initial geostatic stress analysis under at-rest condition is conducted to know the state of stress in the foundation before placing any fill. The resulting state of stress was used as the reference state for the subsequent staged loading. The deformation from this initial geostatic analysis is not accounted for in subsequent analysis.

To capture realistic stresses on diaphragm wall, the contact between the cut-off wall and the surrounding soil is simulated as interface element. The stiffness property of the interface is assumed equal to that of the surrounding soil. The smear effect during installation of stone column is not accounted for in this study. Modulus of elasticity (E) is provided as a function of effective stress for the foundation soil and hydraulic conductivity (k) of soft soil which reduces as the soil compresses and void ratio change is inputted using a k-modifier as a function of effective vertical stress. The hydraulic conductivity in the y-coordinate direction is assumed to be equal to the hydraulic conductivity in the x-coordinate direction. Load response ratio i.e. the pore-pressure response to an applied load is taken as 100%.

The hydraulic boundary condition used for stone columns is a constant total head equal to the elevation of the natural ground while for granular blanket at the top of the stone columns, is taken to be as zero pressure head.

To ensure the efficacy of the cut-off wall, the governing criteria are its imperviousness and its deformations compatible with those in the surrounding soil without development of cracks. The factors influencing the stress-strain behaviour of the cut-off wall include the ground improvement in the surrounding soil, contact interface element properties between the wall and the surrounding soil, the modulus of elasticity of plastic concrete, and the type of junction between cut-off wall and clay core. The diaphragm cut-off wall is subjected to large dead load from the dam body, the water head difference between upstream and downstream water level i.e. unbalance seepage force, the effective lateral earth pressures, skin friction and self weight. End of construction loading condition may not be of much importance if diaphragm wall is so located that loading on either side is reasonably symmetrical. The critical loading condition for diaphragm wall is when the reservoir is full. However, this study evaluates the behaviour of the diaphragm wall till the end of construction stage only.

For ensuring the stability of slope at all times during construction, a check on the factor of safety (more appropriately the stability factor) has been kept. The slope stability is analysed using stress state from the coupled stress-strain-pore water pressure analysis carried out in SIGMA/W. The stresses calculated in the finite element analysis are used in limit equilibrium framework to analyse factor of safety for a trial slip surface. The factor of stability of a slope by the finite element stress method is defined as the ratio of the summation of the available resisting shear force along a slip surface to the summation of the mobilized shear force along a slip surface. The stress values are used to compute the normal stress and the mobilized shear stress at the base centre of each slice. The factor of safety is checked at regular intervals which reveal its variation with time.

5.1 Coupled Numerical Analysis: The Theory

SIGMA/W of GEO-SLOPE International, Ltd. is formulated to solve soil consolidation problems using a fully coupled formulation. A fully coupled analysis requires that both the stress-deformation and seepage dissipation equations be solved simultaneously. The theoretical formulation for this rigorous method is discussed below.

When coupled, three equations are created for each node in the finite element mesh. Two are equilibrium (displacement) equations and the third is a continuity (flow) equation. Solving all three equations simultaneously gives both displacement and pore-water pressure changes. In the un-coupled consolidation formulation the seepage analysis is solved independently of the volume change analysis. The incremental change in pore-water pressures from the seepage solutions are used at each load step in the stress-deformation calculation in order to determine the change in effective stresses.

5.1.1 *Constitutive equation for soil structure*

The incremental strain-stress relationship for an unsaturated soil medium can be written as equation 4

$$\{\Delta\sigma\} = [D]\{\Delta\epsilon\} - [D]\{m_H\}(u_a - u_w) + \{\Delta u_a\} \tag{4}$$

where:

ε = normal strain

σ = normal stress

u_a = pore − air pressure

u_w = pore − water pressure

$[D]$ = drained constitutive matrix

H = unsaturated soil modulus for soil structure with respect to matrix suction $(u_a - u_w)$

$\{m_H\}^T = \left[\frac{1}{H}\frac{1}{H}\frac{1}{H}\ 0\right]$

5.1.2 *Flow equation for water phase*

The two-dimensional flow of pore-water through an elemental volume of soil based on Darcy's law and expressed in terms of pore-pressure is described by the following partial differential equation 5:

$$\frac{\partial}{\partial x}\left(k_x \frac{1}{\gamma_w}\frac{\partial u_w}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y\left(\frac{1}{\gamma_w}\frac{\partial u_w}{\partial y} + 1\right)\right) + Q = \frac{\partial \theta_w}{\partial t} \tag{5}$$

where;

k_x, k_y = the hydraulic conductivity in x and y direction, respectively

u_w = pore − water pressure

γ_w = the unit weight of water

Q = a sink or source specified boundary flow

θ_w = the volumetric water content

t = time

5.1.3 *Finite element formulation for coupled analysis*

In a coupled consolidation analysis, both equilibrium and flow equations are solved simultaneously.In SIGMA/W, the finite element equilibrium equations are formulated using the principle of virtual work, which states that for a system in equilibrium, the total internal virtual work is equal to the external virtual work. The coupled equations 6 and 7 are used for finite element analysis:

$$[K]\{\Delta\delta\} + [L_d]\{\Delta u_w\} = \{\Delta F\} \quad (6)$$

$$\beta[L_f]\{\Delta\delta\} - \left(\frac{\Delta t}{\gamma_w}[K_f] + \omega[M_N]\right)\{c\} = \Delta t\left(\{Q\}|_{t+\Delta t} + [K_f]\{y\} + \frac{1}{\gamma_w}[K_f]\{u_w\}|_t\right) \quad (7)$$

where;

$$[K] = \sum[B]^T[D][B] \quad (8)$$

$$[L_d] = \sum[B]^T[D]\{m_H\}\langle N\rangle \quad (9)$$

$$\{m_H\} = \langle\frac{1}{H}\frac{1}{H}\frac{1}{H}\ 0\rangle \quad (10)$$

$$[K_f] = \sum[B]^T[K_w][B] \quad (11)$$

$$[M_N] = \sum\langle N\rangle^T\langle N\rangle \quad (12)$$

$$[L_f] = \sum\langle N\rangle^T\{m\}[B] \quad (13)$$

$$\beta = \frac{E}{H}\frac{1}{(1-2\nu)} = \frac{3K_B}{H} \quad (14)$$

$$\omega = \frac{1}{R} - \frac{3\beta}{H} \quad (15)$$

$[B]$ = gradientmatrix (strainmatrix)	$[K_f]$ = elementstiffnessmatrix
$[K_w]$ = hydraulicconductivitymatrix	$\langle N\rangle$ = rowvectorofshapefunctions
$[D]$ = drainedconstitutivematrix	$[M_N]$ = massmatrix
$[K]$ = stiffnessmatrix	$[L_f]$ = couplingmatrixforflow
$[L_d]$ = couplingmatrix	$\{m\}^T$ = isotropicunittensor, $\langle 1\ 1\ 1\ 0\rangle$
$\{\Delta\delta\}$ = incrementaldisplacementvector	δ = nodaldisplcement
$\{Q\}$ = theflowatboundarynodes	K_B = bulkmodulus
R = amodulusrelatingtheΔvolumetricwatercontentwithΔmatricsuction	

5.2 Results and Discussions

Analysis was carried out for six cases wherein the number of days taken for construction of dam is varied. A hypothetical situation of the dam being constructed in a single stage is used for comparison purpose. For the stage constructed cases, the time of construction of each lift is varied from 2 days, 3 days, 6 days, 9 days and in one case with a dissipation phase of 3 days. The factor of safety of the upstream slope is checked at, after every 5 lifts to look at the trend. Table 5 and Figure 4 shows the factors of safety obtained in various cases at different times. On comparison of FoS in case 3 and 6, it may be noted that a dissipation phase of 3 days after every lift has increased the FoS tremendously.

Table 5 : Cases analysed for optimising the rate of staged construction along with FoS at various stage

Case No.	Case Brief	FOS at the end of construction	After 20 lifts	After 15 lifts	After 10 lifts	After 5 lifts
1	Single stage construction 1 lift-1 day total Dissipation Phase-0 day each	0.668	-	-	-	-
2	Incremental staged construction 25 lifts-2 days each Dissipation Phase-0 days each	1.070	1.128	1.355	1.860	2.231
3	Incremental staged construction 25 lifts-3 days each Dissipation Phase-0 days each	1.081	1.142	1.372	1.881	2.232
4	Incremental staged construction 25 lifts-6 days each Dissipation Phase-0 days each	1.113	1.182	1.416	1.934	2.255
5	Incremental staged construction 25 lifts-9 days each Dissipation Phase-0 days each	1.139	1.217	1.453	1.963	2.276
6	Incremental staged construction 25 lifts-3 days each Dissipation Phase-3 days each	1.512	1.559	1.690	2.193	2.440

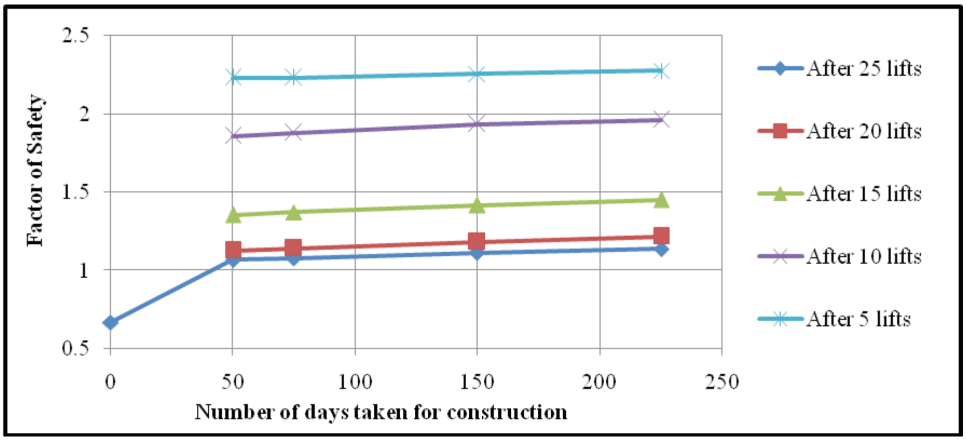

Figure 4 : Factors of safety obtained at different stages of construction when the time taken to construct the complete dam is varied from 50 days to 225 days

As noted above, case 3, having a lift construction time of 3 days (feasible though not practical) with no dissipation phase, is quite critical in terms of FoS. Therefore, the following discussion of results shall pertain to case 3 for assessing the worst case scenario.

1. The ground surface deformation observed in case 3 as depicted in figure 5 shows maximum settlement of about 3 m and a heave beyond dam toe of about 1 m at the end of construction. The reduced settlement around the cut-off wall is on account of ground improvement by deep soil mixing.

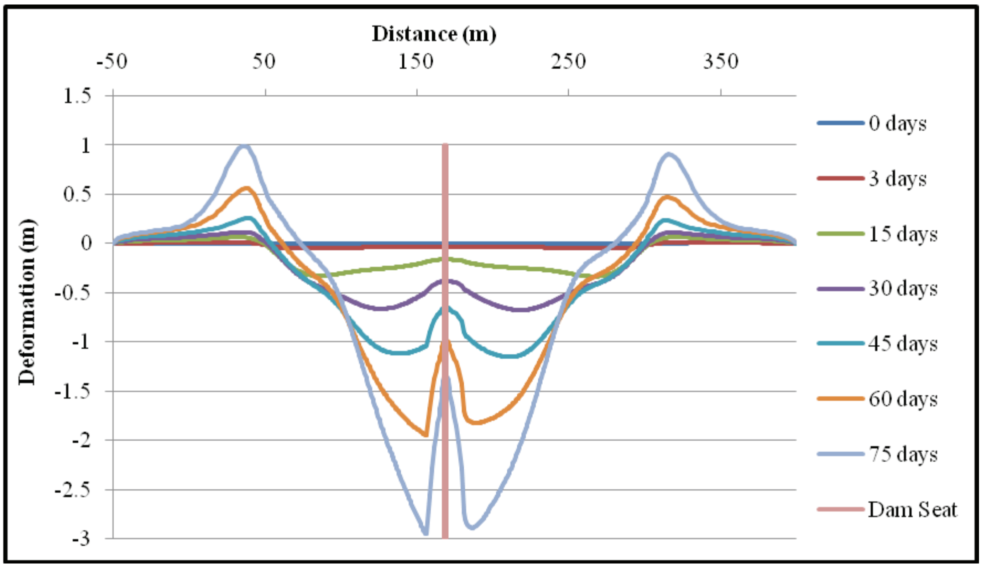

Figure 5 : Ground surface deformation (settlement and heave) at different stages of construction.

2. The lateral deflection of a vertical plane 20 m deep from the ground surface at upstream dam toe is plotted in figure 6. Maximum deflection of about 1.9 m is observed at the end of construction in case 3.

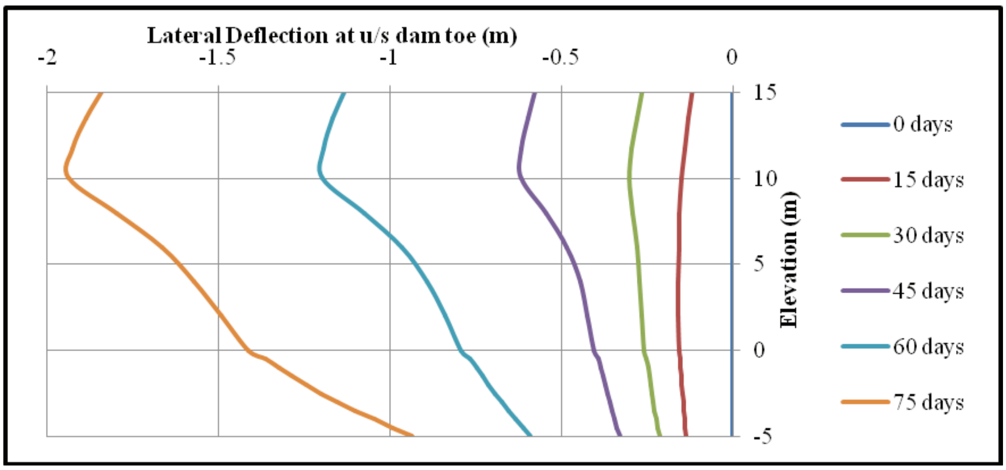

Figure 6 : Lateral deflection of a vertical plane 20 m deep at upstream dam toe at different stages of construction.

3. The vertical strain along the diaphragm wall is plotted in Figure 7. Maximum vertical strain is observed at the top of the diaphragm wall, at end of construction in case 3, is about 0.073. This is greater than the allowable axial strain at failure. It may be noted that strains are reduced in the wall when the surrounding soil is improved.

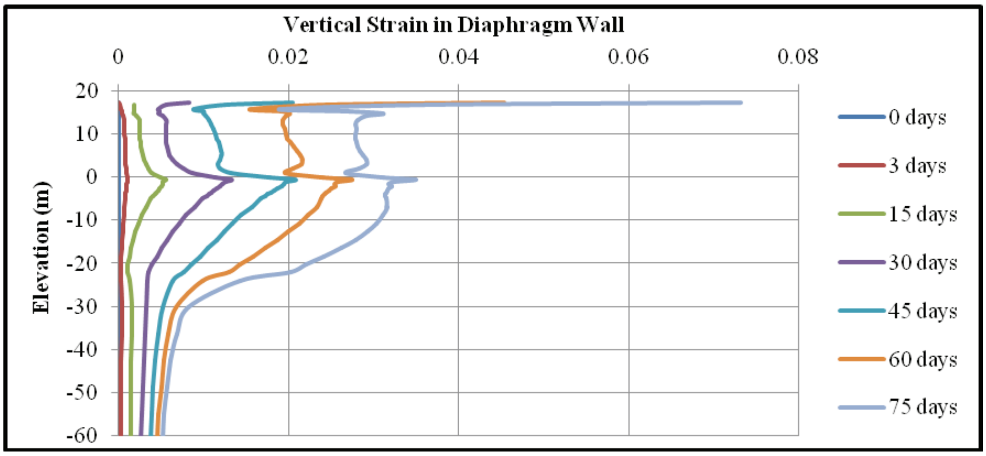

Figure 7 : Vertical strain along the diaphragm wall at different stages of construction.

4. The relative vertical displacement between the clay core and the top of the diaphragm wall is about 2 m. Hence, about 3 to 4 m thick cover of plastic clay above the top of diaphragm wall will be required.
5. The staged construction of ECRF dam founded on highly plastic clay foundation at Gap-II Region-II of Polavaram Irrigation Project, Andhra Pradesh can be safely carried out after ground improvement using stone columns and deep soil mixing. Practically the proposed lift height at field is 600 mm after compaction which may take about at least 1-2 days to place. The time to place each lift will reduce at higher elevation of the dam due to reduced earthwork. With availability of good instrumentation data, the construction time can be minimised to about 2.5 to 3 months, not taking into account the resource constraints.

6. RECOMMENDED INSTRUMENTATION

An extensive field instrumentation installation scheme to monitor and control the rate of construction against failure is suggested. The instrumentations include settlement gauge, inclinometer, piezometer and earth pressure cell that allow observation of the dam foundation system. Sufficient quantity including redundancies, of the best available modern instruments should be used for monitoring the long-term behaviour of the system. Instrumentation next to the diaphragm wall will be effective in understanding its behaviour during the construction.

7. SUMMARY AND CONCLUSION

1. With proper instrumentation and careful monitoring of the collected data, field construction rates can be adjusted and an embankment dam can be safely founded on thick soft deposits.
2. Field instrumentation data may be utilized to validate prediction of numerical model and improve it further for realistic predictions of field settlement behavior during construction. Back analysis will verify the material properties and modeling assumptions.
3. Extent of embedment of diaphragm wall into the core can be decided on the basis on the relative displacement between the clay core and the top of the wall. Top end of the diaphragm wall shall be rounded to avoid stress concentration at the corners.
4. A cover of plastic clay is provided around diaphragm wall to safeguard against any possible damage to the wall or to the impervious core. This plastic clay cover shall be of relatively high plasticity with liquid limit more than 50%. This cover will act as a deformable cushion and will reduce stress concentration. It will also improve the interface contacts.
5. There is always a possibility of differential settlement between the cut-off wall and the foundation, resulting in friction between the cutoff wall and the surrounding soil. With suitable ground modification such differential settlement and consequential friction force on the cut-off wall can be drastically reduced.
6. A plastic concrete of low elastic modulus in the diaphragm cut-off wall whose stiffness is close to the foundation stiffness is recommended. Usually, plastic concrete with modulus of elasticity of about 4-5 times that of the surrounding soil is considered adequate.
7. Intensive ground improvement technique at transition zone shall be ensured to match the rate of settlement of clay and sand layers to avoid damage in clay core of dam.
8. The range of settlement expected in improved ground conditions warrant defensive measures like providing thicker clay core and increasing the constructed height of dam (and the core)to ensure that the free board is not encroached upon.

DISCLAIMER

The authors declare that there is no conflict of interest regarding the publication of this paper. The present exercise is based purely on the academic and research interest of the authors. It does not reflect the professional opinion of Central Water Commission (CWC) or any other organisation, on the matter dealt herewith. The content/findings are not to be construed as an official CWC position.

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