Settlement behaviour of a concrete faced rock-fill dam

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Abstract. Settlement behaviour of a concrete faced rock-fill dam (CFRD), Kurtun dam, located in the East Black Sea Region, Turkey, is investigated. This is the first instrumented CRDF in the Country. Two dimensional plane strain finite element analyses were carried out to assess the total stresses and displacements in the dam both for construction and reservoir filling stages by using computer program PLAXIS. Predicted stresses and settlements are compared with those observed and overall the results are found to be in good agreement for the construction stage. Due to the relatively steep abutment slopes, cross-valley arching has a significant effect on stresses and displacements in the dam body. For the reservoir filling condition, it is seen that, predicted settlements are larger than the observed values, which indicates that during reservoir filling the rock-fill embankment responds more stiffly than it does during the construction stage. The hardening soil model is used to represent the non-linear, inelastic and stress dependent behaviour of rock-fill material. The model parameters are selected from the appropriate values in the literature investigating comparable cases.

Key words. concrete faced rock-fill dams, displacement, finite element analysis, hardening model, stress.

1. Introduction

Today, concrete faced rock-fill dams (CFRD) are very popular all over the world, especially in regions, which receive heavy rain and where impervious soil reserves are insufficient. In the current state of the art, the design of a CFRD is based on experience and engineering judgement (Cooke, 1984). Since, these constructions are important structures; their behaviour should be estimated realistically for both construction and reservoir filling stages. Finite element method is one of the available tools used in the prediction of structural behaviour.

The key point in the analysis is down-to-earth modelling of the stress-strain relationships of rock-fill materials, preferably based on triaxial test results. Considering the particle sizes of rock-fill material, up to 1.2 m diameter, the difficulty in obtaining experimental data, which in our case is lacking, becomes obvious. Although limited, the available triaxial data in the literature indicate that rock-fill

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materials possess highly stress dependent, inelastic and non-linear stress-strain relationships (Marsal, 1967; Marachi et al., 1972; Duncan et al., 1980).

Among the material models used in the available studies carried out in recent years, Duncan and Chang's (1970) hyperbolic model is probably the most common. The hyperbolic model has been utilized in a number of similar research successfully (Khalid et al., 1990; Saboya and Byrne, 1993; Liu et al., 2002).

In the present study, the "hardening soil model", which is essentially a modified implementation of the hyperbolic model available in PLAXIS, is used to represent the rock-fill behaviour (Schanz et al., 1999).

2. Dam Characteristics

2.1. GENERAL

Kurtun is located in East Black Sea Region, 27 km northwest of Torul town of Gümüşhane province, on Harsit River. The main purpose of the dam is to provide energy with an annual out put of 198 GWh.

Construction of the dam embankment was started in 1997 and finished in 1999. After the completion of rock-filling, the construction process had been given a halt for about 1.5 years until the major part of the settlement of rock-fill embankment was completed. The construction of concrete lining was continued there after. The reservoir impounding process was started on 08.02.2002; and the water level reached 630 m elevation on 30.04.2002.

Kurtun dam is 133 m high from the river bed. The side slopes are 1.4:1 and 1.5:1 (H:V) in the upstream and downstream embankment faces, respectively. The slope of the concrete membrane is 1.406:1. The crest extends 300 m. The dam is constructed on a relatively narrow valley with steep slopes. The valley is 45 m wide. Abutment slopes, on the average are 61.32° and 52.04° for the left and right abutments, respectively. Figure 1 shows the largest cross-section with different zones indicated.

Basic geological units in the dam site are granodiorite, diabase andesite and limestone. Granodiorite is the most common type of rock found in the region on which the dam rests. The details of zoning and construction are given in Table 1.

Figure 1. Largest cross-section of Kurtun dam.

Table 1. Materials and construction details for Kurtun dam

Zone	Material				Construction techniques	
	Type	Particle sizes			Layers (m)	Compaction (10 t vibratory roller)
		D _{max} (mm)	Sand (%)	Fines (%)		
1A	Impervious fill	50	>70	>35	0.60	4 passes (static)
1B	Tuvenan alluvium	400	–	<20	0.60	4 passes
2A	Sieved rock (or alluvium)	150	30–55	2–10	0.40	6 passes (4 static + 6 dynamic)
2AA	Filter	20	70–100 40% <0.5 mm	<5	–	–
3A	Selected rock	300	15–45	<5	0.40	6 passes
3B	Quarry rock fill	600	<20	<5	0.80	4 passes + 150 lt/m ³ water
3C	Quarry rock fill	1000	–	<2	1.20	4 passes
3D	Selected rock	2000	–	–	Surface placed rocks	

2.2. INSTRUMENTATION OF THE DAM

Because, it was the first example of its type in Turkey, Kurtun dam was extensively instrumented in order to monitor the performance. Four types of instruments were used for monitoring the behaviour. These were:

- Hydraulic settlement devices, located within the rock-fill (ZDO)
- Hydraulic pressure cells, located within the rock-fill embankment (BO)
- Strain-meters, located in the concrete membrane (GO)
- Surface-mount joint-meters, located in the concrete membrane (DDO)

A total of 33 hydraulic settlement devices and 21 earth pressure cells were installed at three cross-sections and four different elevations. The locations of the devices at the maximum cross-section of the dam are shown in Figure 2. In addition, 16 joint-meters and 6 strain-meters were installed in the concrete membrane

3. Analysis and Results

3.1. GENERAL

As stated in the previous paragraphs, hardening soil model, a modified version of Duncan–Chang model, has been utilized in the analyses. Since no triaxial test results are available, pertinent works from the literature (Saboya and Byrne, 1993) are referred in selecting the hardening soil model parameters. Table 2 shows the range of parameters used in the preliminary analysis. In hardening model, the secant modulus for primary loading corresponding to 50% of shear strength q_f is defined by (Figure 3):

original Duncan–Chang (1970) model. Mohr–Coulomb failure criterion is used for evaluation of q_f in hardening soil model.

Zones 2A and 3A shown in Figure 1 are not expected to have a significant effect on the overall dam behaviour and have not been included in the finite element analysis.

In the analysis of rock-fill dams, cohesion of the rock-fill materials is generally taken to be zero. However, in this study, cohesion is taken as 1 kPa, as suggested in the program manual to improve computational performance.

The rock foundation of the dam is assumed to be rigid. Thus, at the foundation level, no horizontal or vertical settlements are computed. The analyses are carried out considering both end of construction (EoC) and reservoir full (RFC) conditions.

3.2. FINITE ELEMENT MODEL

The dam is analysed by assuming 2-D plane strain conditions. The finite element program PLAXIS is employed. The rock-fill embankment is modelled by 15-node triangular elements with the hardening soil as the material model. These elements have 12 interior stress points located at different coordinates from the element nodes where displacements are out put. The concrete membrane is modelled by five noded linear elastic beam elements. These elements have 50 cm thicknesses and 8 stress points. The material parameters of the concrete are shown in Table 3.

For a preliminary analysis, each construction stage was represented by a 10-m thick layer. It was observed that reducing the layer thickness renders the simulation better while extending the run time. Finally, a layer thickness of 5 m was decided as agreeable. The last 6 m’s of the dam from the top was not included in the mesh, since the parapet wall unnecessarily complicated the geometry, nevertheless the weight of the wall was taken into account in the model as a surcharge of 100 kPa. The finite element mesh is shown in Figure 4. This mesh consists of 15,417 nodes and 22,608 stress points.

3.3. RESULTS OF ANALYSES

3.3.1. *Settlements*

3.3.1.1. *End of Construction (EoC)*. Analyses were carried out using the range of parameters indicated in Table 2 for EoC condition, assuming that the rock-fill embankment consists of the material 3B only. Best results are obtained when the following parameters are selected:

Table 3. Linear elastic model parameters used in concrete membrane

Material	E (MPa)	d (m)	γ (kN/m ³)	ν
Concrete	28,500	0.50	23.50	0.20

Figure 4. Finite element mesh used in the analyses.

A comparison of measured settlements with the calculated ones using the material parameters shown in Table 4, are presented in Table 5.

It is seen from Table 5 that, there is an overall agreement except for the instruments located at El 555, where the predicted settlements are larger than the observed values. This discrepancy may be given to the cross-valley arching effect, which counteracts to reduce the settlements. Since the analysis is based on 2-D plane strain assumption, the arching effect can not have been reflected in the analysis.

Table 4. Parameters assigned to 3B materials, adopted from the preliminary analysis

γ (kN/m ³)	E_{50}^{ref} (kPa)	$E_{\text{oed}}^{\text{ref}}$ (kPa)	m	R_f	c_{ref} (kPa)	$\phi(^{\circ})$	$\psi(^{\circ})$
21	21,000	21,000	0.25	0.75	1	45	10

Table 5. Comparison of results for EoC (using only 3B material)

Axis	Instrument	Elevation (m)	Computed settlement (mm)	Observed settlement (mm)
A–A	ZDO-1	555.00	452	311
B–B	ZDO-11	575.00	680	609
C–C	ZDO-2	555.00	1323	1113
	ZDO-12	575.00	1397	1417
	ZDO-20	600.00	869	836
D–D	ZDO-3	555.00	1553	1460
	ZDO-13	575.00	1838	2019
	ZDO-21	600.00	1592	1592
	ZDO-30	625.00	771	621
E–E	ZDO-4	555.00	1623	1607
	ZDO-14	575.00	1971	2155
	ZDO-22	600.00	1822	1861
	ZDO-31	625.00	1099	717
F–F	ZDO-5	555.00	1491	1313
	ZDO-15	575.00	1712	1669
	ZDO-23	600.00	1360	1462

In the next step, 3C rock-fill material was also included in the model. Initially, it was expected that the material in zone 3C had a lower stiffness than 3B material, since it had been compacted in thicker layers than those of 3B material (Hunter and Fell, 2003).

An analysis was conducted by keeping the exponent m the same for Zones 3B and 3C material ($m = 0.25$) and selecting E_{50}^{ref} values as 19,000 and 23,000 kN/m² for 3C to represent different stiffness.

It was observed that, the computed settlements did not comply well with the measurements on axis F–F, which lies in zone 3C, when $E_{50}^{\text{ref}} = 19,000$ kN/m². It was found that, the settlements were in better agreement if E_{50}^{ref} was increased to $E_{50}^{\text{ref}} = 23,000$ kN/m². Table 6 shows the results of all three analyses.

3.3.1.2. Reservoir Full Condition (RFC). First impounding is a critical loading condition for dams, since most of the post-construction settlements take place at this stage during which reservoir water load causes both horizontal and vertical displacements in the embankment.

During impounding zone 3B deforms in unloading and the unloading modulus is suggested to be taken as a reasonable multiple of primary loading modulus (Nobari and Duncan, 1972; Fitzpatrick et al., 1985; Saboya and Byrne, 1993). However, this unloading behaviour cannot be incorporated in the analyses, since the available version of the computer program, PLAXIS 7.1 do not call for a separate unloading modulus.

The concrete membrane is assumed to be impervious, without cracks. The water pressure was applied as a triangular load distribution acting perpendicular to the concrete membrane. The results of this analysis are shown in Table 7.

Table 6. Comparison of results for different material properties assigned to zone 3C for EoC

Axis	Instrument	Elevation (m)	Observed Settlement (mm)	Zone 3C (Stiff) (mm)	Zone 3C (Weak) (mm)	Only Zone 3B (mm)
C–C	ZDO-2	555.00	1113	1347	1346	1323
	ZDO-12	575.00	1417	1426	1425	1397
	ZDO-20	600.00	836	882	877	869
D–D	ZDO-3	555.00	1460	1583	1624	1553
	ZDO-13	575.00	2019	1868	1916	1838
	ZDO-21	600.00	1592	1604	1662	1592
	ZDO-30	625.00	621	767	794	771
E–E	ZDO-4	555.00	1607	1621	1794	1623
	ZDO-14	575.00	2155	1977	2157	1971
	ZDO-22	600.00	1861	1822	1976	1822
	ZDO-31	625.00	717	1083	1151	1099
F–F	ZDO-5	555.00	1313	1465	1712	1491
	ZDO-15	575.00	1669	1687	1967	1712
	ZDO-23	600.00	1462	1337	1552	1360

Table 7. Comparison of computed and observed settlements for RFC

Axis	Instrument	Elevation (m)	Predicted Settlement (mm)	Observed Settlement (mm)
A-A	ZDO-1	555.00	433	371
B-B	ZDO-11	575.00	442	384
C-C	ZDO-2	555.00	166	44
	ZDO-12	575.00	246	173
	ZDO-20	600.00	334	238
D-D	ZDO-3	555.00	98	52
	ZDO-13	575.00	137	83
	ZDO-21	600.00	167	127
	ZDO-30	625.00	170	146
E-E	ZDO-4	555.00	65	28
	ZDO-14	575.00	87	78
	ZDO-22	600.00	103	73
	ZDO-31	625.00	108	143
F-F	ZDO-5	555.00	35	12
	ZDO-15	575.00	45	64
	ZDO-23	600.00	53	52

It is seen that maximum settlements occur close to the upstream membrane and diminish towards the downstream face, as expected. As can be seen from the table that computed settlements are somewhat larger than the observed values. This is attributed to the fact that, since the water load is applied as a surcharge the rock-fill material responds in primary loading, consequently primary loading modulus is used by the computer, whereas, in the actual case, unloading modulus must contribute to settlements up to a certain water level (Saboya and Byrne, 1993).

Table 8. Comparison of vertical total stresses for EoC and RFC

Axis	Instrument	Elevation (m)	Overburden stress (kPa)	Observed stress (kPa)		Predicted stress	
				EoC	RFC	EoC	RFC
C-C	BO-2	555.00	1029	931	1029	1135	1357
	BO-10	575.00	609	623	684	723	900
E-E	BO-3	555.00	1995	—	—	1577	1645
	BO-11	575.00	1575	1123	1266	1245	1287
	BO-16	600.00	1050	896	927	831	846
F-F	BO-4	555.00	1375	1024	997	1377	1410
	BO-12	575.00	955	927	943	998	1023
G-G	BO-1	555.00	—	40	230	49	765
	BO-9	575.00	—	65	258	50	571
	BO-15	600.00	—	135	415	52	329
	BO-20	625.00	—	27	33	54	91

3.3.2. Total Stresses

The comparison of observed and predicted total stresses for EoC and RFC are given in Table 8. Also included in this table are the overburden stresses.

An examination of Table 8 shows that, for EoC, predicted stresses and readings are close to each other, in general. It is reported that instrument BO-3 failed recording during construction, therefore readings are missing. There is an unexpected discrepancy for the instrument BO-15, which lies on G–G axis, just beneath the concrete membrane, probably caused by improper installation.

In case of RFC, measured and predicted stresses are also in compliance, except for the instruments on axis G–G, which is parallel to the concrete membrane (i.e. BO-1, BO-9, BO-15, and BO-20). These deviations are given to the fact that these instruments were relocated prior to the construction of concrete facing, consequently prior to impounding.

During RFC, the stresses increase as the water load increases. The stress increase is pronounced more in regions close to the upstream membrane and negligible in regions close to the downstream face. It is also seen that the differences between predictions and observations are somewhat larger than those for EoC, which may be explained by the softening effect of water on the rock-fill materials (Nobari and Duncan, 1972).

3.3.3. Stress and Displacement Contours

Figures 5–12 show the stress distribution both for EoC and RFC. The effect of reservoir filling can clearly be seen at the upstream half of the dam. Horizontal stresses display a similar distribution as the vertical stresses. In particular, reservoir water load pushes the dam towards downstream which results in positive shear stresses both in upstream and downstream halves of the dam, as pointed out in (Khalid et al., 1990).

Figures 13–16 illustrate the computed vertical and horizontal displacements. As can be seen from the figures, for EoC, the upper part of the upstream face tend to move downstream while the lower part displays the opposite behaviour. This behaviour is the same but mirrored at the downstream half. Since horizontal

Figure 5. Horizontal stresses for EoC (stresses are in kPa, “–” indicate compression).

Figure 6. Vertical stresses for EoC (stresses are in kPa, “-” indicate compression).

Figure 7. Shear stresses for EoC (stresses are in kPa).

Figure 8. Stresses in out of plane direction for EoC (stresses are in kPa “-” indicate compression).

Figure 9. Horizontal stresses for RFC (stresses are in kPa, “-” indicate compression).

Figure 10. Vertical stresses for RFC (stresses are in kPa, “-” indicate compression).

Figure 11. Shear stresses for RFC (stresses are in kPa, “-” indicate compression).

Figure 12. Stresses in out of plane direction for RFC (stresses are in kPa “-” indicate compression).

Figure 13. Horizontal displacements for EoC (displacements are in cm) (downstream is positive).

Figure 14. Vertical displacements for EoC (displacements are in cm) (“-” indicates settlement).

Figure 15. Horizontal displacements for RFC (displacements are in cm) (downstream is positive).

Figure 16. Vertical displacements for RFC (displacements in cm) (“-” indicate settlement).

displacement behaviour of Kurtun dam was not monitored, we do not have the opportunity of comparing the results with the observations.

In the upstream part, computed maximum horizontal displacement is 19.5 cm at El 551.2, located at 81.80 m upstream of the dam axis. In the downstream part, computed maximum horizontal displacement is 17.3 cm at El 550.00, located at 89.30 m downstream of the dam axis. When compared with vertical displacements, calculated horizontal displacements are relatively small.

For RFC, maximum horizontal displacement is found to be 36.09 cm on concrete membrane at El 560.63. Computed horizontal displacements compare to those in previous studies (Nobari and Duncan, 1972). The computed maximum settlements are 199.76 cm at El 580.00 for EoC and 52.79 cm at El 565.63 for RFC, respectively.

4. Conclusions

In this study, a two dimensional finite element analysis of CRF Kurtun dam is carried out and the computed displacements and internal stresses compared with those measured *in situ*. The rock-fill material is represented by the hardening soil model which is a modified version of Duncan and Chang's hyperbolic model. The material parameters are adopted from the available data from the literature. The structure is analysed for both EoC and RFC conditions. The results obtained from two dimensional plane strain non-linear finite element analysis using hardening soil model are found to be in relatively good agreement with the *in situ* readings. The computer program PLAXIS can satisfactorily reflect the deformational characteristics of rock-fill masses. Since the valley is narrow; the cross-valley arching effect should be significant in reducing the vertical displacements. It has been seen that, when a reduced modulus is assigned to zone 3C fill, computed settlements deviate more from those observed; this suggests that deformational characteristics of zone 3C material, when well compacted, can be quite similar to those of zone 3B material.

It is believed that monitoring lateral deformations and stresses in future constructions will throw more light in understanding the actual behaviour of CFRDs.

The study indicates that the method followed can be used to predict performance of concrete faced rock-fill dams for end-of-construction condition and for reservoir filling, with some confidence.

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