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Stress-Strain Analysis of Different Concrete Cut Off Wall Connection Systems Case Study: Karkheh Storage Dam

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ABSTRACT

The concrete cut off wall is usually used to control the seepage through the foundation of earth dams. It is usually made of plastic concrete. It can be connected to the core of the earth dam using different connection systems. The difference between cut off wall and core material results stress concentration in the connection zone. In fact the main cause is the large difference between stiffness of the cut off wall concrete and clayey core of the dam. This makes the stress-strain behavior complex in the connection zone. In the present study, six different details for the connection system of the cut off wall to the earth dam core were identified. The Karkheh storage dam in Iran with a plastic concrete cut off wall was selected as the case study for investigation of the behavior of different connection systems. The connection systems were modeled and the stress-strain behavior in connection zones was analyzed at the end of dam construction and steady state seepage through the dam. Eventually, the most appropriate connection with the best stress-strain distribution was determined. According to the results the stress strain behavior of different types of cut off connection systems are different. Indeed, the kind of connection between the cut off wall and the core influences the stress-strain distribution in the connection zone drastically. According to the results, the system No. 2 (thick concrete slab at the base level of the core) causes a better stress-strain distribution compared to the other methods of cut off wall connection with the core.

INTRODUCTION

Dam foundation water tightening is applied to control seepage and to reduce uplift pressure under the dam and appurtenant structures, to prevent sliding of downstream structures on weak ground layers. Cut off wall is one conventional method for water tightening of large dam foundation (Shadravan et al., 2004).

The cut off walls are used extensively in cases where using other methods of water tight of foundation, due to high permeability of soil or high groundwater head, is impossible (Millet et al., 1992). The filling materials used in these walls distinguish them from each other, mostly. One of the most common kinds of these walls is the plastic concrete cut off wall which consists of water, aggregate, cement and bentonite.

One of the most important issues in finding appropriate mix design of plastic concrete used in the cut off walls under the dams, especially the dams with a sizable height, is to make a mixture which is not only resistant enough against hydraulic erosions under high gradients and have durability and a steady state during the dam's exploitation, but also has to be able to concord with the deformation of the dam's foundation. It means that the stresses and strains, which the dam's foundation goes, though in the loading of construction, impoundment, and exploitation would be applied to the cut off wall in the foundation, as well. And if the materials in the wall could not stand these stresses and strains, there will occur cracks in the wall and losing sealing will be probable (Shahbazian Ahari et al., 2000).

Also, because there is a high hydraulic gradient in the connection of the cut off wall and the clayey core, and because there will be noticeable settlement in the wall itself due to the time factor and the pressure from the above backfill; erosion, leakage, and cracks in the wall are very probable (Shahbazian Ahari, 1999). Therefore, another factor that should be considered in designing any connection system of the dam's foundation and especially the cut off wall is the connection of the wall and the body of the dam. Joining the connection system of the foundation and the dam should be designed to control the leakage in that area and avoid breakage and separation of the wall and the dam. This may be reached by different details for connection system as follows:

- Penetration of the cut off into the core
- Thick concrete slab at the base level of the core
- Combination of cut off penetration into the core and the concrete slab

- Compaction grouting around the connection zone in foundation
- Clayey soil besides a concrete cap
- Clayey trench

Considering the connection systems mentioned above and the difference in stiffness and deformability of the cut off wall, core and foundation in the connection zone, there is the probability of stress concentration and unequal settlements in different loading stages of the dam. The unequal settlements and stress concentration may cause the weak operation of the cut off wall and the core, and eventually a weak and malfunctioned connection system.

The present study, therefore, deals with evaluation and comparing the stress strain behavior in different, above-mentioned connections zones with modeling and numerical analysis. The purpose of the study is to clarify the condition of the stresses and strains induced in the area of the connection at two levels of end of construction and steady state seepage in order to identify the connection with the best operation.

CASE STUDY

Karkheh storage dam is the largest dam, in terms of reservoir and volume of fill placed, constructed in Iran. It is a central Core, zoned embankment dam 127 meters high, 3030 meters long, with an embankment volume of 32 million cubic meters. The dam crest is located in +234 MSL and the minimum level of the foundation is +106 MSL. The normal water level is in +220 MSL (Mahab Ghodss Consulting Engineers, 1998).

The dam Foundation water tightness is mostly achieved by means of a cut off wall. The characteristics of the cut off wall material (plastic concrete) were assigned in such a way to ensure the required impermeability, deformability and strength. The final wall surface area is about 150,000 square meters. Moreover, the foundation's depth is varied in different places based on the location of impermeable layers. The depth of the wall is determined based on the seepage analysis done in different stages and economical factors and the wall's thickness is determined based on the allowable hydraulic gradient, hydraulic fracturing pressure, and the drilling facilities. Therefore, the depth of the wall in deepest section is about 80 meters while the average of depth is about 50 meters. With a length of 3030 m, it was vertically built in the dam foundation along the dam axis. The wall thickness is 1 meter at the valley and in the right abutment. At some location of the left abutment, the thickness of the wall is chosen to be 0.8 meter (Shadravan et al., 2004 and Mahab Ghodss Consulting Engineers, 1998).

In addition, the foundation of this large earth dam consists of alternative layers of conglomerate and mudstone, in which the conglomerated layers have much more impermeability, resistance and elastic modulus than the mudstone layers. Figure 1 depicts the cross section of the dam and its foundation and cut off wall. Table 1 shows the parameters and specifications of the average material used, which are obtained from laboratory tests, field tests and back analyses for the materials in Karkheh earth dam, its foundation, and cut off wall.

NUMERICAL MODELING

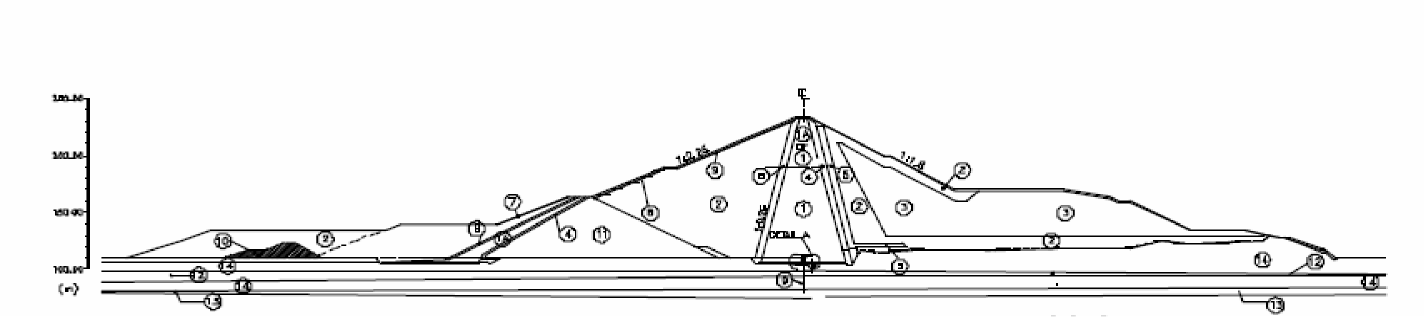
The dam, foundation and cut off wall were modeled with PLAXIS software in the largest section (Fig. 1). In this section, the cut off wall stretches 25.5 meters below the core and is fixed in a mudstone layer.

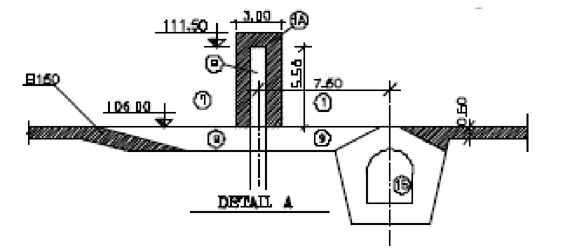
The specifications of the materials used in modeling is shown in Table1. In order to control the parameters used in modeling, the settlement of the axis of core was contrasted with the instrumentation used in the core of the dam and also the results of the software CA2 (Niromand, 1999). The results confirm that there is a considerable agreement in these three cases as shown in Fig. 2. Therefore, it can be concluded that there is a considerable consistency between the parameters used in modeling and the real characteristics of the materials in the dam.

The stress-strain analysis was performed for the two phases. The end of construction and steady state seepage through the dam were considered as the stages of the analysis. It was assumed that the dam is constructed in 15 layers. Moreover, elastic-plastic Mohr-Coulomb model was considered for the soil. Figure 3 shows the finite element mesh generated in the stress-strain analysis.

In order to analyze the stress-strain behavior in different connection systems, the six connections shown in Fig. 4 were modeled. All of the connection systems are numbered. These numbers are representative of each connection system in this study. Geometric details used in modeling all these connections are depicted in Table 2. The following assumptions are considered in numerical modeling:

- The cut off wall width is considered 1 meter in all cases.
- The Same parameters are used in the cut off wall and the concrete slab.
- Due to the lack of test data, some of the parameters considered in connections No. 4, 5, and 6 are assumed according to the specifications of materials in the core and foundation. These data can be seen in Table 3.
- The properties of the material are the same in the grouted zone.
- The cap has 3 meters length from both sides and a thickness of 1 meter in the fifth system and the material used in the cap is chosen the same as what is used in the cut off wall.
- The trench width is 3 meters in bottom and the slope of its walls is 3V:1H.





1. Impervious core (mudstone mixed with sandy gravel)
1A. Impervious core (mudstone)
2. Sandy gravel
3. Conglomerate or sandy gravel
4. Sand filter
5. Gravel filter and drain
6. Sand-gravel filter
7. U/S slope protection using limestone riprap
8. U/S slope protection using soil cement
9. Plastic concrete cut off wall
10. Pre-coffer dam
11. Main cofferdam
12. Mudstone No. (-1)
13. Mudstone No. (-2)
14. Conglomerate
15. Inspection gallery

Fig. 1. Cross section of Karkheh storage dam (Mahab Ghodss Consulting Engineers, 1995)

Table 1. The parameters and specifications of different parts of the dam (Mahab Ghodss Consulting Engineers, 1998)

Parameters	Shell	Core	Filter	Cut off wall	Mudstone layers	Conglomerate layer (1)	Conglomerate layer (2)	Conglomerate layer (3)
Dry unit weight (kN/m^3)	20	17.4	19	21	19.5	21	21	21
Saturated unit weight (kN/m^3)	22	20.2	20	22	21	23	23	23
Permeability coefficient (cm/s)	10^{-4}	5×10^{-7}	10^{-3}	1×10^{-7}	5×10^{-8}	4.5×10^{-2}	1.1×10^{-3}	6.1×10^{-4}
Elastic modulus (kN/m^2)$\times10^4$	11	3.5	7	400	12	80	100	100
Poisson’s ratio	0.25	0.35	0.27	0.25	0.3	0.25	0.25	0.25
Undrained cohesion (kN/m^2)	-	70	-	800	-	-	-	-
Drained cohesion (kN/m^2)	0	30	0	700	70	85	85	85
Undrained friction angle (degree)	-	6	-	28	-	-	-	-
Drained friction angle (degree)	39	20	35	33	22	39.4	39.4	39.4
Dilation angle (degree)	10	2	8	10	5	10	10	10

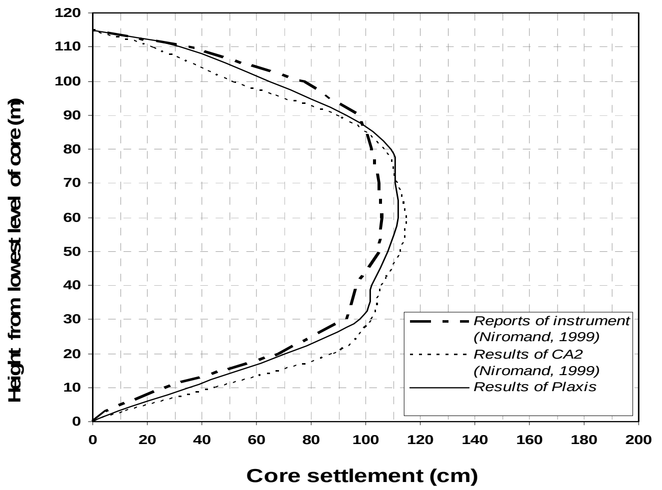


Fig. 2. Settlement changes in various level of the core axis in 115 meters embankment height above the foundation

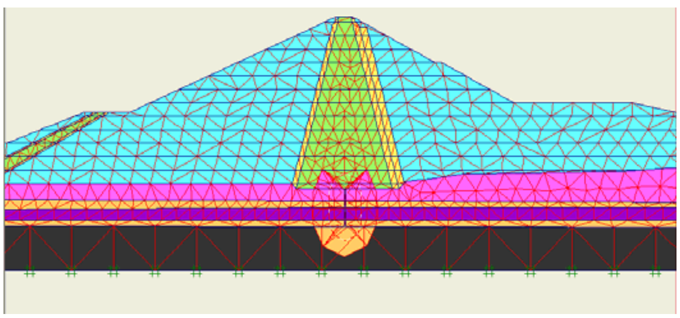


Fig. 3. Finite element mesh generated for the dam

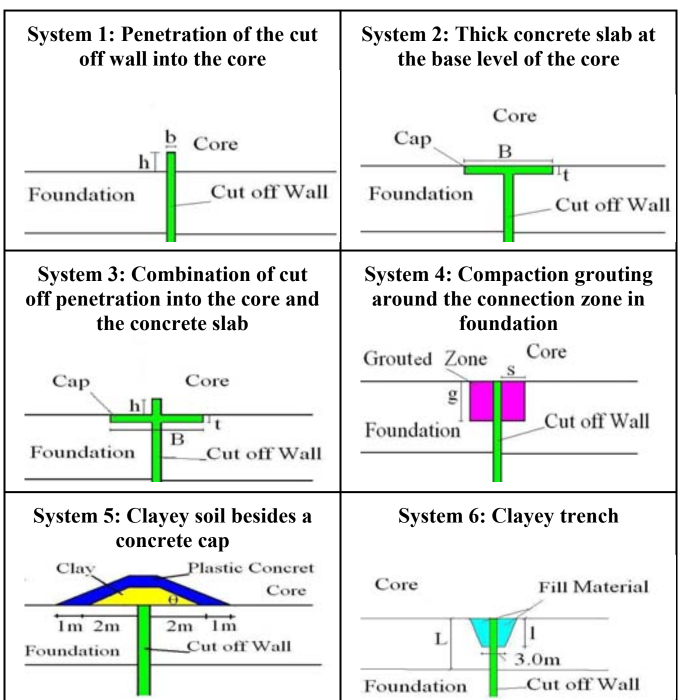


Fig. 4. Different connection systems

Table 2. Geometric details of different connection systems

Connection system No.	Geometric details
1	h/H=1/30
2	B=4 (m) , t=1 (m)
3	B=4 (m) , t=1 (m) , h/H=1/30
4	S=2 (m) , g=2 (m)
5	θ=30º
6	l=4 (m)

Table 3. Properties of the materials used in connections No. 4, 5, and 6.

Parameters	Grouted zone	Plastic clay	Fill material
Dry unit weight (kN/m^3)	22	16.5	19
Saturated unit weight (kN/m^3)	23.2	18.9	21
Permeability coefficient (cm/s)	1×10^{-4}	5×10^{-7}	5×10^{-7}
Elastic modulus (kN/m^2)$\times10^4$	150	1	10
Poisson's ratio	0.22	0.4	0.3
Undrained cohesion (kN/m^2)	86	90	40
Drained cohesion (kN/m^2)	75	60	25
Undrained friction angle (degree)	32	4	18
Drained friction angle (degree)	40	15	26
Dilation angle (degree)	10	0	3

EVALUATION OF THE RESULTS

As it was mentioned earlier, in the present study, two stages namely the end of construction and the steady state seepage are taken into consideration. In this research, we evaluated and contrasted maximum total and effective stress and strains in the connection zones of different systems. Stress points taken into consideration in the core for the present analysis are maximum 0.5 meter distant from the connection zone. Moreover for better analyzing of the results, the cases in which the wall is connected to the core with no system, is also reported.

Stresses

In order to study the stress condition in two stages of the end of construction and steady state seepage, the shear and principal stresses in the core at the mentioned two stages were compared in different connection zones. In all connections, the principal stresses are compressive.

Sheer stress: As it can be seen in Fig. 5, the maximum shear stresses increases in different connection systems compared to the case of no connection system. Comparing different connections it was observed that the least shear stress occurs in connection 2 and the most shear stress occurs in connection 3 at the end of construction state. Also the least shear stress occurs in connection 4 and the most shear stress occurs in connection 5 in case of the steady state seepage.

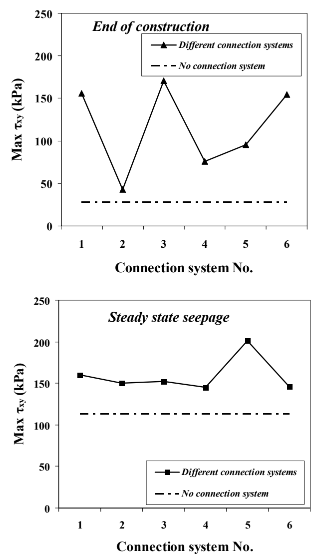


Fig. 5. Comparison of shear stresses in different connection zones at the two phases of the end of construction and steady state seepage

Principal stresses: As it is shown in Fig. 6, the major principal stress (σ_1) increases when a connection system is used. However, the least value is associated to connection systems 2 and 4.

Moreover, according to Fig. 7, the total and effective minor principal stress (σ_3) in the two stages of analysis occurs in different connections is less than total and effective minor principal stress induced when no system is used.

Comparing the results for different connection systems it can be seen that the least minor principal stress induces in connection 5 at the end of construction. However, the most value induces in connection system 2 at this stage.

Also the least total and effective minor principal stress occurs in connection system 6 and the most values occur happens in connection 2 in case of the steady state seepage.

As it is shown in this figure, the minor principal stress is always compressive. Therefore it can be concluded that there is no possibility for hydraulic fracture occurrence.

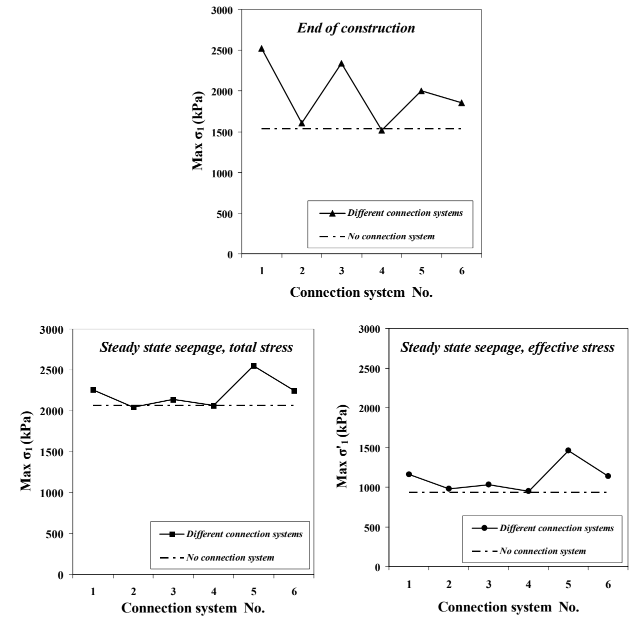


Fig. 6. Comparison of major principal stress in different connection zones at the two phases of end of construction and steady state operation.

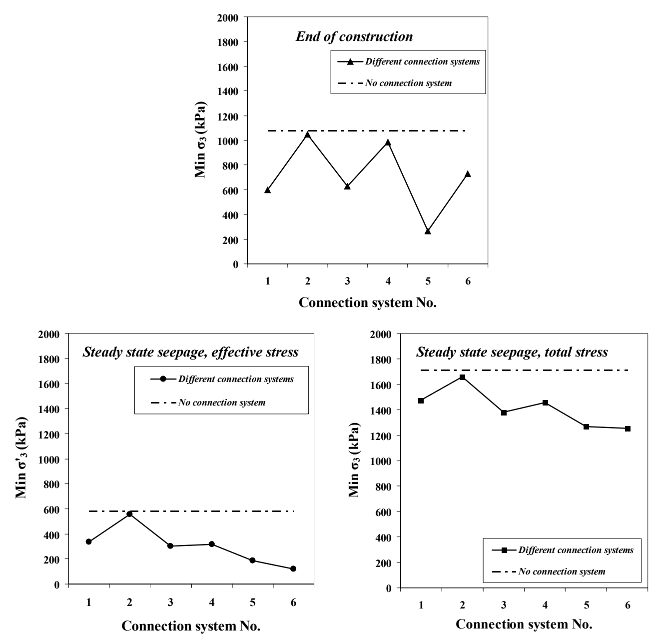


Fig. 7. Comparison of minor principal stress in different connection zones at the two phases of the end of construction and steady state seepage

The maximum of the ratio of major principal stress to minor principal stress (σ_1/σ_3) in both stages of analysis is shown in Fig. 8. According to this figure, the minimum of the ratio occurs in connection 2 at the end of construction. It is the same for the case of the steady state seepage. However, the ratio of the total stresses remains nearly the same for different connections in this stage.

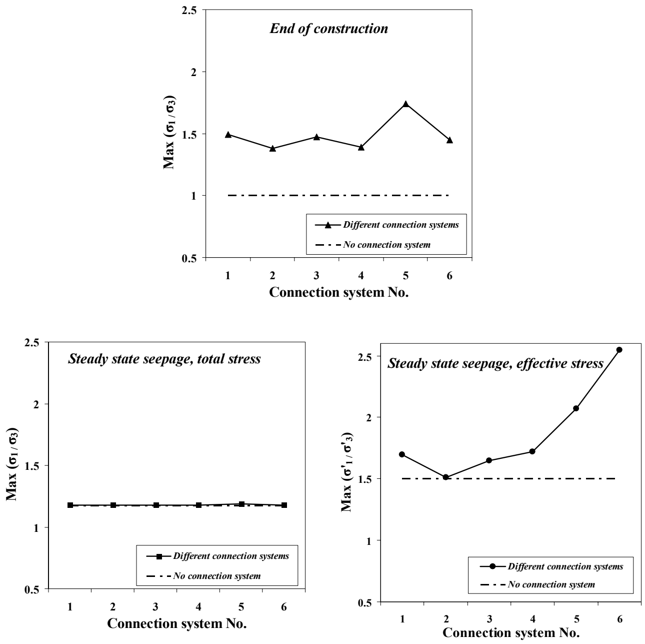


Fig. 8. Comparison of the ratio of major principal stress to minor principal stress in different connection zones at the two phases of the end of construction and steady state seepage

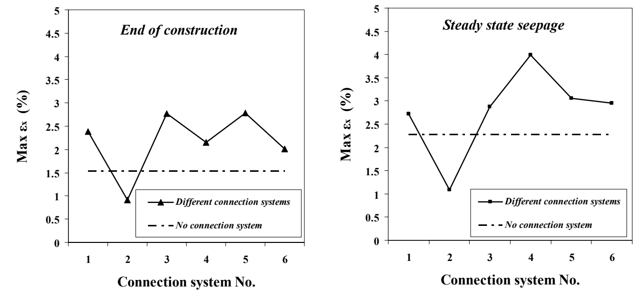


Fig. 9. Comparison of horizontal strains in different connection zones at the two phases of the end of construction and steady state seepage

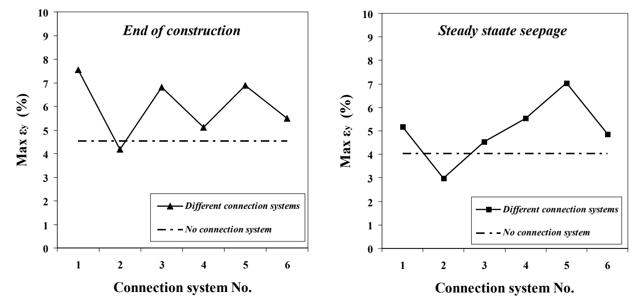


Fig. 10. Comparison of vertical strains in different connection zones at the two phases of the end of construction and steady state seepage

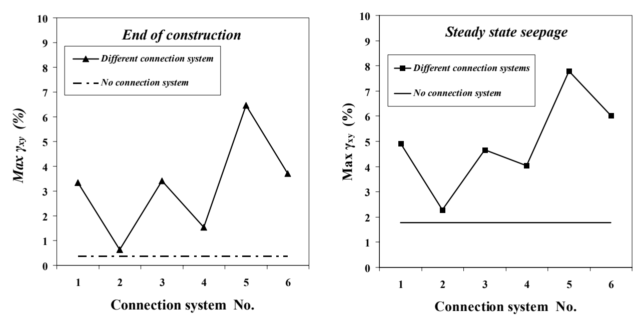


Fig. 11. Comparison of shear strains in different connection zones at the two phases of the end of construction and steady state seepage

Strains

The components of normal and shear strains are compared in different connection systems at the end of construction and steady state seepage. The results of these analyses are as bellows:

Vertical and horizontal strains: Figure 9 shows that the maximum vertical strain in the two stages of analysis is less in connection 2 compared to the other ones. This is also correct for the horizontal strains as shown in Fig. 10.
The maximum vertical and horizontal strains in this type of connection are even less than the case without any connection system.
The most vertical strains occur in connections 1 and 5 at the end of construction and the steady state seepage respectively. Moreover, the maximum strains occurred in connections 4 and 5 at the end of construction and the steady state seepage respectively.

Shear Strain: As it can be seen in Fig. 11, the maximum shear strains, in the two stages of analysis in different connection zones are more than the maximum shear strains when there is no connection system.
Comparison of different connection systems shows that the least shear strain occurs in connection 2 and the most shear strain occurs in connection 5.

Factors of safety against shear stresses

The following equations were used to determine the factor of safety values against the shear stresses in different connection systems:

$$F.S = \tau_{all.} / \tau_{ext.} \quad (1)$$

$$\tau_{all.} = \sigma_n . \tan(\varphi) + c \quad (2)$$

$$\sigma_n = (2\sigma_1 . \sigma_3) / (\sigma_1 + \sigma_3) \quad (3)$$

In these equations $\tau_{ext.}$ is the existing shear stress and σ_1, σ_3 are the maximum and minimum principal stresses in each connection zone. Moreover, σ_n is the normal stress and τ_{all} is the allowable shear stress. Using the above-mentioned formula, the minimum value of the factor of safety against shear stress is calculated in each connection zone at the two phases of the end of construction and the steady state seepage. Figure 12 shows the computed factor of safeties for different connection systems. According to this figure, the maximum value of the factor of safety is associated to the connection system 2 at the end of construction. However, the factors of safety values are nearly equal at the steady state seepage phase for different connection systems.

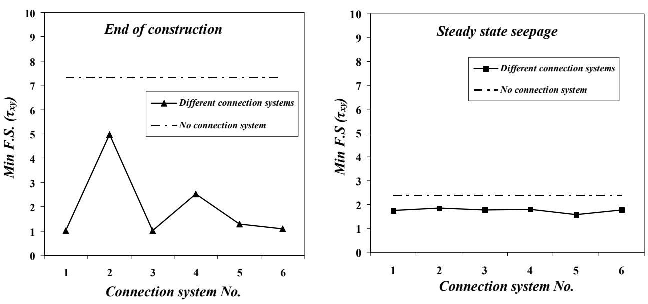


Fig. 12. Comparison of factors of safety against shear stress for different connections at the two phases of the end of construction and the steady state seepage

Investigation of the failure criterion

Figure 13 shows the status of principal stresses for all stress points considered in connection systems. The Mohr-Coulomb failure criterion is also drawn for the core material. This figure is used to determine the percentage of plastic points in each connection system.
The ratio of the number of plastic points to the total stress points considered in each connection system is determined and the results are shown in Fig. 14. As indicated in this figure, the minimum percentage of the plastic points occurs in the connection system 2. This figure also shows that the maximum percentage of plastic points occur in connection system 5.

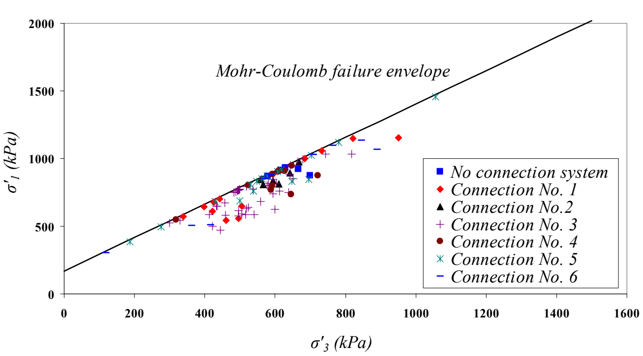


Fig. 13. Investigation of the failure state for different connection systems

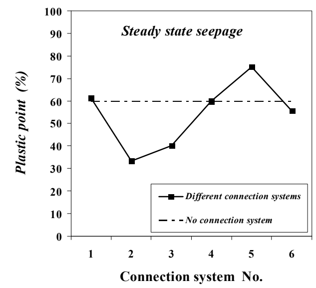


Fig. 14. Percentage of plastic points in each connection system

Comparison of different connection systems

Tables 4 and 5 show a summary of the results of stress-strain analysis performed on different cut off wall connection systems in Karkheh dam at the end of construction and steady state seepage.
The results show that connection system No. 2 which consists of a thick concrete slab at the base level of the core has a smoother stress distribution and lower shear stress. Moreover, consideration of the strains at the end of construction and steady state seepage show lower deformations for this connection type. Also the factor of safety against the shear stresses was the most for this connection system among the other ones with the least plastic points.
Therefore, it seems that, the least stress and strain concentration would occur in this connection system and it has a steadier state compared the other ones. Although, the studies in this paper refers only to the static loads and the dynamic analysis should also be taken into consideration.

Table 4. Summaries of the results of stress-strain analysis on different connection systems in Karkheh dam at the end of construction phase

Connection System	1	2	3	4	5	6
min(τ_{max})		#				
min(σ_1)		#		#		
max(σ_3)		#				
min($\sigma_1/\sigma_3)_{max}$		#				
min($\varepsilon_{x, max}$)		#				
min($\varepsilon_{y, max}$)		#				
min($\gamma_{xy, max}$)		#				
max(F.S.)		#				

CONCLUSIONS

1- Regarding to the Mohr-Coulomb's failure envelope, it can be seen that all points are in shear failure mode and no point is in tension failure mode.

2- The minor principal stress is compressive in all connection systems which shows that there is no probability of hydraulic fracture occurrence in connection zone.

Table 5. Summaries of the results of stress-strain analysis on different connection systems in Karkheh dam in the steady state seepage phase

Connection System	1	2	3	4	5	6
$\min(\tau_{max})$				#		
$\min(\sigma_1)$		#		#		
$\max(\sigma_3)$		#				
$\min(\sigma'_1/\sigma'_3)_{max}$		#				
$\min(\varepsilon_{x,max})$		#				
$\min(\varepsilon_{y,max})$		#				
$\min(\gamma_{xy,max})$		#				
max(F.S)		#				
Max. plastic points		#				

3- The results of the stress-strain analysis at two phases of end of construction and steady state seepage show that the failure of the core does not occur for any of the connection systems.

4- Comparison of the results show a smoother stress distribution, lower shear strains, more factor of safety and less plastic points for connection system 2 which consists of a concrete slab at the base level of the core. This kind of connection is recommended for a better stress-strain distribution based on the static analysis results. However, dynamic analysis is required to investigate this subject.

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