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Seepage analysis underneath the headwork of Chanda Mohana Irrigation Scheme, Sunsari, Nepal

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Abstract

Seepage is one of the major causes of failure of irrigation headworks constructed on permeable foundation in the Terai (plain) Region of Nepal. There is an urgent need to upgrade these schemes in order to meet the increasing food demand. A seepage problem was seen at the headwork of Chanda Mohana Irrigation Scheme, Sunsari situated at the confluence of Budhi and Katle Streams from the very beginning of the commissioning in 2000. The headwork consists of a weir (65 m span) with under-sluice (7 m opening). This research was focused in the seepage analysis underneath the headwork.

MSeep model (2-Dimensional) was used for seepage analysis. Different scenarios were developed to represent the different works carried out in 2000, 2009 and 2010. The results indicate that the exit gradient was within the range of permissible safe exit gradient without considering the lateral flow. In contrast, the exit gradient was increased above the maximum permissible limit considering lateral flow. Due to continuous seepage flow, the horizontal and vertical hydraulic conductivities in 2009 might have been increased to 2.5 times and 1.3 times respectively than the values in 2000. This has reduced the rate of head-loss and hence increased the exit gradient. Therefore, this is one of the major causes for the failure of the structure in 2010 regardless of the maintenance carried out in 2009.

The research finding suggests that the ongoing maintenance work from 2010 is safe only if the pressure grouting recovers the soil parameters to the stage during construction in 2000, which is generally not achievable. So, considering the hydraulic conductivity and porosity of the soil as the average values of 2000 and 2009, extension of the downstream floor with additional sheet piles is proposed as an alternative measure for achieving the safety of the structure.

Keywords: Seepage analysis, Piping, MSeep model, Chanda Mohana Irrigation Project

1 Introduction

Importance of water resources has been recognized by Nepalese farmers and hence they have been constructing irrigation systems at their own initiative since long ago (Pradhan, 1988). Agricultural sector plays a crucial role in the Nepalese national economy providing employment opportunities to 66 percent of the total population and contributing about 39 percent in the GDP (http://www.doanepal.gov.np, 2010). Although the Nepalese economy mainly depends on agriculture, the optimal production is not being achieved because of failure of irrigation structures, mainly headwork. These schemes should be upgraded to meet the increasing food demand. 80 - 90% of the required increase in food production will have to be realized on existing cultivated land in coming decades (Schultz *et al.*, 2009). The Terai (plain) Region is suitable for agricultural production. Seepage is one of the major causes of failure of irrigation headworks constructed on permeable foundation in the Terai (plain) Region of Nepal.

Failures of dams by piping action have occurred since the earliest dams were constructed around 2900 BC. Out of all dam failures in US, 20% are due to the problem of piping (http://www.damsafety.org/news/?p=412f29c8-3fd8-4529-b5c9-8d47364c1f3e, 2010). 46% of all large embankment dam failures up to 1986, excluding dams constructed in Japan pre 1930 and in China are related with piping failure (Foster *et al.*, 2000). According to the research done by Richards *et al.*, (2007), the recent data from National Performance of Dams Program together with Jones (1981) and Lane (1934) shows that about 267 dams have failed due to piping failure. Cracks and fissures, weakening of the shell material due to burrowing animals, root growth penetration from vegetation and internal erosion or seepage are some causes of piping failure (Moayedi *et al.*, 2011).

The CMIS[5], located in Sunsari district within the Koshi zone of Eastern Development Region of Nepal had seepage even at the very beginning of commissioning of the headwork in 2000 (CMIP, 2001). It was constructed

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in 2000 to irrigate 1800 hectares of command area. The headwork of CMIS lies at the confluence of Budhi and Katle Stream. Initially, the confluence point of Budhi, Katle and Jala Stream was at the same location where the present east (left) head regulator exists. From the construction of headwork, the Jala Stream is being diverted towards the East which meets two streams at the downstream of the headworks. Just after the operation of canal in October 2000, soil subsidence was seen near the Eastern head regulator and water boils with fine sands were seen at the downstream of sheet pile line in left under-sluice portion. After the immediate maintenance in 2000, again piping problem was seen in 2009. Although some temporary measures were taken in 2009 to save the structure from piping, the vulnerable condition was noticed on the first week of January 2010. It was seen that the weir structure with it's under-sluice had piping problem. The upstream floor at the left under-sluice was cracked and settled forming a ditch of size 5 m x 3 m x 2 m. Hollow formation was seen beneath the left abutment wall near the settled upstream floor (Prasad, 2010). Hollow formation was also noticed from the upstream floor in weir portion nearby left divide wall to the settled floor at the upstream under-sluice portion. Furthermore, the left abutment wall was settled and tilted towards the East. As a result, the gate at the under-sluice portion was also tilted towards the East and the gap in between the bridge decks above the weir is also increased. It is found that major maintenance works were carried out in the headwork of CMIS in 2000, 2009 and 2010 even after the completion of the scheme.

Overall span of weir = 65 m, Weir portion: clear span = 40 m, crest height = 3 m, gate height = 1.5 m
Undersluice portion: opening = 7 m (on both banks), crest height = 2 m, gate height = 2.5 m

The reduced level of pond level by closing the gates is 70.0 m. Furthermore, the reduced level of downstream deepest floor is 63.9 m. Thus, the head difference is 6.10 m. Floor length at the under-sluice portion and weir portion is 49.75 m and 44.75 m respectively. The sheet piles are provided only at the end of upstream and downstream of floor but not in the sides of the abutment wall. This shows that the sheet piles are not closed. Location and layout map of CMIS is shown in Figure 1 and Figure 2 respectively.

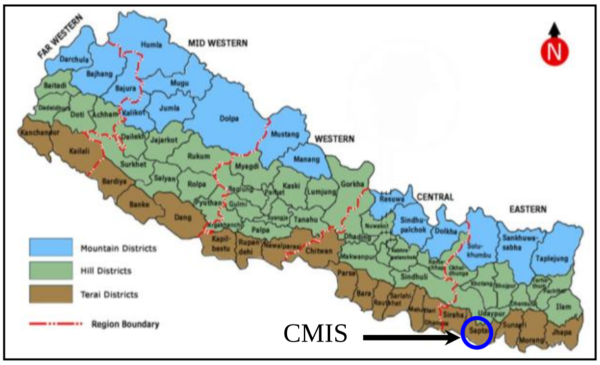


Figure 1: Location map of CMIS Figure 2: Layout map of CMIS after failure(Google-Earth, 2010)

2 Development of Problem and Works Carried Out

In 2000

Just after the operation of canal in October 2000, soil subsidence was seen near the Eastern head regulator. It was thought that the subsidence was local due to insufficient compaction of soil. Backfilling was done in the subsidence area. Canal was operated for 10 days as per crop water requirement. Again soil subsidence was seen near previous subsidence area when canal was operated for about a week in December 2000 for wheat. The subsidence area was backfilled. In this period, water boils with fine sands were seen at the downstream of sheet pile line in left under-sluice portion. At every watering period of wheat, due to water ponding in the upstream of weir, soil subsidence was seen near head regulator but entry point of seepage water and seepage path was not known. Considering the suggestions given by different experts, laying of geo-membrane up to 25 m length upstream at existing upstream floor level to increase seepage path by 25 m and 0.3 m earth filling with compaction was done. Geo-membrane was loaded with 0.3 m thick clay and gabion of 0.3 m thick. Excavation in the subsidence area up to depth of seepage hole and filling with clay cement mixture with layer wise compaction, earth fill in old course of Jala Khola was done. Fixation of geo textiles at the joint of abutment wall and stone masonry wall with nut bolts up to height of 2.5 m was also done (CMIP, 2001).

[5] Chanda Mohana Irrigation Scheme, Sunsari, Nepal

In 2009

Water boiling from the concrete blocks near divide wall at the end of downstream sheet pile line of left under-sluice portion as well as at the end of downstream sheet pile line in weir portion was observed. It was estimated that seepage water was passing from upstream sheet pile line and was escaping from downstream sheet pile line. The maintenance work was carried out for 30 m length upstream from end of upstream sheet pile line to increase the seepage path by 30 m (CMIP, 2009).

In 2010

It was seen that the weir structure with its under-sluice had piping problem where water was seeping through downstream floor. The boiling at the downstream was seen. It had created big hollow at the upstream floor near the left divide wall in under-sluice portion and the upstream floor was cracked and settled (Prasad, 2010).

According to the engineers involved in that scheme, the settlement had an area of about 5 m x 3 m at the upstream of under-sluice portion. After diverting the water into the Jala Khola, it was noticed that the height of hollow was about 2 m and the hollow was beneath the foundation of abutment wall too. Furthermore, the hollow was also formed not only towards divide wall but also towards the downstream portion as well. The connection of hollow along upstream of weir portion near divide wall, divide wall, under-sluice portion and abutment wall was observed. The red rectangular area in Figure 3 shows the hollow formed area at upstream floor. The settlement of abutment wall also observed. Due to this, the gate of under-sluice portion was also tilted towards east. In addition to this, the gaps in the joints of the bridge deck were also increased due to the settlement of floor and abutment wall and this is shown by red rectangles in Figure 4.

Figure 3: Hollow formed area at U/S floor

Figure 4: Increased gap in bridge deck slab joints

3 Exit Gradient and Factor of Safety

The submerged weight of soil is the resultant of force of gravity and the force of buoyancy. The force of gravity, W acts in downward direction.

$$\boldsymbol{W = \gamma_w * (1 - \varepsilon) * S_s} \quad \textbf{...(1)}$$

Where, γ_w is unit weight of water,
ε is porosity of soil material,
S_s is specific gravity of soil particles.

The weight of displaced water, (buoyancy) B acts in upward direction = $\boldsymbol{\gamma_w * (1 - \varepsilon)}$ **...(2)**

The resultant of these two forces, $\boldsymbol{Ws = W - B = \gamma_w * (1 - \varepsilon) * (S_s - 1)}$ **...(3)**

This is the submersed weight of unit volume of soil which always acts in the downward direction as the specific gravity of soil (sand) is greater than 1.

The upward disturbing force, F is the pressure gradient at that point.

$$\boldsymbol{F = \frac{dp}{dl} = \gamma_w * \frac{dh}{dl}} \quad \textbf{...(4)}$$

Where, h is residual head to be dissipated.

For critical condition, $\boldsymbol{F = Ws}$ **...(5)**

$$\boldsymbol{\gamma_w * \frac{dh}{dl} = \gamma_w * (1 - \varepsilon) * (S_s - 1)} \quad \textbf{...(6)}$$

$$\boldsymbol{\frac{dh}{dl} = (1 - \varepsilon) * (S_s - 1)} \quad \textbf{... (7)}$$

Where, $\frac{dh}{dl}$ is the rate of head loss or the exit gradient.

There are large numbers of uncertain parameters in the subsoil. The subsoil may not be homogeneous. The stratification may not be same throughout the foundation area. There may be local intrusion of very porous media or impervious media which may help to concentrate the flow from all around or deflect the flow lines. There may be the faults and fissures in the sub soil. The uncertainties in sub soil formations plays vital role to deviate the results from theory. Failure of any normal structure should be well high impossible if the exit gradients were considered purely from the academic perspective (Khosla *et al.*, 1962).

While designing safe structures, the uncertainties and deviations in subsoil must be considered. So, a factor of safety must be taken to the critical value of exit gradient to get a safe value of exit gradient. Table 1 shows typical safety factors recommended by Khosla.

Table 1: Recommended safety factor for exit gradients by Khosla

S.N.	Soil type	Safety factor
1	Shingle	4 to 5
2	Coarse sand	5 to 6
3	Fine sand	6 to 7

Source:(Khosla *et al.*, 1962)

4 Methods and Materials

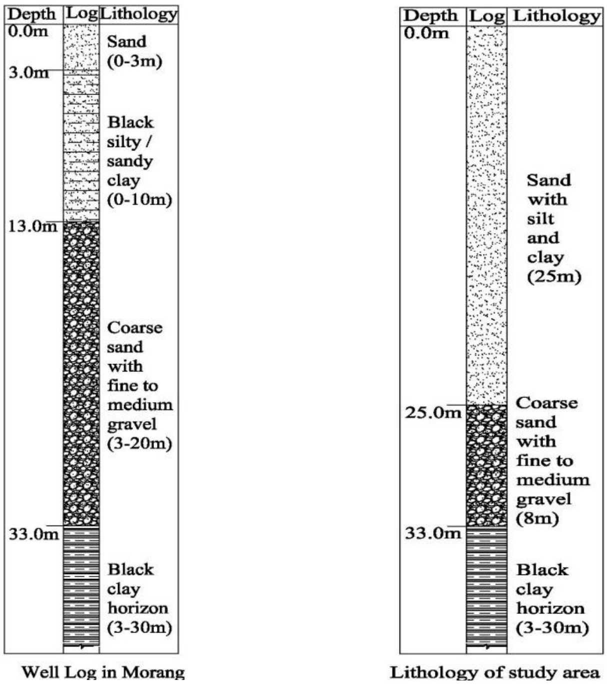
4.1 Physical Dimensions of Headwork Site

The details of physical dimensions of structures (plan and cross section of headwork structures) were taken during the site visit. It consists of both the primary and secondary data as underground dimensions are based up on as built drawings found in Eastern Irrigation Development Division No.-2, Inaruwa, Sunsari.

4.2 Geology of the Study Area

The study area is in the Southern part of the Terai Region. This plain is composed of interlocked alluvial deposits of the Gangetic plain. As the rivers flow from the Northern part to the Southern part, the alluvium was created by the river systems in this zone. The grain size of the sediments in this zone decreases towards south. The sediments consist of gravel, sand, silt and clay. The lithology of the upper 30 m or so is rapidly changing over very small distances (Kanzler *et al.*, 1989).

In Sunsari district, the chances of getting at least 5 m to 8 m of sand and/or gravel deposits are very high, and that a shallow well to supply water to villagers can be constructed without much uncertainty (Kanzler *et al.*, 1989). Test report of well log in Amahibela (X=518,125 m, Y=2,927,750 m), Sunsari nearby study area (X=516,644 m, Y=2,929,186 m) up to 24.4 m depth was reviewed. It shows that the top soil consists of sandy clay with thickness of 3.1 m. Sand of 3 m thick was found below top soil. Below this sand layer, an impervious layer of 1.5 m thick clay was noticed. Sand of 10.7 m thick layer and 6.1 m thick gravel and boulder were observed further below the clay layer (Kanzler *et al.*, 1989). Previous test report of well log in Ramnagar (X=510,875 m, Y=2,932,125 m), Sunsari up to 21.3 m depth shows that the top soil contains sandy clay with thickness of 5.2 m. Sand with gravel and pebbles was found in remaining 16.1 m depth (Kanzler *et al.*, 1989).



Source: (Bhattarai, 1988)

Figure 5: Lithological sections

Earlier studies carried out in Morang district (near to Sunsari district) show that it is also underlain by alluvium. As shown in Figure 5, the thickness of the top soil ranges from few centimetres to 3 m. Black silty to sandy clay of thickness few centimetres to 10 m is encountered below the top soil. The clay content is decreasing towards the North. Below this there is a layer of 3 m to 20 m consisting of course sand with fine to medium gravel. Shallow tube wells tap the water from this zone in the local areas. A black clay horizon of 3 m to 30 m is encountered below this layer. Further below this black clay horizon, 5 m to 40 m thick gravel and coarse to fine sand was observed (Bhattarai, 1988).

The geo-hydrological studies conducted from soil log test at the headwork area of Chanda Mohana Irrigation Scheme shows that different sizes and colour of sand mixed with silt were found up to 25 m depth (CMIP, 2001). From above information, sand and loamy sand are taken respectively at upstream and downstream of headwork up to the depth of 25 m and remaining 8 m of depth is taken as coarse sand with fine to medium gravel for the lithology of the study area. Below this depth, black clay horizon is considered which has the characteristics of impervious layer.

4.3 Laboratory Works for Soil Parameters

Soil samples from both the upstream and downstream of the headwork were excavated from 4 m below the river bed during the site visit. The soil samples were tested in the Central Material Testing Laboratory of Tribhuvan University, Institute of Engineering, Pulchowk Campus, Nepal. The Grain size distribution test, minimum and maximum dry densities, saturated bulk density, hydraulic conductivity and specific gravity were conducted for both the upstream and downstream samples.

The Grain size distribution test: The grain size distribution curves of upstream and downstream samples are shown in Figure 6.

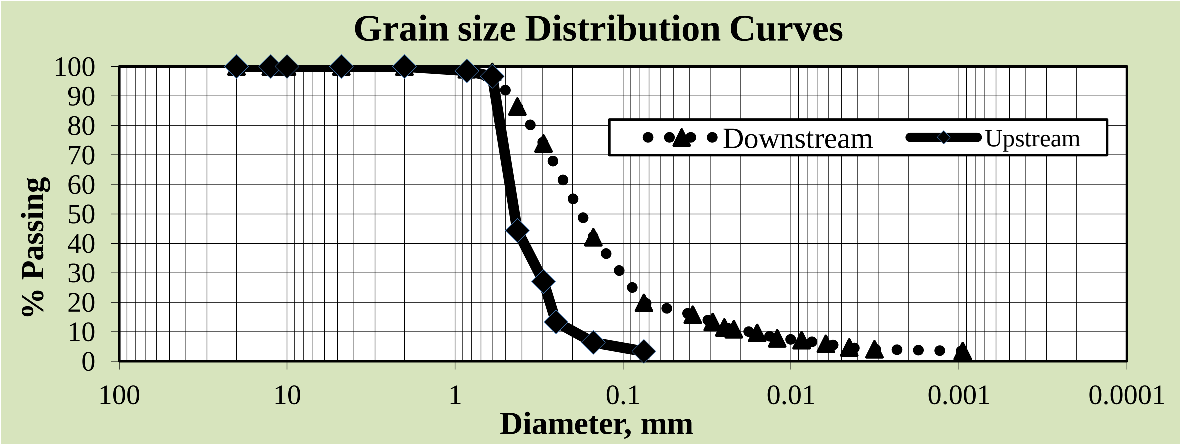


Figure 6: Grain size distribution curves

As per USDA soil classification system, upstream sample belongs to sand while downstream sample belongs to loamy sand. The D60, D50 and D30 of both samples are given in Table 2. The D60 means particle size such that 60% of the soil is finer than this size.

Table 2: D60, D50 and D30 of upstream and downstream sample

S.N.	Parameter	Upstream sample	Downstream sample
1	D60	0.48 mm	0.21 mm
2	D50	0.41 mm	0.19 mm
3	D30	0.31 mm	0.10 mm

Dry Density, Saturated bulk density, Specific gravity

The minimum dry density, maximum dry density, saturated bulk density and specific gravity values of upstream and downstream samples are shown in Table 3.

Table 3: Minimum dry density, maximum dry density, saturated bulk density and specific gravity values

S.N.	Parameter	Upstream sample	Downstream sample
1	Minimum dry density	1324 kg/m^3	1319 kg/m^3
2	Maximum dry density	1433 kg/m^3	1525 kg/m^3
3	Saturated bulk density	1622 kg/m^3	1740 kg/m^3
4	Specific gravity	2.69	2.78

Hydraulic conductivity test

As the hydraulic conductivity depends up the compaction of the soil layer, hydraulic conductivity was found in the laboratory for 30%, 60% and 90% compaction of soil sample for both the upstream and downstream samples. To find the hydraulic conductivity, constant head method was used and the rate of flow was measured. The hydraulic conductivity values are shown in Table 4.

Table 4: Hydraulic conductivity values

S. N.	Relative density (%)	U/S sample		D/S sample	
		Permeability, Ky (m/s)	Permeability, Kh (m/s)	Permeability, Ky (m/s)	Permeability, Kh (m/s)
1	30	$3.43*10^{-05}$	$3.75*10^{-05}$	$6.70*10^{-05}$	$1.26*10^{-04}$
2	60	$3.27*10^{-05}$	$3.47*10^{-05}$	$5.04*10^{-05}$	$5.10*10^{-05}$
3	90	$3.22*10^{-05}$	$3.23*10^{-05}$	$4.33*10^{-05}$	$4.50*10^{-05}$

4.4 Hydrology of the Study Area

4.4.1 Rainfall

The daily rainfall data observed at Tarhara Station situated nearby study area was analyzed. The annual average rainfall from 1984 to 2008 is 2006 mm.

4.4.2 Catchment area

The total catchment area of basin is delineated as 112 km^2 from ARCGIS using ARC Hydro. The digital elevation map is downloaded from http://srtm.csi.cgiar.org/ (2010). The catchment area is shown in Figure 7.

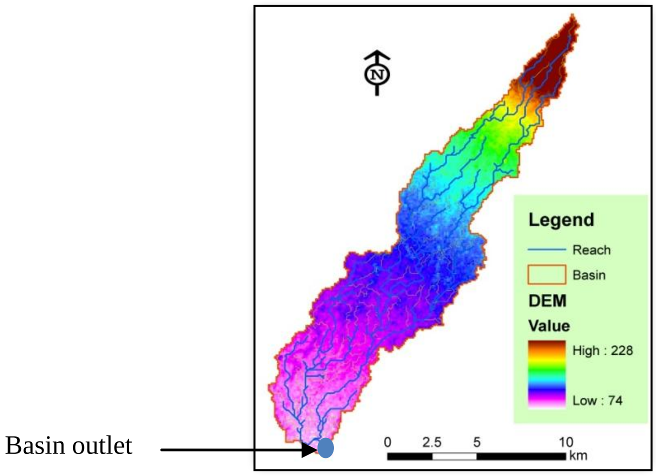


Figure 7: Catchment area of the basin (Budhi and Katle Khola)

4.4.3 High Flood Discharge

The high flood discharge was calculated from SCS method developed by His Majesty's Government of Nepal-Department of Irrigation, UNDP / World Bank (HMG-DoI/UNDP/WorldBank, 1990). It mainly includes the estimation of design daily rainfall and high flood discharge.

The design discharge for 50 years return period is 569 m^3/s. The design discharge considered during design was 550 m^3/s (CMS *et al.*, 1997). The calculated design discharge at present is quite close to that used in design.

4.5 MSeep Model Setup

4.5.1 Introduction to MSeep

MSeep model, Version 1.0 was first released by GeoDelft in 1988 and is being used for the two dimensional groundwater flow problems. In this research, MSeep model, version 6.7, developed in 2002 was used for the seepage analysis. Vertical two dimensional stationary groundwater flow in layered soil structures with boundary conditions is analysed by Cross section method in MSeep. An element mesh of iso-parametric triangles is created for the geometry (GeoDelft, 2002).

The cross section model in MSeep is based on Darcy's law and the continuity equation. Two dimensional steady flow in terms of Laplace equation used in MSeep is (GeoDelft, 2002),

$$\frac{\partial}{\partial x}\left(-K_x\frac{\partial \emptyset}{\partial x}\right)+\frac{\partial}{\partial y}\left(-K_y\frac{\partial \emptyset}{\partial y}\right)=Q \quad \text{...(8)}$$

Where $\emptyset$ is potential (piezometric head), *Kx* and *Ky* are permeabilities in *X* and *Y* directions and *Q* is the discharge of internal sources. Within MSeep, a negative discharge means an outflow which indicates that seepage occurs in that concerning nodal point. Finite element method is used to solve the Laplace equation.

4.5.2 Schematization

The schematization is done to represent the site condition as per the drawings and information found during the data collection. Different layers are created to represent different soil conditions as well as to represent the sheet piles. In contrast, extra layer is created at the end of sheet piles so that node exists at its end which gives the exact length of sheet pile as MSeep calculates the length of sheet pile to its nearest node.

4.5.3 Material Properties

Coarse sand/ fine to medium gravel

The permeability of coarse sand with gravel is taken as 1E-4 m/s (Harr, 1962) for both the upstream and downstream portion. The D60 is taken as 12.5 mm. The porosity and saturated bulk density are taken as the average of sand and gravel for both the upstream and downstream portion. The porosity values are taken as the average of gravel and sand which is 0.415 for both upstream and downstream portion. The saturated bulk density for upstream is 17.1 kN/m^3 and 17.7 kN/m^3 for downstream.

Sand

The horizontal and vertical permeability used for upstream sand was 51.14E-6 m/s and 50.35E-6 m/s respectively. The porosity of 0.48 and saturated bulk density of 16.2 kN/m^3 were used. The D60 was 0.48 mm. Similarly, the horizontal and vertical permeability of downstream loamy sand was 34.7E-6 m/s and 32.7E-6 m/s respectively. The values of porosity, saturated bulk density and D60 used were for calculation were 0.48, 17.4 kN/m^3 and 0.21 mm respectively.

Gravel filter

The average permeability of gravel is 11.6E-3 m/s or 1000 m/day (Zhou, 2009). The average porosity of gravel is 0.35 (USACE, 1999). The D60 and saturated bulk density are taken as 25 mm and 1800 kg/m^3 respectively.

Gabion works with boulder filling

As the permeability of stones having diameter 100-300 mm is 0.3 m/sec (USACE, 2006), the permeability of gabion works with stone filling was also taken as 0.3 m/s. The porosity of gabion works used was 0.3 (http://www.erosioncontrol.com/november-december-2003/gabions-for-erosion-2.aspx, 2011). The D60 and saturated bulk density for gabion works with boulder filling were considered to be 250 mm and 2200 kg/m^3 respectively.

Concrete blocks

The permeability of concrete blocks with gaps in between is calculated considering it as the rocks and joints in between them. The permeability of concrete blocks with gaps in between has been determined assuming it as a rock and joints in between the rocks. Fracture characteristics of rock mass such as rock quality designation (RQD), joint number (Jn), joint roughness (Jr), joint hydraulic conductivity (Jk), joint aperture factor (Jaf) and joint water factor (Jw) were assigned a numerical value depending up on fracture properties and hydropotential value is obtained from them which is again modified into permeability (Gates, 2002).

For cross sectional area of 5.5 m x 1.0 m for under-sluice portion as per as built drawings, the permeability is 1.19E-5 m/s. Similarly, for cross sectional area of 4.5 m x 1.0 m for weir portion as per as built drawings, the permeability is 1.45E-5 m/s. Considering the gap in the joints as 10-20 mm, the porosity of 0.03 is taken. The saturated bulk density is considered as 2400 kg/m^3.

4.5.4 Boundary Conditions

The maximum water depth at the upstream is 4.5 m. So boundary conditions for upstream is given considering water depth of 4.5 m. From geological analysis of study area, the impervious layer is kept at 33 m below the upstream river bed. No flow boundary condition is taken to represent impervious layers. For the downstream portion, water level at the river bed level is considered which follows the boundary with potential with the elevation equals river bed level elevation with respect to impervious layer of aquifer. Boundary conditions for nodes were given accordingly to represent flow or no flow condition.

To analyse the seepage from Jala Khola, a section along the weir portion, under-sluice portion, abutment wall and Jala Khola is considered. The location of Jala Khola 200 m far from headwork site which is the original flow point of the river towards headwork is considered for analysis. This location was chosen as it is preferable for the river to flow in the pre-existing path. The bed level and water level at this location are given as boundary conditions for Jala Khola. As the critical condition for seepage at headwork site is for non flooding season, maximum water level at the Jala Khola during non flooding season is given as the boundary condition. The bed level of Jala Khola at this location as obtained during site visit is 67.70 m amsl. As per local people and member of water users' association, the maximum water level during non flooding season at that location is 1.25 m. So, water depth of 1.25 m is taken for analysis.

4.5.5 Optimization of grid size and river bed length

The Finite element model requires optimization of the element size (grid size). As the minimum element size for maximum exit gradient is found as 0.7 m x 0.7 m, the element size of 0.7 m x 0.7 m is taken for the calculations. The exit gradient increases until the river bed length is equal to 1.5 times the length of structure and there is no impact on exit gradient in further increment in length of river bed. So, the river bed length is considered as 1.5 times the length of structure.

5 Modelling for Different Scenarios

MSeep model was run for different scenarios to represent the site condition as different works were carried out in different time periods. Site condition after construction in 2000, after maintenance in 2000, 2009, 2010 were analysed and a new section is recommended. So, five different scenarios were analysed.

The model was run for sections of under-sluice portion and weir portion separately for the flow from upstream to downstream without considering lateral flow. Exit gradients for each section were calculated. The schematization for under-sluice portion without lateral flow is shown in Figure 8. The schematization for weir portion is shown in Figure 9. Different types of materials are shown in different layers. Each layer has its own colour as shown in legend in respective figure. The sheet piles are denoted by vertical green lines. Water levels at upstream and downstream are denoted by horizontal blue lines.

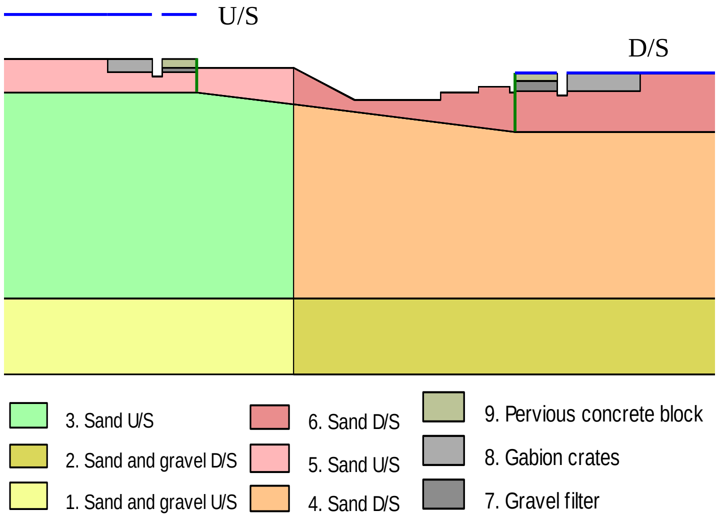


Figure 8: Schematization for under-sluice portion without lateral flow for Scenario-I

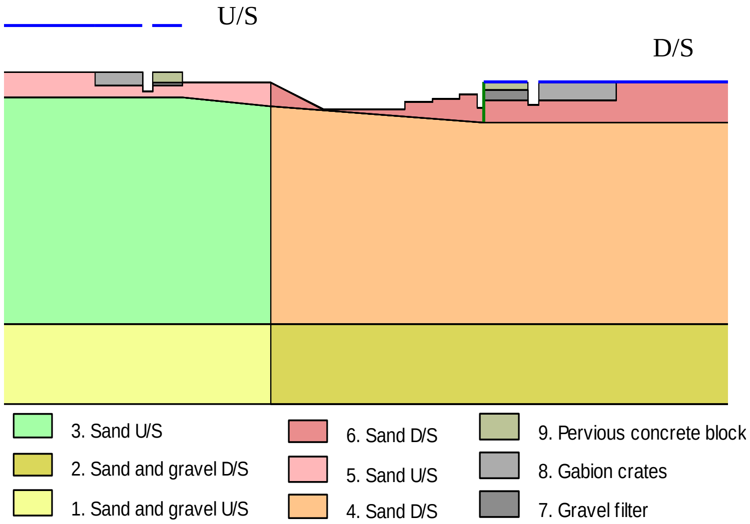


Figure 9: Schematization for weir portion for Scenario-I

For the lateral flow consideration, section along weir portion - under-sluice portion - abutment wall - Jala Khola was considered in upstream weir crest line. The potential (residual head) obtained above from weir portion at the line of weir crest beneath the upstream floor is used for the boundary condition for the seepage analysis along weir portion - under-sluice portion - abutment wall - Jala Khola. The river bed and water level in Jala Khola were the boundary condition for Jala Khola end. The schematization for lateral flow consideration is shown in Figure 10.

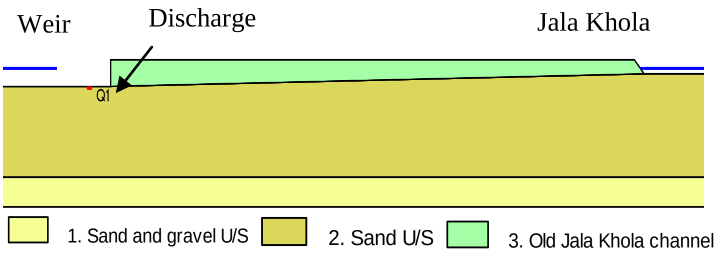


Figure 10: Schematization for lateral flow consideration for Scenario-I

The seepage discharge outgoing from the system from under-sluice portion was simulated until the potential beneath the under-sluice portion from this condition is equal to the potential from upstream to downstream without considering seepage.

This seepage discharge was added at the point at beneath weir crest at under-sluice portion for the flow from upstream to downstream along the under-sluice portion. When a discharge point is added in MSeep, this discharge goes all around the point (even in to upstream of the point). In reality, it does not go to upstream. So, the discharge point is given in sloping portion of weir to reduce this effect. From this, exit gradient at the under-sluice portion was obtained considering lateral flow. The schematization is shown in Figure 11.

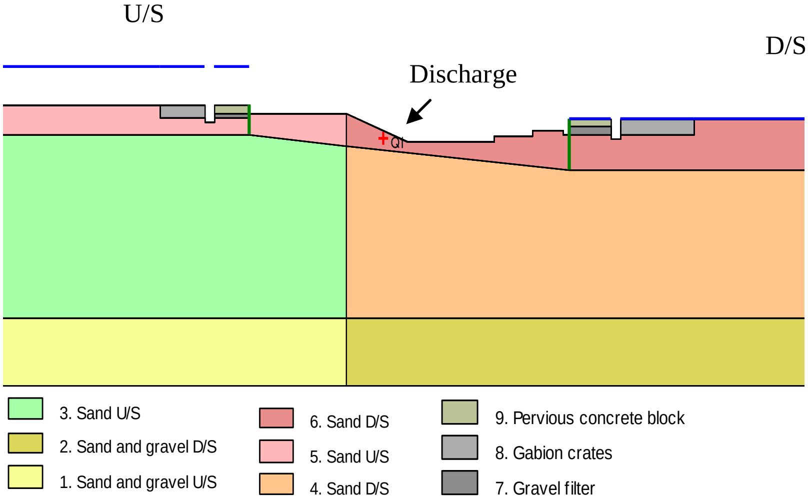


Figure 11: Schematization for under-sluice portion with lateral flow for Scenario-I

6 Results and Discussion

6.1 Scenario-I: As Built Condition in 2000

For undersluice portion, the residual head beneath the structure at the upstream of weir crest line was 35.906 m. The exit gradient at under-sluice portion without considering lateral flow was calculated as 0.151 which is below the maximum theoretical permissible exit gradient i.e. 0.154. This shows that the structure was safe if there was no lateral seepage discharge.

The exit gradients in weir portion with and without upstream sheet piles were 0.144 and 0.146 respectively. The residual head beneath the structure without upstream sheet pile is 36.326 m at the upstream of weir crest line which was used for boundary condition to consider lateral flow.

The residual head at the weir portion (36.326 m) is higher than residual head at the under-sluice portion (35.906 m). This shows that there is possibility of seepage flow from weir portion to under-sluice portion. As the sheet piles near the divide wall in weir portion were not found during upstream floor excavation in 2010, this became a favourable condition for the seepage flow to enter into the under-sluice portion from weir portion.

The higher equipotential values near Jala Khola clearly indicate that seepage flow occurs from Jala Khola towards the structure. Although the Jala Khola was diverted towards east after headwork construction, there is possibility of seepage since impervious barrier was not provided at the bank of Jala Khola.

Lateral discharge flow into the under-sluice portion was found from trial and error until the residual head beneath the under-sluice portion was same as that was found earlier, 35.91 m. The lateral discharge was 2.25E-5 m^3/s. The discharge, 2.25E-5 m^3/s obtained from lateral flow was then inserted into the under-sluice portion.

After the research conducted in the Kafrein dam in Jordan, it was concluded that the seepage water was mainly flowing through faulted bed rock and alluvium deposits beneath the abutment of dam; seepage water was originating from the location used as borrow area during construction, some 1 km upstream of dam (Malkawi *et al.*, 2000). Although the borrow area might have filled by sediments, there was seepage from this area. Similarly, seepage coming from Jala Khola was observed even though earth filling was done in its previous channel after the diversion.
Due to lateral flow, the head is increased and thus the exit gradient is increased from 0.151 to 0.163.

The theoretical minimum and maximum safe exit gradient limit for soil having specific gravity of 2.78 and porosity of 0.48 is;
Lower limit = (1-0.48) x (2.78-1)/7 = 0.132 (considering safety factor of 7)
Upper limit = (1-0.48) x (2.78-1)/6 = 0.154 (considering safety factor of 6)

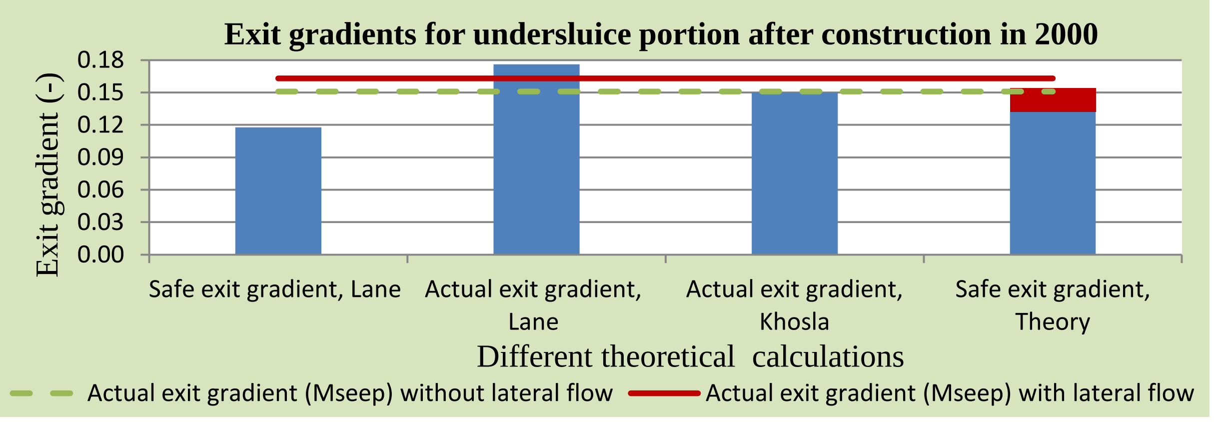


Figure 12: Comparison of exit gradients for under-sluice portion under Scenario-I

The exit gradient as per Lane is also greater than his safe recommended exit gradient. The red dark shaded area in Figure 12 for safe exit gradient, Theory is the maximum permissible exit gradient. Although the lower values of exit gradient are safe for structure, it increases the cost. The red shaded area is the range for economical constructions with safe structure. This shows that exit gradient without lateral flow is within the maximum safe limit. Exit gradient by Khosla does not consider the effect of lateral seepage flow. So, it is lower than the actual exit gradient. The actual exit gradient without lateral flow is almost same with the exit gradient by Khosla. As the maximum limit of safe exit gradient was exceeded with the lateral flow into the under-sluice portion, the structure might have failed due to piping action in 2000 during the commissioning period.

Exit gradients for weir portion are shown in Figure 13. It shows that the upstream sheet pile does not have vital role in reducing the exit gradient. It might be because of the bottom level of upstream cut-off wall and bottom of upstream sheet pile is only 0.60 m. The exit gradient, Lane is higher than recommended by him. The exit gradient from model is within the maximum theoretical permissible limit. Although the exit gradient is not much affected by the absence of upstream sheet pile, it might have provided a path for the seepage water to flow from weir portion to under-sluice portion. Since the residual head at the weir portion was greater than the residual head at under-sluice portion, the absence of upstream sheet pile might have become a favourable condition to flow seepage water into under-sluice portion. This was reflected at site as well at 2010 failure condition as hollow beneath the structure was expanded from under-sluice portion to the weir via this location.

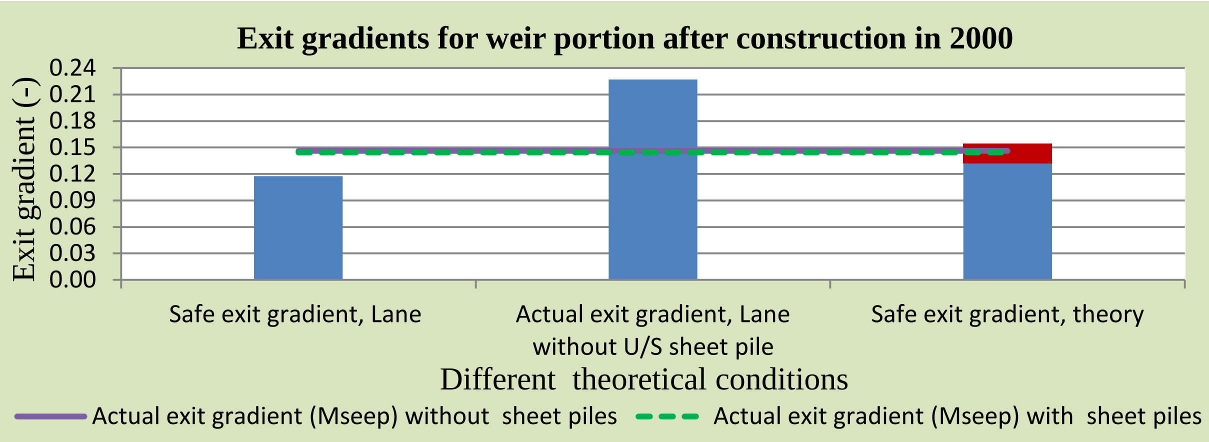


Figure 13: Comparison of exit gradients for weir portion under Scenario-I

6.2 Scenario-II: After Maintenance in 2000

The potential beneath the structure at the upstream of weir crest line had decreased from 35.906 m to 35.207 m. As in under-sluice portion, the residual head at weir portion beneath the structure without upstream sheet pile at the upstream of weir crest line had reduced to 35.481 m. The exit gradient without upstream sheet pile was 0.122.

The lateral seepage discharge was computed with same process as in scenario-I. The lateral seepage discharge was 1.95E-5 m^3/s. Because of lateral flow, the head in increased. The exit gradient is increased from 0.130 to 0.143 which is below the maximum theoretical permissible exit gradient.

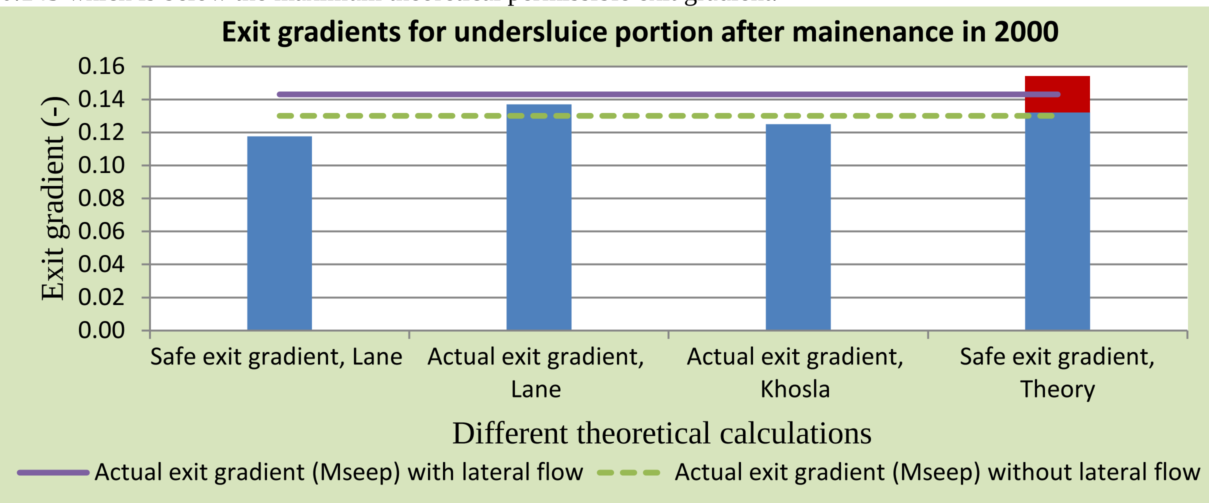


Figure 14: Comparison of exit gradients for under-sluice portion under Scenario-II

Figure 14 shows that the exit gradient according to Lane is greater than his safe recommended exit gradient. The actual exit gradient without considering lateral flow is slightly higher than exit gradient by Khosla. Both exit gradients are less than permissible theoretical exit gradient. Actual exit gradient with considering lateral flow is within the maximum theoretical permissible limit. As the maximum limit of safe exit gradient was not exceeded, the structure was safe. This result supports the site condition as the structure was functioning until 2009 after maintenance works in 2000. This shows that safe exit gradient by Lane assumes a higher safety factor.

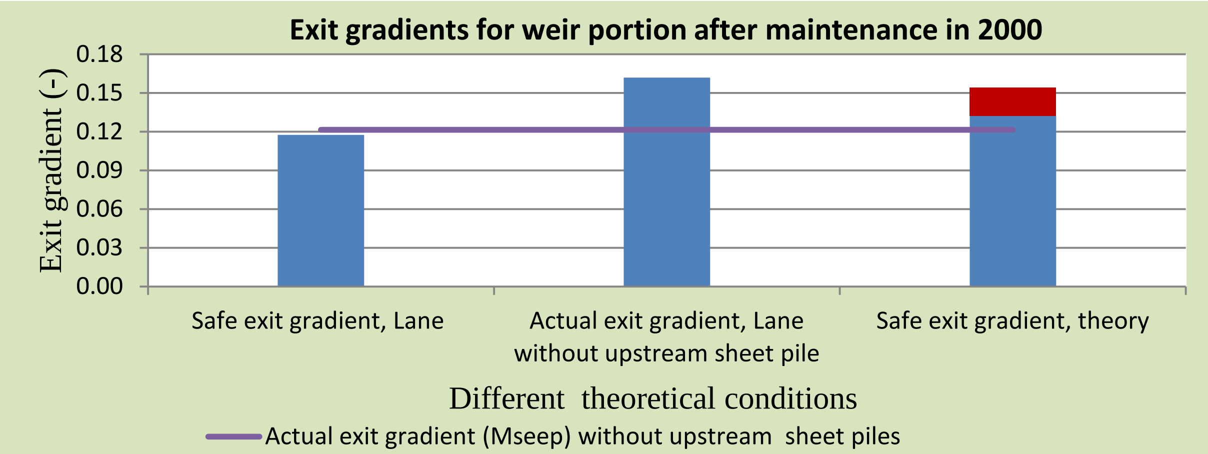


Figure 15: Comparison of exit gradients for weir portion under Scenario-II

From Figure 15, it is clear that the exit gradient, Lane is higher than recommended by him. In contrast, the exit gradient from model is within the safe theoretical permissible limit. It highly supports the site condition because the structure was functioning until 2009 after maintenance carried out in 2000. This illustrates that safe exit gradient by Lane assumes a high safety factor.

6.3 Scenario-III: After Maintenance in 2009

As shown in Figure 16, the exit gradient should have decreased after maintenance in 2009 if the soil parameters were same as previous in 2000. An exit gradient by Khosla (0.122) is slightly lower than the exit gradient by MSeep without considering lateral flow (0.126), both being less than permissible theoretical exit gradient. The lateral seepage discharge and actual exit gradient might be 2E-5 m^3/s and 0.140 respectively. Failure in 2010 should not have occurred in this situation as the exit gradient, 0.140 was within permissible limit. But, the headwork failure was seen in 2010 which indicates the change in soil parameters due to sub soil erosion by seepage water.

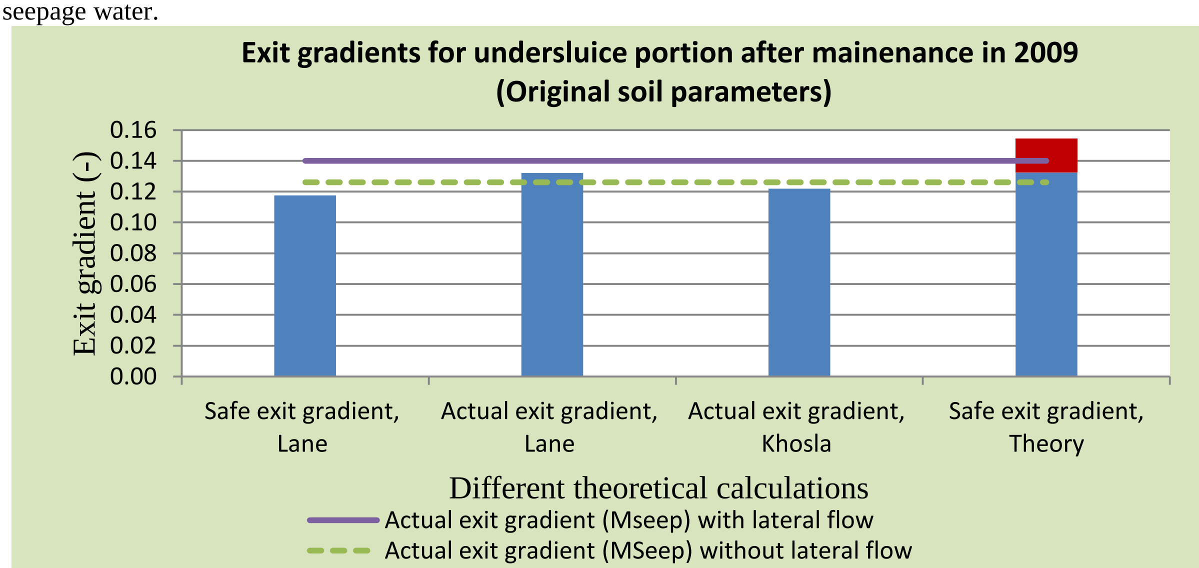


Figure 16: Comparison of exit gradients for under-sluice portion for Scenario-III (Original soil parameters)

Seepage discharge was increased and water boiling was seen in the downstream portion by experiments. The channel (cavity) size increases with higher seepage discharge (Pornprommin *et al.*, 2010). It might have made cavities beneath the structure.

So, new soil parameters as found in laboratory for relative density of 30% were used for analysis. In 2009, the horizontal and vertical hydraulic conductivities might have increased by 2.5 and 1.3 times than previous which reduced the rate of head loss and hence increased the exit gradient. Therefore, the structure was again failed at 2010 although there was a maintenance activity carried out in 2009.

The rise in reservoir water level in Ataturk dam probably eroded clay fillings in karstic limestone and ungrouted locations which resulted in the formation of hollow features and cavities (Unal *et al.*, 2007).

Seepage analysis carried out on Fordyce Dam, Sierra Nevada Mountains concluded that the measured seepage discharge of about 25 cubic feet per second can be achieved if the hydraulic conductivities of the concrete slab and cut-off wall are 25 times higher than that were initially estimated, or the hydraulic conductivity of alluvial channel is 10 times higher than that was initially estimated (Lee, et al., 2006).

The potential beneath the structure at the upstream of weir crest line was 35.897 m. The exit gradient without considering lateral flow is 0.159.The residual head beneath the structure without upstream sheet pile at the upstream of weir crest line was 36.065 m. The exit gradient without upstream sheet pile was 0.151.

The residual head in under-sluice portion (35.897 m) is lower than that at weir portion (36.065 m) and Jala Khola (36.45 m). The higher heads at weir and abutment portion made seepage water flow into under-sluice portion. The lateral seepage discharge was 1.7E-5 m^3/s.

As a result of lateral flow, the head is increased and thus the exit gradient is increased to 0.168. Figure 17 shows that the exit gradient according to Lane is slightly greater than his safe recommended exit gradient. The exit gradient by MSeep without considering lateral seepage flow is much higher than exit gradient by Kholsa. This is due to the fact that Khosla theory does not consider the effect of change in soil parameters like hydraulic conductivity, porosity. So, there is no change in exit gradient by Khosla. Exit gradient with lateral flow (0.168) is above the maximum theoretical permissible limit (0.148). At site, the structure was failed in 2010 because of the exit gradient exceeded the maximum limit of safe exit gradient.

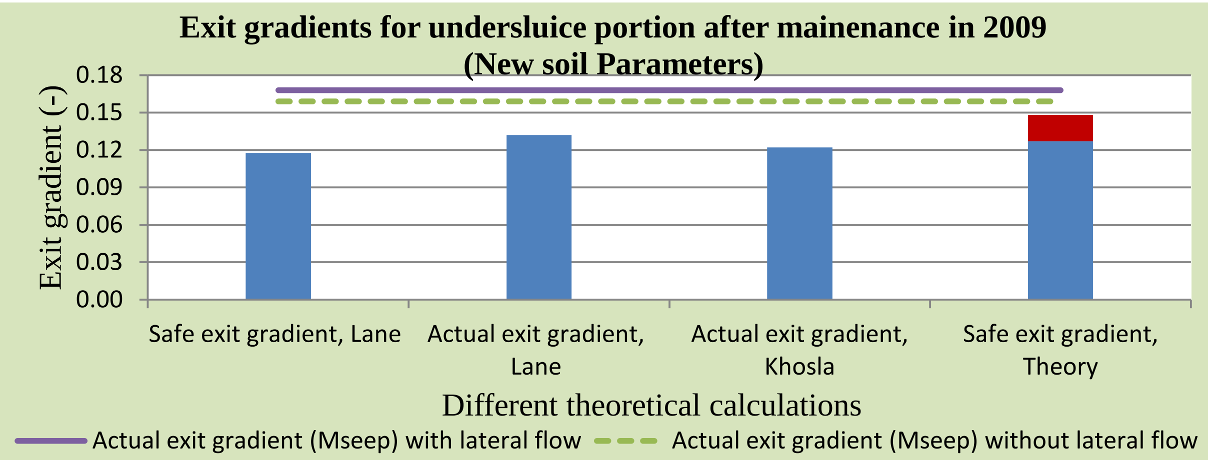


Figure 17: Comparison of exit gradients for under-sluice portion for Scenario-III (New soil parameters)

Exit gradients for weir portion are shown in Figure 18. The exit gradient, Lane is higher than recommended by him. The exit gradient from MSeep (0.151) is also higher than the maximum theoretical permissible limit (0.148) which made the structure fail in 2010.

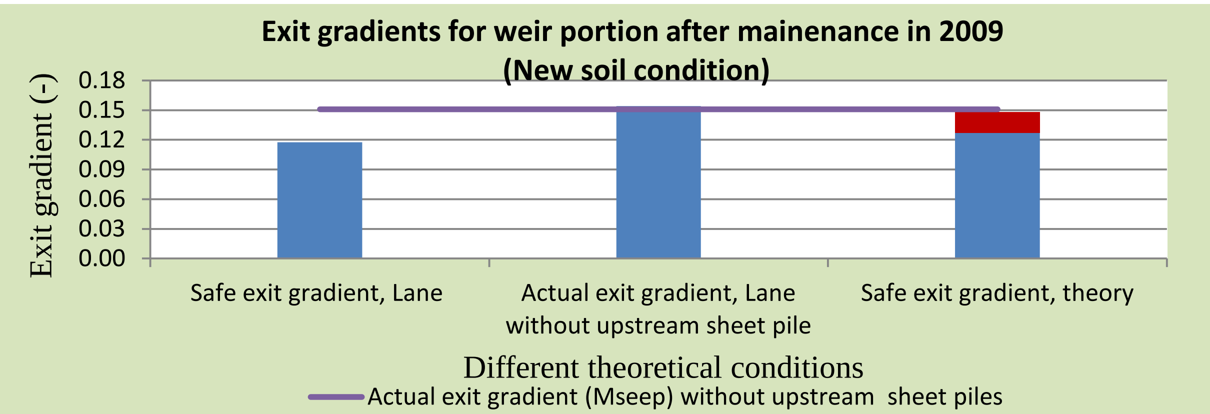


Figure 18: Comparison of exit gradients for weir portion for Scenario-III (New soil parameters)

6.4 Scenario-IV: After Maintenance in 2010

After the maintenance work of 2010, the hydraulic conductivity and porosity of sub soil decreases. Different exit gradients for under-sluice portion are shown in Figure 19 considering the soil parameters might have reached into original condition as in 2000. The graph shows that the structure is safe if the soil parameters have reached into original condition as in 2000. The structure is safe for all exit gradients. The exit gradient by MSeep without lateral flow (0.131) is almost same with exit gradient by Khosla (0.134) as the soil parameters are taken as in 2000. The exit gradient with lateral flow is 0.138.

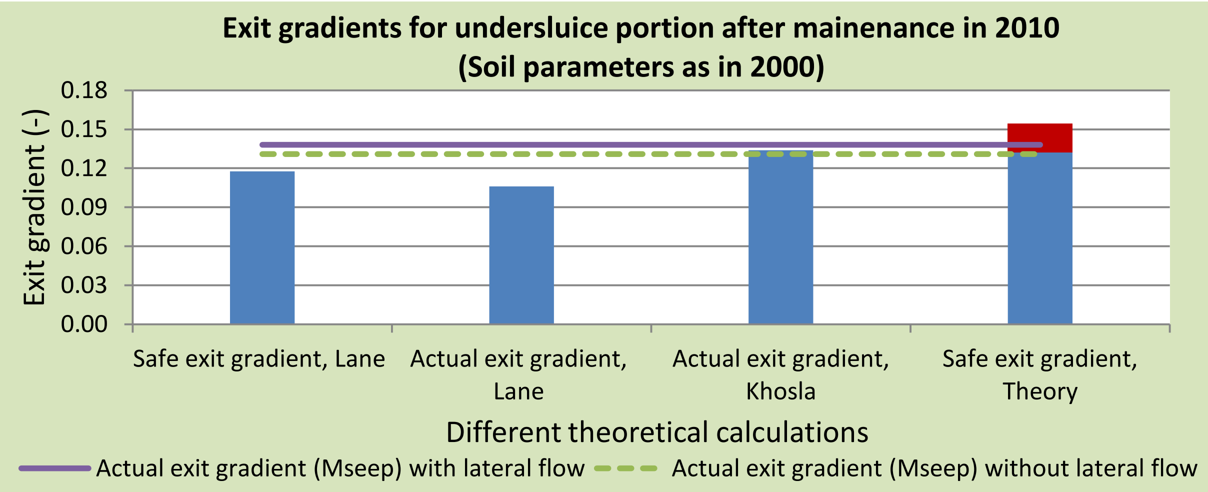


Figure 19: Comparison of exit gradients for under-sluice portion for Scenario-IV (Original soil parameters)

Grouting improves the physical characteristics of the surrounding rock mass which mainly increases the rock mass stiffness and reduces the rock mass permeability (Marence, 2008). Furthermore, permeability can be decreased up to 20 times by consolidated grouting (Marence *et al.*, 2005).

In addition, the efficiency of grout curtain improved in Ataturk dam after additional grout treatment since there was no increase in leakage and a relative decrease as compared to former measurements (Unal *et al.*, 2007). Grouting work was carried out in three different phases in Chanda Mohana Irrigation Scheme in 2010. So, the efficiency of grouting might have been increased and thus the hydraulic conductivity might have decreased after grouting works.

Although about 53,336 kg of grouting material was consumed on the right side of dam below 20 m depth and about 5,666 kg was consumed on the left side of Kafrein dam. It was concluded that the seepage water was mainly flowing through faulted bed rock and alluvium deposits beneath the abutment of dam; seepage water was originating from the location used as borrow area during construction, some 1 km upstream of dam and little seepage contributed by left abutment (Malkawi *et al.*, 2000).

In addition, though 825 tons of cement, 28 tons of bentonite, 4.8 tons of sand and 0.2 tons of sodium silicate has been used for the grouting operations in Kalecik dam, it was concluded that the leakage path moved towards the spillway from dam embankment but the total discharge of the springs did not change (Turkmen, 2003).

Furthermore, drilling and grouting (1989 to 2003) with 17,700 kg of cement, bentonite and sand were done on left side up to 35 m depth in the Armagan dam, Kirklareli, northwestern Turkey. Research showed that grouting on left bank was not completely successful (Ünal *et al.*, 2008). This shows that grouting is not always fully successful to control the seepage problems.

About 175,000 kgs of cement was used to make cement slurry and was sent by pressure grouting in the headwork of Chanda Mohana Irrigation Scheme. The technicians involved in maintenance work claimed that the hollow formed areas are now more or less completely filled. According to Curt, *et al.* (2011), visual inspection (like cracking, differential movements, seepage, surveillance, sinkhole formation etc) is a key item of visual assessments during dam reviews. The construction techniques in developing countries like Nepal are not of high quality. For the safety of structure, the soil parameters after pressure grouting with cement slurry is assumed as medium with respect to the original condition and the condition at the time of failure in 2010. The horizontal and vertical hydraulic conductivities for sand upstream were assumed as 8.86E-5 m/s and 5.87E-5 m/s and for downstream sand as 3.61E-5 m/s and 3.35E-5 m/s respectively. The porosity for upstream and downstream was considered as 0.49.

There is head loss due to new sheet pile at the upstream. In contrast, there is not much effect of new sheet pile at the downstream end. This is logical as the distance between these two piles is only 2 m, stream lines do not reach up to the bottom of impervious floor in the space between two piles. Thus, there is not much head loss in between two sheep piles as it was expected. The exit gradient without lateral flow is 0.147. The potential beneath the structure at the upstream of weir crest line is 36.00 m. Optimization on sheet pile spacing shows that at least 5 m of spacing between two sheet piles results in the head loss between them.

The residual head beneath the structure without upstream sheet pile at the upstream of weir crest line was 36.131 m. Although the additional downstream sheet pile is near to the existing one, as it is deeper than the existing, its effect can be seen. Thus, there is no head loss in between two sheep piles. The exit gradient without upstream sheet pile reduced to 0.116.

Lateral flow is now analysed with additional sheet pile under abutment wall after maintenance in 2010. The higher heads at weir portion (36.131 m) and Jala Khola (36.45 m) than in under-sluice portion (36.00 m) had caused the lateral flow into under-sluice portion.

Due to flow barrier, the seepage discharge has decreased. The lateral seepage flow is 1E-5 m^3/s. Due to additional impervious floor and sheet piles both in upstream and downstream as well as improved soil parameters; the exit gradient at under-sluice portion with seepage discharge is 0.152.

For the porosity of 0.49 and specific gravity of 2.78, theoretical minimum and maximum safe exit gradient limit is;
Lower limit = (1-0.49) x (2.78-1)/7 = 0.130 (considering safety factor of 7)
Upper limit = (1-0.49) x (2.78-1)/6 = 0.151 (considering safety factor of 6)

The comparison of exit gradients for under-sluice portion is shown in Figure 20. As new soil parameters were considered in modelling, the exit gradient without later seepage is much higher than exit gradient by Khosla. The structure seems to be safe from Khosla exit gradient. But, it has not considered the lateral flow as well as effect of change in values of soil parameters. The vertical distance of sheet piles in inner sides is taken to calculate the lane's exit gradient though the stream line do not reach up to floor level in between the downstream sheet piles which can not be known from Lane's theory. It tells us that the exit gradient by Lane is less than the recommended which is safe for structure. The exit gradient including seepage flow is slightly above the maximum theoretical permissible limit. This shows that the structure is still at the verge of failure.

Although major maintenance works were carried out three times after the completion of the headwork, the results show that it is still not safe. If the higher safety factor than that recommended by Khosla had taken for seepage analysis during the design, the structure would not have failed.

It is difficult and quite expensive to control the quantity of seepage that occurs after construction (Uromeihy *et al.*, 2007). The repeated maintenance works due to lower safety factor cost more than one time construction cost resulting from higher safety factor. As the structure is not fully safe even after maintenance in 2010, further additional maintenance works are recommended under Scenario V.

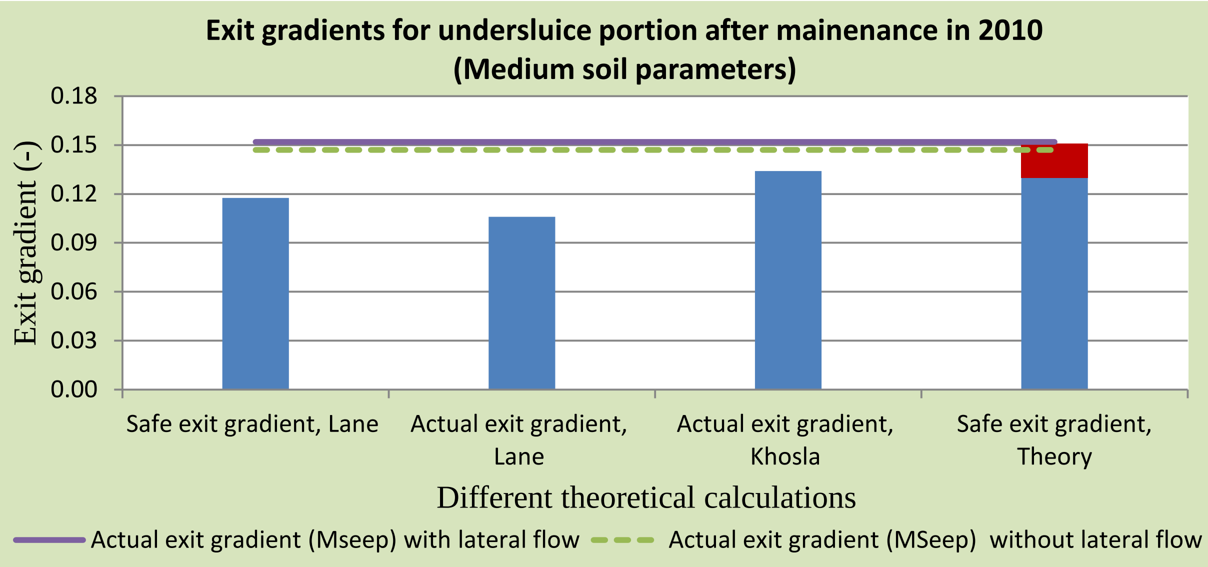


Figure 20: Comparison of exit gradients for under-sluice portion for Scenario-IV (Medium soil parameters)

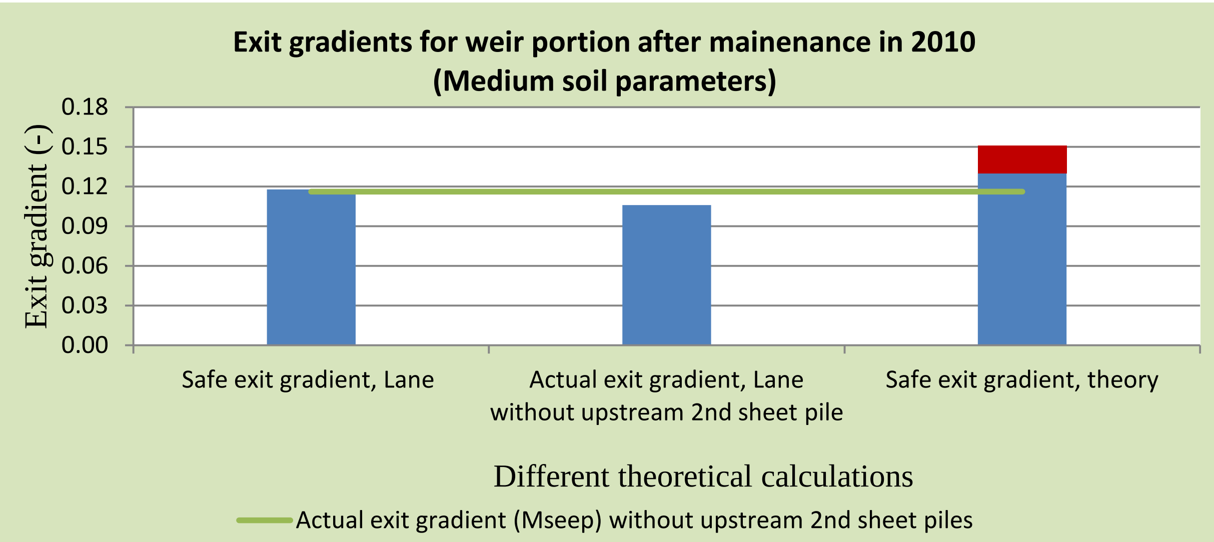


Figure 21: Comparison of exit gradients for weir portion for Scenario-IV (Medium soil parameters)

Figure 21 shows that exit gradient by Lane is less than the recommended exit gradient in the weir portion. In addition, exit gradient from MSeep is also less than theoretical permissible exit gradient after maintenance in 2010. This shows that further maintenance works are not needed for weir portion. So, no maintenance works are suggested for weir portion.

6.5 Scenario-V: Proposed Maintenance Works

From Scenario-IV, it is seen that maintenance works are needed in under-sluice portion for the safety of the structure. So, additional maintenance works are proposed herewith.

Instead of the pervious concrete blocks at the downstream of under-sluice portion, impervious concrete floor is considered with additional sheet pile up to the same depth as existing sheet pile depth. At the downstream of additional sheet pile, pervious concrete blocks of 10 m length above gravel filter as proposed in 2010 is analysed. Now, the stream lines reach up to the bottom of impervious floor in between existing and additional sheet pile. It is due to increased distance between the existing and additional sheet piles. The potential beneath the structure at the upstream of weir crest line in under-sluice portion is 36.146 m. So, lateral seepage flow is calculated using residual head of 36.146 m at under-sluice portion and 36.131 m in weir portion. Additional maintenance works at weir portion is not suggested as it is already safe.

The residual heads at weir portion and under-sluice portion are almost the same. In contrast to the previous scenarios, the difference in residual heads at weir portion, under-sluice portion and abutment wall is not much different. This reduced the lateral seepage flow as the head difference is less. The lateral seepage flow is 9.5E-7 m^3/s. This seepage discharge is too less in comparison to the previous scenarios. The exit gradient with lateral seepage flow is 0.127 which is below the safe permissible theoretical exit gradient.

Two more alternative analyses were made. They are:

- Additional impervious floor and sheet pile at upstream instead of at the downstream;
- Additional impervious floor and sheet pile both at upstream and downstream.

The comparison of exit gradients is shown in Figure 22.

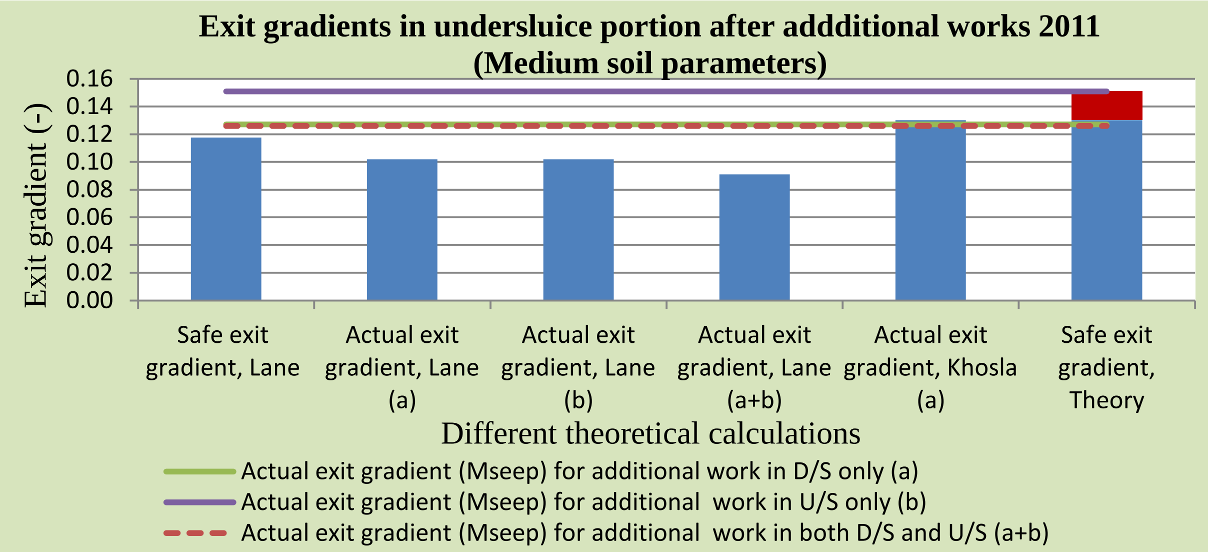


Figure 22: Comparison of exit gradients for under-sluice portion for Scenario-V

Figure 22 shows that there is no significant reduction in exit gradient due to additional work in upstream portion. The sheet piles at upstream mainly reduces pressure at the upstream floor. The upstream pile has little effect in reducing the pressures at the downstream portion as the spacing of the upstream and downstream sheet piles is generally much more than the range of influence of either (Khosla *et al.*, 1962). This argument is supported by Figure 22. As the additional work in upstream as well as in both upstream and downstream is not effective to reduce the exit gradient, it will be more economical to implement additional work in downstream portion only. Therefore, additional impervious downstream floor and sheet pile is necessary for the safety of the structure.

7 Conclusions and Recommendations

After the results and discussions, conclusions and recommendations are made based on the findings of the analysis. The conclusions and recommendations are as follows:

7.1 Conclusions

1. In the headwork structure with weir portion and under-sluice portion, the residual head in lateral direction along the weir axis may not be the same due to different floor length in weir portion and under-sluice portion. The difference in residual head becomes a favourable condition for lateral seepage flow.

2. Although the Jala Khola was diverted into new channel, there is seepage flow from new Jala Khola channel to the headwork. This shows that the flow path before diversion becomes a preferable seepage flow path for the river if impervious barriers are not provided.

3. The headwork structure as built in 2000 was safe if there was no lateral seepage flow. Due to total lateral discharge of 2.25E-5 m^3/s from weir portion as well as from Jala Khola, the exit gradient at the under-sluice portion increased from 0.151 to 0.163, exceeding the maximum permissible safe exit gradient of 0.154. As a result of this, the failure was seen in commissioning period in 2000.

4. After the maintenance works in commissioning period 2000, the exit gradient in under-sluice portion is reduced to 0.143 which is less than the maximum permissible exit gradient, but is still higher than the

minimum recommended. As the exit gradient is below maximum permissible exit gradient, the structure was functioning until 2009.

5. Soil parameters might have changed until 2009 due to subsoil erosion. The horizontal and vertical hydraulic conductivities might have increased to about 2.5 and 1.3 times greater respectively with respect to 2000. This reduced the rate of head loss. Due to lateral seepage discharge of about 1.7E-5 m^3/s and reduced rate of head loss, the structure failed within one year after the maintenance in 2009.
6. The ongoing maintenance work from 2010 is only safe if the pressure grouting improved the soil parameters as that in 2000 which is often not possible. The structure is still at the verge of failure considering the soil parameters as medium with respect to that in 2000 and in 2009.

7. Additional impervious floor of 9 m length with sheet pile of 4 m depth is necessary at downstream for the safety of the structure.

7.2 Recommendations

1. The results showed that there is seepage water flow from Jala Khola to headwork. So, it is necessary to reduce this seepage flow. Impervious barrier nearby Jala Khola bank might be effective in reducing the seepage discharge from Jala Khola. Providing filter material nearby abutment wall could help for safe exit of seepage water from Jala Khola.

2. The results from the MSeep model suggested that the current ongoing maintenance work is not still safe from failure of headwork. Therefore, the findings of the research strongly recommend to have an additional impervious floor of 9 m with sheet pile of 4 m depth at of the downstream of the headwork.

3. Khosla method is adopted instead of MSeep model in general practice of seepage analysis in Nepal. However, a factor of safety provided by Khosla (1962) could not be considered if there is a lateral seepage flow. Therefore, it is recommended to use higher factor of safety than in current practice for considering lateral seepage flow.

4. Although it was not a part of research analysis, the construction equipments and techniques largely affect the quality of the work. Generally, sheet piles are manually inserted below the foundation. It is difficult to insert the sheet piles manually into higher depth. During the sheet pilling works, joints between the sheet piles may not be fully water tight. The sheet pile may not go vertically downward everywhere when it is done manually. The deflection of sheet pile at higher depth increases the gap between the sheet piles. Furthermore, seepage water may pass through this gap. This process reduces the creep length and the rate of head loss decreases. In addition, the exit gradient increases. It could be better to adopt mechanical method of sheet piling.

7.3 Further Research

1. The results depicted that although the Jala Khola was diverted into new channel, there is seepage flow from new Jala Khola channel to the headwork. This seepage problem becomes 3-dimensional when lateral seepage flow is also considered. As 2-D model is a simplification of 3-D problems, seepage analysis by 3-D model could be a way forward for more accurate analysis.

2. Water chemistry, dye tracer method etc might be useful to estimate lateral seepage discharge at the field level.

3. The time dependent relation of sub soil erosion with varying exit gradient should be investigated to understand the relation among them.

4. The conclusions made in this research are from a case study of Chanda Mohana Irrigation Scheme, Nepal. However, the problem associated with seepage failure is common in other irrigation schemes in Nepal. Therefore, further research is required to explore seepage analysis in other irrigation schemes of Nepal. The outcome of further research from various schemes can provide a guideline to make a general conclusion for seepage failure in irrigation schemes in Nepal.

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