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Optimization of River Diversion Structure and Cofferdam in Sunkoshi–Marin Diversion Scheme

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Abstract

A river diversion system has a large proportion of cost despite of it's temporary use . In this ground, it requires optimization of a diversion structure including conveyance system in terms of economic feasibility and safety. This study evaluates and optimizes the temporary diversion scheme with cofferdam in Sunkoshi-Marin diversion scheme. The optimization ensured economic viability and safety during construction. Two river diversion options (i) surface diversion and (ii) tunnel diversion were analysed and compared. Numerical analysis in HEC-RAS was carried out to calculate the requirement of channel width and depth of flow for floods varying from 2 to 20 years return period. The cost of different diversion options was calculated to optimize the surface option and compare with the sub-surface option. The cost of the diversion works was compared with the cost of loss due to the flooding of structures for the optimization of protection works. The analysis showed the cost of diversion works corresponding to the 10 years return period diversion flood well balances the loss due to flooding. The cost of diversion by tunnel is more than three times of surface diversion option. The study recommends surface diversion option as the optimal solution during the construction of Sunkoshi-Marin diversion headworks.

Keywords

diversion flood, HEC-RAS, Optimization, surface diversion, Sunkoshi-Marin, tunnel diversion

1. Introduction

River diversions consist of a system of structures and measures that intercept river runoff upstream of a project site, transport it around the work area, and discharge it downstream for the project construction operations. Despite of it's temporary use, a diversion system has a large proportion of total Engineering, Procurement, Construction (EPC) cost (Hong et al., 2023). In this ground, optimization of a diversion structure including conveyance system in terms of economic feasibility and safety is required (Hong et al., 2023). The design flood estimation of a diversion structure primarily depends upon characteristics of flood frequency, basin size, construction period, the type of main dam, and damage or loss caused by flooding during construction. The different design standards have propoesd different return periods for a diversion structure; for example, International Commission On Large Dams (ICOLD) suggests 10 years return period for a diversion structure for a concrete gravity dam (ICOLD (International Commission on Large Dams), 1986). The Korean dam design standard suggests 1 to 2 years of return period for the same condition (KWRA (Korea Water Resources Association), Republic of Korea, 2011).

Three principal aprroaches namely historical event-based, return-period, and risk-based govern the hydro-system infrastructure design (Tung et al., 2006). Among of them, return-period approach is onsidered as a practical approach. The risk-based approach consists of advanced procedure. It evaluates different alternatives considering the trade-off between the investment cost and the expected economic losses due to failure.

In Sunkoshi-Marin diversion multi-purpose project, headworks is proposed in the form of barrage consisting of a low-head dam with number of large gates control the discharge. Barrage gated spillways are proposed to pass design flood of 12,328.00 m3/s corresponding to 1,000 years return period and check flood 15,630 m3/s corresponding to 10,000 years return period. The construction of headworks covers 164 m long and 20 m height diversion dam across the Sunkoshi River. In addition, settling basin and flushing chute are proposed at the right bank of the river. The construction of these structures require temporary flood diversion structures to isolate the construction area for both construction purpose and protection of hydraulic structures as well.

In this background, this study aims to achieve the specific objectives (1) numerical simulation of diversion flow to check the reqirement for constrcition of channel width and (2) optimization of surface and sub-surface temporary river diversion structure of Sunkoshi Marin diversion headworks.

2. Study Area

Sunkoshi Marin Diversion Multipurpose Project (SMDMP) is proposed as a run-of-river basin diversion scheme planned mainly to provide irrigation facilities in Bagamti River Basin. The project aims to augment water at the head reaches of Bagmati Irrigation Project by diverting water from Sunkoshi into Bagmati River through Marin Khola, a major tributary of the Bagmati River. The project covers the area between 490 m and 390 m above mean sea level geographically.

Figure 1: Sunkoshi Marin headworks site at Khurkot

Figure 2: Weir axis at headworks site

The Headworks site is located in the Lesser Himalayan zone of eastern Nepal. It's location is at Khurkot of Sindhuli district, Bagmati Province (Figure 1, Figure 2).

3. Methodology

The methodology of study is primarily divided into two parts focusing on its objectives. First part includes the specific objective (1) with numerical simulation in HEC-RAS to calculate the requirement of channel width and depth of flow for floods varying from 2 to 20 years return periods. For this, the study requires flood frequency and peak flood analysis including diversion flood. The second part covers specific objective (2) in which quantities of principal construction items will be estimated for the river diversion works. Unit price data will be developed from rate analysis of each items of construction activities using established rate analysis procedures and available standard norms. For optimization, the cost of diversion with surface and tunnel diversion will be compared and analysed.

3.1 Numerical simulation

3.1.1 Flood Frequency Analysis

The selection of diversion flood estimation method depends upon the availability of data, importance of structure and the level of risk to be adhered. Design flood estimation may be carried out on the basis of either event-based or continuous simulation modelling methods [5,6]. Commonly three event-based approaches (a) probabilistic, (b) deterministic, and (c) empirical methods are used at-site design flood estimation [7,8]. Design flood estimation may be carried out by probabilistic method if adequate length and quality of historical data are available (Cordery and Pilgrim, 2014). Deterministic methods basically lump all heterogeneous catchment processes into a single process to enable the estimation of the flood event with correlation of rainfall event assuming the average catchment condition (Rahman et al., 2002). Empirical methods relate peak discharge to catchment size with incorporation of physiographical and climatological indices (SANRAL, 2013). Hence, statistical approach of flood frequency analysis is selected for deriving design flood making use of available long-term data set of Khurkot Station.

3.1.2 Peak Flood Estimation by Statistical Methods

Flood flow records are available at Khurkot gauging station (2 km downstream from intake site). Flood frequency analysis was adopted for design flood estimation from annual instantaneous floods from Khurkot station. Consistency of data were checked in computer software to identify the presence of trend and jump if any. The result of the consistency check is shown in Table 1.

Table 1: Check for Data Consistency Using Statistical Tools

H_0: Homogeneous data
H_a: Change in the data in a date

Test	99% confidence Interval	P-value	Remarks
Standard Normal Homogeneity Test	(0.483, 0.509)	0.496	Null Hypothesis H_0 cannot be rejected
Buishand's test	(0.251, 0.273)	0.262	Null Hypothesis H_0 cannot be rejected
Pettitt's test	(0.108, 0.124)	0.116	Null Hypothesis H_0 cannot be rejected

3.2 Diversion Flood

Statistical methods namely Log Pearson Type III distribution, Log Normal distribution, Pearson Type III distribution, Normal distribution and Gumbel distribution were used for flood frequency analysis. The results obtained based on those methods are tabulated in Table 2.

Table 2: Flood Estimates (m^3/s) in Khurkot Station under different methods

Return Period (Years)	Distribution				
	Normal	Log Normal	Pearson Type-III	Log Pearson Type -III	Gumbel
2	4123	3841	3886	3780	3856
5	5490	5277	5374	5247	5291
10	6204	6230	6297	6287	6241
20	6794	7145	7137	7335	7153

For all of the above mentioned distributions, some goodness of fit tests viz. Chi-square test, Kolmogorov-Smirnov and Andersn Darling tests were carried out for the peak instantaneous data. The outputs of goodness of fit tests are shown in Table 3.

Table 3: Goodness of fit test results

Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Square	
	Statistic	Rank	Statistic	Rank	Statistic	Rank
Gumbel	0.216	4	3.692	4	4.089	4
Log Pearson Type III	0.113	2	0.479	2	2.689	3
Log Normal	0.101	1	0.405	1	2.549	2
Normal	0.147	3	1.119	3	1.010	1

The Log Normal distribution was found to be the best fit distribution from the tests. The capacity of a diversion work is defined by the hydrological safety. In general, the larger the design floods, the higher the cost of diversion works. The selection of discharge capacity of the diversion structures is fundamental for (i) definition of cofferdam height (stage - discharge relationship) and (ii) definition of size of conveyance system such as diversion tunnel diameter, channel width and depth of submersible cofferdams.

The selection of the design flood for the diversion works depends on the risk analysis. A more conservative design flood has to be considered for situation where overtopping during construction would have disastrous results. A basic mathematical expression for damage is E=R*D where, E - mathematical expectation of damage, D - estimation of damages resulting from a failure of the diversion scheme (for instance overtopping and destruction of a cofferdam). Includes damages downstream and at site. T - return period of a flood for which no damage or destruction should occur, C - total cost of diversion scheme having required capacity (corresponding to T), R - Risk of occurrence of a flood larger than diversion capacity during diversion period. The damage is calculated by taking the certain percentage of the initial construction cost of the tunnel and coffer dam.

Based on site condition and volume of flood to divert, two river diversion options (i) surface diversion and (ii) tunnel diversion are considered for the study. This option consists of construction of cofferdam and flood walls to divert the river flow to the required direction and to isolate the construction zone in left and right banks respectively. The diversion works are planned in two stages.

3.2.1 Stage-I Diversion of River to Right Bank

This stage consists of diversion of river water towards right bank to isolate the construction works at left bank. The flood wall is proposed at the mid river such that the river flow is concentrated within 75 m from the right bank. The upstream cofferdam will divert the flow towards right bank. This diversion is chosen first as the river channel is deeper along right bank such that the flow can be easily diverted. In addition, this part of left bank is mostly dry for about 6 months each year.

3.2.2 Stage-II Diversion to Left Bank

After completion of structures at the left bank the river flow will be diverted towards left bank. The flood walls will be constructed at the edge or on top of the completed structures and a new upstream and downstream cofferdam at the right bank.

The hydraulic design of the channel proposes passing the flood of different return periods. The allowable mean velocity in the channel is assumed equivalent of 2 years return period flood. Numerical analysis in HEC-RAS is carried out to calculate the requirement of channel width and depth of flow for floods varying from 2 to 20 years return period. For this purpose, the base width of the stage -I channel (along right bank) is assumed to be 75m. The height of the wall is fixed by flood analysis in HEC-RAS.

3.3 Optimization of diversion options

The channel is proposed to divert the river flow around the construction area. Two channels for two stages of diversion are to be constructed. The channel for stage I is along the right bank. The stage II channel is on top of the concrete works constructed in stage I.

The cost of different diversion options will be calculated to optimize the surface option and compare with the sub-surface option. Quantities of principal construction items will be estimated. Unit price data are developed from rate analysis of each items of construction activities using established rate analysis procedures and available standard norms. The cost of the diversion works will be compared with the cost of loss due to the flooding of structures for the optimization of protection works.

4. Results and Discussions

4.1 Numerical Simulation

Numerical analysis in HEC-RAS is performed to calculate the requirement of channel width and depth of flow for floods varying from 2 to 20 years return period. During the calculation, the base width of the stage -I channel (along right bank) is assumed to be 75 m.

4.1.1 Stage- I Diversion

The 1D steady flow analysis for the computation of water surface elevation and flow velocity is performed using HEC-RAS. The flow area is considered through right bank only with the channel constriction of 75 m width (Figure 3). The simulation results are presented in Figures 3, 4 and 5 respectively. The details of outputs are presented in Table 4.

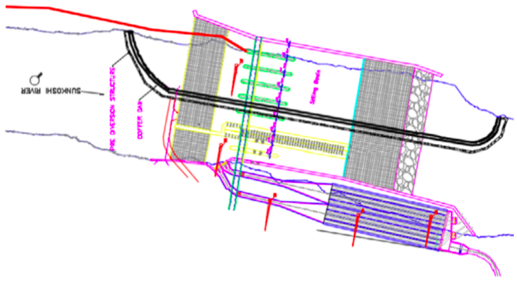

Figure 3: Location of stage-1 River diversion

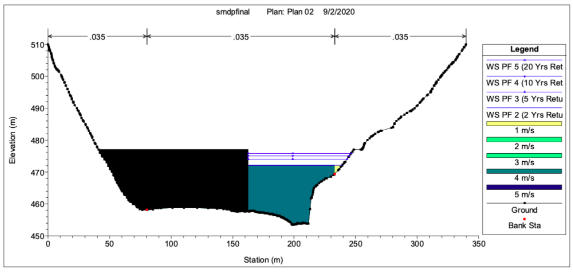

Figure 4: Cross section at weir axis

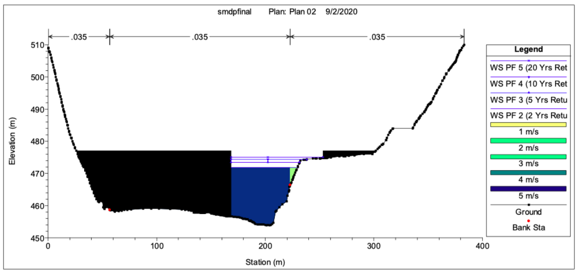

Figure 5: Cross section at 54 m d/s from weir axis

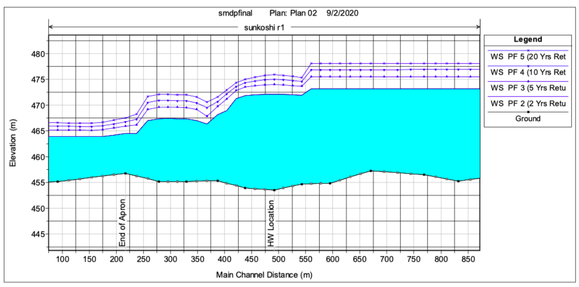

Figure 6: L-profile

Table 4: Water surface profile and velocity for stage -I diversion (75 m channel flow)

River Station	Profile	Discharge (m^3/s)	Min Channel Elevation (m)	Water Surface Elevation (m)	Channel Velocity (m/s)	Remarks
569.35*	PF 1	139.77	454.75	458.40	1.14	At 67.63m U/S from weir axis
	PF 2 (2 Yrs)	3841	454.75	467.38	2.63	
	PF 3 (5 Yrs)	5277	454.75	469.68	2.88	
	PF 4 (10 Yrs)	6230	454.75	471.07	3.03	
	PF 5 (20 Yrs)	7145	454.75	472.32	3.16	
551.936	PF 1	139.77	454.70	458.38	1.20	At 50.21m U/S from weir axis
	PF 2 (2 Yrs)	3841	454.70	466.17	5.33	
	PF 3 (5 Yrs)	5277	454.70	468.12	6.02	
	PF 4 (10 Yrs)	6230	454.70	469.27	6.42	
	PF 5 (20 Yrs)	7145	454.70	470.31	6.78	
535.20*	PF 1	139.77	454.31	458.37	1.15	At 33.48m U/S from weir axis
	PF 2 (2 Yrs)	3841	454.31	466.00	5.55	
	PF 3 (5 Yrs)	5277	454.31	467.90	6.28	
	PF 4 (10 Yrs)	6230	454.31	469.03	6.71	
	PF 5 (20 Yrs)	7145	454.31	470.04	7.09	
518.46*	PF 1	139.77	453.92	458.36	1.10	At 16.74m U/S from weir axis
	PF 2 (2 Yrs)	3841	453.92	465.77	5.86	
	PF 3 (5 Yrs)	5277	453.92	467.57	6.69	
	PF 4 (10 Yrs)	6230	453.92	468.64	7.17	
	PF 5 (20 Yrs)	7145	453.92	469.61	7.58	
501.723	PF 1	139.77	453.53	458.36	1.03	Weir Axis
	PF 2 (2 Yrs)	3841	453.53	465.90	5.46	
	PF 3 (5 Yrs)	5277	453.53	467.75	6.23	
	PF 4 (10 Yrs)	6230	453.53	468.85	6.66	
	PF 5 (20 Yrs)	7145	453.53	469.87	7.01	
483.72*	PF 1	139.77	453.65	458.36	1.00	
	PF 2 (2 Yrs)	3841	453.65	465.89	5.42	
	PF 3 (5 Yrs)	5277	453.65	467.77	6.13	
	PF 4 (10 Yrs)	6230	453.65	468.90	6.54	
	PF 5 (20 Yrs)	7145	453.65	469.92	6.88	
465.73*	PF 1	139.77	453.78	458.35	0.94	
	PF 2 (2 Yrs)	3841	453.78	465.96	5.18	
	PF 3 (5 Yrs)	5277	453.78	467.87	5.87	
	PF 4 (10 Yrs)	6230	453.78	469.00	6.26	
	PF 5 (20 Yrs)	7145	453.78	470.02	6.60	

The results justify the sufficiency of river constriction to 75 width in the right bank in stage–I of river diversion.

4.1.2 Stage II Diversion

The simulation is carried out for passage of diversion flood through left bank in stage II diversion work (Figure 7). At the end of stage-I diversion four barrage bays are supposed to be completed. The channel width is considered same as the total width of four barrage bays. Figures 8, 9, and 10 presents the flow simulation for 2-20 years return period flood.

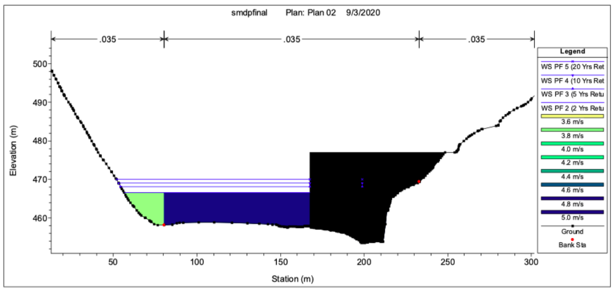

Figure 8: Cross section at weir axis

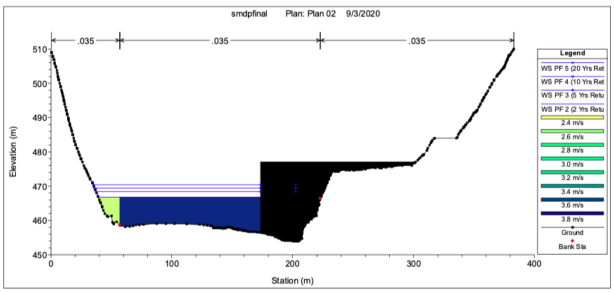

Figure 9: ross section at 54 d/s from dam Axis

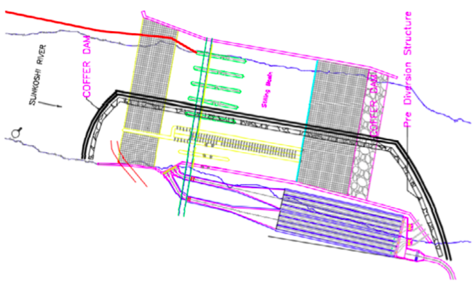

Figure 7: Location of stage-II diversion

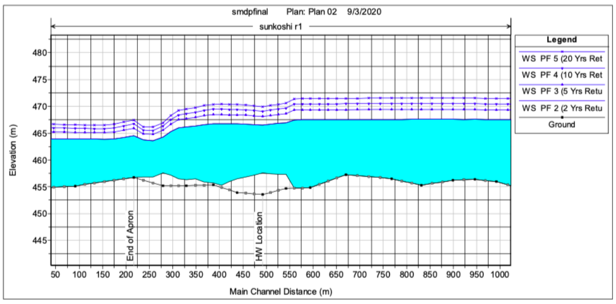

Figure 10: Longitudinal profile

The simulation results show that the water way equivalent to four barrage bays completed after the stage-I is sufficient for the diversion requirement in stage–II.

4.2 Optimization of diversion options

The cost of different diversion options was calculated to optimize the surface option and compare with the subsurface option.

4.2.1 Surface diversion

Quantities of principal construction items were estimated for the river diversion works. Unit price data are developed from rate analysis of each items of construction activities using established rate analysis procedures and available standard norms. The cost of the surface river diversion for SMDMP head work corresponding to different return period flood is summarized in Table 5.

Table 5: Cost of diversion at different return period flood

Return period (Years)	Cost of Pre-Diversion works (US$)	Cost of Cofferdam (US$)	Cost of seepage control (jet grouting) (US$)	Grand Total Cost (US$)	Cost for each stage (US$)
Stage I					
2	117,886.11	3,499,531.83	8,181.82	3,617,417.94	3,625,599.76
5	227,351.78	4,005,430.60	8,181.82	4,232,782.38	4,240,964.20
10	235,772.22	4,444,041.04	8,181.82	4,679,813.25	4,687,995.07
20	312,836.36	4,852,423.95	8,181.82	5,165,260.31	5,173,442.13
Stage II					
2	117,886.11	1,579,425.84	8,181.82	1,697,311.95	1,705,493.77
5	227,351.78	2,027,957.32	8,181.82	2,255,309.10	2,263,490.92
10	235,772.22	2,130,623.51	8,181.82	2,366,395.73	2,374,577.55
20	312,836.36	2,349,579.55	8,181.82	2,662,415.91	2,670,597.73

The overall cost of the diversion work including pre-diversion work and cofferdam is shown in Table 6. The variation of surface diversion cost with respect to return period is shown in Figure11.

Table 6: Summary of cost (US$) for different return period flood

Diversion stages	Return Period (Years)			
	2	5	10	20
Left (Stage-1)	3,617,417.94	4,232,782.38	4,679,813.25	5,165,260.31
Right (Stage-2)	1,697,311.95	2,255,309.10	2,255,309.10	2,662,415.91
Grand Total	5,322,911.71	6,496,273.30	6,943,304.26	7,835,858.22

The cost of diversion works was compared with the cost of loss due to flooding of structures for the optimization of protection works. The analysis showed cost of diversion works corresponding to the 10 years return period well balances the loss due to flooding.

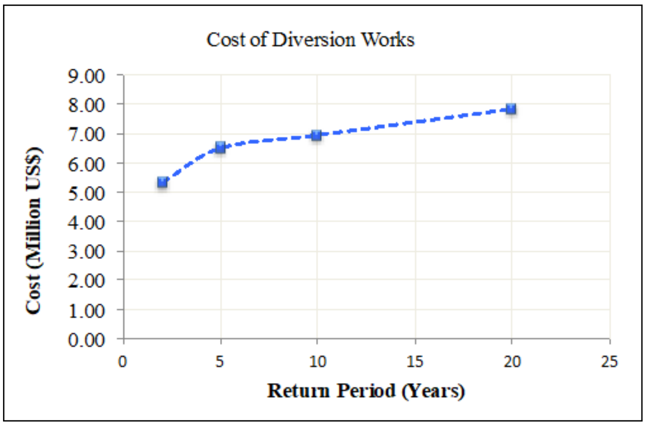

Figure 11: Variation of surface diversion cost with return period

4.2.2 Tunnel Diversion

The selection of the tunnel diameters is based on mainly two factors (i) discharge carrying capacity of the tunnel equivalent to 10 years return period flood and (ii) the geological, geo-morphological characteristics of the area including available construction methodology and technology. The diversion tunnels should carry the flood discharge of 6230 m3/sec during construction time. For smooth and safe passage of this flood two diversion tunnels of 10 m diameter are proposed. That diameter is the maximum size of tunnel for the existing geological condition of the area.

The proposed tunnel invert level is 458.27 masl at the inlet, dropping to 454.89 masl at the outlet over an average length of almost 1.325 km. Under normal and dry season flows, the tunnel will flow part full, but will generally flow full during the monsoon season. Flow velocities are high but not unprecedented. The upstream diversion cofferdam is developed from the initial closure or starter cofferdam and provides protection the downstream construction site. The cost estimates are based on unit price data from rate analysis of different items of construction derived using established estimating procedures. Quantities of principal construction items are estimated based on preliminary design calculation. The overall cost of the river diversion through tunnel is US$ 22,541,279.64 (Table 7).

Table 7: Summary of cost (in US$) for tunnel diversion option (10 years return period flood discharge)

Description	Cost (US$)
Pre- diversion work	235,772.22
Cofferdam construction	3,610,273.26
Tunnel construction	18,695,234.16
Total	22,541,279.64

4.3 Cost comparison

The cost of diversion with surface and tunnel diversion for 10 years return period flood discharge was estimated around US$ 6,943,304.26 and US$ 22,541,279.64 respectively. The estimated amount for tunnel diversion is more than three times of surface diversion (Figure 12).

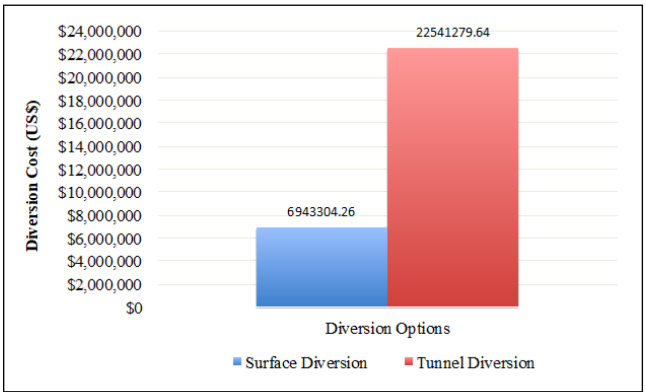

Figure 12: Comparision of cost for surface and tunnel diversion options

5. Conclusions

The study evaluates and optimizes the temporary diversion scheme with cofferdam in Sunkoshi-Marin diversion project. Two river diversion options (i) surface diversion and (ii) tunnel diversion were compared. Numerical simulation is performed using HEC-RAS to calculate the requirement of channel width and depth of flow for floods varying from 2 to 20 years return period. In stage –I diversion the flow area through right bank is sufficient with the channel constriction of 75 m width. In stage –II the channel width equivalent to total width of four barrage bays that is supposed to be completed in satge-I is sufficient for diversion requirement. The cost of the diversion works is compared with the cost of loss due to the flooding of structures for the optimization of protection works. The analysis shows cost of diversion works corresponding to the 10 years return period diversion flood well balances the loss due to flooding. The cost estimate of diversion with surface and tunnel diversion for 10 years return period flood discharge are around US$ 6,943,304.0 and US$ 22,541,279.0 respectively. The estimate amount for tunnel diversion is more than three times of surface diversion option. Surface river diversion option is recommended as the optimal solution during the construction of SMDMP headworks.

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