Landslide at Govindghat – Investigation and stabilization measures

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Abstract. A cloudburst and flash flood in June 2013 in the state of Uttarakhand, India caused landslide at Govindghat located along a major highway. Large boulders in soil matrix were encountered in the area. Water seeping through the open pores between the boulders and erosion of the soil were the major causes of the slope-failure. Erosion of the accumulated debris on the valley side during periods of high discharge in the area also added to the instability. The stabilization measures adopted include a secured drapery of rock anchors and steel mesh on the hill side, nailed gabions at road level, reinforced soil wall to ensure the desired road width and river-side gabion wall for erosion protection.

Keywords: Landslide, Cloudburst, Boulder-soil Matrix, Seismic Refraction tests, Hill-slope Stabilization, Erosion Protection.

1 Introduction

A cloudburst in the state of Uttarakhand in northern India triggered several landslides in the fragile Himalayan hill slopes and created havoc in the region. A flash flood in the river Alaknanda flowing in the region further exacerbated the situation. Several buildings and roads in the area were severely damaged / washed away. Of the many landslides that blocked traffic along the National Highway 58 as a result of this extreme event, this paper presents the case study of a major landslide at Govindghat. The causes of the slide and the stabilization measures adopted are discussed.

2 History and Background of the Slide

A satellite image in 2012 shows no landslide and overall stable conditions along the highway with vegetation on the hill-slopes (See Fig.1). Satellite image of 2014 illustrated in Fig. 2 shows a major landslide along the highway. Since this is the third landslide in the vicinity of Govindghat town, it is identified as Govindghat III.

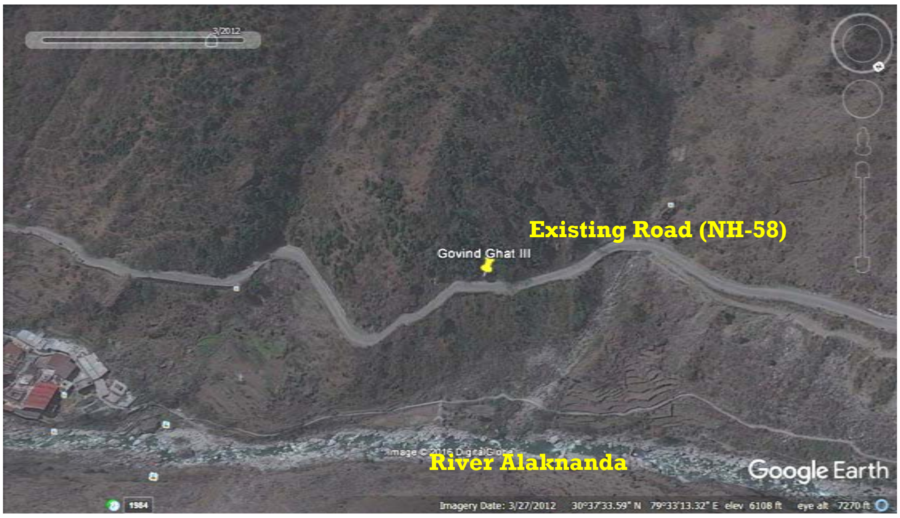


Fig. 1. Satellite Image – 2014 shows no landslide in the area

Fig. 2. Satellite image in 2014 shows major landslide above and below the highway

3 Site Conditions

3.1 Regional Geology

Geologically, the area belongs to a tectonic fore-deep in the Lesser Himalayas [1]. Govindghat belongs to the Central Crystalline group of rocks and is in the Pandu-keshwar Formation. The main rock types in this formation include whitish to grey quartzite inter-banded with mylonite-gneiss and garnet-kyanite schist [2]. The quartzites are medium to coarse grained, massive, cross-bedded and folded. The inter-banded mica-schists are crenulated and contain fibrolite, kyanite and garnet.

In the landslide area, rocks are not exposed except as a soil-boulder matrix. The boulders are angular and vary in size from less than 0.5 m to 5 m. These boulders are cemented in a sandy matrix. Fig. 3 presents photographs of the area.

Fig. 3. Slopes adjoining the highway at Govindghat with boulder-soil matrix. The photo on the left shows strata above road level and the one on the right shows the strata below road level

3.2 Stratigraphy

The landslide area at Govindghat III was investigated by two boreholes and one seismic refraction test. Limited boreholes were drilled because of site constraints; however the borehole data and seismic refraction test were sufficient to clearly assess the stratigraphy.

Fig. 4 illustrates the borehole drilling in progress. Fig. 5 presents a photo of the seismic refraction test being done.

Fig. 4. Borehole drilling in progress

Fig. 5. Seismic refraction test in progress

The study indicated the presence of large size boulders and cobbles in a soil matrix to substantial depth, probably extending to the riverbed.Most of it was landslide debris material. The higher recovery and RQD values are due to the large size of the boulders. Fig. 6 presents the seismic profile along with the borehole data superimposed on it.

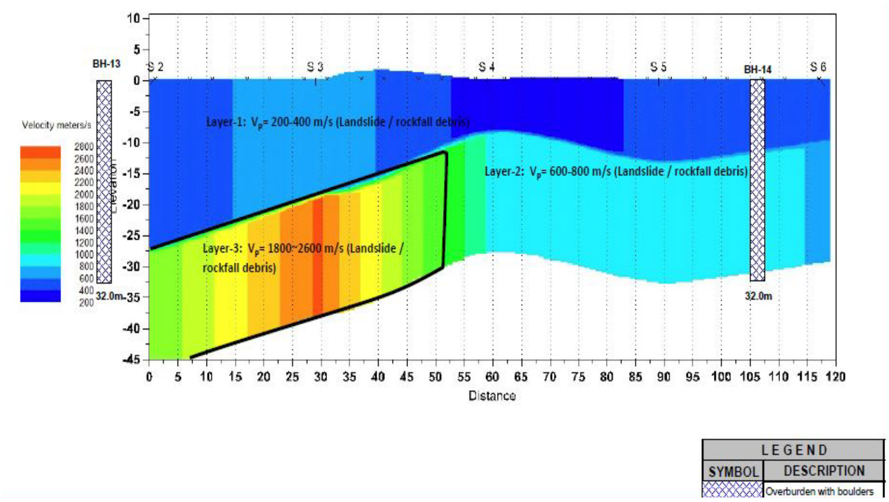


Fig. 6. Geophysical profile together with borehole data

4 Problem Description and Causes of the Slide

After a detailed study of the slide, the destabilizing forces identified include the following:

- Seepage of water through the open pores through the large boulders on the hillside;
- Erosion of the soil between the boulders-soil matrix during heavy rains, thereby triggering landslide in the foothill below the road; and
- Erosion of the toe on the river side during high discharge of the river.

These are discussed in detail in the sections below. The chainages mentioned are in two parts – the distance in kilometers (whole number) plus additional distance in metres from reference point. For example Chainage km 499+998 means the distance from the reference point is 499.998 km.

4.1 Chainage km 499+988 to 500+030

In a 42 m stretch between Chainage km 499+988 to 500+030, large boulders ranging in size from 1 m to 5 m are seen (See Fig. 7). There is substantial fine material between the large pores that tend to get washed away as rainwater percolates through them. This makes the boulders prone to sliding. Pore-water pressure builds up due to the water percolation which in turn triggers rockfall. The damage to the non-engineered hand-made gabion wall at the toe of the slope highlights the problem.

Fig. 7. Stretch with large boulders and open pores. Note the damage to the non-engineered handmade gabion wall

4.2 Chainage km 500+030 to 500+138

In this stretch, the the material on the hillslope consists of loose overburden soil and fractured rock (See Fig. 8). The fines get eroded due to seepage during heavy rains and this triggers the landslide. Consequently, the soil debris causes water-logging of the road as illustrated on Fig. 9.

Fig. 8. Loose soil with boulders

Fig. 9. Water and debris accumulated on road

4.3 River side Erosion

Erosion of the toe along the Alaknanda River at Govindghat is also one of the causes of landslides in the area. The loose alluvium accumulated on the toe of the river gets eroded, particularly during floods and periods of high discharge during monsoons. This increases the instability, thereby creating conditions favorable for a landslide. The impact of toe erosion is illustrated in Fig. 10.

Fig. 10. Toe erosion on valley side

5 Stabilization Measures Adopted

Stabilizing measures implemented include the following:

1. For retention of proposed 12-m wide road, reinforced soil wall of 8 m maximum height has been constructed. The wall is designed for the traffic load and very high seismic loads (as per BS 8006 Part-1 [3] and for seismic loading condition, FHWA-NHI-10-024 [4]). The stability analysis was performed using inhouse built software for both static and seismic conditions. This software allows the user to conduct the stability checks using the Limit Equilibrium Method and to do checks of different possible failure mechanism in heterogeneous strata layers with all possible load combinations described in BS 8006.

Reinforced soil system is advantageous in terms of overall performance, better uniform pressure distribution at the base of the wall and environmentally friendly. Since the base is flexible, differential settlement can be accommodated.

A typical of software analysis is presented in Fig. 11 and the summary of factor of safety (FoS) is given in Table-1.

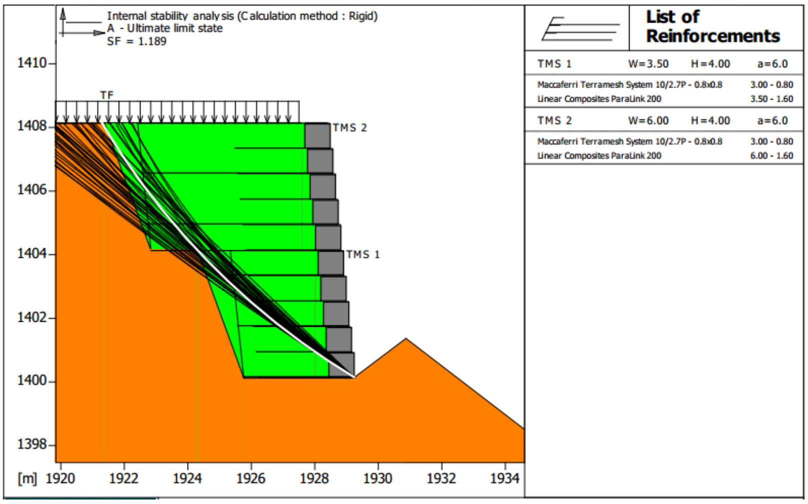


Fig. 11. Slope stability analysis of 8m high reinforced soil wall

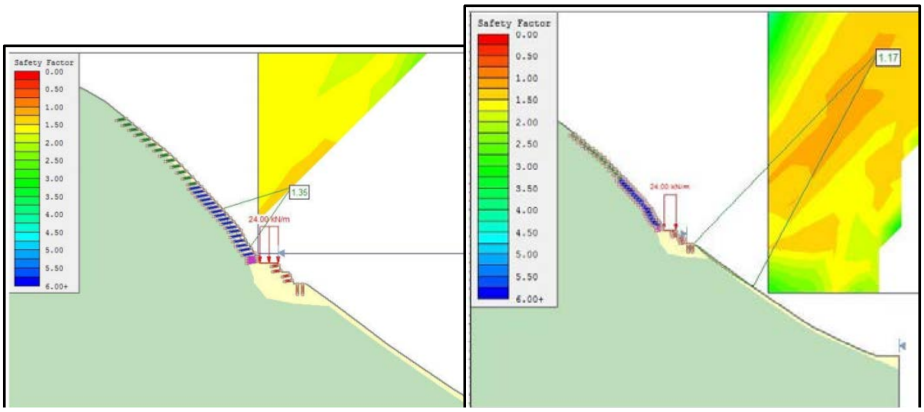
Table -1 Summarized factor of safety for 8m high reinforced soil wall

Stability checks	FoS (Static case)		FoS (Seismic case)	
	Required (BS 8006 Part-1)	Actual	Required (FHWA-NHI-00-043)	Actual
Global stability	1	1.377	1	1.197
Sliding	1.3	4.845	1.125	2.242
Bearing capacity	1.35	1.150	1.875	2.598
Overturning	2	8.255	1.5	1.784
Internal stability	1	1.189	1.125	1.272

2. For protecting the toe on hill side, nailed gabion wall of 3m height was provided for entire stretch. By providing nailed gabion, cutting into the hill is reduced as compared to gabion retaining wall, thereby minimizing disturbance to the in-situ strata.

   Gabions act as a robust fascia for the nails which helps in preventing erosion of the toe and also act as a drainage toe. The nails provide the stabilization effect.The analysis of nailed slope with gabion facia/drapery system (steel mesh as facia) and global stability of overall system was carried out using software – ‘SLIDE’ of Rocscience Inc., USA. This software evaluates the safety factor or probability of failure of circular or non-circular failure surfaces using vertical slice or non-vertical slice limit equilibrium methods

   A typical software analysis output is shown in Fig. 12and the summary of factor of safety (FoS) is given in Table-2.



(a) (b)

Fig. 12. Slide analysis results (a) for hill side (static) FOS = 1.35 (b) for valley side (seismic) FOS = 1.17

Table -2 Summarized factor of safety hill side and valley side slope

Stability checks	FoS (Static case)		FoS (Seismic case)	
	Required (IRC HRB 15)	**Actual**	**Required (IRC HRB 15/IRC 75)**	**Actual**
Hill side	1.3	1.35	1.05	1.06
Valley side	1.3	1.61	1.05	1.17

3. On the hill-side, secured drapery consisting of rock anchors with steel mesh was used to secure the large boulders (designed as per Indian Roads Congress IRCHRB Special Report 23 [5]) all along the road in the landslide zone. For, the design of secured drapery, inhouse built software was used. The analysis for the design of for hill side slope protection and rock-fall mitigation was performed using this software, a limit equilibrium approach for the design of secured drapery. A typical output file of the analysis can be seen in Fig.13. The summary of factor of safety used in the analysis for seismic conditions is given in Table 3. Suitable bioengineering measures are being implemented to enhance the stability of the area.

Table -3 Summarized factor of safety rockfall mitigation (seismic case design)

Stability checks	Required FoS	Actual FoS
Bar design	1	1.61
Mesh design	1	6.43
Serviceability design	1	2.36

Results

Bar design check (Slope SF)		Mesh design check		Serviceability design check	
1.61	Satisfied	6.34	Satisfied	2.36	Satisfied

Bar design

Stabilizing forces [kN]	312.86
Driving forces [kN]	194.38
Ratio Stabilizing/Driving forces	1.61
Angle between perpendicular to slope and bar axis [°]	35.00
Minimum acceptable steel yield stress [MPa]	474.14
Effective cross section of bar [mm²]	525.83
Sliding plane stabilizing forces - per anchorage [kN]	215.59
Minimum drilling diameter (NOMINAL) [mm]	47.00
Anchor pull-out force from load on the mesh [kN]	8.62
Anchor pull-out force due to global instability [kN]	55.70
Maximum pull-out force (total) [kN]	55.70
Minimum bar length in the stable rock mass [m]	1.60
Minimum length (bar) in the unstable rock mass [m]	2.80
Minimum total bar length [m]	4.70

Serviceability

Maximum acceptable displacement [m]	0.26
Calculated mesh displacement [m]	0.11
Displacement Ratio	2.36

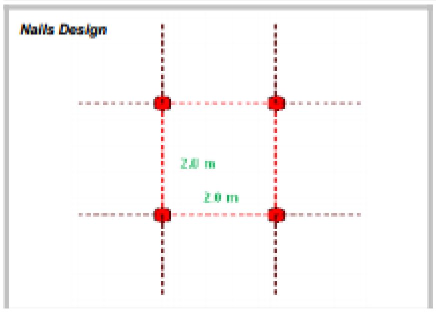
Mesh design

Admissible tensile stress of the mesh [kN/m]	48.80
Max. tensile stress within the mesh [kN/m]	7.69
Force-strength ratio	6.34
Potential unstable volume on joint - case A [m³/m]	0.54
Potential unstable volume on joint - case B [m³/m]	0.00
Potential unstable volume on joint - case C [m³/m]	0.00
Maximum rock volume that can slide between anchors [m³/m]	0.54
Maximum rock weight that can slide between anchors [kN/m]	13.93
Sum of driving forces acting on the sliding plane [kN/m]	10.24
Sum of stabilizing forces acting on the sliding plane [kN/m]	4.61
Punching forces acting on the mesh [kN]	2.91
Average angle between deformed mesh plane and rock surface [°]	4.09

Features of the instability

Pressure on the average slip surface [MPa]	0.05
Initial dilatance of the most dangerous joint [°]	0.08
Total unstable volume controlled by each anchorage [m³]	8.00
Total unstable weight controlled by each anchorage [kN]	208.00

Geometry



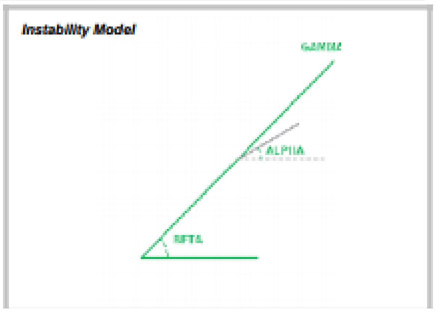


Fig. 13. Analysis of secured drapery system

4. Runoff water was channelized by providing adequate drainage measures including surface run-off deflector and roadside drain which discharges the water collected into a nearby culvert as shown in Fig. 14 (designed as per IRC SP 42 [6]).Fig.15 illustrates the scheme adopted.Rainfall intensity wascalculated as per IRC:SP013 and was measured as 14.58 cm/hr. Catchment area delineation was done in Arc-GIS software was used and was calculated as 5.1 hectare. Design discharge from catchment was calculated as per IRC SP 42 and was come out as 1.66 m^3/sec. Manning's equation was used to calculate allowable discharge for roadside drains and was come out as 2.71m^3/sec with the factor of safety of 1.63.

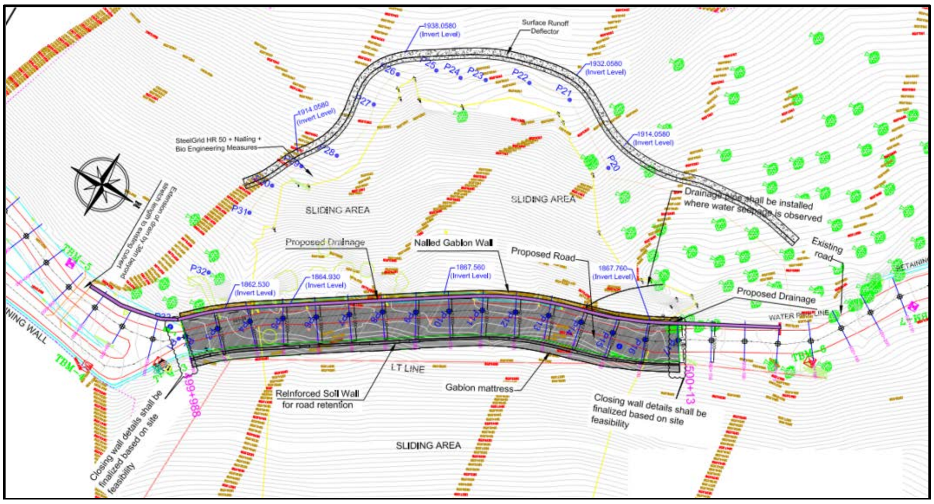


Fig. 14. Drainage plan showing mitigation measures on hill side of the road

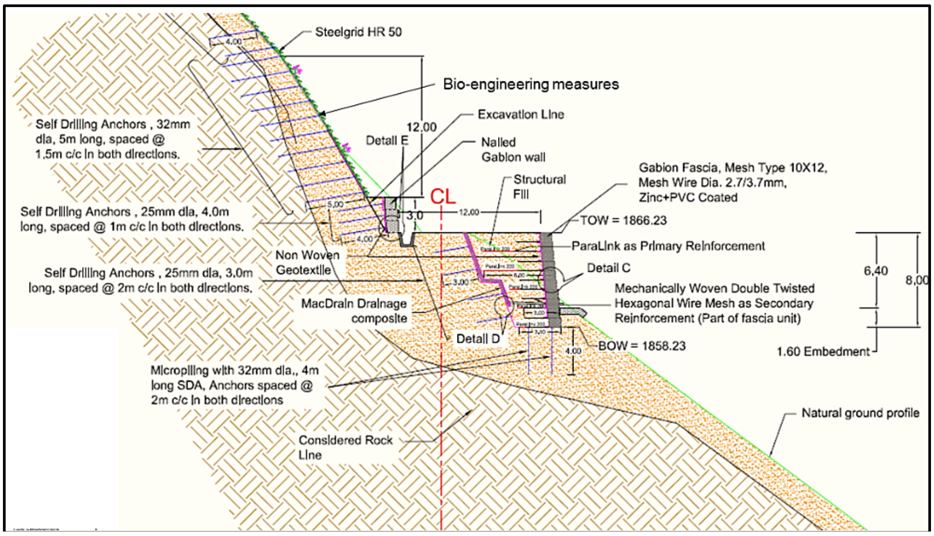


Fig. 15. Reinforced soil wall with secured drapery on hill side and nailed gabions to arrest rockfall (schematic sketch, not to scale)

5. For protecting the toe exposed to the river, a gabion toe-wall has been provided for the whole 150m stretch along with the gabion mattress as launching apron to protect the exposed toe and reduce erosion as per IRC 89 [7]. The mattress was covered with big boulders (min. 800 mm size). The gabion mattress act as a launching apron and boulders provided added protection against large boulder impacts to the structure flowing along the river during floods. The protection has been done to 1

m above the high flood level. Fig. 16illustrates the details. The stability analysis of gabion toe wall was performed using inhouse built software. The stability of gabion toe wall i. e. both internal and external stabilities were checked using limit equilibrium method and the global stability was checked using theories of Bishop (1955)[8]. &Jambu (1956)[9]. External stability of each individual gabion wall against sliding, overturning and foundation bearing pressures were checked and verified. Internal stability of gabion layers was checked by taking apparent cohesion and apparent angle of internal friction between gabion layers. A typical output of software analysis is presented in Fig. 17 and the summary of Factor of Safety (FoS) is given in Table 4 and Table 5.

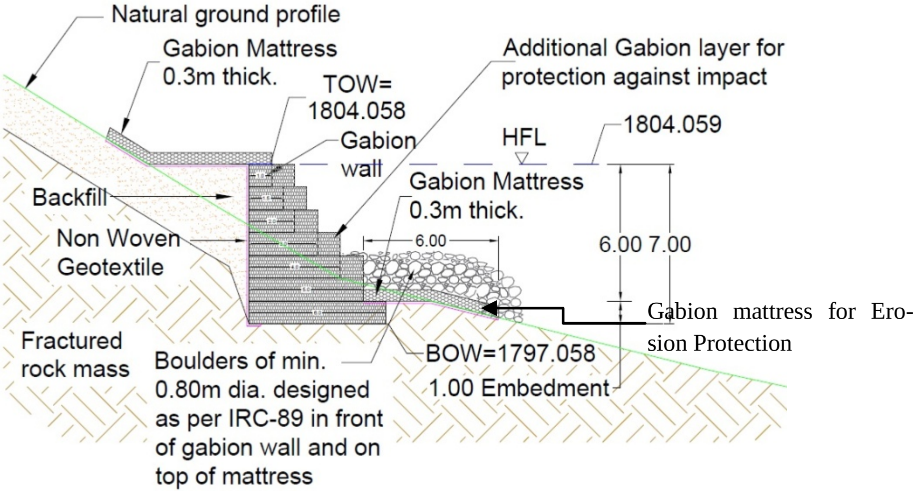


Fig. 16. River-side Gabion Wall for bank protection(schematic sketch, not to scale) Note: HFL: High Flood Level (m)
TOW: Level of Top of Wall (m) BOW: Level of Bottom of Wall (for 1 m embedment)

The gabions and mattress provided for the project were as per IS 16014 [10].

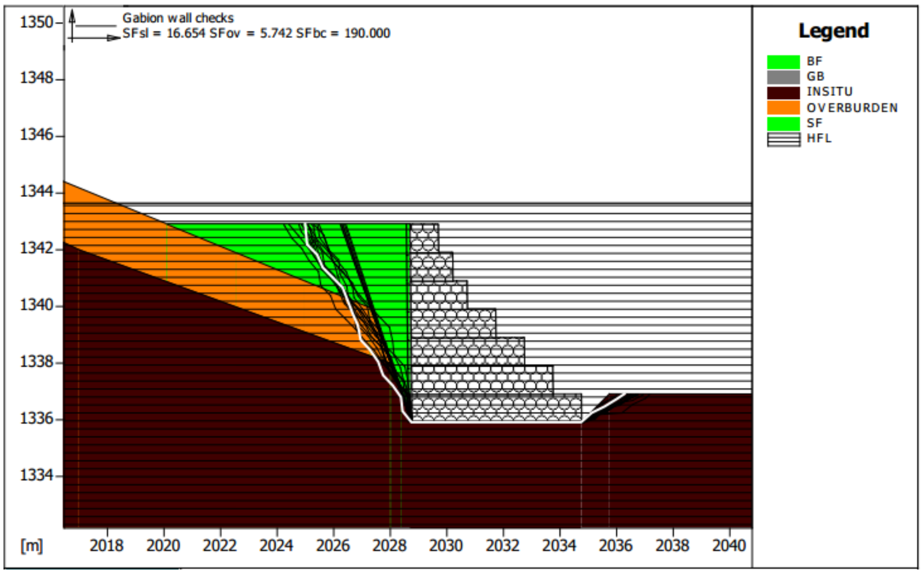


Fig. 17. Slope stability analysis of 7m high gabion toe wall

Table -4 Summarized factor of safety for 7m high gabion toe wall (Static case)

Stability checks	**FoS (Static case)**		
	Required (IS 14458 Part-2)	**FOS for 7m design height atHFL**	**FOS for 7m design height with Drawdown**
Global stability	1.3	2.177	1.705
Sliding	1.5	16.654	6.585
Bearing capacity	2.5	190	63.750
Overturning	2	5.742	2.159

Table - 5 Summarized factor of safety for 7m high gabion toe wall (Seismic case)

Stability checks	**FoS (Seismic case)**	
	Required(FHWA-NHI-00-043)	**FOS for 7m design height at LWL**
Global stability	1	1.409
Sliding	1.125	5.765
Bearing capacity	1.875	60
Overturning	2.5	3.845

6 Present status

The project is recently completed and the photographs of the adopted remedial measures at the site are shown in Fig. 19, Fig. 20 and Fig. 21.

Fig. 19. Photograph showing the hill side rehabilitative measures (3-m high nailed gabion wall for toe stabilization and secured drapery system to secure large boulders adopted at site

Fig. 20. Photograph showing reinforced soil wall constructed at site for road retention

Fig. 21. Photograph showing the bank protection works adopted at site

7 Concluding Remarks

A landslide was triggered on the hill-slopes of National Highway 58 at Govindghat in Uttarkhand state, India by a cloudburst and flash flood in June 2013. The geotechnical problems identified were:

– Seepage of water through the open pores through the large boulders on the hill-side;
– Erosion of the boulders and soil during heavy rains, thereby triggering landslide in the foothill below the road; and
– Erosion of the toe on the river side during high discharge of the river.

Engineering solutions adopted to stabilize the area include the following:

- stabilizing the hill-slope above road level with a secured drapery of rock anchors and steel mesh;
- installing 3-m high nailed gabions at road level for toe stabilization;
- constructing a reinforced soil wall on the valley side (max. 8 m high) to ensure that the desired road-width of 12 m is achieved with necessary stability;
- adequate drainage planning to channelize water into drainage channels for disposal; and
- protecting the toe erosion on river-side with a gabion wall and mattress.

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