A Technical Note on

Landslide Protection in Sinking Zone of Darjeeling Himalaya Using Reinforced Soil Technique

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1. INTRODUCTION

Protection walls along the hill and valley side of a road bench form an important feature of hill roads. Conventional masonry and stone infilled gabion structures constructed as breast or toe retaining walls, often show signs of distress within one or two seasons of its construction especially in sinking zones and areas subject to significant lateral ground movements. These are relatively rigid structures and cannot adapt to ground settlements or wall deformations as efficiently, when compared with a reinforced soil construction. Besides, they also impose a higher structural weight to the foundation, being an externally stabilised system. A reinforced soil construction on the contrary is an internally stabilised system. Here, the reinforcing layers in the reinforced soil zone interact with the soil-reinforcement composite system in order to resist potential shear failures or slips. The foundation pressures are also spread over a wider base width and hence are more effective for construction in sinking hilly terrain.

The flexibility of the structure, without causing any structural distress is the prime beauty of reinforced soil construction. This is of particular advantage in areas subjected to seismic effects and dynamic loading.

For construction of moderate to high retaining structures in hilly terrain, the above considerations are helpful over conventional systems.

2. MAJOR CAUSES OF LANDSLIDES

Out of 17,00,000 km roads in India, 44,000 km are under Himalayan terrain. On an average, about 24 million cubic meters of debris are produced by mass movements on roads in the Himalaya each year. The natural causes of landslides are lithology (rock and slope types), structure of rock and slope, morphology of slope, relative relief, drainage and vegetative cover. The main natural catalysts to the cause of landslides are earthquake and heavy rainfall. The Himalayas are young in their formation, fragile, geologically unstable, and seismically active. They undergo metamorphosis and changes in their constitution lead to the formation of destabilising forces within the soil structure. The Himalaya is longitudinally dissected by faults. These faults are neo-tectonically active and tend to converge northwards towards Tibet upto a distance of 1500 km. The slip rate across Nepal is about 18mm / year and the Himalayas are rising constantly at a rate of 2 to 12 mm/year. This makes the Himalaya seismically active that causes weakness of rocks and slopes, prone to landslide. Besides, rainfall is the single largest exogenous factor triggering off landslides in this region. In Darjeeling Himalaya the average rainfall recorded is 4000mm / year during 1997 - 98 with a peak rainfall intensity of 500mm / day during July 1998 [Source: Darjeeling Highway Division, PWD(Roads)].

Deforestation for livelihood and construction of roads (blasting sometime leads to liquefaction of soil) also cannot be ignored as significant landslide triggering factors.

3. MAJOR CONSTRAINTS FOR THE CONSTRUCTION OF REINFORCED SOIL STRUCTURE IN HILLY TERRAIN

The geomorphological and topographical nature of the hilly terrain may create many constraints from the point of design and construction. The main problems while constructing a reinforced soil structure in a hilly terrain are enumerated below:

- Lack of firm and stable foundation.

- Excavation instability that limits the scope of excavation (reinforcement length, to the face of the structure) limiting the height of construction in many cases.

- Intermittent occurrence of bed-rock that limits the reinforcement length in the reinforced zone. The bedrock is an incompressible stratum that cannot fail internally. Standard design methodology may be appropriate for assessing the internal stability of reinforced slopes, while their use for assessing the overall dimensions for external, global and compound stability failures is not appropriate.

- Presence of unaccounted groundwater sources and seepage lines falling into the excavation in the reinforced zone. Uncompacted soils with high hydraulic conductivity resulting in easy penetration by groundwater. Creation of new surface run-off paths, gullies and ‘jhoras’ due to construction activity, that was unaccounted in the design.

- Overall instability of the hill slope and terrain due to geological instability of the mountains in the area.

- Non-availability of requisite construction equipments and selective backfill material due to inaccessibility to site and high lead of transportation, low labour skills and productivity rate.

- Non-availability of front-space for mobilisation of tools and plants, storage and handling of construction materials.

- Presence of rivers, streams and brooks at the toe of the hill bench. The water course in many cases tend to shift velocity flow vectors due to debris deposition at toe (that restrict flow paths and create short spurs) from landslides in mid-slope areas of hill along the course. The shift in course is very unpredictable and sudden, especially during high discharge seasons (monsoon months). This leads to instability of the lower slope reaches of hill bench either from landslides (that create debris deposition at toe) or scour at the toe from change of flow paths.

- Unpredictable behaviour of the upper reach hill slopes from muck and debris flows, rock topple, multiple slips, etc. that change the geometric configuration of the reinforced soil construction.

Each of the above has a considerable effect on the final design of the reinforced slope, and many of these events are often impossible to be foreseen with some degree of certainty until construction is in progress.

Thus, specific project sites can result in significantly different solutions compared to those provided in standard design charts and drawings.

Under these conditions, the design, construction and service performance of a reinforced soil structure in hilly terrain possesses a significant challenge to the engineer.

4. THE REINFORCED SOIL PROTECTION WALLS IN DARJEELING HIMALAYA

The Darjeeling District of West Bengal State, India lies in lower Shivalik range and is very prone to landslide due to the unstable formation of hills in this region.

PWD (Roads), Government of West Bengal, and BRTF (under project Swastik) are primarily responsible for the construction and maintenance of roads in this territory. It is indeed the most challenging task to maintain the integrity of such hill roads in this area, during and just after the monsoon rains. Natural hazard and disaster management often becomes a routine function of the concerned officials, as a part of the civil engineering activity.

The two most dreaded areas under the preview of the PWD (Roads) are “Paglajhora” and “Bwalukhope” sinking zones. These areas have a long history of being distressed during and just after the rains. During the other seasons, the topography of the area look normal, just like any other hill bench section. But, during the rains muck flow, slides and slips are a common feature of this area.

Different options for earth retention using gabions, rubble masonry, etc were tried by the department for the past few years, but only to limited success. These walls could only function well for merely one season or so and thereafter collapse or show severe signs of distress.

As a non-conventional technique, the reinforced soil option was adopted on an experimental basis in the Bhalukhope sinking zone and other zones in Darjeeling hill roads. The most critical stretch of the hill road in this zone was selected to assess the efficacy of this technique.

The general features of the protection wall and the engineering aspects are given in Fig. 1 and described in the section below.

4.1 Engineering Considerations

Reinforced soil walls of height varying from 2 metres to 6 metres were constructed on the hill side and valley side of the road benches in the various areas of Darjeeling hills. The total cumulative length of the wall was in excess of 100 metres. Most critical locations were at Bhalukhope towards Mongpoo and Hotel Golai towards Mirik. All these walls were designed for a vertical slope of 70° to horizontal using wrap around facing construction.

4.2 Design Basis

The design of the reinforced soil toe wall was performed in conformance with the RTA QA Specification R57, Edition 1, Revision 0 - August 1997 that provides guidelines for "Design of reinforced soil walls", a publication of Roads and Traffic of New South Wales, Australia using Jewell's charts (1990). The design was also verified by using the MGRSW software program available from TCMirafi, USA.

A tie-back method of internal stability analysis and a gravity-mass external stability analysis was used for design and this is consistent with the current practices in geotechnical engineering. Rankine's earth pressure theory was used for both internal and external stability analysis.

The properties of soil in the reinforced soil used for design are as follows:

Soil type	*Friction angle*	*Cohesion*	*Bulk unit weight*
Silty sand (with presence of clay)	28 degrees	Ignored	20 kN/m^3

The friction angle is determined by conducting consolidated un-drained direct shear tests. It was assumed that the soils in the reinforced zone exhibit uniform strength properties and is isotropic and homogeneous. Due to the presence of bed-rock at shallow depths the possibilities of slip circle failures were eliminated.

A dead load surcharge of $10kN/m^2$ on crest was considered in the design to account for the collection of debris at the top of the wall. A nominal embedment of 300mm was considered in the design to minimise the effect on the disturbance of foundation. The minimum reinforcement length used was 2m which was considered to be adequate, as grid form reinforcements develop adequate pull-out resistance at shorter lengths than strip forms of reinforcement.

Effect of seismic loading was not considered in design because of lower wall height though the zone is under classification VI as per IS:1893:1984.

4.3 Soil Reinforcing Element

The soil reinforcement material used for construction were polyester geogrids having an ultimate strength of 55 kN/m and 40 kN/m and a long-term design strength of 17.6 kN/m and 12.8 kN/m respectively, for 50 years at 40 percent ultimate strength for 1 percent post-construction serviceability strain requirement. The effect of uncontrolled construction damage factor was considered to reduce the ultimate strength by 20% (i.e. FOS, construction and installation damage = 1.25). Effects of environmental damages were ignored. The factors of safety towards manufacture and variation of test results (MARV were used) were considered unity. The commercial name of the product is MIRAGRID 5XT and MIRAGRID 3XT respectively.

The geogrid is consisting of PET fibres that are knitted or woven together to form well defined grid pattern with specific aperture sizes and then coated with PVC. The coating penetrates down into the fibres and become an integral part of the structure. Besides acting like a protective layer against chemical degradation in adverse soils, the caoting also helps in improving the dimensional and UV stability and reduce construction damage effects. Manufacturers have chosen PET as the base polymer for geogrids for their high strength to weight ratio, relatively lower creep property and ease in construction and installation.

PET geogrids are stretched during their manufacturing process, that causes the long molecular chains to realign in the direction of strain thus providing much higher tensile strength than it would otherwise. This production process called “preferred orientation”, which is a polymeric equivalent to strain - hardening of steel.

Also the well-defined apertures in the geogrid allow the soil to interact with all surfaces of the ribs and the granular fill material can “strike through” for maximum pullout resistance.

PET geogrids, with their rougher surfaces reply much more on the development of friction and all-surface areas of the grid. For these geogrids, the bearing resistance of the transverse ribs and the junction efficiency are not relied on for working within the working loads defined by the long-term design strength, as has been demonstrated by pull out testing with the transverse ribs removed.

4.4 Backfill Soil

Locally available soil was used, with a blend of imported select backfill material for use in the reinforcement soil zone. The local material consisted of a high percentage of fines which have potential to develop secondary consolidation or creep from realignment of the soil skeleton. This in turn pose problem of internal settlement and wall deformation. The earth works was done in soil lifts of not exceeding 200mm by using baby rammers and manual tamping. However, it was conceived that a compaction less than 70 percent of standard proctor density could have been achieved by using this procedure. The constraints towards availability of equipment and skilled work force have already been discussed earlier.

4.5 Drainage Considerations

Poor internal drainage from ingress of water from retained backfill zone and from saturation by groundwater leads to accumulation of pore water (hydrostatic pressure), that creates considerable destabilising forces for the reinforced structure. One of the most important reasons of failure of earth retaining structure in these areas is improper drainage provision. This issue was therefore critically examined in the design. Accordingly, transient pore pressure coefficient of 0.2 was considered in the design. This corresponds to 2m of water head at the base of the structure for a wall of height of 5.0m.

Internal drainage require lesser attention for backfill soils having a permeability greater than 10^{-3} m/sec. or containing less than 5% of fine elements smaller than 80 microns. Such materials could be classified as self-draining. However, the drainage will require critical attention when a rapid draw down takes place and the permeability of fill is insufficient. It is advisable to provide drainage galleries to cut-off the pheratic lines in order to arrest hydrostatic pressure build-ups within the reinforced soil zone.

In order to prevent excess build-up of pore water pressure under short-term considerations, a 450mm thick aggregate drainage bay (chimney drain) was provided behind and below the reinforced soil wall. A non-woven geotextile (Mirafi 180N) satisfying piping and permeability criteria was used to wrap the chimney drain to prevent contamination of fines in the aggregate drainage layer from leaching of soils from behind and below the structure. The geotextile choice was also governed by the mechanical properties of the product, based primarily on construction damage factors. A 150mm diameter PVC perforated pipe was positioned at the toe of the wall, with a longitudinal slope of 1 in 400, leading the collected water to the culvert positioned towards the end of wall. Proper quality control in gradation of the aggregate as designed for (19.1mm to 9.1mm well graded) could not be maintained at site due to accessibility and availability of material.

4.6 Facing

Reinforced soil wall and slopes consist of select backfill soil, reinforcing element and a form of facing. The facing is considered to provide a mere support to take care of the local erosion and ravelling of soil caused by weathering. It is also required to provide an external form and acceptable finish to the structure. Structurally, it also provides local support to the soil between reinforcement layers in the active zone.

Though it is conceived that no genuine forms of earth pressure is transmitted on the facing, in reality however, some amount of the horizontal soil pressure and reinforcement tension reaction are transmitted to the facing during the construction of the wall and also in service stages. Apart from this, the facing may also be subjected to relative deformations and settlements in the structure. For wrap around construction facing, the tolerance towards settlement and lateral deformation of the structure is much greater than the rigid forms of the facing using discrete panel or moulder blocks.

Hessian bags were filled with agricultural soil with fertilizers and wrapped around with geogrids with a tie length of 900mm. Seeds to grass the face where impregnated in the gunny bags. The type of vegetation selected was the variety that could grow fast in the lateral direction (horizontal creeping variety of grass) and establish a good matrix with the facing soil. Eventually, the vegetative cover would erupt through apertures of the geogrid and provide a green finish, coherent with the natural scenery of the area (Plate 1). The local vegetation would also flourish in the tropical environment in this area.

Vertical spacing limits of reinforcing geogrids were restricted to 300mm to take care of lateral deformations.

5. PERFORMANCE OF REINFORCED EARTH SOIL

The reinforced soil wall at all locations performed satisfactory to the expectation levels, considering the site and workability constraints. The use of marginal fills and inadequate compaction resulted in some uncommon deformation and internal settlement of the structures. In the Bhalukhope wall, such occurrence was aggravated because of the presence of jhoras behind a portion of the wall. However, these are merely serviceability issues and not quite important for such hill roads, especially on the hillsides. However, these are critical considerations when such structures support the road bench. The wall was designed to provide adequate factor of safety against limit equilibrium collapse mechanisms. The potential lateral deformation and settlement were ignored at design stage due to unexpected difference that could arise during construction and the design board (Plate 2).

6. CONCLUSIONS

The practical constraints of construction of Reinforced Soil Wall in difficult and inaccessible hilly terrains have been well understood, considering the critical differences between design and construction that could occur in such sites. No formal instrumentation and monitoring programme was done for the wall, but visual inspection during and after the construction of wall and thereafter when the structure had served more than two monsoon season, showed that none of the structures were in danger of collapse at any time during the service. These cases clearly indicate that geosynthetics can be used effectively for the limited landslide control programmes in Indian conditions, where very difficult and different environment, work and material control persists. Outward deformation and settlement are not indication of the failure of the structure in such hill roads which are a resultant of pre-adjustment of the stresses in composite soil reinforcement system. However, use of select quality fill and proper compaction tools and quality control can take care of deformation and internal settlements in the structure. Additional set back and facing pre-batter also help in controlling the final wall alignment. The use of clips and pins in the wrap and tie reinforcements can also help to control the movements in some cases.

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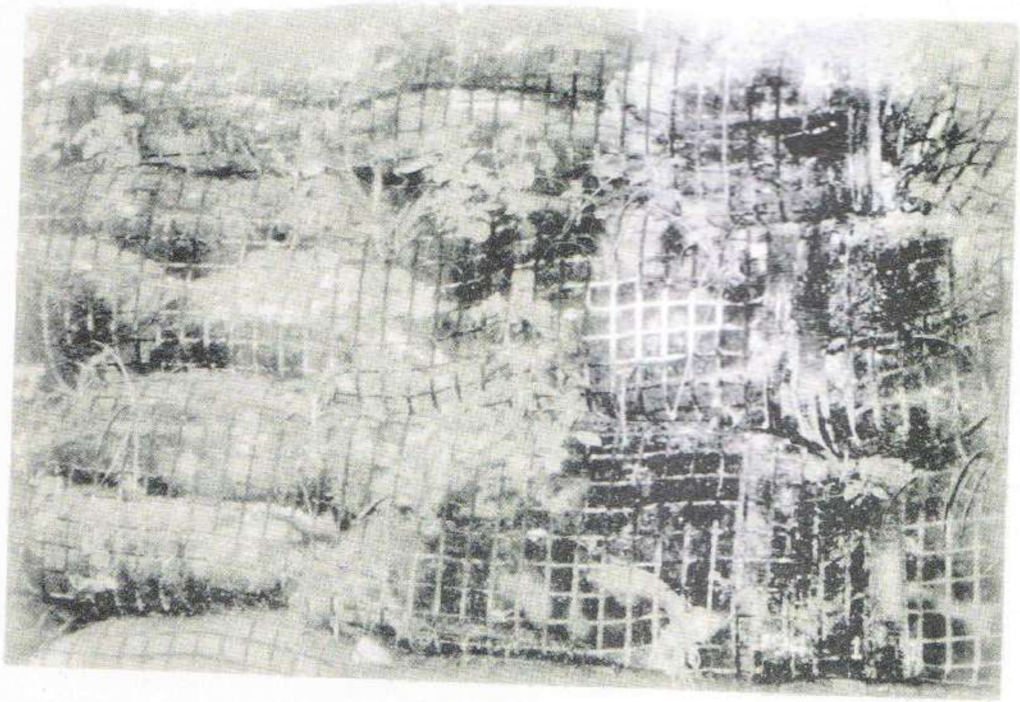

Plate 1: Vegetation germinating through geogrid apertures

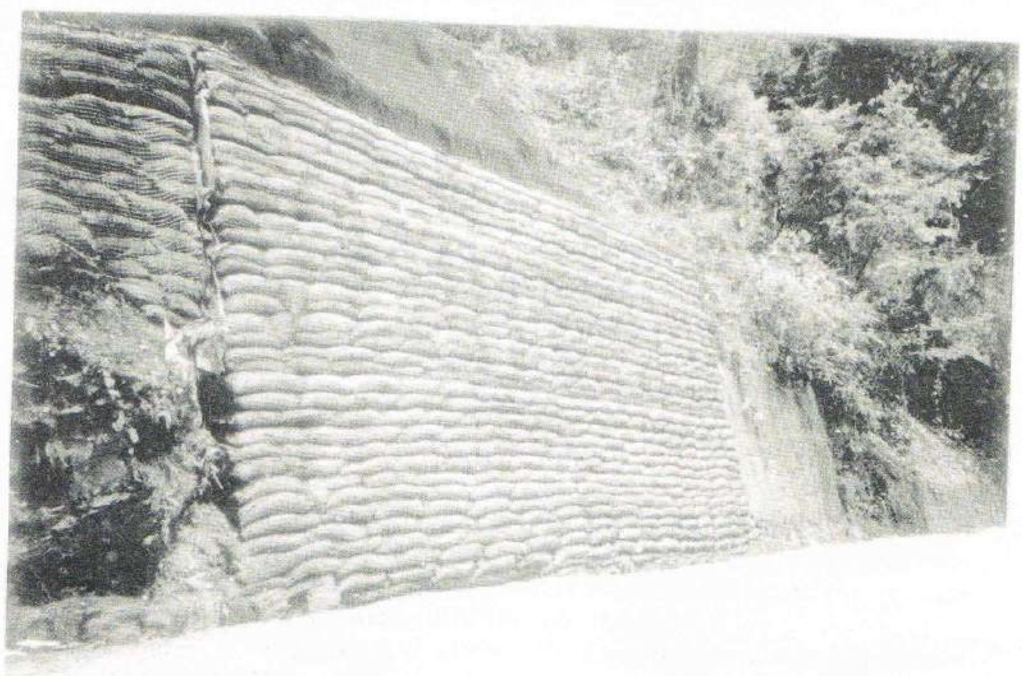

Plate 2: Protection wall at Hotel Golai towards Mirik