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Design of a Geo-Grid Reinforced Soil Wall of Height 15m Using ReSSA Software

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Abstract— As the highway construction in hilly areas progress, the 4 laning of exiting highway is proposed due increase in traffic. The side of the highway where a slope stabilisation or soil retaining is required for higher depths duet presence of vertical slopes, the various retaining structures such retaining wall and gabion wall can be constructed, but these are feasible and economical for smaller heights that is up to 7m or less. To further retain a soil various slope stabilisation techniques can be used. Geo-grid Reinforced soil wall is technique that requires use of geo-grid and coir mat to construct a soil wall like structure to provide an area and stabilized road way for propagating traffic. This paper involves study and design of a Geo-Grid Reinforced Soil wall for site in Himachal Pradesh on project of Four laning of Kiratpur to Ner-Chowk of Section of NH 21 from km 73.000 to km 186.500 in States of Punjab and Himachal Pradesh with Project length of 84.380 km and 327.000 lane km

Keywords— Slope Stabilisation, Soil Reinforcement, Design of RS wall,

I. Introduction

This research paper involves study and design of reinforced soil wall based on site conditions The various test on soil were performed to get soil properties. The design considers two slope stabilisation technique the first is Bishop Method (Bishop, 1955), and the second is Spencer Method (Spencer, 1967). Both the methods are briefly discussed later in the paper. The site conditions are analysed along with the Original Ground Level(OGL) and the Proposed Finish Road Level (FRL) of the site and soil investigation are also done. The design is done using the ReSSA software. For studies purpose only one section of the highway is taken i.e. from chainage km 18+640 to km 18+650

II. Analyses Type

There are two methods used for analysis, the first is Bishop Method (Bishop, 1955), it is valid to round slip faces. Although this method does not strictly satisfy the equilibrium conditions are not satisfied by this method but the outcomes are very near to the results obtained by the other complicated stability methods. If the strength of soil changes gradually then Bishop Method is applicable because of circular failure mechanism. It is valid for soil in which the translational failure occurs for example soft clay over granular soil.

Spencer Method is used in the second method in the ReSSA software for analysis. This method follows the conditions of equilibrium. This process assumes zero cohesion force between the soil particles. It considers the effect of strengthening material in form of layers. That is, it takes in account the direct sliding along every layer. It uses the two and three-part wedge. Which is the deciding factor the reinforcement quantity of the reinforcement in this the geo-grid.

Spencer method considers both rotational and translational failure and hence somewhat preferred more than bishop method.

III. Introduction to ReSSA Software

ReSSA allows the user to input soil strata containing up to 25 different soils, use of tension crack, varieties of surcharge loads, seismicity, and water pressure. Water pressure can be introduced via a phreatic surface or by using twenty lines each representing a different piezo metric head. Invoking water pressure enables the designer to conduct effective stress analysis or mixed type of analysis; total stress ignores pore water pressures. Mixed analysis means that in predetermined layers of soil, the shear strength of soil will be calculated based on effective stresses (i.e., using drained shear strength parameters) while in others it will use strength based on total stress (i.e., un-drained shear strength parameters).

Mixed analysis can be useful in many cases where reinforcement is used; e.g., reinforced slope comprised of granular, free-draining soil over saturated clay in which case the clayey foundation will likely exhibit an un-drained behaviour at failure while the granular backfill will practically exhibit drained strength. As a result, ReSSA is capable of assessing the required reinforcement strength and layout, including pull-out resistance, under effective or total stress conditions thus enabling the assessment of waterfront structures.

In computing the available strength along each geo-synthetic layer, ReSSA considers pull-out resistance at the reinforcement rear-end implementing user-prescribed factor of safety. However, in a sense, mechanism similar to pull-out can occur also in the front-end of each layer. In this case, the soil may slide outwards relative to the anchored reinforcement. The geo-synthetic strength feasible at its 'front-end' depends on the 'connection' strength at the face of the slope. To calculate the geo-synthetic strength at points away from the slope face, the resistance developing along the soil-reinforcement interface is added to the connection strength, not to exceed the long-term allowable strength of the reinforcement. The user needs to specify the connection strength; for reinforcement that terminates at the face of the slope it would be zero, for wrap-around with sufficiently long re-embedment it would be the strength of the reinforcement, and for attached facia (e.g., blocks or gabions) it would be the actual connection strength. ReSSA calculates the strength distribution along each layer based on the given interaction parameters, connection strength, overburden pressure and specified pull-out resistance factor of safety. In stability calculations ReSSA uses the strength value at the intersection with each analysed slip surface, be it rotational or translational (two- or three-part wedge).

For present study, only drained conditions were assumed and accordingly study parameters were input at the modelling stage.

IV. THE PRESENT SITE CONDITIONS

A. *Original Ground Level and Proposed Finished Road Level*

TABLE I
OGL OF RHS SIDE OF PAVEMENT FOR CHAINAGE 18+640

22.36	20.75	18.63	16.52	14.36	11.19	9.20	6.76	4.32	2.03	0.00
574.442	573.223	572.328	571.534	570.622	569.885	569.301	568.837	568.372	567.247	566.123

TABLE II
OGL OF LHS SIDE OF PAVEMENT FOR CHAINAGE 18+640

0.00	-0.27	-2.02	-3.59	-5.98	-8.59	-11.06	-13.54	-16.46	-19.40	-21.62	-23.84
566.123	566.024	566.045	565.950	565.428	564.976	564.191	563.475	562.598	561.770	561.299	560.727

TABLE III
FRL OF RHS & LHS SIDE OF PAVEMENT FOR CHAINAGE 18+640

10.25	8.25	6	4	1	0	-1	-4	-5	-8.25	-10.25
573.107	573.107	573.174	573.224	573.374	573.374	573.374	573.224	573.174	573.107	573.107

Graph Plotting OGL and FRL against Offsets

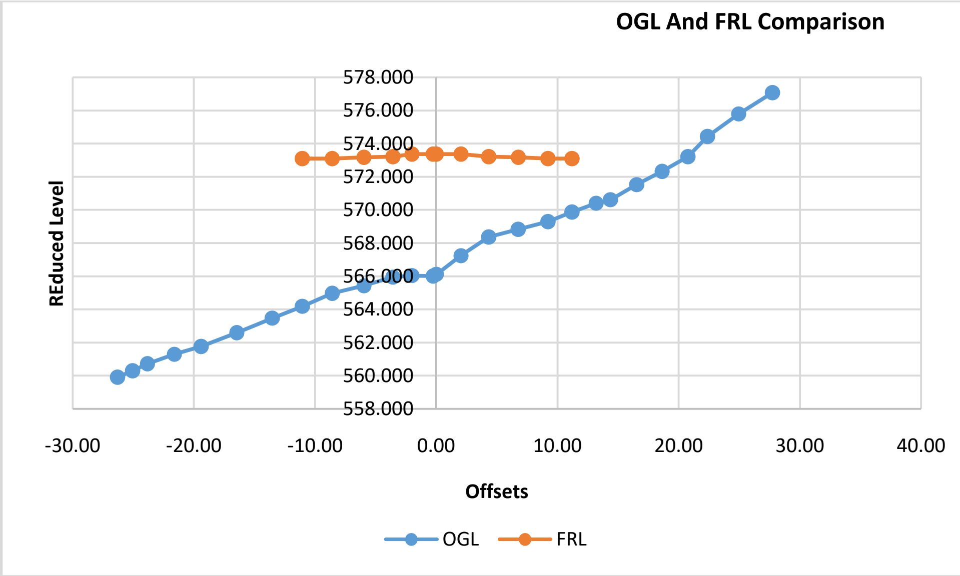

Fig 1 A sample line grah representing the OGL & FRL Cross-section

B. Soil parameters

TABLE IV

TABLE REPRESENTING ALL SOIL PARAMETERS OBTAINED FROM SOIL TESTING

Data obtained from site: **(to be confirmed)**	Soil Parameters Engineered fill for new embankments: (form BA at Ch 18+640–18+860) Design value of internal angle of friction, φ = 32° Unit weight, ϒ = 19.0 kN/m3 Cohesion, c = 0.0 kN/m2 Ultimate Bearing Capacity = 240 kN/m^3 Foundation Soil: As per GI DATA_5 Boreholes, "CONSOLIDATED GEOTECHNICAL LOG AT CHAINAGE: Km. 18+640–18+860" Load Surcharge Live load surcharge (embankment), Qv-l = 24.0 kPa Seismic Factor Seismic zone V Peak Ground Acceleration, A0 = 0.36g Kh = 0.09 Kv= +/- 0.50

V. DEFINING MATERIAL PROPERTIES & ANALYSES OF RS WALL

C. Tenax Geo-grids

TENAX TT and Flexageogrids are mass-produced by extruding and mono-directional sketch of high-density polyethylene (HDPE) grids. The design pull force Ta of the fortification is generally controlled by the opposition of the reinforcement or by the strength in the geo-grid conforming to the maximum distortions well-matched with serviceability. The allowable Resistance of a geo-grid is found as a segment of the Long Term Design Strength (*LTDS*) by means of a POF.

TABLE V
TABLE REPRESENTING TYPES OF FILLING MATERIALS TO BE USED

	Soil Layer	Unit Weight	Internal Angle of friction (degrees)	Cohesion, c , (kPa)
1	Filling Material (Soil)	19.0	32	0.00
2	Moderately Weathered Sandstone	23.5	20	120
3	Highly Weathered Sandstone	23.5	15	100

TABLE VI
TYPES OF GEOGRIS USED AS PER THEIR STRENGTH

Type	Geosynthetic Designated Name	Ultimate Strength Tult (kN/m)
1	FLEXA 5	96
2	FLEXA 7	132
3	TT 160	160

D. Properties Tenax Geogrid

TENAX TT and Flexa geogrids are also constructed by weaving and uni-directional stretching of high-density polyethylene (HDPE) grids. The design pull Ta of the reinforcement is generally governed by the opposition of the reinforcement or by the strength in the geogrid corresponding to the maximum deformations compatible with serviceability. The allowable Resistance of a geogrid is determined as a fraction of the Long Term Design Strength (*LTDS*) by means of a Partial Safety Factor:

$$T_{all} = \frac{LTDS}{fs_{total}} \quad \ldots\ldots 3$$

where: *LTDS* = *TCS* = design tensile strength (Serviceability limit state) according to Creep Strain Analysis;

$$fs_{total} = (fs_{construction} \cdot fs_{chemical} \cdot fs_{biological}) \quad \ldots\ldots 4$$

The design strength Ta is determined by applying a further global Safety Factor FSg to the allowable resistance Tall. Depending on the importance and the design life of the structure, this value ranges between 1.30 ÷1.50.

$$T_a = \frac{T_{all}}{FS_g} \quad \ldots\ldots 5$$

The LTDS is a function of the creep phenomena of the geogrids, temperature and time; it is determined after creep tests. In Table VI are given the suggested Long Term Design Strength (LTDS) at 20° C.

TABLE VII
LTDS IN KN/M FOR DIFFERENT GEOGRIDS

Geogrid Type	**Design Strength at 20 °C *TCS* (kN/m) up to 120 Years**
Flexa 2	17.28
Flexa 3	29.86
Flexa 5	43.44
Flexa 7	59.73
TT 160	75.47

$f_{s_{total}}$, shall be obtained by multiplying several Partials Factors of Safety (Koerner, 1994) to account for several possible aging factors (eq. 4). The biological and chemical Safety Factors for TENAX TT and Flexa geogrids are equal to 1.00 for all typical conditions found in natural soil; the manufacturing technology and polymer used for Tenax geogrids (HDPE) are such to prevent any aging in consequence of chemical and biological aggression. The geogrid are made with high quality polyethylene (HDPE) the most inert polymer type and therefore are chemically and biologically resistant. Tests results performed on TENAX geogrids at Geosyntec Laboratory (1991) in USA, using the E.P.A 9090 Test Method, have shown that the HDPE extruded geogrids are not damaged by any synthetic leachate at typical temperature conditions found in soil. Furthermore, the TENAX geogrids have high resistant to micro-organisms attack (aerobic and anaerobic bacterium) and macro organisms (rodent and termite).

When soil, especially crushed gravel, is spread on geogrids and is compacted, geogrids suffer damages due to local punctures, indentations, abrasions, cuttings and splitting inferred by the aggregate. Every type of geogrid suffers a different degree of damage which can be assessed by tensile tests performed on both damaged and control (undamaged) products. On this subject extensive independent test programs have been performed for evaluating the residual tensile strength of different geosynthetics after a full scale compaction damage trial. The results of these tests for geogrids and different soil types are summarized in the following Table VIII.

TABLE VIII
$F_{CONSTRUCTION}$ FOR DIFFERENT TYPES OF SOIL

Soil type	**Ø max. of the particles**		**fsconstruction**
Silt and Clay	< 0.06 mm		1.00
Pulverized fuels ashed	Variable		1.00
Fine and medium sand	0.06 - 0.6 mm		1.00
Coarse sand and fine gravel	0.6 - 6 mm		1.00
Gravel	6 - 40 mm		1.00
Ballast, sharp stones		< 75 mm	1.03
		< 125 mm	1.07

Table 6 – $f_{sconstruction}$ for different types of soil

Considering the soil that will be used a reduction factor equal to 1.00 can be used for every geogrid, except for Flexa 2 for which a reduction factor 1.10 applies.

E. Stability Analyses

For steepened reinforced slopes, there are three failure modes (Figure 4)

1. Internal, where the failure plane passes through the reinforcing elements

2. External, where the failure surface passes behind and underneath the reinforced mass

3. Compound, where the failure surface passes behind and through the reinforced soil mass

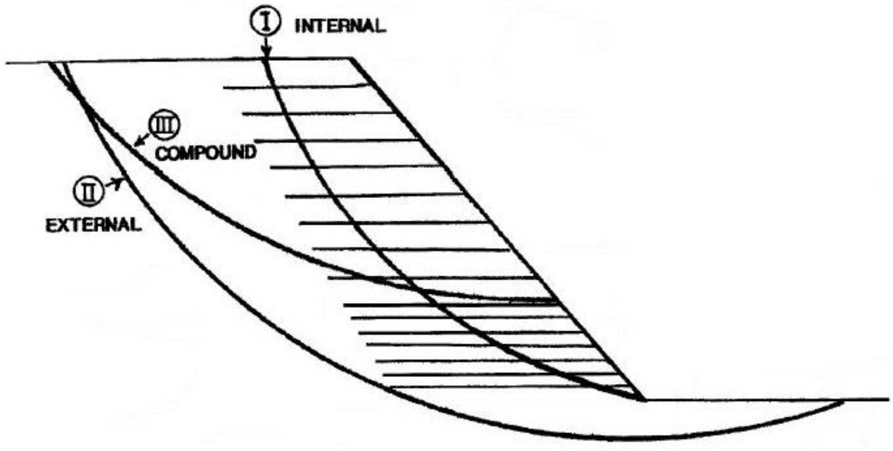

Figure 2 – Failure modes for reinforced soil slopes

Design Data

Performance requirements

External stability:

Static conditions

- sliding: FS ≥ 1.3

- Deep seated (overall Stability-Static): FS ≥ 1.3

Deep seated global stability

Evaluate potential deep-seated failures surface behind the reinforced soil zone to provide

$$F.S. = \frac{M_R}{M_o} \geq 1.3\ minimum$$

Seismic Stability: FS >= 1.1

Dynamic Stability

Perform a pseudo-static analysis using a seismic ground coefficient A, obtained from local building code and a design seismic acceleration A_m equal to A_m=A/2. Reinforced Soil slopes are clearly yielding type structures, more so than walls. As such A_m can be taken as A/2 as allowed by AASHTO in Division 1A-Seismic Design 6.4.3 Abutments (AASHTO, 2002) and Appendix A11.1.1.2 (AASHTO, 2007)

F.S. Dynamic >= 1.1

Compound failure:

- Overall compound failure: FS ≥ 1.3

Internal slope stability:

- Overall internal stability: FS ≥ 1.3

- Pullout resistance: FS ≥ 1.5

Minimum anchorage length: Le = 1.0 m

Soil Parameters

Engineered fill: Design value of internal angle of friction, φ = 32°

Unit weight, ϒ = 19.0 kN/m3

Cohesion, c = 0.0 kN/m2

Foundation Soil: as per GI DATA_5 Boreholes, "CONSOLIDATED GEOTECHNICAL LOG AT CHAINAGE:

Km. Ch 18+640–18+860"

Load Surcharge

Live load surcharge (embankment), Qv-l = 24.0 kPa

Seismic Factor

Maximum ground acceleration coefficient, A0 = 0.36g

Horizontal ground acceleration coefficient, kh = 0.5 x A0 = 0.18

Vertical ground acceleration coefficient, kv = ± 0.5 x kh = 0.09

F. Analyses performed

The embankment has been studied according to the design parameters above mentioned considering external and global stability (deep seated circles and sliding at the base and along every geogrid layer) and internal stability (failure surfaces starting from the edge point of every geogrid layer). Analyses have been performed in static and seismic condition. In seismic condition lower FS are acceptable, as shown before (for dynamic loading Fs should be > 1.00; however, a minimum F_s of 1.10 was searched even in seismic conditions).

The drawings to be followed for construction

The drawing represent the construction Reduced level and length of geogrid to be used in the construction

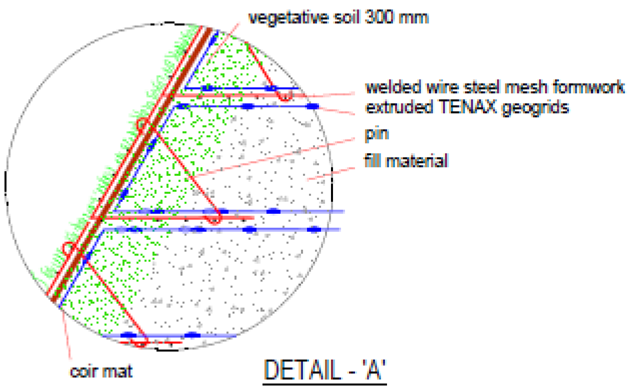

Figure 3 Represents the Facing Detail of RS wall

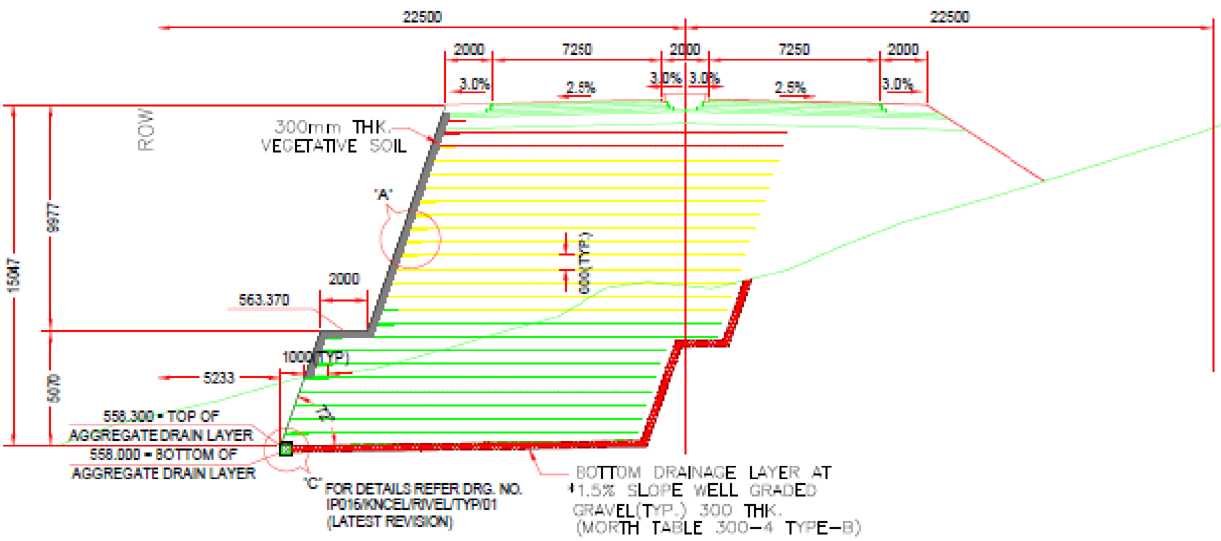

Figure 4 Represents the Cross Section of the RS wall at Chainage 18+640 to 18+650 Km

Notes:

1. All dimensions are in Millimeters, unless otherwise specified.
2. All chainages & level in meters, unless otherwise specified.
3. Codes followed is IRC SP 102:2014/ MoRTH Section 3100
4. Seismic factors to be taken as per IRC: 6: 2010 & IS 1893 {I} : 2002
5. Reinforced fill should be compacted in layers not more than 200mm thick to achieve 97% compaction of Relative density.
6. 300 mm thick soil on surface should be vegetative soil
7. Aggregates in the drain layer should be laid at slope of minimum 1.5 %, it can be increased as per site requirements
8. Appropriate longitudinal drains & Chute drains shall be proved as final drainage plan of Roadway

VI. . CONCLUSION

The design for RS wall can be more reliable and stable from software, the inclusion of geo-grid to strengthen the soil has great effects. The use and application of geo-synthetics are still under progress. The Geo-Grid Reinforced RS wall is great solution for slope stabilization instead of the conventional methods which are convenient for small heights.

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