15 Aug 2008, 11:00am - 12:30pm

Practical Lessons from Failure of a Reinforced Soil Retaining Wall on a Major Highway

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ABSTRACT

A "Reinforced Soil Retaining Wall" (RSR) wall of 10.5m height collapsed about 5 years after construction. HDPE geogrids were used as reinforcement. The facia panels suffered outward movements during construction and the deformations continued to increase. A 16m long stretch of the RSR wall failed and the failure wedge cut through four layers of reinforcement. The paper analyses the role of different factors in causing the failure. Tension tests were carried out on the geogrids exhumed from the failed zone. A significant loss in the strength of these grids was found which may be attributed to the high ambient temperatures in the area where the RSR wall is located. The influence of other factors in causing the failure are also brought out and discussed.

INTRODUCTION

Reinforced soil retaining walls (referred as RSR walls) with granular fill and high strength reinforcement are an economical alternative to gravity as well as reinforced concrete retaining walls. RSR walls were introduced in India in the mid 1990 s, C.R.R.R.I.,(1995) and have steadily gained wider usage. The RSR wall case study being presented here was designed in 1999-2000 and built in 2000-2001. The RSR wall started experiencing large significant outward deformations during and after construction. By September 2006 severe distress was noticed in some stretches of the RSR wall and the need for control measures was apparent. A 16m stretch of the RSR wall collapsed in Nov 2006. Following the distress and collapse, a detailed study and analysis of the design and construction was carried out to identify the causes for distress and failure and workout the remedial measures.

DETAILS OF RSR WALL

The maximum design height of the RSR wall is 10.5m. Typical cross section of the RSR Wall along with the location of the geogrid reinforcement at different levels is shown in Fig.1.This configuration of geogrids is as provided at the design stage and given in the design report prepared by the design consultant. The rated strength of the geogrids used in the project ranged from 45 to 160 kN/m (ultimate tensile strength).The facia elements consisted of 2.0m high, 1m wide RCC panels of 150 mm thickness. In some locations the panel height was higher, at 2.5m.Also over most of the length of the RSR wall, the height of the top most panels varied from 0.5 to 1.0m to fit with elevation of the top of the road pavement.

A starter length of geogrid of appropriate length was placed at the required location, when the panels were cast. About 350mm length of the starter stub projects out of the panel. The main reinforcement is connected to the starter by a synthetic connector and spread out at the time of construction.

A capping beam of inverted U shape, 600mm wide, 250mm in height was provided over the upper most panel and 50mm dia. pipe post was attached to the coping beam A 1.0m wide foot path of 600mm thick PCC with a steel guardrail along the inner edge of the foot path were provided for traffic safety.

CONSTRUCTION DETAILS AND INITIAL DISTRESS

As per construction guidelines, the initial batter to be provided was 1 in 30, to be adjusted to 1 in 40 after the first lift. It was also stated that the wall face would adjust to vertical after a period of time. No details were provided as to how the batter was to be maintained and checked. As construction proceeded, it was observed that the facia panels developed an uneven profile and appeared to have moved outwards. Construction was completed, reportedly, paying greater attention to precautions such as ensuring no heavy compaction equipment operates with in 1.5m of the wall edge, stretching the geogrids taut before the fill material is placed and etc.

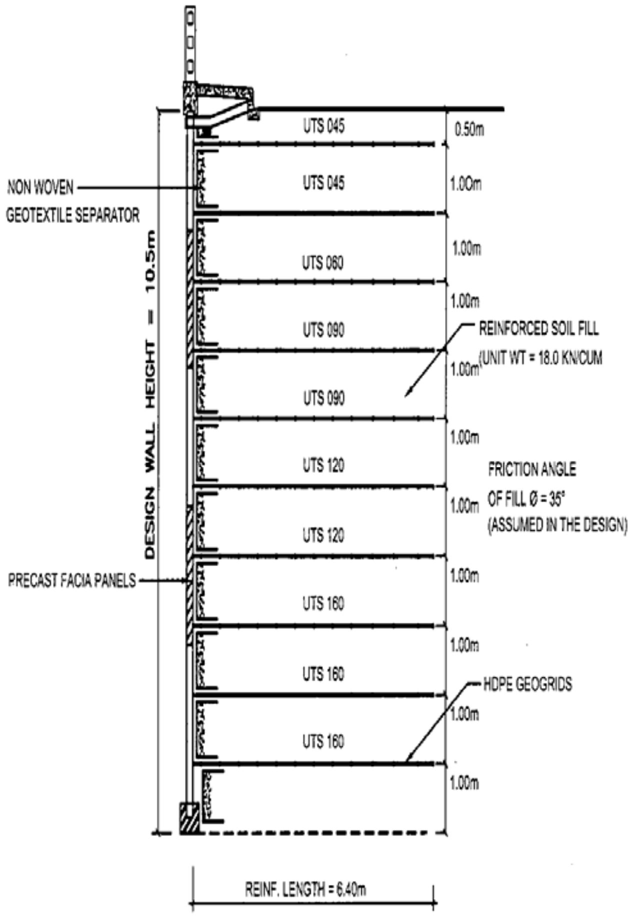

Fig.1. Cross Section of RSR Wall as per Design Report

Fig .2: Guard rail and pipe moved out of alignment due to large outward movements of Facia Panels

By mid 2001, construction work including the pavement was completed. However, the outward movements of the facia panels were found to be in the range of 200 to 250mm in the highest sections of the RSR wall. Expert opinion at that time suggested that facia movements shall be monitored regularly for one year. The data obtained could be used to project the future behaviour of the RSR wall. No such action was taken possibly because the contract did not have any provision for such activity. It was also felt that the wall movements would stabilize as the construction gets completed. A set of measurements taken in July2003 indicated a small increase in the outward movements of the facia panels and on this basis it was assumed that the movements were indeed stabilizing. However, more alarming was the tilt and rotation of the capping beam observed at many of the sections. Such movements of the capping beam were visible at many sections in 2004 and were of large magnitude where the height of the RSR wall was maximum. In retrospect it appears that the significance of the tilt and rotation of capping beam was not adequately appreciated at that time.

An attempt to prevent further movements of the capping beam was made as follows: Temporary props were set up from the ground level. Capping beam sections were pushed back, as much as possible to their original position, and then were fixed rigidly to the top most panel by brackets. However, movements of facia and capping beam continued.

In Aug-Sep 2006, cracks appeared at the junction of the pavement and the footpath. Gaps also developed between the outer edge of the foot path and facia panels. The guard rail at the inner edge of the footpath and the steel pipe post, both located on the capping beam, got out of alignment (Fig. 2). The facia movements appear to have attained alarming proportions.

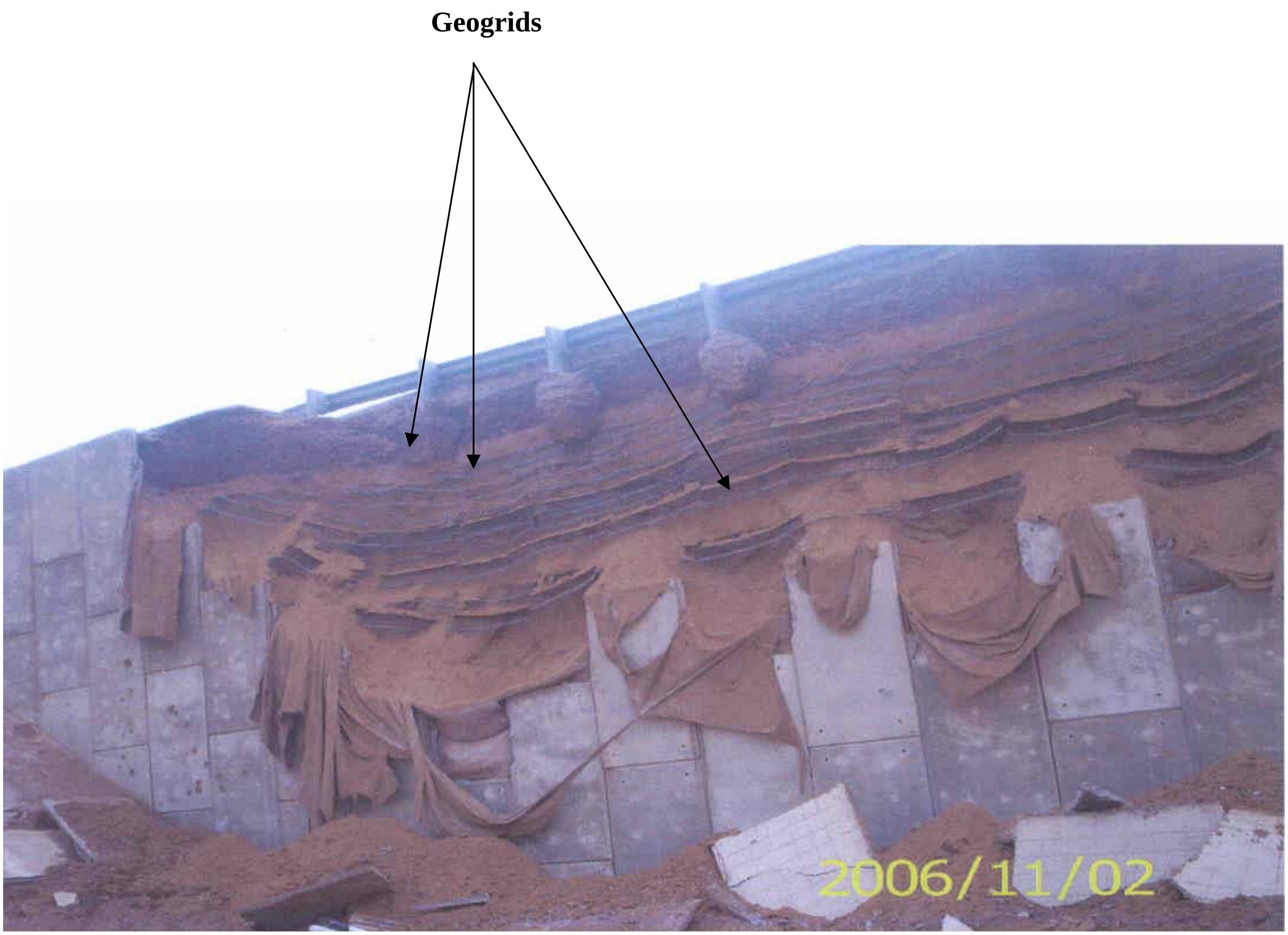

Fig.3.View of collapsed RSR wall, Exposed Geogrids are seen over the collapsed fill

The deformations and movements continued and as per information available outward deformation reached 400-450mm at some locations. A section of the RSR wall collapsed in Nov 2006(Fig.3). A wedge of fill material along with facia panels fell outwards. In Fig. 3, Geogrids can be seen hanging over the failure surface. The thickness of the wedge at top was found to be ranging between 1.6 to 2.0m, along the 16m of collapsed stretch. The failure surface conformed very closely to a circular arc. The wedge extended to about 4.5 to 5.0m down from the top.

REVIEW AND ANALYSIS OF DESIGN

As per design document, the design follows the guidelines of the geogrid manufacturer as well as B.S.8006-1995(Code of practice for reinforced soils and other fills), which is widely used in India and provides a conservative design. Although the project requirements did not specify adherence to B.S.8006-1995, an expert review at the end of 2000,i.e; before the start of construction concluded that the design generally follows the above code.

However, detailed review of the project design indicated significant deviations from the manufacturer's design guidelines as well as B.S.8006-1995.It was also found that the properties of fill materials do not satisfy the requirements of the project specifications. The impact of the deviations from the design norms and inadequacies of fill material, on the stability and deformations of the RSR wall is discussed in the following sections

Choice of Reduction Factors

The ultimate tensile strength of the geogrid is reduced by reduction factors to account for loss of strength due to creep, extrapolation of data, construction damage, environment of the fill etc. The values of each of these factors are provided by the manufacturer from carefully conducted tests. Of all the factors creep is most critical and is also temperature dependent, more so in the case of HDPE geogrids. Typical value of cumulative reduction factor for HDPE grids is about 4 to 4.5, at 20 ˚C. The allowable stress determined as above has the effect of keeping the initial and 'end of design life strains' at low acceptable levels, thereby keeping the deformations of the RSR wall at low values.

It may be mentioned that even as at present there is no widely adopted design methodology based on strain calculations. Hence, adoption of proper values of reduction factors is critical in the design of RSR wall. The manufacturer's guidelines have not provided values of the partial factors. Instead a lumped parameter of 2.12 was provided. To this, an additional reduction factor of 1.30 is applied to account for construction, biological factors and junction strength. Thus the cumulative reduction factor provided by the manufacturer works out to 2.12×1.3=2.76.The manufacturer's guidelines also specify an overtension factor of safety of 1.50. Thus, to derive the allowable or usable tensile strength from the ultimate value, the reduction factor works out to 2.76×1.50=4.14.In further discussions, the values of 2.76 and 4.14 are referred to as overall 'reduction factors'.

<u>Length of Reinforcement</u>

Further, the length of geogrid provided was 0.6H, where H is the design height of the RSR wall, as suggested in manufacturer's design guidelines and no absolute minimum length is specified. However, B.S.8006-1995 specifies 0.7H as the minimum reinforcement length with 3.0 m as the absolute minimum .Length of geogrid is critical from considerations of global stability and pullout resistance. The project RSR wall has adequate stability against global failure. Safety against pullout is discussed subsequently.

<u>Properties of Fill Materials</u>

Project specifications required that the fill material shall be predominantly coarse grained and not more than 10 percent of particles shall be finer than 75 microns. No requirement of minimum friction angle for the fill material was stipulated. Tests on fill material carried out prior to the construction showed the fines content to be in the range of 25 to 30 percent. A few samples showed as much as 70-75percent fines. At the same time test results were also given stating the angle of friction to be in the range of 34˚ to 37˚. Even though the result is incongruous, it was used as the basis of RSR wall design with angle of friction as 35˚.

Subsequent to failure, fill samples were collected and tested in independent laboratory. The results confirmed the percentage of fines to be 30, but the angle of was found to be 29˚ only, a much lower value. Considering the granular composition of the fill material and compacted densities measured in the field, the lower friction angle is considered representative of the fill rather than the higher one. Clearly such discrepancy in the design friction angle has a large adverse effect on the level of stability of the RSR wall.

INFLUENCE OF REDUCTION FACTORS AND FRICTION ANGLE ON THE DESIGN

Table 1 shows allowable tension for the geogrids for the following parameters:

(a) Combination of reduction factor of 2.12 and overtension factor of safety of 1.3,giving an overall reduction factor of 2.76, as used in the design
(b) Combination of reduction factor of 2.12, overtension factor of safety of 1.5 and global factor of safety of 1.3 as ought to have been used as per manufacturer's guidelines, giving an overall reduction factor of 4.14.

Table 1 Effect of Reduction Factors on Allowable Tensile Strength of Geogrids

Ultimate tensile strength kN/m	160	120	90	60	45
Allowable tensile strength kN/m (overall reduction factor =2.76)	57.9	43.4	32.6	21.7	16.3
Allowable tensile strength kN/m (overall reduction factor =4.14)	38.6	28.9	21.8	14.5	10.8

The design is based on friction angle =35˚, whereas test results have shown this to be 29˚ only. Lower friction values imply higher tensions in the geogrids for the same spacing. The design adopts 1m uniform grid spacing. Table 2 shows tensile force in the geogrids for friction angle of 35˚ and Table 3 shows that for 29˚, and the tensile force in the geogrids is compared with allowable values for both the overall reduction factors shown in Table 1. All comparisons are for design wall height of 10.5m.

CALCULATION OF TENSILE FORCE AND PULLOUT RESISTANCE

In the design report, tension in the geogrids was calculated using the formula:

$$T_i = T_{hi} + T_{qi} \qquad \text{---- (1)}$$

where T_i = *tensile* force in the ith geogrid layer

$$= [K_a \sigma h_i] V_i \qquad \text{---- (2)}$$

and T_{qi} = tensile force from the uniformly distributed surcharge on top of the wall at ith layer.

$$= K_a q V_i \qquad \text{---- (3)}$$

K_a = Coefficient of active earth pressure

$$= \frac{1 - Sin\phi}{1 + Sin\phi}$$

ϕ = angle of fraction of the fill material

value of $\phi = 35^0$ was used in the report, and $K_a = 0.271$

γ = unit weight of fill (18 kN/m^3)

q = surcharge (10.8 kN/m^2)

V_i = vertical spacing of geogrids

h_i = height of the fill above the i^{th} layer.

Pullout resistance was calculated using the formula

Pullout resistance of i^{th} layer,

$$P_{\gamma i} = 2C_{po} \; L_{ei} \; \sigma_{vi} \; \tan \phi \qquad \text{---- (4)}$$

$P_{\gamma i}$ = pullout resistance of i^{th} layer

$\sigma_{vi} = (\gamma h_i + q)$ = Vertical load on the ith geogrid layer.

C_{po} = soil – geogrid pull out coefficient (=0.85)

L_{ei} = length of geogrid in the resisting zone, in the i^{th} layer.

$= L_i - L_{ai}$

L_i = total length of geogrid in the i^{th} layer.

L_{ai} = length of goegrid in the active zone in the i^{th} layer.

Table 2. Comparison of Tensile Force (ø =35˚) with Allowable Tensile Strength

Depth from Top (m)	Vertical Spacing of Geogrids (m)	Ultimate Tensile Strength of Geogrid (kN/m)	Total Tensile Force (kN/m ø=35˚)	Allowable Tensile Strength for Reduction Factor =2.76 (kN/m)	Remarks	Allowable tensile strength for reduction factor =4.14 (kN/m)	Remarks
9.5	1.0	160	51.6	57.9	Considered safe as per design	38.6	Level of safety is not adequate
8.5	1.0	160	46.5	57.9	-	38.6	,,
7.5	1.0	160	41.3	57.9	-	38.6	,,
6.5	1.0	120	36.2	43.4	-	28.9	,,
5.5	1.0	120	31.1	43.4	-	28.9	,,
4.5	1.0	90	25.9	32.6	-	21.8	,,
3.5	1.0	90	20.8	32.6	-	21.8	Level of safety is adequate
2.5	1.0	60	15.7	21.7	-	14.5	-
1.5	1.0	45	10.6	16.3	-	10.8	,,
0.5	1.0	45	5.4	16.3	-	10.8	,,

Table 3. Comparison of Tensile Force (ø =29˚) with Allowable Tensile Strength

Depth from Top (m)	Vertical Spacing of Geogrids (m)	Ultimate Tensile Strength of Geogrid (kN/m)	Total Tensile Force (kN/m ø=29˚)	Allowable Tensile Strength for Reduction Factor =2.76 (kN/m)	Remarks	Allowable tensile strength for reduction factor =4.14 (kN/m)	Remarks
9.5	1.0	160	65.8	57.9	Level of safety is not adequate	38.6	Level of safety is not adequate
8.5	1.0	160	59.3	57.9	-	38.6	,,
7.5	1.0	160	52.8	57.9	-	38.6	,,
6.5	1.0	120	46.2	43.4	-	28.9	,,
5.5	1.0	120	39.7	43.4	Level of safety is adequate	28.9	,,
4.5	1.0	90	33.2	32.6	-	21.8	,,
3.5	1.0	90	26.6	32.6	-	21.8	,,
2.5	1.0	60	20.1	21.7	-	14.5	,,
1.5	1.0	45	13.5	16.3	-	10.8	,,
0.5	1.0	45	7.0	16.3	-	10.8	Only this layer has adequate safe

Table 4. Comparison of Pullout Safety for Different Friction Angles

Depth from Top (m)	Ultimate Tensile Strength of Geogrid (kN/m)	Angle of Friction of Fill ø˚	Failure Plane Length (m)	Effective Reinforcement Length (m)	Total Tensile Force (kN/m)	Pullout Resistance (kN/m)	Factor of Safety
0.5	45	35	5.21	1.18	5.41	28.03	5.11
0.5	45	29	5.90	0.49	7.01	9.29	1.33

Tables 2 and 3 clearly bring out the differences in safety level of the RSR wall, based on the values of the parameters used in the design vis-à-vis the more appropriate and realistic values. From Table 3 it is clear that only the geogrid at elevation of 0.5 m from the top has adequate level of safety in tension.

Considering pullout, normally the top most layer is critical, and the same is discussed now. Pullout is not directly influenced by the allowable tensile strength of the geogrid. The angle of friction of the fill has greater role as it influences the tensile force, effective reinforcement length and the mobilized pullout resistance. Pullout comparison for ø=35˚ and for ø=29˚ for geogrid length of 0.6H is given in Table 4 below for the top most layer of reinforcement.

It is seen from table 4 that with the angle of friction of the fill equal to 29˚, the failure plane length is longer and the effective reinforcement length is correspondingly shorter, the tensile force is higher and pullout resistance is lower, as compared to the values with friction angle of 35˚ and as a result the pullout factor of safety is 1.33, which value is lower than the minimum required value of 1.5.

From considerations of critical factors involved in the design, viz. reduction factors used in arriving at the allowable tensile strength ,factors of safety and friction angle of fill material, it is clear that the design is very much on the unsafe side. The nil or inadequate factors of safety in overtension imply that the stiffness of the RSR wall is inadequate, ab-initio. This has caused the wall to be susceptible to experience large deformations, as indeed has been observed. Full scale studies on a HDPE reinforced soil retaining wall (Chew S.H.and J.K. Mitchell 1996) showed that the wall movements are higher by 20% for reinforcement length of 0.6H, as compared to those if the reinforcement length was 0.7H.

Discrepancy in the Spacing of Geogrids

Besides the above factors which have shown a cumulative effect on the large and continuing deformations of the RSR wall, a very unusual factor was found during the investigations related to its distress and failure, and this factor had an equally large destabilizing influence.

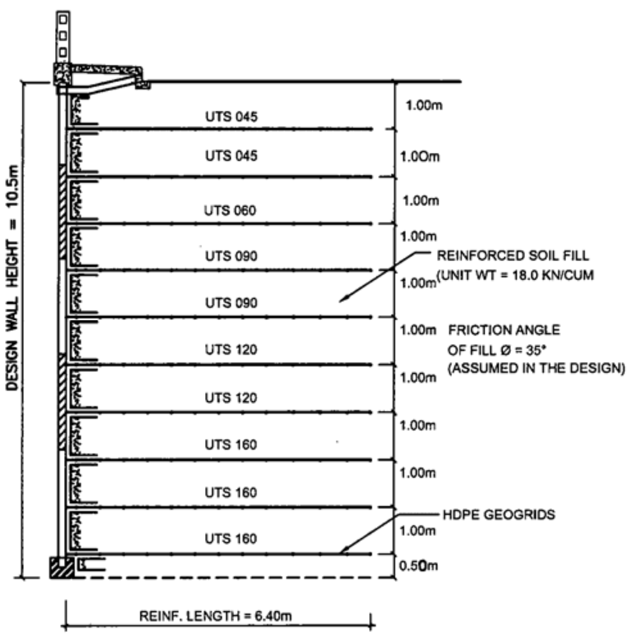

Fig.4. Cross Section of RSR wall as Built

Note:

1. Spacing of lowest and top most grid differs from that in the design.
2. Friction of angle of fill was found to be 29^0, thus lower than assumed in the design.

It was found that the spacing of geogrids as adopted in the construction was different from that given in the design report. The change was made by the designer, without however, evaluating the effects of the same. In the design report, the top most geogrid layer was provided at 0.5m below top of the pavement level with subsequent layers of reinforcement being spaced at 1.0m intervals each. The stresses in all the geogrid layers were calculated for this pattern of reinforcement layout, as shown in Tables 2, 3 and 4. On the other hand drawings provided by the designer, for use in construction, showed the lowest reinforcement layer at 0.5m above the foundation level, with other layers being spaced at 1.0m intervals from thereon. This has resulted the topmost layer being located at 1.0 to1.1m below the top of the RSR wall, instead of 0.5m below as used in the design calculations. The design guide lines of the manufacturer specify that the geogrid layer immediately above the foundation level shall be spaced at not more than 0.5m .However, in altering the vertical layout of geogrids to satisfy this requirement, no attention was apparently paid to the increased distance between the top of the RSR wall and nearest geogrid layer.

Careful check of the drawings provided for construction showed that most of the top panels, whose height is up to1.0m, did not have any geogrid attached to them. The height of these panels is between 0.5 and 1.0m, varying to keep in line with the top profile of the RSR wall.

A pit was excavated adjacent to one of the panels, where the height of the RSR wall is10.5m, and it was found that the top panel of 1.0m height did not have any geogrid attached to it and the geogrid was further 10cms below i.e; at a depth of 1.10m from the top of the RSR wall. Fig. 4 shows cross section of the RSR wall with geogrid spacing in “as built” condition.

Following the collapse and failure of the RSR wall, it was found that in this section also, the topmost geogrid layers was placed 1.10m below the top of the pavement level. Thus, the excavation carried out earlier and the exposed geogrids observed in the failed section, both confirm that the spacing of geogrids adopted in the construction differs from the one given in the design.

As a result of these of these deviations, over a large part of the RSR wall the top most panels were just inserted into the ones below and on sides. The capping beam was also resting on the top of the poorly fixed top panels. This arrangement caused the capping beam to experience large tilt as the top panels moved outwards. Fixing of the capping beam to the top panels was, hence of no use at all. In a ripple effect, the worst placed top panels caused the panels by the sides and below as well, to move out. Further, the panel shape was such that the joints between the panels were continuous from top to bottom of the RSR wall, and the width of the panels as well as the geogrid was 1.0m.No overlap was provided for the geogrids in their in their width direction. This combined configuration of panels and geogrid had the effect of considerably decreasing the stiffness of the RSR wall.

The large outward movements of the RSR wall are a cumulative effect of the factors discussed above.

Results of Tensile Strength Tests on Exhumed Geogrid Samples

As stated earlier, a part of the RSR wall collapsed in early Nov 2006.The failed wedge extends for about 16m length and 5m depth below the top. The width of the wedge ranges from about 2.0 to 1.6m at the top. Four layers of geogrids failed in tension and snapped at the junction with the facia panels. As the fill soil and the facia panels fell down, the four layers of geogrids were left hanging out of the fill. The ultimate tensile strength of these geogrids was 45,45,60 and 90 kN/m in sequence from top to bottom of the wedge.1m long samples were cut from the top two projecting geogrids and were tested for tensile strength in a reputed independent laboratory. Geogrid samples from two lower layers were of inadequate length for testing as the wedge narrowed down at these levels. The results of these tests showed that current tensile strength was less than the initial value as well as the peak failure strain was lower. The stress-strain curves at the installation stage and failure stage are shown in Fig.5. The strains in the geogrids at the critical depths of 1.1 and 2.1m for the calculated stress values (ref Table 4) are summarised in Table 5.

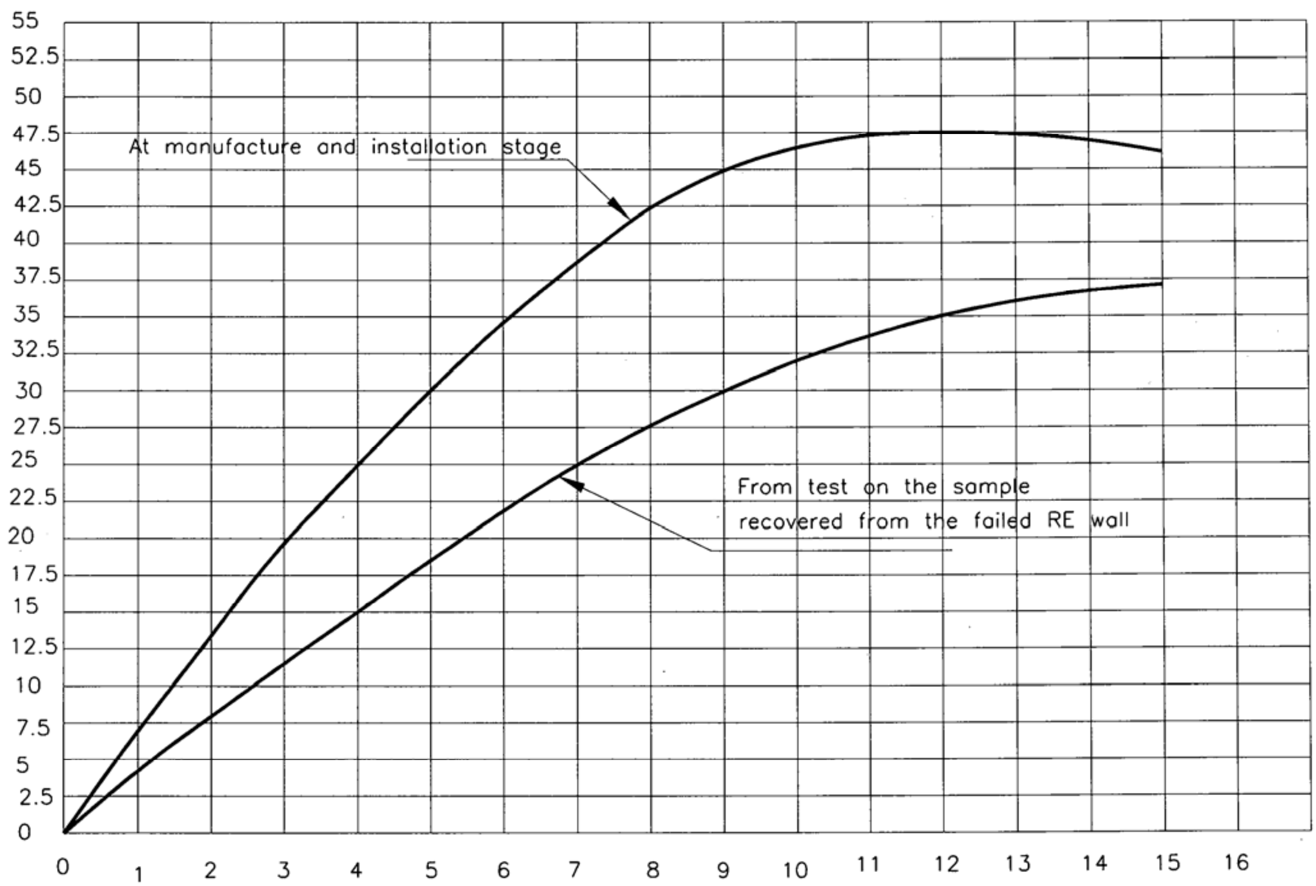

Fig.5. Stress-Strain Relationship of the Geogrid (UTS 45 kN/m) – at Installation and at Failure

Table 5. Comparison of Stress-Strain Curves for Failed Geogrids

Tensile Stress (m)	Estimated Strains (kN/m)	Estimated strains at design stage (%)	Estimated strains at failure stage (%)
1.1	10.9	1.6	2.8
2.1	17.4	2.6	4.6

Results of tensile strength tests show that the geogrids have lost about 15% of strength during the period 2001-2006 i.e. from installation to failure. Table 6 gives the comparison.

Table 6. Comparison of Stress-Strain Relationship of Geogrids as Installed and at Failure

	At Failure		As Manufactured
	Sample 1	Sample 2	
Tensile strength at 2% strain kN/m	7.69	6.0	11.0
Tensile strength at 5% strain kN/m	18.5	19.9	25.0
Ultimate tensile strength kN/m	38.0	37.2	45.0
Ultimate strain %	15.0	14.5	11.0

The test results show that tensile strength at failure is less than that at manufacture by as much as 16% and strains are higher by about 35% as compared to the values at manufacture. The loss of strength of geogrids is attributed to creep. High temperatures in the area of location of the RSR wall also appear to have increased the extent of loss strength due to creep. The failed wall has S-W exposure and wall temperatures (˚C) would be in excess of 60 for atleast 3months and in excess of 50 for another 3 months per year. The manufacturer's data for reduction factors does not provide values for temperatures in excess of 30˚C. Further the designer chose to adopt values of reduction factors applicable for 20˚C. That the creep characteristics of HDPE geogrids are highly sensitive to temperature has not been adequately factored into the design. The collapse of the RSR wall can be attributed to the rupture of geogrids which have lost part of their strength due to creep. The project is located in hot semi-arid zone. The average annual rainfall is in the range of 400 to 500mm. There was no high rainfall in the weeks preceding the rainfall. Rainy season normally ends in the middle of September.

Results of Stability Analysis

Stability analysis was carried out for circular failure wedges 5m deep and width of 1.6and 2.0m at the top. The trial wedges formed a part of circular arc of 9.2m dia. and 9.33m dia. for 2.0mdeep and 1.6 m deep ones, respectively. The analysis was carried out using Bishop's simplified method. Angle of friction for the fill material was set equal to 29^0,as determined from the post-failure tests. Fig. 6 shows the range of the width of the failure wedge observed in the filed. Fig. 6 also shows the failure surface with 2m top width, which was used in the stability analysis. The wedge was divided into 8 slices for the analysis.

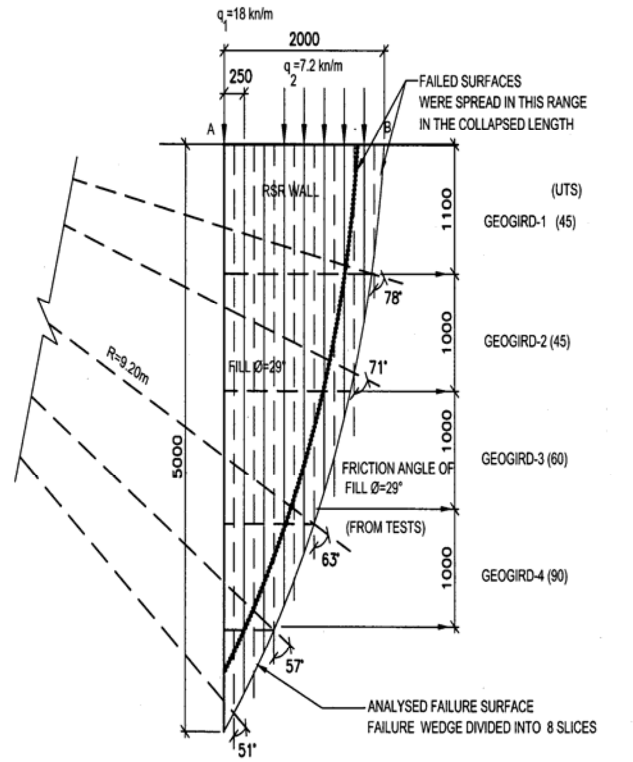

Fig.6. Section of RSR Wall showing Critical Surface Analysed

Remedial Measures to Stabilise the RSR Wall

Remedial measures to stabilize the RSR wall are presented briefly. As the highway carries high volumes of traffic, the road can not be closed even for a few days. Immediately after failure, gabion wall, with stone backfilling and woven steel mesh baskets was installed along the failed face for a length of 20m. Fig. 7 shows the failed section of the RSR wall along with the gabion wall built to restore stability. The stepped gabion wall has a base width of 8 m, with gabions of 1 m height. The need for the urgent restoration of the highway to uninterrupted traffic movement dictated the choice of gabion wall as a stabilizing measure. Although gabion wall looks unwieldy, it was installed very fast, in less than 7 days, largely with manual labour.

Fig. 8 shows the cross section of distressed stretch, where 6 m long nails were installed. The installation was done by drilling a borehole of 50 mm diameter through the facia panel and fill, and subsequently grouting the hole. The diameter of steel nails is 20 mm. The average density of steel nails was on nail per sq. m of face area. This remedial measure was carried out over a 40 m of length of RSR wall which had experienced outward movements of up to 450 mm. The RSR wall continues to be under observation. Design of steel nails and related aspects are

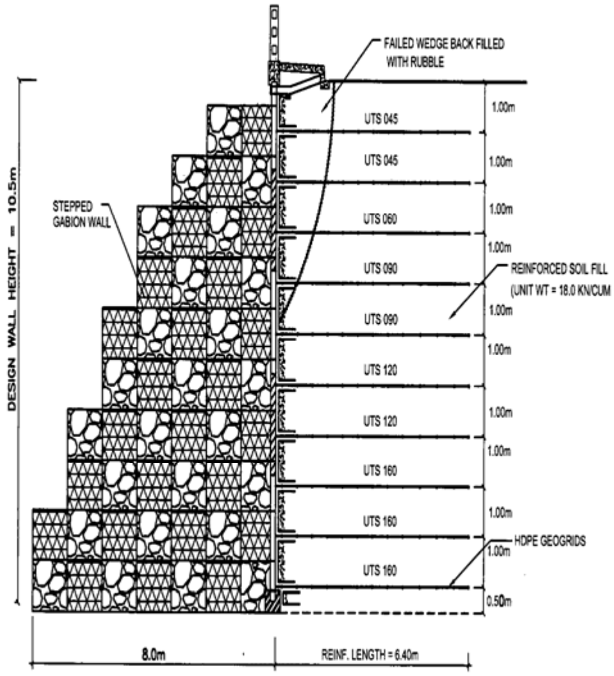

Fig. 7. Stepped Gabion Wall Built to Protect the Failed Section

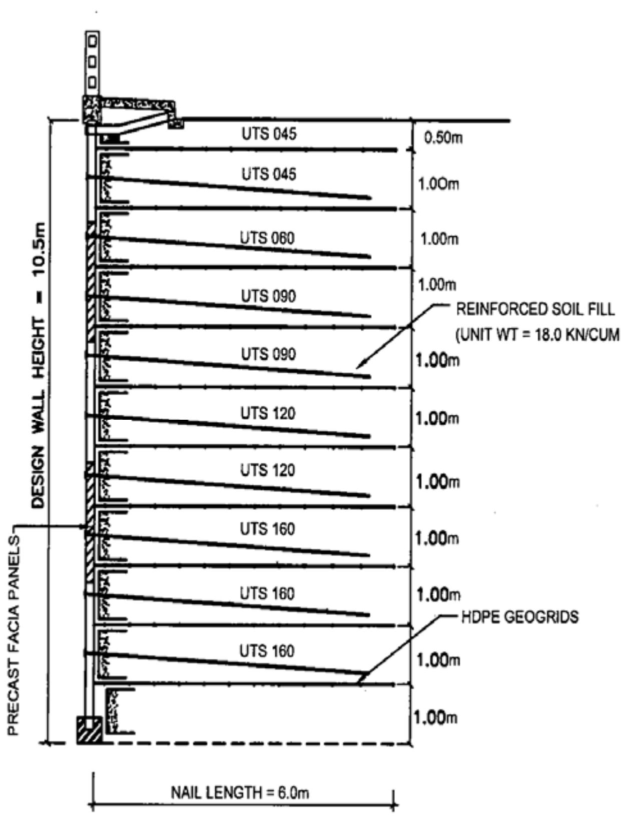

Fig.8. Cross Section of RSR Wall with Steel Nails

beyond the scope of the paper. This was completed within a few days and traffic could move unhindered. In other sections having severe distress and needing strengthening of the RSR wall, 6m long steel nails are installed, at a density of one nail per sq.m of face area.

LESSONS LEARNT FROM ANALYSIS OF FAILURE

Various deficiencies in the design that resulted in the failure of the RSR wall have been discussed in detail in the preceding sections. The lessons learnt from the analysis are summarized in the following paras:

a) The reduction factors used in arriving at the design strength values of the geogrids were low and lower than the manufacturer's recommended values. Further, the creep properties of HDPE geogrids are highly temperature sensitive and this factor was not taken into consideration in the design.

   As a result, the design stress values were higher than desirable.

b) The fill soil had fines passing 75 microns in the range of 25-30%. Still, the friction angle of 35 was considered possible at the field levels compaction of 95% standard Proctor compaction density. However, careful tests in independent laboratory showed the friction value of the fill to be 29.This discrepancy implies that the actual stress values in the geogrids are higher than the values shown in the design. Also, safety factors in pull out would be far less than estimated, importantly, for the critical top layer.

c) The design adopted does not adequately conform to the provisions of any design manual or code, but chooses bits and pieces from such documents in an arbitrary manner.

d) It appears that the large deformations observed ought have been heeded as early warning signs. The optimistic interpretation that wall movements would stabilize following completion of construction did not materialize. Each of the factors discussed in the previous sections had the effect of lowering the safety margin available.

e) It may be concluded that the failure was initiated as outward deformations were accumulating with time due to the high initial level of stress in the geogrids. The errors in the spacing geogrids which resulted in the top panels being left unsupported had also aggravated the deformations. Creep process, accentuated by the high ambient temperatures, had accelerated the progress of deformations. The error in the spacing of geogrids leaving the top panels unsupported triggered the deformation process, as well as rendered their control difficult.

It may hence be concluded that the failure is a cumulative effect of the various deficiencies and is progressive in nature.

f) It is not the usual practice to quantitatively estimate wall deformations nor is there any widely accepted method for the same. At the point of time the project was taken up it is indeed very rare to consider deformations as apart of the design.

It appears necessary that future design methods and codes establish reliable methods for working out deformations and also their progression with time.

The present case study amply bears out the need for the same.

g) When a structure under construction shows signs distressed behaviour, it would be advantageous that the structure is monitored irrespective of whether such provision is available in the contract. Such a step would help any changes to be made in the design or construction at right time, ensure the stability of the structure as well as avoid costly remedial measures at a later date.

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