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"Design of Reinforced Soil Walls By Lrfd Approach"

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ABSTRACT: *In this paper, the detail of Load and Resistance Factor Design (LRFD) approach which is based on reliability concept for the design of Reinforced Soil (RS) walls is presented. For conventional methods i.e Allowable Stress Design (ASD) Method, the factor of safety is applied only to resistance and loads are considered without variations. For LRFD method, factor of safeties are applied for both load and resistance. Due to availability of large statistical data and economy, this method is preferred. An attempt is made to solve one numerical example of geosynthetic RS walls due to soil self-weight plus permanent uniform surcharge using LRFD as well as other conventional methods (ASD) viz. FHWA, Modified Rankine, NCMA and B.S Code Methods and the results of the LRFD methods are compared with conventional design methods and concluding remarks are presented. The various equations are obtained based on various curves plotted by using ASD and LRFD approaches. From these equations it is clear that if FOS against tensile rupture is known for any RS wall having 7m height and same properties and environmental conditions as mentioned in current study then FOS against pullout failure and pullout capacity can be computed for these walls.*
Keywords: *ASD, LRFD, Modified Rankine Method, pullout failure, Reinforced Soil Wall.*

1. INTRODUCTION

1.1 Introduction: In traditionally, the Reinforced Soil (RS) walls are designed using Allowable Stress Design (ASD) approach. As RS walls being geotechnical structure, a lot of uncertainties are involved in geotechnical parameters and hence there is ample scope of an economical design of RS wall. Presently there are guidelines for Load and Resistance Factor Design (LRFD) approach which is more economical than ASD approach due to proper FS. RS wall can design for both external and internal considerations;
a) External stability checks: Sliding, Bearing capacity, overturning about the toe of the wall.
b) Internal stability checks: Tensile overstress, Pullout Resistance, Facing connection overstress

1.2 Conventional Methods / Allowable Stress Design (ASD) Method:

There are various conventional methods through which the RS walls can be analysed (Koerner et al. 2001)

- a) A Modified Rankine approach.
- b) The Federal Highway Administration approach (FHWA)
- c) The National Concrete Masonry Association approach (NCMA)
- d) British Standard Code method (BS Code Method BS 8006:1995)

Fundamental equation governing ASD is given by,

$$\boxed{R_n / FS \geq \Sigma Q_i} \qquad (1)$$

Where, R_n = Nominal Resistance, ΣQ_i = Sum of all Loads, FS = Factor of Safety.
Graphically, the ASD process can be illustrated as shown in Fig. 1 which is one of the principal limitations of ASD, wherein the values of Q and R_n are assumed to be unique such that they have a probability of occurrence of unity.

1.3 Limitations of ASD:

- Does not adequately account for variability of loads and resistances. The FS is applied only to resistance. Loads are considered to be without variations.
- Does not represent a reasonable measure of strength which is more fundamental measure of resistance than the allowable stress.
- Selection of FS is subjective and does not provide a measure of reality in terms of probability of failure.

1.4 Load and Resistance Factor Design (LRFD):

In LRFD, the resistance side is multiplied by a statistically-based resistance factor φ which value is usually less than one. As applied to the geotechnical design of RS wall, φ accounts for factors such as weaker foundation soils than expected, poor construction of the RS wall and its materials such as earth, geogrids or steel strips that may not completely satisfy the requirements in the specifications.

The load components on the right side are multiplied by their respective statistically based load factors, γi, whose values are usually greater than one. Because the load effect at a particular limit state involves a combination of different load types, Qi, each of which has different degrees of predictability, the load factors differ in magnitude for the various load types. Therefore, the load effects can be represented by a summation of γi Qi products. If the nominal resistance is given by R_n, then the safety criterion can be written as:

$$R_r = \varphi R_n \geq \Sigma \eta_i \gamma_i Q_i \quad (2)$$

Where:
φ = Statistically-based resistance factor (dimensionless), R_n = Nominal resistance,
η_i = Load modifier to account for effects of ductility, redundancy and operational importance (dimensionless),
γ_i = Statistically-based load factor (dimensionless), Q_i = Load effect.
Because of above equation involves both load factors and resistance factors, the design method is called Load and Resistance Factor Design (LRFD). For a satisfactory design, the factored nominal resistance should equal or exceed the sum of the factored load effects for a particular limit state. Load and resistance factors are chosen so that in the highly improbable event that the nominal resistance of the RS wall elements is overestimated and at the same time the loads are underestimated, there is a reasonably high probability that the actual resistance of the RS wall elements should still be large to support the loads. From Fig. 2, it implies that safety margin for ASD method is more as compared to that of LRFD method due to unfactored loads and resistance in ASD. Therefore, LRFD method is more economical as compared to ASD method.

2. AIMS AND OBJECTIVES OF THE STUDY

Review of Load and Resistance Factors Design (LRFD) approach and its results are compared with conventional design methods (ASD methods) viz. FHWA, Modified Rankine, NCMA and B.S Code Methods and conclusion are drawn.

2.1 LRFD Calibration of Pullout Limit Test: The LRFD calibration of RS wall using geogrid as a reinforcement and soil self weight plus permanent uniform surcharge as a loading condition is used in current study. Hence, its limit state function for pullout failure is given by,

$$\varphi P_C - \gamma_Q T_{max} \geq 0 \quad (3)$$

Here, P_c = Nominal calcul... R_n),
T_{max} = Nominal calculated maximum reinforcement load (Q_n),
φ = corresponding resistance factor, γ_Q = corresponding load factor applicable to internal MSEW stability,

2.2 AASHTO Modified Simplified Method for Load Models:- The maximum reinforcement load T_{max} using the AASHTO Simplified Method is computed as (For Self wt + uniform surcharge),

$$T_{max} = \lambda S_v K_r \overline{\sigma}_v + \lambda S_v K_r q$$

$$T_{max} = \lambda S_v K_r \gamma_b (Z + S) \quad (4)$$

Where, λ = Bias factor (Cu... ...ed AASHTO = 0.3 & 0.15)
S_v = Vertical spacing of the reinforcement layer,
K_r = Lateral earth pressure coefficient (1.7-1.2Ka for Steel strips and for geosynthetic = Ka),
σ_v = Normal stress due to the self-weight of backfill (γ_b Z) and equivalent height of uniform surcharge pressure ($S = q/\gamma_b$),
γ_b = Bulk unit weight of soil, z = Depth below crest of the wall, q = Uniform distributed surcharge.

Reinforcement Load Data and Bias Statistics:

The reinforcement load data for 7 m high RS walls containing surcharge load (q) varying from 10 kPa to 30 kPa and angle of internal friction (ϕ) for backfill varying from 28^0 to 36^0, is available from different case studies reported by Allen et al. (2002), Miyata and Bathurst (2007a,b) and Bathurst et al. (2008b). This data is used to compute maximum tensile load T_{max} (Calculated load) in the geogrid at each layer using Eq 4. By knowing measured load (Q), the load bias can be computed at each layer of geogrids using equation given by Bathurst et al (2008).

The constant coefficient λ is called bias factor which introduced in Equation 4. When λ = 1, the current AASHTO Simplified Method is used to compute maximum tensile load in each geogrid layer for ϕ backfill whereas, when λ = 0.3 and 0.15, the Modified AASHTO Simplified Method is used to compute maximum tensile load in each geogrid layer for ϕ and C-ϕ backfill soil cases, respectively.

2.2.1 Current AASHTO Simplified Method (λ = 1):

Fig 3 shows measured versus calculated (T_{max}) load values using the current AASHTO Simplified Method for all wall cases in the database used in this study with cohesionless soil (ϕ) backfills and none of the data points fall above the 1:1 correspondence line. In this case, the calculated load values are an order of magnitude higher than the measured value. As the mean of load bias values is μ_Q = 0.68, hence, it concludes that measured load values (Q) are 68% of the calculated load values (T_{max}).

2.2.2 Modified AASHTO Simplified Method (λ = 0.30):- The current AASHTO Simplified Model for calculation of reinforcement loads for operational (prepared) conditions is very poor for frictional (ϕ) backfill soil, because the current AASHTO simplified model over-estimates the loads by a factor of three. This deficiency can be corrected empirically by using λ = 0.30 in Eq 4 to compute T_{max}. Also, the data points fall above and below of the 1:1 correspondence line. For this case, mean bias value nearly equal to 1 and COV = 0.28.

2.2.3 Modified AASHTO Simplified Method (λ = 0.15):-

In order to extend the utility of the modified Simplified Method to (c-ϕ) soils, a complication that arises when all data points are considered is an undesirable dependency between load bias values X_Q and calculated load T_{max}. This deficiency can be corrected by dividing the load data based on calculated T_{max} into two or more groups, or filtering the data (Bathurst et.al. 2008). However, this will result in different resistance factors for different load ranges and thus complicates design. The strategy ultimately adopted in the current study to minimize load bias dependency was to remove selected bias values. After many attempts, the best filter criterion for c-ϕ soil wall cases is to remove all load bias values corresponding to calculated T_{max} < 0.5 kN/m (Bathurst et.al. 2008) as shown in Fig 5.

2.3 Modified AASHTO Simplified Method for Pullout Capacity Models:

According to AASHTO (2010) and FHWA (2009) the ultimate pullout capacity for sheet geosynthetics (geotextiles and geogrids) is estimated as,

$$\boxed{P_c = 2\,(F^*\alpha)\,\sigma_v\,L_e} \tag{5}$$

An alternative expression that used in practice is (Huang and Bathurst 2009),

$$\boxed{P_c = 2\,(\Psi \tan\phi)\,\sigma_v} \tag{6}$$

Here, L_e = anchorage len

F^* and α = dimensionless parameters,

$\Psi = \tan\phi_{sg}/\tan\phi$ = dimensionless efficiency factor ,

ϕ_{sg} = peak geosynthetic-soil interface friction angle = δ

In the FHWA document, the following default values are recommended: α = 0.8 for geogrids and α = 0.6 for geotextiles, and $F^*=2/3\tan\phi$ (Huang et.al 2009).

Pullout Test Database

The pullout resistance data for 7 m high RE walls containing surcharge load (q) varying from 15 kPa to 55 kPa and angle of internal friction (ϕ) for backfill varying from 28^0 to 40^0, is available from different case studies reported by Huang and Bathurst (2009). The tests are carried out in general conformity with ASTM D 6706 (2007).

As reported by Huang et.al (2009), there are five models used to measure pullout capacity of geogrid in RS walls which are listed in Table 1. Out of these models, Model 1 corresponds to the case where a single (average) value of $F^*\alpha$ is computed from a set of pullout tests. Model 4 uses a bi-linear approximation to the efficiency factor Ψ. As demonstrated by Huang and Bathurst, both models have strong bias dependencies with normal stress and therefore they are omitted from the current study. Therefore, model 2, model 3 and model 5 are used in current study.

2.3.1 Model – 2: First-order approximation to measured F*α

In this approach, back-calculated values of $F^*\alpha$ using Eq 5 are determined from a set of tests performed on the same soil-geogrid combination at different normal stresses. A first-order (linear) approximation is then fitted to the data. Fig 6 shows that measured (P_m) versus predicted (P_c) resistance values plot tightly around the 1:1 correspondence line. The quantitative accuracy of the model is confirmed by the bias statistics which have a mean and COV value of 1.03 and 0.13 respectively.

2.3.2 Model – 3: FHWA method with default values $F^*\alpha$ =0.8x (2/3) tan ϕ_s

Model 3 corresponds to the current FHWA (2009) geogrid pullout model. However, unlike Model 2, soil-geogrid pullout tests are not carried out. Rather, the default value α = 0.8 is used and F* is computed using ϕ of the soil. Fig 7 shows, measured versus predicted pullout resistance values. Most of the data fall above the 1:1 correspondence line and the bias mean is μ_R = 1.20. Hence, Model 3 under-estimates the pullout capacity.

2.3.3 Model – 5: Non-linear model

The general form of the n ng and Bathurst (2009),

$$\boxed{P_{corr} = \beta\,(P_c)^{1+k} = \beta\,(2\,\sigma_v\,L_e\,F^*\,\alpha)^{1+k}} \tag{7}$$

Here, dimension-dependent terms β and (1+k) are equal to 5.51 and 0.629 when pullout capacity is computed in units of kN/m (Bathurst 2009). Implementation of Model 5 is a two-step process. First calculate the pullout capacity (P_c) using Eq 5 with the default value for F* and α = 0.8. Then, compute the corrected value (P_{corr}) using the power function expression in Eq 7. Thus for model 5, the mean is 1.12 and COV is 0.50.

3. RESULTS AND DISCUSSION

From the above analytical investigation, the results of mean and Coefficient of Variation (COV) for different Load and pullout capacity models are tabulated as shown in Table: 2

To incorporate the effect of Load and Resistance Factors in design of RE wall, the following numeric example is solved using LRFD approach which is already solved by Koerner et.al (2001) using different ASD methods.

Consider a RS wall as shown in Fig.8 having following properties:

- Height of wall (H) =7m,
- Length of wall (L) = 5m, Surcharge (q) = 15kPa,
- Reinforced soil properties: $\phi_r = 32^0$, $\gamma_r = 18$ kN/m^3
- Backfill properties: $\phi_b = 30^0$, $\gamma_b = 17$ kN/m^3
- Foundation soil properties: $\phi_f = 30^0$, $\gamma_f = 17$ kN/m^3

3.1 External Stability Considerations:

3.1.1 The FOS Consideration: Refer Table: 6

In the Modified Rankine's approach, the frictional force is computed by taking into account only the weight of reinforced soil mass i.e. it neglects surcharge effect for conservative side. Hence, it has less frictional resistance thus FOS is less (2.07) for this method. In BS Code approach, the frictional coefficient is taken approximately equal to 1/3 to 2/3 of tanϕ where, ϕ is angle of internal friction hence, more FOS (2.15) as compared to Modified Rankine's approach. In LRFD approach, the resistance is reduced whereas the load effect is increase as explained earlier. Therefore, it has least factor of safety than other ASD approaches (1.70). The coefficient of friction in NCMA approach depends upon types of the soil which controls the sliding (reinforced, drainage and foundation) as given by Koerner et.al. 2001. Hence frictional resistance of NCMA approach is more as compared to FHWA and Modified Rankines approaches, hence more FOS (2.87).

3.1.2 Eccentricity Consideration: Refer Table: 7

In Modified Rankine approach, overturning moment can be computed by adding moments due to earth pressure and surcharge loading for safer side. Also, total vertical load (ΣW) is the sum of weight of soil mass and surcharge loading therefore eccentricity is maximum and also equal to BS Code approach because it is attributed to the ratio of difference between resisting moment and overturning moment to total vertical load. Thus, the eccentricity is given by, e = (B/2) - $\bar{x}$, hence e is more (0.64m) as compared to Modified Rankine approach. In LRFD approach, the location of resultant is at middle half of the base. Hence, $\bar{x}$ gets decrease and eccentricity increase (0.70m).

3.2 Internal Stability Considerations:

3.2.1 Tensile Failure: In Modified Rankine approach, the vertical stress (6_v) is due to self weight of reinforced soil and surcharge effect, hence it is more. As the design strength (T_{des}) is a function of vertical stress, it is also more and thus FOS $= \frac{\text{Tult}}{\text{Tdes}}$ is less. In BS Code approach, the maximum vertical stress (6_{vmax}) is given by sum of direct and bending stress which is less and hence FOS is more as compared to Modified Rankine's approach. In LRFD approach, the empirical adjustments are made by using bias factor λ to the tensile load models to match measured reinforcement loads in RE walls under operational conditions. Therefore, in case of LRFD approach FOS is most as compared to other ASD approaches as shown in Fig 1.

Table 8 shows factor of safeties against Tensile Failure for different depth of the RE wall.

The active earth pressure distribution on RE wall is triangular in nature having zero pressure at top and linearly increases to maximum at bottom. Therefore, the vertical spacing of geogrids is minimum at bottom and gets increases from bottom to top. As the tensile force is a function of vertical spacing of geogrid layers, it is maximum at top gets decreases with depth of the wall. Therefore, the FOS is also more at the top of the wall and gets decrease continuously with depth of the wall. The trends of FOS for all five methods are approximately same whereas trend of Modified Rankine, FHWA and NCMA approach matches with each other as shown in Fig 9. The FOS for LRFD approach is more as compared to other approaches may be because in LRFD approach, the maximum tensile force gets decrease due to bias factor λ for Modified Simplified AASHTO method for Load model.

3.2.2 Pullout Failure

In Modified Rankine approach, the pullout capacity (P_c) can be computed by assuming interaction coefficient and coverage ratio, due to this pullout capacity gets decrease and hence FOS also gets decrease. In BS Code approach, to compute Pc, the average stress at resistive zone is assumed instead of maximum stress and hence FOS is more as compared to Modified Rankine approach. In LRFD approach, to compute pullout capacity, five deterministic models are used. The resistance factor $\varphi = 0.58$ is taken to compute pullout capacity in the current study from model 2 which requires actual laboratory pullout tests. Hence, for LRFD approach, FOS may be least for all layers of RS walls as compared to ASD approaches.

The Rankine's failure plane inclined by making an angle of (45+ϕ/2) with horizontal hence effective length is lesser at top and gets increase from top to bottom. As pullout capacity is the function of effective length, pullout capacity as well as FOS is less at top and more at bottom as shown in Fig 10.
Table 9 shows factor of safeties against Pullout Failure for different depth of the RS wall.
Now the graph is plot between FOS against pullout failure on normal scale versus FOS against tensile rupture on semi-log scale for all five approaches together as shown in Fig 11.
From Fig 11, it is observed that the trend of Modified Rankine, FHWA and NCMA approaches are approximately parallel to each other. On the other hand, the trend of BS Code and LRFD approaches are approximately parallel to each other. The equations of the trend lines for various approaches and their R^2 values are tabulated in Table 10.
From the equations stated in Table 9, it is observed that if FS against tensile rupture is known for any RS wall having 7m height and same properties and environmental conditions as mentioned in current study then FS against pullout failure can be computed for these walls.
In the Table 9, the equations are used only for that RE walls which has same dimensions, same material properties and same environmental conditions as that of RE wall used in present study. Hence, these equations are not universal equations but can be converted into universal equations by further work.
Now for critical FS against tensile rupture (FS_{TR}) =1.5, the FS against pullout failure can be computed corresponding to critical FS for all approaches using equations which is tabulated in Table 10.

4. FIGURES AND TABLES

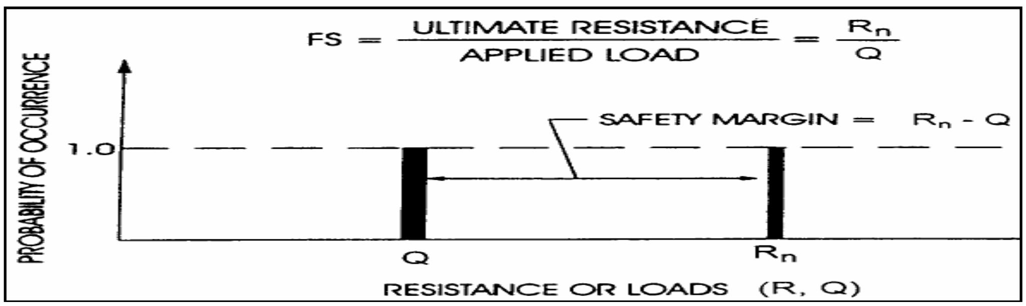

Fig 1: ASD Design Approach (FHWA 2001)

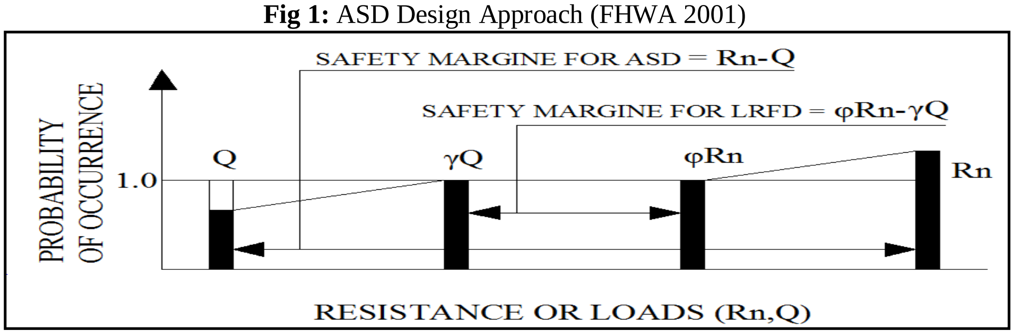

Fig 2: Combination of ASD and LRFD Approach

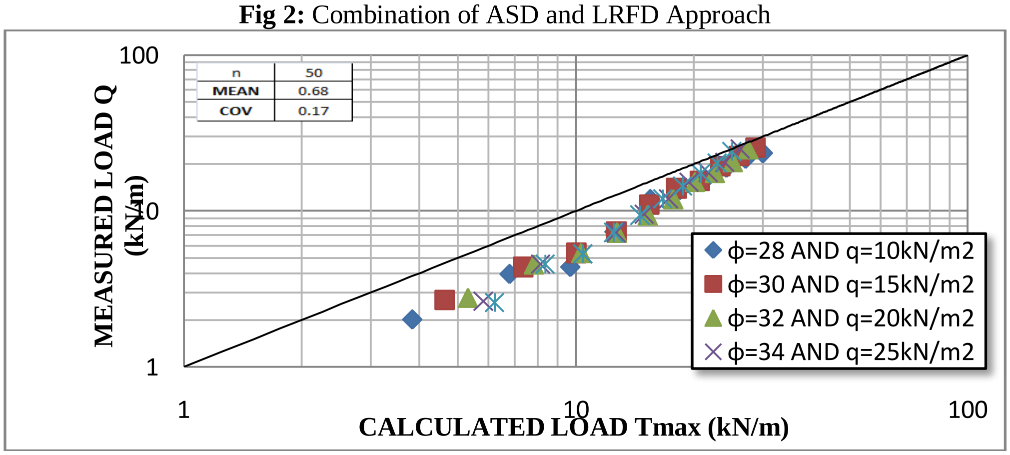

Fig 3: Measured vs Calculated Load values for λ = 1.0

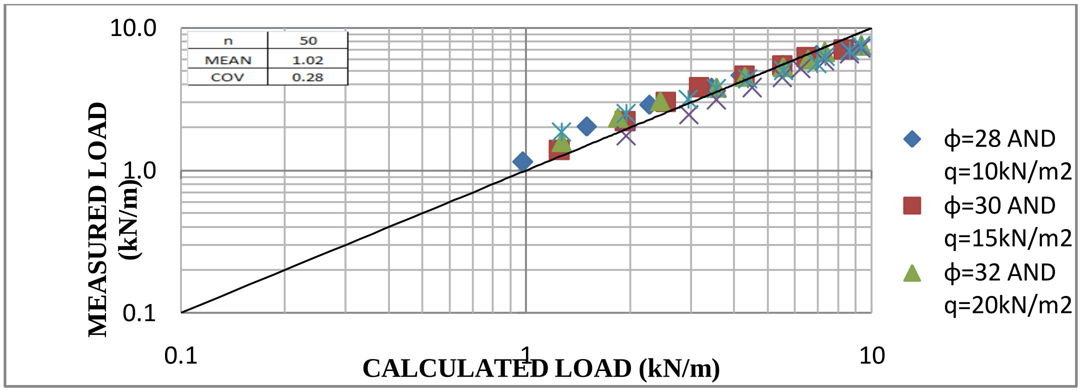

Fig 4: Measured vs Calculated Load values for λ = 0.30

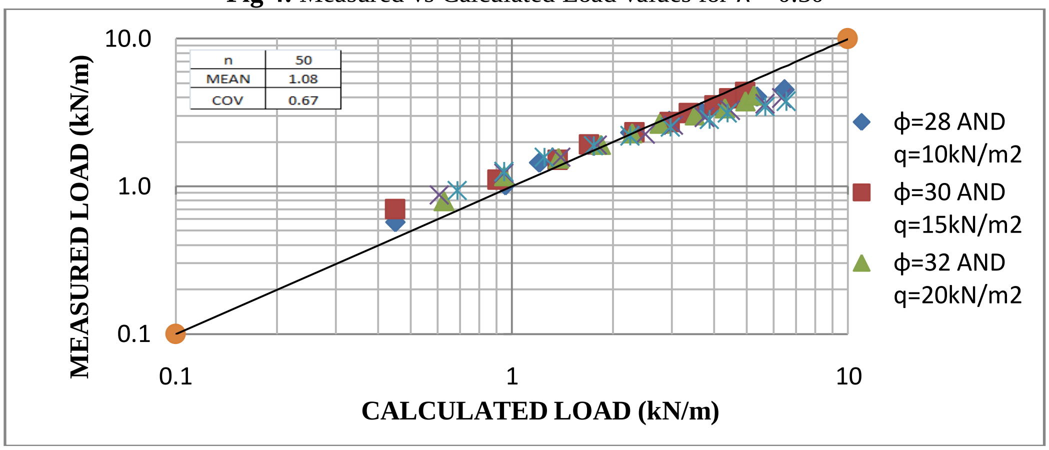

Fig 5: Measured vs Calculated Load values for λ = 0.15

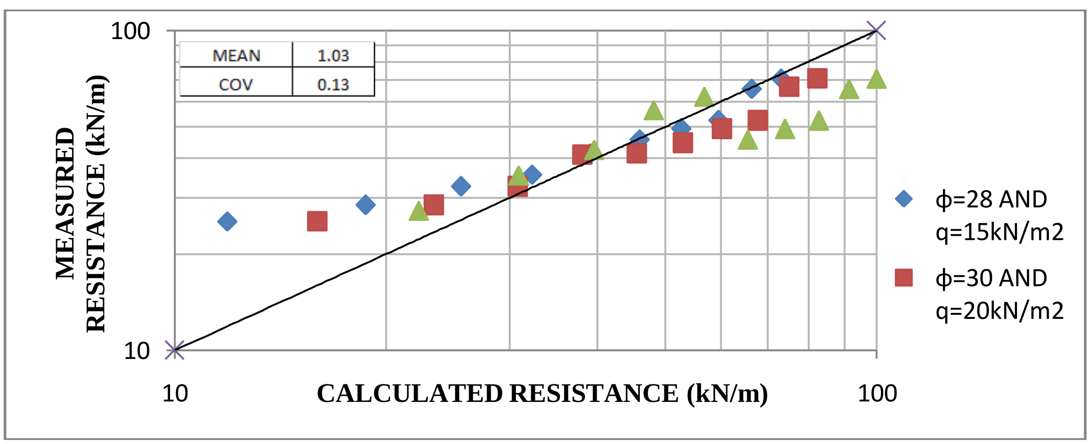

Fig 6: Measured vs Calculated Pullout Resistance values for Model 2.

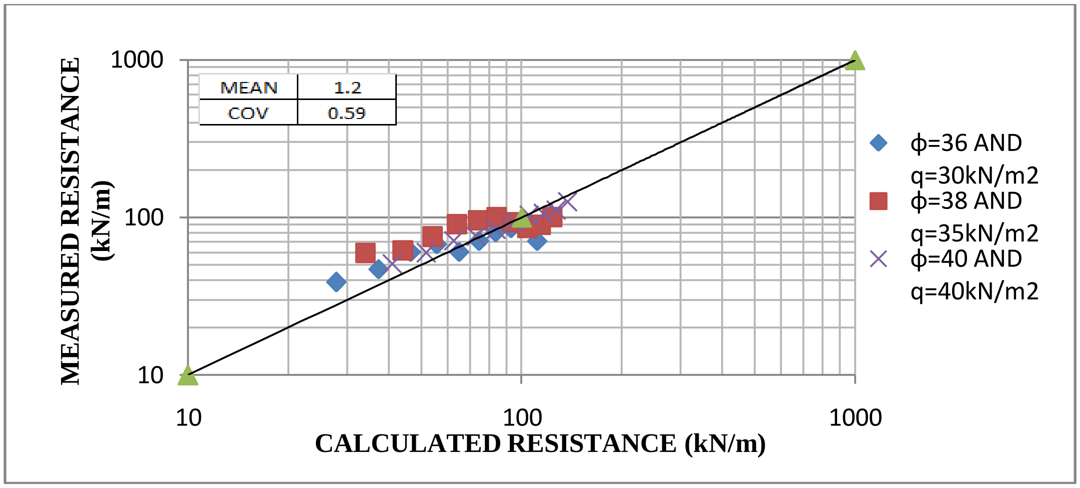

Fig 7: Measured vs Calculated Pullout Resistance values for Model 3.

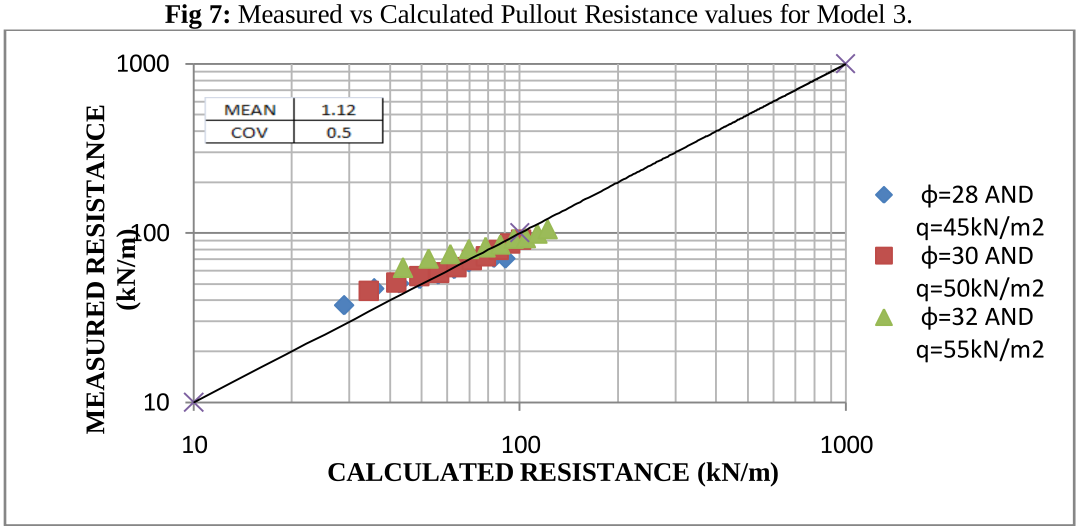

Fig 8: Measured vs Calculated Pullout Resistance values for Model 5

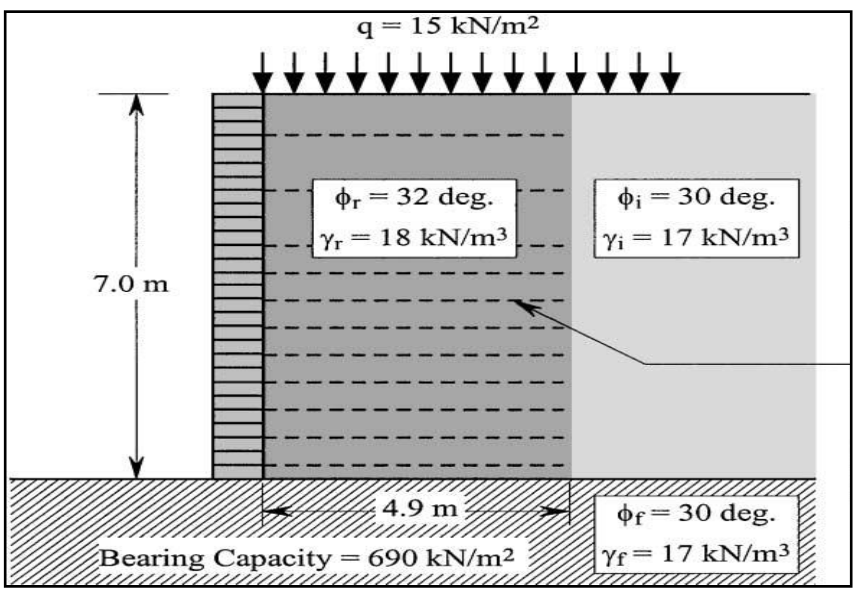

Fig. 8: Typical RE wall having modular facing block.

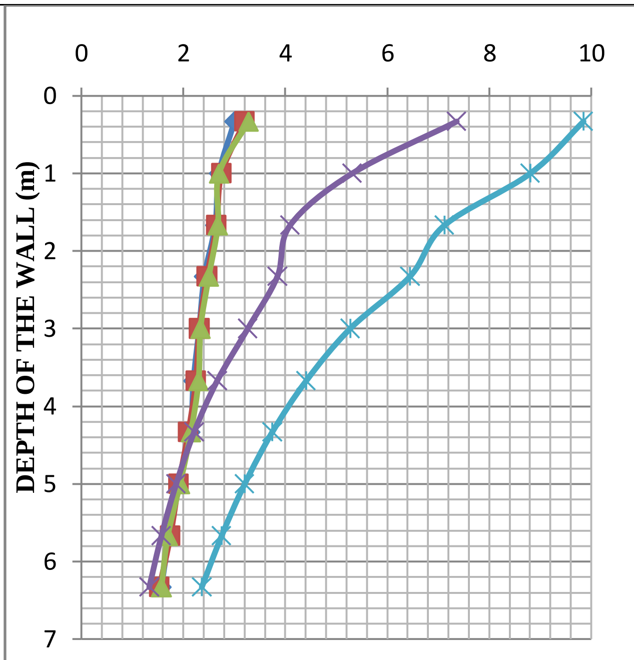

Fig 9: Factor of Safety against Tensile Rupture

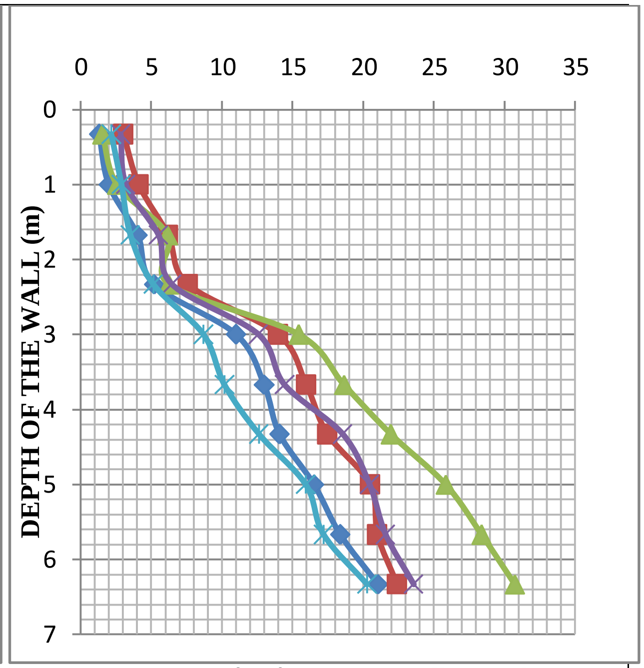

Fig 10: Factor of Safety against Pullout Failure

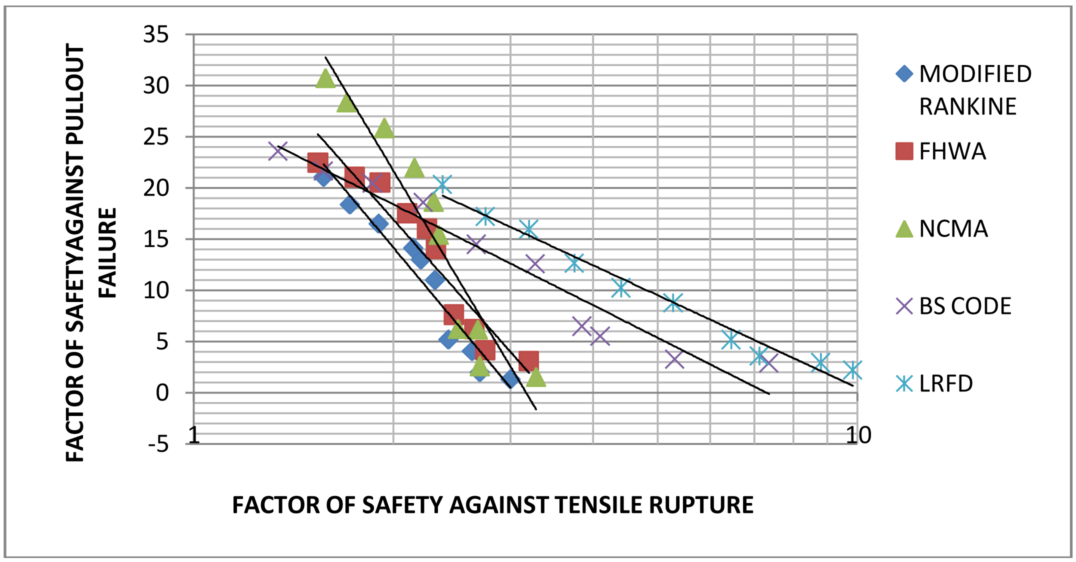

Fig 11: Factor of Safety against Pullout Failure vs Factor of Safety against Tensile Rupture for five Design Approaches of RE Wall.

Table 1 Pullout Models, their description and their use in current study.

Pullout Model	Description	Use in Current Study
Model- 1	Average measured F*α	No
Model- 2	First-order approximation to measured F*α	Yes
Model- 3	FHWA method with default values F*α =0.8x (2/3) tan ϕ_s	Yes
Model- 4	Bi-linear model	No
Model- 5	Non-linear model	Yes

Table 2: Summary of Load bias statistics (X_Q) for T_{max} using current and Modified AASHTO Simplified method.

Parameter	**SOIL TYPE**		
	Frictional (ϕ- Soil)		(c-ϕ Soil)
	Current Model λ = 1.0	Modified Model λ = 0.30	Modified Model λ = 0.15
n (Number of data points)	50	50	50
μ_Q (mean)	0.68	1.02	1.08
COV_Q (Coefficient of variation)	0.17	0.28	0.67

Table 3: Bias Statistics for different pullout capacity model types

Model	**Description**	**Bias Statistics**	
		Mean μ_R	COV_R
2	First-order approximation to measured F*α	1.03	0.13
3	FHWA method with default values (F*α = 0.8 x (2/3) tan ϕ)	1.20	0.59
5	Non- linear model	1.12	0.50

Table 4: Computed resistance factor φ for P_f = 0.01 (β = 2.33) and selected load factors γ_Q

PULLOUT MODELS		LOAD FACTORS γ_Q	RESISTANCE FACTOR (φ)		
			CURRENT LOAD MODEL (λ=1) ϕ-SOIL	MODIFIED AASHTO LOAD MODEL	
				(λ=0.30) ϕ - SOIL	(λ=0.15) C- ϕ - SOIL
MODEL-2	MEAN μ_R= 1.03	1	1.18	0.49	0.28
		1.35	**1.6**	**0.58**	**0.38**
	COV_R = 0.13	1.75	2.07	1.03	0.5
		2	2.37	1.18	0.57
MODEL-3	MEAN μ_R= 1.20	1	1.29	0.43	0.34
		1.35	**1.73**	**0.56**	**0.46**
	COV_R = 0.59	1.75	2.28	0.75	0.57
		2	2.59	0.88	0.63
MODEL-5	MEAN μ_R= 1.12	1	0.87	0.33	0.46
		1.35	**1.19**	**0.43**	**0.62**
	COV_R = 0.50	1.75	1.55	0.58	0.8
		2	1.76	0.66	0.91

Table 5: Summary of recommended resistance factor values for β = 2.33 and γ_Q = 1.35 using current and modified AASHTO simplified Method.

Resistance (Pullout) Model	**Resistance Factor φ**		
	Load Models		
	Current AASHTO	Modified AASHTO	
	λ = 1	λ = 0.30	λ = 0.15
	ϕ - SOIL	ϕ - SOIL	c-ϕ SOIL
Model- 2	1.00*	**0.58**	0.38
Model- 3	1.00*	0.56	0.46
Model- 5	1.00*	0.43	0.62

Notes: * Calculated φ values are greater than one but φ for design should be capped at one.

Table 6: Comparison of FS for External Stability Consideration

SR. NO	EXTERNAL STABILITY CONSIDERATION					
	STABILITY CONSIDERATION (FOS)	MODIFIED RANKINE	FHWA*	NCMA*	B.S CODE	LRFD
01	FS against foundation sliding (≥ 1.5)	2.07	2.11	2.87	2.15	1.70
02	FS against bearing capacity (≥ 2.0)	3.59	3.66	5.53	3.17	2.25
03	FS against overturning (≥ 2.0)	3.63	N.A	4.93	5.52	3.57

Note: * Indicates the ASD methods which consider the sloping backfill hence they are not compared with other methods.

Table 7: Comparison of FS for External Stability Consideration (Eccentricity Consideration)

SR NO	STABILITY CONSIDERATION	MODIFIED RANKINE	FHWA*	NCMA*	B.S CODE	LRFD
01	Eccentricity (m) (≤ B/6 = 0.83m)	0.64	0.63	0.42	0.64	0.70**

Note: ** Indicates location of resultant at middle half of the base i.e. e ≤ B/4 = 1.25m

Table 8: Comparison of FS for Tensile Rupture Consideration (FS >1.5)

Z (m)	FS AGAINST TENSILE RUPTURE				
	MODIFIED RANKINE	FHWA*	NCMA*	B.S CODE	LRFD
0.33	3	3.2	3.28	7.36	9.84
1	2.7	2.75	2.7	5.31	8.80
1.67	2.63	2.65	2.68	4.10	7.12
2.33	2.42	2.47	2.5	3.85	6.45
3	2.31	2.32	2.34	3.27	5.28
3.67	2.2	2.25	2.3	2.67	4.41
4.33	2.14	2.1	2.15	2.22	3.75
5	1.9	1.91	1.94	1.86	3.2
5.67	1.72	1.75	1.7	1.67	2.75
6.33	1.57	1.55	1.58	1.54	2.37

Note: * Indicates the ASD methods which consider the sloping backfill hence they are not compared with other methods.

Table 9: Comparison of FS for Pullout Failure Consideration (FS >1.5)

Z (m)	FS AGAINST PULLOUT FAILURE				
	MODIFIED RANKINE	FHWA*	NCMA*	B.S CODE	LRFD
0.3	1.3	3.02	1.54	2.85	2.19
1	2	4.1	2.58	3.22	2.89
1.6	4.03	6.18	6.17	5.5	3.54
2.3	5.18	7.58	6.25	6.45	5.14
3	11.01	13.98	15.42	12.56	8.71

3.6	13.01	15.98	18.64	14.46	10.20
4.3	14.08	17.48	21.97	18.54	12.61
5	16.5	20.5	25.83	20.45	15.95
5.6	18.37	21.02	28.37	21.6	17.18
6.3	21.04	22.42	30.74	23.57	20.30

Note: * Indicates the ASD methods which consider the sloping backfill hence they are not compared with other methods.

Table 10: Equations of the trend lines and their R^2 values for various approaches.

DESIGN APPROACH	EQUATION OF TREND LINES	R^2 VALUE
MODIFIED RANKINE	$FS_{Pu} = -33.6 \ln(FS_{TR}) + 37.47$	0.940
FHWA	$FS_{Pu} = -31.9 \ln(FS_{TR}) + 39.02$	0.906
NCMA	$FS_{Pu} = -47.0 \ln(FS_{TR}) + 54.26$	0.906
BS CODE	$FS_{Pu} = -14.1 \ln(FS_{TR}) + 28.16$	0.947
LRFD	$FS_{Pu} = -13.0 \ln(FS_{TR}) + 30.46$	0.977

5. CONCLUDING REMARKS

From Table 10, it is observed that for Critical FS against Tensile Rupture, the corresponding FS against Pullout failure for NCMA approach is the highest (35.20) hence, it has more anchorage length whereas for BS Code approach is the lowest (22.44) hence, it has less anchorage length. The FS against Pullout failure varying from 35.20 to 22.44.

1. This study includes the results of rigorous LRFD calibration for the geogrid pullout limit state in geosynthetic RS walls due to soil self-weight plus permanent uniform surcharge.
2. The modifications to the current AASHTO Simplified Method are proposed and new default pullout models are used.
3. Depending on the reinforced soil type (frictional & cohesive- frictional) and the pullout model adopted, the resistance factor (φ) varies in the range of 0.38 to 0.62. While these values are lower than φ = 0.90 recommended by AASHTO.
4. An important practical benefit of using Model 2 with actual laboratory pullout data over the default Model 3 and non-linear Model 5 is that the Model 2 allows a higher resistance factor (φ) to be used for design; the result is shorter reinforcement lengths and hence more cost-effective wall design outcomes.
5. By using ASD and LRFD approaches, the various equations are obtained (Table 9) based on various curves plotted (Fig 11).
6. From these equations it is clear that if FOS against tensile rupture is known then FOS against pullout failure can be computed and hence F*α. Therefore, no need to perform the pullout tests for particular height.

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