IRC:SP:102-2014

GUIDELINES FOR DESIGN AND CONSTRUCTION OF REINFORCED SOIL WALLS

INDIAN ROADS CONGRESS
2014

GUIDELINES
FOR
DESIGN AND CONSTRUCTION OF
REINFORCED SOIL WALLS

Published by:

INDIAN ROADS CONGRESS

Kama Koti Marg,
Sector-6, R.K. Puram,
New Delhi-110 022

August, 2014

Price : ₹ 800/-

(Plus Packing & Postage)

IRC:SP:102-2014

First Published : August, 2014

(All Rights Reserved. No part of this publication shall be reproduced, translated or transmitted in any form or by any means without the permission of the Indian Roads Congress)

Printed by India Offset Press, Delhi

1000 Copies

Contents

Digitized by the Internet Archive
in 2014

https://archive.org/details/govlawircy2014sp102

PERSONNEL OF THE BRIDGES SPECIFICATIONS AND STANDARDS COMMITTEE

(As on 6th January, 2014)

1.	Kandasamy, C. (Convenor)	Director General (RD) & Spl. Secy. to Govt. of India, Ministry of Road Transport and Highways, Transport Bhavan, New Delhi
2.	Patankar, V.L. (Co-Convenor)	Addl. Director General, Ministry of Road Transport and Highways Transport Bhavan, New Delhi
3.	Pathak, A.P. (Member-Secretary)	Chief Engineer (B) S&R, (Ministry of Road Transport & Highways, Transport Bhavan, New Delhi

Members

4.	Agrawal, K.N.	DG(W), CPWD (Retd.) Ghaziabad
5.	Alimchandani, C.R.	Chairman & Managing Director, STUP Consultants (P) Ltd., Mumbai
6.	Arora, H.C.	Chief Engineer (Retd.) MORTH, New Delhi
7.	Bagish, Dr. B.P.	C-2/2013, Vasant Kunj, Opp. D.P.S. New Delhi
8.	Bandyopadhyay, Dr. N.	Director, STUP Consultants (P) Ltd. New Delhi
9.	Bandyopadhyay, Dr. T.K.	Joint Director General (Retd.) INSDAG, Kolkata
10.	Banerjee, A.K.	Chief Engineer (Retd.) MoRT&H, New Delhi
11.	Banerjee, T.B.	Chief Engineer (Retd.) MoRT&H, New Delhi
12.	Basa, Ashok	Director (Tech.) B. Engineers & Builders Ltd., Bhubaneswar
13.	Bhasin, P.C.	ADG (B), (Retd.), MoRT&H, New Delhi
14.	Bhowmick, Alok	Managing Director, Bridge & Structural Engg. Consultants (P) Ltd., Noida
15.	Bongirwar, P.L.	Advisor, L&T, Mumbai
16.	Dhodapkar, A.N.	Chief Engineer (Retd.) MoRT&H, New Delhi
17.	Ghoshal, A.	Director and Vice President, STUP Consultants (P) Ltd. Kolkata
18.	Joglekar, S.G.	Vice President, STUP Consultants (P) Ltd., Mumbai
19.	Kand, C.V.	Chief Engineer (Retd.), MP, PWD Bhopal
20.	Koshi, Ninan	DG(RD) & Addl. Secy., (Retd) MOST, New Delhi
21.	Kumar, Ashok	Chief Engineer (Retd.), MoRT&H, New Delhi
22.	Kumar, Prafulla	DG (RD) & AS, MoRT&H (Retd.) New Delhi
23.	Kumar, Vijay	E-in-Chief (Retd.) UP, PWD,
24.	Manjure, P.Y.	Director, Freyssinet Prestressed Concrete Co. Mumbai
25.	Mukherjee, M.K.	Chief Engineer (Retd.) MoRT&H, New Delhi
26.	Nagpal, A.K.	Prof. IIT, New Delhi
27.	Narain, A.D.	DG (RD) & AS, MoRT&H (Retd.) New Delhi

28.	Ninan, R.S.	Chief Engineer (Retd.) MoRT&H New Delhi
29.	Pandey, R.K.	Chief Engineer (Planning), MoRT&H, New Delhi
30.	Parameswaran, Dr. (Mrs.) Lakshmy	Chief Scientist (BAS), CRRI, New Delhi
31.	Raizada, Pratap S.	Vice President (Corporate Affairs). Gammon India Ltd. Mumbai
32.	Rao, Dr. M.V.B.	A-181, Sarita Vihar, New Delh
33.	Roy, Dr. B.C.	Senior Executive Director, M/s. Consulting Engg. Services India (Pvt.) Ltd. Gurgaon
34.	Saha, Dr. G.P.	Executive Director, Construma Consultancy (P) Ltd. Mumbai
35.	Sharan, G.	DG (RD) & Spl. Secy (Retd.) MoRT&H, New Dellhi
36.	Sharma, R.S.	Chief Engineer (Retd.) MoRT&H, New Delhi
37.	Sinha, N.K.	DG(RD) & SS, (Retd.) MoRT&H New Delhi
38.	Subbarao, Dr. Harshavardhan	Chairman & Managing Director, Construma Consultancy (P) Ltd. Mumbai
39.	Tandon, Mahesh Prof.	Managing Director, Tandon Consultants (P) Ltd., New Delhi
40.	Thandavan, K.B.	Chief Engineer (Retd.) MoRT&H, New Delhi
41.	Velayutham, V.	DG (RD) & SS (Retd.) MoRT&H, New Delhi
42.	Viswanathan, T.	7046, Sector B, Pocket 10 , Vasant Kunj ,New Delhi
43.	The Executive Director (B&S)	RDSO, Lucknow
44.	The Director and Head, (Civil Engg.),	Bureau of Indian Standards, New Delhi

Corresponding Members

1.	Raina, Dr. V.K.	Consultant (W.B.)
2.	Singh, R.B.	Director, Projects Consulting India (P) Ltd. New Delhi

Ex-Officio Members

1.	President, IRC and Director General (Road Development) & Special Secretary	(Kandasamy, C.), Ministry of Road Transport & Highways, New Delhi
2.	Secretary General	(Prasad, Vishnu Shankar), Indian Roads Congress, New Delhi

Notations and Symbols

Φ	Angle of internal friction of soil/fill
σ_{vi}	is the factored vertical stress at the i^{th} level of reinforcement
B_i	can be taken as (h_i + b) if $h_i \leq$ (2d-b) or (h_i + b) /2 + d if $h_i \geq$ (2d-b)
D	is offset D $\geq H_1 \tan(45 - \frac{\phi}{2})$ or D $\leq \left(\frac{H_1 + H_2}{20}\right)$

where,

H_1 is taller wall and H_2 is shorter wall

F_L	Horizontal shear force applied to the contact of the loaded area
K	Lateral earth pressure coefficient
K_{0j}	earth pressure at rest, K_a is the active earth pressure coefficient
LTD	Long-term design strength
Q	is given by the expression tan (45 - Φ/2) / (d + b/2)
S_L	is the strip loading applied on a contact area of width b
S_{vi}	is the vertical spacing of reinforcement at the i^{th} level of reinforcement
T_{char}	is the characteristic value of tensile strength
T_i	Tensile force/m. running length
b	Width of contact area
d	Distance of CG of load from Facia
f'_1	is the reduction factor due to manufacturing processes
f_1	is the partial load factor for applied concentrated loads (1.2 for combination A)
f_2	is the reduction factor for creep applicable for the design life and design temperature
f_3	is the reduction factor for installation damage appropriate for the fill material particle shape and gradation
f_4	is the reduction factor for environmental damage
h_i	is the distance to the top of the i^{th} layer

where,

P_j is the horizontal width of the top and bottom faces of the reinforcement element at the j^{th} layer per metre run

T_j	is the maximum tension as evaluated from theequation in section 3.2.a above

f_{fs}	is the partial load factor applied to soil self-weight for the same load combination as Tj refer section 3.3 – load combinations
f_f	is the partial load factor applied to surcharge load for the same combinations as T_j – section 3.3 – load combinations
μ	is the coefficient of friction between the fill and the reinforcing element
W_s	is the surcharge due to dead loads only
f_p	is the partial factor for reinforcement pull out resistance – 1.3
f_n	is the partial factor for economic ramifications of failure – 1.1
a'_{bc}	is the adhesion coefficient between the soil and the reinforcement
c'	is the cohesion of the soil under effective stress conditions
f_{ms}	is the partial safety factor applied to c' may be taken as 1.6
h_j –	Depth of the j^{th} reinforcement below top of the structure
P_j	is the horizontal width of the top and bottom faces of the reinforcement element at the
j^{th}	layer per metre run

GUIDELINES FOR DESIGN AND CONSTRUCTION OF REINFORCED SOIL WALLS

BACKGROUND

The IRC B-3 Sub-group headed by Prof. Sharad Mhaiskar has drafted the "Guidelines for Design and Construction of Reinforced Soil Walls" by working assiduously over the last 4 years. In the intervening period the FHWA, British codes issued revised editions in 2009 and 2010 respectively. MORTH has promulgated guidelines in 2013 which gave brief Specifications for Reinforced Soil Walls. The guidelines before finalisation have been shared with all the stakeholders ranging from manufacturers, designers, consultants, owners and contractors.

During the deliberations the sub-committee received diverse suggestions from stakeholders based on their understanding, experience and interest. However, while drafting the guidelines it has been ensured that the design as well as construction proceeds in a safe and conservative manner, keeping in mind the complications faced in addressing serviceability issues post-construction. Experience gained in adopting RS Wall technology on several projects in the country helped the group in drafting these guidelines. State of construction practices as well as QA and QC procedures followed in the country have been uppermost in the mind while drafting the provisions of the guidelines.

Several crucial issues like estimation of Φ, testing of reinforcement as well as connection testing, reinforced and retained soil/fill testing, causes of failure, design methods, classification of reinforcement in to extensible and inextensible types and special cases of geometry, have been addressed in the guidelines. The design principles are based on the limit state approach. The guidelines also include a worked example to demonstrate the provisions in the guidelines.

The initial draft document on "Guidelines for Design and Construction of Reinforced Soil Walls" was discussed in number of meetings of B-3 Committee and document was approved by the B-3 Committee in its meeting held on 14.05.2013 for placing before the BSS Committee. BSS Committee in its meeting held on 18.7.2013 decided that B-3 Committee should first finalise the document based on the comments received and forward the same to IRC Sectt. for circulating amongst H-4 Committee members for holding the joint meeting of B-3 and H-4 Committees.

Accordingly, draft was reviewed by H-4 Committee members and was further discussed in number of meetings of B-3 and H-4 Committees. This document was then discussed and finalised in the joint meeting of B-3 and H-4 Committees held on 23.11.2013 and then discussed again in the meeting of working group constituted by B-3 and H-4 Committees on 3.12.2013. Alongwith the H-4 Committee members (Shri P.J. Rao, Ms. Minimol Korulla, Shri Atanu Mandal, Shri Rajiv Goel and others), valuable suggestions and contributions were received from Shri N.K. Sinha, Dr. M.V.B. Rao, Dr. B.P. Bagish, Shri Alok Bhowmick, Shri R.R. Chonkar, Shri. Shahrokh Bagli, Prof. M.R. Madhav and others. The contribution and guidance of the Shri P.L. Bongirwar, Convenor, B-3 Committee and Shri S.G. Joglekar Co-convenor, B-3 Committee has been immense during the course of the drafting of the guidelines.

Promulgation of these guidelines will ensure that the RS Walls will be designed and constructed with great care and diligence, and would stand the test of time.

The BSS Committee approved this document in its meeting held on 6th January, 2014. The Executive Committee in its meeting held on 9th January, 2014 approved this document. Finally, the document was considered and approved by the IRC Council in its 201st meeting held on 19th January, 2014 at Guwahati (Assam) for publishing by IRC.

The personnel of Foundation Sub-structure Protective Works and Masonry Structures Committee (B-3) and Embankment Ground Improvement and Drainage Committee (H-4) are appended.

1 INTRODUCTION

Reinforced Soil (RS) Walls are in use for more than 40 years world over and for the last 25 years in India and are increasingly being adopted in highway and bridge construction. These applications call for use of relatively new technology and materials. The developments in the theory, design methods and experience of the behaviour of RS Walls gained in laboratories, full scale tests and field applications in India and abroad have brought knowledge from developmental stage to widespread applications in hands of practicing engineers. This powerful method will be increasingly adopted in road and bridge projects. Publication of these guidelines (referred as 'Guidelines' hereafter) covering the design and construction methods for benefit of the new as well as existing users is overdue, as need is felt to bring consistency in design and philosophy adopted by various system suppliers and to ensure minimum standard and criteria for acceptance of materials so that design life of 100 years is assured and to bring uniformity in partial load factors and partial material safety factors fulfilling a long standing need in the field of Highways and Bridges Engineering.

2 SCOPE

The design and construction of approaches retaining soil/fill leading to the open spans of bridges, flyovers, road over-bridges crossing railways and retaining walls of high fills for road embankments are covered in these Guidelines. Reinforced soil improves the load carrying capacity and reduces compressibility under the load. Technical details of such applications are outside the scope of the Guidelines. The use of RS Walls as abutment to carry the loads from open spans is not covered by these guidelines. Reinforced Soil Structures with slope angle of less than 70 degrees to the horizontal are not covered by these guidelines.

The coverage of Guidelines includes, but is not limited to:

- Materials and properties of reinforcement as well as the soil used in its construction
- Types of reinforcements, fills, and facings
- Testing of materials
- Design methods
- Broad method of construction to realise desired properties and behaviour of RS Walls
- Overview of non-conventional ground improvement methods used in RS Walls.

3 ELEMENTS OF RS WALLS, MATERIALS THEIR PROPERTIES

Fig.1 shows the elements of a Reinforced Soil Wall. Following sections describe the characteristics of materials used in construction of RS Walls. In addition to the reinforcing elements, the performance of the reinforced soil structure hinges on retained fill, reinforced fill and the aggregates used in the drainage bay and/or other measures used for ensuring drainage.

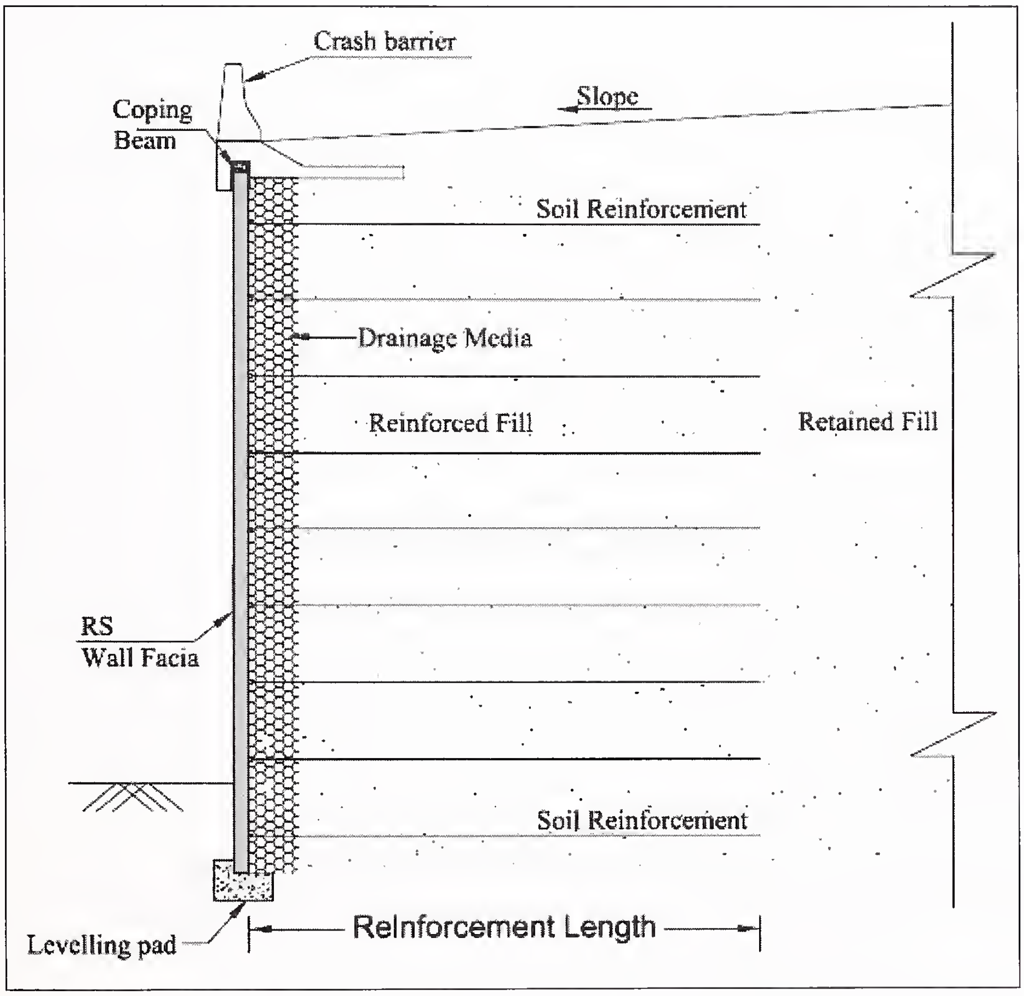


Fig. 1 Typical Cross Section for RS Wall

3.1 Retained soil/fill

The retained fill in case of 2 lane and 4 lane highway projects where the total width is not very significant shall be of same specification as reinforced fill. However, in the case of six lane projects or four lane with slope surcharge it is experienced that large quantity of fill is required. In case the retained fill is not available in requisite quantity, fill meeting the criteria mentioned below may be used.

a) Recommended properties:

   i) Angle of internal friction (ϕ) ≥ 25°

   ii) Plasticity Index ≤ 20

The configuration of retained fill is shown in **Figs. 2A and 2B**.

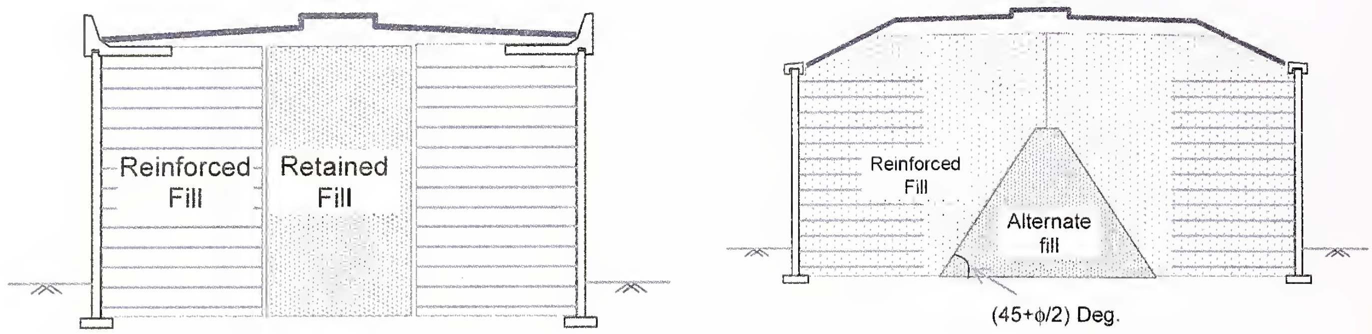


Fig. 2 A

Fig. 2B

Fig. 2 Retained Fill of Different Specifications in 6 Lane Highways and Expressways of Varying Width (Figure Not to Scale)

Retained soil can be natural or borrowed. Properties of the retained soil are essential to determine the lateral earth pressure. In case configuration as per **Fig. 2A** is adopted the earth pressure acting on the reinforced fill will be a function of angle of friction of retained fill. The friction value of the retained fill in this arrangement will be lesser than that of the reinforced fill. Hence the earth pressure acting on reinforced soil mass will be more than the earth pressure compared to the case where the fill material in both zones are identical. Another possible configuration of retained fill is shown in **Fig. 2B**. In this arrangement since the retained fill is extended upto conventional Rankine's failure zone, the earth pressure acting on the reinforced zone is same as per the properties of reinforced fill. Properties of the retained soil/fill like grain size distribution, angle of internal friction (under drained and undrained conditions), Atterberg limits, density, and permeability should be determined before proceeding with design. If a retained fill is not permeable, drainage should be ensured by providing a drainage bay between the retained and reinforced fill as well as the retained soil and the founding soil, if required.

For the retained soil, the value of phi considered in design should be arrived at using a similar approach for the reinforced soil as outlined in the subsequent sections.

3.2 Reinforced Soil/Fill

The reinforced soil/fill is essentially borrowed. Properties of this soil play crucial role in the performance of the RS structure. The soil is borrowed from quarries, river beds etc. It is essential to know if the material borrowed would be consistent with reference to the quantity required and if not, what levels of variations are likely. Besides performance, the cost of the RS structure is also sensitive and dependent on the properties of the reinforced soil. It is desirable that the reinforced fill be free draining with majority of the shear strength component derived from internal friction. The desirable gradation of the reinforced fill is shown in **Table 1**. The gradation proposed would ensure that the fill is well graded, free draining and has adequate shear strength once it is compacted. Properties of the reinforced soil like grain size distribution, Atterberg limits, drained shear strength (peak as well as residual value), permeability, maximum dry density and OMC as obtained from a Heavy Compaction Test (corresponding to Modified Proctor Test) or relative density (whichever applicable), compactibility should be determined before proceeding with design with great care.

Table 1 Desirable Gradation for Reinforced Soil Fill

Seive Size	Percentage Finer (in %)
75 mm	100
4.75 mm	85 – 100
425 micron	60 – 90
75 micron	< 15

The backfill should also have Plasticity Index, PI ≤ 6 and Cu > 2.

Soil/Fill with more than 15 percent passing 75 micron sieve, but less than 10 percent of particles smaller than 15 microns are acceptable provided PI is less than 6 and angle of friction is not less than 30°.

As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to ¾ - in. (19 mm) for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill. Pre qualification tests on reinforcements using fill materials should be conducted before proceeding with design.

While using metallic reinforcement or metallic connection system, it should be ensured that electro-chemical properties of the fill are satisfactory and would not cause or trigger corrosion of the reinforcement. It is desirable that soil should have a resistivity ≥ 5000 ohm-cm at saturation. Metal reinforcement should not be used for soils with resistivity less than 1,000 ohm-cm. Soils with resistivity between 1000 to 5000 ohm-cm may be used provided the water extract from the soil does not show chlorides more than 100 ppm., sulphates do not exceed 200 ppm, and pH ranges from 5-10. Water used for compaction shall have resistivity more than 700 ohm-cm. Besides, the water used for compaction shall comply to permissible limits specified by IS-456-2000 i.e. Sulphates as SO_3 400 mg/lit., Chlorides 2000 mg/l, and pH not less than 6, organic content 200 mg/l, suspended matter 2000 mg/l.

Flyash confirming to IRC:SP- 58 can be used as reinforced as well as retained fill.

The quality of flyash should be controlled through periodical checks to ensure consistency and compliance to specifications.

In many regions of the country reinforced fill as mentioned above is not available. Clayey/ Sandy Gravel (residual soil) , classifying as GC, GM or GC-GM. may be used provided the fines content (defined as combined percentage of silt and clay i.e. -75 micron soil) does not exceed 20 percent. At the same time additional precautions, if required, to ensure that hydrostatic as well as pore pressure is not developed by providing adequate surface and sub-surface drainage system should be undertaken.

It is emphasized that the order of preference of reinforced fill is as follows:-

a) Clean, free draining, non-plastic fill meeting gradation and plasticity requirements specified earlier

b) Flyash confirming to IRC:SP-58.

c) Residual/soil Murum meeting above requirements with due precautions mentioned above.

d) Any other mechanically stabilised soil, blended by mechanical equipment and meeting gradation and other requirements mentioned above may be also used.

In many situations the reinforced fill and retained soil is same. Retained soil properties are used to calculate the lateral pressures; it is desirable to use value of Φ using the approach suggested for the reinforced soil, so that the appropriate lateral pressure is evaluated. Properties (principally shear strength and density) of the reinforced fill determine the length and tensile strength of the reinforcement. It is therefore desirable that the angle of internal friction should be evaluated considering the variability in the backfill, compactibility, state of quality control practised across the country etc.

For lateral pressure as well as for reinforcement calculations (strength as well as length) Φ_{design} should be taken as Φ_{peak}. The value of Φ_{peak} may be obtained by conducting a drained direct shear test as prescribed by IS 2720 part IV. The Direct Shear Test shall be conducted at 95 percent of Modified Proctor Maximum Dry Density or 80 percent of Relative Density. Where the fill contains gravel percentage more than 10 percent it is advisable to carry out a direct shear test using a 305 x 305 mm. direct shear box.

These recommendations are on the assumption that adequate drainage is ensured in the reinforced as well as retained zones.

The value of phi used in design i.e. Φ_{design} in no case shall exceed 34 degrees or average value of Φ_{peak} minus 2 * standard deviation based on the number of tests prescribed in section 4.2, as far as the reinforced fill is concerned, except for GM-GC soils satisfying the properties mentioned in the next paragraph.

Where the soil classifies as GM or GC, is acceptable as per gradation and plasticity norms and if it is ensured that 80 - 90 percent of the quantity of material is available for the project value of Φ_{design} may be based on the results of the large size direct shear box tests mentioned above with a limiting value of 38 degrees.

3.3 Drainage Bay

Normally, RS structures are not designed for hydrostatic pressures. Where hydrostatic pressures are likely due to submergence, the design should account for such pressure. To ensure that these conditions are realised in the field, adequate drainage measures need to be taken. A drainage bay of minimum 600 mm width at the back of the facing is commonly used. Appropriately, profiled blocks are also used for the facia which have provision for placing granular drainage. The desirable gradation of the aggregate used in the bay is indicated in **Table 2**. Besides meeting gradation requirements it should be ensured that the aggregates are not friable, flaky, elongated and are sound in strength. Relevant tests as per MORTH 2013 specifications may be used to judge the suitability of the material used in the drainage bay.

Table 2 Gradation for Drainage Bay

Sieve Opening, mm	Percentage Finer
37.50	90-100
20.00	80-100
12.50	0-20

Alternatively, a geo-composite which ensures adequate drainage may be provided. Specifications for Geo-composite should be as recommended in MORTH Specifications-2013 **Tables 700-9 and 700-10**.

Where RS Walls are provided to support hill cuts, the face of the hill cut is to be considered as a retained fill. To ensure that the run-off and sub-surface water is drained, a drainage bay should be provided between the retained soil and the reinforced soil to ensure proper drainage. The drainage bay should be designed to carry the discharge and should be provided vertically at the back of the retained fill and continued in a horizontal extent to a depth well below the toe of the RS Wall and lead to a drain meant to carry the discharge away from the RS Wall.

3.4 Facing Elements

The facing is provided to prevent spilling/falling over of the fill and also to provide firm anchorage to the reinforcement. Facings should be tough and robust. Facing also provides an aesthetic architectural finish to the RS structure.

The facing system shall be one of the following (Refer MORTH specifications -2013)

a) Precast reinforced concrete panels

b) Precast concrete blocks and precast concrete hollow blocks

c) Gabion facing

d) Wrap around facing using geosynthetics

e) Metallic facing, prefabricated in different shapes including welded wire grid and woven steel wire mesh

f) Other proprietary and proven systems

The facing should be designed to withstand the stresses it is subjected to. Typically the normal stresses arising would be due to the panels/blocks above it and forces and moments arising due to connections to reinforcement.

Connection between the facia panels and the reinforcing element shall be done by using either nut or bolt, HDPE inserts with bodkin joint, hollow embedded devices, polymeric/steel strips/rods/pipes, fibre glass dowels or any other material shown in the drawings. Several types of connections are being used.

In case of modular block facing where the reinforcement is held by friction between the facia block and the reinforcement, the results of pullout test as per ASTM D 6638 shall satisfy the requirement of the Long Term Design Strength of the primary reinforcement.

In the case of block facing the reinforcement is held by friction between the reinforcing element and the blocks. The connection strength is based on the friction between the blocks and reinforcement as well as block to block friction. Typical cross sections of walls using modular, gabion and panel facing using geogrids is shown in **Figs. 3A, 3B and 3C** Few varieties of blocks panels (not exhaustive) used in practice are shown in **Figs. 4A and 4B**.

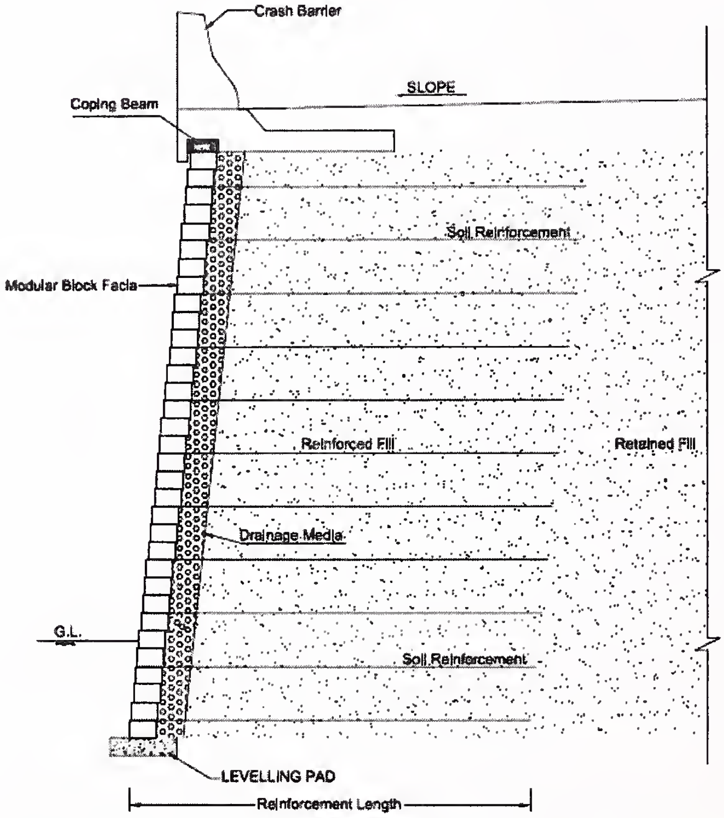


Fig. 3A Typical Cross Section of RS Wall with Modular Block Facia

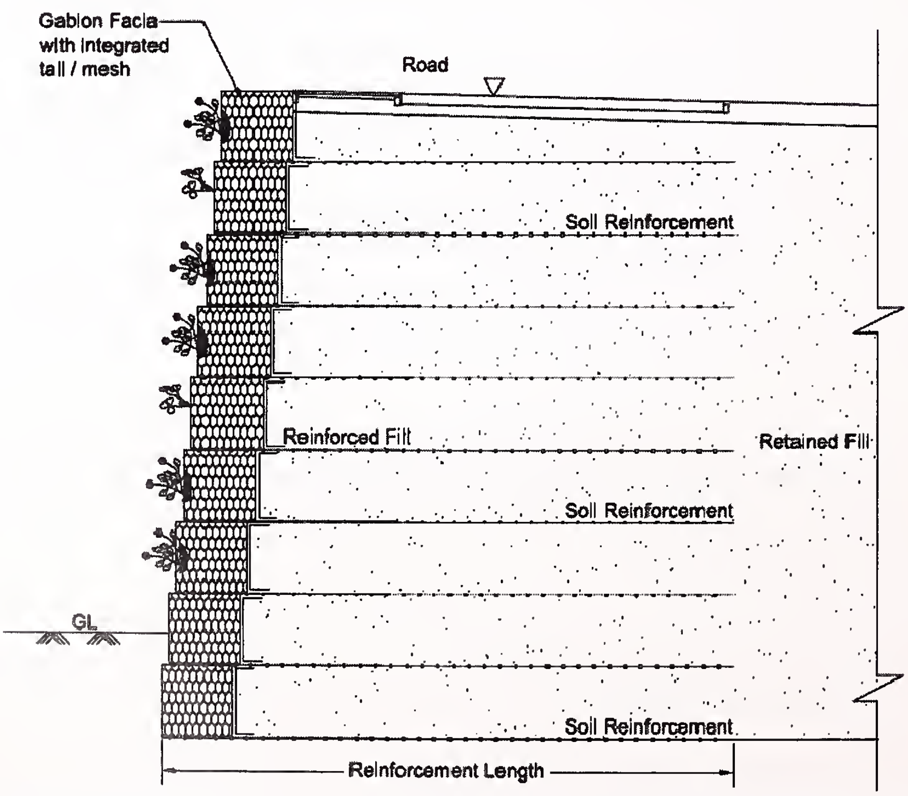


Fig. 3B Typical Cross Section of RS Wall with Gabion Facia with Integrated Tail

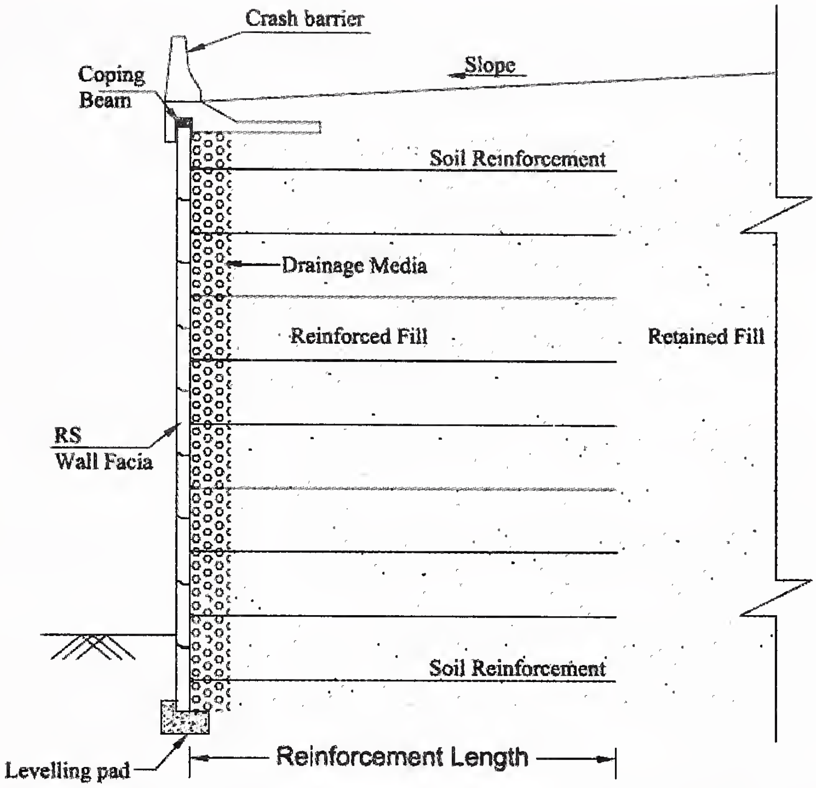


Fig. 3C Typical Cross Section of RS Wall with Concrete Panel Facia

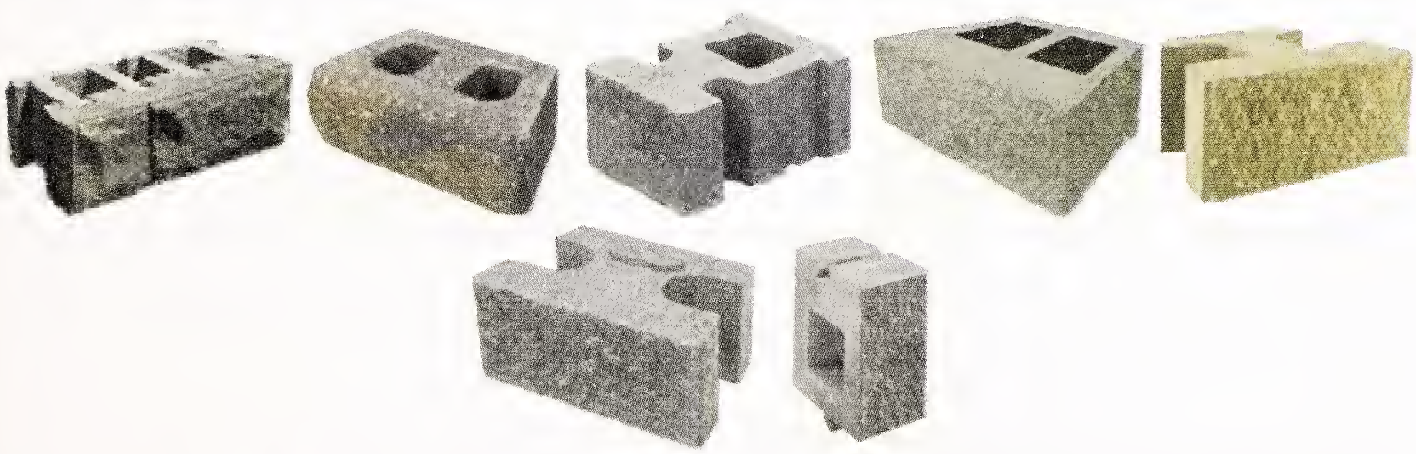


Fig. 4A Modular Blocks with Architectural Finish

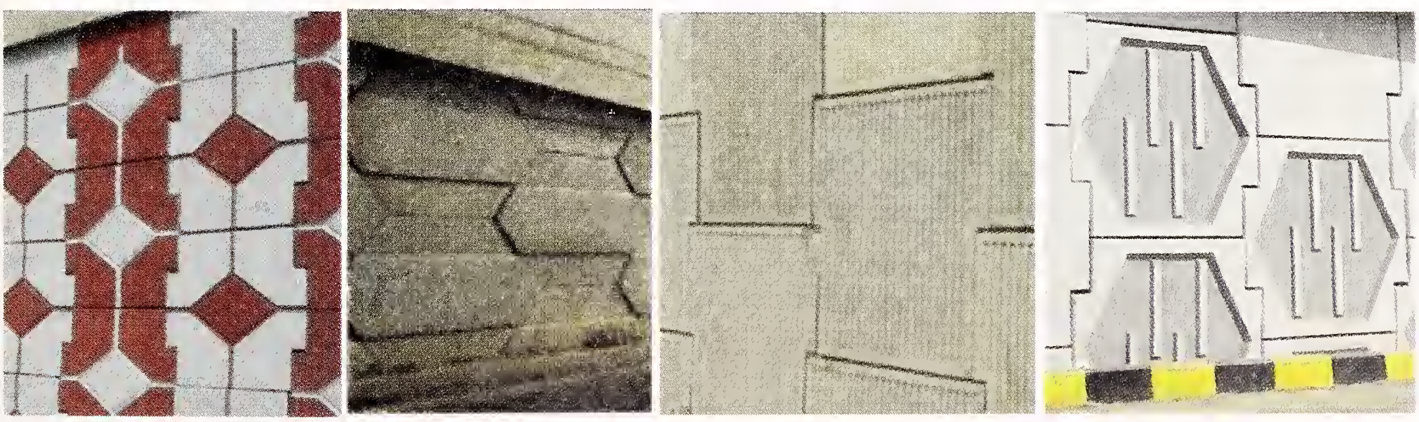


Fig. 4B Panels with Architectural Finish

Concrete (minimum M35 concrete strength) panels should be of minimum 140 mm. thickness (at its minimum), excluding architectural finishes and reinforced as per design requirements, unless otherwise validated by load tests and project reference.

Modular blocks should be manufactured using a block making machine and cast from a cement sand mix to attain a minimum concrete strength (M35) of 35 N/mm^2. In case of blocks, the hollow area shall not exceed 40 percent of the cross sectional area. The outer side of the block shall have a minimum thickness of 85 mm and the inner side 45 mm. Blocks may also be profiled to create hollows between adjacent blocks The hollow space shall be filled with clean, 20 mm. down sound, aggregate to add to friction between the reinforcing grid and facia blocks. The blocks and panels manufactured should have consistency/uniformity in dimensions and shape.

The facing chosen should be compatible with the extensibility of reinforcement. Compatibility to ensure flexibility of the system should be attained by choosing an appropriate combination of fascia and reinforcement.

The RS Wall system may be accepted by the Engineer-in-Charge if it has certification for material (mainly reinforcement) and connection strength, from accredited laboratories referred in IRC:113-2013 and **Table 3**.

In addition, it is desirable to have a CE marking for the reinforcement. Once a system is accepted by the Engineer-in-Charge shall not be changed during construction without the prior approval of the Engineer-in-Charge.

RS Walls because of their superior flexibility offer better resistance in seismic zones. However to ensure that there is no "falling of blocks", block facing can be used in zones where seismic activity is high, typically Zone-IV and Zone V, as defined by IRC:6, with additional measures like mechanical connection which shall transfer 100 percent load of long term design strength in continuity, shear key, pins etc. to enhance the sliding resistance and resistance to falling off. Use of block facing walls in a single rise beyond 10 m. should not be undertaken in these zones. Such walls should be designed using a berm.

Connections of the panel/block with the reinforcement should be clearly defined and tested using relevant ASTM standards. ASTM D-6916 and D-6638, gives test procedures for evaluating block to block friction and block to reinforcement connection strength (**Fig. 5**). Results of these tests should be provided by the supplier.

The connection strength between the reinforcing element and facing shall be as per BS: 8006-2010. It is once again reiterated that the connection strength and layout once used in design calculations, shall not be changed during execution, unless approved by Engineer-in-Charge. The method statement for construction of panels and blocks shall be approved by the Engineer-in-Charge. Designers and construction personnel should note that, several failures have occurred due to improper connections and deviation from the connections proposed in the approved designs.

In addition, the method of construction shall have quality assurance plan and tolerances as specified by Clause 3106.6 of MORTH 2013 Specifications.

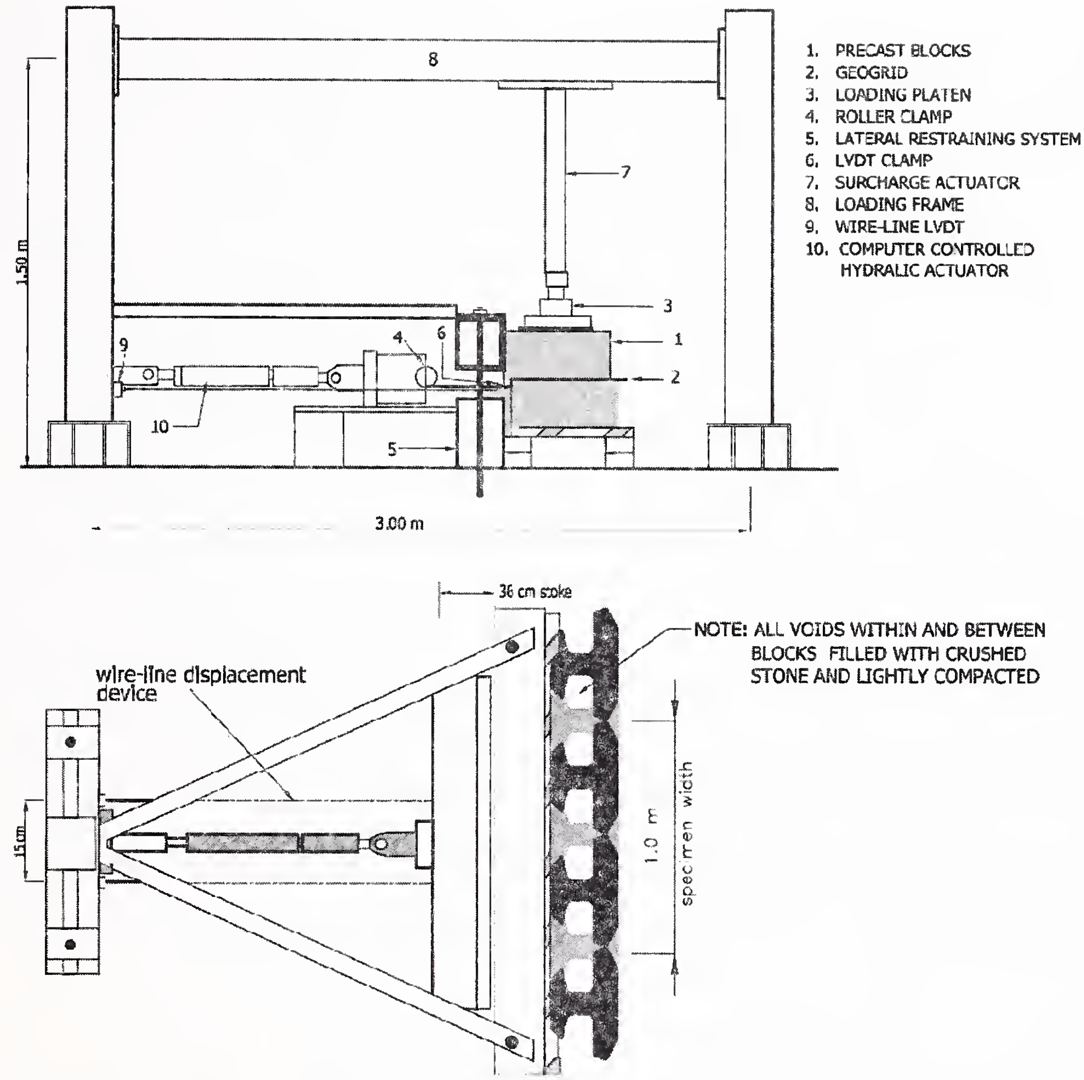


Fig. 5 Block to Reinforcement Friction/Connection Strength Test Details as per ASTM 6638

3.5 Reinforcement

Different types of reinforcements used in reinforced soil walls are:-

- Metallic elements like bars, strips, plates etc.
- Metallic reinforcement in form of mesh
- Polymeric elements like strips, grids, rods, mesh etc.

All types of Reinforcements are taken beyond the Rankine zone into the resistant zone to ensure sufficient bond and anchorage.

Reinforcement used to resist lateral loads can be metallic (typically inextensible) or polymeric (typically extensible). Polymers are visco-elastic materials. Strength of polymeric reinforcement is therefore largely affected by temperature and time (creep). Evaluation of strength should account for these two important factors. The tensile strength should be evaluated by conducting a wide width tensile test (ISO 10319 or ASTM D 6637or EN 10223-3 for woven steel wire mesh). All tests related to the reinforcement should be performed in an independent accredited laboratory which is accredited by a competent authority.

Partial safety factors are used to arrive at the long term design strength. While accounting for creep, design life of the adjoining super structure should be taken in to account. For example, since all the elements of a bridge are designed for 100 years, the strength of the polymeric reinforcement should also be estimated at the end of 100 years. Creep should be estimated keeping in mind the ambient temperature which in many parts of the country exceeds 40°C during summer, though in-situ temperatures may not exceed 30° - 35°C, that too for a short period of the year. The designer shall provide partial safety factors for creep, for 20°, 30° and 40°C degrees. These results should also include creep rupture and creep strain at these temperatures. Partial Safety factor used in design should correspond to the temperature calculated by the procedure given in EN/ISO 20432. The design temperature should be taken as half way between the average yearly air temperature and normal daily temperature (shade temperature) for the hottest month at the site. For obtaining the shade air temperature in a zone as given by IRC-6 may be referred. Creep factors for different temperature should be arrived at by conducting tests as per ISO 20432 in independent accredited laboratories.

Polymeric reinforcement shall have minimum of 10000 hours creep test data or SIM test data at different temperatures to evaluate partial material factor for creep. It is essential that SIM test results be backed by few conventional tests taken to some limited hours.

Manufacturing of reinforcement should confirm to ISO 9001 standard, to ensure that quality processes are stringently followed during manufacturing processes.

Long-term design strength of geosynthetic reinforcement should be determined in accordance with ISO/TR 20432. The long-term design strength (LTD) of geosynthetic reinforcement should be determined as:

$$LTD = T_{char}/\{f_1 * f_2 * f_3 * f_4\}$$

where,

T_{char} is the characteristic value (95 percent confidence limit) of tensile strength

f_1 is the reduction factor due to manufacturing processes

f_2 is the reduction factor for creep applicable for the design life and design temperature

f_3 is the reduction factor for installation damage appropriate for the fill material particle shape and gradation

f_4 is the reduction factor for environmental damage

Metallic reinforcement exhibits relatively negligible creep. The metallic reinforcement should be coated with zinc to delay exposure and eventual corrosion. The zinc coating should confirm to relevant BS/IS Code and in any case should not be less than 140 micron. In addition to the zinc coating, sacrificial thickness of minimum 0.50 mm should be provided on all sides, while designing. The zinc coated metal reinforcement shall be free of holidays and shall be subject to 100 percent inspection, since holidays can accelerate corrosion due to galvanic effect. Care shall be exercised during compaction of granular fill with zinc coated metal strips to ensure the integrity of the coating. Metallic reinforcement should be manufactured in a facility having ISO certification. Metallic elements and fasteners connections should be coated by zinc coating of 80 micron.

3.6 Traffic Barriers

Traffic barriers (Crash barriers) are constructed over the front face of the reinforced walls. Commonly, a friction slab is used to transfer the lateral loads due to impact of vehicles on the Barriers. Typically a friction slab varies from 1500 to 2500 mm width and 250 mm thick depending on the type of crash barrier provided. One aspect to be taken care of is the 'Friction slab' in the approach embankments. Unlike the approach slab which extends throughout the width of the embankments, the friction slab width depends upon the design adequacy extending only for the part of the embankment width. It is necessary to make detailed design for the friction slab taking care of adequate factor of safety against sliding, overturning etc. in addition to the structural design of crash barrier.

If the barrier is placed without a shoulder the lateral force due to the impact of a vehicle would be transferred to the upper layer of the reinforcement. The barrier should be designed for an impact force as stipulated by the IRC Codes/guidelines. The impact force resulting due to the impact shall be distributed equally to the upper two rows of the soil reinforcement, which the reinforcements resist over their full length. It should be structurally sized to resist at least 3 tonnes per metre. A typical arrangement of a crash barrier without a shoulder is shown in **Fig. 6**. Designers should note that there have been several cases of failure of RS Walls due to improper design, detailing and construction of Traffic/Crash barriers. Friction slab shall be designed in a manner consistent with the type of crash barrier provided.

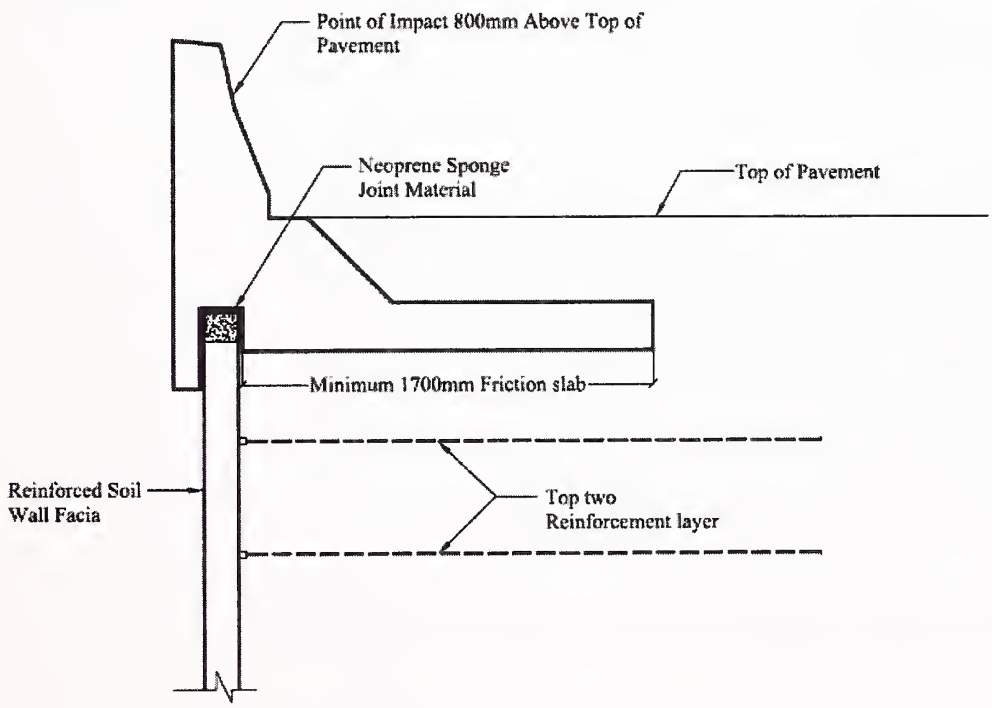


Fig. 6 Crash Barrier Section without Shoulder

4 QUALITY CONTROL TESTS DURING CONSTRUCTION

The different components that go to make up a RS Wall need to be chosen carefully after carrying out appropriate tests. Besides the quality control tests should be also undertaken with stringent control with reference to the tests and their frequency.

Before proceeding with the design of RS Walls, adequate geotechnical exploration should be carried out to ensure that the all necessary soil properties required for design of the wall are available. IRC:78 may referred for details of extent and depth of exploration.

The feasibility of using earth retaining system depends on the existing topography, subsurface conditions, and soil/rock properties. It is necessary to perform a comprehensive subsurface exploration to evaluate site stability, likely settlement, need for drainage, etc., before designing a new retaining wall.

Subsurface investigations are required not only in the area of the construction but also behind and in front of the structure to assess overall performance behaviour. The subsurface investigation shall enable study of conditions that prevail throughout the construction of the structure, such as the stability of construction.

4.1 Tests for Reinforcement

The reinforcement is chosen on the basis of the test results provided by the supplier. Such tests are referred as "Index Tests". All tests should have been conducted in an independent accredited laboratory. The laboratory/tests should have received accreditation from a competent authority. Tensile test results furnished by the manufacturer should be recent, i.e. tests conducted less than a year old, before proof checking and fit for construction approval is accorded.

Tests should include Tensile tests, (Stress strain graph), creep test results, tests to determine resistance to mechanical and environmental damage, raw material used and other properties characterising the reinforcement e.g. Aperture size, wt./sq.m. etc.

The tests performed to evaluate the in-situ/life time performance like resistance to installation damage, environmental damage, creep, type of raw material, carboxyl end group and molecular weight, should be also provided by the supplier. Creep test results for 20°, 30° and 40°C should be provided. The testing should also include tests to evaluate block to block and block to reinforcement testing as specified by ASTM tests mentioned earlier. The supplier should also clearly indicate the methodology of identifying the reinforcement vis-à-vis its strength in the field.

Certification/test report for long-term creep shall state the reinforcement products in their final configuration (finished product); creep test run on the raw material (resin/yarn) used in the manufacture of the geosynthetic material shall not to be considered as representative of the finished product. The manufacturer shall give a Declaration of Conformity (DoC) stating that the products supplied for a project fully comply with the product that were tested for creep and for which the creep test report/certification are submitted.

Polyester geosynthetics shall have molecular weight greater than or equal to 25000 g/mol and carboxyl end groups less than or equal to 30 mmol/Kg. Manufacturer shall furnish test results of these parameters and sign a declaration that the same raw material is used in the finished product.

Once the design is approved the reinforcement arriving at the site should be tested for tensile strength, in an independent accredited laboratory, at frequency of 1 set per 5000 sq. m. of wall facia area or two sets of samples whichever is higher. Samples should be drawn randomly from the reinforcement at site in presence of the user or his representative.

Metallic reinforcement should confirm to Clause 3103 of MORTH 2013 Specifications.

4.2 Tests for Reinforced and Retained Fill

The soil which is proposed to be used as reinforced fill shall be tested to ascertain the suitability for required quantity, grading, type and availability of required quantity etc.

The soil to be used as retained fill behind the reinforced fill, in case it is not natural soil, shall be tested for its shear characteristics and permeability to evaluate earth pressure, drainage characteristics etc. for external stability of the wall.

The backfill is tested at two stages. The first stage is to ascertain the suitability of the fill while the second stage to ensure that the backfill envisaged in design is used during construction.

To ascertain the suitability of the fill, samples should be drawn from the borrow area by drawing a grid of 25 m c/c to full depth, logging and sampling for ascertaining suitability of the borrow material as per MORTH 2013 Specifications. Following tests shall be carried out as per Indian Standards.

i) Sieve Analysis – IS: 2720 Part - 2 tests per 3000 cu.m. of soil

ii) Atterberg Limit Tests- IS: 2720 Part- 5 – 2 tests per 3000 cu.m. of soil

iii) Compaction Tests – IS: 2720 relevant part corresponding to modified as well as Standard Proctor test – 2 tests per 3000 cu.m. of soil

iv) Direct Shear Tests – IS: 2720 Part 13 & 39 to ascertain the peak angle of shearing resistance. The tests should be done at 95 percent of Modified Proctor Density at -2 percent of OMC at a frequency of 1 per 3000 cu.m. of fill

During construction the quality control should be exercised by conducting one set of density test of 3000 sq.m. of compacted area considering the importance of compaction in reinforced soil walls. (Clause 903.2.2 of MORTH 2013) One set shall consist of 6 tests. The density tests shall be carried out in accordance with IS-2720 Part 28. Density measurement by nuclear gauge may be carried out as an alternative. For such a test the number of tests per set shall be doubled. If the retained fill is borrowed tests mentioned above should be carried out at same frequency of reinforced fill. Frequency during construction shall be as per MORTH 2013 Specifications.

4.3 Materials for Concrete

Materials used for making concrete blocks and panels shall be as per specification and tests as specified in IRC:112.

5 DESIGN PRINCIPLES

Limit state principles are used in design of Reinforced Soil Walls. Two limit states considered in design are:

a) Ultimate limit state (collapse loads)

b) Serviceability limit state (ensuring that deformations are within prescribed limits).

Limit state design for reinforced soil walls uses partial safety factors applied to imposed loads, materials used and overall safety factor to include the consequences of failure. In the limit state approach, disturbing loads are increased by multiplying specified load factors to arrive at design load, while resisting forces are reduced by dividing by the specified material factors to arrive at the design strengths. In addition, for Reinforced Soil walls, a partial safety factor, f_n is used to account for consequences of failure, frequency of occurrence of loads etc., while considering the ultimate limit state, various potential failure mechanisms are considered. Internal and external stability is considered for different potential failure mechanisms.

In addition to normal principles of design on the basis of which earth retaining structures are designed, consideration has to be given to soil/reinforcement interaction while designing Reinforced soil structures.

Analysis is done in two distinct parts External Stability and Internal Stability.

External stability deals with stability of the reinforced block as a unit, while internal stability deals with mechanisms of transfer of lateral pressures to reinforcement and related mechanisms involved.

Once the design loads (serviceability load , or working load) are carried by the metallic reinforcement such as bars, plates etc. at an axial strain less than the strain in the soil, the reinforcement is classified as "inextensible" reinforcement. Polymeric reinforcements which are characterised by temperature and time dependent strains (creep) are normally classified as extensible reinforcements. However, Polymeric and other reinforcements which show less strain as compared to soil strain may be also classified as inextensible reinforcement.

Normally, when the design load is sustained at a total axial strain 1 percent or less the reinforcement is classified as inextensible (BS-8006).Where the design load is sustained at total axial tensile strain exceeding 1 percent the reinforcement is classified as extensible.

The onus of categorising the reinforcement as extensible or inextensible, based on one of the above mentioned approach, will entirely lie on the supplier. The supplier should provide tensile test results from an independent accredited laboratory/certifying agency in support. The test to classify the reinforcement needs to be done only at the design stage. Such a test need not be repeated with local soil. Alternatively, full scale instrumented test results can be accepted to classify the reinforcement.

Two methods are commonly used for analysing the internal stability of the reinforced soil structures. The "tie back wedge method" follows principles used in classical analysis of anchored retaining walls. The "coherent gravity method" is based on monitored and observed behaviour of reinforced structures using inextensible reinforcement. For inextensible reinforcements, lateral pressures are estimated using K_0 i.e. earth pressure at rest at top and shall be linearly tapered to active earth pressure coefficient K_a at 6m (height measured from top) and below, while when extensible reinforcement is used, active pressure should be considered in design throughout the height of the wall.

Limit state approach using partial safety factors is used for design of reinforced soil structures and structural elements also.

5.1 External Stability

RS Walls with uniform slope have been successfully constructed in India upto heights 20 m using polymeric reinforcements as well as metallic reinforcements. For heights exceeding 15 m the walls should be designed with a berm/step at an intermediate height, if polymeric reinforcements are used.

External stability of the reinforced soil mass/block is checked for three different conditions:-

a) **Bearing and tilt failure:-** Bearing pressure exerted by the reinforced soil mass on the founding strata should be such that there is sufficient margin against failure. The design should achieve a Factor of Safety of at least 1.4 in the limit state, after considering eccentricity and resultant pressures. It should be also ensured that the eccentricity is less than L/6 to avoid development of tension. Passive pressure in front of the wall should not be considered in the stability calculations. Minimum depth of embedment shall be 600 mm or H/20 whichever is more. It should be noted that H is the mechanical height, which is the vertical distance measured from the inner edge of the wall to the mid-height of the road crust. While constructing retaining walls on existing roads, especially concrete roads, the requirement of minimum embedment depth of 600 mm may be relaxed to 400 mm, provided precautions are taken to ensure that the front of the wall is protected from excavation post-construction and sufficient resistance is available to prevent sliding. Arrangement shown in **Annexure A0**, which uses a beam and anchor rods at the toe of the wall, can be adopted to ensure adequate lateral resistance towards sliding. Where the bearing capacity is not adequate, suitable ground improvement measures need to be undertaken. After construction of walls excavation in front of the walls should be strictly prohibited as it is likely to reduce the stability of the wall. **Annexure A1** gives a summary of ground improvement measures commonly used. The purpose of giving brief information of ground improvement methods is to highlight methods beyond the conventional approaches which are overlooked by designers.

b) **Sliding and overturning:-** Factor of Safety towards sliding and overturning due to lateral pressures imposed should be adequate. FS of at least 1.2 in the limit state should be achieved.

c) **Global stability:-** FS against a slip circle failure should be checked. FS of at least 1.30 under static conditions and 1.10 under dynamic/earthquake loads should be ensured considering un-factored load. The analysis should also check possible failure modes included the possibility of deep seated failure. The global stability analysis shall be carried out using standard software having capability of modelling reinforcement, different geometries of the wall and failure modes.

The soil properties of the reinforced soil, retained soil and the loads considered in the stability calculations are shown in **Figs. 7 and 8** shows the load combinations for internal and external stability.

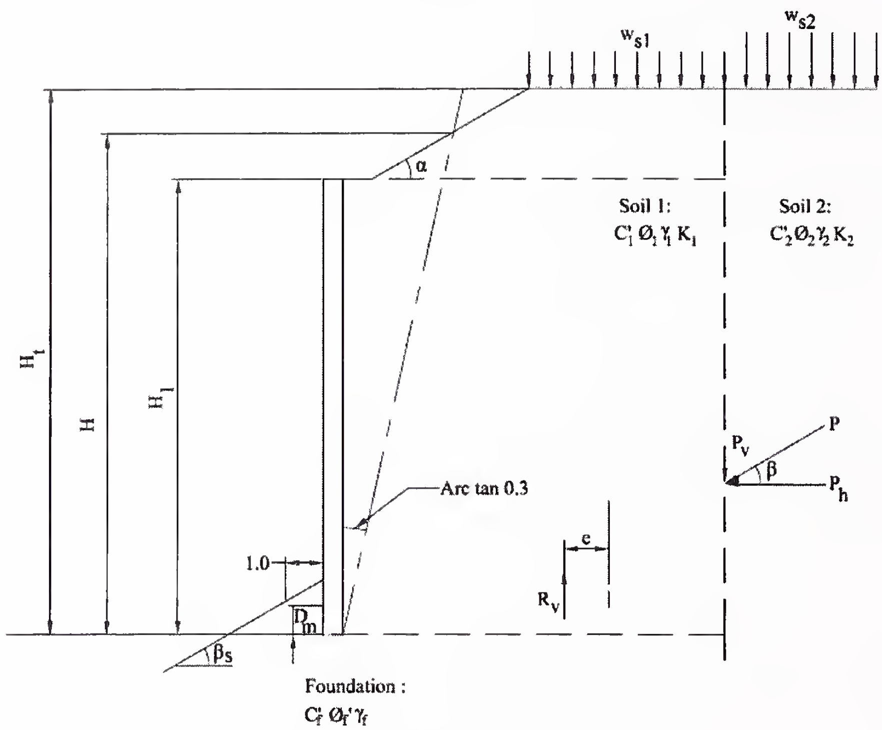


Fig. 7 Definition of Soil Properties and Loads (BS: 8006 - 2010)

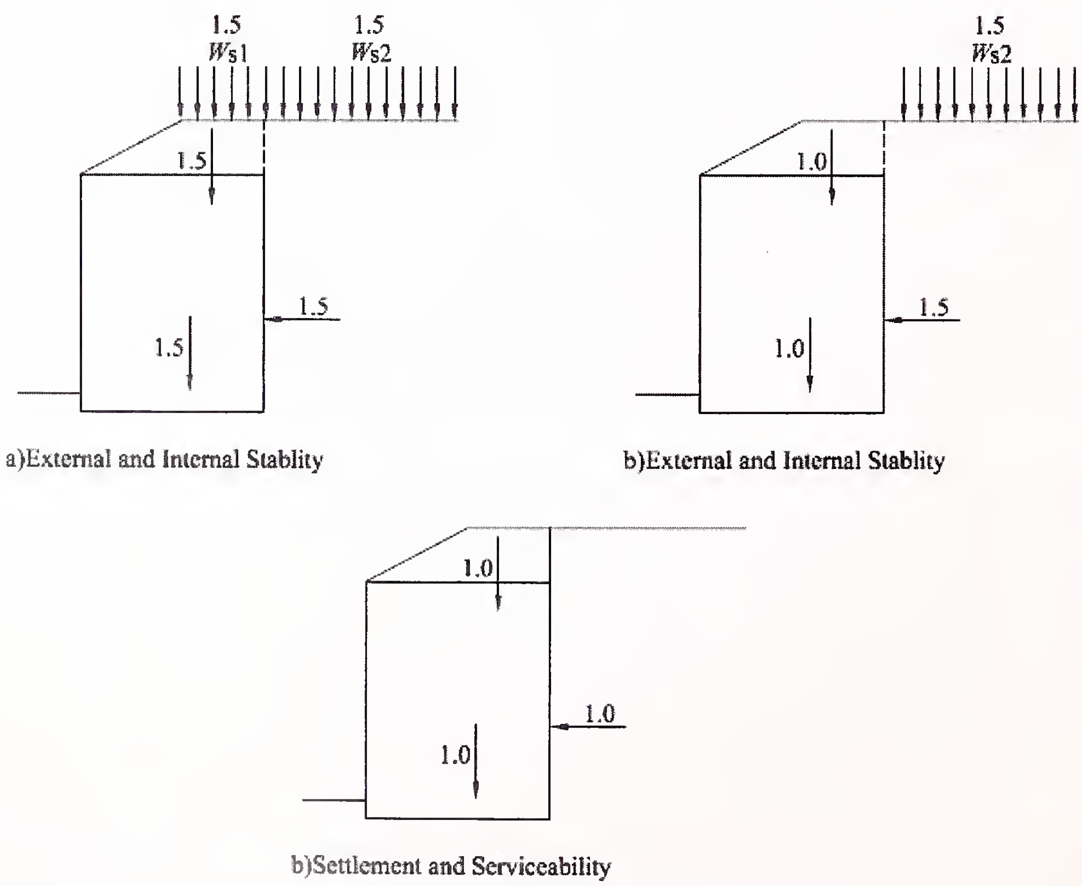


Fig. 8 Load Combinations for External and Internal Stability (BS: 8006 - 2010)

5.2 Internal Stability

a) Tie Back Wedge Method

The maximum ultimate limit state tensile force T_i to be resisted by a particular layer of reinforcement will be summation of lateral pressure arising due to self-weight of the fill, surcharge caused by external loading, strip loading applied on the top of the fill and shear applied at the contact of the strip loading. **Figs. 9A and 9B** show the different forces to be taken in to design. Tensile force/m running length can be calculated using the expression given below:-

$$T_i = K_a \sigma_{vi} S_{vi} + K_a S_{vi} (F_1 S_L/B_i) + 2S_{vi} F_1 F_L Q (1 - h_i Q) \qquad \text{Eqn. ... 1}$$

where,

K_a is the active earth pressure coefficient, taken as $(1 - \sin \Phi)/(1 + \sin \Phi)$

For a sloping surcharge and wall Ka may be estimated using the following equation

$$K_a = \{(\mathrm{Sin}^2(\alpha + \Phi))/[\sin^2\alpha * (\sin(\alpha - \delta)[1 + (\sqrt{\sin(\Phi + \delta) * \sin(\Phi - \beta)})/(\sin(\alpha + \beta))]^2\}$$

where,

α is the wall face makes with the horizontal

β is the slope makes with the horizontal

Φ is the angle of internal friction

δ friction angle between the soil and the wall which can be taken as $0.67*\Phi$

σ_{vi} is the factored vertical stress at the i^{th} level of reinforcement after considering Meyerhof's distribution i.e. eccentricity. A distribution of 2v:1h should be considered for evaluation of the vertical stress due to strip load only.

Note: $\sigma_{vi} = R_{vi}/(L-2e)$.

S_{vi} is the vertical spacing of reinforcement at the i^{th} level of reinforcement

F_1 is the partial load factor for applied concentrated loads (1.2 for combination A)

S_L is the strip loading applied on a contact area of width b

d is the distance from the inner face of the wall to the centre of the strip load

h_i is the distance to the top of the i^{th} layer

B_i can be taken as $(h_i + b)$ if $h_i \leq (2d - b)$ or $(h_i + b)/2 + d$ if $h_i \geq (2d - b)$

F_L Horizontal shear force applied to the contact of the loaded area

Q is given by the expression $\tan(45 - \Phi/2) / (d + b/2)$

Reduction in lateral pressure and tension, due to cohesion is not to be considered and neglected since cohesion may be lost under certain conditions.

The resistance of the i^{th} layer should be checked for rupture and adherence. In addition to the above checks the wedge stability of the mass above the i^{th} layer should be also checked.

Wedge stability is ensured when friction forces acting on a potential failure plane and the tensile resistance offered by the reinforcement are able to resist loads tending to cause movements.

Annexure A2 gives the design details for Rupture, Adherence and Wedge stability checks.

These checks will also ensure that the length of the reinforcement in the resistant zone is adequate.

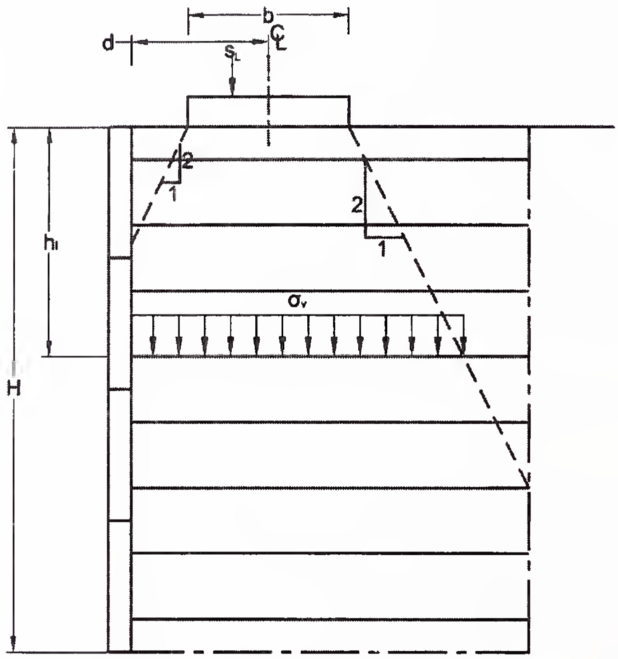


Fig. 9A Dispersal of Vertical Strip Load Through Reinforced Fill - Tie Back Wedge Method (BS: 8006 - 2010)

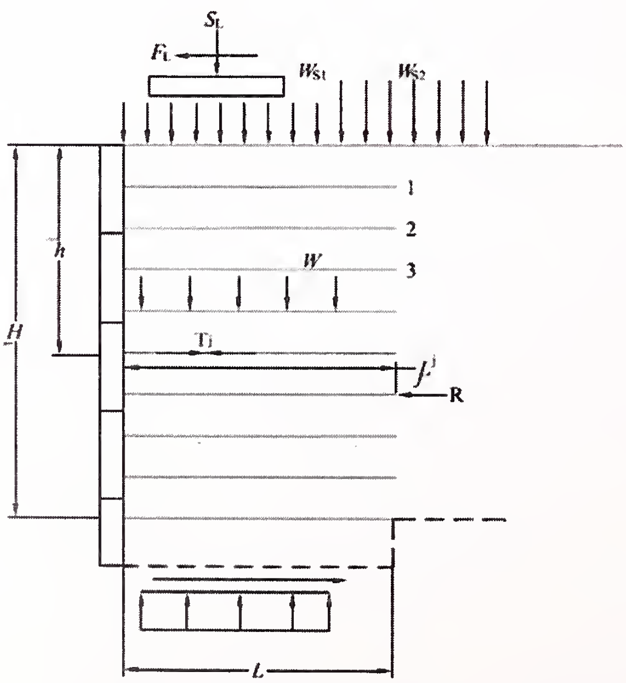


Fig. 9B Internal Stability – Effects of Loads to be Considered (Tie Back Wedge Method (BS:8006-2010)

b) Coherent Gravity Method

For the ultimate limit state and serviceability K is taken as K_0 at the top and linearly reducing to K_a at 6.00 m and below this depth. In case of inextensible reinforcement the lateral earth pressure coefficient K_o at top should be taken as 1 - sin ϕ to K_a at 6 m depth. Equation 2 may be used for evaluating tensile force/m Adherence capacity of the reinforcement and long term rupture should be checked. (Refer **Figs. 10 and 11**).

$T_i = K\, \sigma_{vi} S_{vi} + K{*}F_2{*}0.50{*}Q{*}[(F_B{*}(y/h_j) - F_B\,(z/h_j)]{*}\, S_{vj} + (2F_2 F_L\, S_{vi}/x) * (1- h_j/x)$ ... Eqn. 2

where,

K is the lateral earth pressure coefficient

x is given by (d + b/2)

K is the lateral earth pressure coefficient

y = (d' + b') and z = (d' - b')

S_{vj} is the vertical spacing of the reinforcement at the jth level

$F_B = (2/\Pi) * [(X/(1+X^2)) + \tan^{-1}(X)]$ where $\tan^{-1}$ X is in radians And

$X = (y/h_j)$

Q is the pressure beneath the footing as shown in **Fig. 8**.

h_j is the depth of the top up to the reinforcement

F_2 is the partial load factor for external load

F_L is the horizontal shear applied on the strip contact area

Reduction in lateral pressure and tension, due to cohesion is not considered since cohesion may be lost under certain conditions.

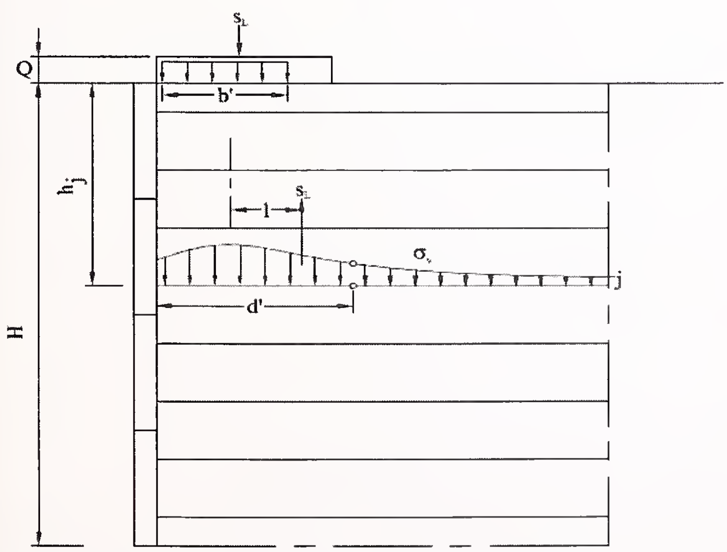


Fig. 10 Dispersal of Vertical Strip Load through the Reinforced Fill - Coherent Gravity Method (BS:8006-2010)

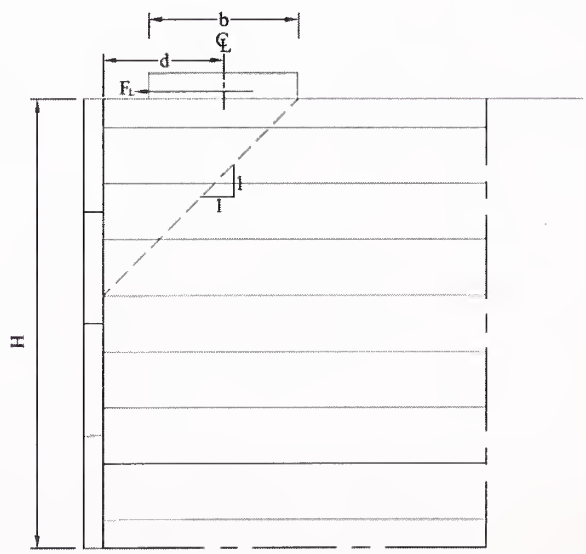


Fig. 11 Dispersal of Horizontal Shear Load through the Reinforced Fill - Coherent Gravity Method (BS:8006-2010)

Maximum Tension line for a coherent gravity structure can be approximated as shown in **Fig. 12**. This is commonly referred as Tension line 2. When a structure is subjected to strip load another tension line needs to be considered. This line is called as tension line 1 (BS-8006). Tension line 1 is defined in **Fig. 13**. The tension in the reinforcement should be calculated for three positions i.e. at the facing, along tension line 1 and tension line 2.

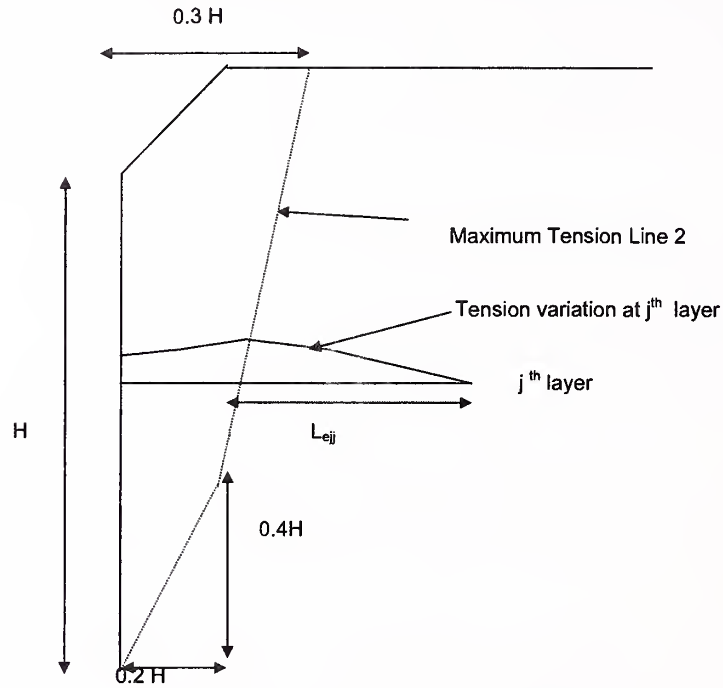


Fig. 12 Tension Line 2 for the Coherent Gravity Method (BS:8006-2010)

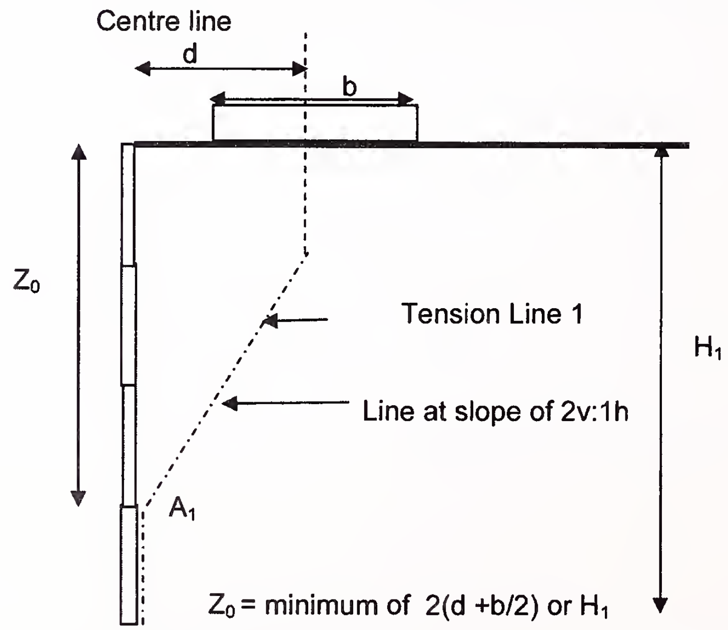


Fig. 13 Tension Line 1 for Coherent Gravity Method (BS:8006-2010)

For the tensile force at facing the first part of equation 2 should be multiplied by a constant a_0 while other components would remain unchanged. The value of a_0 may be taken as 0.85 for $h_j \leq (1.5\ H_1 - 3Y)$ where Y is the width of the active zone below the strip loading and can be taken as $1 - 0.15\ (H_1 - h_j)\ (H_1 - (1.5\ H_1 - Y))$.

For the tensile force along tension line 1 the first part of equation 2 should be multiplied by a constant a_o while the other components would remain unchanged. The value of a_o may be taken as 1 for $h_j \leq b$, or $a_0 + (1 - a_0)(Z_o - b)$ if $b < h_j < Z_0$ and equal to a_0 if $h_j > Z_0$.

For Tension force along tension line 2 equation 2 should be used.

The resistance of the i^{th} layer should be checked for rupture and adherence. In addition to the above checks the wedge stability of the mass above the i^{th} layer should be also checked. Wedge stability is ensured when friction forces acting on a potential failure plane and the tensile resistance offered by the reinforcement are able to resist loads tending to cause movements.

For both the methods described above global stability checks should be carried out in addition to local stability checks.

As far as rupture is concerned it should be ensured that

$$T_D/f_n \geq T_j$$

where,

T_D is the strength of the reinforcement at the end of the design life

f_n is the factor for ramifications of failure (refer **Table 3**)

T_j is the tensile force to be resisted by the j^{th} layer

The check for rupture should be carried out for both the methods (tie-back and coherent gravity).

Minimum length of reinforcement shall be 0.7 H or 3 m, whichever is more, where H = design height of the RS Retaining Wall. The design height should be calculated as depth of embedment plus the height above the ground level.

Some reinforced soil walls use a passive block at the end of the reinforcement to derive the benefit of the passive resistance in resisting the tension. Full passive resistance should not be taken in design since the strain required to mobilise the passive force is large. Such a strain level can be only attained when the deformation in the RS Walls is beyond the permissible level. The design calculations may only include 20 percent of the passive resistance.

5.3 Partial Safety Factors for Material and Load

a) A partial FS (f_n) 1.05 should be used for design of Reinforced soil structures for road/rail projects to account for ramifications of failure.

b) Reinforcement

   i) Polymeric

      1. Partial Safety Factor (f_1) for Manufacturing, quality control processes, estimated strength based on statistical reliability etc. should be as recommended by ISO standards Refer **Table 3**. Products manufactured from plants not following ISO manufacturing standards shall not be used.

Table 3 Summary of Partial Safety Factors

Sr. No.	Case	Partial Safety Factor
	Design	
1	Ramifications of failure, f_n	1.1
	Loads in different load combinations	
2	Case A	
	Dead Load	1.5
	Lateral Pressure	1.5
	Traffic Load	1.5
3	Case B	
	Dead Load	1.0
	Lateral Pressure	1.5
	Traffic Load behind the block	1.5
4.	Case C	
	Dead Load	1.0
	Lateral Pressure	1.0
	Earthquake Load	1.0
	Material Properties	
	Soil	
5	Angle of Internal friction ϕ_{design}	1.0
	Reinforcement	
6	Manufacturing processes as per ISO (f_1)	As per ISO recommendations
7	Creep effects (f_2) for polymeric Reinf.	*
8	Mechanical Damage (f_3) , for polym. Reinf.	*
9	Environmental Damage (f_4) , for poly. Reinf.	*
10	Metallic reinf. Over ultimate stress	1.5
	Reinforcement to reinforced soil interaction	
11	Sliding across reinforcement for ult. State	1.3
12	Sliding across reinforcement for serviceability	1.0
13	Pull out resistance of reinforcement for ult. State	1.3
14	Pull out resistance of reinforcement for serviceability state	1.0
	External Stability using Limit State	
15	Bearing and Tilt	1.4
16	Sliding	1.2
	Serviceability Limit State	
17	Slip circle , global stability, static (with no load factor)	1.3
18	Slip circle, global stability, dynamic (with no load factor)	1.1

* Relevant test data in support of these factors has to provided by the supplier.

* Tests should be conducted in accredited laboratories using materials proposed to be used.

* Tests for creep, mechanical damage and environmental damage can be more than a year old at the time of proof checking.

* Tests for Tensile strength should be recent. (not more than a year old at the time of proof checking)

* Connection tests can be done at IIT Madras, IIT Delhi or any other accredited laboratory where such facilities are available in the country.

* Tensile Tests on polymeric reinforcement can be conducted in laboratories listed in IRC:113-2013.

2. Partial Safety factor (f_2) to account for creep effects applied to laboratory tests data. This factor will be based on tests carried out in an independent accredited laboratory as spelt out in earlier sections.
3. Partial Safety Factor (f_3) for mechanical damage during construction should be based on tests carried out by an independent accredited laboratory. FS would depend on the type of reinforced fill.
4. Partial Safety Factor (f_4) for environmental damage should be based on tests carried out by an independent accredited laboratory.

Above partial safety factors should be used to reduce the long term strength to design strength and applied to the Minimum Average Roll Value for the product.

ii) Metallic

A partial FS (f_5) of 1.5 should be used to arrive at design strength once the ultimates tress of metallic reinforcement is estimated using relevant ISO test method.

a) Partial Safety factor for loads

The reinforced soil structure should be designed for following load combinations

Combinations A (generates maximum tension requirements)

1.5 * Dead Load + 1.5 * Lateral Pressure + 1.5 * Traffic load behind the Reinf. Block + 1.5 Traffic load on the block +Dynamic Load (Earthquake)

Combination B (critical for overturning and maximum eccentricity of resultant at base

1.0 * Dead Load + 1.5 * Lateral Pressure + 1.5 * Traffic load behind the Reinf. Block + Dynamic Load (Earthquake)

Combination C (critical for serviceability- deformations)

1.0 * Dead Load + 1:0 * Lateral Pressure + 1.0 Dynamic Load (Earthquake)

Annexure A3 gives the details of forces to be considered in external stability and internal stability due to seismic forces. The calculations are based on FHWA-10-024 & FHWA-10-025. Alternatively, the analysis can be also performed using the AFNOR NF P94 270

b) Partial Safety factors for reinforced soil

i) A partial FS of 1.0 may be used for angle of internal friction.

c) Partial Safety factor for reinforcement interaction
   i) Sliding across reinforcement surface 1.3 for ultimate limit state and 1.0 for serviceability
   ii) Pull out resistance of reinforcement 1.3 for ultimate limit state and 1.0 for serviceability.

Table 3 summarises the Partial Safety Factors.

Few cases of Reinforced Soil Walls with complex geometrics are shown in **Annexure A4** for the benefit of the users. **Annexure A5** gives a sample calculations Wall using see above mentioned principles.

5.4 Serviceability and Settlements

Settlement of the founding soil and the compression of the reinforced mass contribute towards the total settlement of a reinforced soil structure. Settlement of the founding soil can be estimated by conventional theories. Post construction settlement of the founding soil should not exceed 100 mm for discrete panels/and blocks which result in flexible structures. Settlements arising due to internal compression are normally small once compaction is done effectively. However the facing should be able to cope up with the internal compression. Typical safe vertical movements the fascia should resist should be taken as 1 in 150. Total settlement can affect functionality in a specific manner, differential settlements produces severe effects on the completed structure (although reinforced soil mass is known to be more accommodative as far as differential settlements are concerned), Typically differential settlement of 1 in 100 are considered as safe for discrete concrete panels facings (1 in 500 for full height panels).

5.4.1 *Construction tolerances and serviceability*

Reinforced soil structures deform during construction if proper care is not taken during compaction and erection. Due consideration should be given to ensure that during construction and post construction deformations and strains are within limits. Reinforced structures.

i) Should be visually acceptable, free of bulges and erratic alignments.
ii) should follow smooth curves.
iii) wall faces should not deform to cause spalling/cracking of face panel/s, closing/opening of joints.

Specifications outlined in section 3106.6 of MORTH-2013 document shall be referred for construction and serviceability tolerances, dimensional tolerances of panels and blocks.

5.5 Spacing and Layout of Reinforcement in Reinforced Soil Walls

The spacing of reinforcement shall be established based on the design principles. However, in the actual layout of reinforcing elements, the following shall be adhered to as provided Clause A2 in the MORTH-2013 guidelines.

i) To provide a coherent reinforced soil mass, the vertical spacing of primary reinforcement shall not exceed 800 mm, in all types of reinforcement.

ii) For walls constructed with modular blocks and deriving their connection capacity by friction, and also for any other facia configurations, where connection capacity is by friction, the maximum vertical spacing of reinforcement shall be two times the block width (measured from front fact to back face of the block). Further, the maximum spacing of reinforcing elements shall not exceed 800 mm in all cases.

The maximum height of facing left unreinforced

a) above the uppermost reinforcing layer and

b) below the lowest reinforcing layer, shall not exceed the width of the block (measured from the front face to back face of the block.)

iii) In case modular blocks are used for facia, no more than one intervening block shall be left without having primary reinforcement.

iv) In case of wraparound facings for walls, the maximum spacing of reinforcing elements shall not exceed 500 mm, to protect against bulging.

v) Where panels are used, the maximum spacing of reinforcement shall not exceed 800 mm. The spacing of nearest reinforcing element shall be such that maximum height of facing above uppermost reinforcement layer and below the lower most reinforcement layer does not exceed 400 mm.

vi) Reinforcement spacing worked out from the design procedures shall be configured to fit the above parameters.

Whereas the role of the primary reinforcement is to carry the tensile forces in the reinforced fill, secondary reinforcement may be required to protect the slope face from local sloughing and instability depending upon the facia configuration adopted. Where secondary reinforcement is used, stability of the area near the slope face shall be checked separately.

Where metallic type facia elements are used, the lower part of the facia element may be extended into the fill to serve as a secondary reinforcement. In other types of facia, geogrids may also be used as a secondary reinforcement. The length of the secondary reinforcement shall be adequate to provide local stability in the vicinity of the slope face.

6 CONSTRUCTION OF RS WALLS

The performance of the RS structure hinges not only on design but to a larger extent on the care and accuracy to which the construction is carried out. Construction of Reinforced soil structures should be therefore given due importance especially since it involves layer wise construction. The responsibility of the construction of a RS Wall will solely lies with the main contractor. Steps involved in construction are described below in the following section.

A) Placing and fixing of reinforcement and facing elements with backfill

1) Foundation treatment, if required, shall be first completed to ensure that design parameters are attained. It should be noted that use of RS Wall structures does not imply that “no foundation treatment is required.”

2) The plan of the structure shall be marked on ground as per approved drawings.
3) Excavate and compact the base the ground to the embedment depth and required width, to a dry density of 95 percent of the Modified Proctor Density.
4) The trench shall be backfilled using reinforced fill, levelled and well compacted to achieve 95 percent Modified Proctor Maximum Dry Density.
5) An initial levelling pad of 150 mm thick using (minimum) M15 plain cement concrete having suitable width to be placed below the first row of fascia layer.
6) The first layer of face block or element on the base and level envisaged in the drawing.
7) The alignment of the block/facing element must be checked regularly to make sure the wall is straight or curve as per drawing.
8) The required thickness of drainage material shall be placed at the back facing block/panel and in the hollows of facing block. The drainage material shall be compacted with vibratory plate compactor and within the block cavities. No heavy compaction equipment should be allowed to operate within 1.5 m of the back of face panel.
9) Placing the reinforced soil backfill behind the drainage zone and compacting to a minimum of 95 percent Modified Proctor density/80 percent Relative Density. The backfill should be placed and compacted in layers. The compacted thickness of each layer shall not exceed 200 mm. At no stage of construction the compaction or any other equipment shall be allowed to operate directly on the reinforcement.
10) When in direct contact, the backfill material and the drainage material shall be separated using permeable non-woven geotextile.
11) The successive face element shall be placed as per required line and level. In several cases outward movement has been observed due to poor connection of reinforcement with face element. Provisions given in Section 3.4 should be followed for details of connections. The same procedure shall be repeated until the final layer of reinforcement is reached.
12) Before placing the drainage material and backfill, the reinforcement should be cut to length and placed on top of the face block. The reinforcement should be stretched to ensure that there are no wrinkles and the reinforcement is taut.
13) Care should be taken to ensure that geogrid is slightly away from the external junction of outside face of fascia block. This will ensure that the geogrid does not protrude out of the wall and is prevented from UV ray exposure.
14) Second layer of facia block is laid over the geogrid, so that geogrid is completely interlocked between the blocks. The above procedure is repeated for subsequent geogrid layers.

15) Where panels are used, the reinforcement should be connected to connector embedded in the panel. The connection envisaged should be clearly indicated in the approved design and outlined in the "good/fit for construction drawings". At no stage of construction the details envisaged in design should be changed.

16) When panels are used it is desirable to keep an initial inward better. It should be in accordance with MoRTH 3106.3 which states that it may be necessary to set facing unit at an additional batter than as provided in the drawings since there is a tendency for initially positioned units of facia to lean outward as the fill material is placed and compacted. Care and caution shall be taken to accommodate this phenomenon. At the end of the construction, the face may have a slight residual inward batter.

17) Where the retained fill is borrowed and is different from the reinforced fill the construction should progress simultaneously.

It is desirable that face of the blocks be profiled to have an inward batter of 2-4 degrees.

7 COMMON CAUSES OF FAILURE OF RETAINING WALLS

Design and construction of Reinforced Soil walls is an involved process requiring due diligence and quality control. Moreover, repairs and remedial are often laborious, difficult, time consuming, expensive, often ineffective in the long run and in most of the cases impossible to implement. The designer and the owner should therefore be cautioned that while there are several advantages of using RS Walls these are not realised unless careful consideration is given to design as well as construction procedures. Failures can be in serviceability as well as collapse.

Common causes of distress and/or failures are summarised below.

Cause	Effect
Design Stage	
Inadequate Investigations regarding founding soil – typically erroneous or inadequate data for classification, shear strength, stratification	Excessive differential/total settlement resulting in Bulging/leaning of face panels and uneven riding surface in plan, bearing capacity failure - leading to excessive distortion or collapse
Inadequate investigations regarding borrow area material to be used as reinforced soil – typically data which would give reliable and consistent knowledge regarding shear strength and permeability properties of the entire borrow area fill	Difficulty in compaction, Buildup of hydrostatic pressure if the fill contains high percentage of fines resulting in bulging and/or leaning of fascia.
Inadequate inputs regarding reinforcement properties - typically data for creep, strength etc.	Excessive strain in the reinforcement resulting in bulging and or/local failures in the long run.

Cause	Effect
Inadequate drainage bay design to ensure drainage from retained fill	Excessive hydrostatic pressure/development of pore pressure resulting bulging and/or leaning of fascia.
Construction Stage	
Inadequate/Improper leveling pad construction as far as material, level etc. is concerned	Excessive settlement resulting in distortion/ leaning of the wall, and uneven riding surface
Compaction not meeting specifications	
Reinforced fill not meeting specifications in gradation, permeability plasticity characteristics etc.	Clogging of drainage pipes
Improper drainage details like perforated pipe details, laying, location in plan and elevation, outlet levels, etc.	
Improper Connection to fascia at variation with respect to specifications/drawing	Leaning and eventual collapse of panels/blocks leading to local failures/bulging of walls
Change in Connection details	
Heavy Compaction equipment coming within 1.5 m of the face of the wall.	
Drainage bay material not meeting specifications	
Initial batter not provided in panels	

8 REFERENCES/BIBLIOGRAPHY

A) International Standards

1) "Code of Practice for Strengthened/Reinforced Soils and Other fills", BS 8006 1:2010.
2) "Design and Construction of Mechanically Stabilised Earth Walls and Reinforced Soil Slopes - Volume I and II", US DoT, FHWA-10-024, November 2009.
3) AFNOR NF-P94-270-"Geotechnical Design- Retaining Structures- Reinforced and Soil Nailing Structures"

B) Other Standards

1) MORTH Specifications for Roads and Bridges – Published by IRC - 5th Edition, 2013.
2) "Plain and Reinforced Concrete- Code of Practice", IS:456, 2000
3) "Design of Earthquake Resistant Structures- Part 1", IS:1893, 2002

4) "Compendium of Indian Standards on Indian Standards on Soil Engineering, Part-1, Laboratory Testing of Soils for Civil Engineering Purposes SP 36", 1987.

5) Standard Specifications and Code of Practice for Road Bridges, Section II – Loads and Stresses (Revised Edition), IRC:6-2014.

6) "Guidelines for Use of Flyash in Road Embankments", IRC:SP:58-2001.

7) "Standard Specifications and Code of Practice for Road Bridges", Section VII – Foundations and Substructure (Revised Edition) IRC:78-2014.

8) "Code of Practice for Concrete Road Bridges" - IRC:112-2011.

9) "Guidelines for the Design and Construction of Geosynthetic Reinforced Embankments on Soft Subsoils" IRC:113-2013.

10) Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi - Rib Tensile Method - ASTM D 6637-11.

11) Standard Test Method for Determining Connection Strength Between Geosynthetic Reinforcement and Segmental Concrete Units (Modular Concrete Blocks) ASTM D 6638.

12) Standard Test Method for Determining the Shear Strength Between Segmental Concrete Units (Modular Concrete Blocks) ASTM D 6916-6c-2011.

13) Guidelines for the determination of the long-term strength of geosynthetics for soil reinforcement ISO/TR 20432:2007.

14) Geosynthetics - Wide-width Tensile Test ISO 10319:2008.

15) Certification (Quality Management System) - ISO-9001-2008.

C) Bibliography

1) "Reinforced Soil and its Engineering Applications", Swami Saran, I.K. International Pvt. Ltd., 2005.

2) "Designing with Geosynthetics", Robert M. Koerner, 6th Edition, Volumes 1 and 2, Xlibris Corporation, 2012.

3) "Geotechnical Earthquake Engineering", Steven K. Kramer, Pearson Education, 1996.

4) "Review of Design of Reinforced Earth Retaining Walls for Flyovers", Chonkar, R.R., Journal of the Indian Roads Congress, December, 2001.

5) "Review of Design Procedures of various Earth Retaining Structures including Seismic Effects", B.N. Sinha and Harjot Singh, Journal of the Indian Roads Congress, Paper 574, October-December 2011.

6) "Mechanically Stabilized Earth (MSE) Walls & Reinforced Soil Slopes (RSS) Indian Scenario: A Comprehensive Review", Rajeev Goel, Journal of the Indian Roads Congress, Paper No. 520 Vol. 67-1, 2007.

7) "Considerations for Reinforced Soil Walls in Urban Flyovers" Rajeev Goel Journal of the ING-IABSE Volume 43, Number 3, September 2013.

Annexure A0

(Refer Clause 5.1)

BEAM AND ANCHOR RODS TO ENSURE ADEQUATE LATERAL RESISTANCE FOR RS WALLS RESTING ON CONCRETE/ROCK SURFACES

Where RS walls on located on a concrete/rock surface it is difficult to emebd the wall to a depth of 400 mm. To ensure adequate lateral resistance arrangement shown below can be used. The arrangement comprises constructing a beam with anchors designed to withstand the lateral earth pressure arising due to the retained earth.

A 300 mm x 300 mm RCC beam may be provided over the concrete pavement and dowel bars be provided in that beam and anchored in the pavement below. This beam will have to be designed for the horizontal forces generated in the RE Wall.

Typical arrangement showing the beam and anchor arrangement is shown in **Fig. A0** below

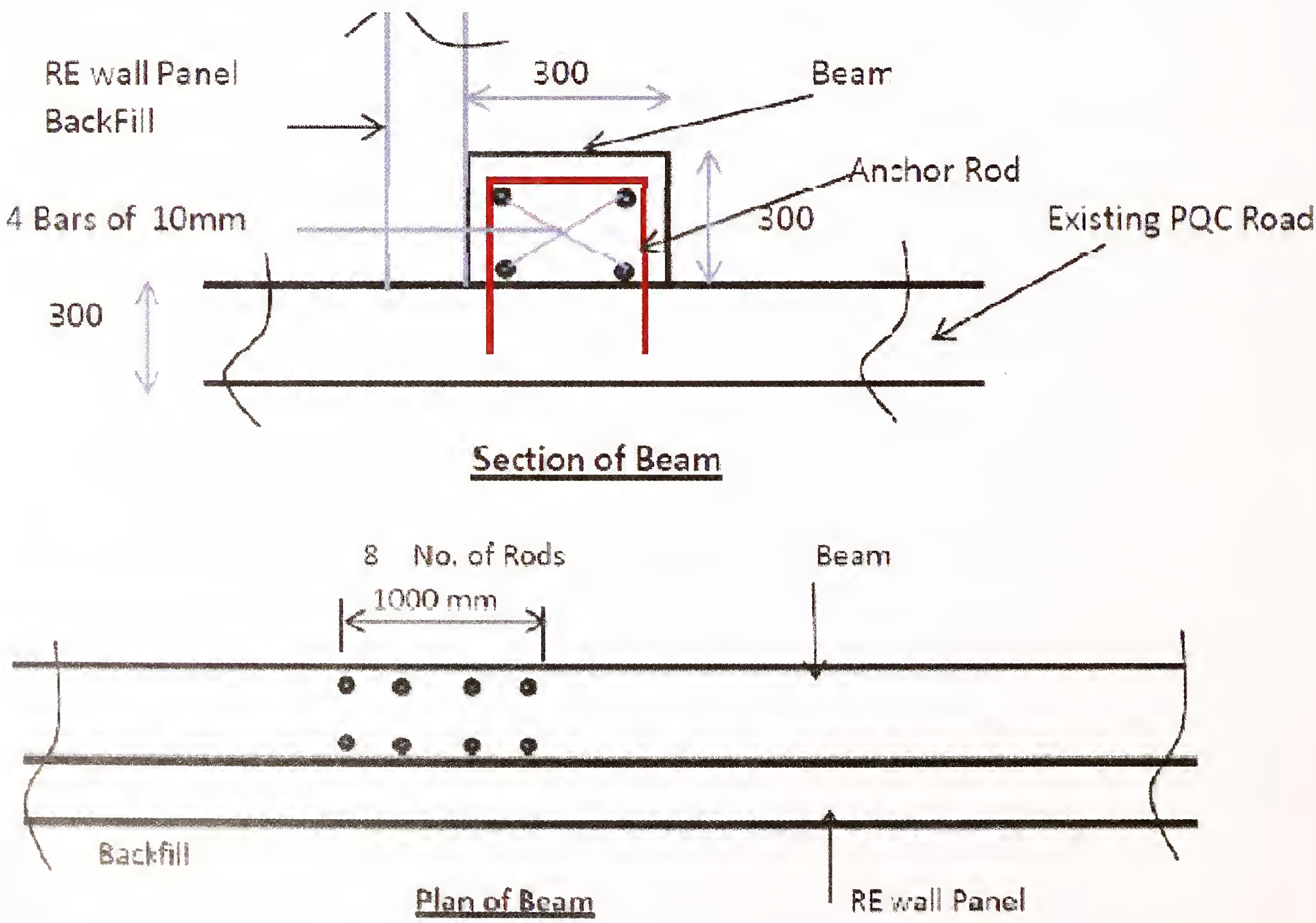


Fig. A0 Beam and Anchor Arrangement for RS Walls Resting on Concrete Surfaces

Annexure A1

(Refer Clause 5.1)

GROUND IMPROVEMENT METHODS FOR ENHANCING BEARING CAPACITY AND STABILITY

A1.1 Cohesionless Soils

Common methods of improving deep cohesionless soils are:

a) Heavy Tamping- It is the most economical way in which the ground can be improved up to a substantial depth, of say 15.00 m. The method consists of dropping a weight W through a distance H, thus imparting an energy of W*H per blow. The entire area can be covered by this method to improve the ground to desired shear strength. Initial trials need to be carried out to prove the efficacy of the method and also to fine tune the coverage of the entire area. This method should be only used in areas where resulting vibrations do not affect the utilities, underground structures and buildings. This method should not be considered in urban settings.

b) Blasting – Igniting a charge in a borehole to cause initial liquefaction and /or change in volume thus causing the soil to settle to a new compact relative density. This method can be adopted only when resulting vibrations do not affect structures and utilities. Therefore, this method should not be considered in urban areas or areas where utilities and underground structures are likely to be affected.

c) Vibrofloatation – A patented method where a "vibrofloat" compacts the soil by vibrations and the resulting column is filled with gravel.

d) In case the improvement is not required up to a substantial depth, replacing the soil or excavating the soil and backfilling it with reinforcement (commonly bi-axial geo-grids) would enhance the bearing capacity. Since the precise distribution of forces is not fully understood, it is recommended that, where possible, the maximum limit state tensile force T_r should be provided in one reinforcement layer. The reinforcement be restricted to a maximum 2 layers. The enhancement in Bearing Capacity can be judged by estimating the Bearing Capacity Ratio (BCR)/Pressure Ratio (p_r) which is defined Bearing Capacity in reinforced soil to that in the unreinforced soil at the same settlement. The tensile strength of the reinforcement can be calculated once the required BCR and the location of the reinforcement and the number of layers are known. It is not advisable to use this method of ground improvement for deep cohesive soils.

e) Use of Geocells to improve the Bearing Capacity of both cohesive and cohesionless soils.

  i) Geocells are three dimensional, axisymmetric, interconnected cells made up of geosynthetics that are used to improve the properties of base courses by providing lateral confinement to increase the strength and stiffness and reduce permanent surface deformation. Geocells are filled with soil/granular material or appropriately graded, forming a mattress for increased bearing capacity and manoeuvrability on loose or compressible subsoil.

  ii) Geocells are placed directly on the subsoil surface and propped open in an accordion-like manner. The opened honeycomb-like spaces are filled with cohesionless material and compacted using vibratory hand-operated plate compactors.

  iii) In terms of design, geocell systems mechanics are quite complex to assess. One can estimate the enhancement in Bearing Capacity using approaches cited in the literature.

  iv) Empirical equations are available to assess the efficacy of the geocell mattresses.

  v) Geocell mattresses are formed by adopting different construction techniques. Such mattresses are also referred to as Basal mattresess.

  This method can be used where ground improvement is required for shallow depths say upto 1.00 to 1.50 m.

f) In case the improvement is not required upto a substantial depth, replacing the soil or excavating the soil and backfilling it with reinforcement (commonly bi-axial geo-grids) would enhance the bearing capacity. Such reinforcement can be used in 2-3 layers upto a depth of about 1.00 to 2.00 m. The enhancement in Bearing Capacity can be judged by estimating the Bearing Capacity Ratio (BCR) which is defined Bearing Capacity in reinforced soil to that in the unreinforced soil. The tensile strength of the reinforcement can be calculated once the required BCR and the location of the reinforcement is known. It is not advisable to use this method of ground improvement for deep cohesive soils.

A1.2 Cohesive Soil

Deep cohesive soils can be also improved by:-

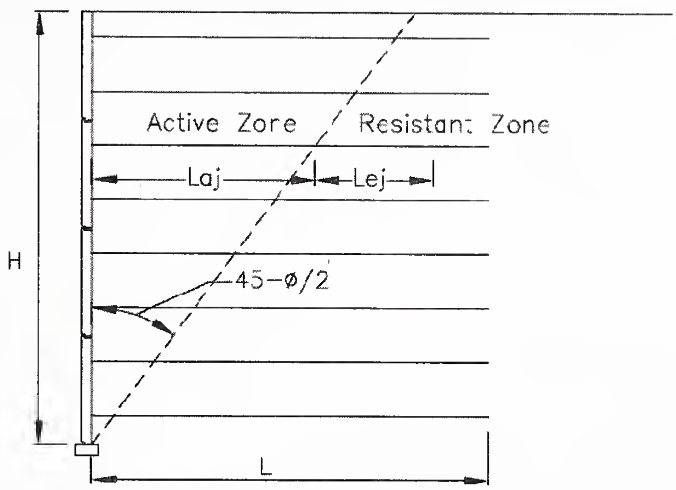
1. Geocell mattresses/Basal Mattresses as mentioned above
2. Stone Columns
3. Band drains with preloading
4. Chemical stabilisation

For details of ground improvement techniques, HRB SOAR No. 13, HRB SOAR No. 14 and IRC:113-2013 shall be referred.

Annexure A2

(Refer Clause 5.2)

ADHERENCE CHECK FOR THE REINFORCEMENT



Laj = Length of reinforcement in active zone
Lej = Length of reinforcement in resistant zone

Fig. A2.1 Parameters for Adherence Check (BS:8006)

The length of the reinforcement is the length provided in the active zone plus that in the passive zone

For the length of the reinforcement in the Passive or resistant zone is checked from the following equation which checks the adherence of the reinforcement.

$$P_j \geq \frac{T_j}{\dfrac{\mu\ L_{ej}\ (f_{fs}\ \gamma_1\ h_j\ + f_f W_s)}{f_p f_n} + \dfrac{a'_{bc}\ c\ L_{ej}}{f_{ms} f_p f_n}}$$

where,

- P_j is the horizontal width of the top and bottom faces of the reinforcement element at the j^{th} layer per metre run
- T_j is the maximum tension as evaluated from the equation in section 5.2.a above
- f_{fs} is the partial load factor applied to soil self-weight for the same load combination as T_j refer Section 5.3 – load combinations
- f_f is the partial load factor applied to surcharge load for the same combinations as T_j – Section 5.3 – load combinations
- μ is the coefficient of friction between the fill and the reinforcing element
- W_s is the surcharge due to dead loads only
- f_p is the partial factor for reinforcement pull out resistance – 1.3
- f_n is the partial factor for economic ramifications of failure – 1.1
- a'_{bc} is the adhesion coefficient between the soil and the reinforcement
- c' is the cohesion of the soil under effective stress conditions

f_{ms} is the partial safety factor applied to c' may be taken as 1.6

h_j – Depth of the j^{th} reinforcement below top of the structure.

For inextensible reinforcement the adherence should be checked beyond line 1 and 2 and checked with reinforcement tension at each of these points. The adherence capacity of T_j of each layer of reinforcement can be calculated using the equation given below:-

$$T_j \leq ((2B\mu)/(f_p f_n)) * \int_{L-Laj}^{L} f_{fs}\sigma_v (x)\, dx$$

where,

T_j is the maximum tensile force resisted by the j^{th} layer 2 indicates two faces of the reinforcement

B is the width of the reinforcement

μ is the value of coefficient of friction

f_p is the partial factor for pull out resistance (typical 1.3)

f_n is the partial factor for ramifications of failure (1.1)

f_{fs} is the partial load factor for different combinations refer Section 5.3

σ_v is the vertical stress along the length x of the reinforcement.

L is the length of the reinforcement

L_{aj} is the length of the reinforcement beyond the tension zone.

Wedge Stability Checks

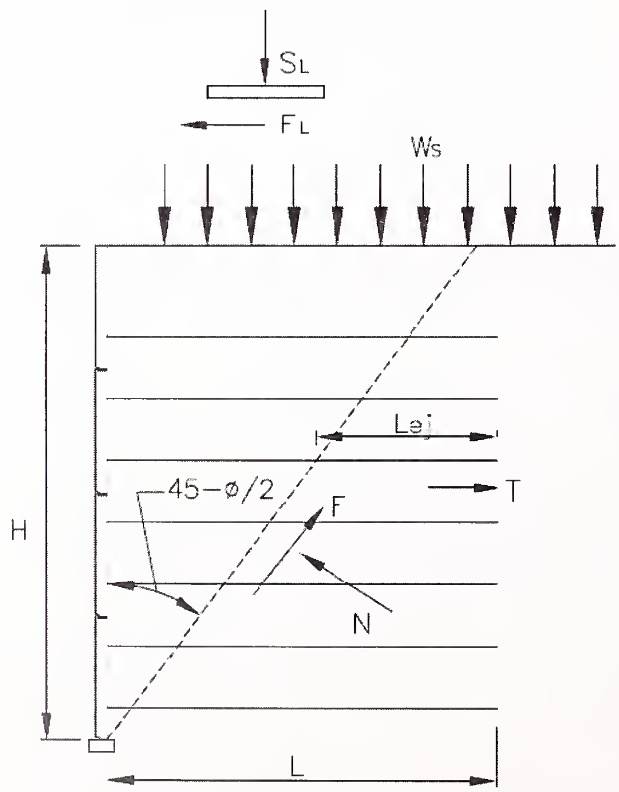


Fig. A2.2 Forces for Wedge Stability Check (BS:8006)

The stability of the wedge at each level of reinforcement should be checked. Wedge stability ensures that friction and tensile resistance due to the bond of the reinforcement is sufficient to resist the loads tending to cause the movement.

For reinforced soil the total resistance of the layers of elements anchoring the wedge is satisfied by

$$\sum_{j=1}^{n\ \text{layers}} [T_{Dj}/f_n] \geq T$$

or

$$\sum_{j=1}^{n\ \text{layers}} \{[(P_j L_{ej})/(f_p f_n)] * [(\mu f_{fs}\ \gamma\ h_j) + (\mu\ f_f W_s) + (a'_{bc} * c'/f_{ms})]\} \geq T$$

Lesser of the two values should be used in the summation to assess the stability

- T_{Dj} is the design strength of the reinforcement at the j^{th} layer
- T_j is the maximum tension as evaluated from the equation in Section 5.2.a above
- f_{fs} is the partial load factor applied to soil self-weight for the same load combination as T_j refer Section 5.3 – load combinations
- P_j is the horizontal width of the top and bottom faces of the reinforcement element at the j^{th} layer per metre run
- f_f is the partial load factor applied to surcharge load for the same combinations as T_j – Section 5.3 – load combinations
- W_{sv} is surcharge due to dead loads only
- f_p is the partial factor for reinforcement pull out resistance – 1.3
- f_n is the partial factor for economic ramifications of failure – 1.1
- a'_{bc} is the adhesion coefficient between the soil and the reinforcement
- c' is the cohesion of the soil under effective stress conditions
- f_{ms} is the partial safety factor applied to c' may be taken as 1.6

Annexure A3

(Refer Clause 5.3)

SEISMIC FORCES TO BE CONSIDERED IN EXTERNAL & INTERNAL STABILITY

(Refer FHWA-NHI-00-043)

This annexure of the guidelines provide for seismic analysis of RE wall as provided by FHWA-NHI-00-043 approach. Designer can adopt seismic analysis as per AFNOR and other FHWA guidelines as an alternative. As reinforced earth wall is more flexible compared to a relatively rigid R.C.C retaining wall the inertial effects due to horizontal acceleration is considered on part of reinforced volume of earth and vertical acceleration is neglected. Also only 50 percent of dynamic increment on the earth pressure of retained fill is considered. No dynamic increment on earth pressure due to live load surcharge is considered unlike gravity type retaining walls. Salient features of seismic analysis are as under.

Peak horizontal acceleration is selected based on the seismic zone (design earthquake) given as A, Acceleration co-efficient (refer IS:1893-2002).

I. Maximum acceleration developed in the wall is obtained from equation

$$A_m = (1.45 - A)\,A \qquad \text{(Eqn. A3.1)}$$

where,

A = maximum ground acceleration co-efficient

A_m = maximum wall acceleration co-efficient

II. Horizontal inertia force on block of soil mass (shaded area in Fig. 1) P_{IR} and incremental thrust due to retained fill P_{AE} are obtained by the following formulae:

$$P_{IR} = 0.5\,A_m \times \gamma \times H^2 \qquad \text{(Eqn. A3.2)}$$

$$P_{AE} = 0.5\,\Delta K_{AE} \times \gamma \times H^2 \qquad \text{(Eqn. A3.3)}$$

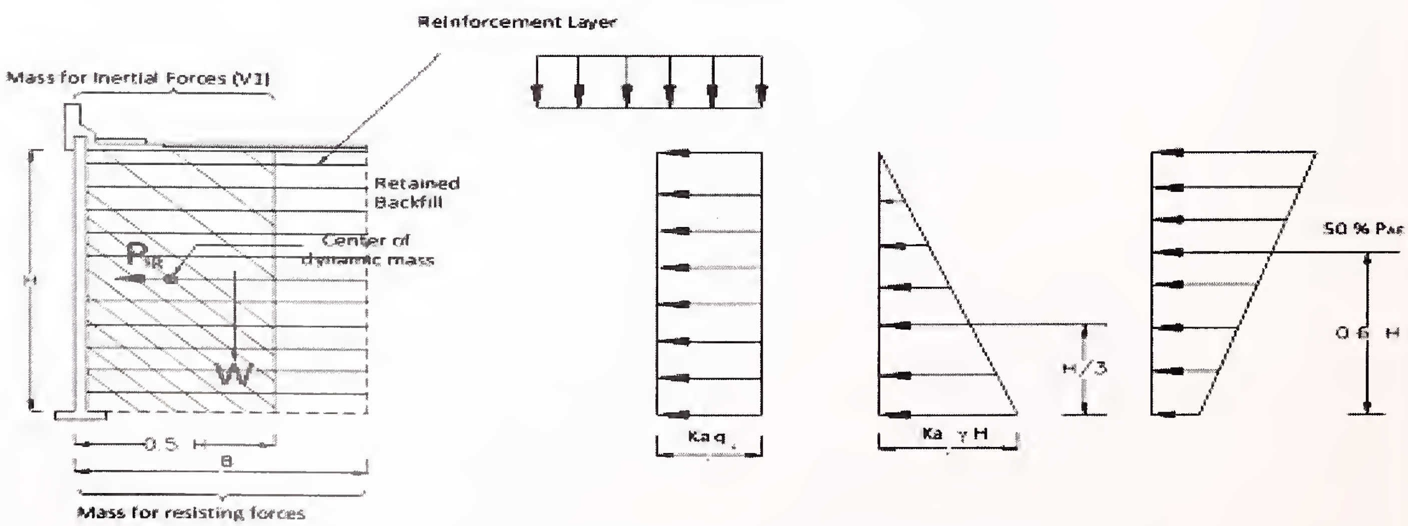


Fig. A3.1 External Forces on Wall (Horizontal Backfill)

ΔK_{AE} is obtained by calculating KAE by putting β = 0 from the Eq. (4) deducting there from values of K_{AE} by putting β and λ both zero in Eq. (A3.4)

$$K_{AE} = K_{AE} = \frac{\cos^2(\Phi - \lambda - 90 + \alpha)}{\cos\lambda \cos^2(90-\alpha)\cos(\beta + 90 - \alpha + \lambda).\left(1 + \frac{\sqrt{\sin(\Phi+\beta)\sin(\Phi-\lambda-\beta)}}{\sqrt{\cos(\beta+90-\alpha+\lambda)\cos(\beta-90+\alpha)}}\right)^2} \quad \text{(Eq. A3.4)}$$

where,

β = earth slope, α = wall slope (see **Fig. 1**), Φ = soil friction angle.

Both these forces are horizontal and added to the horizontal static forces. Full inertia force on the part of reinforced soil and only 50 per cent of dynamic thrust due to earth pressure by retained fill are considered for stability analysis. **Fig. 1** shows the static and dynamic forces for horizontal backfill.

In case of sloping backfill the inertia force and dynamic earth pressure increment are obtained on the basis of increased height of H_2, determined as follows. **Fig. A3.2** shows force diagram for seismic condition.

$$H_2 = H + \frac{\tan\beta \; 0.5H}{1 - 0.5 \tan\beta} \quad \text{(Eqn. A3.5)}$$

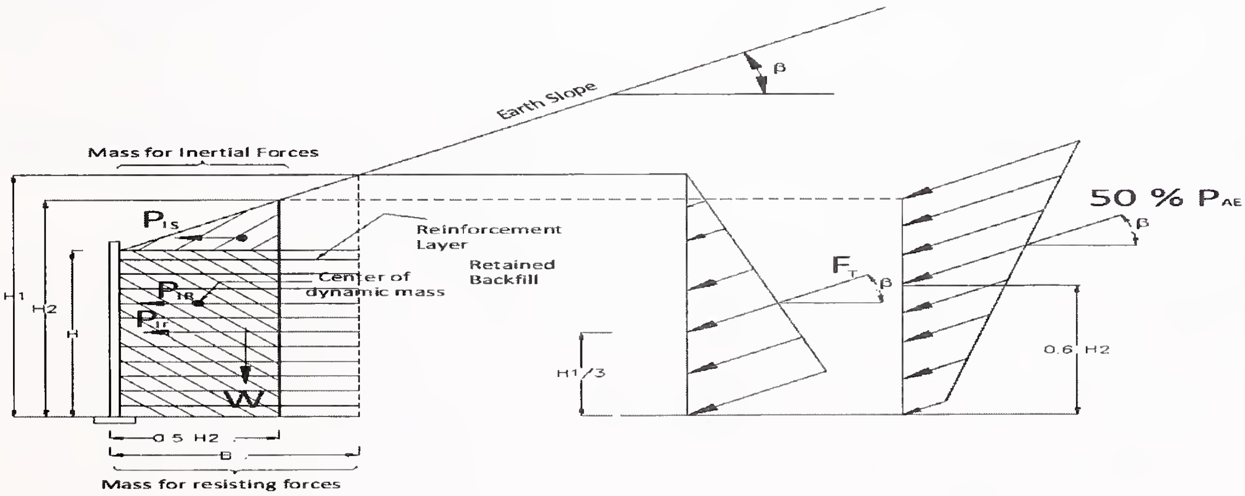


Fig. A3.2 External Forces on Wall (Inclined Backfill)

ΔK_{AE} Shall be obtained as difference of values of K_{AE} by Eq. 4 and value of K_{AE} obtained by putting β = 0 in Eq. 4. Force shall be obtained by the following formulae:

$$P_{IR} = P_{ir} + P_{is} \quad \text{(Eqn. A3.6)}$$

$$P_{ir} = 0.5\, A_m\, \Upsilon\, H_2\, H, \quad \text{(Eqn. A3.7)}$$

$$P_{is} = 0.125\, A_m\, \Upsilon\, H_2^2 \tan\beta \quad \text{(Eqn. A3.8)}$$

$$P_{AE} = 0.5\, \Upsilon\, H_2^2\, \Delta K_{AE} \text{ (Sloping backfill)} \quad \text{(Eqn. A3.9)}$$

Where P_{ir} is the inertia force caused by acceleration of the reinforced backfill and P_{is} is the inertia force caused by acceleration of sloping soil surcharge above the reinforced backfill where width of mass contributing to P_{IR} is equal to $0.5\,H_2$.

The analysis is completed by obtaining sliding, overturning and bearing stability of wall considering all forces such as F_T, P_{IR}, and 50 percent of P_{AE} and weight of reinforced soil block as detailed for static/dynamic analysis of normal retaining wall. Inclined slope is not supposed to have live load surcharge and will need no consideration.

A3.1 Internal Stability

The internal stability shall be analyzed as follows:

Internal failure can occur mainly in two ways.

- Failure by excessive elongation or breakage of tensile element
- Failure by pull-out of tensile element

The following steps are needed for analyzing internal stability -

1) Selection of type of reinforcement
2) Location of critical failure surface
3) Maximum tensile force at each reinforcement level, static and dynamic
4) Maximum tensile force at the connection to the facing
5) Pull-out capacity at each reinforcement level

Based on research and experiments the critical failure surface has been established to depend on type of reinforcements whether the same is extensible or inextensible. **Figs. A3.3(a) and A3.3(b)** shows these two cases giving relevant details. The maximum tensile forces T_{max} in the element occurs along the line shown in **Figs. A3.3 (a) & A3.3 (b)**. The anchorage and strength (shear and bending) of the face panel are also required to withstand the forces to which these are subjected to from the earth pressure of the backfill.

The failure of tensile element by rupture can take place at the maximum tensile stress point and solely is a function of its tensile strength. The tensile force is a function of earth pressure due to self-weight of earth above the element and other super imposed loads, may be live or dead load, and coefficient of earth pressure. The pull out resistance depends on the embedment length of tensile element beyond the failure line (effective length), the inter face friction coefficient and the vertical loads (resultant pressure. of all loads above the level of element). The first step is evaluation of T_{max} in the tensile element at particular level. Then the strength of the element after allowing for various reduction factors is checked whether the same can safely withstand T_{max}. Then the pullout resistance of the tensile element in the resistant zone is evaluated to check whether the element can withstand T_{max} without getting pulled out. The FoS is also evaluated against strength of reinforcement as well as pullout resistance. Thus internal stability of the wall at this level of reinforcement is ensured. Similar checking is done at levels of all the tensile elements to ensure safety of the R.E Wall. These steps are mentioned in Para A3.1.1, A3.1.2 & A3.1.3.

A3.1.1 *Calculation of Maximum Tensile forces in the Reinforcement Layers*

At each reinforcement level the horizontal stress σ_H along the potential failure line from the weight of the reinforced fill γ_r Z (Z = elevation of reinforcement) plus uniform surcharge load q is obtained as

$$\sigma_H = K_a \sigma_v \qquad \text{(Eqn. A3.10)}$$

Where, $\sigma_v = \gamma_r Z + q$, K_a = Active Earth Pressure Coefficient. The horizontal stress is multiplied by the area of the wall affecting the reinforcement known as tributary area A_t and this leads to T_{max}

$$T_{max} = \sigma_H A_t \qquad \text{(Eqn. 3.11)}$$

K_a should be evaluated for extensible and inextensible reinforcement based on the approach given in Section 5.2

A3.1.2 *Internal stability with respect to rupture (breakage of reinforcement)*

The allowable reinforcement strength is obtained from its ultimate strength after allowing for various reduction factors and applying a factor of safety. For wall to be safe against rupture of reinforcement the allowable strength is required to be more than T_{max} at the level of the reinforcement then

$$T_{max} \leq \frac{T_{ultimate}}{RF_D RF_{ID} RF_C\, R_c} \qquad \text{(Eqn. A3.12)}$$

where, RF_D is durability reduction factor, RF_{ID} is reduction factor due to installation damage, RF_C is the reduction factor due to creep and coverage ratio $R_c = b/S_h$, with b the gross width of reinforcing element and S_h is the center-to-center horizontal spacing between reinforcements (in case of Geogrids R_c = 1).

A3.1.3 *Internal Stability with respect to pull-out failure:*

The tensile element to be safe against pull out the following criteria is satisfied.

$$T_{max} \leq \frac{1}{FS_{po}} F^{*} \gamma Z_p L_e C R_c \alpha \qquad \text{(Eqn. A3.13)}$$

where,

FS_{po}	=	Safety factor against pullout ≥ 1.5.
T_{max}	=	maximum reinforcement tension.
C	=	2 for strip, grid and sheet type reinforcement.
α	=	scale correction factor.
F*	=	Pullout resistance factor
R_c	=	Coverage ratio
Z_p	=	Overburden pressure, including distributed dead load surcharge, neglecting live load (traffic load)
L_e	=	Length of embedment in the resistant zone.

If the criterion is not satisfied for all reinforcement layers, the reinforcement length has to be increased and/or reinforcement with a greater pullout resistance per unit width must be used, or the vertical spacing may be reduced which would reduce T_{max}.

The total length of reinforcement, L, required for internal stability is determined from.

$$L = L_a + L_e \qquad \text{(Eqn. A3.14)}$$

Where, L_a is obtained based on the relationship as drawn from the **Fig. (A3.3)**.

For RE wall with extensible reinforcement, vertical face and horizontal backfill:

$$L_a = (H - z)\tan\frac{(45-\emptyset)}{2} \qquad \text{(Eqn. A3.15)}$$

Where, z is the depth to the reinforcement level,

For walls with inextensible reinforcement from the base up to H/2:

$$L_a = 0.6\,(H - z) \qquad \text{(Eqn. A3.16)}$$

For upper half of wall $L_a = 0.3H$.

A3.1.4 *Internal stability under seismic condition*

Seismic force induces an inertial force P_I acting horizontally, in addition to the existing static forces. This force causes dynamic incremental resulting in increase in the maximum tensile forces T_{max} in the reinforcements.

It is assumed that the maximum tensile force line shown in **Figs. A3.3(a) & A3.3(b)** does not change during seismic condition. The dynamic increment acts on the weight of full wedge (backfill in the active zone) shown hatched in **Figs. A3.3(a) & A3.3(b)** and this total incremental inertia force P_I is distributed among the different reinforcements proportionally (T_{md}) to their "resistant area" (L_e) per unit wall width basis.

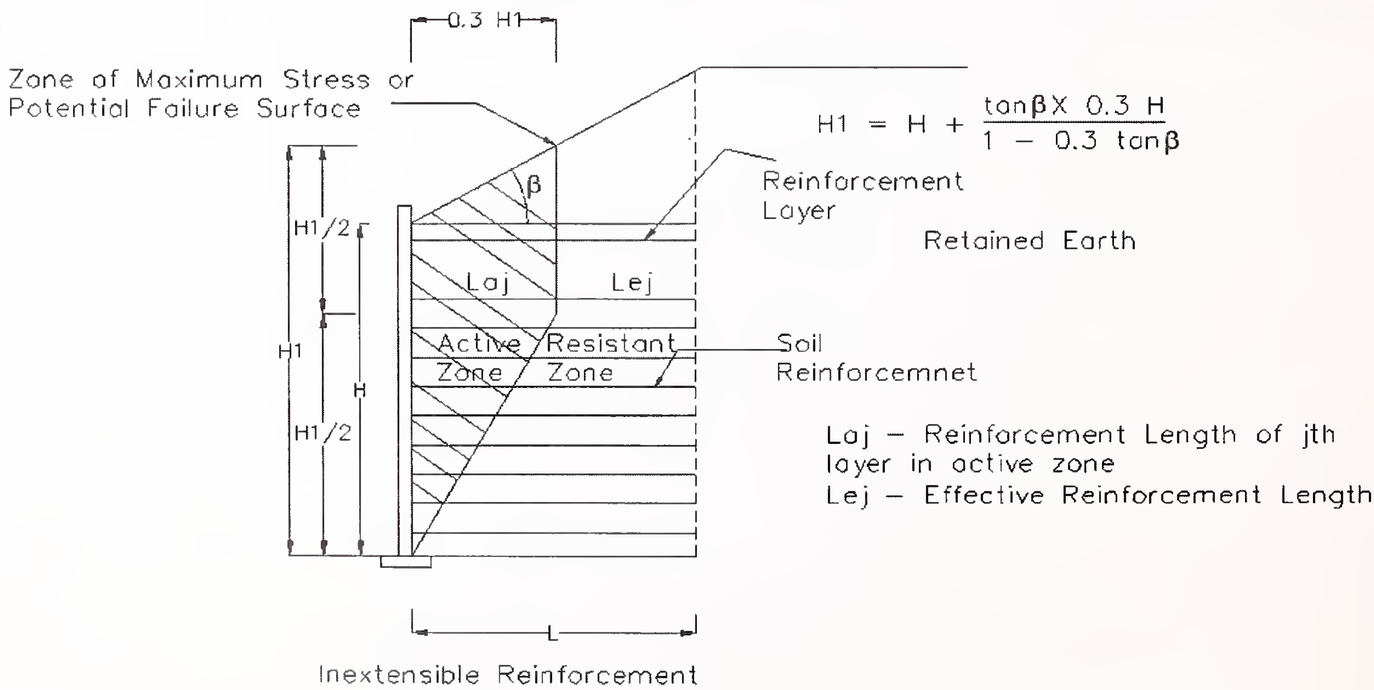


Fig. A3.3(a) Location of Potential Failure Line in Inextensible Reinforcement

This dynamic induced T_{md} is added to static T_{max} in particular reinforcement to get the total tensile force (T_{total}) in that reinforcement. This T_{Total} is considered for the internal stability with respect to breakage and pull out of the reinforcement in seismic condition considering seismic factor of safety of 75 percent of the minimum allowable static factor of safety. The following procedure is followed.

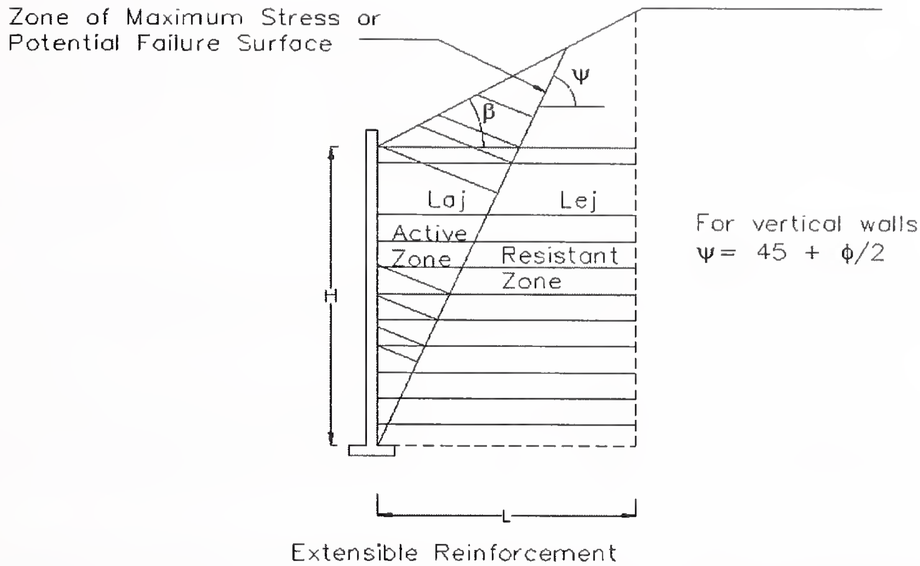


Fig. A3.3(b) Location of Potential Failure Line in Extensible Reinforcement

1) Calculate maximum acceleration in the wall and the force P_I per unit width acting above the base:

$$P_I = A_m W_A \qquad \text{(Eqn. A3.17)}$$

$$A_m = (1.45 - A)\,A$$

Where, W_A is the weight of the active zone (Shaded area in **Figs. 3(a) & 3(b)**), A is the maximum ground acceleration coefficient and A_m is maximum wall acceleration coefficient

2) Calculate the dynamic increment Tmddirectly induced by the inertia force P_I in the reinforcements by distributing P_I in the different reinforcements proportionally to their "resistant area" (L_e) on a load per unit wall width basis. This leads to:

$$T_{md} = P_I \frac{L_{ei}}{\sum_{i=1}^{n} L_{ei}} \qquad \text{(Eqn. A3.18)}$$

The multiplier of P_I in Eq. A3.18 is the resistant length of the reinforcement at level divided by the sum of the resistant length for all reinforcement levels.

3) T_{max} for static condition is already explained.

4) The maximum T_{Total} is

$$T_{total} = T_{max} + T_{md} \qquad \text{(Eqn. A3.19)}$$

For steel reinforcement stability is calculated as

$$T_a \le \frac{T_{Total}\,(0.75)}{R_c} \qquad \text{(Eqn. A3.20)}$$

For geosynthetic reinforcement rupture, the reinforcement must be designed to resist the static and dynamic components of the load as follows:

For the static component:

$$T_{max} \leq \frac{S_{rs} R_c}{0.75\ RF\ FS} \qquad \text{(Eqn. A3.21)}$$

For dynamic component, where the load is applied for a short time, creep reduction is not required and therefore,

$$T_{md} \leq \frac{S_{rt} R_c}{0.75\ FS\ RF_D RF_{ID}} \qquad \text{(Eqn. A3.22)}$$

Therefore, ultimate strength of geosynthetic reinforcement required is

$$T_{ult} = S_{rs} + S_{rt} \qquad \text{(Eqn. A3.23)}$$

Where S_{rs} is the reinforcement strength per unit width needed to resist the static component of load and S_{rt} the reinforcement strength needed to resist the dynamic or transient component of load.

For pullout under seismic loading, for all reinforcements, the friction coefficient F* should be reduced to 80 percent of the static value leading to:

$$T_{Total} \leq \frac{P_r R_c}{0.75\ FS_{PO}} = \frac{C\ (0.8\ F^*)}{0.75\ X\ 1.5} \gamma Z' L_e R_c \alpha \qquad \text{(Eqn. A3.24)}$$

Annexure A4

(Refer Clauses 5.3)

RS WALL OF COMPLEX GEOMETRICS

1. Broken Slope

This geometry is regarding change in slope of back fill near the RE Wall. Two situations can arise at site depending on the distance from the wall where the slope changes. Where, however, the change in slope takes place beyond 2H (Height of the wall) all computation of earth pressure co-efficient etc. shall be done taking surcharge slope without any modification.

There could be two situations, first where the slope changes within the length of reinforcement and second where the slope changes beyond the reinforcement length but before 2H distance from the wall. Both these cases are covered in following sketches. In both cases the surcharge angle (β) is required to be modified by angle (I) and all computation of earth pressure co-efficient etc. are done using modified surcharge angle. Rest of the computation is same as for wall without broken slope.

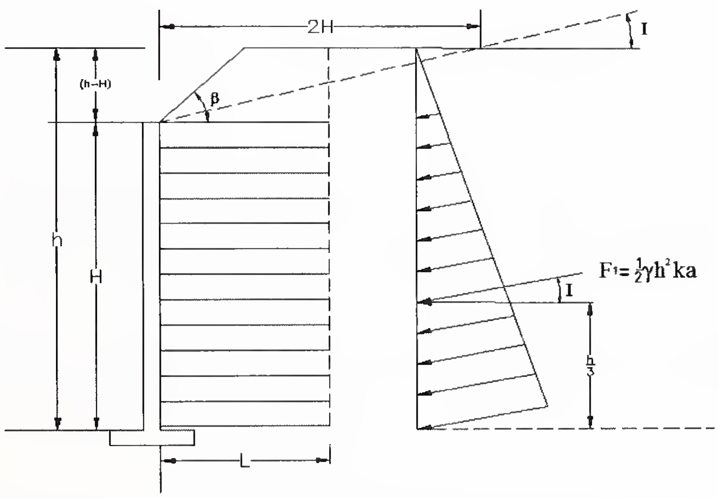


Fig. A4.1 Where Slope Changes within Reinforcement Length

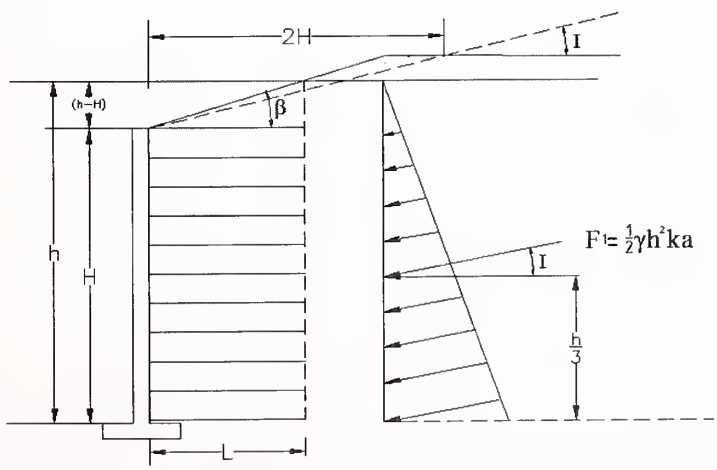


Fig. A4.2 Where Slope Changes Beyond Reinforcement Length and within 2H

K_a is obtained for inclined surcharge with modified slope I (i.e. δ = β = I).

2. Back to Back Wall

The back to back wall are those which are near each other such that the reinforced portion of wall come within the active failure wedge which might form beyond the reinforced soil volume. As the walls comes closer the earth pressure by the back fill on the reinforce block decreases and at a point when the reinforcing element of the two wall overlap by $0.3H_2$ (where H_2 is the height of the shorter wall) the earth pressure becomes zero. In a position where the wall is at a distance such that $D \geq H_1 \tan(45° - \phi/2)$ where there is no interference of active wedge with the reinforced volume, they will behave as independent walls and stability shall be obtained accordingly. In between these two positions the earth pressure shall be linearly interpolated for analyzing sliding and over-turning.

The sketches given below show these situations.

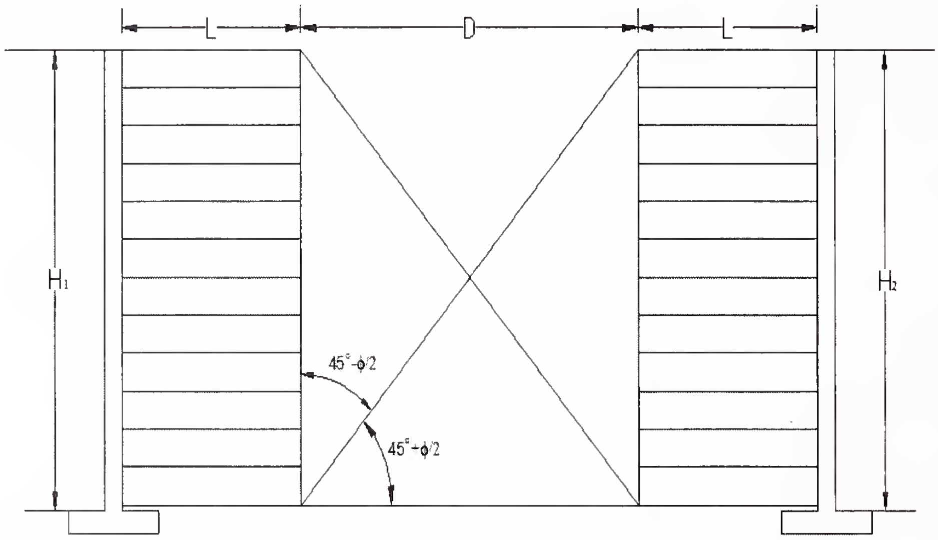


Fig. A4.3 $D \geq H_1$ (tan 45°- $\phi/2$), the Walls shall be Independent

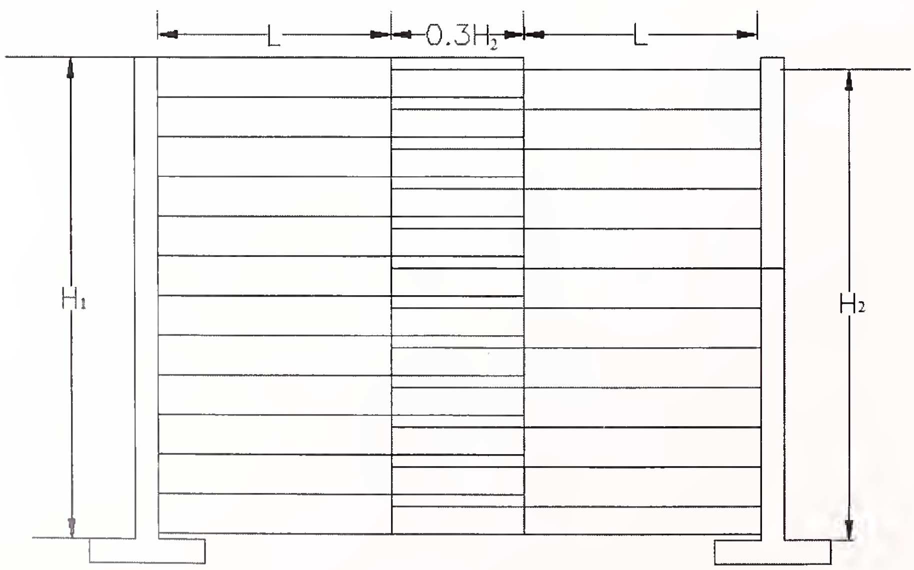


Fig. A4.4 Overlap is 0.3H2 and more, no Earth Pressure from Back Fill

H_1 is taller wall, H_2 is shorter wall
Back to Back Wall

3 Superimposed Walls

Total height of embankment can be covered by providing stepped RE Wall each covering one part of height. Such arrangement is known as Superimposed Wall where the design of RE Wall may fall in following categories depending on offsets.

Where the offsets "D" are:-

i) $D \leq \left(\frac{H1+H2}{20}\right)$, the wall shall be designed as one single wall of height $H = H_1 + H_2$ (**Fig. 4.5**) where L_1' is more than or = $0.7H_1$ & L_2' is more than or = $0.6\,(H_1 + H_2)$

ii) $D \geq H_2 \tan(90° - \phi r)$ the wall shall be designed as independent walls without any effect on the lower wall. The surcharge load on the lower wall $\sigma_l = 0$.

iii) $\left(\frac{H1+H2}{20}\right) < D \leq H_2 \tan\left(45° - \frac{\phi r}{2}\right)$, the wall shall be designed as superimposed wall. Internal line of failure shall be shifted by D' as shown in **Fig. 4.6**. The surcharge load due to upper wall on lower wall shall be $\sigma_l = \gamma H_1$.

iv) $H_2 \tan\left(45° - \frac{\phi r}{2}\right) < D \leq H_2 \tan(90° - \phi r)$. The wall internal stability tensile force lines are as indicated in figure and vertical pressure as given in **Fig. 4.8**.

These relationships are somewhat empirical and geometrically derived.

The global stability of the system (slip circle failure through backfill) has to be checked considering all the walls in different tires together for all cases covered by (i), (ii), (iii) and (iv) above. The upper wall is considered as a surcharge for the lower wall in computing bearing pressure. For location of the failure line **Figs. 4.5, 4.6 and 4.7** are referred.

The vertical pressure for internal design is considered as given in **Fig.4.8** where $H_2 \tan(45° - \phi/2) \leq D < H_2 \tan(90° - \phi_r)$.

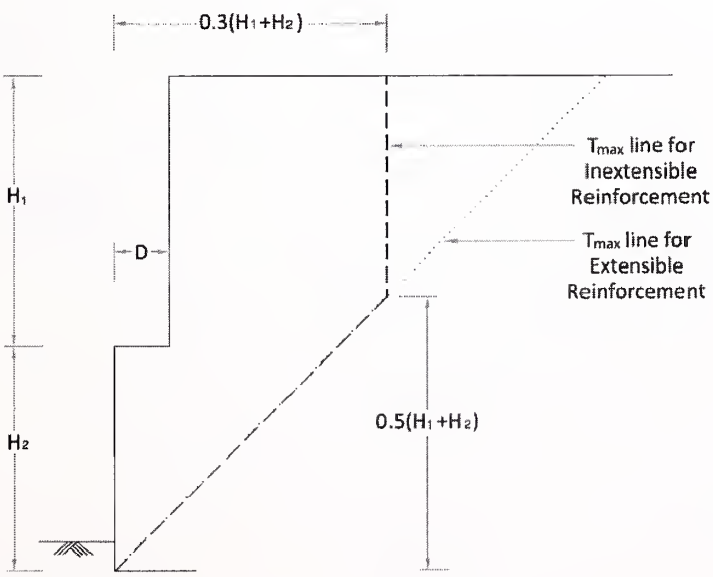
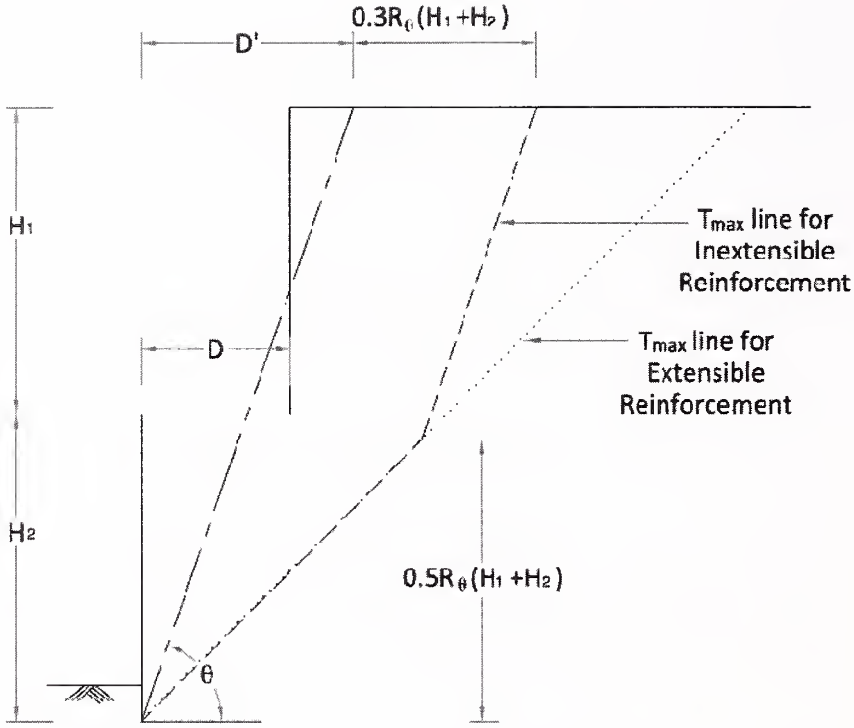


Fig. A4.5 $D \leq \frac{H_1+H_2}{20}$,



$$D' = 2D\frac{H_1}{H_1+H_2} \qquad R\theta = \frac{\phi-\phi_r}{90-\Phi\theta_r}$$

Fig. A4.6 $\frac{H_1+H_2}{20} < D \leq H_2 \tan\left(45-\frac{\phi_r}{2}\right)$

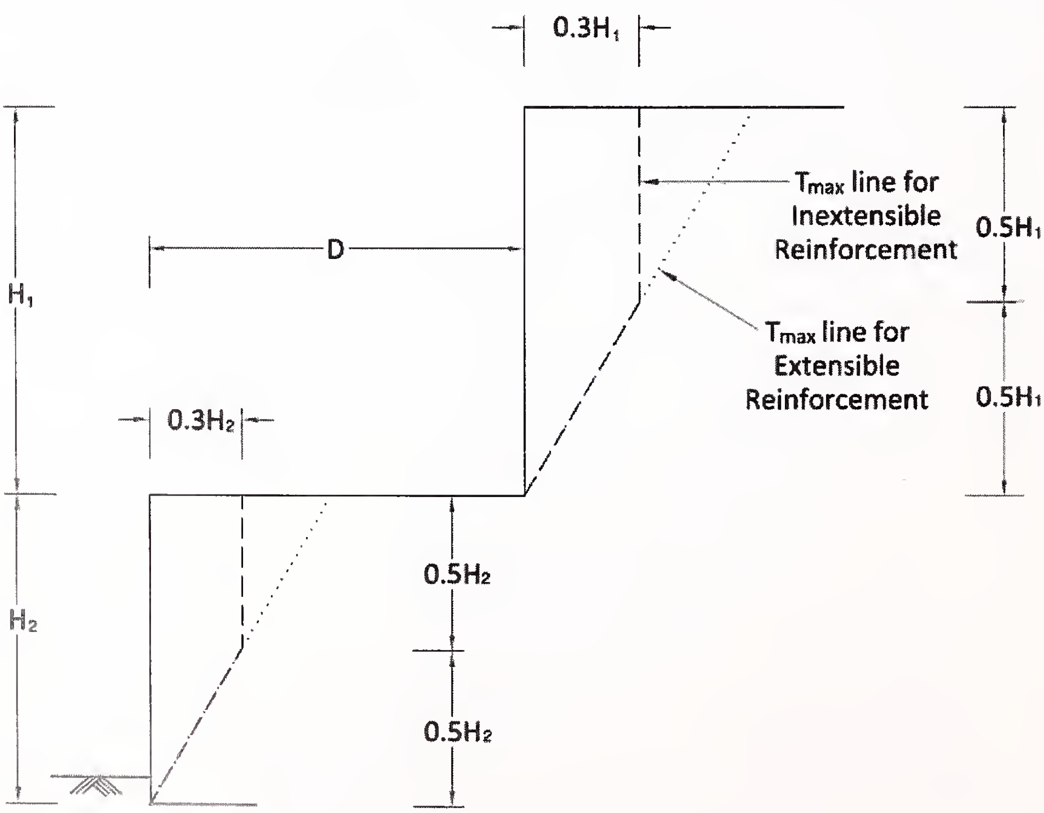
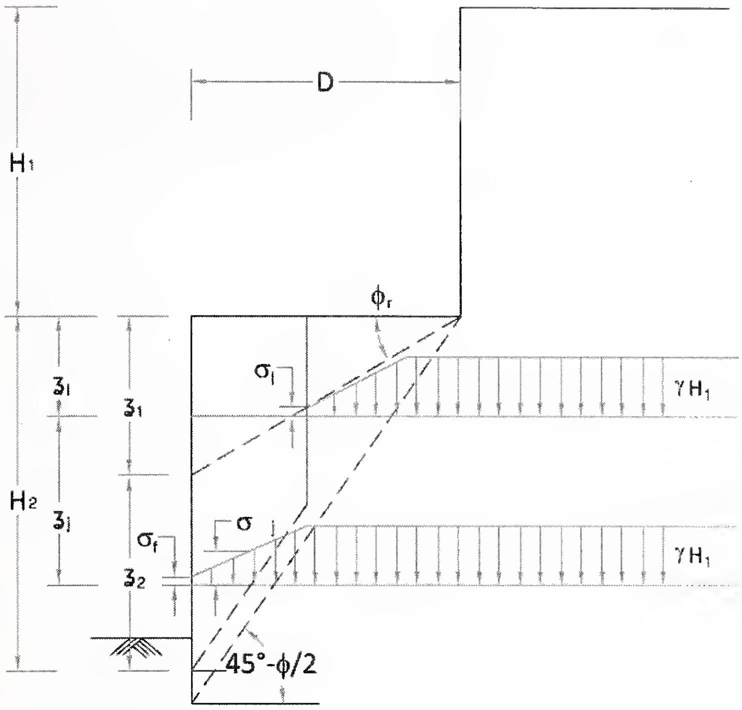


Fig. A4.7 $D > H_2 \tan\left(45^\circ - \frac{\phi_r}{2}\right)$



$$z_1 = D\tan\phi r,\ z_2 = D\tan\left(45 + \frac{\phi r}{2}\right),\ \sigma_f = \frac{z_j - z_1}{z_2 - z_1}\gamma H_1$$

Fig. A4.8 $H_2\tan\left(45 - \frac{\phi_r}{2}\right) < D \leq H_2\tan(90 - \Phi r)$

Annexure A5

Typical Calculations for Reinforced Soil Wall (Static)

Title	Hand Calculations for modular block wall of height 10.75 m
Reference	BS 8006-1:2010
Date :	
Designed by :	
Checked by :	
Approved by :	

Design Input Parameters

Reinforced Soil Data

Angle of Internal friction	Φ_1	32°
Unit wt	γ_1	18.5 kN/cu.m

Retained Backfill Soil Data

Angle of Internal friction	Φ_2	30°
Unit	γ_2	18.5 kN/cu.m

Foundation Soil Data

Cohesion	C_3	0 kPa
Angle of Internal Friction	Φ_3	30°
Unit wt	γ_3	18.5 kN/cu.m

Crash Barrier Data

Strip load due to crash barrier	Q	15.45 kPa
Live Load	Ql	23 kPa

Live Load should be considered as per provisions of IRC:78-2014

Water table is considered below the influence zone.

General Shear Failure is considered.

COMPUTATION OF EXTERNAL STABILITY FOR 10.75 m HIGH WALL BY STATIC ANALYSIS

Coefficient of active earth pressure (k_a):

For reinforced soil :

$k_a = (1-\sin\emptyset)/(1+\sin\emptyset)$

k_{a1} = 0.307 $\emptyset_1$ = 32 °

Wall batter θ = 4.23 °

For retained backfill soil :

k_{a2} = 0.333

Mechanical wall height H = 10.75 m i.e 10.15 m +0.6 m road crust

Length of reinforcement L = 7.60 m

Foundation properties :

Minimum embedment depth D_m = 1 m

Unit weight of foundation soil γ_f = 18 **kN/m³**

Summary of partial factors to be used

Partial factors		**Ultimate limit state**	**Serviceability limit state**
Soil material factors :	to be applied $\tan\emptyset'_p$ (f_{ms})	1.0	1.0
	to be applied C' (f_{ms})	1.6	1.0
	to be applied C_u (f_{ms})	1.0	1.0
Soil/reinforcement interaction factors :	Sliding across surface of reinforcement (f_s)	1.3	1.0
	Pullout resistance of reinforcement (f_p)	1.3	1.0
Partial factors of safety	Foundation bearing capacity : to be applied to q_{ult} (f_{ms})	1.4	NA
	Sliding along base of the structure or any horizontal surface where there is soil-soil contact (f_s)	1.2	NA

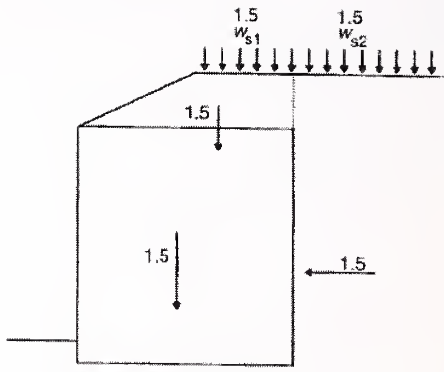
Partial load factors for load combinations associated with walls			
Effects	**combinations**		
	A	**B**	**C**
Mass of the reinforced soil body (f_{fs})	1.5	1.0	1.0
Mass of the backfill on top of the reinforced soil wall (f_{fs})	1.5	1.0	1.0
Earth pressure behind the structure (f_{fs})	1.5	1.5	1.0
Traffic load: On reinforced soil block (f_q)	1.5	0	0
Behind reinforced soil block (f_q)	1.5	1.5	0

CASE :A

LOADS

Self weight of Reinforced Soil Wall

		Lever Arm		
$V_1 = \gamma_1 * H * L * f_{fs}$				
$= 18.5*10.75*7.6*1.5$	**kN/m**	X1 =	3.8	m
= **2267.18**	**kN/m**			



a) External and internal stability

Strip load due to Crash Barrier

$V_2 = Q \times b * f_{fs}$

width of the Strip Load		b =	1.6	m
$V_2 = 15.5*1.6*1.5$	**kN/m**	X2 =	0.8	m
= **37.08**	**kN/m**			

Vertical load due to Live Load :-

$V_3 = Ql \times L * f_{fs}$

$= 23*7.6*1.5$	**kN/m**	X3 =	3.8	m
= **262.2**	**kN/m**			

Resultant Vertical Load :-

$R_v = V_1+V_2+V_3$

= **2566.46 kN/m**

Horizontal Forces

Earth pressure behind reinforced soil block

$P_1 = 1/2 * k_{a2} * \gamma_2 * H^2 * f_{fs}$

$= 0.5*0.333*18.5*10.75*10.75*1.5$

= **534.22 kN/m**

Earth Pressure due to Live Load : $P2 = ka_2 * Ql * H * f_{fs}$

$= 0.333*23*10.75*1.5$

= **123.56 kN/m**

Check for Sliding along the base CASE:B (Critical case for sliding)

For long term stability where there is soil to soil contact at the base of the structure

$$f_s R_h \leq R_v(\tan \varnothing'_p / f_{ms}) + (C' * L / f_{ms})$$

R_h	is the horizontal factored disturbing force
R_v	is the vertical factored resultant force
$\varnothing'_p$	is the peak angle of shearing resistance under effective stress conditions
f_{ms}	is the partial materials factor applied to $\tan\varnothing'p$, C', C_u
f_s	is the partial factor against base sliding
L	is the effective base width for sliding

Sliding force (R_h) = (P_1+P_2) = **657.78 kN/m**

Resisting force = $(R_V * \tan\Phi'_p) / f_{ms}$

= **886.91 kN / m**

789.34 ≤ **886.91** ***O.K***

Hence structure is safe in sliding stability

Check for Bearing Failure **CASE :A**

Overturning Moment

$M_o = (P1 * H/3 + P2 * H/2)$

= **2578.4 kN-m/m**

Resisting Moment

$M_R = (V1 * L/2 + Lc/2 \times V2 + V3 * L/2)$

= **9641.3 KN-m /m**

Eccentricity (e) of resultant load Rv about the centre line of the base of width L

$$e = L/2 - \frac{(M_R - M_O)}{\Sigma(V_1+V_2+V_3)}$$

$$= 3.8 \quad \frac{7062.8}{2566.46}$$

$$= 3.80 - 2.752$$

$$= 1.048 \text{ m} < \mathbf{L/6}$$

$$< 1.27$$ ***O.K***

Bearing pressure q_r due to Meyerhof distribution

$$q_r = \frac{R_V}{L-2e}$$

L is the reinforcement length at the base of the wall

R_v is the resultant of all factored vertical loads

$$= \frac{2566.46}{5.504}$$

$$= \mathbf{466.29} \text{ kN/m}^2$$

$$q_r \leq q_{ult}/ f_{ms} + \gamma * Dm$$

f_{ms} is partial material factor applied to q_{ult}

Ultimate bearing capacity of foundation soil $q_{ult} = \{c_f\, Nc + q\, N_q + 0.5\,(L-2e)\, \gamma_f\, N_\gamma\}$

for $\varphi = 30^\circ$

$N_c = 30.14$

$N_q = 18.4$

$N_\gamma = 22.40$

$L-2e = 5.50$

$$q_{ult} = qN_q + 0.5\,(L-2e)\,\gamma_f\,N_\gamma$$

$$= 1440.80 \quad \text{kN/m}^2$$

$$q_r \leq \quad 1047.14 \quad \text{kN/m}^2$$

$$466.29 \leq \quad 1047.14$$ ***O.K***

Hence foundation is safe against bearing capacity failure

COMPUTATION OF INTERNAL STABILITY FOR 10.75 m HIGH WALL BY STATIC ANALYSIS

Check for Rupture

For reinforced soil :

k_{a1} = 0.307

Ø = 32 ° r_1 = 18.5 KN/m^3

For retained backfill soil :

k_{a2} = 0.333

H = 10.55 m

L = 7.60 m

Foundation Properties

Dm = 1 m

r = 18 kN/m^3

LOADS

Self weight of Reinforced soil wall

				Lever Arm	
$V_1 = \gamma_1 * H * L$					
=	**1483.33 kN/m**		X1 =	3.8	m

Vertical load due to strip load

Q x b		where	b =	1.6	m
V_2 =	**24.72 kN/m**		X2 =	0.8	m

Vertical load due to live load

$V_3 = Q_l \times L$				
=	**174.8 kN/m**	X3 =	3.8	m

Horizontal Forces

$P_1 = 1/2 * k_{a2} * \gamma_2 * H^2$

= **343.02 kN/m**

4) E.P due to L.L $P_2 = k_{a2} * Q_l * H$ 80.84 **kN/m**

Check for internal Sliding

Calculation for bottom layer of Geogrid

For long term stability where there is reinforcement to soil contact at the base of the structure

$$f_s R_h \leq R_v(a' * \tan Ø'_p / f_{ms}) + (C' * L' / f_{ms})$$

R_h	is the horizontal factored disturbing force
R_v	is the vertical factored resultant force
$Ø'_p$	is the peak angle of shearing resistance under effective stress conditions
f_{ms}	is the partial materials factor applied to tan Ø'p, C', C_u
f_s	is the partial factor against base sliding = 1.3
L'	is the effective base width for sliding

R_h = $(P_1 + P_2) * f_{fs}$

635.80 kN/m

R_v = $(V_1 + V_2)$

1508.05 **kN/m**

826.53 ≤ 848.57 **O.K**

MOMENTS

Overturning Moment

Mo = $(P_1 * H/3*f_{fs} + P_2 * H/2 *f_{fs})$

Mo = 2449.1 kN-m/m

f_{fs} = 1.5 from Table 12 of BS 8006-1 : 2010

Resisting moment

$M_R = (V_1 * L/2 + Lc/2 \times V_2 + V_3*L/2)*f_{fs}$

M_R = 9481.0 KN-m /m

Eccentricity, e = $L/2 - \dfrac{(M_R - M_O)}{\Sigma(V_1+V_2+V_3)*1.5}$

$= 3.8 \dfrac{7031.9}{2524.28}$

= 3.80 - 2.786

= **1.014 m** < **L/6**

< 1.27 ***O.K***

Elevation of Geogrid Layer

E1 = 0.2 m First layer from bottom

E2 = 0.81 m Second layer from bottom

S_{vj} = 0.5*(E2-E1)+E1

0.505 m

$\sigma_{vj} = R_{vj} / (L-2e)$

= **446.42 kN/m^2**

$T_{pj} = ka_1 * \sigma_{vj} * S_{vj}$

= **69.23 kN/m** (for bottom grid)

Considering the crash barrier as a strip load and calculating the tension due to the strip load of width 1.6m

$T_{sj} = (k_a * S_{vj} * f_f * S_L) / D_j$

where **$D_j = ((h_j + b)/2) + d$**

calculating T_{sj} for the bottom most Grid layer

K_a = 0.307 b = 1.6 m

S_{vj} = 0.505 m d = 0.8 m

f_f = 1.5

S_L = 24.73 KN/m where Crash barrier area (A) = 0.989 m^2

Density of Concrete γ_c= 25 KN/m^3

$S_L = A*\gamma_c$

D_j = 6.875 m where h_j = (10.75-0.2) for bottom grid

T_{sj} = 0.836 KN/m

$T_j = T_{pj} + T_{sj}$

T_j = **70.07** KN/m

	Geogrid Data					
	Geogrid Type #1	Geogrid Type #2	Geogrid Type #3	Geogrid Type #4	Geogrid Type #5	Geogrid Type #6
Tult(kPa)	40.0	60.0	80.0	100.0	120.0	150.0
Strength Reduction factors						
Durability (RF_d)	1.15	1.15	1.15	1.15	1.15	1.15
Installation damage(RF_{id}) (based on type of soil)	1.1	1.1	1.1	1.1	1.1	1.1
Creep (RF_{cr}) (Varies based on temp of soil)	1.51	1.51	1.51	1.51	1.51	1.51
T_{design} (kPa) $=T_{ult}/(RF_d x RF_{id} x RF_{cr})$	20.94	31.41	41.88	52.35	62.82	78.53

For Type #6 : $T_D =$ Tult $/(RF_d*RF_{id}*RF_{cr})$

= 78.53 **kN/m**

$f_n = T_D / T_j$

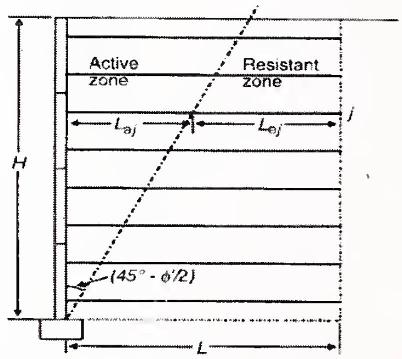
1.1 >1.1 from Table 9 of BS: 8006 **O.K**

Check for Pullout

Inclination of failure surface w.r.t horizontal $\delta = 45 + \emptyset/2 = 45 + 32/2 =$ 61 °

Elevation from bottom E1 = 0.203 m

wall batter (w) :- 4.23 °



$$Le = L - \frac{E_i}{Tan(\psi)} + E_i \times Tan(\omega)$$

Effective Length Le = 7.47 m

Perimeter of the j_{th} layer :

$$P_j \geq \frac{T_j}{\frac{\mu L_{ej}(f_{fs}\gamma_1 h_j + f_f w_s)}{f_p f_n} + \frac{a_{bc} c' L_{ej}}{f_{ms} f_p f_n}}$$

C' = cohesion of the soil $= 0$ KN/m²

f_{fs} = 1.5 from Table 11 of BS: 8006-1 : 2010

f_p = 1.3 from Table 11 of BS: 8006-1 : 2010

f_n = 1.1 from Table 9 of BS: 8006-1 : 2010

Pj = 2

f_{ms} = 1 from Table 11 of BS: 8006-1 : 2010

$$\mu = \frac{a' \tan \varphi'_p}{f_{ms}}$$

μ = 0.5 where a' = 0.8

h_j = 10.55 m

f_f = 1.5 from Table 11 of BS: 8006

$f_p \leq$ Pj * μ* Le*$(f_{fs}*\gamma_1*h_j)/T_j*f_n$

$f_p \leq$ 28.4 Where fp = 1.3 from Table 11 of BS: 8006-1 : 2010

O.K

Case A

Check for Connection Strength

The ultimate connection strength $T_{ultconn\,(n)}$ at each geosynthetic reinforcement shall be calculated as

$$\mathbf{T_{ultconn\,(n)} = a_{cs} + W_{w(n)}\tan\lambda_{cs}}$$

where $T_{ultconn\,(n)}$ = ultimate connection strength

a_{cs} = apparent minimum connection strength between geogrid reinforcement and block unit

λ_{cs} = apparent angle of friction for connection of geogrid reinforcement and block unit

a_{cs} , λcs shall be considered from Type #6 grid to block connection report

$W_{w(n)}$ = $(H-E_i)*\gamma_u*W_u$

Connection calculations for bottom most Grid :

H = wall height 10.15 m

γ_u = Density of plain concrete = 24 KN/m^3

W_u = Block unit width front to back = 0.305 m E_i = 0.203 m

$W_{w(n)}$ = 72.81 KN/m

a_{cs} = 19.71 KN/m

λ_{cs} = 30

$T_{ultconn\,(n)}$ = 61.77 KN/m

From Case A T_j = 70.07 KN/m

As T_j is greater than $T_{ultconn(n)}$ a secondary reinforcement of 1.0m length is provided at an elevation of 0.609m

Therefore S_{vj} for the bottom most layer for connection = 0.203+((0.609-0.203)/2)

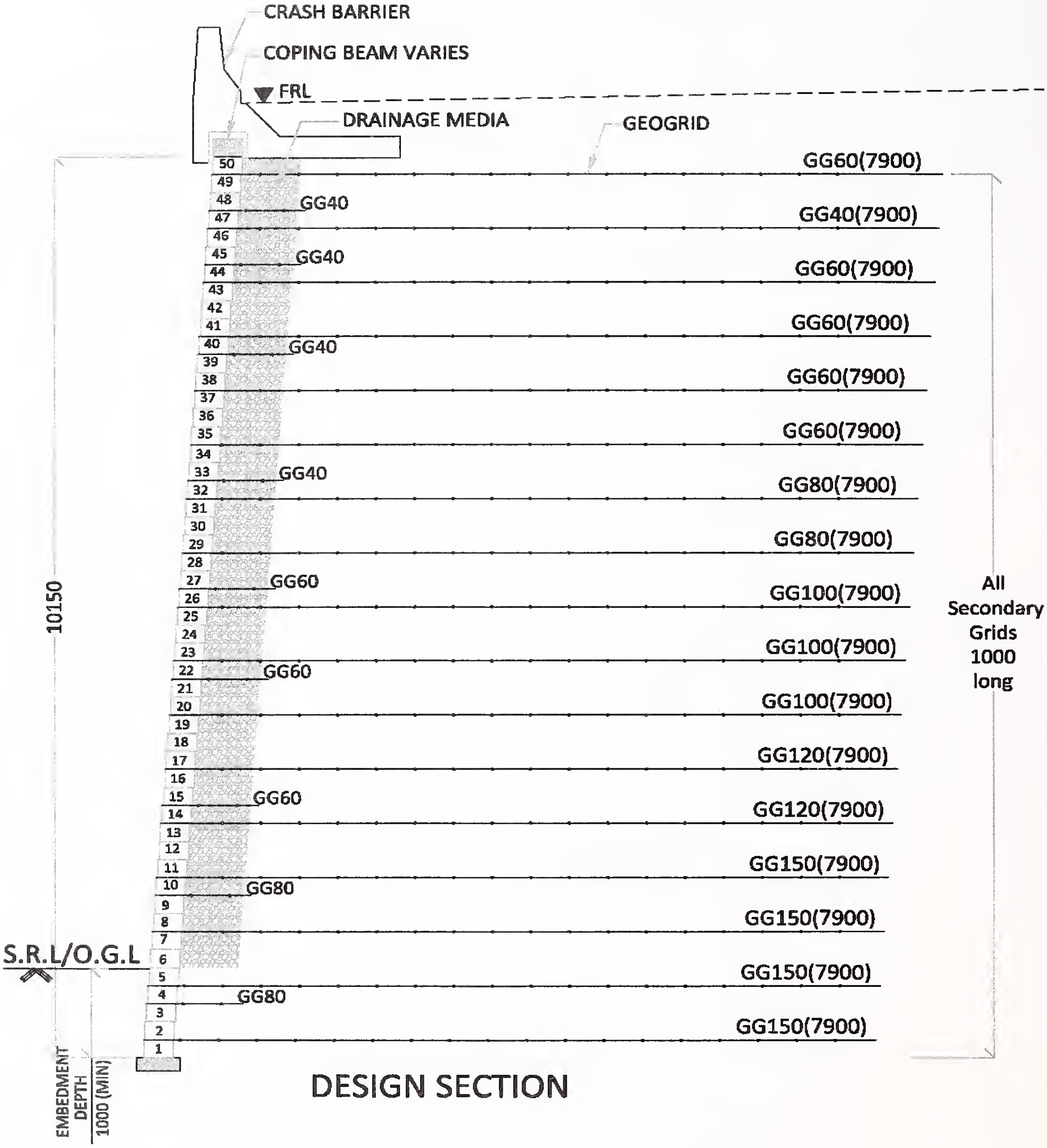
0.406 m

T_j = 70.07*0.406/0.505

56.3 KN/m

$T_j < T_{ultconn\,(n)}$

56.3 < 61.8 **O.K**



DESIGN SECTION

Title	Hand Calculations for Modular Block Wall of Height 10.75 m (Seismic)
Reference	
Date :	
Designed by :	
Checked by :	
Approved by :	

Design Input Parameters

Reinforced Soil Data

Angle of Internal friction	Φ_1	32	°
Unit wt	γ_1	18.5	kN/cu.m

Retained Backfill Soil Data

Angle of Internal friction	Φ_2	30	°
Unit wt	γ_2	18.5	kN/cu.m

Foundation Soil Data

Cohesion	c_3	0	kPa
Angle of Internal friction	Φ_3	30	°
Unit wt	γ_3	18	kN/cu.m

Dead load for 0.6m road crust	Qd	15.45	kPa	
Live Load	Ql	23	kPa	Use IRC-78:2014 provisions
Total Load	Q	38.45	kPa	

Water table is considered below the influence zone.
General Shear Failure is considered.

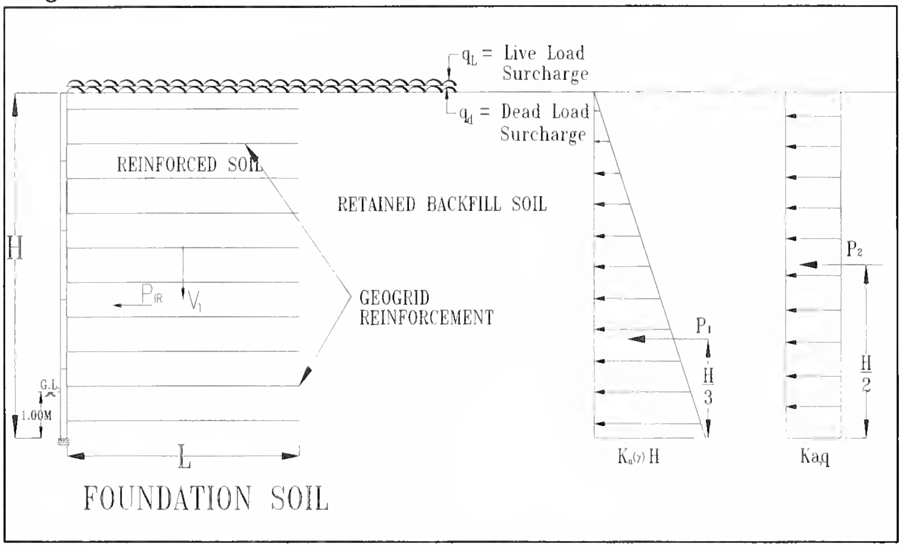
Computations for External Stability for 10.15 m High Wall Under Seismic Loading

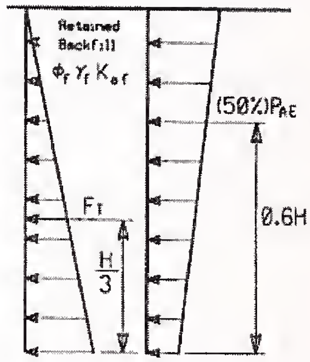
Coefficient of active earth pressure of retained fill

K_{a2} = 0.33

For seismic zone - III, Max hor. Acceleration coefficient A_m = 0.1

Height of the reinforced soil wall	H =	10.15 m
Length of the reinforcement	L =	7.60 m





LOADS

Self weight of Reinforced Soil Wall

$V_1 = \gamma_1 * H * L$

= **1427.09 kN/m**

Vertical Load due to Dead Load

V_2 = Qd x L

= **117.42 kN/m**

Vertical Load due to Live Load

V_3 = Ql x L

= **174.8 kN/m**

Resultant Vertical Load

R = ΣV= V1+V2+V3

= **1719.31 kN/m**

Horizontal Forces

Earth pressure due to backfill soil

$P_1 = 1/2 * K_{a2} * \gamma_2 * H^2$

$=$ **314.48 kN/m**

Earth Pressure due to surcharge

$P_2 = K_{a2} * q * H$

$=$ **128.79 kN/m**

$P_{IR} = 1/2 * A_m * \gamma_1 * H^2$

$=$ **95.30 kN/m**

$P_{AE} = 0.375 * \gamma_3 * H^2 * A_m$

$=$ **69.54 kN/m**

$(50\%) P_{AE} =$ **34.77 kN/m**

where, P_{IR} = horizontal inertia force

P_{AE} = Seismic thrust

Check for Sliding

Factor of safety against sliding = $\dfrac{\text{Resisting Force}}{\text{Sliding Force}} > 1.125$ (1.125 is 75% of 1.5 for static condition)

Sliding force = $P_1+P_2+P_{IR}+P_{AE}$ = **573.34 kN/m**

Resisting force = $\Sigma V * \tan\Phi$

= **891.73 kN / m**

$Fsl = \dfrac{891.73}{573.34}$

= **1.6** > 1.125 ***OK***

Hence structure is safe in sliding stability

Factor of Safety against Overturning

Overturning Moment

$Mo = P_1 * H/3 + P_2 * H/2 + P_{IR} * H/2 + 0.5 * P_{AE} * 0.6 * H$

Mo = 2413.0 kN-m/m

Resisting moment

$M_R = V_1 * L/2 + L/2 \times V_2$

M_R = 5869.1 kN-m /m

$F.S_{ovr} = \dfrac{\text{Reisting Moment}}{\text{Overturning Moment}} > 1.125$

$F.S_{ovr} = M_R / Mo$

$= \dfrac{5869.1}{2413.0}$

= **2.4** > 1.125 **OK**

Hence structure is safe in overturning stability check

Check for Bearing Capacity

MR_{BP} = $V_1 * L/2 + V_2 * L/2 + V_3 * L/2$

MR_{BP} = 6533.378 KN-m /m

Eccentricity, e $= L/2 - \dfrac{(M_{RBC}-M_O)}{\Sigma(V1+V2+V3)}$

$= 3.8 \quad \dfrac{4120.4}{1719.31}$

$= 3.80 - 2.397$

$= 1.403$ m $< L/3$

< 2.53 **O.K**

Calculation of Bearing Pressure

Mayerhoff stress , σ_v $= \dfrac{\text{Total Vertical Load}}{L-2e} = \dfrac{\Sigma V}{L-2e}$

$= \dfrac{1719.31}{4.793}$

$= 358.71$ kN/m^2

Ultimate Bearing Capacity

$q_{ult} = c_f\, Nc + q\, N_q + 0.5\,(L-2e)\,\gamma_f\, N_\gamma$

for $\varphi = 30^\circ$

$N_c = 30.14$

$N_q = 18.4$

$N_\gamma = 22.40$

$q_{ult} = qN_q + 0.5\,(L-2e)\,\gamma_f\, N_\gamma$

$= 1297.49$ kN/m^2

factor of safety , FS $= \dfrac{\text{Ultimate bearing capacity}}{\text{Mayerhoff stress}}$ > 1.875

$= \dfrac{1297.49}{358.71}$

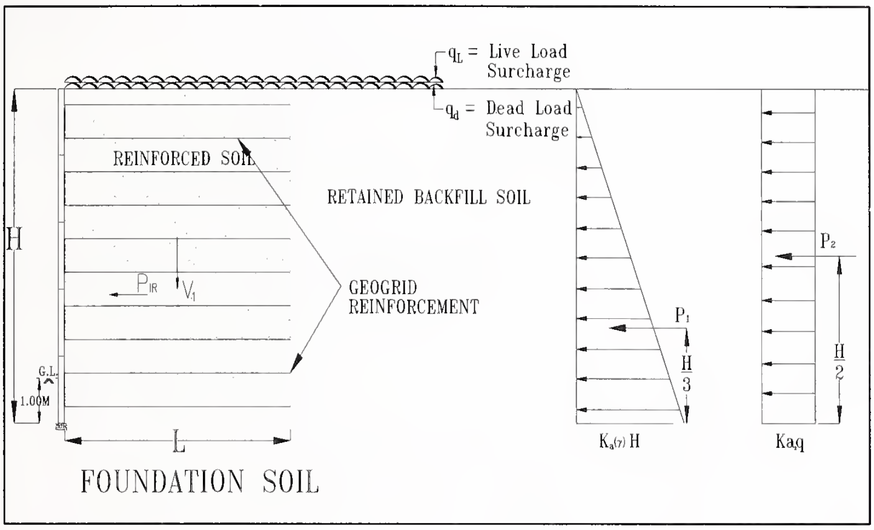
$= 3.62$ > 1.875

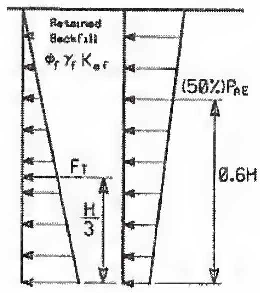
O.K

Hence foundation is safe against bearing capcity failure

Computations for Internal stability for 10.15 m High Wall under Seismic Loading

Height of the reinforced soil wall above the bottom most grid H = 9.947 m (i.e. 10.15-0.203)
Length of the reinforcement L = 7.60 m
For Seismic Zone - III, Max. Horizontal Acceleration coefficient A_m = 0.1





LOADS

Self weight of Reinforced Soil Wall

$V_1 = \gamma_1 * H * L$

= **1398.55 kN/m**

Vertical Load due to Dead Load

V_2 = Qd x L

= **117.42 kN/m**

Vertical Load due to Live Load

V_3 = Ql x L

= **174.8 kN/m**

Resultant Vertical Load

R = ΣV= V1+V2+V3

= **1690.77 kN/m**

Horizontal Force

Earth Pressure due to backfill soil

$P_1 = 1/2 * K_{a2} * \gamma_2 * H^2$

= **302.03 kN/m**

Earth pressure due to Surcharge

P_2 = K_{a2} * q * H

= **126.22 kN/m**

P_{IR} = $1/2 * A_m * \gamma_1 * H^2$

= **91.52 kN/m**

P_{AE} = $0.375 * \gamma_3 * H^2 * A_m$

= **66.79 kN/m**

$(50\%)P_{AE}$ = **33.39 kN/m**

where,

P_{IR} = horizontal inertia force

P_{AE} = Seismic thrust

Check for Sliding for bottom most Reinforcement Layer

Factor of safety against sliding = $\frac{\text{Resisting Force}}{\text{Sliding Force}}$ > 1.125

Sliding force = $P_1+P_2+P_{IR}+P_{AE}$ = **553.17 kN/m**

Resisting Force = $\sum V * \tan\phi$ = 259.84 * tan 30

= **875.25 kN / m**

Fsl = $\frac{875.25}{553.17}$

= **1.6** > 1.125 ***OK*** **Hence structure is safe in sliding stability**

Check for Rupture for bottom most Geogrid

Seismic Loads Produce an inertial force P_I acting Horizontally in addition to static force

P_I = $W_A * Am$

where,

WA = weight of the active zone

= 0.5*γ1*tan(45-ϕ/2)H2

= 507.6 **kN / m**

P_I = 182.8*0.1

= 50.76 **kN / m**

The total maximum tensile load T_{max} per unit width

T_{max} = $S_v * \sigma_H$

= $K_{a1} * \sigma_v * S_v$

Where,

σ_v = Vertical stress at the level of reinforcement

S_v = Vertical spacing of the reinforcement

V_i = Volume of contributory area

T_{max} = $k_{a1} * (\gamma_1 * H + q) * V_i$

= 34.49 **kN / m**

Dynamic Increment (T_{md}) induced by inertia force P_I in the different R/f Proportional to their Resistant Area on Load per unit wall width basis

$$T_{md} = P_I \frac{L_{ei}}{\sum_{i=1}^{n} (L_{ei})}$$

Maximum Tension applied at bottom most reinforcement layer

T_{md} = 4.40 **kN / m**

The maximum tensile force $T_{total} = T_{max} + T_{md}$ = 38.89 kN / m

T_a = $T_{ult} /(RF_d*RF_{id}*RF_{cr}xf_s)$ **kN / m**

For type6 T_a = 78.53

F.S against rupture = $\frac{T_a}{T_{total}}$ > 1.125 **O.K**

= 2.0

Check for Pullout for bottom most Reinforcement Layer

Available Pullout resistance P_r = $\tan\phi_1 * C_i * \gamma z * Le * C * Rc * \alpha$

Where,

C = reinforced effective unit perimeter e.g., C=2 for strips,grids and sheets

α =scale effect correction factor based on Laboratory data

geogrid - soil co.efficient is 80% of its value in seismic conditions $\tan\phi_1 * C_i * (\gamma z) * L_e * C * \alpha$

Available pullout resistance P_r = 1100.0 **kN / m**

F.S against Pullout = $\frac{P_r}{T_{total}}$ > 1.125 **O.K**

= 28.29

Check for Connection strength for bottom most Reinforcement Layer ≥ 1.125

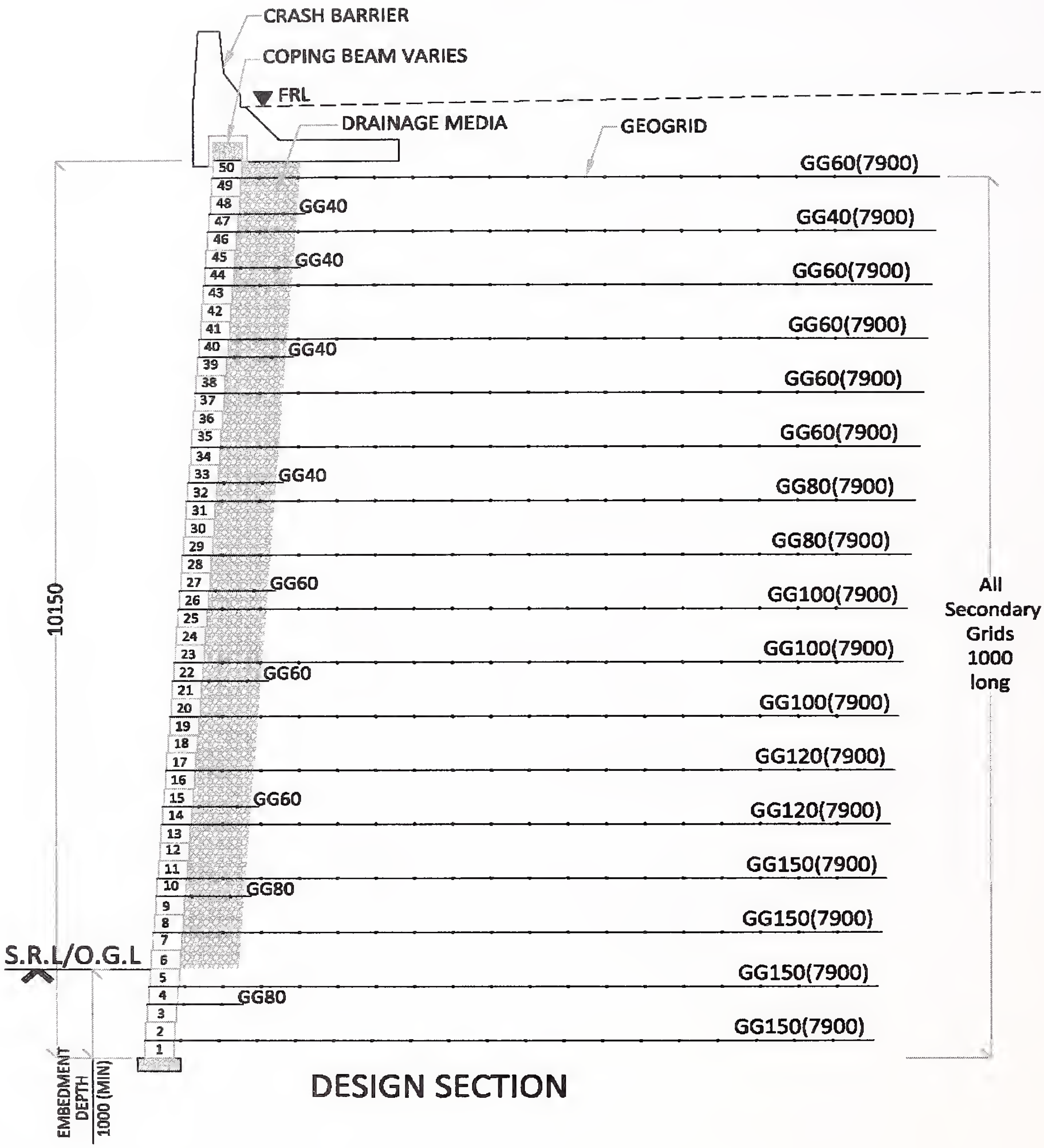
Under Seismic loading the long term connection strength shall be reduced to 80% of its static value

Factor of safety against conection strength = $\frac{T_{conn}}{T_{total}}$

T_{conn} = 61.77 KN/m

80% of T_{conn} = 49.416 KN/m 49.416/31 ≥ 1.125 **O.K**

Factor of safety = 1.6



DESIGN SECTION

APPENDIX

Personnel of the Foundation Sub-structure Protective Works and Masonry Structures Committee (B-3)

Bongirwar, P.L.	Convenor
Joglekar, S.G.	Co-Convenor
Kanhere, D.K.	Member-Secretary

Members

Bagish, B.P.	Kand, Dr. C.V
Chonkar, Ravindra	Mhaiskar, Dr. S.Y.
Dhiman, R.K.	Marwah, M.P.
Deshmukh, Dr. V.V.	Nashikkar, Jayant
Elavarson R,	Nayak, Dr. N.V.
Ganpule, Dr. V.T. (expired on 14.03.2013)	Ray, S. Saha, Smt. S.D Nigade,.
Gandhi, Prof. S.R.	Subbarao, Dr. H.
Jaigopal, R.K.	Saha, Dr. G.P.
Kothyari, Prof. U.C. (expired on 30.12.2012)	Singh, M.N. Singh, Rajeev
Karandikar, D.V.	Velyutham, V.

Corresponding Members

Basa, Ashok	Paul, Dipankar
Dey, Prof. S.	Pitchumani, Dr. N.
Heggade, V.N.	Sarma, R.H.
Mazumder, Prof. S.K.	Tarapore, Dr. Z.S.

Viswanathan, T.

Ex-Officio Members

President, IRC & Director General (Road Development) & Special Secretary	(Kandasamy, C.) Ministry of Road Transport and Highways
Secretary General	(Prasad, Vishnu Shankar), Indian Roads Congress

Personnel of the Embankment, Ground Improvement and Drainage Committee (H-4)

Kumar, Mahesh	Convenor
Rao, P.J.	Co-Convenor
Vittal, U.K. Guru	Member Secretary

Members

Bagli, Shahrokh P.	Lal, B.B.
Chand, Faquir	Mathur, Sudhir
Gajria, Maj. Gen. K.T.	Mazumder, Prof. S.K.
Gupta, A.K.	Mishra, Digvijay
Gupta, Sanjay	Pradhan, B.C.
Jain, N.S.	Ranjan, Gopal
Jain, Naresh Chand	Sangal, M.M.
Jalota, Dr. A.V.	Seehra, Dr. S.S.
Khan, Ms. Shabana	Sen, Samiren
Korulla, Mrs. Minimol	Shahu, Dr. J.T.
Kumar, Jaswant	Sharma, Dr. Ram Prasad
Kumar, Satender	Sreedhar, G

Verma, M.S

Corresponding Member

Madhav, Dr. M.R.	Thomas, Dr. Jimmy
Rajgopal, Dr. K.	Venkatappa, Dr. G. Rao,

Ex-Officio Members

President, IRC and Director General (Road Development) & Special Secretary	(Kandasamy, C) Ministry of Road Transport and Highways
Secretary General	(Prasad, Vishnu Shankar), Indian Roads Congress

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the code/guidelines/manual, etc. from the date specified therein)