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Ravin M. Tailor
S. V. National Institute of Technology, India

Jigisha M. Vashi
S. V. National Institute of Technology, India

Mahesh D. Desai
S. V. National Institute of Technology, India

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Ravin M. Tailor
S. V. National Institute of Technology,
Surat, Gujarat, India-395007

Jigisha M. Vashi
S. V. National Institute of Technology,
Surat, Gujarat, India-395007

Mahesh D. Desai
S. V. National Institute of Technology,
Surat, Gujarat, India-395007

ABSTRACT

India has massive infrastructure development plan in next decade. The safety, cost optimization saving construction time is challenges to engineers. The mass communication progress of 11th & 12th five year plans involves design and execution of large number of underpasses/flyovers through out country.

The problems faced by adoption of foundation practice in India based on interpretation of BS 8006 / 1995 during execution are analyzed. Though not widely publicized, failures of walls or part of facial block wall are reported. To avoid contract schedules quick remedial measures are adopted, which based on consultants and facilities includes stone columns, lime piles, CC slab cover over foundation trench etc.

A relook at entire problem for RE walls or steep slope foundation is reported. The site specific parameters namely construction season, rains during execution, desiccated expansive soils, settlement of parent subsoil for long life, environment – flood ponding are ignored. A sand-gravel 1.5 m pad foundation cannot take above factors in to account. The soil below the pad is rarely evaluated for differential settlement. Cyclically flooded poorly drained geographical areas particularly for long life structures, needs to be looked into.

For Indian fast developing zones a common approach is evolved. This includes specific shallow depth exploration of RE wall foundations, environmental data collection of drainage, flooding and settlement analysis. Depth of trench is site specific depending on desiccated depth and permissible settlement. A model profile of subsoil, replaced relatively impervious fill in trench with or without Geofabrics is presented.

The relook of site specific factors and control of settlement in present practice is justified by case studies presented.

INTRODUCTION

Unpredicted rapid industrialization in western corridor in particular and overall communication in 11th – 12th five year plans have grown road links highways and expressways. This involves handling of large number of under pass flyovers all over country. These flyovers designed by practices based on available BS code 8006:1995 in Indian environment posed some performance problems during construction. The analysis of case studies related to foundation during construction phase was attempted. This led to revised to practice which includes local factors of soil (expansive), fill materials, environment influenced by climate change, water logging of the surrounding for some days by flooding, poor drainage of area, construction practices and plants, design parameters and interpretations of code by designers.

The typical problems, remedial measures for some cases will provide a base for drafting Indian code/standard.

The urban space and no cost constraint of land for public use in rural area, severe limitations of construction materials in parts of India, justified remodelled RE walls/ Reinforced slopes for flyovers and bridge abutments. Such RE walls with varying heights transfers variable stresses on foundation soil below normal ground level. Design of such structures follows BS 8006:1995 code guidelines. It is based on limit state analysis with specified partial factors for loads and properties of materials.

Overall rigid RE block, Fig. 1, is checked for external stability for sliding, overturning and ultimate bearing capacity of foundation soil. The interpretation of code ignores differential settlements of segmental blocks as insignificant. The designs follows codal non cohesive sand-gravel metal as fill for RE wall and 1 to 2.5 m foundation levelling pad. Few design adopted stabilised soils meeting design parameters in foundation levelling pad if durability criteria's are satisfied by shear parameters in levelling pad.

In RE wall/slopes on cohesive, relatively weak soils in foundation, a check of global stability is made. The differential settlement tolerated by RE fill is higher. Giroud and Noiray (1981) do not consider deformation of all components. FEM model considering strain compatibility, are not convenient for routine design (Rowe, 1987; Otani et al, 1998) as parameters at nodal points with reinforcement and heterogeneous soil are variable with time etc. BS code art 7.1 and Fig. 59 provides ground treatments if there is need to improve UBC and reduction in total settlement with time rate of the soft strata.

Normally for C=0 fill material and the shear parameters C' – Φ' predicted at end if life are adopted for basal reinforcement. For over consolidated clays residual Φ_{cv}, C_{cv} are used. If RE wall fill undergoes only small strains peak Φ'_p will be representative.

If reinforcement is polymeric, for long life decreased rupture strength due to creep governs design parameters.

RELEVANT CODAL PROVISIONS:

Foundation related articles of BS 8006:1995 are articles 1.3, 2.8, 5.1, 5.6, 6.5.6, 8.2, 8.4, and 9.4 along with fig 1(c). The provisions of code as practiced are summarised here.

The reinforcement acts as structural element resisting vertical load on compressible subsoil. It provides immediate relief to stress at foundation level. The basal reinforcement at the interface of fill and foundation restricts lateral movements of soil inducing tension in the reinforcement. This in turn increases lateral confinement and results in improved shear resistance of fill. Geogrids do not provide relief against construction pore pressures but geotextile reduces P.W.P in layer below, during compaction of layer.

Once settlement approaches ultimate value, reinforcement has no major function. Typical loads and terms are illustrated in Fig. 1. Minimum length of reinforcement is 0.7H, failure plane is arc tan 0.3, minimum depth of embedment = 0.45 m and depth of pad for foundation is 1 to 2 m subject to stress computations on virgin soil as safe in shear.

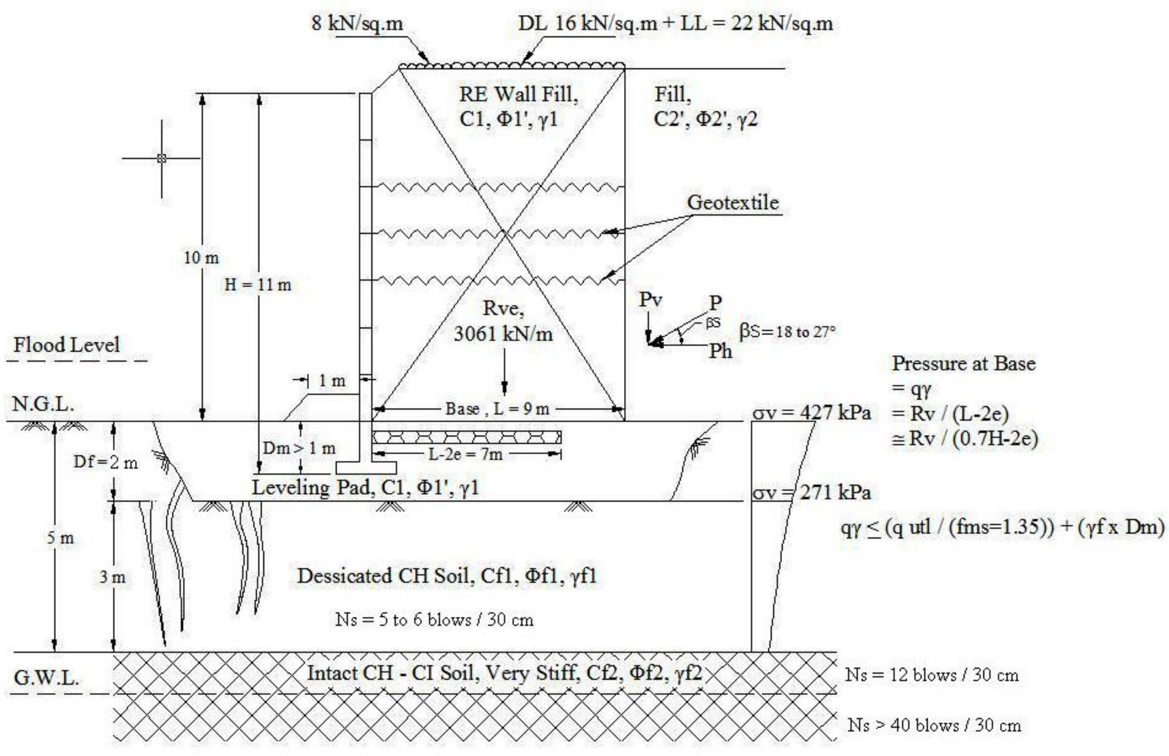


Fig. 1. Sketch of typical segmental RE wall with notations, foundation soil, fill and stresses.

H = 10 m, Foundation 0.0 – 2.0 m Refilled sand pad, 2.0 to 5.0 m desiccated CH clay, 5.0 m onwards intact CI – CH clay, $C_1' = 18$ kPa, $\Phi_1' = 30°$, $\gamma_{b1}' = 18$ kN/m^3.
For RE wall foundation pad compacted fill grading as per BS code, $C_{f2} = 70$ kPa, $\Phi_{f2} = 15°$, $\gamma_{b\,f} = 16$ kN/m^3, UBC = 570 kN/m^2.

Code considers limit state of collapse i.e. rupture or failure of bond in reinforcement backfill. Also it is checked for serviceability, limit state which occurs by excessive deformation of reinforced mass or excessive strain within reinforcement. For the fill of non cohesive materials prescribed, the plane strain at peak stress will be 3 – 5 % and hence strength of polymeric reinforcement availed in the construction phase will be much less. The reinforcement is considered axially stiffer than soil (Hausmann, 1990). Geogrid with tensile strength of 16 – 120 kPa having deformation modulus 150 – 225 kPa. Bond resistance is frictional for C=0 soil and adhesive resistance for $\Phi_u = 0$ soil. Hausmann prescribed minimum load of 50 kN/m and displacement of 25 mm for polymeric reinforcements. The typical dimensions for trial are shown in Fig.1 and for 70 years life of structure, if no data is provided by client 10 kPa surcharge at surface is presumed.

Though code implies design of foundation pad on basis of soil profile up to (2 x Le) depths, many designer ignored subsoil below pad (Typical, Fig.1) considering it as incompressible. The prescribed fill in foundation pad shall be granular 90 mm passing with 600 µm passing fraction 0-25 % and passing 63 µm less than 12%. Cohesive and industrial by products can be used if they satisfy code Art. 3.1.2.2. For cohesive fill, basal reinforcement gain in strength is slow requiring consideration of stress relaxation. For long life creep of polymeric reinforcement could be critical. As per Art. 5.5 long term settlement must be computed by conventional practice considering creep.

TYPICAL RE WALL FOUNDATION SYSTEMS

The practice of design of foundation system is illustrated by case studies on Bombay – Baroda, Bombay – Pune express / highways. This sector is predominantly covered by expansive subsoil for 2 to 6 m depth. On the whole best practices of control of fill materials, control of compaction were adopted by all agencies. Even compacted foundation pad was checked for UBC by plate load tests.

The typical designs are illustrated in Fig. 2. The backfill and pad was granular fill as per BS code in all specifications.

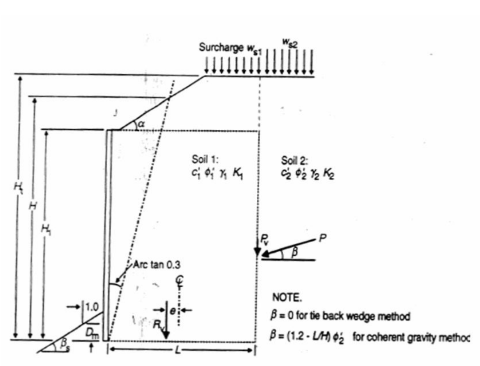


Fig. 2 (a). RE wall-Soil Properties and Principal Loads

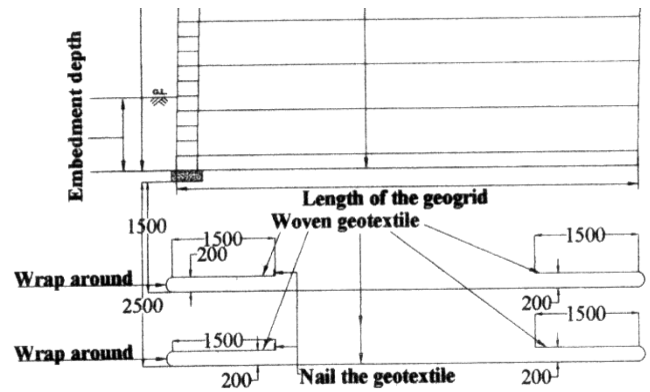


Fig.2 (b). Typical RE wall NH-8 Kamrej Soil Adopting Soil Improvment Depth 2.5 m, Geofabrics as Reinforcement, (Umravia, N. B. et al, 2010)

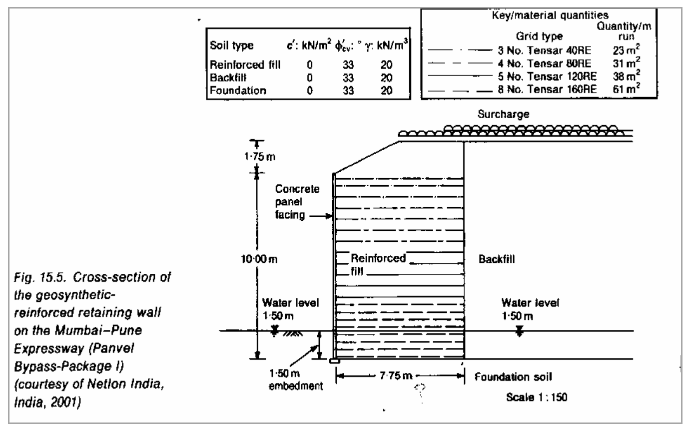


Fig. 2 (c). RE wall Mumbai – Pune Highway Depth of Foundation 1.5 m (Netlon India, 2001)

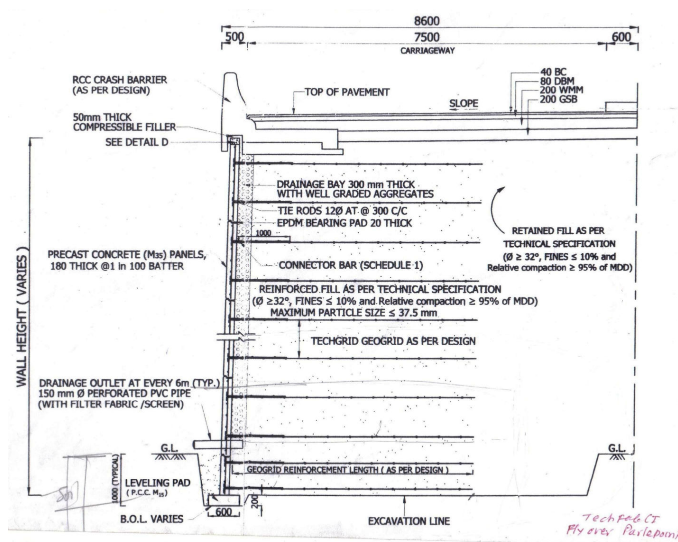


Fig. 2 (d). Typical section of RE wall in Surat City.

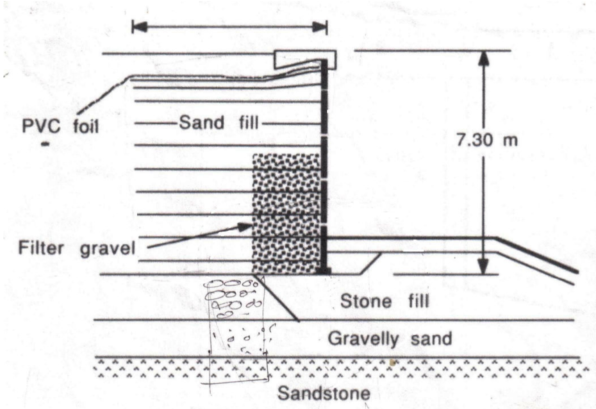


Fig. 2 (e). Typical Section of RE wall on Rocky Subgrade.

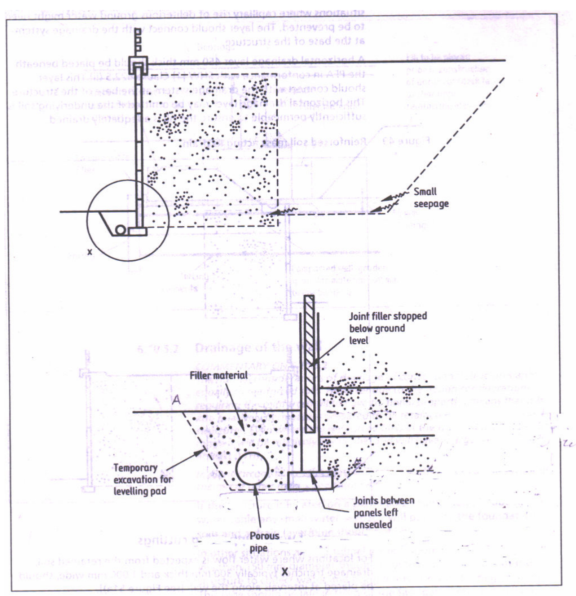


Fig. 2 (f). Details of RE wall foundation and fill drainage updated by BS 8006-1, 2010.

The normal practice of execution is March – April – May i.e. summer. Occasionally in recent years pre-monsoon showers are observed at random in some sites. The work is planned to complete base and part of raising of facial blocks by June – July in general (pre monsoon).

Very limited sectors of only few sites reported some distress in facial block wall during construction phase. They were corrected by the ground treatments. The case studies analysed initiated study to eliminate such problems by modifying the design practice.

INDIAN ENVIRONMENT

The code did not elaborate for typical geological formation of India. Fig. 3 shows vast areas having expansive soil, red murrum, white clay which shows expansive and shrinkage characteristics. Typical properties of massive deposits at surface are shown in Table 1 as under (Tailor R.M. et al, 2011).

Table 1. Geotechnical Properties of Black Cotton Soil.

Property		Values
Grain Size	Gravel (%)	1
	Sand (%)	12
	Silt + Clay (%)	87
Atterberg's Limit	Liquid Limit (%)	55
	Plasticity Index (%)	27
Compaction Test	MDD (kN/cu.m)	15.50
	OMC (%)	21.75
Swelling Test	Free Swell Index (%)	70
CBR (%)		1.77
UCS (kN/sq.m)		59
Permeability (m/s)		8.75×10^{-9}

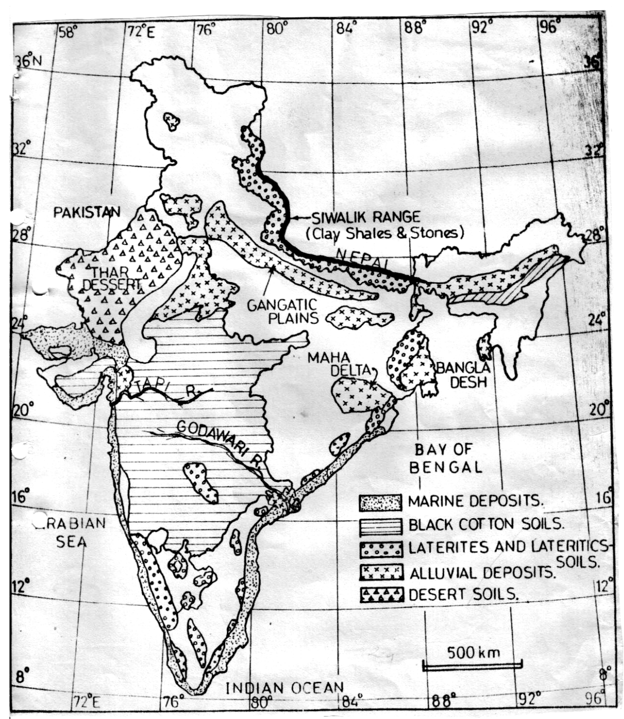


Fig. 3. Map showing the soil deposits in India.

Such deposits are wetted / flooded for 90 days in monsoon and dried in hot summer. Rainfall is average 1000 mm season. Such deposits are in areas with poor drainage and are flood prone in cycles. The structure of top 2 to 3 m of top such clays is structurally desiccated, cracked, clods of soils sometimes in clay stone consistency (Fig. 4), below this same wet intact soil extends to 5 to 7 m in general. For the strata below G.L., exploration in monsoon or by wash boring cannot identify desiccated zone as top strata is clods with water in joints subjected to swelling and shrinkage daily, cyclically.

Fig. 4. Photo showing desiccated Clay Profile.

This strata in summer will be hard clay N_{SPT} > 10 blows/30cm but in post monsoon it will have N_{SPT} 3 to 5 blows/30cm. The insitu CBR > 15 in field in summer reduces to CBR < 2 for top cracked crust in monsoon days. Field open excavation shows cracked zone extends from 2 to 3 m at top. Thus for such soils, present practice of adopting soil profile from limited soil exploration of bores for deep pile for abutment was misleading. Time of exploration, rain cycle vis-à-vis construction schedule and flooding by river/topography or existing rail, road, irrigation embankments are critical for design. As life of structure being 70 years, the land use nearby over 7 decades cannot be anticipated. It also cannot be overlooked. Nearby borrow pits / deep excavation when flooded induces swelling /shrinkage in subsoil damaging well designed expressways. The bearing capacity and differential settlement of RE wall block for different heights will govern the depth of foundation in such cases. The above practice is not safe always.

NEED FOR RELOOK AT DESIGN PRACTICE

In addition to environmental factor discussed above four limitations reported by G. Kempton and Patric Naughton (2005) are:

a) Non consideration of seismic forces in design.
b) Inadequate guidelines for construction to achieve designed performance for long life.
c) Little guidance for design of segmental blocks.
d) No scope for use of alternative fill and reinforcement materials for RE structures. Now revised BS 8006:2010 implies use of alternative fill and polymeric materials but it will take long time before it is adopted in Indian practice.

The interpretation of practice is illustrated in Table 2. To determine depth of foundation of RE wall block, environmental and unknown land use aspects explained earlier are not considered.

Some of site problems of executing RE wall on state highways reported distress during the construction of segmental wall. Tilt, settlement etc. observed had to be remedied by use of stone columns. Re-exploration and designs for subsoil suggested by consultants covered reinforced pad of foundation, increasing depth, insitu lime treatment for wet expansive soil etc. as illustrated in case studies.

This background justified total relooks at foundation model in expansive soil in the typical environment.

SUMMARY OF INDIAN PRACTICE

The client / project consultations invite preliminary proposal for structure including RE wall from specialized firms. They are scrutinised by project consultants with help of geotechnical engineer for site conditions and economics. The designs have following common futures:

1. The investigation of 2 bores for each abutment to 30 m depth is provided by owner. Such deep exploration ignores top 3 to 5 m strata and settlement SPT and test on so called UDS are conducted at 2 to 3 m interval below 3 m. The aim is to provide data for deep foundation.
2. The fill material and hence shear parameters are adopted as per BS 8006:1995, commonly bulk density of 18 to 20 kN/m^3, C' =15 to 20 kPa, Φ' = 30°. Such materials are pervious and have above parameters ensured even if compaction is poor at places. If trench, even partly filled, with standard back fill at some places, was grouted by muddy pre monsoon rain water and the fill was submerged. Climate change is unpredictable so far.

Table 2. Typical illustration of design of depth of foundation of RE wall block

H, Wall Height (m)	L, Width of RE wall @ base, (m)	Trial depth (D_f) below base (m)	UDL base Stress RE wall (kPa)[a]	UBC of soil @ foundation (kPa)[b]	F.S. [d]	Settlement (mm)[c]
6	4.7	1	191	238	1.2	
		2	150	514	3.4	152
		3	124	559	4.5	171

Note:
a) The maximum stress by Mayerhof's approach.
b) UBC of soil in pad of backfill by Terzaghi's theory (properties of backfill C'=3 kPa, Φ'=32°).
c) Elastic settlement of pad, no water table.
d) Factor of safety in shear minimum 3, D_f= 2m is ok.
e) Though settlement varies with L, it is not taken into account by practice in preliminary analysis.

Table 3. Typical stress below base for different height of fill

Design height, H (m)	Length of Reinforcement block (m)	Stress as per design 1 m below base (q_r) (for critical load combination) kPa
3.2	4.1	172
5.6	4.5	293
7.2	5.7	347
10.4	8.2	450

3. The net stress σ_v on effective width (L-2e) is treated as UDL (Mayerhof's approach). A 1 to 2 m thick foundation pad is designed for SBC of 2 $\sigma_{v.}$ some designers checked SBC of vergin soil below. The settlement is indicated based on no W.T. and meagre soil data.
4. The reports are causal about W.T. and probable wetting or surrounding in life of structure.
5. Typical stress variation with height of fill is illustrated in Table 3.

The minimum stress for 10.4 m height (9.4 m) above GL +1 m in foundation is around 450 kPa. The minimum UBC for soil in foundation with F.S = 1.4 is 630 kN/m^2 for worst strata during its life of 70 year. The present data of soil explored in top expansive cohesive soil in summer and monsoon will be different. This variable stress (140 - 450 kPa) for variable width at base of RE block induces settlement which is ignored by most of the preliminary designs. Actual settlement estimated is shown as 150 – 170 mm for H =6 m (Table 2) leads to differential settlement along RE wall length and is again function of time. This cannot be ignored for long term performance.

CONSTRUCTION PRACTICE

Design parameters are influenced by construction practices. The inadequate good construction practice details in BS code 8006:1995 are described by Geoff Kempton et al (2005). C.G. Jenner (2005) discussed good practices explaining proper draining, placement of facing blocks, placement of reinforcement, and placement of fill. The need to prevent construction plants over reinforcement and restricting plant load to 1500 kg within 1 m at back of fall wall are highlighted. This is rarely practice practised at site.

Vibratory and pneumatic compactors are now widely adopted to save time but its impact has not been studied particularly when fill is granular. The improved quality controls are practiced by contractors which includes borrows area survey for fill material, control of OMC and checking of MDD to specified values. The overall foundation pad is tested by 45 cm plate load test for UBC and deformation modulus. The instrumentation of overall performance of foundation and fill is not yet introduced. There are still problems due to misinterpretations of fill and foundation cohesive soils shear parameters using empirical N- Cu correlation on basis of SPT test at shallow depths. Even interpretation of plate load test for backfill has been controversial for size effect of plate and rigid block of RE wall.

In spite of good design, workmanship, using specified materials, some sites during initial stage showed distress in facing block. Some of sites were reinvestigated, and consultants prescribed ground treatments with stone columns, lime piles or lime stabilized soil at base for reinforced wall width etc. Fig. 5 illustrates a typical ground treatment prescribed for damaged block and fill zone of RE wall. This is emergency remedy to avoid construction delay. The probable cause and remedial treatment for all future RE wall foundation is aimed in studies. FEM analysis of failure is explained by Sengupta, A. (2012).

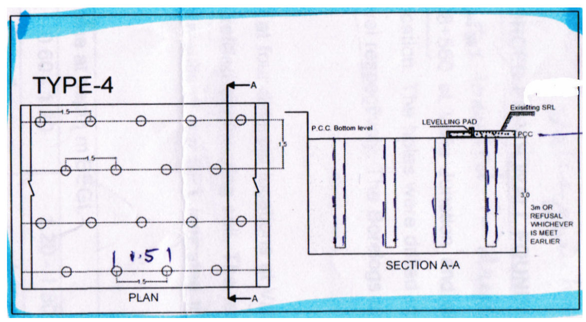


Fig. 5. Ground treatment during construction of foundation for RE wall.

Analysis for probable causes:
1) High vibratory roller used for fill/construction P.W.P in foundation trenches flooded / wetted by rains. The un-drained boundary of soil may cause sloughing, warping of soil mass. Fig.6 shows BS code with modifications.
2) Flooding at site during construction by rain, flood, and water logged surrounding, particularly in desiccated top strata of CH soil. (Fig. 6).
3) Seasonal G.W.L. rise.
4) Starting excavation in April - May, trenches at some sites are fully wetted by pre-monsoon showers, filled up by rain water. The soil suction in desiccated clay (cracks extending 2 m below trench) reduced shear and increased compressibility of CH clayey soil. Heavy compaction stresses induced P.W.P distressed partly raised segmental wall during construction.

5) Investigated in summer but wetted in excavations CH soil behaves as saturated soft clay. Cu for fissured clay on flooding reduced to 40-50 kPa giving net SBC 120 kPa below pad. Thus shear and settlement for a design could fail in shear/sliding and slip even before full load is applied. The fill materials shear parameters as designed will perform with un-drained parameters after wetting & flooding. The desiccated clay cracks make mass semi-pervious.

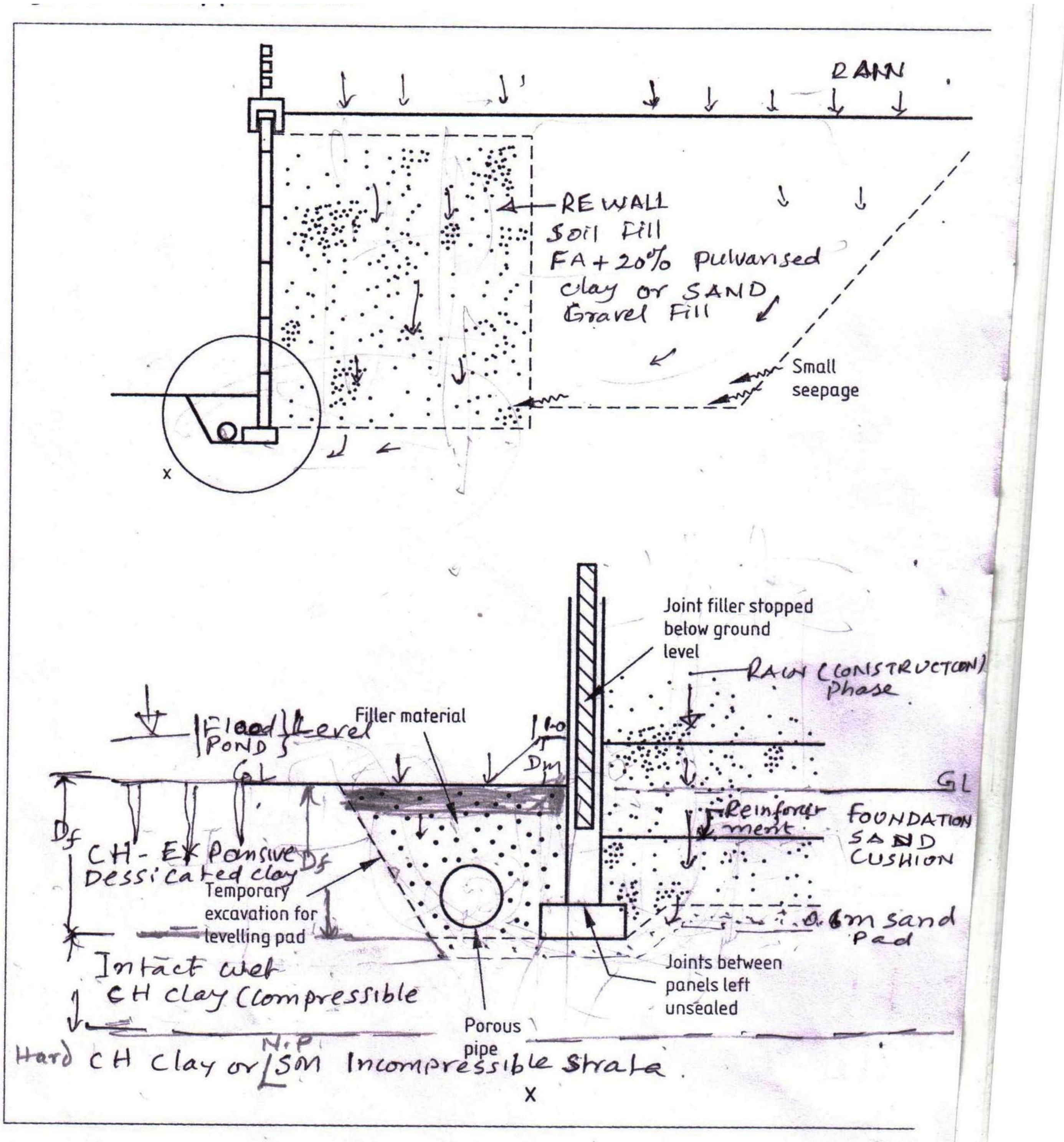


Fig. 6. Ground treatment during construction of foundation for RE wall. Influence of Rain during construction.

REVIEW OF DESIGN

Re-investigation of executed trench for foundation of RE wall on both sides was conducted by quick DCPT test. The Fig. 7 shows considerably wide range of N_c from 4 to 12 blows / 30 cm. Beyond 1.2 m from G.L. the strata below 4.8 m is very stiff unaffected by climate/rain. The excavated trench, backfill of material SC with clay 12 %, PI = 19 when flooded/wetted in rains shows poor shear resistance due to P.W.P due to compaction of fill above G.W.L. The typical model soil profile is shown in Fig. 1.

The un-drained conditions were one of causes of poor performance of the CC blocks during construction. The surrounding clay shows swollen state in around trench but it is not so in summer. Even design fill in front of RE wall (Figure 1) is initially absent in some cases.

To improve shear resistance and stiffness 2 layers of geotextiles at 2 m and 3 m below the GL is proposed for free draining sand fill compacted to @ OMC to MDD as shown in Fig. 8.

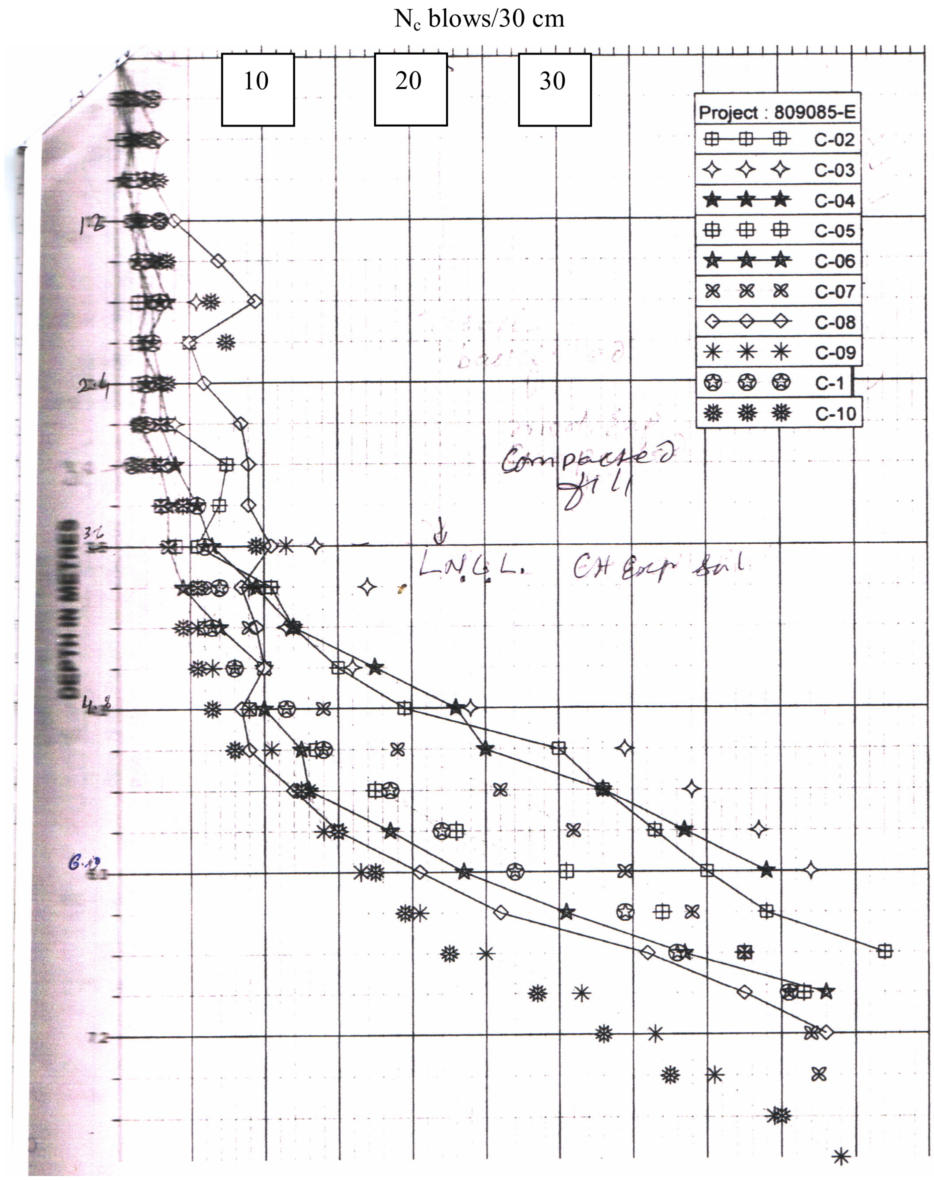


Fig. 7. DCPT test data for typical field in the foundation of RE wall

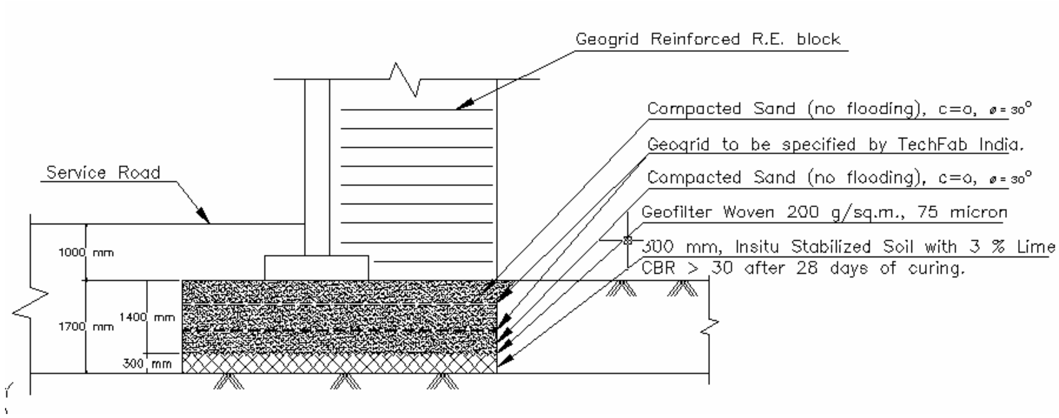


Fig. 8. Reinforced pad for foundation of RE wall

This reinforced sand pad with woven PP geotextile SKAP 300 or equivalent with tensile strength of 56 kN/m @ 12 % strain, 203 g/m2 was adopted. It reduced vertical stress at least 30 %, improve the stiffness and reduce settlement to tolerable limits (25 mm). The drainage during compaction (filter) reduced effect of water logging occasionally non-woven & woven fabric combination is suggested to give drainage & reinforcing effect.

CASE STUDIES OF DESIGN BY PRACTICE

Case Study 1, Kamrej (Strata)

To represent practice preliminary design for junction on NH-8 near Kamrej is summarised.

1. The soil exploration by 4 bores (2 on either side of abutment) indicated 4 to 2 m of stiff MH-CI clays, water content 11 to 25 %, clay content 22 %, LL = 40 %, PL = 23%, average dry weight 1500 kg/m^3, water table 4 m below GL, Cu = 70 kPa, $\Phi u = 0^\circ$, $m_v = 0.11$ cm^2/kg, Ns = 8 blows/30 cm.
2. The strata below 20 m to 30 m is weathered rock with Ns = 15 to 100 blows/30 cm, good rock core ult. Strength = 6000 kPa.
3. Design for wall height H = 9.135 m, L=length of reinforcement = 6.7 m, surcharge slope $\beta = 0$, depth of embedment Df = 1m in sand pad, properties of soil backfill and pad of foundation: bulk unit weight = 19 kN/m^3, $\Phi' = 34^\circ$, C' = 0 kPa, soil below foundation pad: bulk unit weight = 1770 kN/m^3, $\Phi u = 5.4^\circ$, Cu = 70 kPa, Load (LL+DL) = 40 kN/m^2, wall friction $\delta = 2/3\Phi_f = 22.67^\circ$.

Maximum bearing pressure of wall (udl) = 221 kN/m^2, at 2m below GL, UBC of soil at base = 533 kN/m^2, F.S in shear = 2.4, the design suggest 2 m below reinforced soil zone shall be selected foundation subgrade soil compacted to MDD at OMC. The predicted settlement of subsoil below 2 m will be 178 mm. For different heights say 4 to 9 m settlement predicted varies from 126 to 178 mm, causing differential settlement along wall.

The site construction control test by 45 cm plate load test on top surface is shown in Fig. 9. The UBC for test with size correction for effective width is more than 600 kPa. The settlement computed based on E = 18000 kPa was less than 20 mm.

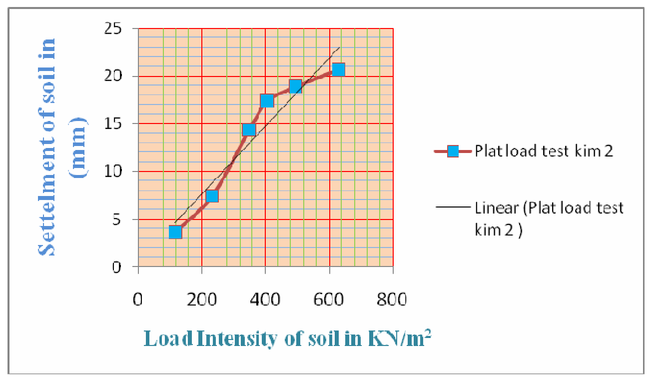


Fig. 9. Typical Load Intensity V_s Settlement Curve for foundation pad

At similar sites there were distress in facing block during construction hence second review was opted by client. The time bound solution of ground improvement was suggested.

Case Study 2

Another typical case study on NH 8 near Bharuch (Gujarat), in 4 bore holes up to 30 m for foundation of abutments explored in oct 2007(monsoon) shows water table beyond 7 m depth. The soil profile shows CH-SC group of clays highly expansive clays with top 2 m showing natural w.c of 27 $\pm$ 2 % with N_{SPT} of 7 to 10 blows/30 cm. The same soil from 2 to 6 m shows water content 16 to 20 % decreasing with depth with N_{SPT} > 15 to 20 blows/30cm. A conventional sand and gravel levelling pad of 1.5 m was provided. The rains water percolated from fill of RE wall under construction and sand gravel pad was fully saturated. The surrounding natural CH soil is impervious below 1.5 m. Some patches were grouted by surface wash clay fractions with rain water from surroundings.

The CC block facing wall founded at 1 m below GL was raised gradually. The post monsoon filling in RE wall under construction generated PWP in foundation pad reducing net SBC of block of RE wall. The compacted sand gravel having net SBC of 290 kPa reduced by water logging (undraind stat) to almost 145 kPa. Stress on facial block wall was obvious output causing distress.

The matter was referred to a consultant for quick solution to keep up schedule of construction. A reinvestigation of construction pad found ok even by plate load test. The tests on constructed fill showed average N_{SPT} for 0-0.6, 0.6-1.2m, 1.2 to 1.8 m as minimum 4 and average 7 to 8 blows/30cm. For the height of wall 8 to 9 m at location required SBC was 250 kPa. The un-drained conditions and likely grouting by muddy rain water of rains at site may lead to shear failure. The loss of moisture in winter and summer if fill and top surfacing is done can cause severe settlements and differential movements of facial block wall.

The typical remedial treatment for the remaining work of RE wall suggested shows 4 rows of 300 mm stone column/sand piles to 4 m depth below finished pad level. The strata of clay below 4 m is CH soil with Cu > 100 kPa and is not fully saturated. A 200 mm thick M20 PCC cover is provided over piles such that rain water seepage is cutoff.

In some cases designers recommend 3m deep trench, 2 to 3 m soil is disc harrowed and mixed with hydraulic lime 6 to 8 % and recomputed by rollers. 0-2 m below backfill is selected fill in layers of GC, SM-SC soil to -2 % OMC and 95 % of MDD to give required bearing capacity.

PROPOSED DESIGN FOR MAGADALLA CROSSING

Conventional design for RE wall shows stress of 450 kPa at base and UBC of soil below 630 kPa for height 10.4 m for work condition. For height up to 6 m stress and UBC is satisfactory with levelling pad of 2 m. The strata 2 to 3 m requires ground improvement for height more than 6.0 m.

The soil profile explored by 4 bores and static cone tests (SCPT) is shown below:

0 – 3.0 m	Desiccated potentially expansive CH soil mixed with road material (Ns = 6 to 6 blows / 30 cm)
3.0 – 5.0 m	Intact CH clay Ns = 10, Cu = 70 kPa, water content 30 to 46 %, Es = 10 MPa.
5.0 – 9.0 m	CH intact clay Ns > 20, Cu > 920 kPa, w = 24 %, W.T. @ 7.5 m, $\gamma d = 1500\ kN/m^3$.

The strata below 9.0 m is stiff and can be treated as incompressible. The properties of clay below 3 m shows Cu > 100 kPa, Φu = 10°, Eu > 10 MPa. The site is in flood plane of river Tapti.

Minimum depth of foundation for fissured desiccated expansive clay for site is 3 m below G.L. The maximum bearing stress for height of 10.4 m is 450 kN/m^2 at 3.0 m below G.L.

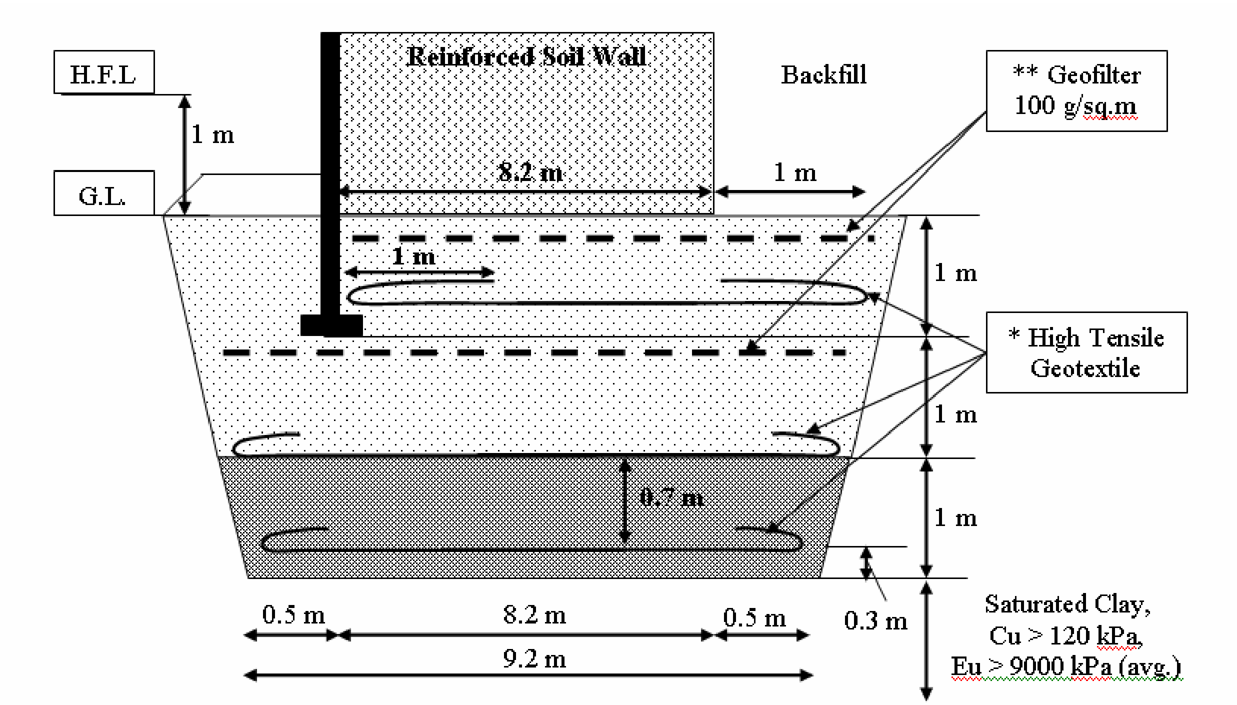
The strata below 3 to 5 m is CH intact clay showing Ns = 10, Cu by triaxial = 70 kPa, Φu= 0, UBC at 3 m below G.L. would be more than 500 kPa. Nc = 10 indicate insitu Cu > 80 kPa.

The fill of 2 m will distribute stresses of RE wall if Φ > 30° for fill. Thus stress at soil contact will be 350 kPa. This will provides adequate factor of safety for heights upto 6 m. For wall height 6 to 10 m, use of reinforced backfill is provided as shown in Fig. 10.

Use of geofilter (non-woven / woven) is provided for filter sand separation. Three high tensile fabric polyester PET 70/70 with tensile strength of 70 kN/m, εf = 12 % is recommended to control settlement of strata by increasing stiffness of fill material. Maximum settlement for height of 10.4 m was 120 mm for Es of flyash composite as 10 MPa. The data of SCPT and odeometer test shows settlement of 88 mm. It will be further reduced by increased stiffness with Geofabric considerably.

REVISED DESIGN CRITERIA

The typical model soil profile is shown in Fig. 11 incorporating state of clayey subsoil in poor draining, high rainfall areas of expansive sub-soils. The major revision is detail exploration of both RE walls for shallow depths of 6 m. The typical DCPT results and bores will provide soil profile model. Special tests of shear parameters in present and submerged un-drained condition and cracked depth of top clays are investigated.



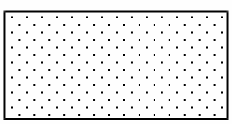
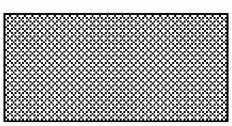
Item	Description
* High Tensile Geotextile	High tensile fabric polyester 70/70 PET type with property attached in Appendix – III. (Minimum width 9.2 m & wrap over & overlap as per standard)
** Geofilter 100 g/sq.m	Geofilter PP / Polyester 100 g/sq.m, 75 micron to drain fill
	Fill of Flyash + Soil (80: 20) with lime 2 %, if soil is expansive mixed by disc harrow in trench. Moisture OMC and minimum 98% proctor compaction, impermeable to water (ϕ = 30°), compaction in layers as per MoRT&H specification
	Sand, SW.SM, Compacted dry in 200 mm layers to MDD. The widening of top length by 1 m at bottom will reduce stress and hence settlement. 10.4 m – q = 450 kPa reduce to 350 kPa. (No flooding is allowed)

Fig. 10. Proposed design for foundation of RE wall at Magadalla

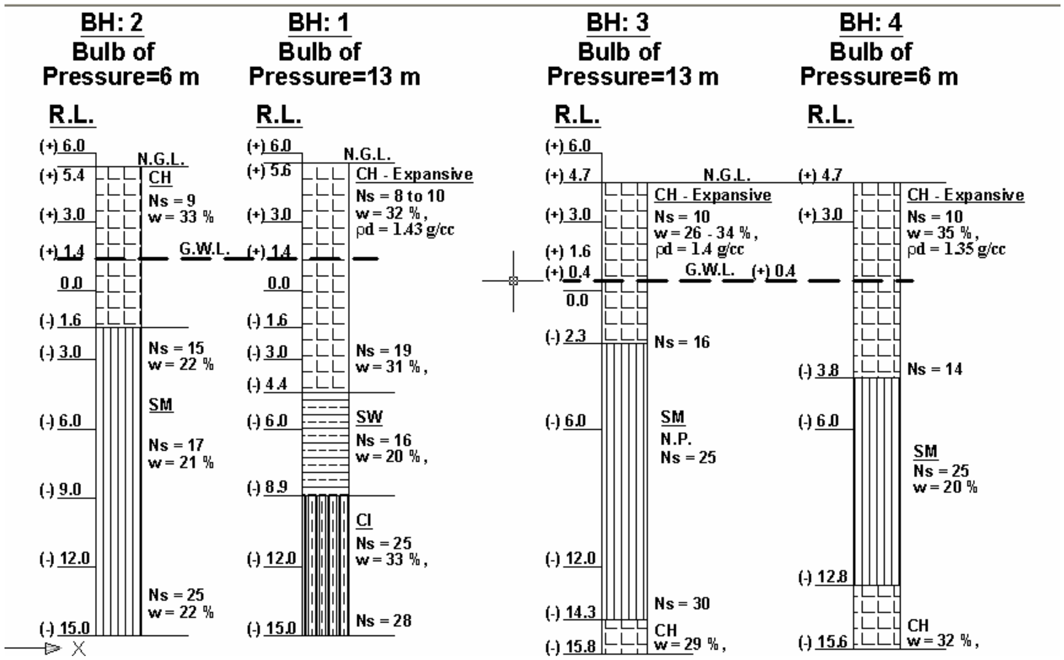


Fig. 11. Typical soil profile for foundation of RE wall.

The backfill of foundation fill of pervious gravel and sand which is pervious is replaced by locally available Flyash with pulverised clay or clay and lime 2 to 3 %. Typical design mix is tested for placement at – 2 % OMC, compacted to 98 % MDD. Mix must satisfy $\Phi = 30°$ and $k = 10^{-5}$ cm/sec giving UBC of more than 800 kPa for B = 6 m. The stress in (L – 2e) will be distributed to trench width. A typical section proposed for Magdalla site is shows in Fig. 12.

(a) Flyash mixing for soil stabilisation

(b) Laying of geotextile

Fig. 12. Photo plate showing construction of foundation for RE wall at Magadalla crossing in progress.

The final stress on virgin clay at 3 m below the ground is compacted. The SBC and settlement of compressible strata of soil is computed considering stiffness of reinforced sand pad.

The fill material in foundation is replaced by local material of Ash Fly and Bottom ash of power plants mixed with 2 % lime and 20 % pulverised black soil. This mix shows OMC = 30 %, MDD = 13.9 kN/m^3 and Cu = 300 kPa, $\Phi u = 38°$. The k value will be 2 x 10^{-5} cm/sec. This material placed on 1 m reinforced SW – SM layer below is laid to a) control rain water seepage from surface & sides, b) reduce settlement due to shrinkage & swelling of top layer, c) provide high tensile woven geotextile and filters to improve stiffness of fill.

CONCLUSION

The study of causes of few distresses on RE wall on express / highways analysed causes of failure. The problem is attributed to the pervious fill in backfill and foundation trenches. The pre-monsoon or rains floods during construction causes water logging in the foundation trench in clayey subsoil.

The C' - Φ' of the foundation pad confined by desiccated clay at base and around, creates un-drained shear state. The fill compaction induced high pore water pressure in planes causing sloughing and low bearing capacity.

Over winter / summer the desiccated clay drains sand by suction causing shrinkage and loss of contact with virgin soil. Thus stability of facial blocks is distressed cyclically, seasonally.

The paper provides for Indian environment and expansive subsoil sectors adopting imperious backfill material flyash with 20 % pulverised clay and designed lime content. In addition need for importance to construction techniques is highlighted. The Paper is intended to discussions to relook design softwares for better performance of structures for worst environment in 75 years long life of embankment. The case studies are intended to illustrate problems and solution without any ulterior motive.

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REFERENCE

BS 8006 [1995], *"Code of practice for strengthened/ reinforced soils and other fills",* British Standards Institution, London, UK.

BS 8006-1 [2010], *"Code of practice for strengthened/ reinforced soils and other fills",* British Standards Institution, London, UK.

Giroud, J.P. and Noiray, L. [1981]. *"Geotextile-reinforced unpaved road design" ASCE Journal of Geotechnical Engineering*, 107(9), pp 1233-1253.

Hausmann, Manfred R. [1990]. *"Engineering Principles of Ground Modification",* McGraw-Hill, New York

Kempton, G. T. and Naughton P. J. [2005], *"Design and construction of reinforced soil walls to BS 8006 and recent advances",* Geosythetics New Horizones, Asian books Pvt. Ltd, New Delhi,

Otani, J.; Ochiai, H. and Yamamoto, K. [1998]. *"Bearing capacity analysis of reinforced foundations on cohesive soil - a finite element analysis",* Geotextiles and Geomembranes, Volume 16, Number 4, August 1998, pp 195-206 (12)

Rowe, R.K. and Soderman, K.L. [1987]. "*Very soft soil stabilization using high strength geotextiles: The role of finite element analysis"*, Geotextiles and Geomembranes, Vol. 6, pp. 53-81.

Sengupta, A. [2012], *"Numerical Study of a Failure of a Reinforced Earth Retaining Wall"*, Geotech Geol Eng (2012), 30: 1025–1034, DOI 10.1007/s 10706-012-9524-3

Shukla, S. K. [2002], *"Geosynthetics and Their Applications",* Thomas Telford Publishing, London, USA, 430 p., ISBN 978-0-7277-3117-3 (Ha rdbound) & 978-0-7277-3783-0 (EBook)

Tailor, R.M.; Desai, M.D. and Shah, N.C. [2011], *"Geotextile as Reinforcement in Flexible Pavement for Swelling Subgrade",* Proceedings of Indian Geotechnical Conference 2011, December 15-17, 2011, Kochi, pp. 565 – 568.

Umravia, N. B. [2008], *"Review of Retaining Earth wall with Recent Development in Reinforced Earth",* M. Tech. (SMFE) Thesis, Applied Mechanics Department, S. V. National Institute of Tech, Surat.

Umravia, N. B.; Vashi, J. M. and Desai, M. D. [2010], *"Need for Relook at Design Practice for Reinforced Earth Wall Foundations",* Indian Geotechnical Conference – 2010, GEOtrendz, December 16–18, 2010, IGS Mumbai Chapter & IIT Bombay, pp 73 – 76.

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