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Performance of Instrumented Walls Retaining Reinforced Earth Fill

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Introduction

The concept of soil reinforcement is not new and in its crude form it has been in use since ancient times. Reinforcement of clay or bricks with reeds or straw for constructing houses, Great Wall of China and Agar-Quf are some of the existing well known examples of earth-reinforcement. Romans had built reed-reinforced earth levees along the Tiber. Reinforced earth concept in its present form was put forward by Vidal (1966). According to Vidal reinforced earth is formed by the association of frictional soil and tension resistant elements in the form of sheets, strips, nets or mats of metal, geosynthetics or fibre reinforced plastics. The reinforcing elements are arranged in the soil mass in such a way to reduce or suppress the tensile strain that might develop under gravity and boundary forces.

A large number of structures have been built throughout the world since the first application of reinforced earth technique in France in late sixties. The concept of earth reinforcement can find its application in two ways in case of retaining walls, e.g., reinforced earth wall and wall with reinforced backfill. Situations can be met with in practice where reinforced earth walls may not provide an ideal solution. This can be true for locations with limited space behind the wall or for narrow hill roads on unstable slopes which may not permit use of designed length of reinforcement. In such circumstances a rigid wall with reinforced backfill may prove to be more appropriate solution. In this (wall with reinforced backfill) technique cohesionless backfill of the wall is reinforced with tensile members that are not tied to the wall. Lateral (active) earth pressure

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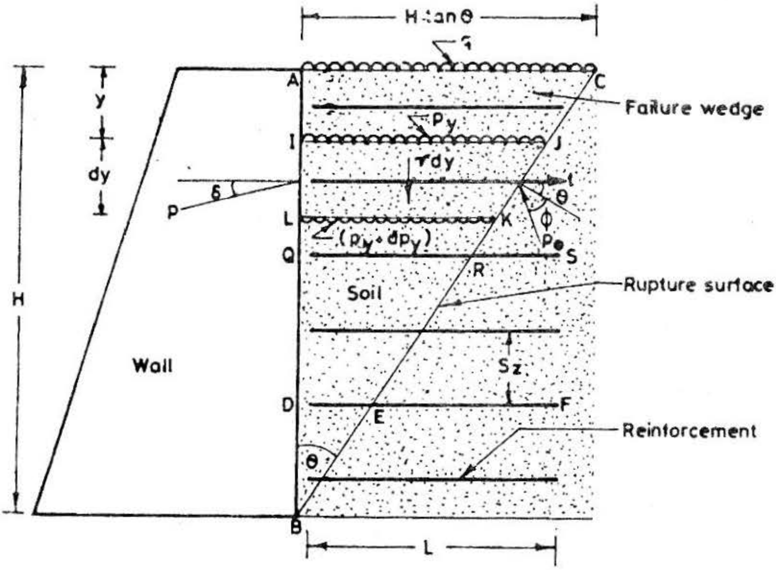

FIGURE 1 : Intensities of Forces Acting on an Element of Soil within Coulomb's Failure Wedge

on the wall gets reduced by this approach thus requiring thinner wall section.

Substantial reduction in the magnitude of active earth pressure on the wall can be affected by reinforcing its backfill Pasley (1882). Based on limited small scale model test results Hausmann and Lee (1978) and Talwar (1981) have also observed similar trends. Presence of uniformly distributed surcharge load on the surface of reinforced backfill (Fig.1) was considered by Garg (1988) in developing non-dimensional design curves to calculate the resultant active earth pressure and the height of its point of application above the base of a wall retaining a reinforced cohesionless fill. He (Garg 1988) has also reported observation similar to Pasley (1882) based on his model test results. Details of methodology for designing such retaining walls are available elsewhere (Garg 1988; Saran et al., 1992). Pinto and Cousens (1996) investigated the behaviour of geotextile reinforced brick faced retaining walls. The authors have tested model walls which were built, backfilled and then surcharged. Results have shown that even short lengths of reinforcement, embedded in the brick masonry courses at designed spacings, significantly improved the behaviour of the retaining wall.

All the above findings are based on the model tests in the laboratory. For the wider application of any new design methodology in actual field cases, it is always better to verify the developed new design approach through instrumented prototype studies. It is with this objective that two instrumented prototype rigid walls, retaining geogrid reinforced cohesionless earthfill, were tested in the field to establish the validity of the analytical approach developed by Garg (1988) and reported by Saran et al. (1992) for designing such walls. The backfill of the wall was subjected to a uniform surcharge load intensity at its top.

Design of Test Retaining Walls

Design Data

Height (H) of each wall	= 4.0 m
Type of masonry	= Brick masonry in 1:6 cement : sand mortar
Angle of internal friction (ϕ) of backfill (determined in laboratory)	= 30°
Angle of soil-wall friction (δ) (Assumed as $2/3\phi$)	= 20°
Average bulk density (γ_b) of earth backfill	= 17.90 kN/m^3
Unit weight of brick masonry	= 22 kN/m^3
Uniform surcharge intensity (θ)	= 5 kN/m^2
Wall base friction coefficient (μ)	= $\tan\delta$ = 0.364
Soil-reinforcement frictional (f*) coefficient	= 0.58
Allowable soil bearing pressure	= 110 kN/m^2
Reinforcing material	= CE 121 geogrid

Design Methodology

Garg (1988) has suggested a design methodology for designing walls retaining cohesionless fill reinforced with strips/mats, that are not tied to the wall and the backfill carries a uniformly distributed surcharge load. The methodology considers the static equilibrium of a horizontal element of soil, within a Coulomb's failure wedge, subjected to various intensities of forces as shown in Fig.1. Shorter length (DE or RS) of the reinforcing strip (DF or QS) is considered to be effective in providing frictional resistance against sliding. No frictional resistance will be offered by a reinforcing strip that lies completely within the failure wedge. Non-dimensional design curves are

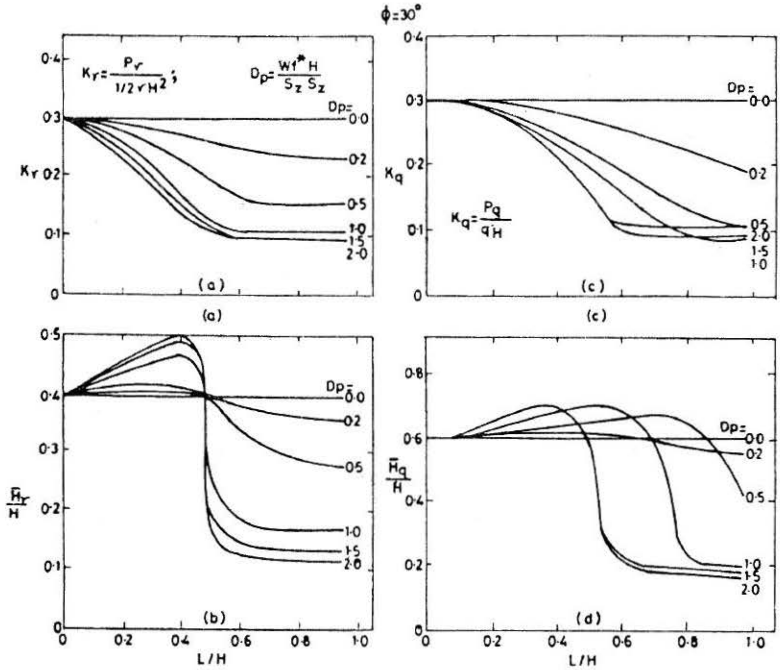

FIGURE 2 : Non-dimensional Charts for Resultant Pressure and Height of Point of Application : (a) a and b due to Backfill; (b) c and d due to Surcharge Loading (ϕ = 30°)

provided (Saran et al., 1992), for different values of angle of internal friction (ϕ), to get the values of active earth pressure coefficients

$$K_\gamma = \frac{P_\gamma}{0.5\gamma H^2}; \quad K_q = \frac{P_q}{qH}$$

and the points of application (H_γ and H_q) of the resultant pressure (P_γ and P_q) above the base of the wall.

The resultant active earth pressure (P) is the summation of:

(a) resultant active earth pressure due to backfill (P_γ) and
(b) resultant active earth pressure due to surcharge load (P_q) i.e. $P = P_\gamma + P_q$.

One such typical set of design curves for ϕ = 30° is provided in Fig.2, which shows that the pressure coefficients (K_γ and K_q) reduce in magnitude with an increase in L/H ratio and $D_p\left(= wf^*H/S_xS_z\right)$, the displacement

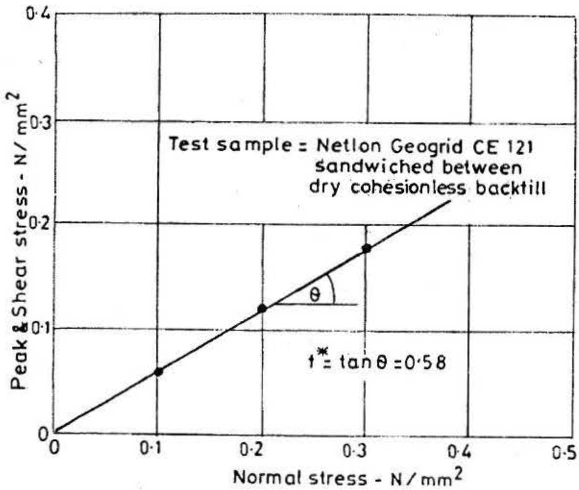

FIGURE 3 : Modified Direct Shear Test Results

coefficient, where w is the width of reinforcing strip, S_x is the horizontal spacing and S_z is thevertical spacing of the reinforcing strips respectively. In general it has been observed that noticeable reduction in the magnitude of earth pressure coefficients (K_γ and K_q) is up to $L/H = 0.6$ and $D_p = 1.0$. Variation of H_γ/H and H_q/H with L/H shows that the point of application of the resultant erth pressures (P_γ and P_q) moves towards bottom of the wall for $L/H \geq 0.5$.

Soil-Geogrid Frictional Coefficient (f*)

Soil-geogrid interfacial frictional resistance was evaluated in the laboratory by conducting sliding shear tests in a modified direct shear apparatus of size 300 mm × 300 mm and 200 mm in height. The lower portion of the box was having an arrangement to clamp the geogrid specimen. The geogrid specimen was sandwiched in between the representative sample of the soil to be used as backfill. The test results are provided in Fig.3 which yield $f^* = 0.58$. Angle of internal friction of the backfill, available at the proposed site of retaining wall, was also determined by direct shear tests in the same modified shear box apparatus. The tests were conducted at dry density of 16 kN/m^3 of the backfill. The geogried CE-121, marketed by Nelton India, is used as reinforcing material.

Design of the Walls

Adopting $D_p = 0.5$; $L/H = 0.6$;

For ϕ = 30° and δ = 2/3ϕ = 20° from Fig.2

K_γ = 0.150,

K_q = 0.188

H_γ = 0.25 H = 1.0 m

H_q = 0.65 H = 2.60 m

Designed sections of the test walls 1 and 2 are shown in Figs.4 and 5, respectively, along with geogrid reinforcement and instrumentation details. Vertical spacing (S_z) of geogrid reinforcement was calculated using the following equation (Saran et al., 1992).

$$T_i = \left[\gamma h_i \left(K_a - K_\gamma\right) + q\left(K_a - K_q\right)\right] S_x S_z \qquad (1)$$

where T_i = permissible tensile strength per meter length of geogrid

h_i = depth at which spacing is required

K_a = Coulomb's active earth pressure coefficient

Adopting S_x equal to 1.70 m in case of wall-1 and 1.0 m in case of wall-2, the permissible vertical spacings (S_z) in case of wall-1 and wall-2 are provided in Figs.4 and 5 respectively.

The Test Site

Experimental prototype studies were carried out at a site located within the Institute Campus. Geologically the site falls within the Gangetic plains of Northern India. The surface accumulation consists of sand and silt and at deeper depths clay. The site was chosen because preliminary explorations, carried out for some other purposes, revealed that the site contains uniform silty sand deposit of sufficient thickness. The depth of ground water table was 3.75 m in dry season and rises to 1.5 m during rainy season. The sub-soil profile along with the penetration data and soil properties is provided in Fig.6.

Construction of Test Facilities and Backfill Operations

Construction of two instrumented test retaining walls (wall-1 and wall-2), along with the three side retaining walls (A, B and C) was taken up simultaneously in February 1996 and was completed in December 1996. None of the wall was constructed more than 0.5 m in height at a time. There

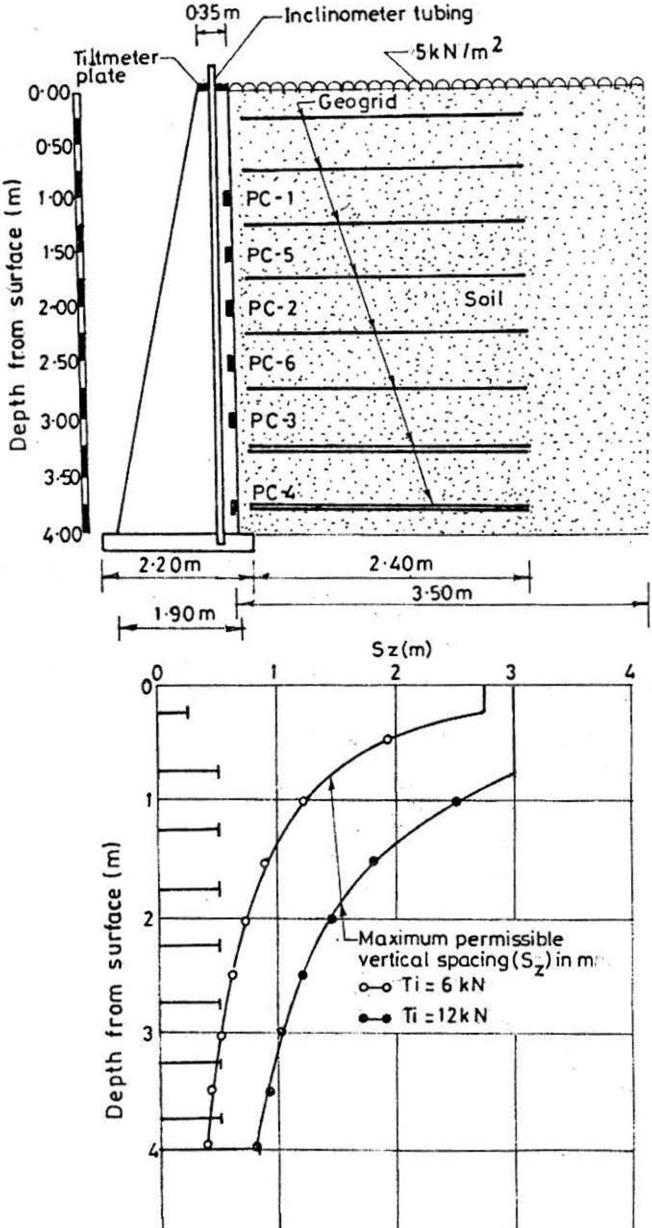

FIGURE 4 : Details of Instrumentation and Geogrid Reinforcement in Case of Test Wall-1. (a) Instrumentation and Geogrid Reinforcement in Position; (b) Permissible Vertical Spacing of Geogrid Reinforcement

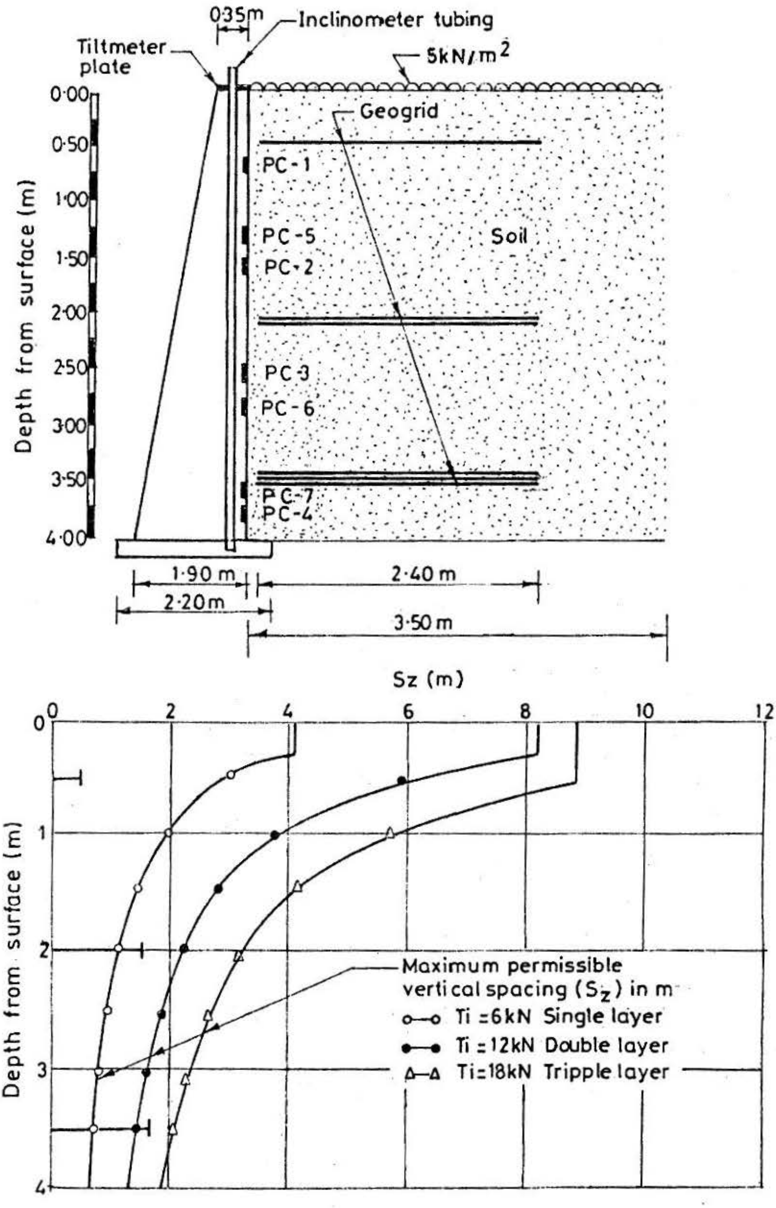

FIGURE 5 : Details of Instrumentation and Geogrid Reinforcement in Case of Test Wall-2. (a) Instrumentation and Geogrid Reinforcement in position; (b) Permissible Vertical Spacing of Geogrid Reinforcement

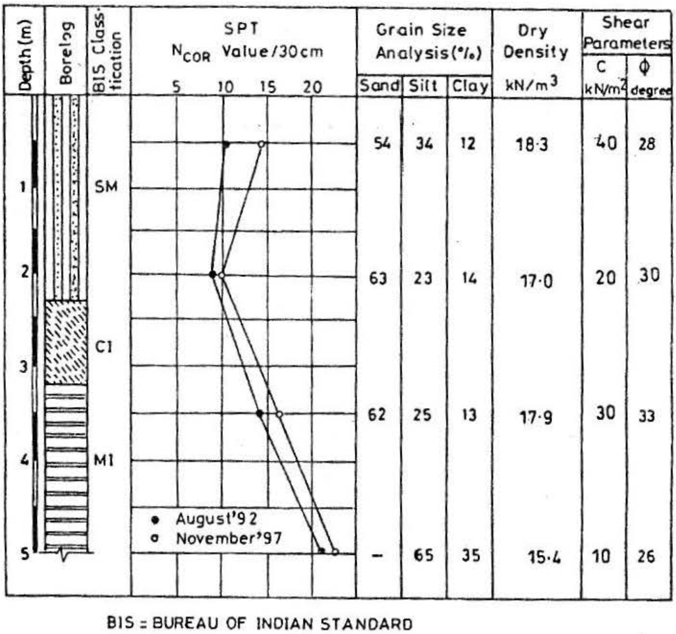

FIGURE 6 : Sub-Soil Profile and Properties of Sub-Soil at Test Site

were stoppages of work in between due to reasons beyond control. Maximum stoppage of work at a time was about 210 days when all the walls were about 2 m in height. Details of rate of loading of the ground due to construction of test facilities and subsequent backfilling operations are provided in Fig.7.

Inclinometer tubings, one in each of the two test walls (wall-1 and wall-2), were installed during the construction of walls. Tiltmeter plates, two in number, were installed on the properly levelled top surface of the test wall-1. Pressure cells on the inner face of both the test walls were installed after their construction was over. All the wire leads of the pressure cells were taken through small size plastic tubes to their respective junction box. One junction box was built for each of the two test walls. One brick masonry column of 45 cm × 45 cm in size was built by the side of each of the two outer side retaining walls (wall-A and wall-C). The height of each column was 4 m. These were used to fix reference datum to measure the tilt of the wall-A and wall-C separately.

Earth backfilling operations were taken up only after the completion of construction of all the five retaining walls to their full heights and after taking initial readings of all the instruments installed on both the test walls

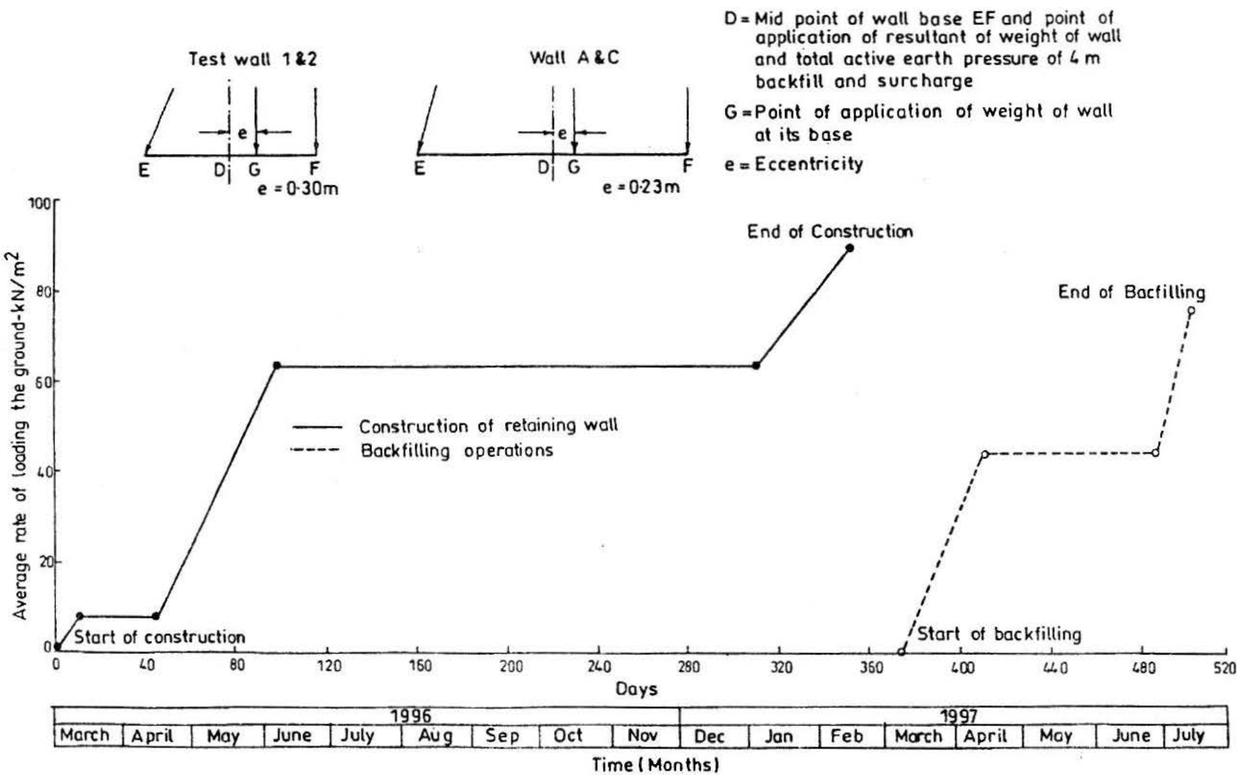

FIGURE 7 : Loading Sequence of Test Site

TABLE 1 : Moisture Content and Density of Backfilled Earth along the Height of Test Retaining Walls (During Backfilling Operation)

Sampling from backfill		Test wall – 1			Test wall – 2		
Date	Height above G.L. (m)	Bulk Density (gm/cc)	Moisture content (%)	Dry Density (gm/cc)	Bulk Density (gm/cc)	Moisture content (%)	Dry Density (gm/cc)
21.2.97	0.15	1.80	10.35	1.63	–	–	–
25.2.97	0.40	1.75	9.40	1.60	–	–	–
25.2.97	0.50	–	–	–	1.81	12.02	1.62
27.2.97	0.75	1.82	12.45	1.62	–	–	–
4.3.97	1.00	–	–	–	1.81	12.00	1.62
7.3.97	1.50	–	–	–	1.82	12.10	1.62
10.3.97	1.75	1.82	11.75	1.63	–	–	–
14.3.97	2.00	–	–	–	1.78	14.30	1.56
19.3.97	2.25	1.79	13.35	1.58	–	–	–
2.7.97	3.25	1.81	12.40	1.61	–	–	–
3.7.97	3.50	–	–	–	1.74	9.15	1.59
4.7.97	3.75	1.72	8.80	1.58	–	–	–
Average dry density				1.607	–	–	1.602

and two outer side walls. The soil, at the back of retaining walls was filled at its natural moisture content (varying from 9 to 14 per cent), in layers of 30 cm thickness. Representative soil samples, from each compacted layer, were collected randomly and tested in the laboratory. Minimum number of such samples invariably ranged between 5 to 8. An average value of each item is reported in the Table 1. Laboratory experiments yielded a maximum dry density of 16.40 kN/m^3 corresponding to an optimum moisture content of 11.00 percent. Earth brought for backfilling purpose on different days was analysed for its grain size distribution and was classified as per BIS 1498-1970.

Instrumentation

Pressure Cells

Strain gauge type pressure cells, 37 mm in diameter and 10 mm in thickness, were installed on the inner faces of wall-1 and wall-2 to monitor the pattern of active earth pressure distribution along the height of each test wall. In all, six pressure cells on wall-1 and seven pressure cells on wall-2 were installed to record lateral earth pressure.

Inclinometer

The Digitilt Inclinometer was used in this case to measure the lateral movement of the two walls. It is a high precision surveying instrument for measuring displacement or deformation. The instrument was lowered down in grooved plastic inclinometer casing installed in the body of the wall. The Digitilt Inclinometer system consists of four units: the movable borehole sensor, the portable digital indicator, the interconnecting electrical cable and the slope indicator guide casing permanently installed in the structure.

Tiltmeter

The Digitilt Tiltmeter was used to monitor the tilt of the retaining wall-1. It consists of three units: portable tiltmeter sensor, portable digital indicator and ceramic tilt plates. Two ceramic tilt plates, 150 mm in diameter with four cylindrical pegs protruding 13 mm above the surface of plate were permanently attached to the retaining wall by means of grout. It can measure tilt up to $\pm 30°$ from horizontal with a sensitivity of 8 seconds of arc at 0° inclination.

Observations

In all 13 sets of observations were taken with the pressure cells, inclinometers, tiltmeters and deformeter in a span of almost two years, i.e.,

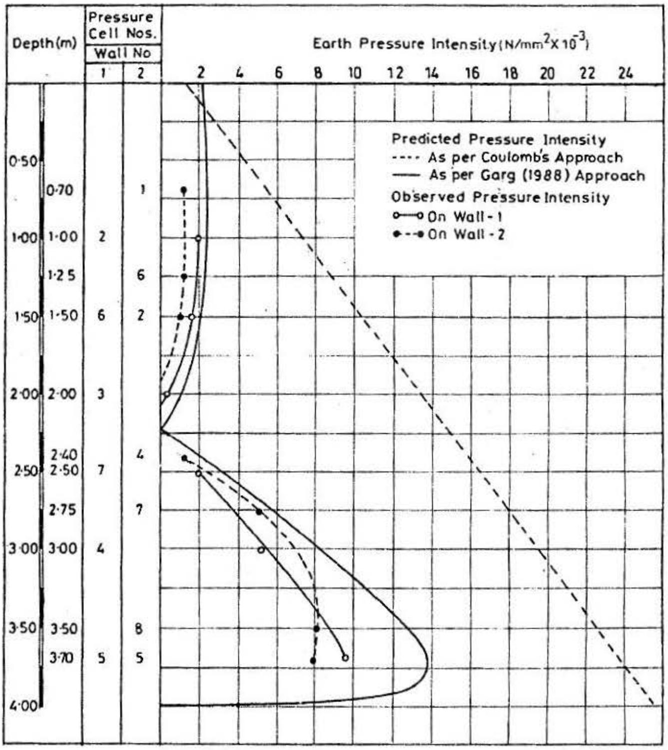

FIGURE 8 : Observed and Predicted Pressure Intensities Variation along the height of Test Walls

from February 97 to March 99. Difference of each subsequent reading w.r.t. its initial reading is worked out and is plotted in respective figures. The observed active earth pressure intensity variations along the height of test walls-1 and wall-2 are provided in Fig.8. The earth pressure is theoretically zero in this case along the height (~ 2.3 m) of walls due to provision of more quantity of reinforcement than required at that place (Garg, 1988). Whereas in practice this situation will never happen. Displacements of test wall-1 and test wall-2, measured with inclino-meter, are provided in Fig.9. Tiltmeter observations on test wall-1 are shown in Fig.10.

Observations of tilt of side-walls (A and C) are given in Fig.11.

Discussion

Active Earth Pressure Intensity

Two curves showing predicted variation of active earth pressure intensity

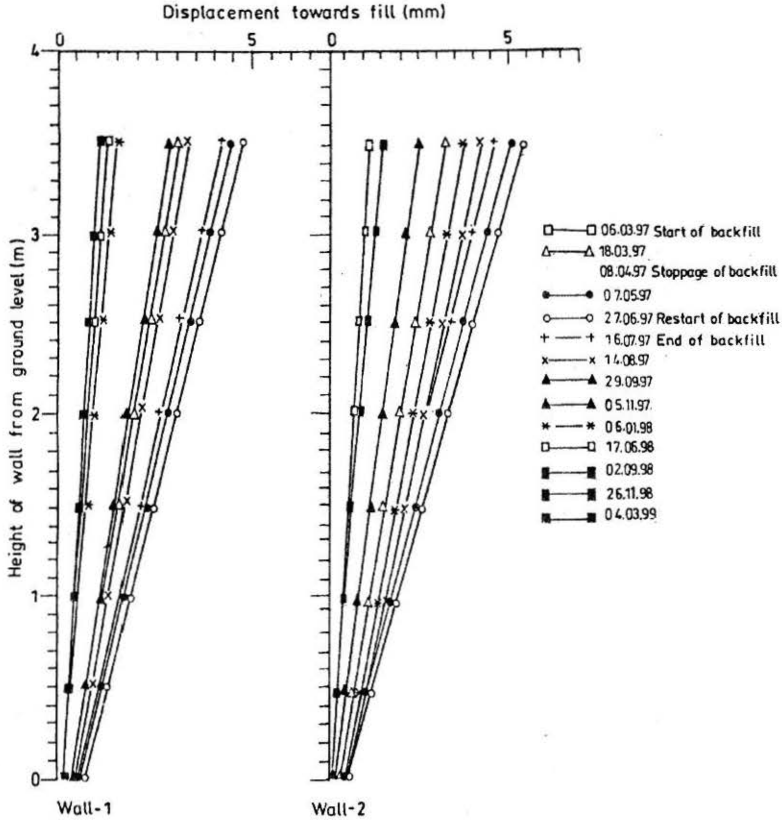

FIGURE 9 : Plots of Inclinometer Data of Test Walls

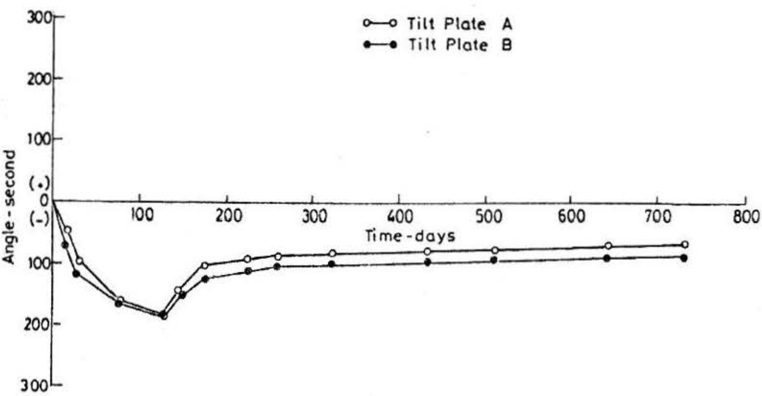

FIGURE 10 : Plots of Observed Tilt of Test Walls

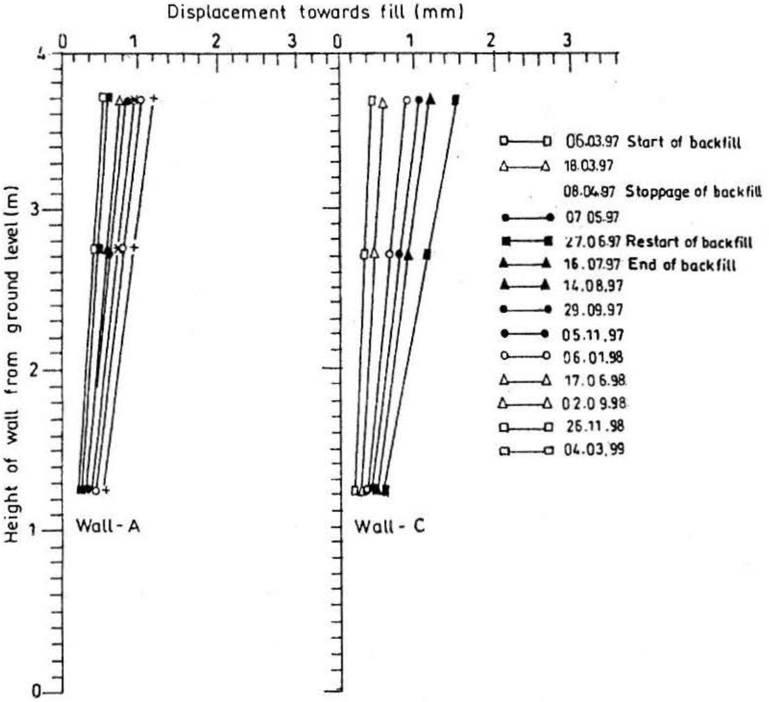

FIGURE 11 : Plots of Observed Lateral Displacements of Side Walls

along the height of the two test walls (wall-1 and wall-2), based on Garg (1988) and Coulomb (1773) approaches, are also provided in Fig.9. It may be noted from Fig.9 that the active earth pressure on a wall retaining reinforced earth fill, is comparatively much less than the active earth pressure due to unreinforced backfill. Both the experimentally observed curves are almost similar in nature and magnitude, indicates that the soil-reinforcement interaction exists effectively for the full depth of vertical spacing (S_z) of 1.50 m. The experimental behaviour of both the test retaining walls compares well with their behaviour predicted by using Garg (1988) approach, suggesting that the developed analytical approach is valid for designing rigid walls with reinforced fill that supports surcharge loading on its surface.

Tilt

Inclinometer observations (Fig.9) on the two test walls indicate that both the test walls have initially tilted inwardly with the magnitude of tilt increasing with increase in the height of walls built. This trend is also confirmed by the tiltmeter observations (Fig.10) on wall-1. The same trend of tilting (Fig.11) was also observed in case of the two side-walls (wall-A and wall-C). But as the backfill was filled almost to its full height, all the walls have started tilting outwardly and a significant proportion of inward tilt was recovered by the time total height of fill was reached and surcharge loading was applied (Figs.9 and 11).

The inward tilting of walls may be due to eccentric loading of the ground by the walls and secondly the test site consists of loose silty sand for a depth of about 2 meters (Fig.6) just below the walls. The magnitude of inward tilt might have been much more than that have taken place, had the construction of all the walls taken place continuously. Due to stoppages of construction work (Fig.7) the ground was loaded in stages at a very slow rate, and that might have helped in improving the ground conditions at the test site.

Conclusions and Recommendations

Based on the outcome of the field study, presented in the paper, on instrumented full size retaining walls with reinforced earth fill supporting surcharge load at its surface, following conclusions and recommendations are made:

1. The analytical approach, for designing rigid walls with reinforced earth fill supporting surcharge load at its surface is validated by the field tests and is recommended for use in appropriate field cases.

2. The proposed design approach provides a conservative estimate of the active earth pressure. Therefore, the walls designed using the proposed analytical approach would always be safe.

3. The extent and pattern of distribution of earth pressure in case of both the test walls (wall - 1 and wall - 2) is almost the same irrespective of the large variation in the vertical spacing (S_z) of the geogrid in the two test cases. This shows that the perfect soil-reinforcement interaction extends over the full depth of larger vertical spacing of 1.5 m also experimented in test wall 2 in this study.

4. The final tilt / displacement of both the test walls at their top as measured by the inclinometer and tiltmeter works out to be 1.67mm and 1.60mm respectively. The two different modes of observations of wall displacement provided a sort of double check on the accuracy of measurements.

Acknowledgement

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Notations

D_p	=	displacement coefficient
f^*	=	coefficient of apparent soil-reinforcement friction
H	=	height of wall
$\bar{H}_q$	=	height of point of application of earth pressure due to surcharge load above base
$\bar{H}_\gamma$	=	height of point of application of earth pressure due to backfill above base
K_a	=	coefficient of active earth pressure
K_q	=	coefficient of active earth pressure for surcharge loading in case of reinforced backfill
K_γ	=	coefficient of active earth pressure for reinforced

backfill

L = total length of reinforcing strip

P = resultant active earth pressure

P_q = resultant active earth pressure due to surcharge loading

P_γ = resultant active earth pressure due to backfill

Py = pressure acting on an element of soil in vertical direction

P_θ = intensity of reaction on failure surface

p = lateral earth pressure intensity on wall

q = intensity of surcharge loading

S_x = horizontal spacing of reinforcement

S_z = vertical spacing of reinforcement

T_i = permissible tensile strength per meter length of geogrid

t = uniformly distributed tensile stress

W = weight of slice or element of soil

w = width of reinforcing strip

y = distance along wall from top

γ_b = average bulk density

δ = angle of sliding friction/angle of wall friction

θ = wedge angle with vertical

μ = coefficient of friction

ϕ = angle of internal friction of soil