*Indian Geotechnical Journal, 28 (3), 1998*

# Experimental Study on Retaining Walls Supported by Vertical Plate Anchors

**K. Rajagopal[†] and V. Sri Hari[‡]**

## Introduction

Retaining walls are required for many civil engineering facilities, e.g. bridge abutments etc. The conventional concrete retaining walls which are based on the gravity principles, are quite expensive to build and need long construction time. Their strength is derived mainly from the mass of the structure developing significant bearing pressures at the base and hence careful design is needed when the subgrade is weak. Because of their rigid nature, these structures cannot withstand differential settlements induced by variations in subgrade strength or due to external disturbances.

In view of the above discussed disadvantages with the rigid concrete type structures, it is advantageous to go in for alternative construction methods which can be built rapidly and economically, and can undergo local over stressing without significant effects. Flexible faced retaining walls made of pre-cast concrete panels supported laterally by tendons anchored within the retained backfill soil provide an alternative to the gravity type retaining walls. Major advantages with this structural methods is the expedient nature of the construction which makes them especially suitable for rehabilitation and reconstruction of highway and railway embankments without interrupting the flow of traffic. Besides these advantages these walls can be built with locally available soil which helps in bringing down the overall construction costs. The flexible nature of these walls allows for large local deformations without catastrophic failures.

---

† Associate Professor, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai – 600 036, India.

‡ M.S. Student (former), Department of Civil Engineering, Indian Institute of Technology Madras, Chennai – 600 036, India.

The concept of anchored retaining walls has received tremendous approval among the practitioners of geotechnical engineering. In recent times many patented forms of anchored retaining wall construction methods have been developed. For example, Public Works Research Institute, Ministry of Construction, Japan has developed a patented form of multianchor type retaining wall construction method. The method consists of installing rows of anchors in the backfill which are connected to the wall panels by thin cables. The backfill is compacted using ordinary roller compactors and hand-held light weight compactors near the wall. This method is routinely used by a Japanese Construction Company Okasan Kogyo Co. Ltd., Tokyo.

A semi-Z shaped mild steel anchor system was patented in India (Singh, 1992a,b) for use in the construction of retaining walls. Singh et al. (1995) reported the design and construction of a 42 m long flexible anchored earth retaining wall embankment in continuation of conventional viaduct, for the new Varuna Bridge of Varanasi.

A large number of researchers have reported their findings on the behaviour of other forms of retaining walls, e.g. tied-back walls and nailed soil walls, e.g. Hanna and Matallana (1970), Hanna and Khurdi (1974), Anderson, Hanna and Shah (1977), Plant (1972), Clough and Tsui (1974), Hua and Shen (1987).

The concept of anchored retaining walls is simple both from the theory and the construction aspects and holds great promise for retaining wall applications. However, in view of limited literature in this area, more research is needed for general application of this technique in India. With this aspect in view, this research work was undertaken. The performance of anchored retaining walls was studied by means of laboratory work. The fundamental aspect of pullout capacity of vertical anchors, which is essential for the design of this class of retaining walls, was studied in a separate paper (Rajagopal and Sri Hari 1998). This paper describes the laboratory studies undertaken in this research and the interpretation of these results.

## Construction of Anchored Retaining Walls

The construction of these walls is easy and needs relatively less construction time. These walls can be constructed using conventional machinery. The various steps involved in the construction of these walls are as follows:

1. A small levelling pad of approximate size of 400 mm width and 200 mm thickness is cast 500 mm below the ground level. This levelling pad has appropriate grooves to support the wall facing and allows for maintaining the level of the wall during the construction. Fig. 1 shows the construction step 1.

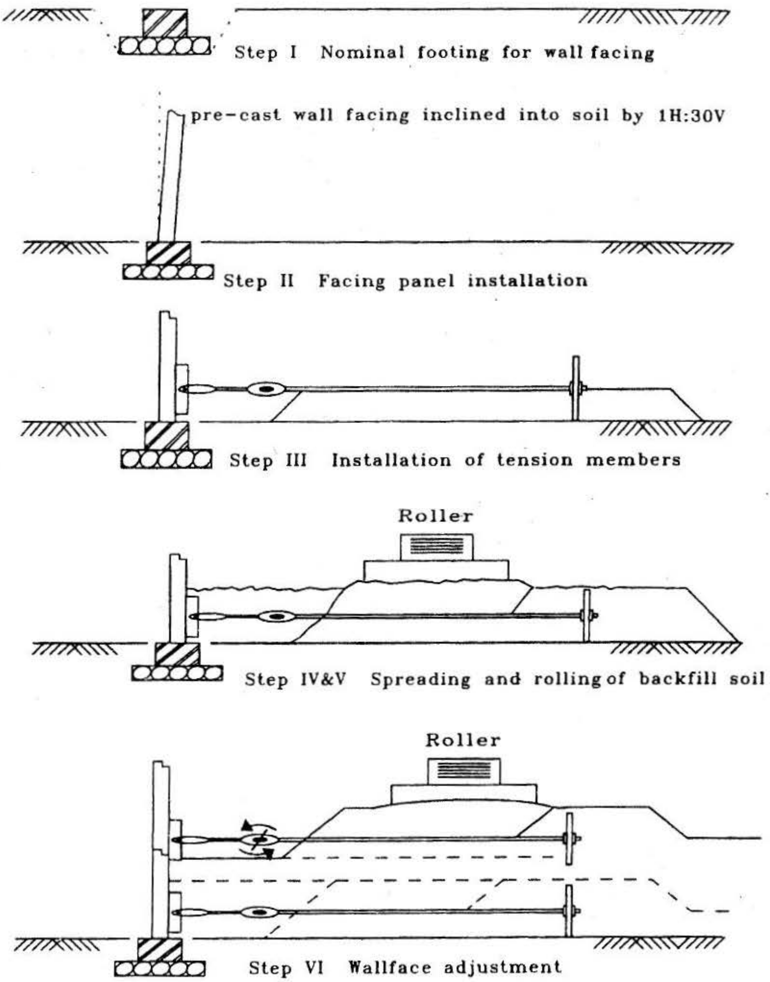



**FIGURE 1 : Construction Procedure of Anchored Retaining Walls**

2. After the construction of the levelling pad, the facing panels (generally made of reinforced concrete) are installed on the marking of the foundation concrete (Step II). These panels are given a slight tilt into the backfill soil of about 2° (lH:30V) so that after the construction induced deformations the wall facing assumes a vertical alignment. These panels have installation fittings to hold the tie-rods connecting the panels and the anchor plates.

3. The first row of anchor plates are then placed in position and the tie-rods extending from the wall facing are connected to the anchor plates

through the installation fitting arrangement. The concrete panels with the tie-rods and the anchor plates are kept in position (Step III).

4. The soil is spread from the anchor plate side to the panel side parallel to the wall (Step IV). Care is taken to see that the panels are not pushed in the outward direction while spreading the soil.

5. The soil thus spread is rolled using conventional rollers. The soil within a distance of 1 m from the facing is rolled using light weight rollers (Step V).

6. The spreading of the soil and rolling is continued till the next level of tie-rods and anchor plates is reached. The tie-rods with anchor plates are placed in position and the soil is spread and rolled up to the level of tie-rods. The next row of wall facing panels are then installed and adjusted over the existing row of panels (Step VI).

7. The above procedure is repeated till the full height of the wall is reached.

## Model Retaining Wall Tests

There are very few publications in the literature on the behaviour of this class of retaining walls. Most of the published papers in this area have considered only the behaviour of ground anchor supported (tied-back) walls or nailed soil walls. The laboratory investigations have been taken up to address some of the issues in the behaviour of this class of retaining walls.

The model retaining walls in the present study are supported on a rigid base. This condition may be similar to a case when the foundation soil below the retaining wall is stiff. The wall is made of a thin aluminium plate and is supported laterally by means of vertical anchor plates.

### *Test Facility*

The test facility for conducting the model tests on retaining walls consists of a test tank, a loading frame, a pre-calibrated proving ring, a hydraulic jack with a pump, a strain meter with a switching unit and load rings to measure the anchor forces. The test tank was 800 mm long, 600 mm wide and 800 mm in height. The loading frame was designed to support loads up to 250 kN. A proving ring of 50 kN capacity was used during the tests to measure the surcharge loads applied on the surface of the backfill. A 250 kN hydraulic jack and pump arrangement was used for applying the required amount of surcharge load. Figure 2 shows the test facility with all the accessories.

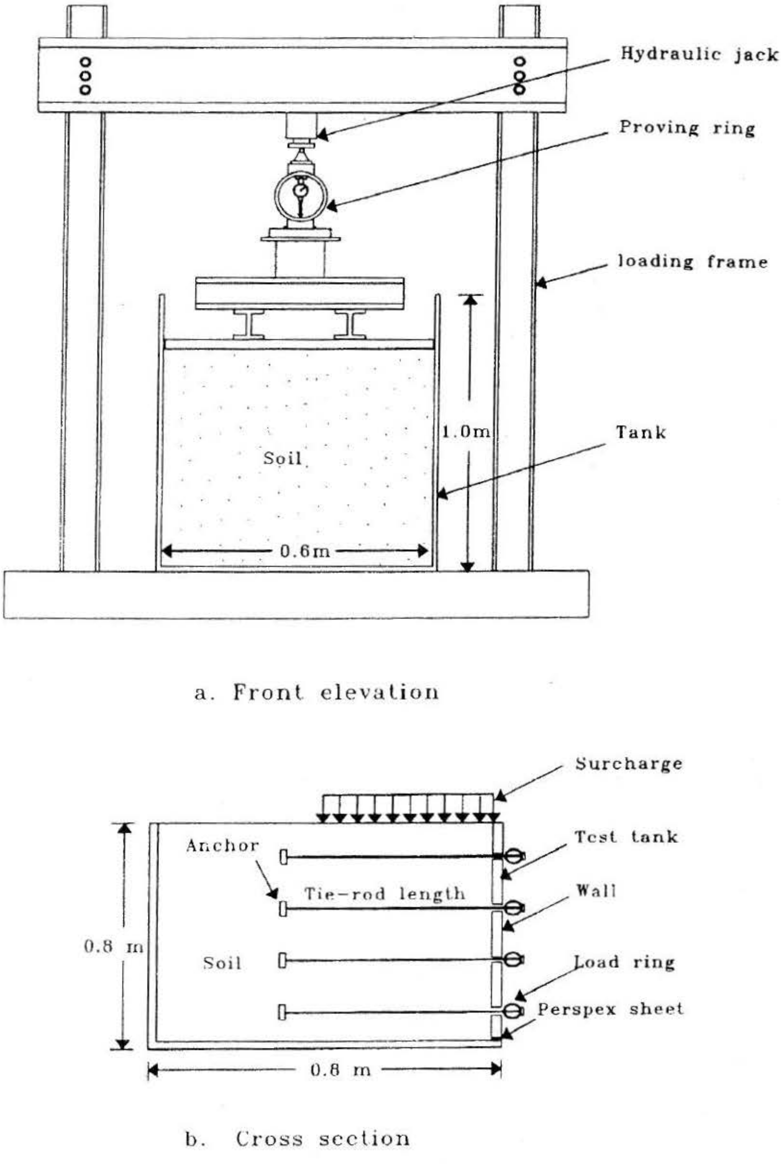



**FIGURE 2 : Schematic of Experimental Setup for Retaining Wall Test**

An aluminium plate of 4.75 mm thickness was used as the model retaining wall for the tests. The model wall (aluminium plate) was 800 mm in height. The width of the facing plate was slightly less than the width of the wall (600 mm) to permit free lateral movements. The tie-rods connecting the anchor plates and the model wall were made of mild steel and were 6 mm in diameter. The length of the tie-rods in the tests was varied from 300 to 500 mm to study the influence of the location of anchors on the behaviour of the wall.

One of the important parameters that needs to be examined during the experimental studies is the passive resistance developed by the anchors in the walls. The anchors were connected to the wall facing through pre-calibrated mild steel load rings to measure the anchor forces. The load rings were made of 30 mm wide pieces of mild steel pipe having an internal diameter of 70 mm and 2 mm thickness. The tie-rods from the anchors were connected to these rings by passing them through diametrically opposite holes as shown in Fig. 2b. Because of this arrangement, the passive force on the anchors is transferred into the load rings as a compression force. The strain induced in these rings due to this compression force gives an indication of the force developed in the anchors. This principle was used to measure the loads developed in the anchors. Resistance type strain gauges were fixed on diametrically opposite sides of these load rings using araldite glue after treating the surface with a primer solution.

A number of resistance type strain gauges were fixed at various heights at the centre of the facing plate to measure the bending moments that develop in the plate during the surcharge load tests.

### *Method of Construction*

Initially, the wall facing was put in place within the test tank and was supported by a clamp arrangement until the end of construction. The verticality of the wall facing was checked using a plumb bob at all stages of construction. The lower end of the wall was made to rest on a perspex sheet in order to allow the wall to move freely in the forward direction during the test. This arrangement was used to avoid the development of lateral frictional force component at the lower end of the wall. Similarly, special care was taken to reduce the influence of side wall friction on the stability of the wall. The side walls of the test tank were lined with two layers of grease coated plastic sheets. These special measures ensure that the lateral support of the wall is derived from the anchor capacities only.

The backfill sand was placed in the tank using the sand raining technique. The properties of this sand for various heights of fall have been reported elsewhere (Rajagopal and Sri Hari 1998). When the level of sand reached the mid-depth of anchor embedment, the anchor was placed in the backfill soil in a vertical position. The anchors were connected to the wall facing through 6 mm diameter tie-rods and pre-calibrated load rings as shown in Fig. 2b. After the anchor and the tie-rod were connected to the facing, further layers of sand were placed in the tank till the next higher level of anchors and the same process was repeated till the sand was placed to the full depth of the tank, i.e. 800 mm. After all the anchors were put in place, the clamps supporting the wall facing were removed.

The surcharge pressure was applied over the full width of the test tank. This pressure was uniformly distributed over the entire area by applying the load through steel channel sections and mild steel plate, Fig. 2a. The load on the backfill soil was measured through a proving ring placed over the channel sections. During the load application, the compression of Styrofoam sheet ensures that the steel loading plate is in contact with the entire loaded area.

### *Different Anchor Layouts*

The testing program has considered the influence of different lengths and inclination of tierods, shapes and sizes of anchors and the strength of backfill soil on the performance of retaining walls supported by vertical plate anchors. In addition, the investigation has also considered the influence of the number of anchors and their location within the wall. Three lengths of tie-rods, viz. 300, 400 and 500 mm were considered. The tests considered various sizes of anchors, viz. 25 mm, 35 mm and 50 mm square anchors and 100 × 50 mm rectangular anchors. Two different relative densities 20% and 41.2%, corresponding to friction angles of 30° and 33° were considered in these studies. Two inclinations of tie-rods were considered. In the case of inclined tie-rods, the plate anchors were placed perpendicular to their axis.

The number of anchors in each wall and their locations were varied by arranging the anchors in different number of rows and columns. To study the influence of these parameters on the surcharge carrying capacity, a total of nine (9) layouts as shown in Fig. 3 were examined during the tests. The configurations 1, 2, 3, 7, 8 and 9 will help in understanding the influence of anchor spacings on the surcharge carrying capacity of the model walls whereas the configurations 4,5 and 6 help in understanding the aspects related to the location of the anchors.

In some of the configurations, it was not possible to test walls with larger size anchors because of the limitations of the test facility as discussed previously. For example, in the case of rectangular anchors of size 100 × 50 mm, only three types of configurations 1, 7 and 8 were possible. In the case of 50 mm square anchors (and equivalent circular anchors), it was not possible to test layouts 3 and 9. However, in the case of 25 and 35 mm size square anchors, it was possible to study the influence of all the nine layouts.

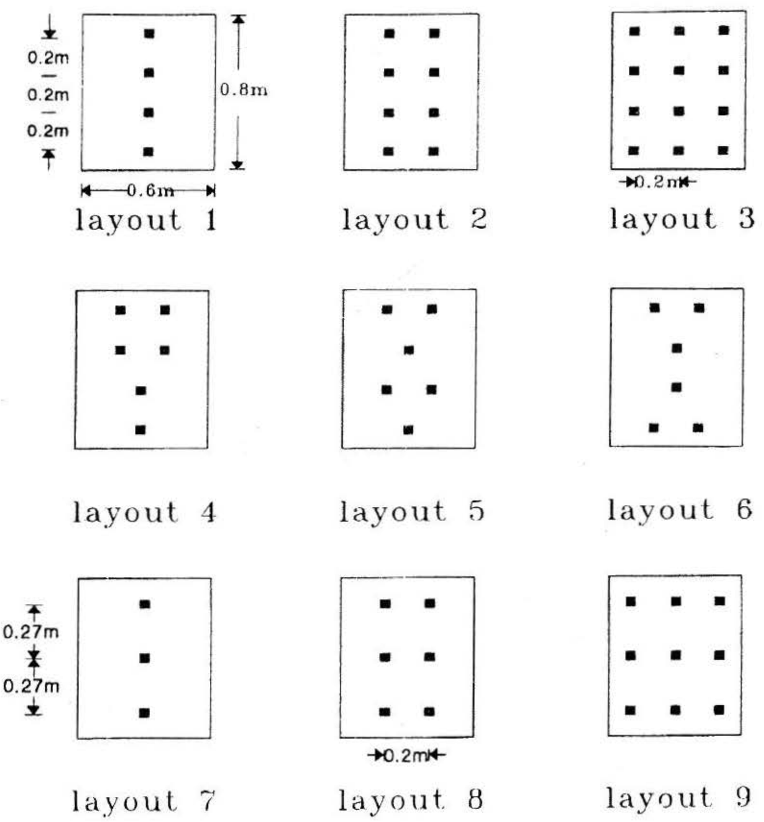



**FIGURE 3 : Anchor Layout for Retaining Wall Tests**

### *Surcharge Load Application*

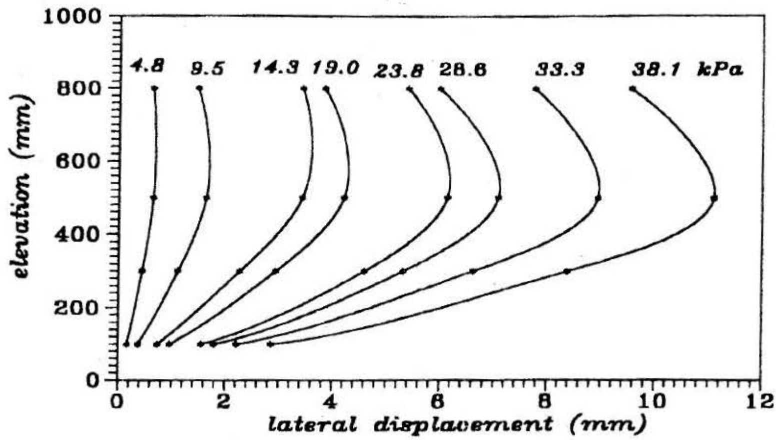
The surcharge pressures were applied on the backfill surface through a flexible Styrofoam sheet of 30 mm thickness to maintain the contact between the loading plate and the soil at all stages of loading. True plane strain conditions were created within the test tank by applying the surcharge pressure over its full width. In the first few trials of the retaining wall tests, the surcharge pressure was applied on the entire backfill surface. This arrangement needed very high surcharge pressures to fail the retaining wall. However, the side walls of the test tank bulged out excessively under high surcharge pressures which prevented the application of full surcharge pressure until the collapse of retaining walls. Even after the side walls were further stiffened with more lateral supports, the retaining walls could not be subjected to failure within the capacity of the tank for this load arrangement. Internal lateral ties were not provided within the tank to prevent the lateral bulging of side walls as these ties themselves may act as passive anchors thus influencing the test results. Hence, it was decided to limit the surcharge pressure to a distance of 0.35 m (0.44 H) behind the wall facing. This configuration is more realistic as loading on most of the retaining walls in the field is applied close to the wall facing e.g. from railway track, vehicular traffic etc.

The surcharge pressure was uniformly distributed over the entire loading area by applying the load from jack to the soil through steel channel sections and mild steel plate, Fig. 2a. The load on the backfill soil was measured through a proving ring placed over the channel sections. During the load application, the compression of Styrofoam sheet ensures that the steel loading plate is in contact with the entire loaded area.

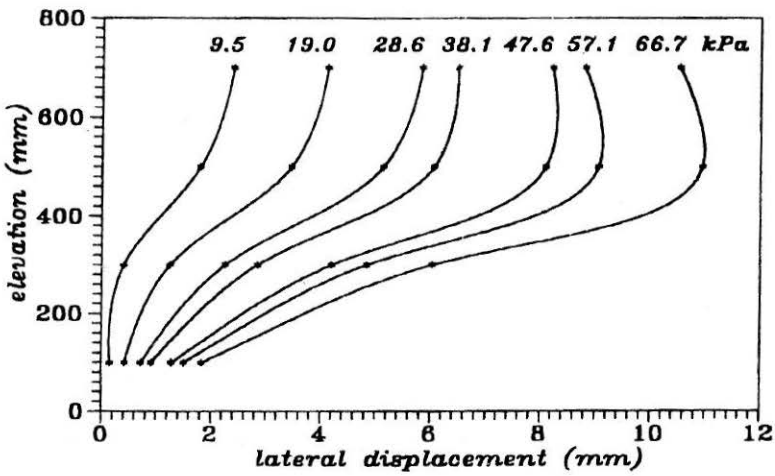
### *Test Procedure*

The initial readings of strain gauges fixed on the load rings and on the wall facing were taken prior to their installation in the tank. Their strains were continuously recorded as the backfill soil was poured into the tank. After the backfill soil was poured to the full depth of tank and the anchors were in place, four dial gauges were fixed at different elevations at the front facing to measure the lateral displacements of the wall. Then the clamps which supported the wall during the construction were removed. Later, the dial gauge readings and the strain readings in various strain gauges were recorded as the readings corresponding to the end of construction stage. The strain gauge readings were recorded using an HBM 75 channel strain meter which can be switched from one channel to the other.

After all the readings corresponding to the end of construction were recorded, the surcharge load was applied on the backfill as discussed in the previous sub-section. The surcharge loading was applied in small increments. Each increment of load was kept constant until the lateral deformations of the wall facing under the surcharge load increment have ceased. At the end of each load increment the lateral deformations of wall facing and the strain readings in various strain gauges were taken. The surcharge pressure increments were applied until the collapse of the wall. The collapse of the wall was defined as the stage at which large lateral deformations occurred when further surcharge increments were applied. The readings which were taken just prior to the collapse of the wall were considered as those corresponding to the collapse stage of the wall. The test was terminated at that stage.




a. layout 8, 35 mm square anchors




b. layout 2, 50 mm square anchors

**FIGURE 4 : Typical Lateral Deformations of Wall Panel at different Surcharge Levels**

## Results and Discussions

A summary of the results obtained from various retaining wall tests is shown in Tables 1 and 2. Typical lateral deformations of walls and anchor forces developed at different surcharge levels are shown in Figs. 4 and 5. Some of these walls have collapsed at the end of construction after the clamp supports were removed whereas the others have failed

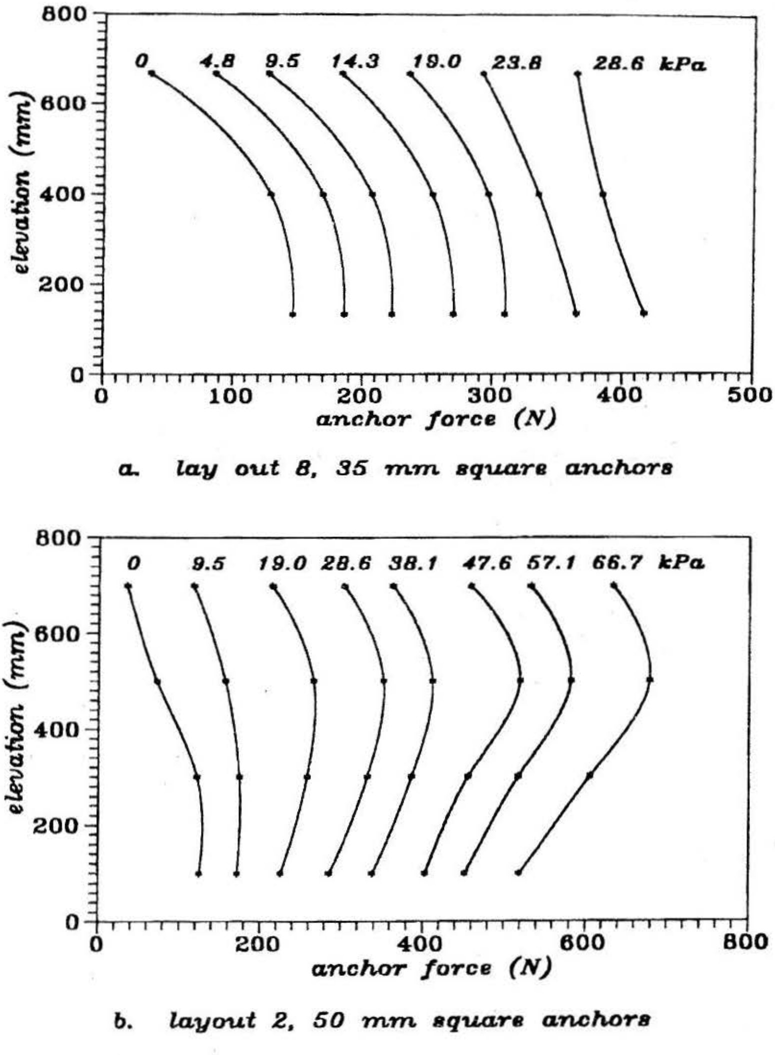



FIGURE 5 : Typical Force Developed in the Tie-rods at different Surcharge Levels

after applying surcharge pressure on the surface of the backfill. The walls did not show appreciable lateral deformation at the end of construction and hence these deformations are not shown in Fig. 4. The deformations are higher in the upper parts of the walls as illustrated in Fig. 4. In general, the anchor forces at the end of construction have linearly increased with depth. Under higher surcharge loads, the anchors near the surface have developed more force compared to the anchors located at the bottom levels. This is because the effect of surcharge applied over a

**Table 1**
**Results from Model Retaining Wall Tests with 400 mm long tie rods**
**($D_r$ = 41.2%, $\phi$ = 33°)**

| Anchor size (BxH mm) | Layout type | Surcharge Pressure ($kN/m^2$) | Sum of Anchor Force (kN) | $k \cong \frac{\sum P_i}{\left[\frac{1}{2}\gamma H_w^2 B_w + Q\right]}$ | $\frac{\delta_{max}}{H}$ (%) |
|---|---|---|---|---|---|
| 25x25 | 1 | ** | ** | ** | ** |
| | 2 | 18.65 | 1.84 | 0.267 | 16.41 |
| | 3 | 32.7 | 2.72 | 0.276 | 12.67 |
| | 7 | ** | ** | ** | ** |
| | 8 | 11.67 | 1.35 | 0.249 | 11.32 |
| | 9 | 16.37 | 1.66 | 0.259 | 8.74 |
| 35x35 | 1 | 9.52 | ** | ** | ** |
| | 2 | 37.40 | 2.90 | 0.268 | 9.56 |
| | 3 | 65.37 | 4.68 | 0.280 | 9.31 |
| | 7 | ** | ** | ** | ** |
| | 8 | 28.07 | 2.33 | 0.263 | 12.00 |
| | 9 | 42.06 | 3.21 | 0.272 | 10.56 |
| 50x50 | 1 | 28.6 | 2.42 | 0.269 | 12.37 |
| | 2 | 65.37 | 4.87 | 0.292 | 10.54 |
| | 4 | 57.14 | 4.74 | 0.317 | 10.60 |
| | 5 | 47.63 | 3.60 | 0.277 | 8.60 |
| | 6 | 38.10 | 3.10 | 0.282 | 13.60 |
| | 7 | 13.99 | 1.56 | 0.264 | 17.82 |
| | 8 | 56.05 | 4.25 | 0.288 | 12.63 |
| 100x50 | 1 | 46.72 | 3.4 | 0.266 | 9.74 |
| | 7 | 18.65 | 1.69 | 0.245 | 11.76 |
| | 8 | 84.11 | 6.08 | 0.295 | 6.64 |
| 56.4 mm Circular | 1 | 28.57 | 2.61 | 0.291 | 12.50 |
| | 2 | 66.70 | 4.97 | 0.293 | 9.26 |
| | 7 | 16.70 | 1.71 | 0.262 | 14.3 |

** walls collapsed during construction

part of the backfill is more on the upper parts of the wall than towards the base portion. This factor has to be accounted for in the design of retaining walls supported by plate anchors.

Various tests were carried out to study the influence of different parameters such as the size and location of anchors, length and inclination of tie-rods. The following subsections discuss the results obtained from these tests.

**Table 2**
**Influence of Various Parameters on the Performance of Anchored Retaining Wall**

| Parameter | Anchor size BxH mm | Layout type | Surcharge ($kN/m^2$) |
|---|---|---|---|
| 20% relative density | 25 × 25 | 8 | 7.13 |
| $\gamma$ = 15.05 $kN/m^3$ | 25 × 25 | 9 | 11.89 |
| L = 400 mm | 35 × 35 | 8 | 19.02 |
| $\phi$ = 30° | 35 × 35 | 9 | 28.53 |
| | 50 × 50 | 8 | 30.90 |
| | 50 × 50 | 9 | 65.91 |
| 500 mm long tie-rods | 25 × 25 | 1 | 1.83 |
| $D_r$ = 41.2% | 25 × 25 | 8 | 14.26 |
| $\gamma$ = 15.5 $kN/m^3$ | 35 × 35 | 1 | 11.89 |
| $\phi$ = 33° | 35 × 35 | 8 | 28.53 |
| | 50 × 50 | 1 | 33.29 |
| | 50 × 50 | 8 | 66.65 |
| 10° tie-rod inclination | 35 × 35 | 1 | 46.26 |
| $D_r$ = 41.2%, $\phi$ = 33° | 35 × 35 | 2 | 57.14 |
| $\gamma$ = 15.5 $kN/m^3$ | 35 × 35 | 3 | 99.66 |

### *Influence of Length of Tie-Rods*

Three lengths of tie rods, viz. 300, 400 and 500 mm were considered in these tests. The retaining walls with 300 mm tie-rod length and all nine anchor layouts have collapsed at the end of construction after the clamp supports were removed or after the application of small surcharge pressures. The conventional Rankine failure plane drawn at an angle of $(45 + \phi/2)°$ from the base intersects the backfill surface at 0.43 m from wall facing. For layouts with four rows of anchors the upper two rows of anchors lie within the failure wedge whereas for layouts with three rows of anchors the top most row of anchors lie within the failure wedge. These anchors which lie within the failure wedge were not able to mobilise enough pullout capacity to support the wall or their capacity was so low that the walls collapsed under small surcharge pressures. This result clearly illustrates that the anchors for these retaining walls should be located away from the Rankine failure wedge.

Most of the retaining walls constructed with 400 mm long tie-rods were able to support significant surcharge loads before collapsing as shown in Table 1. Only for a few layouts, these walls have collapsed at the end of construction. These walls have been discussed under a separate sub-section later in this paper. A few tests were repeated with longer tierods of 500 mm length. The results from these tests are almost similar to those from tests on walls with 400 mm long tie-rods. Hence, they are not presented in the paper. In the case of 25 and 35 mm square anchors, the increase in tie-rod length has not significantly improved the capacity of the walls. In the case of 50 mm size square anchors, the surcharge carrying capacity of the wall has increased by approximately 6% and 18% for the two cases tested. This slight difference in performance can be attributed to the experimental variations. Hence, it can be concluded that the increase in tie-rod length beyond the Rankine failure wedge does not substantially improve the performance of these walls. The results obtained from this series of tests indicate that the minimum length of tie-rod should be chosen taking into account the width of the Rankine failure wedge within the backfill. As the width of failure wedge varies from zero at the base to maximum at the top of wall, it is possible to provide shorter lengths of tie-rods towards the bottom of wall. However, using such differential lengths of tie-rods at different heights may require careful field supervision and may lead to logistic problems. The savings that can be obtained by shortening the tie-rod lengths may not be much while any mistakes in choosing proper lengths of tie-rods at different heights may lead to collapse of wall. Hence, it may be more simpler and conservative to use constant lengths of tie-rods over the full height of wall.

### *Effect of Shape of Anchors*

Three different sizes of square anchors were tested in the model retaining walls. Some of these walls collapsed during the construction stage under the self weight of the soil. In the case of 25 and 35 mm square anchors the configurations of anchors tested were 1, 2, 3, 7, 8 and 9. However in the case of 50 mm square anchors the configurations tested were 1, 2, 4, 5, 6, 7 and 8. Two typical lateral displacements of wall facing and anchor forces at various surcharge levels are shown in Fig.s 4 and 5. A summary of all the test data generated from these experiments is given in Table 1.

A few tests were repeated with circular anchors of 56.4 mm which have the same surface area as those of 50 mm square anchors. The performance of these walls was found to be almost similar to that of 50 mm square anchors as shown in Table 1.

Some tests were also performed with 100 × 50 mm rectangular anchors. In terms of the surface area of anchors, the rectangular anchors have double the surface area of 50 mm square anchors. Only three configurations of anchor layouts were possible in this set of tests because of the limitation on the maximum load that can be applied on the test tank. The configurations of anchors tested were 1, 7 and 9. The results obtained from these tests are also tabulated in Table 1.

A comparison of test results obtained for walls supported with rectangular and 50 mm square anchors showed that the increase in capacity is only marginal and is much less than the increase in the surface area of anchors. Although the surface area of rectangular anchors is double that of 50 mm square anchors, the surcharge load capacity is only marginally higher i.e. the efficiency of rectangular anchors is less than that of square anchors. This result is consistent with that observed in the case of pullout capacity of anchors reported earlier by Rajagopal and Sri Hari (1998).

*Effect of Density of the Soil*

One of the important parameters influencing the behaviour of these walls is the relative density of the soil. The backfill soil was placed at two relative densities corresponding to two heights of fall of 100 and 200 mm corresponding to 20 and 41.2% relative densities. The unit weights of the sand at the two relative densities were 15.05 and 15.5 kN/m3 respectively. The friction angles of soil corresponding to these relative density are 30° and 33°. The surcharge load carrying capacity of the wall increased with the increase in the relative density of the sand as illustrated in Tables 1 and 2.

The increase in surcharge capacity of walls due to the increase in friction angle of backfill soil could be attributed to the following two factors:

i) reduction in lateral earth pressures, and
ii) increase in pullout capacity of anchors.

*Effect of Inclination of Tie-rods*

The influence of the inclination of tie-rods on the load carrying capacity of the walls was studied by conducting tests with tie-rods placed at two inclinations, one in horizontal position and the other at 10° below the horizontal. The results are presented in Tables 1 and 2. It can be observed that there is a slight increase in the surcharge capacity due to the inclination of tie-rods. This increase could be due to the passive earth pressure developed by the soil against inclined tie-rod and the increase in anchor capacity.

*Effect of the Location of Anchors*

Surcharge capacity was found to depend not only on the size and number of anchors but also on the location of the anchors. This result is

**Table 3**
**Surcharge Pressures at Failure for Different Layouts**

| Anchor size (L × H) mm | Layout Number | Total area of anchors ($mm^2$) | Surcharge ($kN/m^2$) |
|---|---|---|---|
| 50 × 50 | 4 | 15000 | 57.14 |
| | 5 | 15000 | 47.63 |
| | 6 | 15000 | 38.10 |
| 50 × 50 | 7 | 7500 | 13.99 |
| 25 × 25 | 3 | 7500 | 32.7 |

clearly illustrated by the surcharge capacities of layouts 4, 5 and 6 for 50 mm square anchors. The same can also be interpreted from the results of 50 mm square anchors in layout 7 and 25 mm square anchors in layout 3. In both the sets, the total anchor plate area provided was the same but there is a substantial difference in the surcharge capacity of these walls as illustrated in Table 3. From the results presented, it can be observed that the configurations with more anchor area towards the upper parts of the wall have higher surcharge capacities. This is because the lateral pressure from surcharge applied on part of the backfill surface is higher at the upper parts of wall than at the bottom (refer Fig. 5b). Hence, the walls which have higher anchor area in the regions with larger lateral forces have shown higher surcharge capacities.

### *Analysis of Collapsed Walls*

As stated earlier, some of the walls collapsed at the end of construction or on application of very small surcharge pressures. The analysis of these walls was performed to understand the failure mechanism of these walls in general. The walls constructed using 25 mm square anchors in layout 1 and 7 and 35 mm anchors in layout 7 had failed at the end of construction. Both the anchor layouts had one column of anchors. Whereas the layout 1 had four rows of anchors, the layout 7 had only three rows of anchors. All these walls were constructed with backfill soil at a relative density of 41.2% ($\phi$ = 33°) and a unit weight of 15.5 $kN/m^3$.

The cause for the above failures can be analysed easily by equating the lateral earth force due to the self weight of the soil and the sum of pullout capacities of the anchors. The pullout capacities of the anchors can be estimated using Eqns. 1 and 2 for shallow ($E_r < 15$) and deep embedment depths ($E_r > 15$) which were reported earlier by Rajagopal and Sri Hari (1998).

$$\frac{P}{L} = S_a C\left[1+\frac{H}{L}\right]^m \gamma H^2 \left(\frac{\sigma_v}{\gamma H}\right)^n K_p^q \tag{1}$$

$$\frac{P}{L} = S_a C\left[1+\frac{H}{L}\right]^m \gamma H^2 \left\{15^n + \left(\frac{\sigma_v}{\gamma H} - 15\right)^r\right\} K_p^q \tag{2}$$

in which
$P$ = pullout capacity of anchors,
$L$ and $H$ = length and height of anchor,
$\sigma_v$ = vertical pressure at the mid-depth of anchor,
$\gamma$ = unit weight of soil
$K_p$ = Rankine passive pressure coefficient,
$S_a$, $C$, $n$, $r$ and $q$ = constants.

The predominant lateral forces in the wall at the time of collapse are due to earth pressures and the anchor forces. The force due to the earth pressures result in collapse of wall while the forces from anchors prevent the wall from collapsing. Considering the horizontal equilibrium just prior to collapse, the stabilising forces should be equal to the destabilising forces. The total active lateral force acting on the wall which causes collapse of the wall is calculated as

$$P_a = \frac{1}{2} K_a \gamma H_w^2 L_w \tag{3}$$

in which
$K_a$ = active earth pressure coefficient,
$\gamma$ = unit weight of the soil,
$H_w$ = height of the wall (0.8 m) and
$L_w$ = width of the test tank (0.6 m).

For the dimensions of the tank and the friction angle of the soil (33°), the active Rankine's lateral earth force is approximately equal to 900 N. If the total pullout capacity of the anchors is less than this active force, the walls would collapse at the end of construction. In the case of 25 mm square anchors for layout 1 which has one column and four rows of anchors (which is more critical than layout 7), the anchor capacities at various depths are calculated using Eqns. 1 and 2 and are given in Table 4. Thus the total pullout resistance the anchors can develop is only 560 N which is much less than the active force of 900 N. leading to the collapse of wall.

Similarly in the case of walls supported with 35 mm size square anchors for layout 7 which has one column and three rows the anchor capacities at

**Table 4**
**Estimated anchor capacities for collapsed wall with 25 mm size anchors**

| S No. | Embedment depth (mm) | Embedment ratio ($E_r$) | Estimated capacity (N) |
|---|---|---|---|
| 1 | 100 | 4 | 25 |
| 2 | 300 | 12 | 125 |
| 3 | 500 | 20 | 190 |
| 4 | 700 | 28 | 220 |
| | | | Total = 560 < 900 |

various depths of anchor locations were calculated using Eqns. 2 and 3 and are given in Table 5. The sum of the anchor pullout capacities in this case is 905 N which is nearly equal to that of the active lateral force acting on the wall and as a result the wall collapsed immediately after construction. Hence, the sum of the anchor forces developed in the wall should always be greater than the force due to the lateral earth pressures of bacldill soil in the wall.

## Overall Discussion of Test Results

The results obtained from these tests have given some important details regarding the behaviour of these walls. Some observations from the experimental and finite element analysis studies are as follows:

1. The anchor forces at the end of construction due to the self weight of backfill soil increased from top to bottom linearly in the same manner as the earth pressure distribution.

**Table 5**
**Estimated anchor capacities for collapsed wall with 35 mm size anchors**

| S No. | Embedment depth (mm) | Embedment ratio ($E_r$) | Estimated capacity (N) |
|---|---|---|---|
| 1 | 133 | 3.8 | 65 |
| 2 | 400 | 11.4 | 325 |
| 3 | 667 | 19.0 | 515 |
| | | | Total = 905 ≅ 900 |

2. The walls constructed with tie rod lengths of 300 mm collapsed during the construction itself. Those constructed with tie rod lengths of 400 and 500 mm did not show appreciable difference in their performance. This result clearly illustrates that the failure plane in the soil lies between 300 and 400 mm behind the wall facing which corresponds to that given by Rankine's theory.

3. Two walls constructed with 25 mm square anchors in layouts 1 and 7 and one wall with 35 mm square anchors in layout 7 have failed at the end of construction. An examination of pullout capacities of anchors for the end of construction state illustrates that the total anchor force developed is equal to the Rankine active lateral force acting on the wall. All other walls have failed at some surcharge pressures applied on the backfill surface.

4. The surcharge capacity of the walls was a function of the shape and size of the anchor, relative density of the soil and location of the anchors.

5. The backward inclination of tie-rods has increased the surcharge capacity of retaining walls. This increase could be due to the additional passive pressures developed in front of the inclined tie-rods and the increase in anchor capacity due to the inclination.

6. In general, the walls with large size anchors failed at higher surcharge loads and the deformations of the walls decreased with increase in the size of the anchors.

7. The lateral deformations at failure of the walls were of the same order at which the anchors have developed peak force during the pullout tests.

8. Surcharge capacity was found to depend not only on the size and number of anchors but also on the location of the anchors. This result is clearly illustrated by the surcharge capacities of layouts 4, 5 and 6 for 50 mm square anchors. The same can also be interpreted from the results of 50 mm square anchors in layout 7 and 25 mm square anchors in layout 3. In both the cases, the total anchor plate area provided was the same but there was a substantial difference in the surcharge capacity of these walls.

9. The surcharge capacity of walls supported by both 50 mm square anchors and circular square anchors having equivalent surface area were found to be almost the same.

10. The surcharge capacity of walls supported with rectangular anchors did

not increase in proportion to their area when compared to the walls supported by square anchors of the same height. Similar result was observed in the pullout tests also.

11. The surcharge capacities per unit anchor area were observed to be lower for retaining walls supported by continuous anchors than those for walls supported by other types of anchors.

The coefficient of lateral earth pressure at collapse state was back-calculated using the anchor forces, total surcharge load applied on the wall and the unit weight of soil using the following relation,

$$K \cong \frac{\sum P_i}{\left[\frac{1}{2}\gamma H_w^2 B_w + Q\right]} \tag{4}$$

in which $\sum Pi$ = sum of measured anchor forces at collapse,
$Q$ = total surcharge force applied on the soil,
$H_w$ and $B_w$ = height and width of wall and
$K$ = lateral earth pressure coefficient.

These back-calculated K values were observed to be close to the Rankine active earth pressure coefficients calculated using the friction angle of the soil as illustrated in Table 6.

## Development of a Design Procedure

As evident from the results of laboratory tests, the surcharge capacity and behaviour of these retaining walls depend on the size, number and location of anchors in the wall. The analysis of walls which collapsed at the end of construction and those which collapsed under the application of surcharge pressures has clearly shown that the active lateral earth pressure conditions prevail at the incipient collapse of these walls.

The data presented above illustrates that the lateral earth pressure at collapse state corresponds to the Rarlkine active earth pressure state. The same may be used in developing a design methodology for these walls. The design consists of choosing the size of the anchor plates and vertical and horizontal spacing of anchors. The design process can be summarised as follows:

- Estimate the total lateral active force on the wall per meter length of wall by considering the unit weight of soil, shear strength properties of soil and any surcharge acting on the surface of the backfill soil.

**Table 6**
**Earth Pressure Coefficients at Collapse of Various Walls**
**($D_r$ = 41.7%, $\phi$ = 33°)**

| Anchor size (L × H mm) | Layout type | Surcharge Pressure (kN/m²) | Sum of Anchor Force (kN) | $k \cong \frac{\sum P_i}{\left[\frac{1}{2}\gamma H_w^2 B_w + Q\right]}$ |
|---|---|---|---|---|
| 25 × 25 | 1 | ** | ** | ** |
| | 2 | 19.0 | 1.84 | 0.264 |
| | 3 | 33.3 | 2.72 | 0.273 |
| | 7 | ** | ** | ** |
| | 8 | 11.9 | 1.35 | 0.247 |
| | 9 | 16.7 | 1.66 | 0.256 |
| 35 × 35 | 1 | 09.5 | ** | ** |
| | 2 | 38.1 | 2.90 | 0.264 |
| | 3 | 66.7 | 4.68 | 0.267 |
| | 7 | ** | ** | ** |
| | 8 | 28.6 | 2.33 | 0.259 |
| | 9 | 42.9 | 3.21 | 0.268 |
| 50 × 50 | 1 | 28.6 | 2.42 | 0.269 |
| | 2 | 66.7 | 4.87 | 0.287 |
| | 4 | 57.1 | 4.19 | 0.280 |
| | 5 | 47.6 | 3.54 | 0.273 |
| | 6 | 38.1 | 3.10 | 0.283 |
| | 7 | 14.3 | 1.56 | 0.261 |
| | 8 | 57.1 | 4.25 | 0.284 |
| 100 × 50 | 1 | 47.6 | 03.4 | 0.262 |
| | 7 | 19.0 | 1.69 | 0.243 |
| | 8 | 85.7 | 6.08 | 0.290 |
| 56.4 mm | 1 | 28.6 | 2.61 | 0.291 |
| Circular | 2 | 66.7 | 4.97 | 0.293 |
| | 7 | 16.7 | 1.81 | 0.279 |

** walls collapsed during construction

- The required lateral movements that the retaining wall has to undergo in order to develop active pressure state in the backfill soil can be estimated as 0.1 to 0.3% of the height of the retaining wall depending upon the type of backfill soil. For clayey soils, the required deformation may be as high as 1% of the wall height. This deformation should be approximately equal to 10% of the anchor height to develop a safe pullout load which is approximately 1/3rd of the pullout capacity. The size of the anchor plate can be decided based on this consideration.

- For the above size of anchor plates, the safe capacity at various levels of retaining walls can be estimated from the equations with a factor of safety of 3.

- The horizontal and vertical spacing can be so chosen that the sum of the pull out capacities of individual anchors is greater than the active force on the wall. If the centre to centre spacing of anchors is less than 3xH, the estimated capacities in the previous step should be reduced to account for the interaction effects.

- If the front facing is made of separate units (as in segmental retaining walls), each unit should be provided with a separate anchor. In such a case, the area of the anchor can be decided based on the size of the facing units and the force coming on each unit.

- The length of tie rods should be chosen such that the anchors are located away from the Rankine failure surface drawn from the base of the wall at an angle of $(45+\phi/2)°$.

The above design procedure is illustrated by giving some design examples for typical field conditions in the following section.

## Design Examples

### *Example 1*

Problem:

It is required to design an anchored retaining wall to retain 5 m height of granular soil which has a unit weight of 18 kN/m$^3$ and a friction angle of 30°.

Solution:

The coefficient of lateral active earth pressure, $K_a = 1/3$.

The total active earth pressure acting on the wall is,

$$P_a = \frac{1}{2} K_a \gamma H_w^2 = 75 \text{kN/m}$$

From empirical data, it can be assumed that a lateral wall deformation of 0.1% to 0.3% of wall height is necessary for developing active earth pressures, i.e. 5 mm to 15 mm of lateral deformation is necessary. As

**Table 7**
**Pullout Capacities of 150 mm Square Anchor at Different Embedment Ratios Design Example No. 1**

| Depth (m) | Embedment ratio ($E_r$) | Pullout load | |
|---|---|---|---|
| | | Ultimate (kN) | Safe (kN) |
| 0.5 | 03.33 | 04.24 | 01.40 |
| 1.0 | 06.67 | 11.70 | 03.90 |
| 1.5 | 10.00 | 21.11 | 07.00 |
| 2.0 | 13.33 | 32.00 | 10.70 |
| 2.5 | 16.67 | 39.30 | 13.10 |
| 3.0 | 20.00 | 41.40 | 13.80 |
| 3.5 | 23.33 | 43.40 | 14.50 |
| 4.0 | 26.67 | 45.30 | 15.10 |
| 4.5 | 30.00 | 47.00 | 15.80 |

the pullout force that is approximately 1/3rd the ultimate pullout capacity of anchors occurs at a deformation of approximately 1/10th the anchor height, the size of anchor can be selected as

$$0.001 \times H_w \text{ to } 0.003 \times H_w$$
$$\cong H/10$$

which gives an anchor size of 50 to 150 mm. Hence 150 mm size square anchors can be provided for this wall. Table 7 shows the capacities of 150 mm square anchors at various embedment depths estimated from Eqns. 1 and 2.

The number of anchors to be provided can be decided from the consideration that the total anchor capacity should be greater than the lateral force on the wall. The anchor arrangement shown in Fig. 6 has a total pullout capacity of 86 kN/m which is greater than the lateral force of 75 kN/m acting on the wall. Hence the design is safe.

*Example 2*

The above retaining wall is re-designed in this example using 100 mm size square anchors to illustrate the effect of size of anchors. Table 8 shows the anchor capacities at various depths of wall. The anchor configuration shown in Fig. 7 has a total pullout capacity of 88 kN/m which is greater than the lateral force of 75 kN/m. Hence the design is safe.

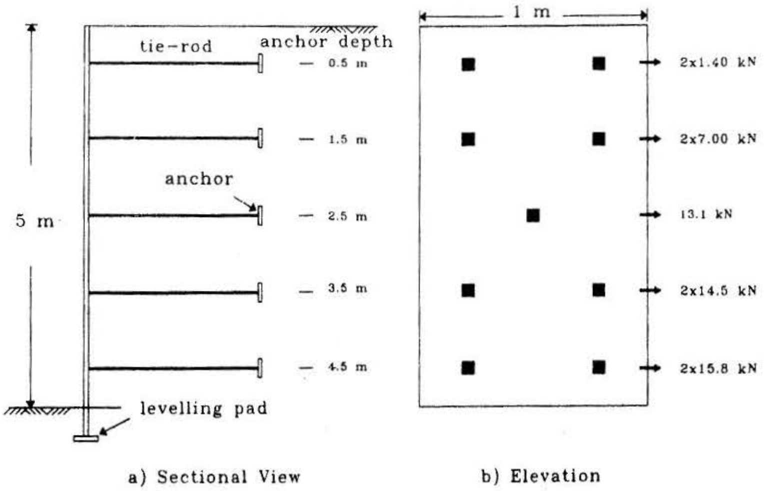



**FIGURE 6 : Anchor Configuration for Retaining Wall in Example-1**

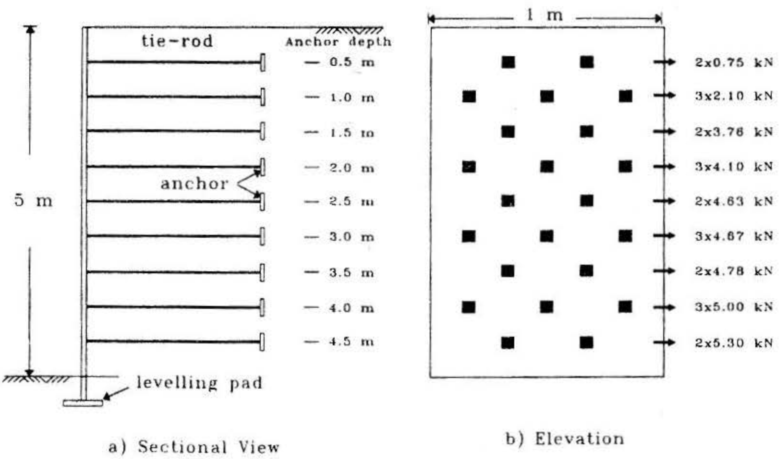



**FIGURE 7 : Anchor Configuration for Retaining Wall in Example-2**

**Table 8**
**Pullout Capacities of 100 mm Square Anchor at Different Embedment Ratios Design Example No. 2**

| Depth (m) | Embedment ratio ($E_r$) | Pullout load | |
|---|---|---|---|
| | | Ultimate (kN) | Safe (kN) |
| 0.5 | 5 | 02.25 | 0.75 |
| 1.0 | 10 | 06.26 | 2.10 |
| 1.5 | 15 | 11.30 | 3.76 |
| 2.0 | 20 | 12.30 | 4.10 |
| 2.5 | 25 | 13.10 | 4.63 |
| 3.0 | 30 | 14.00 | 4.67 |
| 3.5 | 35 | 14.35 | 4.78 |
| 4.0 | 40 | 15.00 | 5.00 |
| 4.5 | 45 | 15.90 | 5.30 |

Though the wall can be designed either way, it is advantageous to go for the smaller size of anchors located all over the retaining wall. The reason being that the wall facing deformations in the case of design with small size anchors is comparatively less.

## Conclusions

This paper has discussed results from laboratory model tests on retaining walls supported by vertical plate anchors. It was found that the lateral earth pressures and internal failure plane behind these walls correspond to that given by Rankine active state. The walls supported by square shaped anchors had higher unit capacity than those supported by rectangular shaped anchors. The length of tie-rods should be chosen such that the anchors are located away from the Rankine active failure wedge. The influence of the vertical location of anchors had significant effect on the overall performance of the retaining walls. The provision of tie-rods with slight inclination in to the backfill wall is beneficial to the strength of the wall.

## Acknowledgements

The facilities extended by IIT, Madras for carrying out this work are gratefully acknowledged by the authors. The authors are also grateful for the useful comments offered by the two anonymous reviewers which has considerably improved the presentation in the paper.

## References

ANDERSON, W.F., HANNA, T.H. and SHAH, S.A. (1977) : "Model Tests on Anchored Walls retaining Over-consolidated Sands", *Canadian Geotechnical Journal*, 14, 214-222.

CLOUGH, G.W. and TSUI, Y. (1974) : "Performance of Tied-back Walls in Clay", *Journal of Geotechnical Engineering Division*, ASCE, 100, 1259-1274.

HANNA. T.H. and KHURDI, I.I. (1974) : "Studies on Anchored Flexible Retaining Walls in Sand", *Journal of Geotechnical Engineering Division*, ASCE, 100, 1107-1120.

HANNA, T.H and MATALLANA, G.A. (1970) : "The Behaviour of Tied-back Retaining Walls", *Canadian Geotechnical Journal*, 7, 373-396.

HUA, Z. and SHEN, C.K. (1987) : "Lateral Earth Pressure on Structures with Anchor Plates", *Journal of Geotechnical Engineering Division*, ASCE, 113, 3, 189-202.

PLANT, G.W. (1972) : "An Assessment of Multi-anchored Retaining Wall Behaviour", *PhD. Thesis*, University of Sheffield, England.

RAJAGOPAL, K. and SRIHARI, V. (1998) : *Experimental Investigations on Pullout Capacity of Vertical Anchors*, Communicated to Indian Geotechnical Journal for possible publication.

SINGH, R.B. (1992a) : "A Combined Semi-Z Anchor and Geotextile System for Soil Reinforcement", *Indian Geotechnical Journal*, 22, 4, 236-244.

SINGH, R.B. (1992b) : "Anchored Earth Technique using Semi-Z Shaped Mild Steel Anchors", *Proceedings of International Symposium on Earth Reinforcement Practice*, Fukuoka, 419-424.

SINGH, R.B., JAIN, K.K., AGARWAL, R.K., MATHUR, S.P. and SARASWAT, S.S. (1995) : "Anchored Earth Approach Embankment for New Varuna Bridge at Varanasi", *Indian Geotechnical Conference*, Bangalore, 1, 375-378.

## Notation

The following symbols have been used in the paper.

$L$ = length of anchor

$C$ = non-dimensional constant

$D_r$ = relative density

$E_r$ = embedment ratio

$h$ = embedment depth of anchor

$H$ = height of anchor

$B_w$ = width of wall

| | | |
|---|---|---|
| $\phi$ | = | friction angle of soil |
| $K_p$ | = | Rankine passive earth pressure coefficient |
| $\gamma$ | = | unit weight of soil |
| Q | = | total surcharge force applied on wall |
| $\sigma_v$ | = | vertical pressure of soil |
| $S_a$ | = | shape factor of anchor |