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# Lateral Earth Pressure Reduction due to Controlled Yielding Technique

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## Introduction

All the structures buried or above the ground level are subjected to lateral earth pressures when they come in contact with the soil medium. In such cases, the earth pressures acting can be subscribed to one of the three conditions that arise out of the soil structure interaction (Terzaghi 1934, Fang and Ishibashi 1986). In situations where the adjacent soil is pushed into it, which is synonymous as the structure moving towards the soil, a passive condition mobilizes and the lateral earth pressures will be the highest. When the structure moves away from the soil, active condition develops where the lateral pressure on the structure will be the least. Once the active condition is reached, increased displacements lead to no further decrease in pressure. The amount of lateral deformation required to achieve these limiting earth pressures depends on the shear strength properties of the soil, mode of deformation of wall (i.e. translation, rotation about the toe or rotation about the top) and height of wall etc (Terzaghi 1934, Fang and Ishibashi 1986, Rajagopal and Bathurst 1992). The earth pressures corresponding to a case where the soil does not undergo any lateral deformation are referred to as at-rest earth pressures. The at-rest earth pressures are higher than those of the active earth pressures. These three situations are to be critically looked into when the structure of concern is a retaining wall and is backfilled. Wherever possible the retaining walls are designed for an active condition by allowing for small lateral deformations thus economizing the design in terms of volume of concrete and steel in the wall section. But, in the case of rigid retaining structures such as basement walls, abutments, box culverts etc., the lateral movement of the wall is curtailed and there can be no scope for the soil to expand laterally.

Since, the lateral expansion of the backfill soil is restrained in rigid structures the lateral earth pressures at the end of its construction are usually very high because of the build up of compaction stresses. The compaction induced stresses are more in case of bridge abutments as good quality soils are used for approach roads and the backfill is highly compacted to reduce the problems of the future settlements (Broms 1971, Duncan and Seed 1986). The magnitude of compaction induced stresses is higher for highly frictional soil which is the preferred backfills behind retaining walls (Clayton and Militisky 1986). In such cases the retaining wall is designed for the at rest condition to

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accommodate the higher induced stresses, where the earth pressure coefficient is taken as 0.8 according to IS: 4651 (Part II)-1969. Even for these rigid structures, an innovative and simple technique is available to allow for lateral expansion of the backfill soil so as to bring down the earth pressures near to active pressures or some times even lesser. The present paper discusses the concept involved in the method and highlights the design aspects and results of a practical application of the controlled yielding technique tried out at a site in the State of Gujarat. The paper also describes the model test set up of a rigid retaining wall and the investigations carried out in the laboratory.

## Controlled Yielding Technique

A vertical layer of compressible medium is introduced abutting the retaining wall between the wall and the backfill during the backfilling process, Figure 1.

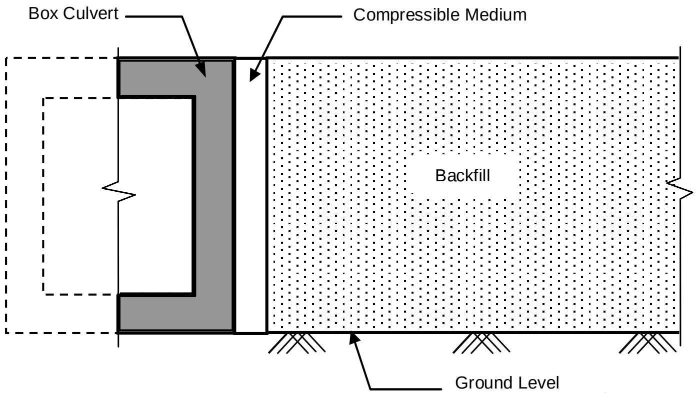


**Fig. 1 Principle of Controlled Yielding**

As the soil is filled up gradually behind the retaining wall, the compressible material placed in between will be experiencing compression allowing the soil to expand laterally. If the thickness of the compressible material is high enough, its compression will enable the soil to undergo sufficient lateral expansion to bring down their lateral pressures to active pressure level. Since, the thickness of the compressible material can be designed to suit the required values of the lateral expansion of the backfill soil, the method introduced to achieve this is termed as the controlled yielding technique (Partos and Kazaniwsky 1987). However, the lateral expansion that can be brought about depends upon the stiffness and stress-strain relations of the compressible material apart from its thickness.

## Earlier Studies in this Area

The fundamental concepts related to the three states of the lateral earth pressures reported by Terzaghi (1934) have shown an efficient and economical way of looking towards the design of the retaining walls. Taking cue from this, many researchers have carried out studies on various methods of reducing lateral earth pressures. It was felt that if the compressible layer can also serve the function as drainage and/or insulating layer apart from its primary purpose of reducing the lateral earth pressures, it could be the most ideal case. Partos and Kazaniwsky (1987) have employed a prefabricated expanded polystyrene bead drainage board to act as both compressible medium and drainage medium in a 10 m high basement wall. McGown et al. (1987, 1988) have demonstrated through model laboratory tests that it is possible to reduce the lateral earth pressures to much below the active levels by providing reinforcement layers in the backfill along with the vertical compressible layer. Edgar et al. (1989) have described the use of corrugated cardboards and removable plywood forms to reduce the lateral load of geosynthetic reinforced bridge fills on bridge abutments to essentially zero.

Saran et al. (1992) have shown that there is a good agreement between the theoretical findings based on limit equilibrium approach and the results from model tests of the case of a rigid wall retaining a reinforced cohesionless fill that carries a uniform surcharge load. In all the above studies, the measured earth pressures were very near or below the active earth pressures. Rajagopal and Bathurst (1992) have investigated the mechanism of controlled yielding through finite element analysis and have developed design charts for choosing the thickness of the compressible medium for different wall heights and soil conditions.

## Field Tests

The innovative technique of controlled yielding was employed successfully to reduce the lateral earth pressures on rigid box culverts over a stretch of 9 km along a new alignment of a highway project in the State of Gujarat for economising the structural design of these culverts. These box culverts are typically 3 to 6 m in height and were structurally designed as rigid structures. At many small crossings on the highway project, these box culverts are also used as bridges. The backfill soil used was a red murum soil with very good compaction properties. These soils exhibited peak friction angle of 35° and a small cohesion when compacted at optimum moisture content.

### Tests on Compressible Material

Keeping in view the main purpose of the compressible material which should not degrade in the presence of water, a number of commercially available materials were explored. After sufficient market study, a most suited product resin bonded rockwool slabs were selected for the purpose, which were generally used as thermal insulators. These materials conform to IS: 8183-1993. These are available in 50 mm thick board form with dimensions 1.52 m $\times$ 1.22 m. The unit weight of this material is approximately 1.10 kN/m$^3$. The stress-strain relations of the material were studied by conducting tests on single slab of 50 mm thick and two slabs with total thickness 100 mm. The test specimen size was 100 mm square. Uniform pressure was applied through a rigid steel plate and the resulting deformations were measured through a dial gauge. Tests were performed in both dry and wet states. Specimens were soaked for 96 hours in water for using them in wet state. Typical compression test arrangement is shown in Figure 2.

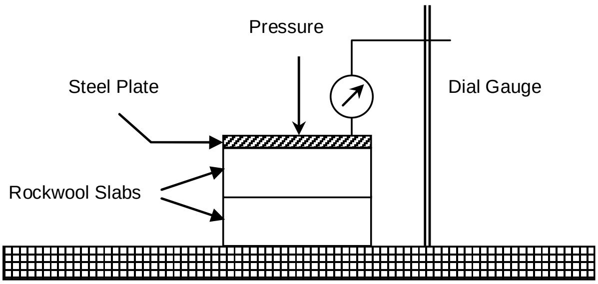


**Fig. 2 Test Set up for Tests on Compressible Material**

The compression tests performed on the material showed that its initial modulus is very small, of the order of 50 to 100 kPa, upto a strain of 30% and thereafter rapidly increases to more than 1000 kPa with increasing strains. This material is ideally suited for the purpose because the backfill soil should expand during the compaction process and further expansion of the soil should cease after the construction is completed. The compressible rockwool slab is fibrous in its form and from the laboratory permeability tests conducted at different compression levels, the material is found to have high permeability characteristics and it is also inert to the presence of water. Further the samples of wet state tests showed no degradation in their stiffness. The stress-strain behaviour of the rockwool slab is shown in Figure 3.

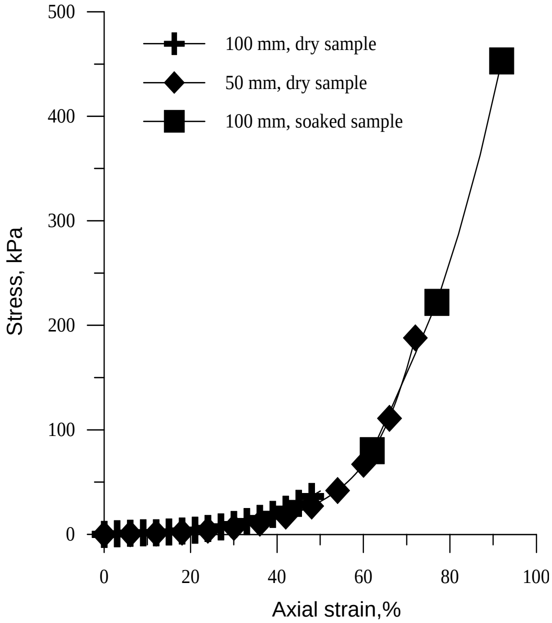


**Fig. 3 Stress - Strain Behaviour of Rockwool Slab**

## Thickness through Finite Element Simulations

The actual thickness of compressible material required to reduce the lateral stress levels to a K-value of 0.33 (which was used in structural design) from the initial K-value of 0.80 was estimated from several plane strain finite element simulations. The properties used in the analyses were obtained from separate laboratory tests on soil and compressible material. The exact sequence of soil placement and compaction to be followed in the field were simulated in the finite element analyses. The variation in the stiffness of the compressible material at different normal pressures was exactly simulated in the analyses as described by Rajagopal et al. (2000). An equivalent surcharge pressure of 35 kPa was applied on the backfill surface to simulate the effects of pavement weight and the traffic loads. The lateral earth pressures for two different thicknesses of the compressible material are shown in Figure 4. It is clear that 100 mm thick rockwool layer will be able to reduce the earth pressures to near active conditions. However, 150 mm thick layer was provided in the construction in order to account for uncertainties in the soil properties, compaction procedures and other factors.

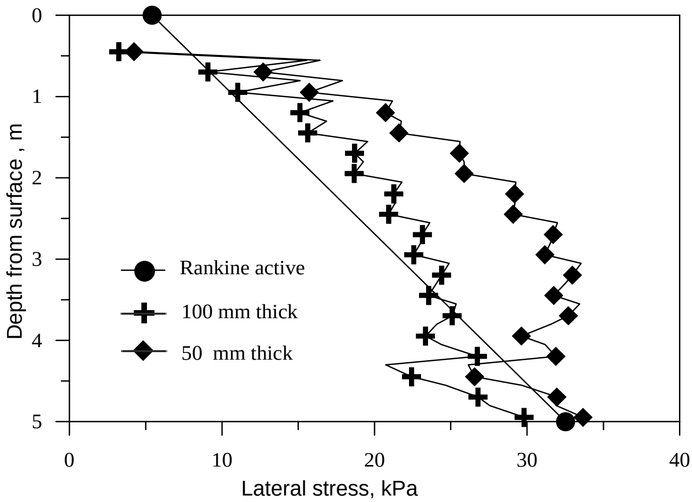


**Fig. 4 Lateral Stresses in Backfill Soil from FE Analyses**

## Sequence of Construction and Backfilling

During construction, these compressible boards with an overall thickness of 150 mm were initially fixed to the walls by gluing. The material was terminated at two lifts of fill below the top of the culvert. A filter medium of 300 mm thickness consisting of 20 mm aggregates and a subsequent layer of 300 mm thick of 10 mm aggregates was placed adjacent to the compressible material. The purpose of the filter medium is to allow the free drainage of water from the backfill. The backfill soil was compacted up to the edge of the filter media i.e. up to 750 mm from the wall using 10 Ton vibro-rollers in layers of 200 mm each.

One section of the box culverts was instrumented with four strain gauge based pressure cells fixed on the wall at different elevations flush with the surface of the wall with cement mortar for verifying the actual lateral earth pressures in the soil.

### Test Results

The data obtained from the measurements at the end of one month after the full construction of an instrumented section is shown in Figure 5. The initial earth pressures were slightly lower than those shown in the figure. The measured earth pressures were very close to those obtained with 150 mm thick compressible layer. The earth pressure measurements could not be continued beyond one month due to pilferage of electrical wires connecting the pressure cells to the reading units.

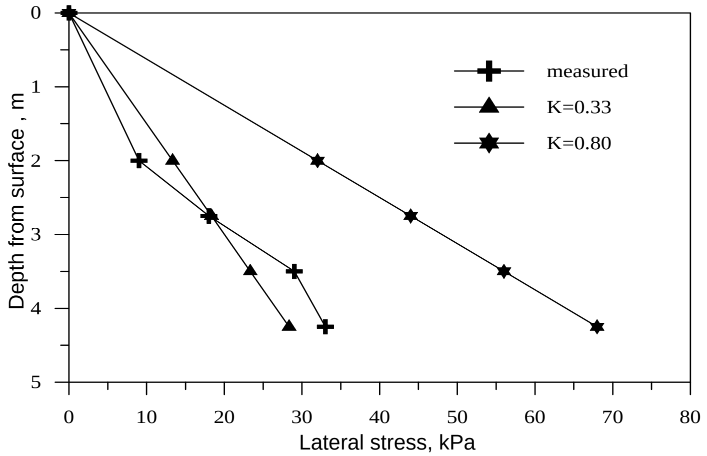


**Fig. 5 Comparison of Different Earth Pressures from Field Tests**

## Laboratory Investigations

### Test Tank

Laboratory experiments were conducted in a specially built rigid concrete test tank of size 1900 mm long, 750 mm wide and 2050 mm height. The backfill can be filled upto a maximum height of 1750 mm in the tank. One of the smaller sides of the test tank served the purpose of a rigid retaining wall. The tank was built from 900 mm below the floor level of the room where it was located, so as to give additional rigidity to its sidewalls and also for the ease of backfilling. The two longitudinal walls of the tank were lined with double layers of greased plastic sheets to reduce the side wall friction effects. The wall was instrumented to record both the lateral earth pressures and the compressions in the compressible medium. The soil in the tank can be subjected to uniform surcharge pressures through an inflatable air pressure bag fixed between the soil surface and rigid plates fixed at the top of the wall as shown schematically in Figure 6.

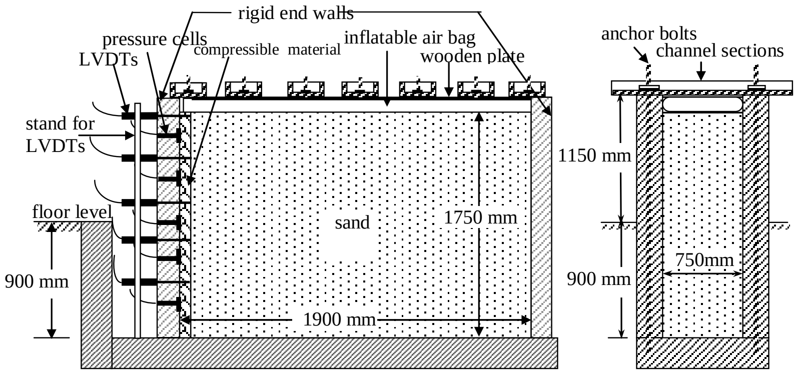


**Fig. 6 Schematic Views of the Laboratory Test Set up**

## Backfill Soil Used and the Compaction Technique

All the laboratory tests were conducted using uniformly graded river sand as the backfill soil. The significant grain sizes of the soil $D_{10}$, $D_{30}$ and $D_{60}$ are 0.20 mm, 0.35 mm and 0.49 mm respectively. The coefficients of uniformity ($C_u$) and curvature ($C_c$) of the soil are 2.47 and 1.25 respectively. The soil can be classified as poorly graded sand with letter symbol SP as per the relevant Indian standard soil classification system IS 1498-1970. The angle of shear resistance of the soil from direct shear tests at 60% relative density was found to be 42° in the normal pressure range of 0 to 100 kPa.

The backfill soil was filled up in 9 layers, of which the first 8 layers were of 200 mm thick and the top layer was 150 mm thick, making its total height 1750 mm. All the layers were compacted uniformly to the required relative density of 60% by the ‘tamping method’. In this method a steel disk with a central hole having a mass of 5 kg was repeatedly lifted by 300 mm and dropped on to a wooden plank of size 700 mm long, 230 mm width and 12 mm thick. The drops from the falling weight were uniformly spread over the wooden plank. At each location, 25 blows spread uniformly over the plan area of the plank were given to achieve a relative density of 60%. The wooden plank was moved over the plan area of the tank so as to compact the soil over the length and breadth of the test tank. The density of the compacted soil was monitored by collecting soil samples in small steel cans embedded in soil during the compaction. The soil very close to the compressible material was gently compacted so as not to damage the compressible material. The sand was compacted to a relative density 60% and the same was maintained in all the tests. This procedure of compaction was developed after a few trials during the initial part of the research work.

## Reinforcement Layers

In some of the tests, the backfill was reinforced with four layers of biaxial geogrid having an index tensile strength of 20 kN/m. The 5% and 10% secant modulus of the reinforcement are 160 and 125 kN/m respectively. All the reinforcement layers were 650 mm wide and were provided horizontally on the finished levels of compacted backfill lifts at depths of 150 mm, 550 mm, 950 mm and 1350 mm from the soil surface. The lengths of the reinforcements were 650 mm each for the bottom two layers and 1000 mm each for the upper two layers. On each layer, strain gauges were firmly affixed using araldite adhesive at three positions one at the middle and the other two at either end of the layer. The lead wires from the strain gauges were connected to the strain measuring units. The geogrid layers were placed on the backfill soil without connecting them to the compressible medium.

## Compressible Materials Used

Two types of compressible materials were used in the laboratory investigations. One is a poor quality material Styrofoam that is preferred for false ceilings and for cushioning electronic goods and the other is fibre glass wool normally used as heat insulators. The Styrofoam and fibre glass wool materials are available in 24 mm and 50 mm thicknesses respectively. Compression tests were performed using a strain controlled testing machine, on both the materials at a slow rate of 0.5% strain per minute to determine their stress-strain behaviour. As shown in Figure 7, the Styrofoam material is relatively stiffer as compared to the fibre glass wool layer especially in the lower range of stresses. No significant change is noticed in the material properties on soaking the specimens for 96 hours in water. This is to suggest that there could not be much variation in withstanding the stresses and compressions both in dry and wet conditions. The Styrofoam is mostly waterproof though there is a greater void space in its structure itself where as fibre glass wool is very porous in its composition and allows drainage.

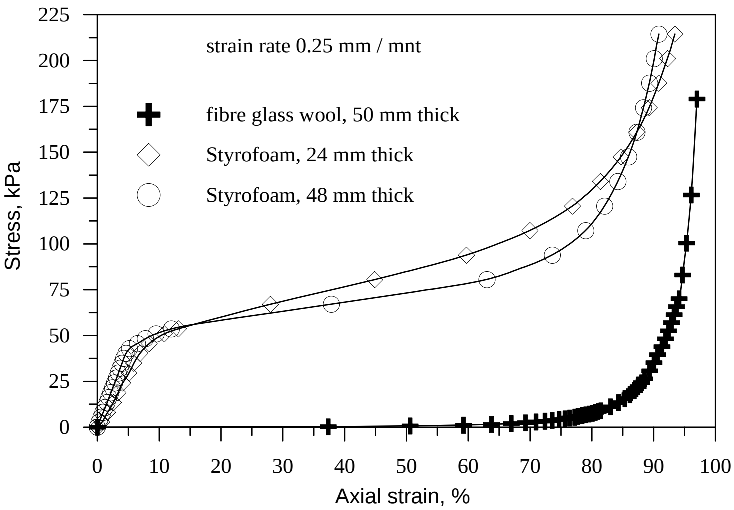


**Fig. 7 Stress-Strain behaviour of Styrofoam and Fibre Glass Wool**

## Surcharge Application

The backfill of the model retaining walls was subjected to uniform surcharge pressure by inflating an air pressure bag placed between the backfill surface and the cross-channels connected between the two sidewalls by the anchored bolts. The top and bottom surfaces of the air bag were covered with a nonwoven geotextile. A 25 mm thick plywood plank was placed between the geotextile and the steel cross channels to distribute the load uniformly on the soil. The air pressure bag, designed for 250 kPa internal pressures, was made of double faced neoprene coated nylon reinforced fabric.

An air compressor capable of supplying air at a maximum pressure of 1000 kPa was run continuously during the test to supply pressurised air. The air was routed through a non-return valve into the air bag. The regulator was adjusted to control the pressure of the air in the air bag to apply the desired pressure in small increments. The actual applied pressure was measured through four pressure cells mounted just below the backfill soil surface. In all the tests, a maximum surcharge pressure of 50 kPa was applied in increments of 5 kPa on the backfill soil. While applying surcharge pressure, each increment of 5 kPa was kept constant until the deformations and lateral pressures reached a steady state, which usually happened in less than 5 minutes at each load increment.

## Test Programme

The different tests conducted in this investigation are listed in Table 1. Some of these tests were repeated to verify the consistency in the test data. Very little difference was observed in the values from different tests indicating the consistency achieved in the test conditions. The measurements made during the tests included the surcharge pressure applied and the corresponding lateral earth pressures and lateral deformations at different elevations of the rigid wall. In addition, the reinforcement strains were measured in tests with reinforcement. These measurements were made at the end of each surcharge pressure increment. The vertical deformations of the soil surface were measured at the end of the test after removing the air pressure bag. The lateral deformations of the external walls were measured to be less than 0.01 mm under the highest surcharge pressure of 90 kPa.

**Table 1 Different Tests Performed**

| *Test Series* | *Type of compressible material* | *Thickness (mm)* |
|---|---|---|
| I | No compressible material | ------- |
| II | Styrofoam | 24, 48, 72 |
| III | Fibre glass | 50,100 |
| IV | Fibre glass wool and layers of biaxial geogrid reinforcement | 50,100 |

A few tests were performed by applying the load and unloading and reloading in order to study the performance of the controlled yielding system under load repetitions. In these tests, the surcharge pressure was increased gradually in increments of 5 kPa to reach a final value of 50 kPa and then reduced back to zero gradually. This makes the first load cycle. The other load cycles were repeated in the same manner. Totally, six cycles of load were applied on each of the backfill systems. After each cycle of loads, the system was left undisturbed for up to 24 hours until all the pressure cell readings and the deformation measurements reached steady state values. Approximately one week was taken to complete one set of repeated load tests.

## Results and Discussion

The measured deformations in the compressible layer for different cases are shown in Figures 8 and 9.

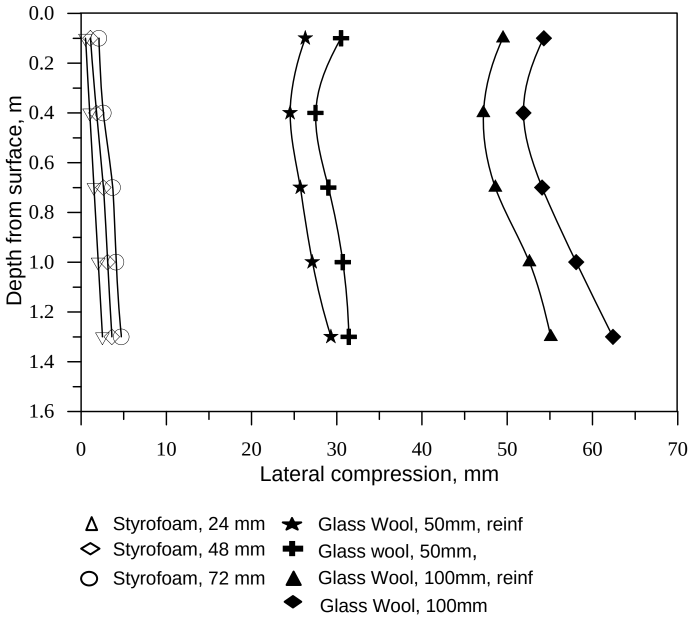


**Fig. 8 Lateral Compressions in Different Systems**

In general, the deformations are higher for softer compressible layers. For the same type of compressible material, the deformations increase with the increase in the thickness of the compressible layer. The deformations have generally increased with depth as shown in Figure 8. Figure 9 shows the variation of lateral deformations at three different depths with increasing surcharge pressures. The rate of increase of lateral displacements with increasing surcharge pressure is found to decrease with depth below the surface. This is because of the higher stiffness of the compressible material at higher initial strain. The material at the bottom depths has higher initial strain due to larger compression at the end of filling and hence higher stiffness leading to only marginal increase in compression under surcharge pressure application. On the other hand, the material at the top of the wall has undergone comparatively lesser strains during filling and hence has lesser stiffness leading to larger compressions under surcharge pressures.

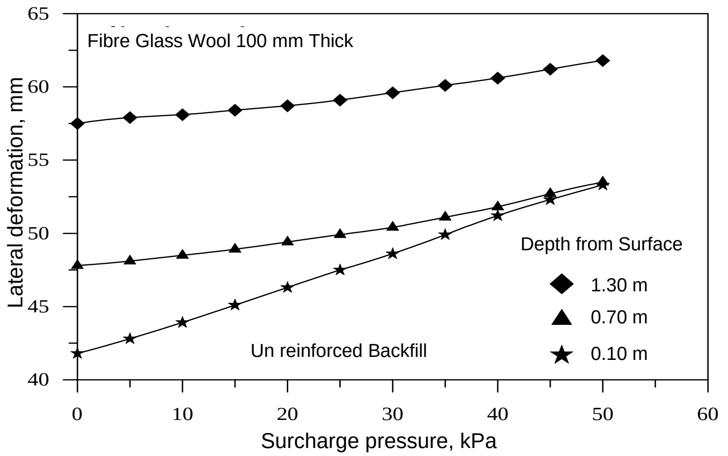


**Fig. 9  Variation of Lateral Deformations with Surcharge Pressures**

The corresponding lateral earth pressures for different cases are shown in Figures 10 and 11. The influence of sidewall friction and the friction on the wall face is clearly evident from the decrease in lateral earth pressures at deeper depths in all the cases. It can be noticed that the provision of compressible layer has significantly decreased the earth pressures transmitted to the wall face. The introduction of horizontal reinforcement layers had resulted in further decrease of lateral earth pressures.

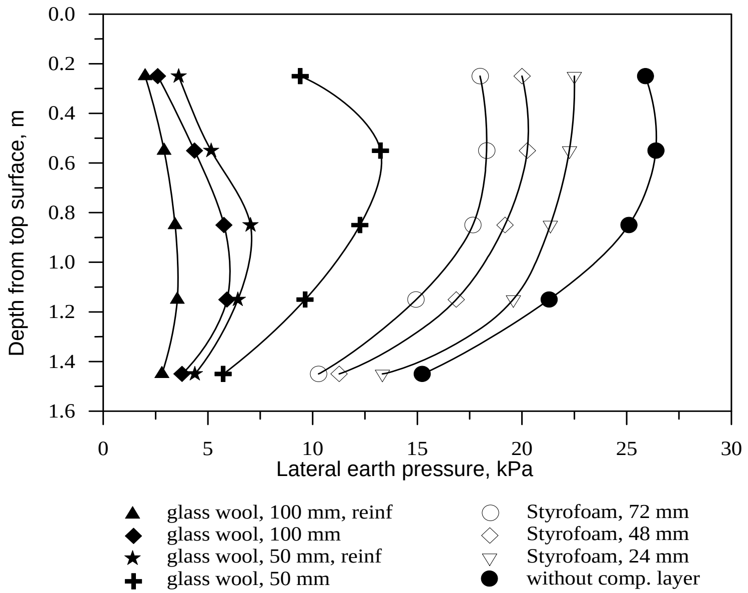


**Fig. 10  Lateral Earth Pressures in Different Systems**

It could be noted that the earth pressures are lower for the cases with larger lateral deformations. The earth pressures are higher for the case with Styrofoam sheets because of relatively lesser lateral straining compared to the fibre glass wool. The earth pressures did not show a linear increase with depth as would be expected. This could have been caused by the effects of wall friction in combination with soil arching. Similar earth pressure distributions were reported by McGown et al. (1987, 1988). The interface friction angle between the sand and the plastic sheet used to cover the side and front facing was measured to be 13°. Hence, significant sidewall effects could be expected.

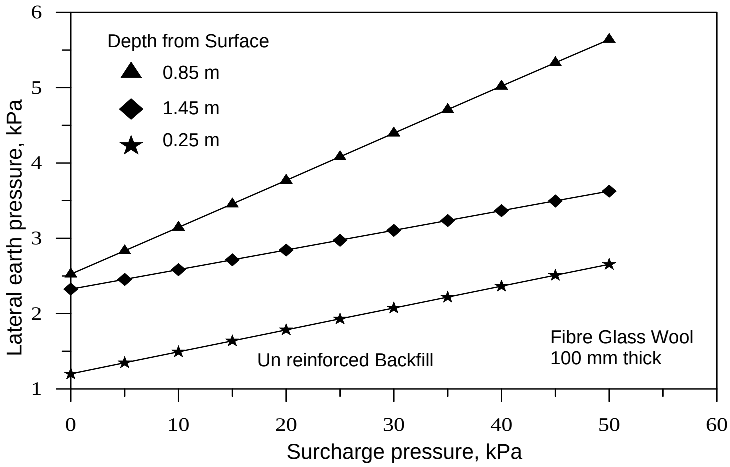


**Fig. 11 Variation of Lateral Earth Pressures with Surcharge Pressures**

The maximum lateral earth pressure for the case without any compressible medium is about 25 kPa near the top surface. As the lateral strains can be taken as zero, the earth pressures in this case will correspond to the at rest condition. The $K_o$ value can be evaluated as the ratio between the lateral stress and the applied vertical pressure as 0.50 which is higher than (1-sin$\phi$). These higher lateral pressures might have happened due to compaction effects during the placement of the soil.

The active earth pressure at 50 kPa surcharge near the top surface of the soil as per the Rankin's earth pressure theory will be approximately 10 kPa for a friction angle of 42º. The measured earth pressures in all the cases with Styrofoam near the top surface are much more than 10 kPa indicating that the soil has not reached the active state. The maximum lateral deformation at the top surface with Styrofoam is about 3 mm which is approximately equal to about 0.17% of the height of soil 1.75 m ($\delta$/H%). As the lateral strain required to mobilise the active state in granular soils is about 0.3 to 0.4% of the wall height, the earth pressures may not have reached the active levels.

The measured earth pressures near the top surface for different cases of fibre glass wool range from 3 to 7 kPa which are much below the active state pressure of 10 kPa. This clearly indicates that the soil has reached the active state in these cases. The lateral deformations for the case of fibre glass wool range from 25 to 55 mm at the top surface corresponding to lateral strains of 1.4

to 3% at the surface of the soil. These deformations are more than adequate to mobilise the active pressure state in the soil and hence the pressures have reached these low values. The earth pressures might have gone below the active values due to the soil arching effects and the reinforcement action. The earth pressures even at deeper depths with compressible medium are much lower than those without compressible layer. The general trend of lower earth pressures with larger lateral deformations has been followed even for the earth pressures at deeper depths.

## Influence of Load Cycling

A few tests were performed by load cycling to examine the effect of repeated load applications on the performance of the controlled yielding system. All these tests were performed with fibre glass wool material with 50 and 100 mm thickness. The results for only the 100 mm thickness are reported because the trends are very much similar. The lateral deformations at the maximum pressure of 50 kPa at the end of all the load cycles are shown in Figure 12. This figure shows the deformations for both the unreinforced and reinforced cases. The reinforced backfill had undergone lesser lateral deformations under the first loading. In the second cycle of loading, large incremental deformations took place near the surface in both reinforced and unreinforced cases. The reinforced system had undergone only minor additional deformations at deep depths while the unreinforced system had undergone larger deformations over the full height of the soil. The same trend has continued in all the load cycles. As the load is re-applied, the incremental deformations went on decreasing. At the end of 6$^{th}$ load cycle, the incremental deformations have drastically reduced for both unreinforced and reinforced cases.

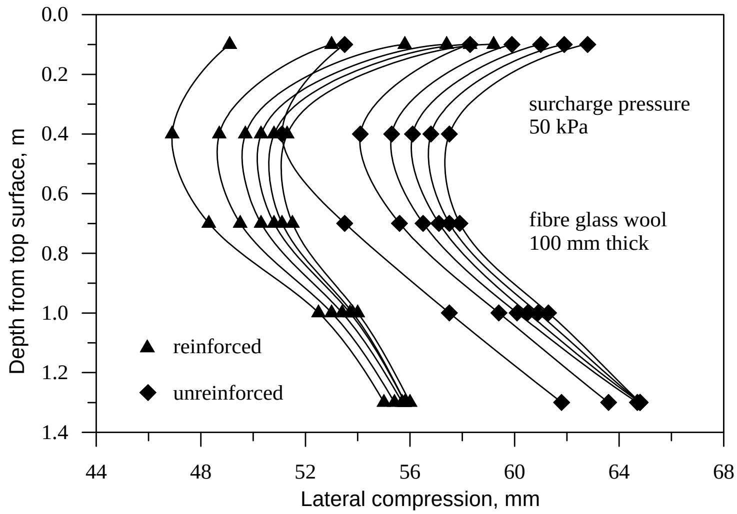


**Fig 12 Lateral Deformations under Cyclic Loading**

The measured lateral earth pressures under load cycling are shown in Figure 13. As in the case of lateral deformations, the changes in earth pressures are more near the surface as compared to the bottom portions of wall. With each load cycle, the earth pressures have increased in magnitude. However, the incremental changes went on reducing until the 6$^{th}$ load cycle when the earth pressures did not change appreciably as could be observed from the figure. The maximum intensity of lateral earth pressures at the end of 6$^{th}$ cycle are 4.5 and 7.5 kPa for reinforced and unreinforced cases respectively. These earth pressures are lower than the corresponding active earth pressures because of soil arching and reinforcement effects. It is clear from these observations that the earth pressures stay below the active values even under load cycling.

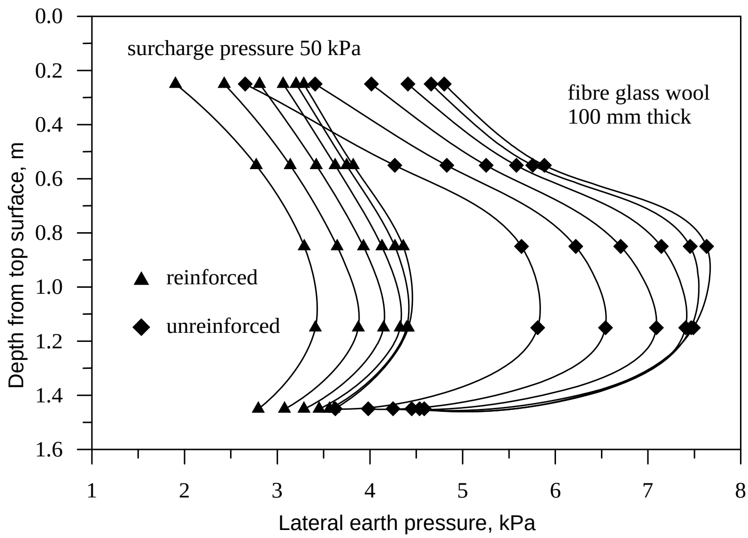


**Fig. 13 Lateral Earth Pressures under Cyclic Loading**

## Subsidence Profile of the Surface

The subsidence profiles of the backfill surface at the end of first and sixth load cycles are measured during the tests for both reinforced and unreinforced cases. The data is presented in Figures 14 and 15 for the cases of 50 mm and 100 mm thick fibre glass wool layers.

These results pertain to the maximum applied surcharge pressure of 50 kPa. The incremental deformations between the 1$^{st}$ and 6$^{th}$ load cycles are much lower for reinforced cases than the unreinforced cases. It is very clear from both the figures that the provision of the reinforcement layers has reduced the magnitude of the subsidence. This is due to the increased stiffness of the soil due to the provision of reinforcement layers.

The back-analyses through 3-dimensional finite element analyses have shown that the equivalent modulus of the reinforced backfill is 50% to 70% more than that of the unreinforced backfill. This clearly explains the reason for lesser subsidence and lesser lateral deformations in the reinforced backfill.

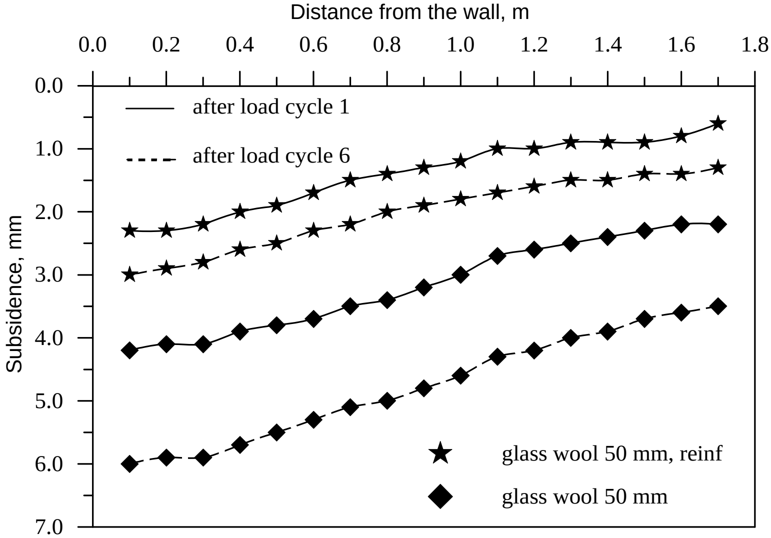


**Fig. 14 Subsidence of Ground Surface with 50 mm thick Glass Wool**

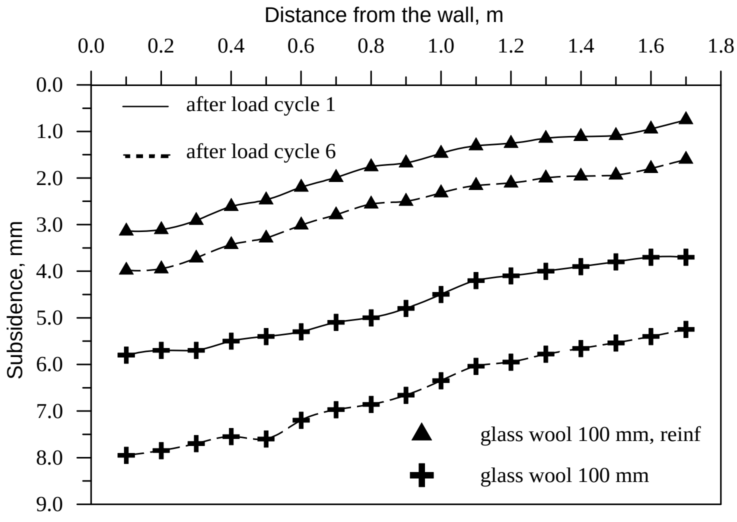


**Fig. 15 Subsidence of Ground Surface with 100 mm thick Glass Wool**

## Conclusions

This paper has reported the field application of controlled yielding technique to reduce the lateral earth pressures on rigid box culverts and results from laboratory model tests to understand the controlled yielding technique. The results from both the field and laboratory tests indicate that it is possible to reduce the earth pressures to or below the active state by controlled yielding.

The field test results showed that the reduction of lateral earth pressures was around 52% against the at rest pressures at a depth of 4.50 m from the surface of the box culvert. From the laboratory model tests, large reductions in lateral earth pressures were achieved when 100 mm thick fibre glass wool was used as the compressible medium. Thus the maximum earth pressure reductions observed under a surcharge pressure of 50 kPa along the retaining wall from bottom to top are 76% to 89 % in the unreinforced backfill system and 81% to 92% in the reinforced backfill system respectively.

The results from repeated load tests have clearly shown that the earth pressures stay below the active state pressures even under load repetitions.

The provision of reinforcement layers in the backfill soil increases the stiffness of the soil apart from reducing the lateral earth pressures transferred to the wall. Both these factors can be ideally employed for the case of bridge abutments. Because of lesser earth pressures, the section of the bridge abutment can be lighter leading to the economy of construction. The increased stiffness of the backfill soil due to the reinforcement may help in preventing the relative settlement at the junction of the abutment and approach road, thereby facilitating a level surface to the moving vehicles.

## References

Broms, B. (1971): 'Lateral Earth Pressures due to Compaction of Cohesionless Soils', *Proceedings of the 6$^{th}$ International Conference on Soil Mechanics and Foundation Engineering*, Budapest, pp 373-384

Clayton, CRI and Militisky, J. (1986): *Earth Pressure and Earth-Retaining Structures*, Blackie & Sons Ltd., Glasgow, UK.

Duncan, J.M. and Seed, R.B. (1986): 'Compaction-induced Earth Pressures under $K_o$ Conditions', *ASCE Journal of Geotechnical Engineering*, Vol. 112, No. GT1, pp.1-22

Edgar, T.V., Puckett, J.A. and D'Spain, R.B. (1989): 'Effects of Geotextiles on Lateral Pressure and Deformation in Highway Embankments', *Journal of Geotextiles and Geomembranes*, Elsevier Science Ltd., Oxford, U.K., Vol. 8, pp 275-292

Fang, A.M. and Ishibashi, I. M. (1986): 'Static Earth Pressures with Various wall movements', *ASCE Journal of Geotechnical Engineering*, Vol.112, No.GT.3, pp.317-333

IS 4651 (Part 2)-(1969): *Code of Practice for Planning and Design of Ports and Harbours: Part 2 Earth Pressure (First Revision)*, *Reaffirmed 2005*, Bureau of Indian Standards, New Delhi

IS 1498-(1970): *Classification and identification of soils for general engineering purposes*, Bureau of Indian Standards, New Delhi.

IS 8183-(1993): *Bonded Mineral Wool (First Revision)*, *Reaffirmed 2004*, Bureau of Indian Standards, New Delhi.

McGown, A., Murray, R.T. and Andrawes, (1987): *Influence of Wall Yielding on Lateral Stresses in Unreinforced and Reinforced Fills*, Research Report 113, Transportation and Road Research Laboratory ,Crowthorne, Berkshire, U.K.

McGown, A., Andrawes, K.Z., Murray, R.T. (1988): 'Controlled Yielding of The Lateral Boundaries of Soil Retaining Structures', *Proc. of Symposium on Geosynthetics for Soil Improvement*, Ed. Rd. Holtz, ASCE, Nashville, Tennessee, USA, pp 193-211

Partos, A.M. and Kazaniwsky, P.M. (1987): 'Geoboard Reduces Lateral Earth Pressures', *Proceedings of the Geosynthetics'87*, New Orleans, Industrial Fabrics Association International, pp 628-639

Rajagopal, K. and Bathurst, R.J. (1992): 'Numerical Investigation of Controlled Yielding of Soil-Retaining Wall Structures', *Journal of Geotextiles and Geomembranes*, Vol. 11, pp. 115-131

Rajagopal, K., Pitchumani, N.K. and Purnanandam, K. (2000): Earth Pressure Reduction Behind Rigid Box Culverts: A Case Study, *Proc. Of Indian Geotechnical Conference*, Mumbai, December 2000, pp 383-386

Saran Swami, Garg, K.G. and Bhandari, R.K. (1992): 'Retaining Wall with Reinforced Cohesionless Backfill', ASCE *Journal of Geotechnical Engineering,* Vol.118, No. GT 12, pp. 1869-1888

Terzaghi, K. (1934): 'Large Retaining Wall Tests (I)-Pressure of Dry Sand', *Engineering News Record*, Vol.112, pp. 136-140