INSTRUMENTATION AND ANALYSIS OF RETAINING WALL

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Submitted By

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ABSTRACT

A reinforced soil retaining structure is usually defined as the vertical wall retaining structure which is constructed by enhancing the stability of backfill of wall with the help of placement of reinforcing materials for eg. Geosynthetics, geogrids and steel strips within the backfill. Retaining walls with Reinforced backfill are structures which are considered as flexible because the friction between the reinforced soil and reinforcement materials and connection parts between walls and reinforcement materials help it in resisting earth thrust through friction. Due to this reason only, retaining walls with reinforced backfill are considered as much better seismic resistant structure than normal retaining walls. Because of this reason, in recent times, retaining walls with reinforced backfill are adopted much more as a railroad bed bearing structure. However analysis of displacement characteristics and acceleration of retaining walls with reinforced backfill because of impact loading of moving loads have been done rarely and hence applicability of reinforced retaining walls have not been scientifically established.

Experiments are conducted in this project through the application of impact as well as dynamic loads to a retaining walls with reinforced backfill in order to study its behavior on the basis of magnitude and location of impact loads.

Shake table tests is performed on retaining wall with reinforced backfill in order to analyze the dynamic behavior and various parameters of wall and compare the results with and without the application of geofibres.

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CHAPTER 1

Introduction

A retaining wall with reinforced backfill is defined as vertical walled retaining structure which is highly stable due to placement of geotextiles which includes geogrids as well as band shape reinforcement materials through the layers of surcharge soil.

Retaining walls with Reinforced backfill are structures which are regarded as flexible as the friction between the soil and reinforcing materials and connection parts between walls and reinforcement materials help it in resisting earth thrust through friction. Tatsuoka (2006) has established the importance of retaining wall with reinforced backfill as seismic resistant soil walled structure. According to his case study, in 1995 during the Kobe Earthquake maximum of gravity retaining wall which were most common type of wall used to sustain loads coming from bed of rail-road were damaged to a large extent or destructed by the severe motion of ground and dynamic forces of earthquake while those backfill is reinforced sustained minimum damages.

Because of high seismic resistency of reinforced retaining walls, IBC(Indian Building Congress) and NHAI(National Highways Authority of India) prepared seismic design criteria for retaining walls with reinforced backfill and hence there is sustained interests in adoption of retaining walls with reinforced backfill as a seismic resistant structure or as a rail road load bearing structure internationally.

However retaining walls with reinforced backfill usually have been applied only to Highways, Bridges, Housing and sports infrastructure and coast protection. Also safety stability and effectiveness of retaining wall with reinforced soil to cyclic as well as dynamic loading and long term repeat loading are not scientifically established. Therefore its usage as a seismic resistant structure or rail road load bearing structure is little.

Bathurst & Raymond and Webster (1989) made use of retaining wall with reinforced backfill in their field experiments by applying it to a rail-road structure. They observed that frictional resistance between soil and reinforcing material was lowered due to the trains vibrations and it caused continuous displacement of retaining wall. Also, seismic design criteria presented by IBC and NHAI are based on a quasi-static limit equilibrium analysis using the Mononobe-Okabe theory making it difficult to understand the actual displacement and acceleration characteristics of retaining wall with reinforced backfill caused by dynamic loading.

In this study, displacement characteristics of reinforced and unreinforced retaining walls were compared when impact and dynamic loading was applied. Behaviors and effects of retaining wall with reinforced backfill against cyclic loading were analyzed by studying the correlation between settlements and strains of geogrids which are used as a reinforcement.

Fig 1: Model of Retaining Wall

CHAPTER 2

Literature Review

2.1. Physical Models

Physical modelling plays an eminent role in evolution of geotechnical understanding. Considering the basic understanding of physical modelling it can be interpreted that if every experiment is considered as a physical model indeed a good model it can be used to increase our confidence in improving the theoretical model.

Physical Modelling is conducted in order to verify theoretical or empirical hypothesis. Hence geotechnical construction is also a form of physical modelling; geotechnical drawing makes assumption about expected performance which may be tested and verified in more or less details depending upon the extent to which reaction of the geotechnical system is desired to be seen. There will be at least a binary indication; has the geotechnical structure achieved the desired objectives or not. A failure will show deficiency in supporting models. If the designer has less confidence in design of supporting models then deep observation and experimentation such as observation of displacements and pore pressures will help in providing the more accurate information about the way in which the materials under observation are behaving.

Reflections on behavior observed then provides the path for better design or modelling in future. (Wood D.M. (2004)

Experimentation on micro elements of soil whether in the laboratory such as triaxial test ,shear test or in situ field tests such as geophysical test, penetration test, pressure meter test etc. would sometimes presume the model in the manner in which soil as a whole is going to behave. Sometimes experience help us in making the models better; it is particularly true in case of strength and stiffness models. These type of models if used for similar materials in the past can be helpful in deciding the rates of loading for different type of tests and also expected levels of stresses, transducers efficiencies for different ranges without much consideration. Routine experimentation involves scaling up or scaling down of present model so that it can be applied to a given material or set of data. Routine experimentation is not conducted to demonstrate that model is inappropriate or to find information which can be used to improve the model.

An all-around structured physical model gives a significant open door in demonstrating cycle. It is continually enticing to expect that a theoretical model (especially assuming numerically it is a rich model) by one way or other embodies truth. We can never demonstrate a theoretical model to be valid or universally accepted; everything we can say about an effective model or an assumption on which that model is based, is that it has not yet been falsified or invalidated. By and large all geotechnical models are most likely in all respects effectively discredited and our enthusiasm as geotechnical engineers is in distinguishing the range inside which the invalidation of individual models is the weakest since it is this which defines the scope of importance of those models.

A structured physical model which retains the basic understanding can be used to check and verify rival theories. Poorly conceptualized physical modelling is no more than data collection. If the models on which the experiments to be conducted are not scientifically correct then there are very less chances that correct data will be gathered; the physical modelling will be engaged in prediction and observation part of loop

2.2. Physical Models: Full-Scale

Physical Modelling involves evaluation of performance during physical testing of complete geotechnical systems. Where there is a distrust between theory and analysis, because of non-satisfaction of assumptions or parameters of material response being too unpredictable or realities of reliable numerical solution being too complex in all such cases physical modelling may seem a suitable route. Physical modelling has the advantage of using real geotechnical materials hence the need of theoretical modelling does not arise. Physical demonstrating of geotechnical frameworks should provide information approving analytical modelling and thus should be able to give a premise to extrapolate from physical model to geotechnical model despite the fact that instrumented and observed geotechnical model would itself be able to act as a physical model

If the physical modelling is being performed because of the uncertainty of ways in which details of geotechnical system might emerge then the optimum strategy is to perform the modelling at maximum scale. Because of unknown behavior of ground it will be beneficial to conduct trials at maximum scale which will resemble loading of real soils under real loading conditions. Full scale tests are normally conducted to ascertain geotechnical parameters which may have a dependence on actual soil fabric and its structural properties thus it is beneficial to use real soils as fabricated by nature.

Trial embankments are a good real life example in which the requirement is to determine the impact of various processes of ground improvement. In this experiment the adoption of different types and spacing of drains in order to accelerate the process of consolidation of soft clay may be analyzed. It is well understood fact that installation of drains through the soil induces a fabric change in soils which are adjacent to drains; it is also considered that the in-situ structure of ground have a decisive effect on flow characteristics. It will have a bearing on flow rates and hence the time of consolidation.

Various methods of ground improvement which will be helpful in enhancing embankment or structural stability without disturbing the rate of consolidation can be considered for example ground reinforcement using grids and fabrics; cement treatment of ground or derivation of columns of compacted course material to provide local strength to ground. It will be possible to model all these at small scale but the details can be best analyzed at full scale.

Information about the performance of piled foundation is enhancing but still there is a widespread conception that theoretical models of pile explaining ground interaction are not accurate and hence not completely dependable. This is because of uncertainties persistent to the processes of pile installation independent of whether it is being jacked, driven or bored as well as to interaction of pile installation method with the ground. Hence test piles are required very frequently so that it can create true installation procedure and ground conditions. The degree of inaccuracy of theoretical models of pile is to such extent that Eurocode 7 clearly states that all design parameters of pile must be related to the results obtained from static pile load tests and must be in line with general expectations.

The most basic advantage of full scale modelling is that it confirms with real ground conditions be it soil, loads or stress histories: these are the parameters that are needed to be factored in geotechnical modelling. Some of these can be controlled for example dimensions of structures created, heights of embankments and size of piles driven. Ground conditions are not in our control. The physical modelling is viewed as a component of organized cycle of theoretical and physical demonstrating. If we are not certain about the ground conditions then we cannot be confident about how to shape our theoretical model and if there is a inconsistency we cannot know whether inconsistency is in theory or in obscure details of underground conditions.

There are some disadvantages also to full scale demonstration. Small scale models help in providing faster results because of their smaller dimensions. For example if an embankment is to be constructed above soft soils for road or airport it will take very large time to complete. If full scale tests are to be conducted to study the rates at which embankments are to be constructed safely, the cost will multiply as the scale of model is increased. Because of the above mentioned reasons small scale modelling is considered better as through this more tests can be performed and more variables and its deviation with different parameters can be studied. Real conditions may complicate or may benefit. If the physical model is to be considered as a verification tool for theoretical model then both must be working under the same conditions.

2.3. Physical Models: Small Scale

If the models are to be experimented with at any other scale then the first task is to validate the model and then to fabricate a path to extrapolate data observed on model scale to parameters that will be observed on prototype: as the purpose of geotechnical modelling is to refine geotechnical practice. Thus supporting theoretical models are essential to extract full scale models from smaller scale. In order to convert small scale models into full scale knowledge of scaling laws and dimensional analysis is required.

A detailed geological model can help in conducting economic and efficient site exploration in order to determine quantitative parameters of soil. Past explorations and knowledge of adjacent sites with similar geologies can help in providing better estimate of the field. Through this properties, nature and mineralogy of soil particles, constitutive models which help in predefining laboratory testing and gradient of field can be predicted. Although geophysical practices can be helpful in drawing broad picture of soil below ground but for specific presentation of structure of ground boreholes are to be drilled regularly at site.

At the micro level, the interaction between the soil which will deform and control various aspects of geotechnical system and rock which will be rigid and deform less will be important in determining the expanse of ground that is to be modelled either physically or numerically. Rock layer will not necessarily act as a boundary layer.

Utilization of various modes of physical geotechnical demonstrating requires that scaling laws must be applied in correct manner so that performance of prototype or geotechnical structure can be interpreted from performance of physical models. For correct application of scaling laws factors which affect the performance of materials which are being used in models need to be understood.

2.4. Scaling Laws

A true model is constructed if all the fundamental laws of dimensional analysis and similitude are satisfied. For geotechnical modelling it is imperative to work with an acceptable model which satisfies ‘first order’ similarity Harris and Sabnis (1999) by considering that some restrictions offered by dimensional analysis are secondary. It will be true particularly in case of shaking table models as it will be difficult to scale loads which are generated due to gravitational forces and it will affect the performance of system and material. In general many academicians have analyzed scaling factors for models and for geotechnical models Harris and Sabnis (1999), Krawinkler (1979), Iai (1989); Schofield and Steedman, (1988) have identified factors related to centrifuge modelling. Stiffness in general is not considered as a factor but it needs to be analyzed thoroughly and it will be in our interests to make use of stiffness instead of strain as an independent factor.

Length

$$\frac{Length(model)}{Length(prototype)} = \eta_l$$

Since reducing dimension is main objective in conducting physical modelling it is most important reduction factor

Density

$$\frac{density(model)}{density(prototype)} = \eta_d$$

Acceleration

$$\frac{acceleration(model)}{acceleration(prototype)} = \eta_a$$

Acceleration factor will be 1 for gravity models which are singular and larger value of n will be considered for models which are experimented with geotechnical centrifuge. It is a basic understanding that dynamic acceleration must be modelled with equivalent gravitational acceleration in same scale. It helps in way that there is same scale factor for different accelerations. It appears reasonable because since vertical acceleration due to dynamic forces are analyzed hence the reaction of soil or various other elements of system will certainly be affected by the way in which dynamic acceleration take up portion of gravitational acceleration.

Stiffness

$$\frac{stiffness(model)}{stiffness(prototype)} = \eta_G$$

The stiffness factor include two terms: first the stiffness which arises due to infinitesimal strain which in different situations is responsible for dynamic response and propagation of waves through the ground; and second medium to large strain deformation parameters of ground which vary nonlinearly. The variation of small strain stiffness is generally considered to be first order and it depends on effective stress level σ. It can be expressed in form of

G α α^n

Experiments shows that exponent α is of order of 0.5 for sands and it is order of 1 for clays. If n=0 it shows that stiffness does not depends on stress level. For all models it has been assumed that material used while experimenting in both prototype and model is same. The advantage of this assumption is that it helps in making density scale factor unity.

For analyzing medium strain deformation behavior the best method in making model and prototype appear similar is to adopt critical state while experimentation.

Soil mechanics and contend for comparative estimations of state variable between the two cases Roscoe and Poorooshasb. For a material with neighborhood basic state line incline λ in an ordinary semi-logarithmic pressure plane, at that point, if the estimation of thickness in respect to the basic state thickness, taking into account pressure change (this relative thickness is the state variable), is to be held, a scale factor m applied on stresses infers a vital change in introductory specific volume and void ratio.

The increment that should be brought in specific volume or void ratio in both model and prototype should be 0.1 in order to maintain similarity. For sand the range of void ratios between maximum to minimum should be around 0.4 as determined by standard procedures. Relative density must be reduced at a high rate in order to maintain similarity. Procedures such as pluviation which are used for preparation of samples of soil will not serve the purpose as through this high void ratio in initial stages cannot be achieved hence loose samples would be prepared which not be able to outlast even first shake of model earthquake. Hence Bolton and Steedman(1985) observed that maximum strength deployed by any soil element after the starting of seismic shaking will be equal to critical state strength. This maximum strength will be independent of density soil has some time earlier.

In case of clayey soil with λ of order of 0.2, the specific volume is to be increased by about 0.8 for a model having a scale factor of 50. Due to this liquidity index will also be incremented by about 0.8. Due to all this steps, samples generated will be soft which will be difficult to handle as they will easily deform upon disturbance.

Strain

Strain occurs because of change in stress in comparison to that of stiffness of material. Scale factor which is adopted for stiffness would have a direct bearing on scale of strains and other parameters which have any dependence on strains.

It is always recommended that strain scale factor should be unity so that geometric similarity is maintained and mobilization of soil stiffness in both model and prototype is similar at different conditions but the strict conditions in which geotechnical modelling is to be carried out often hinders it.

Displacement

Displacement occurs when field of strains merges within the soil mass and hence the displacement scale is obtained by multiplication of strain scale and length scale. Since the behavior of soil is nonlinear, it will be beneficial in modelling that strains at corresponding points of model and prototype are kept same. This will help in making the strain scale factor equal to one. This could be tedious to attain in gravity models which are singular because in these different scale factors are adopted for stresses and stiffness. If the strains between model and prototype are similar then displacements in model and prototype will be in same ratio as dimensions of model are scaled with respect to the prototype.

If in geotechnical system under analysis relative movements is observed on interface which may be between different blocks of soil together responsible for failure or between the soil and structural element for example pile, steel stirrups then the way in which soil particles at interface behave will depend on relative displacement over the interface and hence a small model may not be able to correctly represent the system response.

2.5. Direct Shear Test

Direct shear test can be performed in laboratory or in field in order to find in-situ parameters such as cohesion (c) and angle of internal friction (ϕ). It is preferred test conducted by the geotechnical engineers.

Since in this test there is no mechanism to measure pore pressure hence the test is preferred under drained conditions however it can be conducted under undrained conditions also. There are 2 mechanisms to conduct this test; strain controlled (screw) and stress controlled (pulley plus weight). Strain controlled is generally preferred in most of cases.

First, saturated sample of soil is placed in shear box which has 2 parts; upper and lower. A normal force is applied from top and when expulsion of pore water stops shearing is introduced on predetermined horizontal plane. At constant normal stress shear displacement is given and shear resistance is recorded on proving dial gauge.

Direct shear test has some limitations such as failure plane is fixed which may not always be the weakest plane. Also there is no mechanism to measure pore pressure. Drainage conditions are not in and stress distribution on failure plane is also not uniform. Stress conditions are known only at failure plane hence it is difficult to draw Mohr circle from single observation obtained from test.

It also has some advantages. Equipment's are simple to use and calibrate and test can be performed under different conditions of saturation, consolidation and drainage. Final decision have to be made by weighing advantages with various disadvantages mentioned above. For samples having discontinuities such as rock samples different equipment and procedure have to be adopted.

2.6. Static Cone Penetration Test

Static Cone penetrometer test (CPT) is a useful method when used simultaneously with various other procedures in conducting analysis of engineering structures. Text below details various equipment's of cone penetrometer test and steps to be followed in conducting the tests. It additionally depicts a few techniques for and direction in translating and utilizing the test outcomes. Consistency in all parts of cone penetrometer testing is wanted.

A few penetrometers of different sorts were utilized in Netherlands and Scandinavia starting around 1900. A cone penetrometer combined with a sleeve or shield was first experimented in Holland in 1936. In 1946, the Dutch cone was produced by Goudsche Machinefabriek of Gouda, first as 2500 kg limit mechanical assembly. A couple of years after, this organization started making equipment of 10000 kg and 2000 kg limit. One of the numerous points of interest of static cone penetrometers is the capacity to separate, or expel, the obscure contact powers that create on the push holes. In static penetrometer testing just the resistance from cone point and contact sleeve is estimated.

Mostly static penetrometer tests consists of a drill rig which is inserted into the ground and pushed below with the help of hydraulic jacks. These hydraulic jacks also help in retrieving these rigs. Truck mounted penetrometer rigs which are self contained in a small space are also available. These presents a advantage being more mobile.

In regions where engineering parameters of soil are known earlier, there static cone penetration tests can be helpful in making accurate estimates of settlements and shear strength of underground soil. Since both static and dynamic testing are available, it is not essential that samples to be tested be completely relied upon as it can be disturbed or may even become irretrievable.

These tests are currently in use by SCS in Iowa, Kansas and Nebraska. Various penetrometers are applied in Midwest by the Corps of Engineers and associated engineering companies. Cone penetrometers were first used by SCS in May 1974.

2.7. Surcharge Pressure due to Strip Load

Modified forms of Boussinesq’s equation are adopted in order to analyze lateral surcharge pressure acting against vertical retaining wall due to the action of point load, line load or strip load. For the strip load acting as surcharge on backfill soil, lateral pressure due to unit surcharge load is usually expressed in form of equation. Lateral pressure due to total surcharge load can be computed direcly through manipulation of variables in equation adopted for unit lateral surcharge pressure. (Kim and Barker (2002)

For complete analysis geometric center of lateral surcharge pressure has to be located accurately and the point at which acting lateral pressure is maximum has to be found out. Strip load acting as surcharge on backfill soil has some practical examples such as retaining wall supporting wall footing, railroad and highway. Earth pressure, water pressure are added to surcharge pressure in order to arrive at total lateral pressure which is used to check stability of structure. If already established formulas and equations are used in order to calculate total lateral surcharge pressure it will save a lot of time in arriving at total lateral surcharge pressure and checking the stability. (Jarquio R (1981)

2.8. Active Earth Pressure due to Distance Surcharge

Coefficient of Active Earth pressure adopted for all calculations is generally due to soil weight. However practically earth pressure may have a combined effect of soil weight and surcharge applied to backfill soil to be retained. Such a solution be arrived at which considers the distance at which load is applied since these days load is applied at a certain distance from wall. Elastic solution which are based on Boussinesq's equation are often considered in order to find lateral earth pressure. It is also explained by Jarquio and Misra (1981) however there is a limitation also in this as elastic theory does not consider soil strength and hence its effect is also not considered while calculating lateral earth pressure acting on wall which is opposite to what evidence suggests. Coulomb method having a limit equilibrium approach is better and reliable in active case since in this case solutions derived are similar to as obtained from upper bound limit analysis Chen (1975) and method of Characterisrics Sokolowskii (1965). Coulomb method has an added advantage that it boundary conditions can be varied in earth pressure problems and then tested. (Motta 1994)

2.9. Earth Pressure against Rigid Retaining Walls

Rigid retaining walls having a combined backfill have been well analyzed and documented since decades. A lot of centrifuge model tests were conducted by Frydman and Keissar (1987) through adoption of different height-width ratios in order to analyze earth pressure acting against retaining wall. The earth pressure has been found to transfer through soil elements in backfill. Take and Valsangkar (2001) also analyzed the effect of variation of this ratio on lateral earth pressure who observed that earth pressure becomes nonlinear due to the effect of stress redistribution. It is declared as soil arching effect by Terzaghi (1943). A case study was also presented by O'Neal and Hagerty (2011) presenting earth pressure acting against rigid retaining wall due to backfill which are confined and granular. The observed earth pressure variation confirms effects such as soil arching as well as vertical shear. Based on experimentation, Fan and Fang (2010) observed that active earth pressure calculated for a wall with limited backfill was significantly lower than that obtained through coulomb's method. (Ishibashi and Lee 1982)

2.10. Pressure on Retaining Walls Due to Repeated Loadings

Repetition of surcharge loading on surface of sand backfill which is held by retaining wall results in an increase in residual and buildup pressures as the load cycle increases. The magnitude of residual and buildup pressure increases as surcharge load comes closer to the wall and as these pressures increases wall becomes less flexible. Net pressure distribution which is determined theoretically by Boussinesq and Burmister is lower than the observed pressure distribution. The empirical methods given by Terzaghi and Rowe (1943) gives much more accurate pressure distribution. The solution provided by Broms and Terzaghi (1943) covered all of observed residual pressure for almost all positions of surcharge load. (Sherif and Mackey 1977)

2.11. Geogrid

The geo-synthetic material ie geogrids are constituted of intersecting grids. Geogrids are composed of materials such as polyester, polyethylene and polypropylene.

These grids are composed of material ribs which are crossed or intersected in two directions during the process of manufacturing. One direction is the machine direction which as name suggests is in the direction same as that of manufacturing. The other direction is perpendicular to this and is known as cross-machine direction (CMD).

Matrix structured materials are formed through this process. Apertures are open spaces formed due to intersection of these ribs. Based on longitudinal and transverse alignment of ribs aperture size varies between 2.5cm and 15cm.

Amongst various geotextiles, geogrids are considered much harder and stiffer. In geogrids stresses due to loads are transferred through these junctions hence strength at junction is essential for them to function properly.

Aggregates are being held up or captured together through geogrids. Through this interlocking of aggregates earthquake becomes stabilized mechanically. The spaces in between ie apertures interlocks the aggregates or soil particles resting above them.

Fig 2.Representation of Geogrid confining the Aggregates (Geotextile Testing and Design Engineer, ASTM STP 952, pp 69116)

Geogrids as described earlier helps in redistribution of load over a much larger area. This property of geogrids helps in making pavement more stabilized and tough during construction.

Various mechanisms of geogrids which comes into effect when used for pavement construction are

Tension Membrane Effect

This mechanism is based on vertical stress distribution concept. Vertical stress arises due to deformed shape of membrane as depicted in figure below. This mechanism was initially considered as primary mechanism but later experiments prove that this is major criteria that must be taken into analysis.

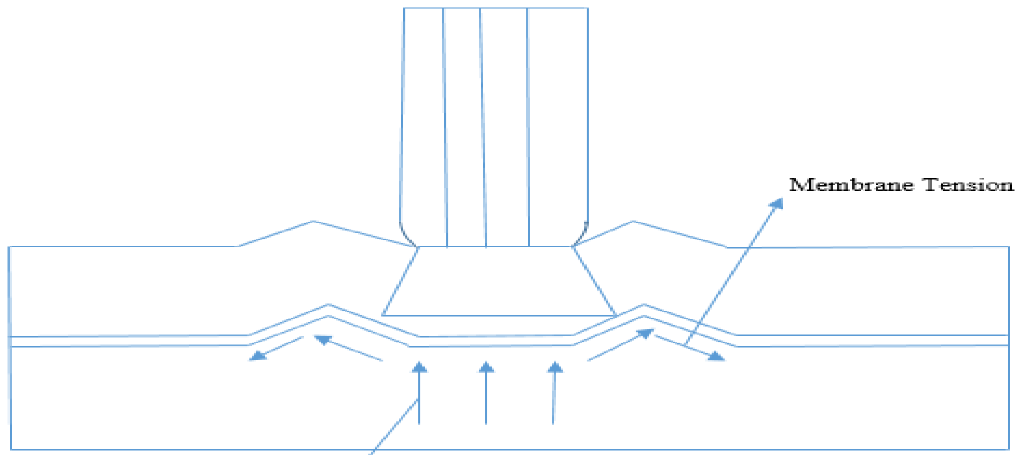

Fig 3.Tension Membrane Effect (Inspiration: Shukla, S.K. and Trivedi, A. 2016"Geotehnical structures with geosynthetics, reinforcement and confinement)

Lateral Restraining Capability

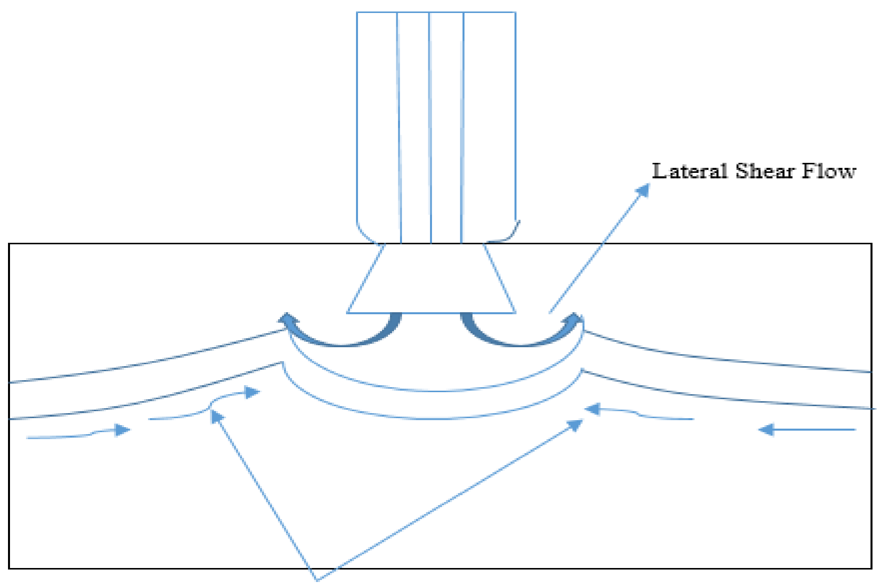

Fig 4. Lateral Restraining Capability (Inspiration: Shukla, S.K. and Trivedi, A. 2016"Geotehnical structures with geosynthetics, reinforcement and confinement)

The stresses produced due to movement of wheels over pavement results in aggregates moving laterally. This movement results in degradation of durability and stability of pavement as a whole. The geogrid installed within the pavement helps in reducing this movement.

Types of Geogrids

On the basis of manufacturing process involved in construction of geogrids it can be classified as

1. Extruded Geogrid
2. Woven Geogrid
3. Bonded Geogrid

Based on the direction of stretching adopted during the manufacturing process it can be classified as

1. Uniaxial Geogrids'
2. Biaxial Geogrids

Uniaxial Geogrid

These type of geogrids are formed by expansion of ribs in the direction same as that of machining i.e. in longitudinal direction. Hence in uniaxial geogrids tensile strength is higher in longitudinal direction and is lower in the direction at right angle to it i.e. in transverse direction.

Biaxial Geogrids

In this case during the process of punching of polymeric sheets, stretching or expansion of grids is carried out in both the directions. Hence tensile strength is equal in both longitudinal as well as transverse direction.

Adoption of Geogrids in Construction of Retaining Wall Structures

During construction of retaining walls geogrids are applied to the backfill soils. Geogrids holds the soil particles together making the retaining wall stable. Reinforcements through geogrids helps in increasing the structural integrity of soils. Through this backfills are confined and loads are also redistributed. The geogrids can be adopted in case of backfill which is soft or sloping ground.

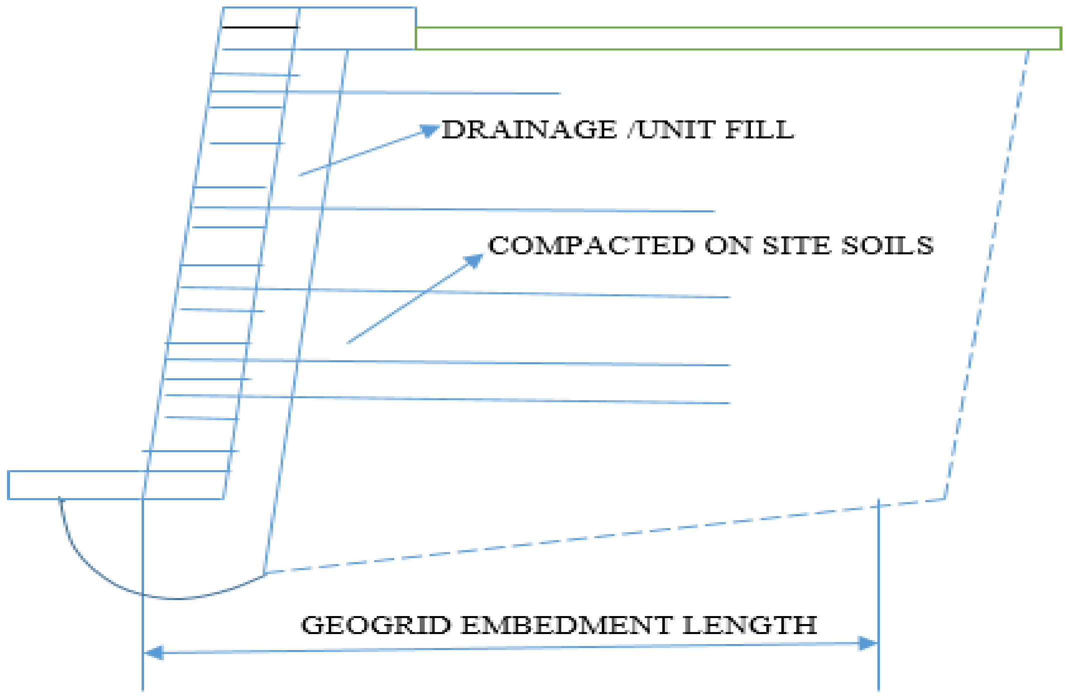

Fig 5. Arrangement of Geogrids in Retaining Walls (Bonaparte, Holtz and Giraud 1987)

As the length of geogrids increases mass of structure also increases. This increased mass will help in constructing taller walls. Through the application of geogrids whole backfill soil mass will behave as a single unit. The type of soil, extent of earth pressure acting against the wall along with various other factors helps in deciding the minimum height from which process of installation of geogrids has to be started.

Various advantages attained by retaining wall which is reinforced through geogrid are

The geogrids applied to the retaining wall makes it much more flexible in nature. The retaining wall whose backfill is reinforced with geogrid can much easily adapt with deformation of foundation in comparison to that of conventional construction since it is much more stiffer.

Due to higher flexibility reinforced retaining wall are much more earthquake resistant.

The construction of reinforced retaining wall can be done in a manner which is more economical than the conventional method. Slope of backfill can be increased which brings about a reduction in cost. Height of wall as well as steepness can be increased through the application of reinforcement materials.

The installed geogrids also provides a protection against deforestation. Hence it provides environmental benefits paving a way forward towards sustainable construction.

Construction of retaining wall with geogrid reinforcement brings quality and the cost of construction is also reduced. This will provide fast and convenient construction.

As the time is passing geogrid reinforced retaining wall is gaining acceptance and its advantages have been well understood and hence its demand has been continuously increasing in field of highways, planning cities, ports and for projects that are focusing on environment.

2.12.Shake Table Test for Dynamic Analysis of Retaining Wall

During some ongoing solid quake, a few disappointments and harms happened in earth retaining structures demonstrated the insufficiency of the conventional pseudo-static plan approach and called attention to the need of execution based criteria for the seismic plan of these structures. This methodology is these days proposed in a few codes and rules for the appraisal of post seismic usefulness of earth-holding structures. In any case, the investigation of the seismic conduct of earth holding dividers speaks to an intricate soil-structure communication issue including cyclic plastic misshaping and huge strains (Zeng and Steedman, 2000). In spite of various investigations attempted to demonstrate the primary trademark this association there is as yet the need of understanding numerous parts of the issue. Hypothetical displaying of this collaboration is very mind boggling since a few variables are engaged with the framework dynamic reaction.

According to worldwide experiences, the reinforced earth walls present a flexible behavior under earthquake loads. However, they can show a considerable deformation. For seismic condition, reinforced earth walls and slopes are often analyzed and designed using pseudo static methods and designers pay less attention to calculate and control seismic deformations.

Most of the reinforced earth walls which were exposed to earthquake in the world have shown lateral deformations but total failure rarely has been reported. The tilting, subsidence, bulging, and face cracking are the common effects of earthquake on reinforced earth walls. There are some instance reports from seismic behavior of reinforced earth walls in practice during the earthquake (Roessing and Sitar 1998) which shows the better performance level of RSWs in comparison to conventional retaining wall systems

2.13. Forced Vibration with Viscous Damping

$mAr^2e^n + cAre^n + KAe^n=0$ (1)

$r^2 + \frac{c}{m} r + \frac{k}{m} = 0$ (2)

Solving equations (1) and (2)

$\frac{c}{2m} > \sqrt[2]{\frac{k}{m}}$ then the roots are real and negative and it is a overdamped case (3)

$\frac{c}{2m} = \sqrt{\frac{k}{m}}$ $r= \frac{-c}{2m}$ then it is a critical damping case (4)

In this case $c=c_c= 2\sqrt{km}$

$\frac{c}{2m} < \sqrt{\frac{k}{m}}$ then the roots are complex and it is a under damped case. (5)

CHAPTER 3

Apparatus and Equipment

3.1. Geometric Model of Retaining Wall

A model of Retaining Wall of size 60cm x 30cm x 30cm was constructed of steel plates having thickness of 3mm. The tank for placing the backfill soil is of size 40cm x 30cm x 30cm. The tank is made up of steel plate at bottom and one side of tank is made up of acrylic sheet of thickness 15mm. Acrylic sheet is provided so as to observe failure pattern of backfill surface due to displacement of retaining wall when load is dropped on backfill surface from a particular height ranging between 10cm and 100cm.

Retaining wall is a 3mm steel plate which is hinged at bottom so that it can displace in both active and passive zone.

Fig 6: Geometrical Model of Retaining Wall

3.2. Loading Mechanism

A 8kg dead load is used is apply load on backfill material of retaining wall. This load can be used to apply static as well as impact load on backfill material. The load is provided with a scale from 0 to 100cm which can be used to determine the height from which load is dropped on backfill material of retaining wall.

Fig 7: Loading Mechanism

3.3. Static Cone Penetrometer and LVDT

This is a instrument which can be used to measure both load acting at the tip of penetrometer and also the displacement at the tip through use of LVDT. Cones of various sizes can be attached to tip to find the load or resistance acting at the tip of cone. Displacement can be measured through narrow stick which is attached to LVDT. This apparatus can be connected with a data logger which is used to record the variation of both displacement and load at tip of static cone penetrometer. A pen drive can be attached to this data logger so as to extract the readings.

Fig 8: Static Cone Penetrometer and LVDT

3.4. High Density Polyethylene (HDPE) Geogrids

Geogrids can be considered as open structures which consist of pre-stressed and high strength bars possessing high creep strength in long as well as short term. Various kinds of geogrids are available these days. Punched and extended sheets of High Density Polyethylene (HDPE) are used to make these geogrids. Bars are placed in direction same as that of principal strength through the process of stretching of sheets. HDPE geogrid design strength can be in higher range with values reaching upto 78 kN/m. The strength of geogrid is employed while providing reinforcements for structures such as embankments, slopes which are steep, retaining walls and various other soil retaining structures. Geogrids makes a bond with soil particles through interlocking and hence serves the purpose of long term reinforcement and hence have emerged as one of most adopted cost reduction method for creating extra horizontal space.

Table 1. HDPE Geogrid Properties from LOVSON ADROIT ENGINEERING LLP

Properties	Values
Wide Width Tensile Strength (kN/m)	54
Creep Strength (kN/m)	21.7
Long Term Design Strength Reduction Factors	
Creep @ 10% strain Limit	2.49
Installation; Sand, Silt & Clay	1.05
Installation; 50mm Minus Well Graded Gravel	1.09
Installation; 75mm Minus Well Graded Gravel	1.13
Installation; 125mm Minus Well Graded Gravel	1.15
Durability/Aging $4 \leq Ph \leq 9$	1.0
Biological Degradation	1.0
Joint Efficiency	1.0

3.5.Shaking Table for Dynamic Analysis

Shaking table is used in order to analyze dynamic behavior of retaining wall in response to vibrations of various frequencies which is provided through shaking table. Shaking table can be used to provide both horizontal and vertical accelerations. There is a steel plate which is provided on the top of shaking table. There are holes on this steel plate which are used to connect structure to this apparatus. Structure is connected to this plate with the help of bolts so that there is no relative motion between the steel plate of shaking table and the retaining wall. Frequency of vibration can be adjusted through a roller which is provided to shaking table.

Specifications of shaking table are:

1. Motion- Vertical and Horizontal

2. Loading Capacity- VST AND HST 35 Kg

3. Operating Frequency- 0 to 25 Hz

4. Amplitude- 0 to 8mm for VST

+5mm 25Hz/+1mm 25H/+2mm 15 Hz and 10mm saw tooth 5H for HST

5.Table Size- 400mm x 400mm with 390mmϕ rotating table

Fig 9: Shake Table

CHAPTER 4

Experiments Conducted and Results

4.1. Classification of Backfill Soil

A. Sieve Analysis

Table 2: Sieve Analysis

IS Sieve(mm)	Weight Retained	% Weight Retained	Cumulative % Weight Retained	Cumulative % Weight passing
4.75	5.3	2.65	2.65	97.35
2.36	6.1	3.05	5.7	94.3
1.18	24.2	12.1	17.8	82.2
0.6	47.6	23.8	41.6	58.4
0.3	73.5	36.75	78.35	21.65
0.15	30.6	15.3	93.65	6.35
0.075	3.7	1.85	95.5	4.5
Pan	9			

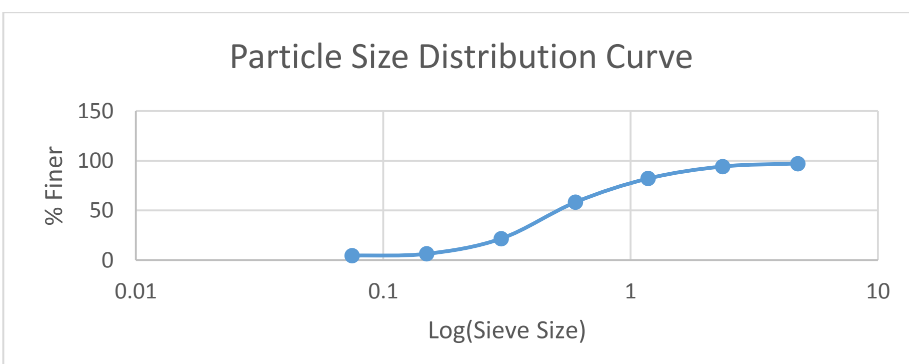

Fig 10: Particle Size Distribution Curve

According to IS Code % passing through 0.075mm sieve is less than 50% hence it is course grained soil.

Since sand fraction (0.075mm<d<4.75mm)>Gravel fraction (d>4.75)

Hence it is classified as Sand.

From the particle size distribution curve

D_{60}=0.65mm D_{30}=0.4mm D_{10}=0.18mm

Coefficient of Uniformity, $C_u = \frac{D_{60}}{D_{10}} = 3.61$

Coefficient of Curvature, $C_c = \frac{D_{30} X\, D_{30}}{D_{60} x D_{10}} = 1.64$

Hence it is a Poorly Graded Sand.

B.Liquid Limit

First water content of natural soil is determined through oven drying method. It is given by W_n=14%

Then liquid limit is determined through Casagrande Apparatus. Soil is filled in Casagrande apparatus and cut by a standard tool having a base width of 12mm. Number of blows required by a soil to flow together a distance of ½" or 12mm is noted at different water contents. Water content corresponding to 25 number of blows is noted as liquid limit of soil.

Table 3: Liquid Limit Determination

Water Content w%	No of Blows
17	38
20	31
28	17

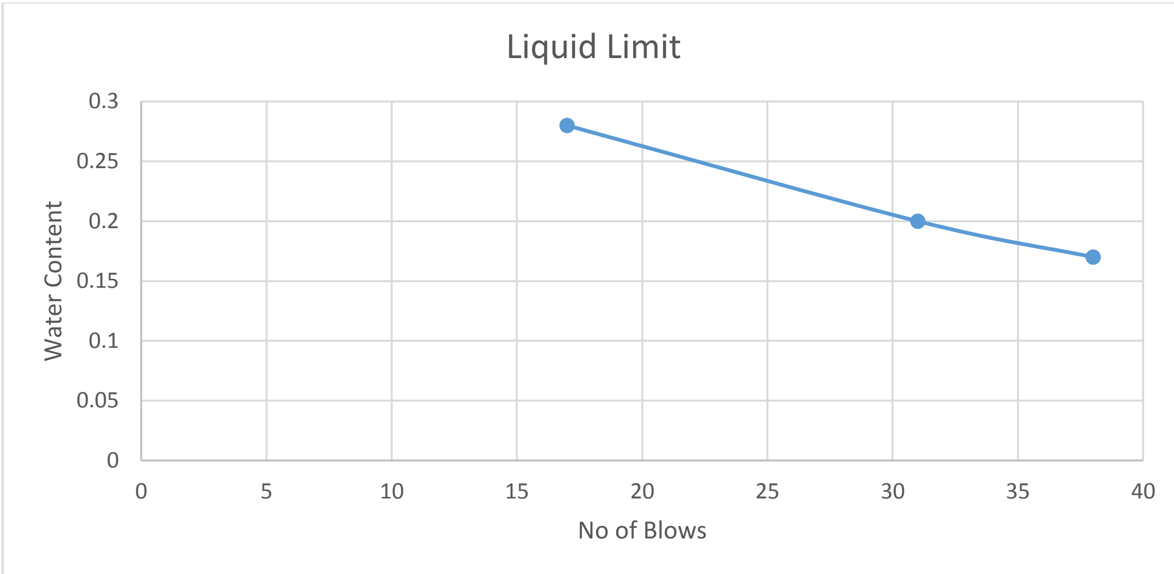

Fig 11: Liquid Limit Determination

From the graph it can be concluded that water content corresponding to 25 number of blows is 0.23

Hence Liquid Limit of soil, w_l=23%

4.2. Direct Shear Test

Direct Shear test is conducted to determine the shear strength properties of soil such as cohesion(c) and angle of internal friction (ϕ). Different amounts of vertical stresses are applied on the soil and shear load on soil at failure was noted to determine the shear strength of soil at failure. These readings were used to plot the Mohr envelope of soil which were used to determine cohesion and angle of internal friction.

Fig 12: Direct Shear Test

Table 4: Direct Shear Test Results

Direct Stress(KN/m^2)	Shear Load(N)	Shear Stress(KN/m^2)
0	17	4.72
50	21	31.56
100	24	45.78
150	28	56.23

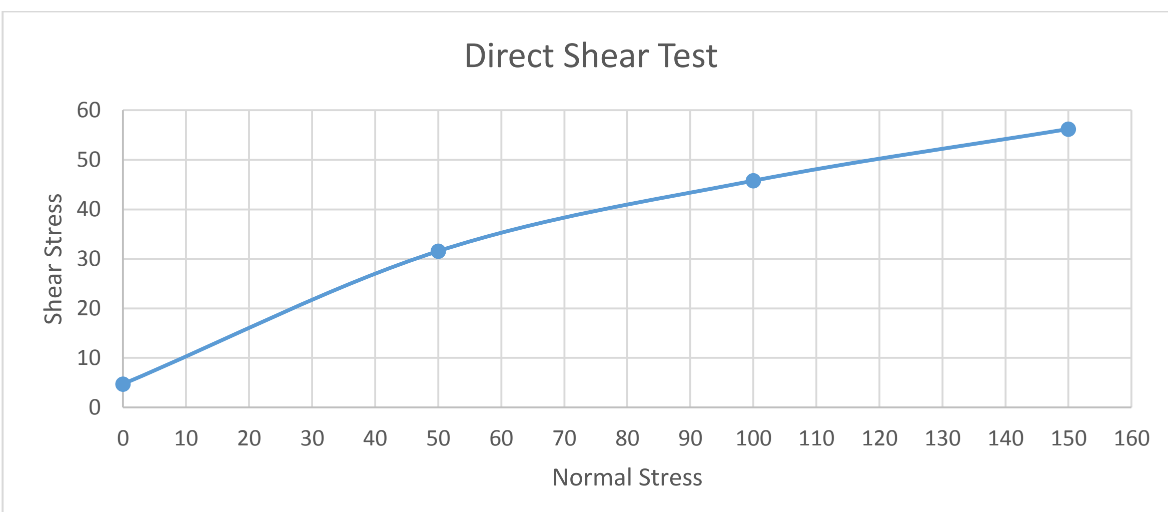

Fig 13: Mohr Envelope

From the direct shear mohr envelope it can be concluded that

Cohesion, C=4.72 KN/m^2

Angle of internal friction=36°

4.3. Displacement of Retaining Wall

In order to get maximum displacement of a retaining wall load was dropped from a height of 20cm and displacement was obtained for 3 conditions ie when load is dropped at a distance of 10cm from wall, 15cm from wall and 20cm from wall. Displacement of wall is coming to be maximum when load is dropped at a distance of 10cm.

Table 5: Displacement of retaining wall at a drop of 20cm

	Displacement when load is applied at a distance of		
Height(cm)	10cm	15cm	20cm
10	0.4	0.3	0.2
20	0.6	0.5	0.4
30	0.8	0.6	0.5

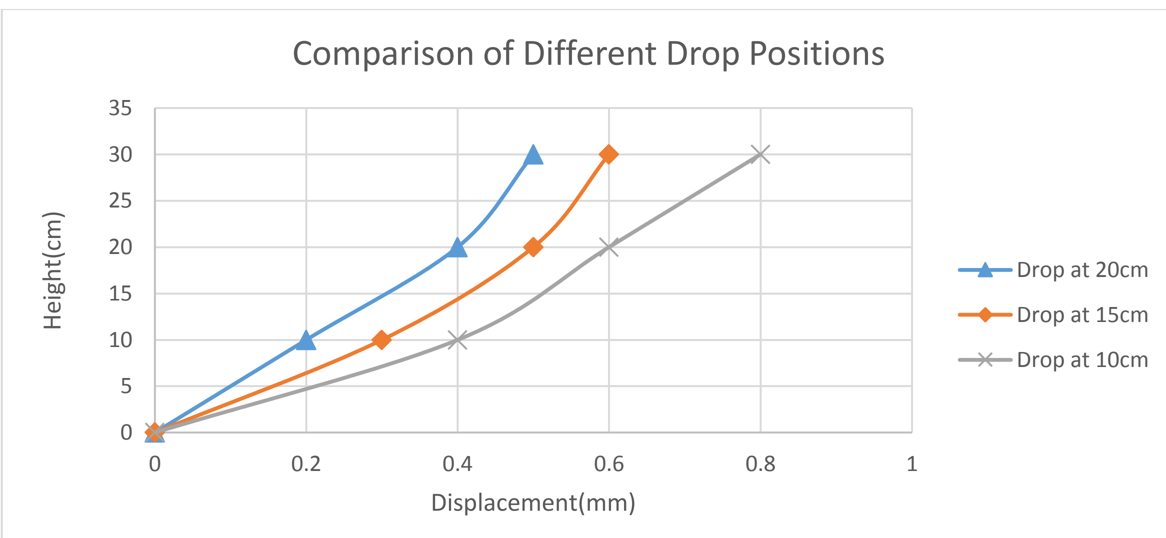

Fig 14: Displacement of Retaining Wall at a drop of 20cm

Since displacement of retaining wall is coming out to be maximum when load is dropped from a distance of 20cm hence now load is dropped from a height of 50cm and displacement of a retaining wall is computed. The displacement of a retaining wall is then compared when a grid of geocells are applied at a height of every 6cm in backfill soil

Table 6: Displacement of retaining wall at a drop of 50cm

Height(cm)	Displacement of retaining wall with geogrid	Displacement of retaining wall without geogrid
10	0.8	0.5
20	1.1	0.8
30	1.4	1.0

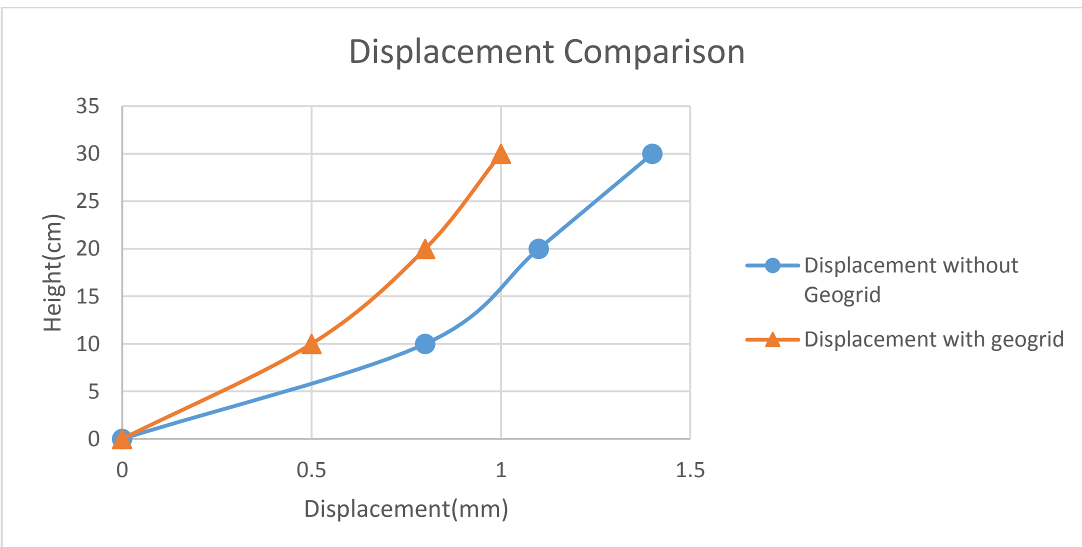

Fig 15: Displacement of retaining wall at a drop of 50cm

4.4. Shake table Test

In order to analyze dynamic behavior of retaining wall in response to accelerations provided by the retaining wall, horizontal accelerations are provided at different frequencies of 15 Hz, 20 Hz and 25 Hz. Since the maximum weight that can be applied to shake table is 35 Kg hence the size of model is reduced keeping the scale factor same as earlier. Now the model has a size of 40cm x 15cm x 15cm with the surcharge tank of retaining wall having a size of 30cm x 15cm x 15cm.

Displacement of retaining wall is gauged at heights of 5cm, 10cm and 15cm through placement of LVDT at corresponding heights which is connected to a data logger. These displacements are observed by providing horizontal accelerations at frequencies of 15 Hz, 20 Hz and 25 Hz.

Table 7: Displacement of Retaining Wall at different Frequencies

Height (cm)	Displacement of Retaining Wall at Frequency of		
	15 Hz	20 Hz	25 Hz
5	3.1	4.3	8.4
10	5.3	6.2	11.1
15	6.2	7.1	13.2

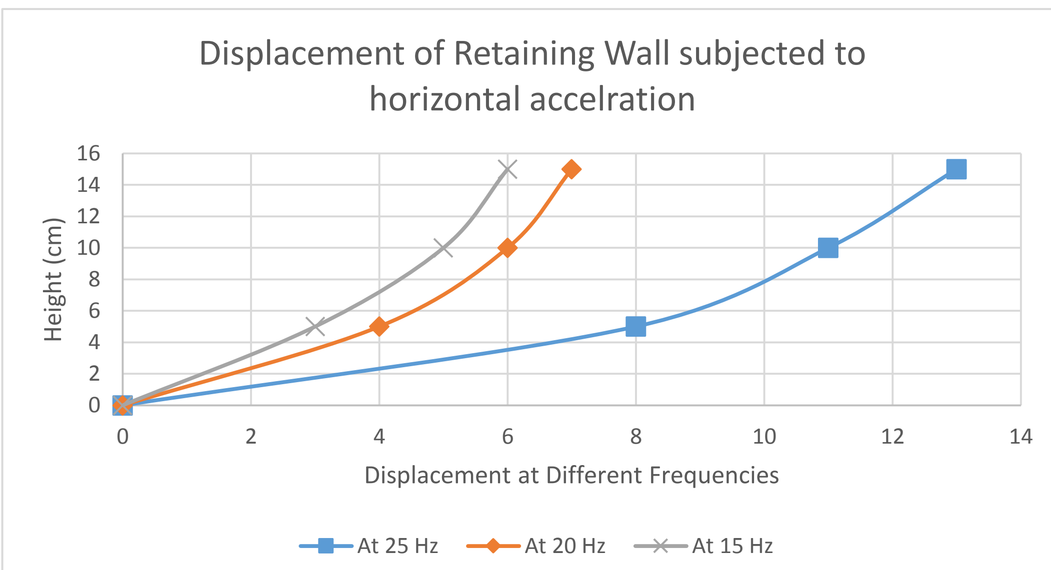

Fig 16: Displacement of Retaining Wall subjected to horizontal acceleration

Soil + Apparatus = 32 Kg

Soil= 14 Kg

Displacement of Retaining wall is suddenly increased at a frequency of 25 Hz. It happens because this frequency matches with the natural frequency of vibration of retaining wall structure.

Table 8: Observed and Calculated Frequency of Vibration at Resonance

Natural Frequency of Vibration	
Observed	Calculated
25 Hz	39 Hz

According to IS Code 1893 (2002) fundamental period of vibration is given by

T_s= 0.085 x $h^{0.75}$= 0.025 sec

Fundamental Frequency, f= $\frac{1}{T}$= 39 Hz

Now the displacement of retaining wall at a frequency of 25 Hz is compared with and without the use of geogrid. First the geogrid is placed in horizontal direction.

Fig 17: Measurement of Dynamic Displacement of Retaining Wall

Table 9: Dynamic Displacement of Retaining Wall with Geogrid

Height	Displacement of Retaining Wall at a Frequency of 25 Hz		
	Without Geogrid	With Horizontal Geogrid	With Vertical Geogrid
5	8.1	5.4	6.2
10	11.3	7.1	8.1
15	13.2	9.2	10.3

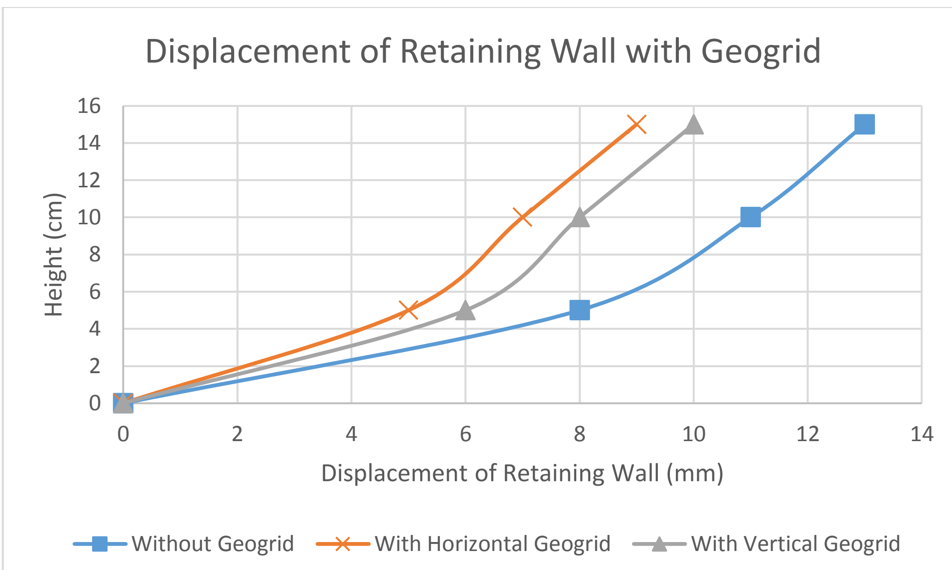

Fig 18: Dynamic Displacement of Retaining Wall with Geogrid

CHAPTER 5

Conclusions

A retaining wall has been successfully modelled in order to determine its Load V/s Displacement characteristics under various loading conditions.

- Displacement of wall decreases as point of application of impact load moves away from the wall.
- As the drop of load increases displacement of wall increases due to more impact stresses being transferred.
- There is significant reduction in displacement of retaining wall on the application of geogrid as a reinforcement in backfill.
- More the stiffer the geogrid is lesser will be the displacement in retaining wall.
- Displacement of retaining wall increases suddenly when the frequency of vibration matches with the natural frequency of vibration of retaining wall.
- Geogrids when placed in both horizontal and vertical orientations help in reducing the dynamic displacements of retaining walls.
- However geogrids placed in horizontal orientation are much more effective in reducing displacement than that in vertical orientation.

CHAPTER 6

Future Scope

- Effect of Different Geogrid used as a reinforcement in backfill on Load V/s displacement characteristics can be studied.
- Different Configurations of Geogrid can be tried i.e. horizontal, vertical, inclined and its Load V/s Displacement characteristics can be studied.
- Spacing’s of Geogrid can be varied and its effect on Load V/s Displacement characteristics can be studied.

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