Chapter 2
LITERATURE REVIEW

2.1 GENERAL

It has been a long time since boxes made of hexagonal mesh fabric, known as gabions, have become an effective technical solution in the design, construction and maintenance of a variety of protective flexible structures. Gabions, by virtue of their matchless strength, excellent engineering adaptability and reliability, have become the chosen building material for a tremendous variety of construction works. These include road construction, river training, weirs, control and training of natural and flood waters, earth retaining structures, water recharge dams, rock slide protection, soil erosion protection and bridge protection.

An extensive literature survey was conducted to identify the research works conducted on gabion faced retaining walls. But it was truly disheartening to note that, even though the construction of these walls is booming up in every nook and corner of the world, the research works conducted to understand the behaviour of these walls are very much limited in number, which is clearly proved at the end of this chapter. This obviously means that the present situation may have disastrous outcomes if the prevailing practice continues. Hence the author shifted her attention to collection of literature based on behaviour of retaining walls in general which may be categorised as gravity walls, reinforced soil walls, segmental retaining walls and gabion faced walls. The studies in this chapter are divided mainly into four groups as follows:

i. Literature on experimental investigations
ii. Analytical studies
iii. Studies on numerical modelling
iv. Economic studies
Literatures on experimental works were collected to understand how the retaining wall can be modelled physically and how the deformations can be measured. Works on finite element modelling of different types of retaining walls were collected to understand how a FE prediction model could be developed to simulate the behaviour of gabion faced retaining walls by modelling the different components individually. Analytical works on retaining walls were collected to understand how the design charts could be developed from the results of the present study.

2.2 EXPERIMENTAL INVESTIGATIONS

2.2.1 Gravity walls

Tweedie et al. (1998) constructed a 4.88m high retaining wall test facility to test tire shreds as retaining wall backfill. The front wall of the facility could be rotated outward away from the fill and was instrumented to measure the horizontal stress. Measurement of movement within the backfill and settlement of the backfill surface during wall rotation allowed estimation of the pattern of movement within the fill. Huang et al. (1999) developed a 2D model retaining wall system to investigate the effect of the bending rigidity of a wall, supported at the top and bottom, on the lateral pressure distribution at completion of backfilling condition.

Briaud et al. (2000) proposed the use of a vertically reinforced wall, a new type of top-down retaining wall. Typically, three to four rows of 1m diameter cemented soil columns were constructed to the depth of soil to be retained. After one year, the horizontal movements and vertical settlements of the wall were very close to the movements of the similar size tieback wall built at the same site. The authors claim this performance as an indication of the viability of this new wall type.

Lee et al. (2001) presented experimental data concerning the lateral earth pressures acting against a small-scale retaining wall, with a backfill consisting of waste foundry sand (WFS) mixtures. It was seen that the lateral earth
pressures on the wall depend on the backfilling sequence, the type and drainage characteristics of the WFS mixture, and the shear strength of the mixtures. Judging from the retaining wall model tests, the authors recommend that the backfilling of a 6-m high retaining wall can be completed in two days with two backfilling stages.

Chen and Fang (2002) investigated the effect of stress-history on the earth pressure at rest, using an instrumented stiff model retaining wall of 1.6m high using dry Ottawa sand. Calculations based on the experimental data indicated that the resultant forces were located at 0.34 H to 0.35 H above the base of the wall. For the backfill, the measured coefficient of earth pressure at rest, $K_o$, was found to be independent of the thickness of the fill.

Fang et al. (2002) conducted experiments on a vertical rigid wall which moved towards a mass of dry sand and the earth pressure was measured. As the wall movement exceeded 12% of the wall height, the authors concluded that the passive earth thrust would reach a constant value, regardless of the initial density of backfill.

Hanna and Khoury (2005) conducted an experimental investigation on the passive earth pressure of overconsolidated cohesionless soil on retaining walls, using a prototype model of a vertical rough wall, retaining horizontal backfill. It was seen that the over consolidation ratio and the soil condition below the founding level significantly affect the value of the coefficient of passive earth pressure on these walls. Design charts and formulae were also developed for practical use.

Bentler and Labuz (2006) used earth pressure cells, tilt meters, strain gauges, inclinometer casings, and survey reflectors during the construction of a 7.9m high reinforced concrete cantilever retaining wall. The passive earth pressure in front of the shear key was found to be less than 10% of the design value and the vertical stress below the heel was greater than that at the toe.
Compaction-induced lateral stresses on the stem were sometimes twice the vertical stress.

Villemus et al. (2007) conducted experimental investigations in the laboratory and in situ on dry stone gravity retaining walls to seek the knowledge necessary to ensure the stability of these structures. From the results of the experiments, a mathematical model was developed for calculating the stability of dry stone retaining walls. These tests also determined the limits of monolithic behaviour of the masonry, thus defining failure and enabling the fulfillment of practical engineering requirements.

### 2.2.2 Reinforced earth walls

Fukuoka et al. (1986) experimented on a 5 m high experimental fabric faced retaining wall reinforced with columns and steel rod anchors. It was observed that the fabric faced multiple anchored retaining wall is economical, easy to construct and very stable and the steel anchor rods can be used for a long time if they are coated with paint or asphalt.

Fabian and Fourie (1988) built large geotextile reinforced clay wall models to investigate the mechanism of clay-geotextile interaction and the effects of a low-cost, non-woven, needle-punched geotextile reinforcement on the load-bearing capacity of a silty clay. No face panels were used. The wrapped back geotextile reinforcement provided the face of the wall. The results of the testing programme were promising and encourage further research into the applicability of cohesive soils in geotextile-reinforced soil structures which might have great economic significance in areas where good quality granular backfill is not readily available.

Horiya et al. (1988) described an experimental study on a reinforced earth retaining wall constructed using the Hi - Tex wall method. The method of construction was using plate anchors (anchors with bearing plate) and a geotextile acting as a membrane. Pullout tests were conducted on the anchors
with bearing plates to understand the sliding behaviour of this new system. After that, execution of the full scale wall was carried out.

In order to evaluate the behaviour and stability of the embankment during and after execution, the deformations in the horizontal and vertical directions of the wall slope, the tensile forces developed in the anchor rods and the stresses developed in the steel pipes were measured.

Rao et al. (1988) presented three dimensional model studies on reinforced earth walls built and tested using “built to failure” technique. The models were prepared using aluminum foil reinforcement and aluminum sheet facing. The model tests revealed that the failure height by Rankine’s theory gives conservative height of failure and that the tension at the joint of reinforcement with the facing is not zero, corroborating the findings of other researchers.

Fannin and Hermann (1990) built and monitored a sloped reinforced soil wall comprising of two sections with a different arrangement and spacing of geogrid reinforcement. The loading conditions used for the study were self weight, a cycle of surcharge load and permanent surcharge loading of the wall crest. Instrumentation was used to measure the force and strain in the reinforcement as well as pressure, strain and temperature in the soil mass. The performance data were collected for 20 months after the completion of construction. The authors found that the mechanisms of behaviour were according to expectations.

Fishman et al. (1993) instrumented an earth reinforced retaining wall to measure the movements of wall faces, lateral earth pressure, vertical stress in the soil mass, strains in the soil and strains along the reinforcement in the field. The wall was reinforced with geogrids and had a full height precast concrete wall facing. They observed that the computation of maximum tension in the reinforcement using the conventional design procedure is satisfactory.
Palmeira and Lanz (1994) presented a study of geotextile reinforced model walls subjected to vertical surcharge. Different reinforcement layouts were tested. Vertical stress distributions were measured at the base of the walls as well as internal displacements and horizontal displacements at the face of the wall. Comparisons between predictions and measurements were also made. The results showed that the reinforcement arrangement used in the wall can significantly affect its face displacements and the stresses at its base.

Wiseman and Shani (1994) described the construction of retaining walls in which the soil is reinforced by panels of welded wire mesh sandwiched between two geotextile layers heat bonded to each other, with precast concrete elements used as facing. Ten separate walls retaining backfill for the city streets with 35° surcharge slopes were constructed in heights varying from 3m to 15m. Pullout tests were conducted on welded wire mesh and geomesh. The authors, from their experience, are of the opinion that geomesh reinforced soil retaining walls have performed very satisfactorily during Israel’s wettest winter in 200 years.

Isabel et al. (1996) described a new method of retaining wall construction that combined the reinforced earth technique with a conventional brick wall. The results showed that even short lengths of reinforcement significantly increased the load carrying capacity of a brick retaining wall. Although brick faced retaining walls do not obey the original principles of flexibility, the inclusion of the geotextile as reinforcement, allowed the construction as a whole to endure considerable movement before collapse, showing a high degree of ductility.

Ochiai and Fukuda (1996) conducted full scale failure experiments to study the behavioural characteristics of geotextile reinforced soil walls with different facings like discrete concrete panels, concrete blocks and expanded polystyrol blocks. With the help of the experiments and FEA, the reinforcing performance and mechanism were studied. From the results, it was found that
the deformation of a wall during the construction can be restrained by the gravity resistance of its facing.

Porbaha and Goodings (1996) built twenty four reduced scale models of vertical and steeply sloping (1 H : 6 V) reinforced soil walls using kaolin as the backfill, reinforced with a nonwoven geotextile simulant, and loaded to failure under increasing self weight in the geotechnical centrifuge. Models were constructed on either firm or rigid foundations, and different lengths of reinforcement were tested. A stability analysis using the two-dimensional limit-equilibrium simplified Bishop method incorporating reinforcement, was found to be a good predictor of the behaviour of the models based on calculated factors of safety at failure.

Iyer et al. (2001) studied the effect of bamboo strips as reinforcing elements with and without braided coir rope interface in model retaining walls. Ferro cement panels were used as facing elements tied to the reinforcing elements and backfill used was sea sand. They concluded that the high modulus of bamboo and the frictional characteristics of braided rope can be best made of in retaining walls up to 2m height.

Garga and O’ Shaughnessy (2002) constructed a 57m high x 17m wide instrumented test fill, comprising both retaining wall and tire-reinforced slope sections to study the performance of tire-reinforced earth fills. Approximately 10,000 tires were used in both cohesionless and cohesive backfills. The testing program included plate load tests, field pull out tests on tire mats, water quality assessment in the field and lab and other complementary lab testing. The authors demonstrated the practical feasibility of constructing reinforced earth fills using scrap tires. The results from the tests showed that the negative wall friction increases the active thrust when the retaining wall becomes more compressible than the backfill. The authors also gave recommendations for the design of retaining walls using scrap tires.
Hegazy (2002) performed laboratory tests to design the connections for a geosynthetic mechanically stabilised earth retaining wall. The connection strength was determined where the connection fails due to rupture of geosynthetic reinforcement (rupture criterion) or pullout of geosynthetic (serviceability criterion), whichever occurs first. A database of laboratory geosynthetic connection tests using different types of geogrid and facing blocks was collected and the design trends of these two parameters were recommended as a function of the confining stress.

Ma and Wu (2004) described the independent full facing reinforced soil wall system and the Fox wall project, with the measured data, and discussed the wall performance. Design implications concerning the initial setback of facing panels, reinforcement strength requirement, and lateral forces on facing panels were also addressed in the paper.

Frankowska (2005) presented the results of the 33-months monitoring of a reinforced wall with geosynthetic wrap-around facing. The wall was exposed to natural weather conditions and the following variables were measured: horizontal and vertical displacements at the surface of the wall faces, the wall settlement and the reduction of basic mechanical properties of geosynthetic after using it as the reinforcement. The observed strength losses of geosynthetic ranged from 83% to 97% over a period of 33 months.

Chen et al. (2007) simulated a clayey vertical geotextile-reinforced earth wall in a wet state due to poor drainage conditions, after several consecutive days of heavy rainfall, by a series of centrifuge models. The models were constructed using clayey soil very close to its liquid limit. Through centrifugal tests on these models, the effectiveness of various reinforcement arrangements was examined. It was seen that for any reinforcement length, there exists a critical value beyond which no further improvement can be attained. Also, smaller vertical reinforcement spacing leads to shorter critical reinforcement length. Four failure modes were observed in this study and their evolution for each reinforcement arrangement was also demonstrated.
Won and Kim (2007) measured the local deformation of geosynthetics, such as geogrids, and nonwoven and woven geotextiles to examine the stability of geosynthetic-reinforced soil (GRS) structures. They proposed a new, more convenient method, to measure the deformation behaviour of nonwoven geotextiles using a strain gauge and examined its suitability via laboratory tests and field trials on two GRS walls.

Shinde and Mandal (2007) carried out several experiments to understand the deformation behaviour of reinforced soil retaining wall with limited fill zones under vertical surcharge strip loading. Panel displacements and strain distribution along geogrid layers were observed. Effectiveness of the reinforced soil wall was also evaluated using a geogrid material. Finite element analysis was also carried out using commercial software PLAXIS version 8 for the above problem without and with anchoring of reinforced soil retaining wall in the limited fill zone. The results were compared and showed good agreement.

Chen and Chiu (2008) performed model tests on nine model geocell retaining walls to examine the effect of the geocells as a major material in retaining structures and also to study the failure mechanism of the said structures under surcharge. Results showed that the deformation on the wall face and the backfill settlement both increased with increasing facing angle and surcharge.

### 2.2.3 Segmental retaining walls

Segmental retaining walls (SRWs) form a new generation of reinforced earth retaining walls where flexible facing units are replaced with modular block facing units. The facing units may be of reinforced concrete, precast concrete or brick units. The reinforcement may be geogrids or geotextiles. An important criterion in the design of these walls is the connection strength between the facing units and the reinforcement. The overall strength of the structure is thus imparted by the rigidity of the facing units and the friction between soil and reinforcement.
A hybrid segmental retaining wall system using both steel and geosynthetic reinforcement failed in 1998. Collin (2001) analysed this segmental retaining wall failure with respect to the design, and construction to determine the cause/causes of the failure. The results of the forensic analysis were presented along with the remedial measures necessary to correct the problem. The analysis identified that the poor connection between the soil reinforcement and the segmental concrete units was the primary cause of the failure. The design of the wall used a proprietary software program that did not consider the connection strength of the hybrid system used. The remediation included the removal of the wall face and reconstruction increasing the connection capacity between the segmental concrete units and the reinforcement by a factor of three.

Yoo and Lee (2003) presented the measured behaviour of an anchored segmental retaining wall. To understand the overall mechanical behaviour of the anchored segmental retaining wall and to confirm the applicability of the design assumptions, an extensive monitoring program was implemented for a 7-m-high anchored segmental retaining wall.

The results showed that the maximum lateral wall displacement was comparable to or less than that of a typical geosynthetic-reinforced wall. The measured and the inferred horizontal earth pressures showed that the horizontal earth pressures exerting on the wall facing were greater than those inferred from the Rankine active state of stress and the Coulomb wedge analysis, approaching the at-rest state.

Yoo and Jung (2004) presented the observed behaviour of a geosynthetic reinforced segmental retaining wall. A 5.6 m high full scale wall in a tiered configuration was constructed and instrumented, in an attempt to examine the mechanical behaviour and to collect relevant data that will help improve the current design approaches. It was shown that for walls on a less competent foundation, significant post construction wall movements may occur.
2.2.4 Gabion faced retaining walls

Garg (1997) highlighted two innovative technologies for stabilisation of slopes. One was a reinforced gabion wall and the other was an anchored drum diaphragm wall implemented successfully in the Garhwal Himalaya during eighties to improve stability of slopes at comparatively lesser cost and time than conventional retaining walls.

Ferriolo et al. (1997) presented the details regarding the use of flexible gabion structures for landslide, road protection and river training works. The authors explained the phenomena of landslide and erosion as modification of equilibrium condition of soil at specific surfaces due to a natural configuration or due to a human activity. Gabion mattresses being flexible structures have added advantages in their use in these areas. It is also mentioned that the internal structure details of the gabions such as opening size, double twist mesh, hexagonal shape, wire diameter, extent of galvanisation, diaphragms and joint details play an important role in the functioning of the structure as a whole.

Simac et al. (1997 A and B) described the design and construction of the MSE walls on the Tellico plains to Robbinsville highway. The walls were built with hybrid wall system components, consisting of geogrid reinforcement and PVC coated gabion baskets. The selection of these materials was based primarily on the presence of a chemically active environment, availability of an economical fill source, aesthetic appearance and overall cost. The paper summarised the design procedure utilised to ensure wall stability along a mountainous highway alignment. The paper also examined how the general MSE design guidelines presented in the project specifications can be augmented with currently accepted methods of analysis to provide a safe but economical wall design.

The authors concluded that hybrid MSE systems like the one mentioned above can be successfully designed by implementing currently accepted methods of analysis for geosynthetic reinforced soil walls. But appropriate
facing connection tests should be done before implementation of this system. The latter paper also describes the specialised laboratory testing that was carried out to ensure that the connection formed between gabion baskets and reinforcement was adequate.

Bergado et al. (2000 B) studied the horizontal deformations of gabion walls on a fully instrumented test embankment reinforced with hexagonal wire, constructed on a soft Bangkok clay foundation in Thailand. The reinforced wall consisted of an inclined gabion facing on one side and a sloping unreinforced sand wall on the opposite side, with a total height of 6 m. Two different types of hexagonal wire meshes were utilized for the study. The wall system was extensively instrumented both in the foundation subsoil and the embankment, in order to monitor the behaviour of the wall both during and after the construction phase. A maximum settlement of 0.35 m was observed 200 days after construction. It was found that there is a direct correlation between displacement and stress in hexagonal wire mesh reinforcement. For both types of meshes, the maximum deformation was observed in the top most layers.

Bergado et al. (2001) conducted pullout tests on hexagonal wire mesh of gabions embedded in silty sand locally known as Ayuttaya sand to investigate the soil reinforcement interaction. Two types of hexagonal wire mesh were tested, namely: (a) galvanised (zinc-coated) which had smaller aperture (cell) dimension of 60 mm x 80 mm and (b) PVC coated which had larger aperture (cell) dimension of 80 mm x 100 mm. The tests were conducted under normal pressures ranging from 35 to 91 kPa and the specimens were pulled at a rate of 1 mm/min. The total pullout resistance of hexagonal wire mesh reinforcement consists of two components, namely: friction and bearing resistance. It was seen that the bearing resistance is higher than the friction resistance for both types of reinforcement. Higher friction and bearing resistances were obtained with increasing normal pressures. The friction and bearing resistances mobilised on the galvanised wire mesh were greater than the PVC-coated wire mesh, due to higher friction coefficient as well as greater number of transverse and longitudinal members (elements) per unit width in the former than the
latter. The authors proposed an analytical method for predicting the pullout resistance and displacement relation using the basic soil and reinforcement properties which agreed reasonably well with the test results.

2.3 ANALYTICAL STUDIES

2.3.1 Gravity walls

Bang and Hwang (1986) developed an approximate analytical solution to estimate the developed lateral earth pressures behind rigid retaining walls experiencing various types of outward movements with horizontal cohesionless backfill soil. Various stages of wall movement, starting from an at-rest condition to a full active condition were included.

Day (1994) examined the effect, which the stiffness of the lateral support system has on the movement of ground and adjacent buildings. The properties of both the supported material and the support system which influence the movement of the retaining structures were identified with the illustrations of the performance of a variety of retaining structures. It was stated that the movement of the ground behind a retaining structure is dependent on the consistency of the ground, the in-situ stress condition and the stiffness of the support system.

Hazarika and Matsuzawa (1996) developed a new numerical method, based on a smeared shear band technique for the analysis of earth pressure that incorporates two shear bands for a localised element. The method, which is valid for plane strain condition, was applied to explain the generation of the active earth pressure against a rigid retaining wall for different modes of the displacement that the wall is likely to undergo.

Chang (1997) presented a simple analysis method for predicting the lateral earth pressure at any wall displacement behind a rotating wall using a modified Coulomb's solution of active earth pressure. The deformation pattern and the associated mobilisation of shearing resistance in the soil as affected by
the wall movement are considered in a simplified manner. The method was validated with solutions from FEM and observations from model tests.

Filz and Duncan (1997) developed a simple theory for calculating the magnitude of vertical shear loads on nonmoving walls. Retaining walls that do not move are customarily designed based on the assumption of at-rest conditions, with no consideration of vertical shear loads applied by the backfill. However, field and laboratory measurements have shown that vertical shear loads do act on nonmoving walls. Typical results from the theory incorporating vertical shear loads were also discussed.

Jalla (1999) presented details of the design methodology of multiple level retaining walls as used in residential construction. Each tier of multiple retaining walls should be safe against sliding, overturning, and bearing capacity failure. In addition, the global slope stability behaviour should also be checked.

Greco (2001) showed that the wall stability against overturning can, however, be assessed using the position of the resultant force on the base, which is unaffected by the assumed thrust surface and contrary to overturning, safety factors against sliding and bearing capacity are unaffected by the assumed thrust surface.

Long (2001) prepared a database of some 300 case histories of wall and ground movements due to deep excavations worldwide. It was analysed that for stiff soil sites, movements were generally less than those suggested in the published well-known relationships. He suggested that, in the case of cantilever walls and for all walls in stiff soils worldwide, design practice is conservative and the inclusion of a cantilever stage at the beginning of a construction sequence seems to be the main cause of unusually large movements.

Kim and Barker (2002) generated values of equivalent height of soil $h_{eq}$ for the live load model in the AASHTO specifications acting on a gravity retaining wall based on the elastic theory for determining soil pressures within
a soil mass due to loads on the surface. The paper discusses the theoretical background, an analytical approach to estimation of actual earth pressure, a number of innovative approaches to obtain a simplified pressure distribution, an extensive parametric study, calibration procedures for the traditional method and recommendations.

Wang (2007) presented approximate but analytical-based solutions for computing the lateral force and centroid location induced by horizontal and vertical surcharge surface loads resting on a cross-anisotropic backfill. The surcharge loading types include: point load, finite line load, and uniform rectangular area load. The planes of cross-anisotropy were assumed to be parallel to the ground surface of the backfill. The results showed that both the lateral force and centroid location in a cross-anisotropic backfill were quite different from those in an isotropic one.

### 2.3.2 Reinforced earth walls

Saran and Talwar (1983) investigated a form of soil reinforcing for retaining walls where the lateral pressure on conventional retaining walls is sought to be reduced by reinforcing the backfill by unattached horizontal bamboo strips. The expressions for intensity of lateral pressure, resultant earth pressure and its point of application were derived in terms of strength parameters of soil as well as the characteristics and distribution of reinforcement.

Sawicki and Lesniewska (1987) dealt with the theoretical analyses of the bearing capacity of reinforced soil retaining walls on the basis of a rigid-plastic model of reinforced soil and limit theorems. The results obtained could be useful in engineering calculations and design of geotextile reinforced retaining walls.

Bauer (1988) analysed earth reinforced structures, reinforced with plastic geogrids, using limit equilibrium and finite element methods. The analyses included surcharge effects and various soil materials. The results
were presented as normalized graphs which can be used as design guides to estimate the critical geometry of a reinforced earth structure for various reinforcing configurations.

Leshchinsky and Perry (1988) evolved a design procedure for geotextile-reinforced walls subjected to uniform surcharge loads. Design charts were included for the evaluation of internal stability of the wall, which were developed from a limit equilibrium analysis.

Bathurst and Simac (1993) described and compared the features of two computer programs GEOWALL and GRSWALL which were written by the first author to allow the geotechnical engineer to design a geosynthetic reinforced soil wall quickly. The authors conclude that GRSWALL allows the user more flexibility with respect to the number of reinforcement types and reinforcement layout. On the other hand, GEOWALL is more advanced as a design aid that will guide the user through the analysis and design of a geosynthetic reinforced soil wall.

Singh and Basudhar (1993) successfully demonstrated the application of a generalized approach to the estimation of the lower bound bearing capacity of reinforced soil retaining walls by using the finite element technique in conjunction with non-linear programming to isolate the optimal solution. The analysis was based on a rigid plastic model for reinforced soil, treating it as a macroscopically homogeneous anisotropic material. The results obtained were found to be in good agreement with the theoretical and experimental data reported in the literature.

Fantini and Roberti (1996) described the study, design and execution of a vegetated geogrid reinforced slope used to recover a degraded area due to the construction of a viaduct in a highway in Italy. Reinforced slopes of height varying from 3 m to 20 m were constructed using different bioengineering techniques, to obtain a well established and permanent vegetation cover.
The authors finally infer that a good use of the geosynthetics can solve extremely difficult problems with due respect to the environment.

Mannsbart and Kropik (1996) reported the construction and analysis of a temporary retaining wall, 2.1m high, in the course of the lifting and widening of a railroad track in Vienna. Due to the limited time and space available, the wall was built using non woven needle punched continuous filament geotextiles as reinforcing elements. In spite of the heavy traffic load, measurements showed that the deformation of the structure was negligible, proving that even low modulus nonwoven geotextiles can fulfill reinforcement functions, especially in low, temporary structures.

Motta (1996) utilised the limit equilibrium analysis applied to a plane failure surface for the evaluation of the active earth pressure on reinforced earth retaining walls under different loading conditions, such as seismic loading, pore water pressures into the fill, vertical and horizontal loads acting on the top at some distance. Pore water pressure effects on the earth pressure were taken into account by means of the pore pressure ratio \( \frac{u}{\gamma H} \). In the design procedure, the analysis also allowed to define the spacing or the number of reinforcement as well as their length according to the failure wedge predicted.

Soong and Koerner (1997) discussed a number of short and long term issues during and after a geosynthetic reinforced soil wall is constructed and its required connection strength.

Garg (1998) dealt with the design, construction and cost economics of a 11m high and 19.5m long random rubble stone masonry wall retaining reinforced earth fill. The cohesionless fill, available at the construction site, was reinforced by geogrids, which were not attached to the wall face. The design philosophy developed by the author for rigid reinforced earth wall was discussed in detail in the paper. It was also reported that the retaining wall with geogrid reinforced earth fill was constructed at 79% of the cost of the retaining wall with conventional earth fill.
Porbaha et al. (2000) applied a kinematic approach based on the framework of limit analysis for the stability analysis of model reinforced vertical and sloping walls with cohesive backfill that were brought to failure under self-weight in a geotechnical centrifuge. A rotational failure mechanism was used to compute critical heights of the unreinforced and reinforced models; and the constrained Simplex method was employed in the optimization scheme. The prototype equivalent heights predicted by the analyses were within the distress range; i.e., development of tension crack and collapse of the retaining walls occurred during centrifuge tests.

Hatami et al. (2001) investigated the structural response of reinforced-soil wall systems with more than one reinforcement type (non uniform reinforcement) using a numerical approach. The selected reinforcement types and mechanical properties represent actual polyester geogrid and woven wire mesh products. The model walls were mainly of wrapped-face type with different reinforcement lengths, arrangements, and stiffness values. Additional wall models with tiered and vertical gabion facings were included for comparison purposes. The numerical simulation of wall models was carried out using a finite difference-based program which included sequential construction of the wall and placement of reinforcement at uniform vertical spacing followed by a sloped surcharge.

Srbulov (2001) analyzed the stabilities of slopes and walls using a method based on limit equilibrium. The results of measurements of axial strains in geogrids of two reinforced steep slopes and two retaining walls were interpreted. However, the results obtained by the method remain only approximate due to necessity to introduce a number of simplifying assumptions.

Horvath (2003) discussed a geosynthetic based earth retention concept. This new concept allows controlled yielding within a retained soil mass by using a compressible inclusion composed of certain types of geofoam geosynthetic that can be used alone or combined with MSE. Two basic ways were outlined,
in which geofoam compressible inclusions are used with earth retaining structures – Reduced Earth Pressure and Zero Earth Pressure concepts. Typical applications of the same were also discussed.

Leshchinsky et al. (2004) developed a rational design methodology for multitiered MSE walls that accurately predicts wall performance. The study presented the results of parametric studies conducted in parallel using two independent types of analyses based on – limiting equilibrium and continuum mechanics. Parametric studies were carried out to assess the required tensile strength as a function of reinforcement length and stiffness, offset distance, the fill and foundation strength, water, surcharge, and number of tiers. It was concluded that limiting equilibrium analyses may be extended to the analysis of multitiered walls.

Bathurst et al. (2005) developed a new working stress method for the calculation of reinforcement loads in geosynthetic reinforced soil walls. As a precursor to this objective, back-analyses of a database of instrumented and monitored full-scale field and laboratory walls was used to demonstrate that the prevailing AASHTO Simplified Method used in North America results in excessively conservative estimates of the volume of reinforcement required to generate satisfactory long-term wall performance.

Mittal et al. (2006) analysed the case of a rigid wall with inclined back face retaining reinforced cohesive frictional backfill subjected to uniformly distributed surcharge load using limit equilibrium approach. The analysis considered the stability of an element of the failure wedge, which is assumed to develop in the reinforced earth mass adjoining the back face of wall. Non-dimensional charts were developed for computing the lateral earth pressure on wall and the height of its point of application above the base of wall. The theoretical findings were verified by model tests on a rigid wall retaining a dry cohesive-frictional soil reinforced by geogrid strips. Experimental results were in good agreement with the theoretical predictions.
A design example was also added in the paper to illustrate the design procedure.

### 2.3.3 Segmental retaining walls

Rimoldi et al. (1997) discussed the various aspects of design methodology, installation and construction practices of geogrid reinforced retaining walls. It was stated that before the design of a reinforced earth retaining wall, the properties of the reinforcement viz., friction, tensile strength, creep resistance, junction strength, chemical and biological resistance should be carefully evaluated. The walls should be checked for external and internal failure modes, facing failures and global stability. The design criteria include tensile overtension failure analysis, geogrid pullout failure analysis and local stability of segmental retaining wall units.

Koerner and Soong (2001) compared three design methods of geosynthetic reinforced segmental retaining walls to one another with respect to their details and idiosyncrasies. This was followed by a numeric example which illustrated that the modified Rankine method is the most conservative, the FHWA method is intermediate, and the NCMA method is the least conservative. A survey of the literature was included where it can be seen that there have been approximately 26 walls which suffered either excessive deformation or actual collapse. The survey described 12 serviceability problems and 14 wall failures. Of the total, 17 of the cases had low permeability backfill soils in the reinforced zone and 8 had uncontrolled or inadequate quality control in the construction of the walls.

### 2.4 NUMERICAL MODELLING

#### 2.4.1 Gravity walls

Bolton et al. (1989) studied the behaviour right up to the collapse of a retaining wall embedded in overconsolidated clay using FE analysis and the results were compared with those of centrifuge modelling. Six nodded triangles were used to model the soil and 8 nodded quadrilaterals to model the diaphragm.
wall. Slip elements of 0.1 mm thickness were used between the retaining wall and the soil to model the interface conditions which may permit relative slip.

Larkin and Williams (1994) developed a simplified computer model of lateral earth pressure and compared its efficiency with some large scale experimental work. The computer model included a linear elastic stress strain relationship with the values of the elastic constants adjusted to be consistent with deformations. The computed force – deformation relationship of the translating wall was compared with that recorded from an experimental set up with a 1m retaining wall backfilled with cohesionless soil. Comparisons between recorded and computed pressure distributions were made for several values of wall translation.

Matsuzwa and Hazarika (1996) carried out a numerical investigation to evaluate the effect of wall movement modes on static active earth pressure. The authors developed new interface elements having bilinear stress – displacement relations and introduced them between the backfill soil and the wall to simulate the frictional behaviour. To avoid the separation between the wall and the backfill soil, during the active movement of the wall, conventional linkage elements were idealized suitably. The active state was defined based on the progressive formation of a failure zone in the backfill. Empirical equations, containing wall movement made as a governing parameter, were derived for calculating the active earth pressure coefficient and the relative height of the resultant active thrust for various angles of internal friction of the backfill.

Filz et al. (1997) constructed massive concrete walls on rock foundations, as well as other nonmoving retaining walls, customarily designed for static earth pressures. In this paper, model test results and case history data were reviewed, the results of finite - element calculations were presented, and a simple design procedure was developed. It was shown that significant economies can result from consideration of vertical shear forces in design of nonmoving retaining walls.
Addenbrooke et al. (2000) used the results from 30 nonlinear finite element analyses of undrained deep excavation in stiff clay to support the use of a new displacement flexibility number in multi propped retaining wall design. The analyses addressed the effects of different initial stress regimes and various values of prop stiffness for the internal supports to the excavation.

2.4.2 Reinforced earth walls

Romstad et al. (1976 and 1978) represented the reinforced earth as a composite material with associated composite properties. The composite model was developed by consideration of a small unit of the material as a fundamental building block called the unit cell. The composite properties defining the stress-strain relationships were predicted by successively considering a number of simple composite stress-strain states and approximately determining the response of the unit cell.

Shen et al. (1976) described an analytical study using the FE program of an instrumented complex field prototype constructed in Southern California. The analytical results were then compared with the field performance data to illustrate the overall behaviour of the structure and the strengths and weakness of the analytical model. In addition, implications of the study relative to the existing design procedures were drawn and recommendations were made for simple reinforced earth structure design.

The analysis of the finite element results and field performance indicated that reinforced earth is a relatively rigid supporting unit in which, under normal conditions, the stress state within the wall is approximately \( k_n \) condition and the backfill just behind the wall approaches \( k_n \) condition. The boundary geometry of the backfill and foundation and compressibility of the foundation material affected the magnitude and direction of the strip forces. Even compressive forces developed in the strip under particular combinations of these parameters.
Naylor (1978) used a special element and formulated a slipping strip analytical model for earth retaining walls using unit cell concept. Soil was assumed to be linear elastic and was represented by six noded rectangular elements. An extra degree of freedom for representing the displacement of a point on the strip relative to the soil matrix in a direction parallel to the strip was given. A parametric study was carried out to investigate the effect of strip slip, fixity at the face, relative longitudinal stiffness of strips and soil as well as stiffness of the foundation.

Ogisako et al. (1988) performed finite element analysis on polymer grid reinforced soil retaining walls. The wall was modelled using the beam element, soil using quadrilateral elements and the polymer grid using truss element whose ends were connected by pin joint. The soil – reinforcement interface and the wall – soil interface were modelled using joint elements. Analyses were performed on various height of the wall and spacings and lengths of the polymer grid. The effect of these parameters on the reduction effect of the earth pressure acting on the wall and the wall deformation was discussed in detail.

Bauer and Halim (1989) reported a composite finite element study on reinforced soil walls constructed with cohesive backfill. Soil was modelled using Duncan and Chang's (1970) hyperbolic stress function. The base was assumed to be rigid. The results showed that the lateral movement of the wall face decreased with a decrease in the wall inclination and the maximum settlement occurred close to the wall face, at the upper part of the wall. The maximum stress in the reinforcing strip depended on the footing location. The direction of load inclination affected the lateral movement and settlement greatly. The introduction of a cohesive backfill was found to reduce the lateral displacement by 25% and settlement by 50% under vertical loads.

Chew and Schmertmann (1990) presented the results of study of the deformation behaviour of reinforced soil walls using a previously validated finite element code. The conventional design procedures for reinforced soil walls consider the overall stability and rupture and pullout capacities of the
reinforcement. But it does not take into account the wall deformations under working stress conditions. The authors attempted to bridge this gap by presenting the summary of a numerical study of the effect of reinforcement length, reinforcement layout and external loadings on the deformation of reinforced soil walls.

The FE code used was capable of modelling the construction sequence and compaction operations. The code could also consider the thrust from the unreinforced soil behind the reinforced soil zone. This modelling method was applied to a range of inextensible reinforcement layouts and external loadings, varying about a reference wall, to predict the end-of-construction wall response.

Karpurapu and Bathurst (1992) described two sets of numerical simulations which were carried out to model the controlled yielding concept. The simulations used finite element method together with a hyperbolic constitutive soil model.

The 1 m high walls were constructed using a continuous stack of 20 articulated platens constrained to move in the horizontal direction by a system of flexible springs and frictionless rollers. Eight-noded quadrilateral elements were used to model both the panels and the soil, and two-noded bar elements were used to simulate the springs. The panels and soil were separated by six-noded isoparametric interface elements. These elements were assumed to have negligible shear stiffness to simulate the teflon surface at the wall that was employed to reduce friction effects in the physical tests. The nodes at the bottom surface of the mesh below the soil were fully constrained. The incremental construction sequence in the physical tests was simulated by building the mesh in 20 rows of elements using several load steps per layer. The material behaviour of the soil was modelled using the hyperbolic constitutive model. Simulations were carried out to generate preliminary design charts for the selection of stiffness and thickness of compressible layers placed
against rigid walls retaining well graded sand backfills compacted to a range of densities.

Bergado et al. (1995) studied the behaviour of a reinforced embankment on soft Bangkok clay by plane strain finite element method. The analysis considered the selection of proper soil / reinforcement properties according to the relative displacement pattern of upper and lower interface elements. A full scale test reinforced embankment with a vertical face wall on Bangkok clay was analysed by this method. The numerical results were compared with the field data and it was concluded that the response of a reinforced embankment on soft ground is principally controlled by the interaction between the reinforced soil mass and the soft ground and the interaction between the grid reinforcement and the backfill soil. The permeability variation of the soft ground was also accounted in the finite element analysis.

Karpurapu and Bathurst (1995) used finite element models to simulate the behaviour of two carefully constructed and monitored large-scale geosynthetic reinforced soil retaining walls. The walls were constructed using a dense sand fill and layers of extensible polymeric (geosynthetic) reinforcement attached to two very different facing treatments. A modified form of hyperbolic constitutive model that includes a dilation parameter was adopted to model the behaviour of the granular soil. The results of analyses show that the finite element model, constitutive models and implementation reported in the study can accurately predict all important features of wall performance.

Ling et al. (1995) discussed the application of the finite element (FE) procedure for simulating the performance of geosynthetic reinforced soil retaining walls. Analyses were performed using a modified version of CANDE code, in which the material properties of the wall (backfill, foundation, geosynthetic and wall face) were expressed using non-linear elastic models. The analytic procedure was validated with the loading test results of a full-scale model comprising silty clay backfill soil and a permeable geotextile. A series of parametric studies was conducted to identify the effects of the
geosynthetic length and the stiffness of the facing and the geosynthetic on the performance of geosynthetic reinforced soil retaining walls. They concluded that the layout of geosynthetic layers affects greatly the performance according to the point of load application and increased wall facing and geosynthetic stiffness improve the performance by restraining the lateral deformations.

Bauer and Brau (1996) proved using back analysis technique that non-wovens can be used in earthworks as reinforcement when the material characteristics that describe the stress strain behaviour of the composite system consisting of soil and geotextile are known. The conventional calculations look at soil and geotextile separately and hence these methods do not suit the calculation of the effect of non-wovens in soil mass.

Ho and Rowe (1996) examined the effect of geometric parameters such as reinforcement length, number of layers of reinforcement, distribution of reinforcement and wall height on the forces developed in the reinforcement. It was shown that the forces developed are largely independent of reinforcement length for reinforcement to wall height ratios equal to or greater than 0.7. For truncated reinforcement schemes with a ratio of less than 0.7, the forces in the reinforcement increased as the length of the reinforcement decreased. The number of layers of reinforcement was not found to significantly affect the total force required for equilibrium provided the reinforcement stiffness density was the same. The analysis provided theoretical support for the common practice of using truncated reinforcement with equal vertical spacing and length equal to 70% (or greater) of the wall height.

Alfaro et al. (1997) obtained the patterns of deformation of the wall and the soft clay foundation beneath the reinforced soil mass based on the results of full scale field tests and FEA. The performance of two reinforced soil test wall - embankment systems constructed on soft clay foundation with different reinforcement types but having the same backfill were used in the investigation. One test facility used steel grid reinforcement and the other used polymer grid reinforcements. Parametric studies were carried out using FEA to examine the
effects of the stiffness of the reinforced soil system and the foundation on the overall deformation characteristics of reinforced soil wall. Results indicated that increasing the stiffness of the reinforced soil system led to lower lateral spreading of the clay foundation owing to more lateral confinement of the underlying soil.

Kobayashi and Porbaha (1997) presented the numerical modelling of the physical tests on scaled geotextile reinforced retaining walls constructed with cohesive backfill. The aim of the work was to predict the position of critical slip surface which is an important factor in the cost effective design of retaining systems. For this, the effects of maximum shear strain contours and plastic yield zones on the positions and traces of slip surfaces were investigated for unreinforced and reinforced vertical walls.

Rowe and Ho (1997) examined the effects of reinforcement stiffness density, reinforcement – soil friction angle, backfill friction angle and facing rigidity on the behaviour of continuous panel faced reinforced soil walls resting on a rigid foundation. It was shown that the interaction between the components of the reinforced system determines the stress distribution in the soil and the manner in which the required total resisting force is distributed in the reinforcement layers.

Helwany et al. (1999) validated a finite element program by comparing its analytical results with the results of a well instrumented large scale laboratory test conducted on a geosynthetic reinforced soil (GRS) retaining wall under well – controlled test conditions. The validated computer program was used to investigate the effects of backfill type on the behaviour of GRS retaining walls. It was shown that the type of backfill had the most profound effect on the behaviour of the GRS retaining wall. It was also shown that the stiffness of the geosynthetic reinforcement had a considerable effect on the behaviour of the GRS retaining wall when the backfill was of lower stiffness and shear strength.
Lee et al. (1999) studied the use of shredded tires as backfill in reinforced earth structures. Triaxial test results were used in the FE modelling (using commercial FE program ABAQUS) of wall and backfill, both unreinforced and reinforced with geosynthetics. Duncan and Chang's (1970) non linear hyperbolic model was used to model the tire shreds and rubber-sand. Both backfill and wall facing elements were modelled with 8 noded 2D solid elements. Linear elastic model was used to represent the wall elements. The geotextile used as reinforcement in the wall backfill was modelled using zero thickness interface elements following Coulomb’s friction mechanism.

The FEA were conducted under at-rest and active conditions. The results were compared with field tests and good estimates of deformations and stresses were obtained for at-rest condition, but showed overestimation for active condition. The analyses indicated that the performance of rubber sand, being both light weight and reasonably strong, compared well with that of a sandy gravel, as a backfill material.

Sreekantiah (2001) used a two dimensional numerical model, based on the finite element method, for investigating the deformation behaviour of a geogrid reinforced soil retaining wall. Relationships between lateral deformation and height of the wall and between vertical settlement and height of the wall were studied numerically and compared with the experimental results on model retaining walls. He concluded that the finite element analysis is capable of the deformation behaviour of a reinforced soil retaining wall with a reasonable degree of accuracy.

Sreekantiah and Sowmya (2001) investigated the behaviour of a geogrid reinforced soil retaining wall using a two dimensional numerical model based on the finite element method. Model retaining walls for different combinations of vertical spacing of reinforcement were investigated for various surcharge loads. Relationships between lateral deformation and height of wall and between vertical settlement and height of wall were studied. The results were verified with experimental data.
Tyagi and Mandal (2001) conducted parametric studies on geosynthetic reinforced retaining walls based on finite element analysis on four different geosynthetic reinforcements and eight different types of backfills to produce 100 combinations. Use of sandwich technique to optimize the cost and strength mobilization of two different backfills was adopted in actual site conditions after validation of FEM results. They concluded that the type of backfill has profound effect on the behaviour of a reinforced soil wall and the stiffness of reinforcement plays a considerable role for soil of lower stiffness and shear strength. The parametric charts developed by the authors will help the geotechnical engineer to choose the appropriate backfill and geosynthetic reinforcement for satisfying the prescribed requirements of maximum displacement, maximum axial strains in reinforcements and safety factors.

Park and Tan (2005) discussed the effects of the inclusion of short fibre in sandy silt (SM) soil on the performance of reinforced walls. The finite element method was used to examine the influence of the reinforced short fibre on reinforced walls. The vertical and horizontal earth pressure, displacement and settlement of the wall face were analyzed. These results were compared to the measured results from two full-scale tests. It was seen that use of short fibre reinforced soil increases the stability of the wall and decreases the earth pressures and displacements of the wall and this effect is more significant when short fibre soil is used in combination with geogrid.

Skinner and Rowe (2005) conducted numerical analysis of a hypothetical 6 m high geosynthetic reinforced soil wall supporting a bridge abutment and approach road constructed on a 10 m thick yielding clayey soil deposit. The soil retaining wall was examined under two-dimensional (plane strain) conditions consistent with normal design assumptions. The finite element mesh used 4335 eight noded isoparametric elements to model the soil, masonry and concrete, 288 linear bar elements (with no significant compressive or bending strength) to model the reinforcement and 1390 interface elements were used between the various materials. The initial geostatic stress conditions in
the foundation were based on the unit weight and effective coefficient of lateral earth pressure at rest ($K_0$) for the soil.

The results of the numerical analysis were compared to current design methodologies to examine the effect of the yielding soil foundation on the behaviour of the wall and abutment. The study included the examination of both the internal and external stability of the wall, and focused on methods of improving the external stability.

2.4.3 Segmental retaining walls

Arab et al. (1998) used the finite element method for analysing two full scale experiments on segmental walls (4.35m x 5m) loaded on the top and constructed with extensible reinforcement of non woven and woven geotextiles, using the FE code GOLIATH. Large deformation analyses were conducted simulating the construction process also. The wall was loaded with a concrete slab having surcharge 130kN/m. For the non woven wall, the predicted settlement was found to be more than the experimental values whereas it was lesser for the woven walls. Parametric studies were conducted varying the geometric and material parameters.

It was seen that the influence of reinforcement length is significant only till length equal to half the height of the wall after which nil effect was noticed. With increasing surcharge and stiffness, settlements also increased. In the absence of a facing or using a facing of low stiffness, caused higher deformations. The effect of position of load was also studied and it was seen that when the footing is placed outside the reinforcement space, a shear band is developed from the downstream side of the slab to the bottom of the facing. Also, there was no significant increase in bearing capacity when the reinforcement length exceeded 0.5 times the height. It was concluded that the reinforcements should extend beyond the potential failure surfaces.

Rowe and Skinner (2001) conducted numerical examination of the behaviour of an 8m high geosynthetic reinforced soil wall constructed on a
layered foundation stratum using FEM. The wall was constructed with 16 segmented concrete facing blocks, a sandy backfill and 11 layers of geogrid reinforcement 6m long. Five additional, one metre long layers of reinforcement were used between the 6m long layers within the upper 5m of the wall to improve the local stability of the facing blocks. The analysis examined the effect of uncertainty regarding the drained and undrained strength of the loam foundation material, its stiffness, the thickness of this soft layer and its position with respect to the bottom of the wall on the calculated behaviour and compared the calculated and observed behaviour from the full-scale test wall.

The field case was idealized as two dimensional and a plane-strain analysis was performed. The finite element mesh used 1697 eight noded isoparametric elements to model the soil, concrete and footing blocks and 1117 interface elements were used between the soil and other materials. The reinforcement was modelled using 417 linear bar elements. The concrete and footing block were treated as elastic materials. An elasto-plastic stress-strain model with Mohr-Coulomb failure criterion was adopted for the continuum elements used for the soil. The interface elements were modelled with a stiff spring in each of the shear and normal directions until slip occurred, at which point deformation could occur along the interface and the normal and shear stresses satisfied Mohr-Coulomb failure criterion. The construction analysis of the wall was conducted layer-by-layer.

Yoo (2004) presented the results of an investigation of a geosynthetic reinforced segmental retaining wall, which exhibited signs of distress and unexpected large lateral wall movements 6 years after wall completion. In an attempt to identify possible causes and to provide mitigation measures, a comprehensive investigation was carried out including wall profiling, stability analyses based on the current design approaches, and finite-element analyses.

In the finite element modelling, the wall facing and the backfill soil were discretised using four noded plane strain elements with 4x4 integration, while the reinforcements and the back-slope were modelled using two-noded truss
elements and three noded triangular plane strain elements respectively. The lateral boundary extends to a distance of three times the wall height (H) from the wall face. Considering the competent foundation condition, the wall was assumed to be located on a non-yielding foundation. The interface behaviour between the wall facing and the backfill soil was modelled using the "contact pair", a special type of interface.

In the analysis, the backfill soil was assumed to be an elasto-plastic material with Mohr-Coulomb failure criterion together with the non-associated flow rule. The wall facing block and the reinforcement were assumed to be linear elastic. In the finite-element modelling, the detailed construction sequence was carefully simulated by adding layers of soil and reinforcement at designated steps. Upon completion of the wall, lateral displacements similar in magnitudes to those measured by the wall profiling were then incrementally applied at the wall face in order to create stress-strain fields similar to those of the actual field walls.

Helwany et al. (2007) described the finite element analyses of two full-scale loading tests of GRS bridge abutments. The aim of the study was to study the complex behaviour of GRS structures in general, and the behaviour of GRS bridge abutments with modular block facing in particular. The study also investigated the performance of the GRS bridge abutments with the variation of backfill properties, reinforcement stiffness properties, and reinforcement vertical spacing.

A plane strain finite element model was developed for bridge abutment and analysed. The soil was simulated utilizing an extended two-invariant geologic cap model which is an elastoplastic model, and the geosynthetic reinforcement using an elastoplastic model with failure. The parameters required for this were deduced from the results of the uniaxial tension tests performed on geosynthetics.
Interface elements were used between the modular blocks and reinforcement, and between the blocks and backfill soil. The interface element used allows sliding with friction and separation. A gravity load was applied in the beginning of the analysis to establish the initial stresses within the backfill soil.

2.4.4 Gabion faced retaining walls

Helwany et al. (1996) proposed a numerical model incorporating a three-parameter dilatant nonlinear incrementally elastic soil model and used it to analyse three well-controlled full-scale geosynthetic reinforced soil (GRS) retaining wall tests. Three tests were conducted to examine the effects of facing rigidity and reinforcement length on the performance of the walls subject to central loading (far from the facing panel). It was then proved that the numerical model is capable of accurately simulating the behaviour of the three tests under service loads, in particular, their sensitivity to facing rigidity and reinforcement length. The validated numerical model was then used to conduct a comparative analysis on four GRS retaining walls subject to front loading (adjacent to the facing panel). The GRS retaining walls comprised four different types of facings types A, B, C and D having different degrees of rigidity. Type A facing was of a continuous cast-in-place concrete facing panel and type B comprised of discrete precast concrete facing units. Both types had gabions immediately behind the concrete facings to facilitate construction. Type C utilised gabions as the sole facing unit and type D was a wrapped around facing. Reinforcement used in the soil was geogrid attached to the facing. The results of the comparative analysis are considered useful as a guide for selecting the proper facing rigidity of GRS retaining walls subject to front loading.

Sand backfill, foundation soil, concrete facings and gabions were modelled using quadrilateral elements, geosynthetic reinforcement using bar elements and loading plate by beam elements. The gabion was considered to behave as a single block and hence was modelled using a single quadrilateral element. From the plane strain compression tests conducted on gabion stacks,
it was clear that the gabions exhibit a linear but inelastic behaviour. However, in the analyses, the gabions were assumed to behave in a linear elastic manner, since the authors are of the opinion that unloading is unlikely to occur in the full-scale tests of the walls.

Bergado et al. (2000 A) simulated the consolidation behaviour of foundation soil below a full scale test embankment on soft Bangkok clay using hexagonal wire mesh as reinforcement. The embankment was underlain by a layered soft foundation. The elastic, perfectly plastic, Mohr–Coulomb model was adopted to represent the fill embankment and weathered clay layer of the foundation soil. The soft soil model was used for predicting the behaviour of both the soft and medium clay layers. The elastic model was used for the stiff clay layer. Bar elements with linear tension strain were used to model the hexagonal wire mesh reinforcement. The elastic, perfectly plastic model was used to simulate the constitutive relationship of the soil–hexagonal wire mesh interface using PLAXIS.

The gabion boxes were filled with crushed rock. The elastic, perfectly plastic, Mohr–Coulomb model was used to simulate the crushed rock inside the gabions. The hexagonal wire mesh in the gabion system was modelled using beam elements. The embankment was modelled as a plane strain problem. The effects of construction sequences were also considered.

The numerical simulation adopted gave a reasonable representation of the overall behaviour of the reinforced soil wall embankment system on a soft foundation through good agreement between the field measurements and simulated values. The important considerations for simulating the behaviour of the reinforced wall embankment were the method of applying the embankment loading during the construction process, the variation of soil permeability during the construction process, and the selection of the appropriate model and properties at the interface between the soil and the reinforcement.
2.5 ECONOMIC STUDIES

Lee et al. (1983) used categories of gravity walls and crib / bin walls and compared them to MSE walls with steel reinforcement. The walls were furthermore subdivided into high (\( H \geq 9.0 \) m), medium \((4.5 < H < 9.0 \text{ m})\) and low \((H \leq 4.5 \text{ m})\) height categories. It was seen that MSE walls with metallic reinforcement are the least expensive wall of the types surveyed at all heights.

Rao and Singh (1988) presented typical results relating to design of reinforced earth walls for varying backfill quality and emphasised the need for full scale field trials in India, for evaluating the techniques as alternative to conventional retaining walls. The global literature was reviewed and proved that the introduction of reinforced earth lowered the cost of structures. It is commented that savings over conventional retaining structures varied between 20% to 50% with an overall average savings in the walls and abutments of 32%.

Durukan and Tezcan (1992) presented a systematic method of determining the possible cost of a reinforced soil-retaining wall and developed an estimate of cost breakdown on the basis of height and length of the retaining wall. Relative economy of reinforced soil retaining walls in comparison with conventional and other types of retaining walls was also determined. For the purpose of illustration, several case studies of cost analyses and a numerical example, were also included.

Koerner et al. (1998) conducted a survey which included four wall categories like gravity walls, crib / bin walls, MSE walls with metal reinforcement and MSE walls with geosynthetic reinforcement. Gravity walls were seen to be the most expensive, with crib/bin walls and MSE (metal) walls significantly less expensive. But the crib/bin walls are rarely above 7m in height. It was also obvious that MSE (geosynthetic) walls are the least expensive of all wall categories and over all wall heights. However, convergence seems to occur within the two different MSE types (metal and geosynthetics) in the high wall height category.
This survey also generated statistical data in providing a mean value, standard deviation and variance. The standard deviation in data is highest with gravity walls, intermediate with crib/bin and MSE (metal) walls, and the least with MSE (geosynthetic) walls. Variance values, however, are similar in all wall categories.

Koerner and Soong (2001) compared the results of survey conducted by various researchers and compared with the results obtained from the survey conducted Koerner et al. (1998). The results showed that MSE walls with geosynthetic reinforcement are the cheapest of all the types considered in the study and MSE walls with metallic reinforcement comes in the second position. The RCC walls were found to be the costliest walls.

Basudhar et al. (2008) dealt with the optimum cost (objective function) design of geosynthetic reinforced earth retaining walls subjected to static and dynamic loading. The design restrictions were imposed as design constraints in the analysis. Choice of the initial designed length and strength of the reinforcement, which are the elements of the design vectors were made in such a way that it forms an initial feasible design vector. The constraints and the objective function being nonlinear in nature, the Sequential Unconstrained Minimization Technique was used in conjunction with conjugate direction and quadratic fit methods for multidimensional and unidirectional minimization to arrive at the optimal (minimum) cost of the reinforced earth wall. Optimal cost tables were presented for different combinations of the loading and the developed procedure was validated by taking up an example problem. It was found from the typical example problem that savings of the order of 7–8% can be made over the conventional design of mechanically stabilised earth (MSE) walls with the aid of design charts presented in the paper.
2.6 NEED FOR THE PRESENT STUDY

After a pervasive literature survey conducted on retaining walls it is concluded that research works on gabion faced retaining walls or even segmental retaining walls (both fall under semi rigid walls category) are very much limited in number. But as mentioned in Chapter 1, the construction of these walls is gaining fast momentum all over the world without understanding the exact behaviour of these walls, as evident from the literature survey. Table 2.1 gives a numerical summary of the literature survey conducted as a part of this thesis work. It has also been eventually found from the literature studies that only a numerical tool like the finite element method can yield a complete picture of the behaviour of the retaining wall system and its components.

<table>
<thead>
<tr>
<th>No. of literature collected</th>
<th>Gravity walls</th>
<th>MSE walls</th>
<th>Segmental retaining walls</th>
<th>Gabion faced walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental studies</td>
<td>11</td>
<td>22</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Analytical studies</td>
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<td>30</td>
<td>2</td>
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</tr>
<tr>
<td>Numerical studies</td>
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<td>31</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>34</strong></td>
<td><strong>83</strong></td>
<td><strong>8</strong></td>
<td><strong>5</strong></td>
</tr>
</tbody>
</table>

Under these circumstances, a two dimensional finite element study is attempted in this work paying individual attention to soil, facing, reinforcement and the interfaces between soil and reinforcement as well as between soil and facing, considering the soil and interface as non linear, to monitor the behaviour of gabion faced retaining wall systems.