Analysis of circular geogrid-reinforced soil-steel bridge.

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Analysis of Circular Geogrid-Reinforced Soil-Steel Bridge

by

André John Kendall Bom

A Thesis
Submitted to the Faculty of Graduate Studies and Research through
The Department of Civil and Environmental Engineering
In Partial Fulfillment of the Requirements for the
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Windsor, Ontario, Canada

2003
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André John Kendall Bom
Abstract

A finite element analysis, by means of ABAQUS, has been undertaken to study the effects of inserting geogrid layers in the soil cover of a circular soil-steel bridge with a diameter of 7.6 m. Two loads are applied to the soil-steel bridge: gravity, as well as a truck load defined by the Canadian Highway Bridge Design Code design truck. Because experimentation and design of geogrids involve the application of a strip load, it is necessary to apply a strip load at the soil surface. Therefore, the rear two axles of the truck, separated by 1.2 m, are the largest loads of the truck and are converted into a strip load in order to model the strip load utilized for study of geogrids.

The study analyzed different depths of cover above the flexible pipe, with unreinforced and reinforced soil cover. Geogrids are placed in soil in order to increase the bearing capacity of the soil. It is expected that the geogrid will increase the load carrying capacity of the soil-steel bridge. Thus, various geogrid layers are placed in the specified depths of cover over the flexible pipe. Two types of strip loading, concentric and eccentric, will be applied, and each was applied until failure of the soil occurs. Concentric load is defined as the strip load applied at the soil surface where the centre of the strip load is directly above the crown of the pipe. Eccentric load is defined by the strip load applied at the soil surface offset a horizontal distance from the crown of the pipe.

Soil behaviour due to the failure load, defined by vertical and horizontal displacements and shear stresses of the soil, are presented for the two types of load cases for the unreinforced and reinforced soil cover above the circular soil-steel bridge. Also, factors of safety are presented, defined by the failure strip load divided by the applied truck strip load from the design code.
To

God my heavenly Father
Jesus Christ
and the
Holy Spirit—
I can do all things through Christ who strengthens me.

Also dedicated to

Mrs. Gertrude Car
April 12, 1930 to August 19, 2003
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List of Variables

\( \phi \)  angle of friction of soil
\( \gamma \)  unit weight of soil
\( \mu \)  Poisson’s ratio
\( \sigma \)  normal compressive stress in the soil
\( \sigma_1, \sigma_3 \)  minimum and maximum compressive normal stress
\( \tau, \tau_{xy} \)  shear stress in the soil
\( B \)  width of rigid strip load, 1.2 m
\( BCR_u \)  ultimate bearing capacity ratio (at failure of soil)
\( c \)  cohesion of soil
\( d \)  distance between pipe nodes for determination of spring stiffness
\( d \)  depth of first layer of geogrid, 0.4 m
\( E' \)  modulus of soil reaction
\( FS \)  factor of safety
\( h \)  effective thickness of pipe wall
\( \Delta H \)  vertical spacing between each geogrid layer, 0.4 m
\( I_s \)  second moment of cross-sectional area of corrugated pipe wall
\( k_n \)  coefficient of normal reaction (spring stiffness)
\( k_s \)  coefficient of tangential reaction (spring stiffness)
\( L \)  length of rigid strip load
\( 2 L_0 \)  total horizontal length to which geogrid is placed in the soil
\( N \)  number of geogrid layers placed in the soil
\( N_{cr} \)  number of geogrid layers to maximize BCR
\( \Delta p \)  pressure due to gravity
\( q_a \)  applied rigid strip truck load, 116 kN/m²
\( q_r \)  rigid strip failure load (force/area) of reinforced soil
\( q_u \)  rigid strip failure load (force/area) of unreinforced soil
\( q_{ult} \)  ultimate failure load on the soil
\( S \)  span of the pipe, 7.6 m
\( u \)  depth of the last geogrid layer
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CHAPTER 1
INTRODUCTION

1.1. General

Intense discussion arose in the late nineteen seventies and early nineteen eighties as to the efficiency of replacing short-span concrete bridges with soil-steel bridges, with spans that exceed 7 metres. Many of the concrete bridges, constructed only a decade or two prior, were losing serviceability too soon. The public and private sector worked together to determine a practical and economical solution to the decaying infrastructure that traversed waterways.

Soil-steel bridges are essentially corrugated steel pipe surrounded and covered with well-compacted soil for optimum support. The soil-steel bridge became the most feasible solution to decaying short-span concrete bridges for numerous reasons. Rapid construction and greater economy are two very important reasons. The Corrugated Steel Pipe Institute [9] list many impressive features of corrugated steel pipe. Due to the interaction with the surrounding backfill soil, a great deal of support is given by the granular material that surrounds the pipe, otherwise known as construction fill, allowing for a great amount of loading. The pipe walls do not have the load-carrying capacity as is the situation for concrete (rigid) pipes, yet the flexible pipe transfers the load to the surrounding construction fill, which stabilizes the structure. Construction fill is relatively inexpensive, provided it does not have to be transported a great distance, and relatively simple to place and compact. When properly compacted, the soil locks together, and supports great compressive stresses. It is a very practical method of attempting to replace deteriorating concrete bridges with large-diameter corrugated steel pipe.

Corrugated plates fabricated by cold forming, galvanized, and fastened together by bolts, make up the pipe wall. The width of each corrugation is usually 152 mm, with depths of corrugation ranging 37 to 60 mm. Once the bedding is shaped and compacted appropriately for the invert of the pipe, the curved plates are assembled. Construction fill is then placed and compacted in 150 mm thick layers from the bedding up to the crown.

Pipe arches, horizontal and vertical elliptical pipes, and circular conduits are the main types of soil-steel bridges. A pipe arch is constructed of corrugated steel plates to
form an arch, placed on concrete footings; they are complex both in construction and analysis. Horizontal and vertical elliptical pipes are simply elliptical pipes constructed of corrugated steel plates. Finally, circular pipes are generally the most widely used. For the purposes of the analysis of shallow cover with reinforcement with geogrids, the large-diameter flexible pipe will be circular. All types of soil–steel bridges constructed are depicted in Fig. 1.1.1.

Figure 1.1.1 Soil-steel bridges [7]
Most circular corrugated steel conduits, both now and in the past, are constructed with smaller diameters. Before the option of computer analysis became available, culverts were designed by empirical knowledge, based on numerous assumptions. But as diameters increased to sizes well above 3 metres, proper design methodology became more critical. Computer analysis became an essential part of the design of large-diameter flexible pipes. Analysis led to the conclusion that flexible pipes of diameters larger than 6 to 7 metres were safe and an acceptable construction method to replace short-span bridges.

However, over the last decade or two, many of the large-span soil-steel bridges have failed. Failure usually occurred in the surrounding soil and most commonly, in the soil above the pipe. The authors of Soil-Steel Bridges [2] emphasize that “soil-steel bridges should be constructed under strict and competent supervision, and with well-established, sound construction”. This is due to the fact that “any deficiencies in the construction process can lead to catastrophic consequences”. These points highlight the need for a knowledge of the mechanisms of failure of the soil-steel structure, not only in the field, but in terms of computer analysis as well. Since failure usually occurs in the soil above the pipe, the likelihood of failure is heightened when the depth of cover is shallow.

In order to minimize the potential for failure in the soil above the pipe, soil-steel bridge design codes include a minimum depth of cover. However, for large spans, as in the case for this analysis (7.6 m), the minimum depth of cover is quite large. The significant soil cover may decrease the economical advantage of the conduit over the short-span concrete bridge. Another problem related to a minimum depth of cover, is related to the geometry of the system; the total design depth, including bedding, pipe span and cover may exceed that which is available or required. The most likely option to date at lowering the code-specified minimum depth of cover is seen in reinforcing the soil above the large-diameter flexible pipe.

Geosynthetics are man-made materials for the purpose of enhancing the qualities of soil. Geotextiles, geomembranes, and geogrids are the three principal categories of geosynthetics [22]. Geotextiles are textile fabric providing filtration, separation or reinforcement of soil. Geomembranes are impermeable sheets used in such places as
landfills to deter vertical movement of fluids. The most significant purpose of geogrids is soil reinforcement. Geogrids now replace the traditional method of reinforcing soil with metal strips. Made of high-strength plastic, geogrids are formed in gridlike configuration. Insertion of geogrids serves two purposes. First, geogrid increases the bearing capacity of the soil investigated, translating into an increase of factor of safety. Second, it is expected that placement of geogrid in the soil layer when subjected to repetitive loading results in the development of composite layers that prevent development of wedge failure in the soil. Wedge failure in the shallow-cover soil is associated with eccentric loading generated by the standard design vehicle recommended by the Canadian Highway Bridge Design Code [7].

1.2. Objective

The research at hand is motivated by the need to strengthen the shallow cover above a circular large-diameter flexible pipe subjected to various types of load conditions. The research refers to previous investigations of similar type that have been conducted on soil-steel bridges. The novelty of this investigation is embodied in the placement of geogrids in the soil cover above the circular soil-steel structure.

The first stage of the research is focused on the determination of the initial data suitable to numerical investigations by means of the finite element method (FEM). It requires development of the effective mesh and soil-steel bridge constitutive parameters. The objective of this stage of research is to determine the effect of various depths of soil cover and soil constitutive parameters on the bearing capacity of the soil-steel bridge interaction system that is presented in terms of the increased factor of safety of the system.

The next stage is connected with the investigation of the effect of placing layers of geogrid in the soil cover on the overall bearing capacity of the large-diameter flexible pipe. The challenge that is met with application of geogrid to reinforce the soil above the pipe system is connected with development of a suitable numerical model in which the interaction between the geogrid and soil material results in the generation of a composite material that is developed around the geogrid when subjected to repetitive loading. The basis for this stage of investigation is provided by the experimental research conducted
on geogrid-reinforced soil under a rigid strip load, or in other words, a shallow foundation. The results of this stage are employed in modeling of the reinforcement of the soil cover over the soil-steel structure when subjected to concentric and eccentric loading. This stage allows for the determination of the increase of bearing capacity and consequently the factor of safety of the soil-steel system that is reinforced by various layers of geogrid.
CHAPTER 2
LITERATURE REVIEW

Experimentation and analysis regarding large-diameter flexible pipes, as well as soil reinforcement, over the last thirty years is abundant. Two terms require definition at this point. First, large-diameter in terms of a pipe is any pipe that has a span greater than 3 m. The second term is flexible; any pipe that can withstand without cracking a deflection at the crown greater than 2 % of the diameter is considered flexible, as opposed to a rigid pipe which would crack at vertical deflection of 2 %.

2.1. Finite Element Analysis

To begin with, a brief explanation of the tool used to analyze the problem of flexible pipes in reinforced soil is required. The finite element method (FEM) is a “method for numerical solution of field problems”, where a “field problem is described by differential equations or by an integral expression” [8]. The geometry of the field problem is divided into small pieces, elements, which are connected by nodes. The entire assembly of elements and nodes is called a mesh, and boundary conditions are applied to the mesh to model the real-world situation. In other words, the periphery of the geometry is restrained in any of the three directions to simulate a boundary. The mesh is designed and analyzed by means of a finite element package, in this case, ABAQUS, to simulate the addition of forces or displacements to the structure.

The majority of studies use plane strain conditions in the analysis of pipes and soil reinforcement. Plane strain conditions are applied to three-dimensional structures in the situation where the length of the structure is very long in comparison to the other two dimensions. More specifically, Cook [8] states “plane strain exists when strain normal to the analysis plane is constrained to be zero”. Examples of plane strain in geotechnical engineering are tunnels, dams, retaining walls and foundations. The analysis of plane strain structures is simplified in terms of computation as compared to the same structure modeled in three dimensions.
2.2. Soil-Steel Structures

In a push to replace concrete bridges with soil-steel structures in the late nineteen seventies and early nineteen eighties, the necessity of proper design and analysis became evident. As computing technology greatly increased in that time period, study of soil-steel bridges was experimentation by means of full- or reduced-scale models as well as by computer analysis. To this day, due to the numerous failures that have occurred, soil-steel bridges are still being studied in order to understand failure modes and to reduce the potential for failure.

One of the earlier studies of a bridge-replacement structure is by McVay and Selig [25]. The Pennsylvania Department of Transportation replaced a concrete bridge with a pipe arch and instrumentation was installed during construction to measure various displacements of the structure. Field inspection determined the soil properties surrounding the pipe arch. In addition, the system was assessed by means of finite-element computer analysis in plane strain. The analysis took into account the backfill placement—without compaction—up to the final surface elevation. Soil was assumed to be linearly elastic, and the modulus of elasticity of soil varied with depth, or vertical stress. For the pipe, in order to model the seams where the weakest part of the wall is located, “the cross-sectional area of the beam element was artificially lowered for the all the beam elements”. From the comparison of experimental results with results of computer analysis many conclusions were determined. A few of the conclusions are:

- the correct soil parameter values are of critical importance;
- the soil should include a soil failure criterion;
- “the finite-element models might be improved by better representing seam slip, soil-culvert interface slip, interface tension release, wall yielding, and wall buckling”.

At the same time as McVay and Selig published their study of the pipe arch in Pennsylvania, Hafez completed a PhD dissertation on soil-steel structures under shallow cover [17]. He studied experimentally and analytically the effects of dead (soil) and live (truck axle) loads on the soil-steel structure in plane strain. The objectives of the research
were to introduce improvements in FEA of large-diameter flexible pipes, and to compare the FEA results with a reduced-scale model. Two of the improvements introduced are improved representation of the interaction between the pipe wall and the soil, as well as the inclusion of a soil failure criterion. Employing ABAQUS, Hafez modeled the interaction between the soil and the pipe wall with interface elements, defined by springs. The soil interacted with the pipe wall through two springs, one in the normal direction, and the second, tangentially. Each spring had a very large stiffness, similar to the pipe and soil connected rigidly together. The second major improvement is seen in the inclusion of the Mohr-Coulomb failure criterion for soil. Once the soil gravity dead load is applied to the soil-steel structure, the live load is applied through the application of a single concentrated force to represent a truck axle load until soil failure occurs [18]. In the reduced-scale model, as well as in plane strain analysis, the live load was applied as a line load at the soil surface. Safety factors were calculated by dividing the failure load, $P_F$, by the allowable truck axle load, $P_a$. Both concentric and eccentric loading of the single truck axle load was analyzed. Several of the conclusions drawn by Hafez from the comparison of experiments and computer analysis are:

- with a number of “over-simplifying idealizations”, FEM properly models the soil-steel structure;
- the soil should be modeled by means of an eight-noded continuum element;
- Mohr-Coulomb failure criterion suitably represents the failure in the soil;
- as depth of cover increases, the safety factor also increases;
- eccentric loading “leads to a reduction in the value of soil failure load, compared to that of concentric loading, up to a limit”.

Hafez makes a number of recommendations, but most notably is the recommendation to further study of reinforced soil in the construction of large-diameter flexible pipes.

While Hafez concentrated his research on soil failure above the pipe, Ghobrial and Abdel-Sayed considered failure in the conduit wall under different configurations [16]. One of these configurations includes the coefficient of soil reaction. In general, the coefficient of soil reaction is used in the discussion of interaction between soil and the
steel pipe; this characteristic of soil is discussed in greater detail in Chapter 3. Identical to the analysis of Hafez, the authors included springs to model the interaction between the two materials employing the coefficient of soil reaction to calculate the normal and tangential spring stiffnesses. While Hafez used a constant value for spring stiffness around the pipe, Ghourial and Abdel-Sayed state that the normal coefficient, $k_n$, varies around the perimeter of the pipe by

$$k_n = \gamma \beta C_d C_\theta (H/D)^{1/2}$$

(2.2.1)

in which $C_d$ and $C_\theta$ are factors defined by $C_d = 4.25 - (0.75D/100)$ and $C_\theta = 0.25 [1 + 5.4 (\theta/\pi)]$, and $\beta = 1$ for dense and compacted granular fill, $H$ is the height of cover above the location of the spring (in), $D$ is the span of the pipe (in), $\gamma$ is the unit weight of the soil (lb/in$^3$), and $\theta$ is the angular coordinate in radians of the spring. A live load is applied at the soil surface by means of two concentrated forces representing two truck axles, assuming that no failure takes place in the soil. The analysis was carried out for conduits with different depths of cover as well as varying span/height ratios.

Sharma and Hardcastle [34] utilized FEA to study the stability of a steel arch that was constructed in Northern Idaho. The steel structure was constructed with ribs for reinforcement, where ribs are metal stiffeners fastened along the length of the culvert, on the exterior perimeter of the pipe at a specified design distance between each rib. The analysis took into account the construction sequence, as recommended by McVay and Selig. The study reviewed three different types of soil compaction, poor, average and good, each defined in the article. In addition to the construction sequence, a live load representing a truck axle was included at the soil surface, above two depths of cover, 0.3 m and 0.6 m. Factors of safety were determined, taking into account the thrust forces in the conduit wall. Among the many conclusions, the most applicable is the importance of the correct soil properties in the finite element analysis.

The most common method to analyze large-diameter flexible pipes has always been, and still is to this day, in plane strain. However, Girges [14] carried out numerical analysis in plane strain and in three-dimensions for comparison purposes, by means of
ABAQUS. He compared the load from a truck axle applied in two dimensions to loads in three dimensions. Specifically, Girges studied the thrust and bending moment around the conduit walls. The soil was modeled using an “eight-node linear displacement solid soil element with three active degrees of freedom” [15]. Between the soil and the pipe, in both two and three-dimensional analysis, a perfect bond was assumed. In other words, springs were not used to represent the interaction between the two materials; springs are used in the analysis to model the sliding that occurs between the soil and the steel pipe. Depth of cover was kept constant during all analysis at 2 m. The most important conclusion related to the present analysis is the statement regarding the high state of stress located directly below the load and above the pipe crown. Girges recommended that studies are needed on methods to increase the load-carrying capacity of the soil-steel structure.

Fernando and Carter [13] further developed the discussion of the influence of loading on the pipe wall for buried pipes. Varying the depths of cover, pipe diameters, effective pipe thickness, and also the ratios of pipe elastic modulus to soil elastic modulus, the authors by means of FEA studied the effects of applying a rigid strip load of varying width directly above the centreline of the pipe at the soil surface. Two simplifying assumptions were required for such a broad analysis. First, the soil was assumed to be linear elastic without a specified failure criterion. Secondly, the assumption was made that the soil and pipe are completely bonded together. Fernando and Carter specify how to best represent the pipe wall using beam elements. The effective thickness of the pipe is calculated “by either assuming the same area as the actual pipe (to model correct axial stiffness) or the same second moment of area (to model the correct bending stiffness), but not both. Usually it is the correct in-plane bending stiffness that is chosen.” The results of the analysis are presented using the thrust and moment in the pipe wall. It was concluded that the effective pipe wall thickness has a significant impact on the thrust and bending moment of the pipe. Also, the ratio of the pipe elastic modulus to the soil elastic modulus did not have as important influence on the results as did the effective thickness.
Finally, and most recently, Abdel-Sayed and Salib [2] analyzed using ABAQUS soil-steel bridges in unreinforced soil under shallow depths of cover. The objective of the analysis was to determine the factors of safety for larger spans than previously studied, with larger depths of corrugation, under various depths of cover. Commonly, the corrugation depth is 51 mm, however, the study was aimed at corrugations of up to 140 mm, greatly increasing the rigidity of the pipe. Two types of loading were analyzed for the larger span soil-steel structures, single axle load as well as two axles separated by 1.2 m. Load was applied both concentrically and eccentrically. Safety factors were determined for all types of loading, the same safety factors as used by Hafez: the failure load from analysis divided by the code-specified design axle load. The failure criterion of the soil is Mohr-Coulomb. Some of the pertinent conclusions are:

- depth of corrugation plays an important role when the conduit span is very large;
- live load is dispersed through the soil and has negligible effect on the pipe when depth to span ratio is 0.3 or greater;
- under eccentric loading, depth of corrugation is more important than for concentric loading;
- the minimum depth of cover specified in the soil-steel bridge design codes is overestimated for large spans, over 9.15 m.

2.3. Soil Reinforcement

Vidal [36] became a great pioneer in the research of reinforced soil due to his study of reinforcing the soil behind retaining walls with metal strips. Presently, the most common material to reinforce soil is geogrid, as discussed in the introduction. The factors contributing to the success of both reinforcing materials are the increased tensile resistance of the soil and the shear resistance from friction between the soil and the reinforcement. The following section outlines some of the studies, experimental and numerical, relating to soil reinforcement.

Binquet and Lee [4] carried out some 65 model strip footing tests to compare the increase of strength of reinforced soil with flat metal strips. The objective of the research
was to expand the knowledge of reinforced earth by means of a small-scale model, and to quantify the improvements of the soil's load-carrying capacity, otherwise known as bearing capacity. The bearing capacity ratio (BCR) was introduced for purposes of comparing the effect of different placements of the metal strips, where BCR is the reinforced soil bearing pressure divided by the unreinforced bearing pressure. The bearing pressure was applied through a surface load 75 mm wide. The soil, properly compacted, contained several layers of aluminum foil, which modeled flat metallic strips. Test parameters varied, such as the depth of the first layer, the number of layers, N, vertical spacing between geogrid layers and different loading pressures. The authors drew many conclusions from the tests. First, "a minimum critical number of layers of reinforcing are required to obtain significant load settlement and bearing capacity improvement". Second, the bearing capacity generally increases for layers of reinforcement of at least six to eight; past this number, no significant increase is noticed. Third, the authors observed that pullout of the reinforcement occurred when N < 2 or 3, and breaking of reinforcement occurred when N > 4. Breaking of ties was observed in the upper layers of reinforcement.

One of the earliest FEA analyses of soil reinforcement is evident in the research of Burd and Houlbys [6]. Burd and Houlbysy studied plane strain load-carrying capacity of unpaved roads over weak subgrades with reinforcement. They applied the membrane element, a one-dimensional structural element, described in the paper. The membrane element can carry axial loading only. In the analysis, the assumption was made that no slipping could occur between the soil and the reinforcement. Excellent interlock is created between soil and geogrid because of repetitive loading; therefore, the assumption of no-slip is justified. Burd and Houlbysy state in their conclusion that the finite element formulation, with the use of a one-dimensional element, accurately models reinforced earth.

In tests carried out on six different geogrids in 1992, the U.S. Army Corps of Engineers concluded that the Tensar BX1100 geogrids performed better than any other geogrids on the market [23]. Because of these results, the geogrid used to reinforce the
soil above the large-diameter flexible pipe in the present analysis will be the Tensar BX1100. More detailed explanation of this particular geogrid and modeling in FEA are explained in a later section.

Omar et al. [30] tested extensively different parameters related to reinforcement placement, similar to the research of Binquet and Lee, except that Tensar geogrid was used instead of aluminum foil. Testing comparisons made use of the bearing capacity ratio introduced by Binquet and Lee to compare the increase in strength of soil due to the reinforcement and varying the placement. Bearing capacity ratios for rigid strip loads and square rigid loading were determined. Objectives of the experiments included:

- the determination of failure loads;
- the determination of the critical ratio of d/B, where d is the depth of the deepest layer of geogrid, and B is the width of the load;
- the determination of the critical ratio of b/B, where b is the total horizontal length of the geogrid beneath the strip load; and
- to determine the BCR for varying number of geogrid layers, N.

Many conclusions were made, and several important figures were obtained, crucial to the design of geogrid. In order to avoid repetition, the conclusions of the study of Omar et al. are not stated here because they are required and included in the section relating to representing the geogrid in FEA.

The use of geogrid in road construction is especially evident in the last decade, yet not as widely as one expect. Perkins [32] wrote a report for the State of Montana Department of Transportation regarding the use of reinforcement under flexible pavements. The study carried out was both experimental and numerical, using ABAQUS. He analyzed a two and three-dimensional section with the following layers: asphalt, reinforcement, base aggregate and subgrade. In addition to the reinforced section, an unreinforced section was also analyzed. The purpose of experimentation and FEA was to determine the effects of repetition of wheel loading, and compare the reinforced to unreinforced soil. The study was quite extensive, and many conclusions were made. Clearly, the most important is the great influence the geogrid has at
increasing the lifespan of the flexible pavement, because the displacements of the roadway are smaller, and serviceability is unaffected.

Examining the effects of the rigidity of the geogrid, among other characteristics, is the objective of the study of Peng et al. [31]. Both experimentation and FEA were carried out in plane strain on seven different geogrids. Because the geogrids are modeled in plane strain, a planar reinforcement replaced the geogrid, similar to the membrane element described previously by Burd. The planar reinforcement had the same properties as the original grid.

Finally, and most recently, Michalowski and Shi [27] researched experimentally and analytically the deformation patterns of geogrid-reinforced sand. The study is more qualitative in terms of pictures of deformation than quantitative. All pictures are presented in plane strain, by means of plexi-glass. The research is an interesting investigation of the displacements occurring in both loose and dense sand, reinforced and unreinforced, when a strip load is applied at the surface. A single layer of reinforcement was placed at two different depths, 0.4 B and 0.8 B, where B is the width of the strip load. One of the more important conclusions of the study is the fact that horizontal displacements of soil above the geogrid are small compared to displacements throughout the rest of the system. This is due to the presence of the reinforcement.

2.4. Soil Reinforcement in Soil-Steel Structure

Research regarding the reinforcement of soil in soil-steel structures is minimal. Because soil-steel structures are an economical alternative to short-span concrete bridges, soil reinforcement above large-diameter flexible pipes should gain more attention in the future. A few of the recent studies are presented.

Noting the recommendations of many studies to increase the load-carrying capacity of the soil-steel structure, Mohammed and Kennedy [28] analyzed two methods to increase the load-carrying capacity. The first approach, transverse stiffeners, strengthens the conduit wall. Transverse stiffeners are curved I-beams equally spaced
along the length of the pipe, fastened to the top half of the pipe. The second method is by means of flat bars placed in the soil above and adjacent to the culvert; those bars adjacent to the culvert are attached to it. The purpose of the flat bars is to divert “a greater portion of the vehicular load to the surrounding soil and away from the conduit.” The flat bars are metal sheets with 1.6 mm thickness and 100 mm width, and the surfaces have notches in order to create greater friction with the soil. The study was carried out in three dimensions on a pipe arch using ABAQUS, and compared with a reduced-scale model. Three parameters of the pipe wall were examined: the displacement, thrust and bending moment. The authors conclude that the ideal method to improve load-carrying capacity of soil-steel bridges is by reinforcing the soil as opposed to the use of transverse stiffeners.

Applying the conclusion from Mohammed and Kennedy regarding reinforcing the soil, Melgar [26] introduced the use of geogrids instead of flat metal bars. With a reduced-scale model, Melgar studied the behaviour of pipe-arches in both reinforced and unreinforced soil, and to make a comparison between the two. Eight geogrid layers were placed in the soil adjacent and attached to the pipe-arch. Gauges were set up to determine the displacements in the crucial area of the pipe. A strip load 250 mm wide was applied at the soil surface and load versus displacement curves and tables are presented. Several conclusions are listed, yet the most important one related to the present analysis is the statement regarding the large increase in load-carrying capacity of the reinforced soil when compared to unreinforced soil. A second important conclusion is that “sudden and catastrophic failure, often found in unreinforced-soil pipe-arches, can be eliminated when using reinforcement. Reinforcing the soil surrounding the pipe-arch changes the failure mode to a more predictable and progressive collapse”. Melgar recommends that further research should be conducted on other soil-steel structure shapes incorporating the use of geogrid to reinforce the soil.

Instead of reinforcing the soil with geogrids, Bathurst and Knight [3] at the Royal Military College of Canada constructed a full-scale model and analyzed a circular conduit buried under shallow depth of cover with a geocell to reinforce the soil. The
geocell has a thickness of 10 or 20 cm, and is “manufactured from thin strips of polymeric material (usually high density polyethylene) bonded or welded together to form a three-dimensional cellular network that can be filled with compacted soil”. The single layer of geocell is placed in the soil cover between the soil surface and pipe crown, the top of the geocell 0.2 m below the soil surface. The result is stiffened soil above the flexible pipe. Experimentation in plane strain included both computer analysis and reduced scale models. The geocell was modelled by means of plane strain continuum elements. Both concentric and eccentric strip loading were analyzed. The research of Bathurst and Knight is very similar to the present analysis for this thesis. However, two of the principal differences are: Bathurst and Knight experimented with geocells, and the present analysis is with geogrids; secondly, the diameter of pipe is 6 m in the study of Bathurst and Knight, as opposed to 7.6 m in the present analysis. Bathurst and Knight stated the following conclusions:

- By introducing one layer of geocell in the soil cover, 0.2 m from the soil surface, the load-carrying capacity under concentric strip load is increased by a factor of four;
- Reinforcing the soil above the buried pipe improves the load carrying capacity such that the minimum depth of cover may be reduced;
- Eccentric loading influences the failure load for reinforced and unreinforced soil-steel bridge.
CHAPTER 3
LARGE-DIAMETER FLEXIBLE PIPE

Analysis and design of soil-steel bridges requires several steel and soil properties, as well as interaction parameters between the soil and pipe. This section summarizes the numerous and important parameters related to the soil-steel structure analysis.

3.1. Soil

Soil underneath the surface in its natural environment is composed of various amounts of sand, clay, gravel, etc, each having varying engineering properties. When a soil-steel structure is constructed, varying engineering properties is unacceptable; proper design stipulates the safest and most economical construction process, which includes the best engineering properties, ensuring stability of the structure. Therefore, compacted construction fill with known engineering properties replaces weaker soil material. Since well-compactd granular material surrounds the structure, optimum support of the pipe is realized.

If the soil surrounding the pipe is a well-compacted homogeneous construction fill, the engineering properties are very favourable. What are the engineering properties? Two properties, which establish the failure criterion of the soil, are angle of friction and cohesion. Angle of friction represents to an extent the degree of compaction of the soil material, and cohesion is a measure of stress of how well particles remain attached to each other. How are these two properties determined?

The two values are determined in the laboratory by means of a direct shear test. As seen in Fig. 3.1.1 (a), soil material is placed in a shear box, and a normal force is applied, while a shearing force is also applied. Normal, \( \sigma \), and shear stress, \( \tau \), are plotted as shown in Fig. 3.1.1 (b). The angle of friction (\( \phi \)) in a granular material is dependent on the amount of compaction. Friction angles range from 26° to 45°, the former for loose sand and the latter, dense sand. A typical friction angle used for design of compacted sand range from 35° to 45°. As for the value of cohesion (c), sand is labeled a cohesionless material—cohesion is zero. When modeling the soil with ABAQUS, the program requires the inclusion of a cohesion value regardless of the soil type. Abdel-Sayed and Salib [2] explain the requirement of a cohesion value in that “a small magnitude of cohesion is imposed into the
finite model to avoid a singularity problem during the runs of ABAQUS". The cohesion value used in the analysis is 7 kPa, which is relatively small.

Figure 3.1.1 (a) Schematic diagram of the direct shear test; (b) plot of test results to obtain the friction angle, \( \phi \); \( c \) = cohesion [10]

A simplified method of analyzing the pipe-soil system is to assume purely elastic soil, a method employed by Fernando and Carter [13]. However, soil should not be modeled as an elastic system because it by no means acts in an elastic manner. Therefore, a soil failure criterion must be specified. Das [11] describes the shear strength of a soil as

\[
\tau = c + \sigma \tan \phi
\]  
(3.1.1)
Equation 3.1.1 is known as the Mohr-Coulomb failure criterion, where the shear strength of the soil determines failure. The equation for shear stress (3.1.1) is written in the simplest form; however, it is most often written in the form of $\sigma_3$ and $\sigma_1$. The most common form of shear stress is seen in Eq. 3.1.2.

$$\sigma_1 = \sigma_3 \tan^2(45 + \phi/2) + 2c \tan(45 + \phi/2)$$  \hspace{1cm} (3.1.2)  

Mohr-Coulomb is the failure condition used in the present analysis using ABAQUS. Equation 3.1.2 defines the minimum compressive normal stress, $\sigma_1$, in terms of the maximum compressive normal stress, $\sigma_3$, angle of friction and cohesion. Each of these variables are presented in the Mohr-Coulomb theory failure criteria presented in Fig. 3.1.2. The use of the Mohr-Coulomb theory for soil is apparently widespread in finite element modeling, as evident in a variety of literature, and this may be credited to the simplicity of calculations in the analysis. If at any point during loading a combination of shear stress and compressive stress in the soil plot above the failure envelope defined by $c$ and $\phi$, and shown in Fig. 3.1.2, soil failure occurs and the analysis ends.

![Mohr-Coulomb failure criteria](image)

**Figure 3.1.2 Mohr-Coulomb failure criteria**

In the majority of finite element analysis, soil is represented by means of quadrilateral continuum elements in plane strain. Continuum elements, according to the ABAQUS Standard User’s Manual [20], are solid elements that may either be linear or quadratic. A linear four-sided continuum element has four nodes at which displacements are calculated, and four interior integration points at which the stresses and strains are calculated.
A first-order differential equation is used to solve linear elements. A quadratic element, however, has eight nodes and nine interior integration points, and a second order differential equation is used to solve the problem. The two types are shown in Fig. 3.1.3.

![Figure 3.1.3 Linear and quadratic plain strain continuum elements [20]](image)

3.2. Corrugated Steel Pipe

Two components are required in the design and analysis of corrugated pipe, the material properties and the geometrical properties. The elastic modulus of the steel pipe is 200 GPa, and yielding of the pipe is taken into account by a plasticity value of 182 MPa. The elastic property of the material allows for the recovery of deformation when loaded; the plastic property denotes the point at which yielding occurs, where deformation is no longer recoverable. Poisson's ratio of steel is 0.3. The diameter of the corrugated circular pipe is 7.6 m.

In order to model structural elements, such as pipe walls, the beam element is chosen. Beam elements are used for “structures in which one dimension is significantly greater than the other two dimensions” [20]. A quadratic beam element is used in the analysis. The quadratic beam element contains two integration points; each integration point is a distance of L/4 (L = beam length) from the beam ends, and has three nodes. Each node can support loads in the horizontal and vertical local axis, and is restrained in the direction of the plane in plane strain analysis.

The flexible pipe in the analysis has a 152 X 51 mm corrugation profile with a thickness of 5.0 mm. The corrugated profile, as depicted in Fig. 3.2.1, is represented in ABAQUS by means of rectangular beam elements; therefore an effective thickness is required, as introduced in the literature review of Fernando and Carter [13]. Determining the effective thickness of the corrugated pipe is simply converting the pipe wall corrugation to a
uniform constant thickness all around the pipe wall, using the second moment of cross-sectional area, or bending stiffness of the corrugated plate 152 X 51 mm with 5 mm thickness. Because the model is in plane strain, all calculations are per meter in the plane, or z-direction. The value for second moment of cross-sectional area for a 152 X 51 mm corrugated pipe of thickness 5.0 mm is 1871 mm$^4$/mm, and the calculations for effective thickness, h, are as shown:

\[
I_s = \frac{bh^3}{12} \text{ per m}
\]

\[
1871 = \frac{(1000\text{mm})h^3}{12(1000\text{mm})}
\]

From the calculations, the value for h is determined to be 28 mm, or 0.028 m, and this is the thickness used for the beam elements in the ABAQUS FEA analysis. It is assumed for simplification purposes that seams are not used in the construction of the corrugated pipe, although this does not occur in actual pipe construction. This assumption is justified because the steel pipe wall is quite thick, at 28 mm, and substantially reduces the opportunity for failure in the pipe wall. This allows for failure to occur in the soil above the circular soil-steel bridge as is typical in soil-steel bridge failures.

![Figure 3.2.1 Corrugation profile of 152 X 51 mm plates](image)

3.3. **Soil-Steel Interaction**

The interaction between the corrugated steel and construction fill, and the support given from one to the other is one of the principal advantages of the soil-steel bridge. Therefore, the interaction should be modeled as best as possible. The following section discusses the characteristic that defines the interaction of the two materials.

The deflection of the uppermost point of the pipe, or crown, is dependent on the rigidity of the pipe. A circular concrete pipe is certainly more rigid than a flexible corrugated
steel pipe, and would therefore deflect less. The soil characteristic that defines the interaction between the soil and pipe is the modulus of soil stiffness, or modulus of soil reaction, $E'$. This value "represents the stiffness of the soil at the soil-pipe interface and is defined as interface pressure ($p$) divided by soil strain ($\varepsilon$)" [1], as indicated by Eq. (3.3.1).

$$E' = \frac{p}{\varepsilon}$$

(3.3.1)

Values of $E'$ for well-compacted soil range from 6.9 to 20.7 MPa, depending on the type of soil.

The simplest manner to model the interaction between soil and pipe in finite element analysis is by assuming a complete bond between the pipe and soil. Examples of this assumption are found in Fernando and Carter [13], Girges [14] and Mohammed and Kennedy [28]. Fernando and Carter explain that the pipe and soil have intimate contact, with no slip between the two when a load is applied. The only situation in which no slip occurs between the pipe wall and surrounding soil is when the deformations are small. They justify the complete bond because in most cases "the pipe tends to settle with the soil unless the soil is very soft relative to the pipe". In Girges' case, the complete bond is justified because "the present work is mainly a comparative study for different cases either in three-dimensional or two-dimensional analysis". Mohammed and Kennedy explain that the contact between the corrugated pipe and soil is an extreme geometrical complexity, therefore, a complete bond is assumed.

In analysis of foundations, it is common to model the interaction between the foundation and soil by means of springs. Similarly, Ghobrial and Abdel-Sayed [16] employ springs between the pipe and soil, as discussed in the literature review. In order to relate the springs of foundations to springs between pipe and soil, three parameters are required: modulus of soil stiffness, $E'$, coefficient of normal reaction, $k_n$, and coefficient of tangential (or shear) reaction, $k_s$. Included with the use of these parameters are two important assumptions:

- "The displacement at a given point on the soil-structure interface is assumed to be directly proportional to the pressure developed at that point."
- $E'$, $k_n$, and $k_s$ are all often assumed to be unique properties of the soil medium, whereas these parameters also depend upon the type of structure, the loading configuration, and the relative stiffness of the structure.” [1]

Springs are attached from the nodes of the beam elements of the pipe wall to the nodes of the plane strain continuum elements representing the soil as shown in Figure 3.3.1. Each spring has a stiffness of value $k_n$ in the normal direction, and $k_s$ in the tangential direction. The coefficient of normal reaction is the measure of normal pressure at the pipe-soil interface divided by displacement due to that pressure (kN/m$^3$). In order to translate this coefficient to a spring stiffness measured as force divided by displacement (kN/m), to properly model the soil steel reaction in the normal direction, Eq. 3.3.2 is used.

$$k_n \text{ (kN/m)} = E'd \quad \text{3.3.2}$$

where $d$ is the distance between nodes at the soil steel interface, as seen in Figure 3.3.1. In the present analysis, the values of $k_n$ for the normal springs around the perimeter range from $1.0 \times 10^8$ to $2.2 \times 10^9$ kN/m. The coefficient of tangential reaction, on the other hand, is “the shear force divided by the tangential displacement at the soil-pipe interface, these being denoted by $V$ and $\Delta_s$, respectively” [1], as shown in Eq. 3.3.3.

$$k_s = \frac{V}{\Delta_s} \quad \text{3.3.3}$$

The authors of *Soil-Steel Bridges* state that because $k_s$ is not dependent on the location of the particular spring, the value for $k_s$ is taken as 20% of $k_n$ at the invert. In the present analysis, the coefficient of tangential reaction is taken as $3.2 \times 10^7$ kN/m.

![Figure 3.3.1 Modeling the interaction between pipe and soil with springs (enlarged representation—located at the soil-pipe interface)](image-url)
To better describe Fig. 3.3.1, as already stated, an eight-node continuum element representing the soil is connected to the pipe's three-node beam element by means of springs. A normal spring connects the node of the continuum to the adjacent node of the pipe element. Tangential springs are more clearly evident in Fig. 3.3.1, one spring connects the upper node of the soil element with the middle node of the beam element, and a second spring connects the middle node of the soil element to the bottom node of the beam element.

3.4. Depth of Cover

A minimum depth of cover provided in design codes is mandatory in order to limit the potential for failure in soil-steel structures. The Canadian Highway Bridge Design Code [7] states a specification regarding the minimum depth of cover over large-diameter flexible pipes. The specified minimum depth of cover is the largest from the three criteria:

a) 0.6 m;

b) \( \frac{D_h}{6} \left( \frac{D_h}{D_v} \right)^{1/2} \); and

c) \( 0.4 \left( \frac{D_h}{D_v} \right)^2 \) m

where \( D_h \) and \( D_v \) are horizontal and vertical spans, respectively, each 7.6 m for the circular conduit. The specified minimum depths of cover in the present case are 0.6 m, 1.27 m and 0.4 m. Thus, according to the code, the minimum depth of cover is the largest of the three values, 1.27 m. Table 3.4.1 lists a summary of the depths of cover and depth-to-span ratios analyzed for the study of concentric and eccentric loading above the pipes. The range of variability of depth of cover analyzed because such is recommended and referred to in other literature, namely Abdel-Sayed and Salib [2].

<table>
<thead>
<tr>
<th>Cover depth-to-span ratio</th>
<th>Depth of cover (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.10</td>
<td>0.76</td>
</tr>
<tr>
<td>0.20</td>
<td>1.52</td>
</tr>
<tr>
<td>0.25</td>
<td>1.90</td>
</tr>
<tr>
<td>0.30</td>
<td>2.28</td>
</tr>
</tbody>
</table>
3.5. Boundary Conditions of the Problem

The boundary conditions of the soil are as follows: fixed at the bottom in both the horizontal and vertical directions, and fixed in the horizontal direction, allowing for vertical displacements along the sides. The soil surface is unrestrained. Figure 3.5.1 depicts the general geometry of the problem, investigated in this thesis, and with the large-diameter flexible pipe buried under 1 m of soil. While Fig. 3.5.1 depicts the general geometry of each analysis, Fig. 3.5.2 portrays a typical mesh of the problem’s geometry analyzed in ABAQUS. The mesh design depends on the depth of cover of soil above the pipe, but this is a typical mesh for the various depths of cover. The ABAQUS input file of the mesh is provided in Appendix A, as well as discussion and explanation of the components of the input file.

![Figure 3.5.1 General geometry of the soil-steel structure analysis under 1 m of soil cover](image)

![Figure 3.5.2 Mesh of circular soil-steel bridge](image)
CHAPTER 4
LOADING

Two components of loading are required in the analysis of large diameter flexible pipes under roadways. The first component is the force of gravity applied to the soil, the dead load, and the second is the force from the weight of the truck, the live load. Loading in ABAQUS is somewhat complex; therefore it is necessary to familiarize the reader with the loading terms.

The terms describing the loading sequence are defined in the ABAQUS Standard User’s Manual [20]. A step is defined as the load history of the problem at hand; the step is where the load sequence is applied. When a problem is non-linear, as is the present case, the load may not be applied all at one time. Therefore, the load is applied in increments. A step is “broken into smaller increments so that the nonlinear solution path can be followed”. The size of the first increment is suggested, and ABAQUS chooses the remaining increments until the failure load is reached. During the analysis, “at the end of each increment, the structure is in approximate equilibrium”. Finally, an iteration “is an attempt at finding an equilibrium solution in an increment”. ABAQUS tries another iteration if the solution is not in equilibrium, and it is easily understood, that for complex problems, many iterations may be required. ABAQUS uses Newton’s method to solve iterations for the non-linear problem. A sample of a typical input file, in Appendix A, demonstrates the loading sequence, which is found at the very end of the input file.

4.1. Dead Load

A considerable portion of loading in the pipe-soil system is the dead load of the soil due to the force of gravity. The engineered fill, though compacted to a high degree, will settle somewhat. This settling is not uncommon in the field, and therefore must be incorporated into the analysis. The force of gravity is applied in engineering by means of the unit weight, of the material, \( \gamma \) (kN/m\(^3\)), presented in Eq. 4.1.1.

\[ \Delta p = \gamma \Delta z \]  

4.1.1
where $\Delta p$ is the pressure from gravity in kN/m$^2$, and $\Delta z$ is the difference in elevation of a soil layer subject to gravity. In the analysis at hand, the unit weight of the soil is taken as 21.2 kN/m$^3$; this value is taken as construction fill that is very-well compacted, so that it is very dense and can support very large loads. In order to incorporate gravity loading, ABAQUS offers the use of a gravity (GRAV) loading command as a distributed load, and is the first step of the analysis of the input file in Appendix A. In soil analysis, it is not uncommon for the gravity load to be “switched on” [1], as is the case here. Each soil continuum element is subject to a downward loading at all times throughout the analysis. Settlement of the soil occurs, and when the gravity load sequence in step one is complete, the next component of loading, live load, is added in step two, as shown in the input file in Appendix A.

4.2. Live Load

The Canadian Highway Bridge Design Code [7] defines the design truck as CL-W, where W is the total truck load in kN. The design truck outlined in the CSA code is the CL-625, shown in Fig. 4.2.1. Typically, wheel loads are modeled as concentrated forces; however, to model wedge failure, one must take the most critical area of loading, which in this case are the two rear axles of the truck, axle numbers 2 and 3 from the figure. They are separated by 1.2 m in the longitudinal direction and 1.8 m in the transverse direction.

In three-dimensional finite element analysis, each wheel load is modeled using a concentrated force. However, the present analysis is concerned with two unique situations: plane strain instead of three dimensions, and wedge failure by means of a rigid strip load instead of concentrated or axle loads. To obtain a rigid strip load, the concentrated wheel loads from axle numbers 2 and 3 in Fig. 4.2.1 must be converted into a strip load for modeling of wedge failure in plane strain. The rigid strip load is formed by all four rear wheel loads, 62.5 kN each, applied to the roadway in a rectangle 1.2 m by 1.8 m. The rigid strip load is necessary because it is connected to the failure of the soil. Failure in the soil does not occur by flexible loads. The four concentrated forces are depicted in Fig. 4.2.2 along with the distributed load, which replaces the four wheel loads. The applied distributed load ($q_a$) of 116 kN/m$^2$ is simply the addition of the four
wheel loads of 62.5 kN divided by the rectangular area. Consequently, the value of $q_a$ is taken as a constant measure of applied load, and is used in the determination of the safety factor.

Figure 4.2.1 Canadian Highway Bridge Design Code truck load [7]
Figure 4.2.2 Live load converted from concentrated loads to strip load

Two approaches of applying live loads are analyzed at the soil surface above the flexible pipe. The first, concentric loading is defined as placing the centre of the strip load directly above the crown of the pipe, as seen in Fig. 4.2.3 a). The four depths of cover analyzed with concentric loading, as outlined in Table 3.4.1 are $h/S = 0.10, 0.20, 0.25$ and $0.30$.

The second loading approach, eccentric loading, is defined as the placement of the strip load offset a horizontal distance ‘e’ from the crown of the pipe. The distance ‘e’ is depicted in Figure 4.2.3 b). Identical to concentric loading, eccentric loading analysis will also occur for the four depths of cover listed in Table 3.4.1. The four eccentric ratios and distances analyzed are listed in Table 4.2.1

Figure 4.2.3 Concentric (a) and eccentric (b) loading
Table 4.2.1 Ratios and Horizontal Distances of Eccentric Loading: span (S) = 7.6 m

| \( e \)  
<table>
<thead>
<tr>
<th>( (S/2) )</th>
<th>Horizontal distance from crown to centre of strip load, ( e ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>0.76</td>
</tr>
<tr>
<td>0.4</td>
<td>1.52</td>
</tr>
<tr>
<td>0.6</td>
<td>2.28</td>
</tr>
<tr>
<td>0.8</td>
<td>3.04</td>
</tr>
</tbody>
</table>

4.3. Safety Factor

Design of a soil-steel structure should include factors of safety. There are several reasons for this. There may be uncertainty in the size or quantity of applied loads over the life of the structure, defects in the steel pipe, poor construction practices such as compacting the soil improperly, and settlement of the structure over its lifespan.

The failure load defines the safety factor. Once gravity dead load is applied in the first step of analysis, the live (strip) load is applied by ABAQUS in increments until a peak failure load is determined. Soil behaves nonlinearly; therefore the load must be analyzed incrementally. Failure is specified, as mentioned, by means of the Mohr-Coulomb failure criterion, as discussed earlier. The failure load of unreinforced soil will be identified as \( q_s \), with units of kN/m², whereas the failure load of the reinforced soil will be identified as \( q_r \), also in kN/m². The safety factor employed in the analysis is defined by

\[
\text{Safety Factor} = \frac{q_r}{q_s} \quad \text{and} \quad \frac{q_r}{q_a}
\]

(4.3.1)

where \( q_a \) is the applied rear truck axle load of 116 kN/m² as calculated in the previous section. The safety factor applies to both concentric and eccentric loading.
CHAPTER 5
GEOGRID

There are all sorts of shapes, sizes, purposes and types of geogrid. For the most part, geogrid is made of polypropylene, a plastic material with high tensile strength and excellent chemical and weathering resistance. The construction fill falls through the apertures, the grid openings, and interlock with the grid to increase the stiffness and strength of the soil, when the load of repetitive type is applied. Since soil is very weak in tension, the geogrid provides a large amount of tensile resistance to the soil. Different types and sizes of geogrids have different amounts of resistance and different interlocking capabilities with the soil. To increase the interlocking capability, compaction of the fill above the geogrid is required. Once a small amount of settling occurs from compaction—carried out by means of heavy equipment through repetitive rolling and vibration—the soil reinforcing ability of the geogrid begins. According to Das [10], and similar to steel reinforcement in concrete, once the geogrid begins to deform from the soil settling, resistant stresses in the reinforcement begin to increase. For that reason, a greater interaction and stability between the construction material and the geogrid causes a greater load-carrying capacity of the soil.

One of the more widely employed geogrid in construction has 25 mm square apertures, rib thickness of 0.76 mm and junction thickness of 2.29 mm. Figure 5.1 portrays the geogrid BX1100 fabricated by Tensar Earth Technologies in Atlanta, Georgia, where B represents biaxial loading, or resistance in two directions, as opposed to uniaxial (U) loading. The model number is represented by 1100; number 1000 was one of the original geogrids manufactured by Tensar, and is now discontinued, replaced by model 1100 because of improvements in fabrication technology and greater strength.

![Geogrid BX1100](image)

Figure 5.1 Geogrid BX1100 fabricated by Tensar Earth Technologies [22]
Three important material properties are needed in the discussion and analysis of geogrids, the elastic modulus, the plastic yield strength in tension, and Poisson’s ratio. Typical properties of polypropylene [21, 24] are as follows: elastic modulus is approximately 1.6 MPa, and the material’s plastic yield strength in tension is approximately 35 MPa. Because it is practically incompressible, the Poisson’s ratio is taken as 0.49.

In order to discuss the failure of soil from a strip load when layers of geogrid are placed in the soil, the design methodology of foundations is applied. In foundation design, the load is applied as a rigid strip load in a homogeneous half space, which is defined as a large quantity of soil, in plane strain, with all the same parameters of elasticity, E, and Poisson’s ratio, μ.

5.1. Bearing Capacity Ratio

Given that the purpose of the geogrid is to reinforce the soil, engineers in foundation design have introduced the bearing capacity ratio (BCR_u) in order to quantify the increase of strength generated in the soil due to the geogrid. The bearing capacity ratio is defined in Eq. 5.1.1,

\[ BCR_u = \frac{q_r}{q_u} \]  

where \( q_r \) is the failure load for reinforced soil, and \( q_u \) is the failure load for unreinforced soil, both introduced in the previous section. The ratio is used in the determination of the ideal geogrid parameters, discussed in the next section, to best reinforce the soil.

5.2. Placement of Geogrid Layers

Four main parameters are included in the placement of geogrids in a homogeneous half space. The parameters are the number of layers, N, the depth of the first layer below the foundation, \( d \), the distance between each layer, \( \Delta H \), and the horizontal distance for which the geogrid extends past the strip load, 2 \( L_0 \). The depth to which the geogrids extend is defined by \( u \), given in Eq. 5.2.1, from Das [10].

\[ u = d + (N-1) \Delta H \]  

Each of these parameters is depicted in Fig. 5.2.1. In the present analysis, \( d \) and \( \Delta H \) are kept constant, while \( N \) is varied.
Figure 5.2.1 Geogrid parameters in foundation design in a homogeneous half space [10]

Each parameter related to the placement of the geogrid plays a significant role in the bearing capacity of the soil, some parameters having more significance than others. As discussed in the literature review, Omar et al. [30] tested each of the parameters to determine the ideal number, best location, and most economical length of geogrid in soil. Most importantly, the parameters depend on the type of soil, clay or sand, and the compaction of the soil, or relative density, including the angle of friction. Similarly, the parameters depend on the foundation width to length ratio, or B/L. Since the analysis is plane strain, L has infinite length and the value for B/L is zero.

The present analysis sets the depth of the first layer of geogrid constant. This is credited to the fact, according to Omar et al., that the maximum values of BCR are determined when d/B = 0.25 to about 0.4. To simplify the analysis, and to follow the work of Omar et al., the ratio of d/B will be kept constant with the average of the range of d/B values, at d/B = 0.333, or d = B/3. Because the strip load is 1.2 m wide, this depth of geogrid equates to 0.4 m. It was also verified with ABAQUS that the depth of 0.4 m is the ideal depth for utilizing the full capacity of the geogrid. Likewise, Omar et al. state that the ideal distance between each layer, ΔH, is equal to B/3, or in this analysis, 0.4 m. Therefore, both the depth of the first layer and distance between the layers are 0.4 m.

The third parameter to review is the horizontal distance to which the geogrid extends in the soil below the strip load, or 2 L_o, which is significant because this length acts against pullout of the geogrid. From Omar et al., the maximum BCR is reached
when $2 L_0/B \approx 8$, which equates to placing the geogrid to a distance of 4.8 m to either side of the centre of the strip load.

Finally, the bearing capacity of the soil is directly related to the number of reinforcing layers. However, the question arises regarding the maximum number of layers for maximum soil strength, in terms of maximizing the economics of geogrids. Tests for the optimum number of geogrid layers has also been carried out in Omar et al.. The variation of $BCR_u$ (Eq. 5.1.1) with $N$ is portrayed in Fig. 5.2.2. It is evident that $BCR_u$ depends on the shape of the load, defined by $B/L$, or width-to-length ratio of the rigid strip load. It is emphasized that Fig. 5.2.2 is for sand with a relative density of 70% and angle of friction of 41°. Also, the geogrid in the experiment is the discontinued B1000, and an up-to-date BCR vs N figure using the new BX1100 are unavailable presently. It is assumed that the two materials produce identical results; this is not the case because BX1100 has improved strength characteristics over the older model.

![Graph showing the variation of $BCR_u$ with $N$ for different $B/L$ ratios](image)

Figure 5.2.2 Variation of $BCR_u$ with $d/B = \Delta H/B = 0.333$ [10]

Soil strengths under rigid strip loading, where $B/L = 0$, do not increase by placing more than five or six layers of geogrids, as seen in Fig. 5.2.2. The results show
that increasing the number of geogrid layers up to \( N_{cr} \) (which defines maximum capacity of the reinforced soil) is associated with progressive stiffening of the soil adjacent to the geogrids in the homogeneous half space. For the present analysis, because a pipe is included—not a homogeneous half space—and different heights of cover are specified, there is a depth restriction for the number of layers of geogrid. Table 5.2.1 presents the appropriate number of layers \( N \) for the various depths of cover analyzed. The table also lists the clearance available between the deepest geogrid layer and the pipe crown.

Table 5.2.1 Depth Restriction for Geogrid Placement for the Various Depths of Cover

<table>
<thead>
<tr>
<th>( h ) ( S )</th>
<th>Depth of cover (m) above pipe crown</th>
<th>Design ( N )</th>
<th>( u ) (m)</th>
<th>Clearance of bottom geogrid above pipe (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.10</td>
<td>0.76</td>
<td>1</td>
<td>0.40</td>
<td>0.36</td>
</tr>
<tr>
<td>0.20</td>
<td>1.52</td>
<td>3</td>
<td>1.20</td>
<td>0.32</td>
</tr>
<tr>
<td>0.25</td>
<td>1.90</td>
<td>3</td>
<td>1.20</td>
<td>0.70</td>
</tr>
<tr>
<td>0.30</td>
<td>2.28</td>
<td>5</td>
<td>2.00</td>
<td>0.28</td>
</tr>
</tbody>
</table>

5.3. Failure Modes

There are three main modes of failure to be concerned with in the design of geogrid. Experimental results in a homogeneous half space show that the modes are shear failure in the soil, geogrid pullout or breaking of the reinforcement.

Shear failure in the soil in the homogeneous half space occurs if the first layer of geogrid is placed at a significant depth below the surface as to allow for the maximum shear to develop. Shear failure is most often evident when depths of soil are greater than \( 2/3 \) \( B \), or in this case, 0.8 m.

The second mode of failure is geogrid pullout, where the geogrid does not fully interlock with the soil, so that there is sliding between the soil and geogrid. Pullout is a rare form of failure; however, when minimal compaction is applied to the soil, complete interlocking between the soil and the geogrid does not occur. In the early years of reinforcement, metallic strips were used to strengthen the soil, and reinforcement pullout was much more often the source of failure. Since the problem at hand uses the technique of effective thickness of geogrid, introduced in the next section, where the geogrid is
thick compared to its actual thickness, the geogrid does not pullout, therefore this mode of failure does not occur.

Finally, the third mode of failure is breaking of the geogrid, or rupture of the material. When the first layer of geogrid is placed at a depth less than $2/3$ B, there are enough layers included in the design ($N>4$), and the layers extend well beyond the economical length of $2 L_o / B \approx 8$, it is expected that the upper layers of reinforcement will break. This is the desired mode of failure, because as geogrid material tensile strength increases as plastics material and fabrication technology increases, the reinforced soil will only gain that much more resistance to failure. In the present analysis, it is possible to view the stresses in each of the geogrid layers by means of the data output. If the axial stresses in the reinforcement are not larger than the plastic yield limit of 35 MPa, the material is still in the region of elasticity, and therefore could not have broken. If the geogrids have not reached plastic yield, therefore, it can be concluded that overall failure in the reinforced circular soil-steel bridge is due to shear failure of the soil between the surface and first layer of geogrid.

5.4. Effective Geogrid Thickness

In three-dimensions, the reinforcement forms a grid-like pattern. However, the FEM problem is plane-strain, and two dimensions represent the three-dimensional geogrid structure. According to Peng et al. [31], the geogrid is modelled “as a planar reinforcing member having the same global material characteristics, such as strength, stiffness and pull-out resistance, as the original grid.” An effective thickness of the reinforcing element (without apertures) in FEA must be determined to properly model the grid formation. The justification for determining an effective thickness is seen in the fact that the geogrid aperture allows for the penetration of adjacent soil when subjected to repetitive loading. The two materials, soil and geogrid, act as one composite material.

Homogeneous half space, as mentioned, is defined as soil, which has all the same parameters in the entire boundary analyzed, these parameters being $E$, modulus of elasticity, and $\nu$, Poisson’s ratio. The half space is employed in foundation and geogrid design, among many other applications, and is applied in the present research to determine effective thickness of the geogrid. Effective thickness determined in the
homogeneous half space of the various geogrid layers will be applied to the geogrid-
reinforced circular soil-steel bridge to determine the factor of safety from a rigid strip
load.

A large boundary of soil representing a homogeneous half space, similar to Fig.
3.5.1, is analyzed in an unreinforced state, as well as reinforced with increasing geogrid
layers to determine the geogrid effective thickness. The unreinforced mesh is portrayed
in Fig. 5.4.1. Reinforcement in ABAQUS is modeled by truss elements. By means of
trial and error and the results from Omar et al., presented in Fig. 5.2.2 regarding BCR_o
and N, the effective thickness of each reinforcing truss element is determined. When two
or more geogrid layers are used to reinforce the soil, the effective thickness for each layer
is kept constant in the trial and error analysis, in order to simplify the determination of the
effective thickness. The trial and error analysis takes place until the the criterion in Table
5.4.1, from Omar et al. for a rigid strip load in a homogeneous half space, are determined.

Figure 5.4.1 Mesh of homogeneous half-space

<table>
<thead>
<tr>
<th>N</th>
<th>Range of BCR values</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.70 - 1.80</td>
</tr>
<tr>
<td>2</td>
<td>2.45 - 2.55</td>
</tr>
<tr>
<td>3</td>
<td>3.20 - 3.30</td>
</tr>
<tr>
<td>4</td>
<td>3.70 - 3.80</td>
</tr>
<tr>
<td>5</td>
<td>4.20 - 4.30</td>
</tr>
</tbody>
</table>
The results of trial and error for the determination of effective thickness for each of the layers, outlined in the previous paragraph, as well as their respective bearing capacity ratios, are presented in Table 5.4.2. The effective thickness for each layer is depicted in Fig. 5.4.2. The increasing bearing capacity of the reinforced system is related not only with the number of geogrid layers but also with the development of stronger composite layer around the geogrid. This fact is translated into increasing the effective thickness as the number of geogrid layers N increases.

### Table 5.4.2 Effective Thickness for Each Number of Geogrid Layer (N), and BCR values

<table>
<thead>
<tr>
<th>N</th>
<th>Effective Thickness (m)</th>
<th>( q_e ) (kN/m²)</th>
<th>BCR (( q_u = 1120 ) kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0010 (1(^{st}) layer)</td>
<td>1940</td>
<td>1.73</td>
</tr>
<tr>
<td>2</td>
<td>0.00347 (2(^{nd}) layer) 0.00347 (1(^{st}) layer)</td>
<td>2770</td>
<td>2.47</td>
</tr>
<tr>
<td>3</td>
<td>0.00503 (3(^{rd}) layer) 0.00503 (2(^{nd}) layer) 0.00503 (1(^{st}) layer)</td>
<td>3600</td>
<td>3.21</td>
</tr>
<tr>
<td>4</td>
<td>0.0131 (4(^{th}) layer) 0.0131 (3(^{rd}) layer) 0.0131 (2(^{nd}) layer) 0.0131 (1(^{st}) layer)</td>
<td>4180</td>
<td>3.73</td>
</tr>
<tr>
<td>5</td>
<td>0.0182 (5(^{th}) layer) 0.0182 (4(^{th}) layer) 0.0182 (3(^{rd}) layer) 0.0182 (2(^{nd}) layer) 0.0182 (1(^{st}) layer)</td>
<td>4780</td>
<td>4.27</td>
</tr>
</tbody>
</table>
5.5. Soil Friction Angle

The determination of safety factors for reinforced homogeneous soil, without an opening for a pipe, depends on the discussed soil strength parameters, mainly angle of friction, $\phi$. However, tests by Omar et al. were conducted on soil with friction angle of $41^\circ$. Therefore, a friction angle in the general proximity of $41^\circ$ must be used in analysis of geogrid-reinforced homogeneous half space and geogrid-reinforced circular soil-steel bridge.

Some general remarks are required regarding other soil friction angles. In geotechnical engineering, it is known that the greater the friction angle, the greater the ultimate load, 

$$ (q_{ult})_{\phi = 35^\circ} < (q_{ult})_{\phi = 40^\circ} < (q_{ult})_{\phi = 45^\circ}. $$

The placement of geogrid with constant spacing and constant equivalent area in soil with different friction angles has differing effects on bearing capacity. By employing the above relationship to reinforced soil, it can be expected that the homogeneous soil medium with $\phi = 35^\circ$ will result in a higher increase of factor of safety than the same
amount of reinforcement placed in soil with $\phi = 40^\circ$, and consequently the same amount
of reinforcement placed in the medium with $\phi = 45^\circ$. This is based on logic, that if

$$(\Delta FS)_\phi = 35^\circ > (\Delta FS)_\phi = 40^\circ > (\Delta FS)_\phi = 45^\circ,$$

then there is a greater increase in bearing capacity caused by the insertion of the geogrid
for soil with a friction angle of $35^\circ$, then for $40^\circ$, and similarly $45^\circ$. While the analysis is
carried out with $\phi = 40^\circ$, the principle of increasing bearing capacity with decreasing
friction angle is an important point to remember.
CHAPTER 6
RESULTS OF THE ANALYSIS OF UNREINFORCED AND REINFORCED HOMOGENEOUS HALF SPACE

Before introducing and discussing the results of the circular soil-steel bridge when a strip load is applied, it is necessary to present and discuss the results of the homogeneous half space, where the homogeneous half space is defined in Section 5.4. In addition to the presentation and discussion of results regarding the factor of safety, it is helpful to present and discuss the results of the soil behaviour at failure for the unreinforced and reinforced homogeneous half space. The presentation of the results of the soil behaviour are in the first section of this chapter, while the discussion of the results of the soil behaviour are found in Section 6.2. The factors of safety for the unreinforced and reinforced homogeneous half space are presented and discussed in Section 6.3.

6.1. Presentation of Results Depicting Soil Behaviour in Unreinforced and Reinforced Half Space

The behaviour of the soil is defined by the following: vertical displacements, horizontal displacements, and shear stresses in the homogeneous half space. Specifically, the soil behaviour of most interest is directly below and immediately to the sides of the strip load. The mesh of the half space is shown in Fig. 5.4.1, and the location of greatest interest is in the refined section of the mesh.

The distribution of vertical displacements, horizontal displacements and shear stresses (τ_{xy}) in the soil at failure are presented from Fig. 6.1.1 to 6.1.6, for the unreinforced and reinforced homogeneous half space. Failure load has significant impact on the vertical and horizontal displacements in the soil, as evident in the figures. Shear stresses in the soil (τ_{xy}) caused by the failure load are significant, as also evident in the figures. The significant changes in the soil behaviour at failure, due to the presence of the geogrid layers, only emphasizes the fact that displacements and shear stresses greatly enhance the discussion of factor of safety for the homogeneous half space.
a) Vertical displacement, U2, in meters

b) Horizontal displacement, U1, in meters

c) Shear stress at integration points, S12, in kN/m²

Figure 6.1.1 Distribution of vertical (U2) and horizontal (U1) displacements, and shear stresses (S12) at failure for unreinforced soil in homogeneous half space
a) Vertical displacement, $U_2$, in meters

b) Horizontal displacement, $U_1$, in meters

c) Shear stress at integration points, $S_{12}$, in kN/m$^2$

Figure 6.1.2 Distribution of vertical ($U_2$) and horizontal ($U_1$) displacements, and shear stresses ($S_{12}$) at failure for reinforced soil, $N = 1$, in homogeneous half space
a) Vertical displacement, $U_2$, in meters

b) Horizontal displacement, $U_1$, in meters

c) Shear stress at integration points, $S_{12}$, in kN/m$^2$

Figure 6.1.3 Distribution of vertical ($U_2$) and horizontal ($U_1$) displacements, and shear stresses ($S_{12}$) at failure for reinforced soil, $N = 2$, in homogeneous half space
Figure 6.1.4 Distribution of vertical (U2) and horizontal (U1) displacements, and shear stresses (S12) at failure for reinforced soil, N = 3, in homogeneous half space.
Figure 6.1.5 Distribution of vertical (U2) and horizontal (U1) displacements, and shear stresses (S12) at failure for reinforced soil, N = 4, in homogeneous half space
Figure 6.1.6 Distribution of vertical (U2) and horizontal (U1) displacements, and shear stresses (S12) at failure for reinforced soil, N = 5, in homogeneous half space
6.2. Discussion of Results Depicting Soil Behaviour in Unreinforced and Reinforced Half Space

At failure, unreinforced soil does not show any positive (upward) displacement, while in reinforced soil, as the number of geogrid layers increase, the failure load also increases, therefore, the vertical displacements also increase. This phenomenon is due to the greater stiffness generated in the soil due to the reinforcement. Since the failure load increases as the number of reinforcement layers increases, the soil is given the opportunity to displace a greater amount. In all cases of reinforced soil, vertical displacements to either side of the strip load increase upwards (positively), while in the soil below the load, the displacements are downward (negative). Positive vertical displacements are not evident in the unreinforced system because soil failure occurs before there is an opportunity to displace upward. Wedge failure is depicted in the vertical displacements to either side of the rigid strip load, but is even more visible in the horizontal displacements of the soil.

As a strip load is applied, the soil below the load not only moves vertically downward, but also outwardly in a symmetrical fashion, away from the load. Deep below and far horizontally from the load, horizontal displacements in the soil tend to zero, unaffected by loading. Viewing the horizontal displacements for the unreinforced system, as well as reinforced soil with \( N = 1 \), wedge failure is clearly noticeable. The situation changes, however, as the number of geogrid layers increases to \( N = 2 \) and greater. Failure in the soil with greater number of geogrid layers does not demonstrate a clearly defined wedge. On the other hand, it appears that the largest horizontal displacements are restricted to the portion of soil above the first layer of geogrid. Explanation for the restriction of displacements above the first geogrid layer is again attributed to the very large stiffness in the soil created by the large number of geogrids, thereby keeping the soil from moving any significant amount, except in the 0.4 m of soil between the first geogrid layer and the soil surface.

Shear stresses, depicted in Fig 6.1.1 c) through 6.1.6 c) in the soil caused by a rigid strip load, are similar to the shear stress contour in soil under shallow rigid foundations, discussed by Das [10]. In all cases, the maximum shear stress, or soil failure, occurs at approximately 0.6 m below the soil surface. For the soil with the larger
number of geogrid layers, this equates to soil failure directly between the second and third geogrid layer. Because the horizontal (axial) stresses in the geogrid have not reached the yield limit, it is concluded that failure of the system is located at a depth of 0.6 m below the soil surface, below each boundary defining the 1.2 m strip load.

6.3. Factors of Safety of Unreinforced and Reinforced Half Space

As stated in Chapter 4, the factor of safety is defined as the failure load determined in ABAQUS divided by the applied strip load of 116 kN/m². The failure loads, q₀ and qᵣ, are listed in Table 5.4.2. The factor of safety for the unreinforced soil, N = 0, and reinforced soil with 5 geogrid layers, for the homogeneous half space are presented in Table 6.3.1. The factors of safety are high, but this is because the applied load, qₐ, is low compared to soil strength. It is clearly evident that well-compacted soil can very safely support the rigid strip load caused by two truck axles. When the soil is reinforced with geogrid, the safety factor significantly increases. These factors of safety, along with the vertical and horizontal displacements and shear stresses of soil portrayed in Fig. 6.1.1 through 6.1.6, are essential for the purpose of comparison with the results of the circular soil-steel structure under concentric and eccentric loading, presented and discussed in the next two chapters respectively.

<table>
<thead>
<tr>
<th>N</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>9.66</td>
</tr>
<tr>
<td>1</td>
<td>16.7</td>
</tr>
<tr>
<td>2</td>
<td>23.9</td>
</tr>
<tr>
<td>3</td>
<td>31.0</td>
</tr>
<tr>
<td>4</td>
<td>36.0</td>
</tr>
<tr>
<td>5</td>
<td>41.2</td>
</tr>
</tbody>
</table>

Table 6.3.1 Factor of Safety for Unreinforced and Reinforced Soil in Homogeneous Half Space
CHAPTER 7
RESULTS OF THE ANALYSIS OF CONCENTRICALLY LOADED CIRCULAR SOIL-STEEL BRIDGE UNDER UNREINFORCED AND REINFORCED COVER

With the inclusion of a large-diameter flexible pipe, and the soil-steel parameters that are involved, which are discussed in Chapter 3, the results become more complex than the homogeneous half space without a pipe. The results of the soil behaviour at failure for the unreinforced and reinforced cover of a concentrically loaded circular soil-steel bridge are presented first in this chapter, then a discussion of the soil behaviour follows in Section 7.2, while plots of factors of safety verses various depths of unreinforced and reinforced cover over the circular soil-steel bridge are presented in discussed in Section 7.3.

7.1. Presentation of Results Depicting Soil Behaviour in Unreinforced and Reinforced Cover of Concentrically Loaded Circular Soil-Steel Bridge

The behaviour of the soil cover above the flexible pipe should also be discussed to enhance the discussion of the factor of safety of the concentrically loaded circular soil-steel bridge. The behaviour of the soil cover is defined in terms of the same three soil components as the homogeneous half space, vertical and horizontal displacements, as well as shear stresses.

For the soil-steel bridge subjected to a rigid strip load, the soil cover may either be unreinforced or reinforced with one to five geogrid layers, each geogrid spaced vertically 0.4 m. Figures 7.1.1 to 7.1.16 depict the three components of soil behaviour of the concentrically loaded soil-steel bridge having increasing depths of cover and varying layers of geogrid. Identical to the homogeneous half space, the greatest amount of displacements and stresses will be located in close proximity to the strip load, therefore, a close-up portrayal of the soil behaviour between the soil surface and the pipe crown is displayed. The distribution of vertical and horizontal displacements and shear stresses in the soil are discussed in the next section.
Figure 7.1.1 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.10, concentrically loaded
Figure 7.1.2 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 1, over circular soil-steel bridge, h/S = 0.10, concentrically loaded
Figure 7.1.3 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.20, concentrically loaded
Figure 7.1.4 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 1, over circular soil-steel bridge, h/S = 0.20, concentrically loaded
Figure 7.1.5 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 2, over circular soil-steel bridge, h/S = 0.20, concentrically loaded
Figure 7.1.6 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 3, over circular soil-steel bridge, h/S = 0.20, concentrically loaded
Figure 7.1.7 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.25, concentrically loaded
Figure 7.1.8 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 1, over circular soil-steel bridge, h/S = 0.25, concentrically loaded
Figure 7.1.9 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, $N = 2$, over circular soil-steel bridge, $h/S = 0.25$, concentrically loaded
Figure 7.1.10 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 3, over circular soil-steel bridge, h/S = 0.25, concentrically loaded
Figure 7.1.11 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.30, concentrically loaded
Figure 7.1.12 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 1, over circular soil-steel bridge, h/S = 0.30, concentrically loaded
Figure 7.1.13 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 2, over circular soil-steel bridge, h/S = 0.30, concentrically loaded
Figure 7.1.14 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 3, over circular soil-steel bridge, h/S = 0.30, concentrically loaded.
Figure 7.1.15 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 4, over circular soil-steel bridge, h/S = 0.30, concentrically loaded
Figure 7.1.16 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 5, over circular soil-steel bridge, h/S = 0.30, concentrically loaded.
7.2. Discussion of Results of Concentrically Loaded Circular Soil-Steel Bridge with Unreinforced and Reinforced Soil Cover

Upon review of the distribution of vertical displacements in the soil, in general it appears that the application of a failure rigid strip load, $q_{ult}$, on the soil surface located over the flexible bridge structure results in development of a trapezoidal wedge. The dark blue trapezoidal shape in the form of a trapezoid, directly below the load and above the pipe crown in all cases of concentric loading, Fig. 7.2.2 a) through 7.2.17 a), denotes the greatest amount of vertical displacements. If the trapezoidal wedge is compared to the vertical displacements of the homogeneous half-space, Fig. 6.1.1 a) through 6.1.6 a), it appears that the wedge is a small portion of the greater extent of vertical displacements that would have occurred below the strip load if the circular pipe was not present. The smaller (shorter) base of the trapezoidal wedge is equal to the width of the application of load, whereas the larger base is developed on the level of the crown of the flexible soil-bridge structure. Since the geometry of the structure has plane of symmetry that coincides with the plane of symmetry of the load, the deformation of the system is also symmetrical.

The shallow soil cover, $h/S = 0.10$, does not allow for the development of the deformation directed upwards which typically appears on both sides of the applied load when the soil is investigated for the bearing capacity. The application of one layer of geogrid has two-fold effect. It allows for an increase of the value of bearing capacity, which requires larger deformations. Therefore, in this case, for shallow soil cover, the deformation on both sides of the load applied is directed upward, whereas in the area below the applied load, the soil moves downward. It is also worth noting that for a circular soil-steel bridge with shallow soil cover the considerable deformation of variable size and value are developed around the circumference of the bridge structure. Due to the flexibility of the pipe, the deformations are extended to both sides of the soil.

The increase of soil cover has beneficial effect on the behaviour of the soil-steel bridge system, in terms of protection. The failure of soil over the pipe crown is seen in the expansion of the trapezoidal wedge (marking the greatest vertical displacements), highlighted in blue. For the soil cover of $h/S = 0.20$, the developed trapezoidal wedge is more than for the case with $h/S = 0.10$. It is worth noting that failure of soil over the soil-
bridge system with cover h/S = 0.20 requires the more advanced development of
deformation around the circumference of the flexible pipe.

As previously stated, placement of a single layer of geogrid over the circular
soil-steel structure results in the increase of bearing capacity of the system that is
achieved with larger deformations of the soil above the pipe. The advancement of
deformation of the soil during failure refers to larger movement of the soil upwards at the
soil surface and downwards below the applied load (over the crown of the pipe). It is
important to note that for h/S = 0.20 and placing two and three layers of geogrid, the
failure of the soil requires development of large deformation of soil over the crown,
whereas the remaining soil is subjected to uniform compression. Therefore, it is fair to
say that for this case the two and three layers of geogrid basically increase the stiffness of
the entire soil-steel bridge.

The failure of the unreinforced soil-bridge system for h/S = 0.25 is distinctively
different than the two previous cases. The difference refers to the clearly defined
mechanism of failure, which consists of a well-developed trapezoidal wedge failure over
the crown of the flexible pipe, whereas the remaining soil is subjected to very uniform
compression. The placement of the first layer of geogrid does not change the pattern of
failure. Upward movement of the soil develops on either side of the loaded area. The
deformation has well-localized character leaving the deformation of the rest of the soil in
homogeneous compression. The placement of the second and third layer of geogrid
results in the distribution of some portion of vertical deformation between the soil-
geogrid composite layers. It is then observed that the composite layers transfer the load
to the lower part of the soil, on either side of the flexible pipe, becoming more
compressed and deformable.

A further increase of depth of soil cover to the value of h/S = 0.30 intensifies the
behaviour of the circular soil-steel bridge system that was observed and discussed for h/S
= 0.25. The similar conclusions regarding the placement of geogrid and their effect on
the failure and associated deformations drawn during discussion of the case h/S = 0.25
can be extended to the soil-bridge system with soil cover h/S = 0.3.

Horizontal displacements generated by the rigid strip load pushing downwards
up until soil failure are shown in Fig. 7.2.2 b) to 7.2.17 b). It is interesting to note that the
soil moves horizontally above a large-diameter flexible pipe for unreinforced and reinforced (N = 1) soil in the same fashion as the homogeneous half-space. This is true for all depths of cover that have unreinforced soil and soil that is reinforced with one layer of geogrid. The soil displaces in the form of a wedge that touches the shoulders of the conduit and proceed outwards away from the load. From this observation, it can be stated that the flexible pipe plays a significant role in the development of horizontal displacements due to loading. However, the pipe plays less of a role when a greater number of geogrid layers are placed in the soil cover. Again, identical to the homogenous half-space, when the number of geogrid layer is two or greater, the geogrid restricts the horizontal movement of the soil, where large horizontal displacements may only typically occur directly above the first geogrid layer, to either side of the rigid strip load at the soil surface.

In the case of shear stresses in the soil, there is a critical combination of vertical and horizontal displacements occurring above the pipe crown due to the rigid strip load that eventually causes failure in the soil. The shear stresses are depicted in the contour plots of S12, in Fig. 7.2.2 c) through 7.2.17 c). Identical to the homogeneous half-space, it appears that the maximum shear stress in the soil typically develops approximately 0.6 m to 1.0 m below the soil surface, regardless of depth of cover or number of geogrid layers. It can be concluded that although the large-diameter flexible pipe decreases the amount of bearing capacity of the soil due to the large void in the soil, when compared to the half-space, the location of the maximum shear stress in the soil due to the rigid strip load does not change. What the pipe also limits, besides the bearing capacity of the soil, is the development of large loops of shear stress in the soil, evident in all cases of unreinforced and reinforced soil of the half-space, outlined in the discussion of foundation engineering [10]. Therefore, it can also be concluded that though the location of the maximum of shear stress in the soil at failure may not move significantly in comparison with the half-space, the full development of shear stress capacity in the soil is certainly restricted due to the presence of the large-diameter flexible pipe.
7.3. **Factors of Safety of Concentrically Loaded Circular Soil-Steel Bridge with Unreinforced and Reinforced Soil Cover**

It is important to remind the reader at this point that in the analysis of the geogrid-reinforced circular soil-steel bridge, the effective thickness of the truss element, which models the geogrid, is the same as determined in the homogeneous half space, where the effective thicknesses for each layer is presented in Fig. 5.4.2.

Firstly, as depth of soil cover increases, the factor of safety, defined by the failure load of the system divided by the applied truck strip load of 116 kN/m², also increases. This is attributed to the fact that a larger soil cover provides a greater degree of protection than a shallow cover. Factors of safety for varying depths of cover are presented in tabular format in Table 7.3.1, whereas they are plotted in Fig. 7.3.1. Identical to the homogeneous half-space, reinforced soil generates a greater factor of safety than unreinforced soil, and with increasing geogrid layers; the factor of safety also increases. For the case of shallow depth of cover, h/S = 0.10, increase of factor of safety from reinforced to unreinforced soil is not greatly significant. This may be equally stated of soil with h/S = 0.20, for the larger number of geogrid layers, N = 2 and N = 3. However, when the depth of cover above the pipe is extensive, h/S = 0.25 and 0.30, the geogrid layers play an important role in significantly augmenting the factor of safety. This may be described by the fact that because the soil depth is large, the reinforced soil has a greater stiffness, similar to the homogeneous half-space.

70
Table 7.3.1 Factors of Safety for Unreinforced and Reinforced Depth of Cover

<table>
<thead>
<tr>
<th>h/S</th>
<th>N</th>
<th>Factor of Safety $q_r/q_a$ &amp; $q_u/q_a$</th>
</tr>
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<td></td>
<td>3</td>
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Figure 7.3.1 Factors of safety for unreinforced and reinforced soil cover, under varying depths of cover, for concentrically load circular soil-steel bridge
CHAPTER 8
RESULTS OF THE ANALYSIS OF ECCENTRICALLY LOADED CIRCULAR SOIL-STEEL BRIDGE UNDER UNREINFORCED AND REINFORCED COVER

Results connected to the analysis of the application of an eccentric rigid strip load to a circular soil-steel bridge with unreinforced and reinforced cover are much more extensive than the results connected to the application of a rigid concentric strip load to the circular soil-steel bridge, and even more extensive than the results connected to the application of a rigid strip load to an unreinforced and reinforced homogeneous half space. For example, for each depth of cover above the pipe, \( h/S = 0.10, 0.20, 0.25, 0.30 \), and specified number of geogrid layers, \( N = 0 \) to \( 5 \), there are four separate eccentric loads to apply, \( 2e/S = 0.2, 0.4, 0.6, 0.8 \). In addition to the large quantity of load cases, sixty-four to be exact, failure may not specifically occur in the soil cover above the crown of the pipe. Additionally, the question arises regarding the critical eccentricity of load, defined by the lowest factor of safety for \( 2e/S \), for each depth of cover, and each number of reinforcement layer in the soil cover over the flexible pipe of 7.6 m diameter. The soil behaviour results are presented in Section 8.1, while a general discussion of the results of soil behaviour under eccentric load over the circular soil-steel bridge follows in Section 8.2. Due to the large quantity of load cases, the unreinforced and reinforced cases of soil cover above the pipe are discussed separately in their respective sections, 8.3 and 8.4.

8.1. Presentation of Results Depicting Soil Behaviour in Unreinforced and Reinforced Cover of Eccentrically Loaded Circular Soil-Steel Bridge

Soil cover behaviour above the soil-steel bridge at failure, defined by vertical and horizontal displacements of the soil, as well as the shear stresses in the soil, is crucial at enhancing the discussion of the factor of safety. Figures 8.1.1 to 8.1.64 depict the three components of soil behaviour for the eccentrically loaded soil-steel bridge having increasing depths of cover, varying layers of geogrid, and varying eccentricities of load. A close-up portrayal of the soil behaviour between the soil surface and the pipe crown is displayed for this is the area of most interest. The discussion of the results of soil behaviour follows in the next three sections.
Figure 8.1.1 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.10, eccentrically loaded, 2e/S = 0.2
Figure 8.1.2 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.10, eccentrically loaded, 2e/S = 0.4
Figure 8.1.3 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.10, eccentrically loaded, 2e/S = 0.6
Figure 8.1.4 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of unreinforced soil over circular soil-steel bridge, h/S = 0.10, eccentrically loaded, 2e/S = 0.8
Figure 8.1.5 Distribution of vertical (U2) and horizontal (U1) displacements and shear stresses (S12) of reinforced soil, N = 1, over circular soil-steel bridge, h/S = 0.10, eccentrically loaded, 2e/S = 0.2
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8.2. General Discussion of Results of an Eccentrically Loaded Circular Soil-Steel Bridge with Unreinforced and Reinforced Soil Cover

As seen in the distribution of vertical and horizontal displacements and shear stresses in the soil in Fig. 8.1.1 to 8.1.64, moving the strip load along the soil surface of the soil-steel bridge with variable depth of soil cover encounters considerable complexities. The complexity of the static analysis of the system investigated is associated with the fact that the axis of symmetry for geometry of the system and the axis of symmetry of the moving load do not coincide. The finite width of the moving load, 1.2m, when vertically projected on the corresponding segment of the flexible bridge shell, encounters the flexible segment of soil-steel bridge structure that is inclined to the horizontal, the inclination being denoted by $\alpha_1$ and $\alpha_2$ in Fig. 8.2.1. This means that a difference exists between the depths of soil cover under two boundary points defining the width of the moving load. This fact affects the deformation of the wedge failure in the local sense, in the vicinity of the applied load, or more specifically, the shaded area labeled as the non-symmetrical area in Fig. 8.2.1. Consequently, the wedge of failure is not symmetrical in the local sense, with respect to the axis of symmetry of the load, as well as the global sense, with respect to the axis of symmetry of the global geometry.

Figure 8.2.1 Axis of symmetry of the geometry and load
Moreover, the inclination to the horizontal of each segment of the bridge shell increases when the position of the moving load from the global geometrical axis of symmetry increases. Then the projecting wedge, depicted as the shaded area in Fig. 8.2.1, demonstrates the increasing lack of symmetry with respect to the axis of symmetry of loading.

The discussed features of eccentrically loaded soil-steel bridge with soil cover of variable thickness are transferred also to the system that is reinforced by the placement of geogrid. To the knowledge of the author, no research was conducted on the behaviour of bearing capacity of soil-geogrid system over a circular soil-steel bridge that allows for the development of the nonsymmetrical wedge of failure due to variable thickness of the soil cover. The discussion of results of the behaviour of the unreinforced soil cover is found in the following section, while reinforced soil cover behaviour is discussed in Section 8.4.

8.3. Discussion of Results of an Eccentrically Loaded Circular Soil-Steel Bridge with Unreinforced Soil Cover

The depth of cover, h/S, combined with eccentricity, e/(S/2), is the decisive factors that affect the advancements of vertical and horizontal deformations associated with failure. The vertical deformations both in the global and local sense are not symmetrical. In general, the mode of failure due to eccentricity of the rigid strip load over the large-diameter circular flexible pipe is characterized by the development of a nonsymmetrical trapezoidal wedge of failure, with a median almost normal to the shell of the pipe. This means that as the value of 2e/S increases, the diversion from the vertical of the median of the trapezoidal wedge of failure also increases, as shown in Fig. 8.3.1. The situation depicted in Fig. 8.3.1 is evident in the coloured layouts of vertical displacements, U2, in all eccentric loading cases for the various depths of cover, unreinforced and reinforced, Fig. 8.1.1 a) to 8.1.64 a). In the case of horizontal displacements, Fig. 8.1.1 b) to 8.1.64 b), the location most directly affected by the rigid strip load is directly to the left of the strip load, or above the crown, in all cases of eccentricity. To the right of the rigid strip load there is evidence of horizontal displacement, but not as significant as the horizontal displacements that forms above the pipe crown. Also, there appears to be an area of very large horizontal displacement at the
shoulder of the pipe, in other words at the location where the trapezoidal wedge of the soil in Fig. 8.3.1 comes into contact with the pipe. As the eccentricity increases, it appears that the horizontal displacements at the shoulder of the pipe also tend to increase. This demonstrates a very significant area of displacement, which occurs horizontally and vertically where the trapezoidal wedge touches the pipe.

Figure 8.3.1 Trapezoidal wedge of failure normal to the pipe

The deformations associated with variable $2e/S$ for $h/S = 0.1$ show that the advancement of the irregular deformations follow the increasing values of $2e/S$ that define the departure from the global geometrical axis of symmetry. The most distant location of the moving load defined by $2e/S = 0.8$ that generates soil failure above the flexible bridge is associated with large vertical and horizontal movement of the soil on the side more remote from the crown of the bridge. It is opposed to upward deformation that can develop over the crown of the bridge during wedge failure. The deformations associated with variable $2e/S$ and $h/S = 0.2$ follow the patterns of deformations of the soil-bridge system with soil cover $h/S = 0.1$ with visible advancement of deformations.

For $h/S = 0.25$ and $0.3$, it is observed that the differences in development of deformations for $2e/S = 0.6$ and $0.8$ are attributed to the increasing depth of soil cover ($h/S$). Consequently, this fact affects the irregularity in the resulting factor of safety.

Similar to concentric loading, it is possible to deduce where shear failure in the soil occurs. Maximum shear stress in the soil, due to a rigid strip load, typically occurs at the soil-steel interface at the shoulder of the pipe. Once again reviewing Fig. 7.4.1, at the point where the right side of the trapezoidal wedge touches the corrugated pipe is the

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location most critical combination of vertical and horizontal displacements causing shear failure in the soil. Another location of maximum shear stress is at a depth of between 0.6 and 1.0 m below the left boundary defining the rigid strip load. As the rigid strip load moves further from the global line of symmetry, the locations of maximum shear stress also move further from the line of symmetry. The maximum shear stress below the left boundary moves further right with greater eccentricity, and the maximum shear stress at the soil-pipe interface moves clockwise along the pipe perimeter.

8.4. Discussion of Results of an Eccentrically Loaded Circular Soil-Steel Bridge with Reinforced Soil Cover

For small soil cover defined by the value of $h/S = 0.1$, which allows for the placement of only one geogrid layer, the presence of this geogrid does not greatly affect the deformation of the bridge-soil interaction system when compared with the unreinforced case. Basically, the placement of the geogrid increases the factor of safety of the soil-steel bridge system when the analysis is conducted for suitable parameters of the system investigated.

As the thickness of cover ($h$) and eccentricity ($2e/S$) increase, the most beneficial effect of reinforcement of the system by the placement of geogrids on bearing capacity of the system is observed for large values of $2e/S$. This is associated with larger size of deformation required to cause the failure of the system. Large vertical deformations occur when the soil cover is reinforced with 3 layers and with eccentricity of $2e/S = 0.6$ and 0.8; these cases exhibit much larger vertical deformations under a rigid strip load when compared with corresponding unreinforced cases or when compared with reinforced soil in the soil-steel bridge containing smaller number of geogrid layers.

The increased values of depth of cover-to-span ratio, $h/S$, allow for the application of the larger number of geogrids, N. The effective performance of the geogrids at the stage of failure requires the considerable increase of the size of deformations that can develop. Consequently, the large area of the soil involved becomes stronger. The increase of the soil cover to the value of $h/S = 0.25$ is of key importance in the analysis when compared with the case $h/S = 0.1$ on the performance of the circular soil-steel bridge. It is clear that the placement of even one geogrid is associated with
much larger deformations of the entire soil medium during the failure. Consequently, this fact results in a larger value of the safety factor, presented and discussed in Section 8.5. The pattern of vertical displacement/deformation is very consistent when the number of geogrid layers increase. The special case is observed for $2e/S = 0.8$ of $h/S = 0.25$, where the failure implies larger deformations that are developed around the application of the load. This fact generates a large value of $q_n$, which implies a large factor of safety.

In general, when $h/S = 0.30$ and $2e/S$ as well as $N$ are variable, the failure of reinforced soil, $q_n$, requires the development of much larger deformations than those associated with $q_n$, or the unreinforced soil failure. The mechanism or mode of failure developed for unreinforced soil-steel bridge structure turns out to be the same for the reinforced system. For this case, the insertion of even one layer of geogrid allows for the generation of larger deformations than for the unreinforced case. Consequently, the large deformation of stronger (reinforced) soil is translated into a larger value of $q_r$ and increased factor of safety. Particularly large deformations are associated with $h/S = 0.30$, $N = 3$, for $2e/S = 0.4$. For the data discussed, the pattern of deformation shows consistent traits, as seen in figures outlining the vertical displacements.

Horizontal displacements for reinforced soil demonstrate similar trend as the unreinforced soil, where the most significant displacements occur to the left of the rigid strip load, above the crown of the pipe. In addition to large horizontal displacements at the soil surface to the left of the load, there are significant horizontal displacements at the shoulder of the pipe, similar to unreinforced soil. It appears that inserting geogrid layers does not limit the significant horizontal displacements at soil-pipe interface. As eccentricity increases, so does the impact of the strip load on the shoulders of the pipe wall.

In the review of the shear stresses, inserting geogrid layers does not affect the location of maximum shear in the soil. Maximum shear stress ($\tau_{xy}$) continues to form at a depth of between 0.6 and 1.0 m to the left of the strip load, as well as at the soil-pipe interface, similar to unreinforced soil.
8.5. Factors of Safety of Eccentrically Loaded Circular Soil-Steel Bridge with
Unreinforced and Reinforced Soil Cover

Firstly, the circular unreinforced and geogrid-reinforced soil-steel bridge with
depth of cover h/S = 0.10 under eccentric loading demonstrate similar soil behaviour, as
shown in Fig. 8.1.1 and 8.1.2. Similarly, the curve of factor of safety verses eccentricity
presented in Fig. 8.5.1 for both unreinforced and reinforced soil cover are parallel to each
other, except when 2e/S = 0.8. While the single layer of geogrid suitably reinforces the
soil when eccentricity is large, it does not significantly increase the bearing capacity
when the rigid strip load is applied with a small eccentricity. The increase in bearing
capacity is very small, similar to concentric loading under shallow depth of cover.
Defining the critical eccentricity as the horizontal distance from the crown of the pipe
that generates the lowest factor of safety, in this case, the critical eccentricity is
somewhere between 2e/S = 0.4 and 0.6.

For the case of depth of cover of h/S = 0.20, reinforcing the soil cover gives the
impression that the geogrid plays a greater role in increasing the bearing capacity of the
soil-steel bridge than when the depth of cover is shallow, h/S = 0.10. Thus, the factors of
safety increase to a larger proportion under eccentric loading when the soil cover is
deeper. It is interesting to note that unreinforced soil and reinforced soil with three
geogrid layers have the same critical eccentricity, 2e/S = 0.6. At the same time,
reinforced soil with one and two geogrid layers exhibit similar critical eccentricities,
between 2e/S = 0.6 and 0.8. When these values are compared with soil cover h/S = 0.10,
it is observed that as the depth of cover increases, the critical eccentricity also tends to
increase.

Increasing the depth of cover to h/S = 0.25 also demonstrates an increase in the
effectiveness of placing geogrid layers. The same may also be stated for h/S = 0.30. In
the case of h/S = 0.25, the critical eccentricity is evident when 2e/S = 0.6. At this
eccentricity, the geogrid is not too effective, because FS does not improve significantly
when N increases. At 2e/S = 0.8 for h/S = 0.25, the reverse is true, for the FS increases to
a much greater extent, proving the viability of the geogrid. For h/S = 0.30, the critical
eccentricity is 2e/S = 0.8, discounting the irregularity of N = 1 under eccentric loading
2e/S = 0.6. This irregularity is explained in the previous section regarding soil behaviour.
Figure 8.5.1 Factors of safety for unreinforced and reinforced soil cover, $h/S = 0.10$, for eccentrically loaded circular soil-steel bridge
Figure 8.5.2 Factors of safety for unreinforced and reinforced soil cover, h/S = 0.20, for eccentrically loaded circular soil-steel bridge.
Figure 8.5.3 Factors of safety for unreinforced and reinforced soil cover, h/S = 0.25, for eccentrically loaded circular soil-steel bridge
Figure 8.5.4 Factors of safety for unreinforced and reinforced soil cover, $h/S = 0.30$, for eccentrically loaded circular soil-steel bridge.
CHAPTER 9
CONCLUSIONS AND RECOMMENDATIONS

While the presentation and discussion of results has entirely focused on the soil behaviour and factors of safety, the structural component of the pipe wall has been neglected all together. Appendix B presents the axial forces (thrust) and bending moments in the pipe wall. There is significant literature that discusses the force and moment component in the pipe wall. However, the focus of this research remained completely on the soil cover of the soil-steel bridge. Numerous conclusions and several recommendations are made regarding the unreinforced and geogrid-reinforced soil cover above a circular soil-steel bridge, under concentric and eccentric loads, in the plane strain analysis.

9.1. Conclusions of Concentric and Eccentric Loading of Geogrid-Reinforced Circular Soil-Steel Bridge

The following conclusions may be drawn out of the present study of circular soil-steel bridges:

1. The finite element method is a very useful method of studying the characteristics of soil failure above large-diameter flexible pipes subjected to both concentric and eccentric strip loads, with unreinforced and reinforced soil cover.

2. Increasing the depth of cover above the circular soil-steel bridge, for both unreinforced and geogrid-reinforced soil cover, under concentric and eccentric loading, also increases the factor of safety, defined by the failure load of the soil divided by the applied strip load of the design truck, 116 kN/m².

3. Increasing the number of geogrid layers from an unreinforced soil cover to a reinforced cover over a circular soil-steel bridge exhibits increased vertical displacements of the soil to either side of the rigid strip load, due to a higher load applied at the soil surface to cause failure because of the greater stiffness, or load-bearing capacity, of the reinforced soil. The very
large vertical displacements that associate with the failure of the soil from a rigid strip load, are evident in soil cover with four or five geogrid layers and may call into question the potential loss of serviceability of the roadway.

4. Under concentric loading, trapezoidal wedge failure, defined by means of maximum vertical displacements, is evident between the rigid strip load at the surface and the pipe crown. Under eccentric loading, the trapezoidal wedge develops at a skewed angle from the load to the shoulder of the pipe.

5. Maximum shear stress ($\tau_{xy}$) in the soil cover over the soil-steel bridge, causing failure under a concentric rigid strip load, is typically located 0.6 to 1.0 m below the soil surface, directly between the rigid strip load and the crown of the pipe. For soil reinforced with two layers of geogrid or greater, 0.6 to 1.0 m depth equates to soil failure between the first and second geogrid layer. Under an eccentric strip load, the maximum shear stress ($\tau_{xy}$) is located at the soil-pipe interface, at the location defined by the right-hand side of the trapezoidal wedge denoted by maximum vertical displacements.

6. Under a concentric load, increasing the number of geogrid layers in the soil cover above the circular soil-steel bridge increases the factor of safety. However, the geogrid does not play as effective a role in increasing the factor of safety of the circular soil-steel bridge when the cover is shallow, $h/S = 0.10$, as much as it increases the bearing capacity when the soil cover is deeper, $h/S \geq 0.20$.

7. Under an eccentric load, increasing the number of geogrid layers in the cover above the flexible pipe increases the factor of safety. However, the application of an eccentric load leads to a critical eccentricity, defined as the eccentricity of load above the circular soil-steel bridge which generates the lowest factor of safety. It appears as depth of cover, $h/S$, increases, the critical eccentricity, $2e/S$, also tends to increase. Therefore, when the soil cover is shallow, $h/S = 0.10$, eccentricity plays a greater role on the soil
cover above the pipe, and eccentricity plays less of a role when the depth of cover is deeper, $h/S \geq 0.25$.

8. While improvement is apparent in the bearing capacity of the soil-steel bridge by the placement of geogrid, this by no means disqualifies the importance of uncompromised construction practices of the soil-steel bridge. The soil still requires the code-specified compaction especially in the area of contact between the soil and pipe wall as well as the soil cover.

9.2. Recommendations for Further Study

The following recommendations are made:

1. As finite element modelling becomes more technologically advanced, it would be favourable to compare the plane strain analysis employing the method of effective geogrid thickness with a three-dimensional model.

2. Since FEA demonstrated significant improvement in the soil cover by means of geogrid, it would be beneficial to carry out a study on a full-scale model.

3. Due to the number of years which have passed since the experimentation and publication of results of testing of Omar et al. using the discontinued geogrid model BX1000, it seems appropriate that up-to-date testing is required on presently available geogrid, due to the advancement of technology in the last decade.

4. The use of geogrid, due to the increase of bearing capacity of the soil, is not only productive in the construction and lifespan of the soil-steel bridge, but also is incredibly helpful in the stability and increased lifespan of a typical roadway. It is recommended that geogrid be used in the construction of roadways.
References


APPENDIX A

Typical ABAQUS Input File of Circular Soil-Steel Bridge with Reinforced Soil Cover Over, Concentrically Loaded
A1. ABAQUS Input File

The following is a typical ABAQUS input file for the purpose of explanation. For this randomly selected file, the different components of the file are discussed. The file is for reinforced (N=2) soil cover of h/S = 0.25, concentrically loaded. The input file is divided into important sections, defined and briefly summarized at the end of the input file. The mesh is displayed in Fig. 3.5.2.

SECTION 1
**************************************************
*NODE,INPUT=h25s.inp
7001, -14, 5.32
7002, -11, 5.32
7003, -8, 5.32
7004, -6.5, 5.32
7005, -5.72, 5.32
7006, -3.071, 5.32
7007, -1.309, 5.32
7008, -0.6, 5.32
7009, 0, 5.32
7010, 0.6, 5.32
7011, 1.309, 5.32
7012, 3.071, 5.32
7013, 5.72, 5.32
7014, 6.5, 5.32
7015, 8, 5.32
7016, 11, 5.32
7017, 14, 5.32
7101, -14, 4.92
7102, -11, 4.92
7103, -8, 4.92
7104, -6.5, 4.92
7105, -5.72, 4.92
7106, -2.911, 4.92
7107, -1.309, 4.92
7108, -0.6, 4.92
7109, 0, 4.92
7110, 0.6, 4.92
7111, 1.309, 4.92
7112, 2.911, 4.92
7113, 5.72, 4.92
7114, 6.5, 4.92
7115, 8, 4.92
7116, 11, 4.92
7117, 14, 4.92
*NSET,NSET=LEFT,GENERATE
2001, 5001, 100
*NSET,NSET=RIGHT,GENERATE
2081, 5081, 100
*NSET,NSET=BOTTOM,GENERATE
2001, 2081, 1
SECTION 2

*****************************************************************************

**Define Pipe elements:
*ELEMENT,TYPE=B22,ELSET=PIPE
  1, 1, 2, 3
  55, 55, 56, 1
*ELGEN,ELSET=PIPE
  1, 27, 2
*BEAM SECTION,SECTION=RECT,ELSET=PIPE,MATERIAL=STEEL
  1, 0, 0.28
*****************************************************************************

**Define soil elements:
*ELEMENT,TYPE=CPE8,ELSET=SOIL
  101, 101, 103, 303, 301, 102, 203, 302, 201
  155, 155, 101, 301, 355, 156, 201, 356, 255
  355, 355, 301, 501, 555, 356, 401, 556, 455
  501, 501, 503, 703, 701, 502, 603, 702, 601
  507, 507, 509, 5049, 707, 508, 609, 708, 607
  509, 511, 4849, 5049, 509, 611, 4949, 609, 510
  511, 513, 4649, 4849, 511, 613, 4749, 611, 512
  513, 515, 4449, 4649, 513, 615, 4549, 613, 514
  515, 517, 4249, 4449, 515, 617, 4349, 615, 516
  517, 519, 4049, 4249, 517, 619, 4149, 617, 518
  519, 521, 3849, 4049, 519, 621, 3949, 619, 520
  521, 3847, 3849, 521, 523, 3848, 621, 522, 623
  523, 3845, 3847, 523, 525, 3846, 623, 524, 625
  525, 3843, 3845, 525, 527, 3844, 625, 526, 627
  527, 3841, 3843, 527, 529, 3842, 627, 528, 629
  529, 3839, 3841, 529, 531, 3840, 629, 530, 631
  531, 3837, 3839, 531, 533, 3838, 631, 532, 633
  533, 3835, 3837, 533, 535, 3836, 633, 534, 635
  535, 3833, 3835, 535, 537, 3834, 635, 536, 637
  537, 3833, 537, 539, 4033, 637, 538, 639, 3933
  539, 4033, 539, 541, 4233, 639, 540, 641, 4133
  541, 4233, 541, 543, 4433, 641, 542, 643, 4333
  543, 4433, 543, 545, 4633, 643, 544, 645, 4533
  545, 4633, 545, 547, 4833, 645, 546, 647, 4733
  547, 4833, 547, 549, 5033, 647, 548, 649, 4933
  549, 549, 551, 751, 5033, 550, 651, 750, 649
  551, 551, 553, 753, 751, 552, 653, 752, 651
  3801, 3801, 3803, 4003, 4001, 3802, 3903, 4002, 3901
  3849, 3849, 3851, 4051, 4049, 3850, 3951, 4050, 3949
*ELGEN,ELSET=SOIL
  101, 27, 2, 2, 2, 200, 200
  501, 3, 2, 2
  2001, 40, 2, 2, 9, 200, 200
  3801, 16, 2, 2, 6, 200, 200
  3849, 16, 2, 6, 200, 200
*****************************************************************************

**Define reinforcement, using truss elements:
*ELEMENT,TYPE=T2D2,ELSET=REINF1
  7001, 7001, 7002
*ELEMENT,TYPE=T2D2,ELSET=REINF2
  7101, 7101, 7102
*ELGEN,ELSET=REINF1
7001, 16, 1, 1
*ELGEN,ELSET=REINF2
70101, 16, 1, 1
**************************************************************************
**Define spring elements, connecting pipe and soil:
*ELEMENT,TYPE=SPRINGA,ELSET=N
1001 , 1 , 101
1002 , 2 , 102
1003 , 3 , 103
1004 , 4 , 104
1005 , 5 , 105
1006 , 6 , 106
1007 , 7 , 107
1008 , 8 , 108
1009 , 9 , 109
1010 , 10 , 110
1011 , 11 , 111
1012 , 12 , 112
1013 , 13 , 113
1014 , 14 , 114
1015 , 15 , 115
1016 , 16 , 116
1017 , 17 , 117
1018 , 18 , 118
1019 , 19 , 119
1020 , 20 , 120
1021 , 21 , 121
1022 , 22 , 122
1023 , 23 , 123
1024 , 24 , 124
1025 , 25 , 125
1026 , 26 , 126
1027 , 27 , 127
1028 , 28 , 128
1029 , 29 , 129
1030 , 30 , 130
1031 , 31 , 131
1032 , 32 , 132
1033 , 33 , 133
1034 , 34 , 134
1035 , 35 , 135
1036 , 36 , 136
1037 , 37 , 137
1038 , 38 , 138
1039 , 39 , 139
1040 , 40 , 140
1041 , 41 , 141
1042 , 42 , 142
1043 , 43 , 143
1044 , 44 , 144
1045 , 45 , 145
1046 , 46 , 146
1047 , 47 , 147
1048 , 48 , 148
1049 , 49 , 149
1050 , 50 , 150
*ELEMENT, TYPE=SPRINGA, ELSET=T

1051, 51, 151
1052, 52, 152
1053, 53, 153
1054, 54, 154
1055, 55, 155
1056, 56, 156

1101, 101, 2
1102, 102, 3
1103, 103, 4
1104, 104, 5
1105, 105, 6
1106, 106, 7
1107, 107, 8
1108, 108, 9
1109, 109, 10
1110, 110, 11
1111, 111, 12
1112, 112, 13
1113, 113, 14
1114, 114, 15
1115, 15, 116
1116, 16, 117
1117, 17, 118
1118, 18, 119
1119, 19, 120
1120, 20, 121
1121, 21, 122
1122, 22, 123
1123, 23, 124
1124, 24, 125
1125, 25, 126
1126, 26, 127
1127, 27, 128
1128, 28, 129
1129, 129, 30
1130, 130, 31
1131, 132, 31
1132, 133, 32
1133, 134, 33
1134, 135, 34
1135, 136, 35
1136, 137, 36
1137, 138, 37
1138, 139, 38
1139, 140, 39
1140, 141, 40
1141, 142, 41
1142, 143, 42
1143, 144, 43
1144, 145, 44
1145, 146, 45
1146, 147, 46
1147, 148, 47
1148, 149, 48
1149, 150, 49
1150 , 151 , 50
1151 , 152 , 51
1152 , 153 , 52
1153 , 154 , 53
1154 , 155 , 54
1155 , 156 , 55
1156 , 101 , 56

*****************************************************************************
**Define stiff elements at surface to model strip load
*ELEMENT,TYPE=CPE8,ELSET=STIFFELEM
701, 701, 703, 903, 901, 702, 803, 902, 801
755, 755, 701, 901, 955, 756, 801, 956, 855

SECTION 3
*****************************************************************************
**Define pipe material behaviour.
*MATERIAL,NAME=STEEL
*DENSITY
7.860
*ELASTIC
200E6, 0.3
*PLASTIC
282E3, 0

*****************************************************************************
**Define soil element behaviour.
*SOLID SECTION,MATERIAL=S,ELSET=SOIL
*MATERIAL,NAME=S
*ELASTIC,TYPE=ISO
207E3, 0.30
*DENSITY
2.141
*MOHR COULOMB
40, 40
*MOHR COULOMB HARDENING
7, 0

*****************************************************************************
**Define reinforcement (TENSA R BX1100) behaviour:
*SOLID SECTION,ELSET=REINF1,MATERIAL=TENSA R
0.00347
*SOLID SECTION,ELSET=REINF2,MATERIAL=TENSA R
0.00347
*MATERIAL,NAME=TENSA R
*ELASTIC
1.656E6, 0.49
*NO COMPRESSION
*PLASTIC
35.88E3
*DENSITY
0.9
*EMBEDDED ELEMENT,HOST ELSET=SOIL
REINF1
REINF2
**Define spring stiffness.**
*SPRING,ELSET=T

3.2E7
*SPRING,ELSET=N

1.6E8

**Define stiff element material**
*SOLID SECTION,ELSET=STIFFELEM,MATERIAL=STEEL2
*MATERIAL,NAME=STEEL2
*DENSITY
7.860
*ELASTIC
200E6, 0.3

SECTION 4

**BOUNDARY**
BOTTOM, 1, 2
LEFT, 1
RIGHT, 1

SECTION 5

**STEP**
*STATIC
0.0625, 1
*DLOAD
PIPE, GRAV, 9.81, 0, -1
SOIL, GRAV, 9.81, 0, -1
REINF1, GRAV, 9.81, 0, -1
REINF2, GRAV, 9.81, 0, -1
*END STEP

**STEP**
*STATIC
0.0625, 1
*CLOAD
901, 2, -10000
*EL PRINT,ELSET=PIPE,FREQUENCY=5
SF1, SM1
*EL PRINT,ELSET=REINF1,FREQUENCY=5
S11
*EL PRINT,ELSET=REINF2,FREQUENCY=5
S11
*OUTPUT,FIELD,VARIABLE=PRESELECT
*OUTPUT,HISTORY
*END STEP
A2. **Explanation of Input File**

The above input file is divided into five sections, and each one is briefly explained. The six sections are:

1. Node definition;
2. Element definition;
3. Material definition;
4. Boundary conditions; and
5. Loading sequence.

First, ABAQUS offers the choice of either generating the nodes of the geometry or inputting them from a text file. Either way, the node requires an identifying number, as well as the x and y coordinates, in ABAQUS, these are specified as coordinates 1 and 2 respectively. In the present case, the input file both obtains the nodes from an input file, h25s.inp, as well as specified nodes: 7001 to 7117 (these nodes are for two layers reinforcement). The set of nodes, NSET=LEFT, RIGHT, and BOTTOM are generated to form the boundary conditions that will be defined in Section 4.

Second, ABAQUS forms elements from the defined nodes from the previous section. An element number, and three node numbers identify the beam elements defining the pipe circumference. While the beam elements forming the pipe circumference are generated with the *ELGEN command, the plane strain 8-node continuum elements of the soil (CPE8) are defined both by generation and by specifying them outright. An element number and eight nodes define the 8-node continuum element. The next type of element is the truss, acting as the geogrid reinforcement. The truss element is defined by a number and two nodes, and the elements are generated by means of the *ELGEN command. Two layers of reinforcement are included in this particular analysis, therefore each are specified by the ELSET command, REINF1 and REINF2. The next element requiring definition is the spring element, which connects the pipe to the soil. Three springs are connected to each node; one normal spring connecting the three-node beam element to the eight-node continuum element, as well as two tangential springs. A spring number, as well as the beam node number and continuum element number define the spring elements, both normal and tangential. The connection of the two types of springs is better explained in Section 3.3, and more specifically in Fig. 3.3.1. The last element defined is the 1.2 m stiff continuum element, which is used to transmit a concentrated force into a strip load at the soil surface.

Third, the material properties for each element must be specified. The first material is the steel of the pipe, with a density value included in the input for the purpose of gravity loading, a modulus of elasticity (200 GPa), Poisson’s ratio (0.3), and plastic yield limit (282 MPa). The second material defined is the soil. The elasticity of the soil is specified as 207 MPa, Poisson’s ratio as 0.3, the density is 2141 kg/m$^3$, and the soil failure criterion, as mentioned in Chapter 3 is the Mohr Coulomb failure criterion. The angle of friction of 40° is specified, and angle of dilation equals the angle of friction. As previously mentioned, a cohesion value of 7 kPa is included in the analysis to avoid numerical singularity (very early and incorrect failure). The geogrid material is the next
material defined. For the case of two layers of geogrid, the effective geogrid thickness was determined as 0.00347 m²/m for each layer. The elasticity modulus is 1.656 GPa, Poisson’s ratio is 0.49 (practically incompressible), and the *NO COMPRESSION command keeps the truss element from supporting a compression force, because the geogrid in the field may not support compression forces either. The plastic yield limit of the geogrid is 35.88 MPa, and the density for gravity calculation is 0.9 kg/m³. The *EMBEDDED ELEMENT command specifies that the geogrid is a composite material of the soil, similar to steel reinforcement in concrete. This command specifies that there is no slip between the geogrid and the soil that the two materials interlock. The springs require a stiffness to be defined; as previously mentioned in Chapter 3, the normal springs have a stiffness of $1.6 \times 10^8$ kN/m, and the tangential spring is 20% of this value, $3.2 \times 10^7$ kN/m. Finally, the stiff element, which transfers the concentrated load into a strip load at the soil surface, is defined as steel, with the same material properties as the pipe wall, without a plastic yield limit.

Fourth, the boundary conditions are identified. The boundary conditions in the input file dictate where the nodes are fixed, allowing for zero displacements in the direction listed. In this case, the bottom nodes are fixed in the vertical and horizontal direction, while the right and left side of the geometry are fixed only in the horizontal direction, allowing for the settling of the soil in the vertical direction, modelling the real-world scenario.

Fifth and final is the load sequence. The loading definitions are outlined in Chapter 4, and the terms are employed here. The gravity load, *DLOAD, is applied first (step 1). The load is applied in increments at a ratio commencing with $(0.0625/1)\times$(Total Load), and then ABAQUS computes the load increments thereafter. Typically, the gravity load is applied in six or seven increments. The step ends when the full gravity load is applied, and the gravity load remains fully applied through the next load sequence, the live load in step 2. The concentrated load, *CLOAD, is applied at node 901 in the vertical (2) downward (negative) direction. The value of 10,000 kN is an overexaggerated value that if reached, the problem will not have failed due to the applied load. The first increment will be applied at $(0.0625/1)\times$(10,000), or 625 kN, and incrementally thereafter, up until failure. The command lines that follow the concentrated force command line, *CLOAD, are all commands for data output. The first output requested (*EL PRINT,ELSET=PIPE) is numerical: the axial forces and bending moments, SF1 and SM1 respectively (data for the plots in Appendix A). The second data output request is the axial stresses in the two layers of reinforcement, *EL PRINT,ELSET=REINF1. The third and final output requests, *OUTPUT,FIELD and *OUTPUT,HISTORY are in regards to the deformed and contour plots in ABAQUS CAE, where results are displayed.
APPENDIX B

Axial Force (Thrust) and Bending Moments in Pipe Wall
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall
Centric Loading, h/S = 0.10

Thrust (kN/m)
(Negative = Compression)

θ (degrees) (counter-clockwise from invert)

invert
 crown
invert

--- Unreinforced Soil
--- Reinforced, N = 1
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Centric Loading, h/S = 0.20

- Unreinforced Soil
- Reinforced, N = 1
- Reinforced, N = 2
- Reinforced, N = 3

Thrust (kN/m) (Negative = Compression)

θ (degrees) (counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Centric Loading, h/S = 0.20

Bending Moment (kN m/m)
invert 90 180 270 360

θ (degrees)
(counter-clockwise from invert)

- - Unreinforced Soil
- - Reinforced, N = 1
- - Reinforced, N = 2
- - Reinforced, N = 3
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Centric Loading, h/S = 0.25

-1700
-1500
-1300
-1100
-900
-700
-500
-300
-100
100

Thrust (kN/m) (Negative = Compression)

0 45 90 135 180 225 270 315 360

θ (degrees) (counter-clockwise from invert)

invert  crown  invert

- Unreinforced Soil
- Reinforced, N = 1
- Reinforced, N = 2
- Reinforced, N = 3
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Centric Loading, h/S = 0.25

- Unreinforced Soil
- Reinforced, N = 1
- Reinforced, N = 2
- Reinforced, N = 3

Bending Moment (kN m/m)

θ (degrees)
(counter-clockwise from invert)
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Centric Loading, h/S = 0.30

Thrust (kN/m) (Negative = Compression)

θ (degrees) (counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Centric Loading, h/S = 0.30

- Unreinforced Soil
- Reinforced, N = 1
- Reinforced, N = 2
- Reinforced, N = 3
- Reinforced, N = 4
- Reinforced, N = 5

Bending Moment (kN m/m)

θ (degrees) (counter-clockwise from invert)

invert
crown
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.10, Unreinforced Soil

![Graph showing thrust vs angle with different symbols for 2e/S values of 0.2, 0.4, 0.6, and 0.8.]
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.10, Reinforced Soil, N = 1

Thrust (kN/m) (Negative = Compression)

θ (degrees) (counter-clockwise from invert)

-600 -500 -400 -300 -200 -100 0 100 200 300

2e/S = 0.2
2e/S = 0.4
2e/S = 0.6
2e/S = 0.8
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, $h/S = 0.10$, Reinforced Soil, $N = 1$
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.20, Unreinforced Soil

Thrust (kN/m) (Negative = Compression)

θ (degrees) (counter-clockwise from invert)
invert 0 45 90 135 180 225 270 315 360

-800 -700 -600 -500 -400 -300 -200 -100 0 100

- 2e/S = 0.2
- - 2e/S = 0.4
- ▲ - 2e/S = 0.6
- O - 2e/S = 0.8

invert
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.20, Unreinforced Soil

Bending Moment (kN m/m)

θ (degrees)
(counter-clockwise from invert)
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.20, Reinforced Soil, N = 1

Thrust (kN/m)

\( \theta \) (degrees)

(counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, $h/S = 0.20$, Reinforced Soil, $N = 1$

Diagram showing bending moment vs $\theta$ (degrees), with lines for different $2e/S$ values: $0.2$, $0.4$, $0.6$, and $0.8$. The x-axis represents $\theta$ (degrees) counter-clockwise from invert, and the y-axis represents Bending Moment (kN m / m).
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.20, Reinforced Soil, N = 2

- 2e/S = 0.2
- 2e/S = 0.4
- 2e/S = 0.6
- 2e/S = 0.8

Thrust (kN / m) (Negative = Compression)

θ (degrees) (counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.20, Reinforced Soil, N = 2

- - - 2e/S = 0.2
- - - - 2e/S = 0.4
- -  2e/S = 0.6
- - - -  2e/S = 0.8

Bending Moment (kN m / m)

θ (degrees)

(counter-clockwise from invert)
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.20, Reinforced Soil, N = 3

Thrust (kN/m) (Negative = Compression)

θ (degrees)
(counter-clockwise from invert)

0 45 90 135 180 225 270 315 360
invert crown invert

- 2e/S = 0.2
- 2e/S = 0.4
- 2e/S = 0.6
- 2e/S = 0.8
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.20, Reinforced Soil, N = 3

Bending Moment (kN m/m)

θ (degrees)
(counter-clockwise from invert)

invert
crown
invert
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.25, Unreinforced Soil

[Graph showing thrust vs. angle θ (degrees)]
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.25, Unreinforced Soil

- • 2e/S = 0.2
- ■ 2e/S = 0.4
- ▲ 2e/S = 0.6
- ○ 2e/S = 0.8

Bending Moment (kN m / m)

θ (degrees)
(counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.25, Reinforced Soil, N = 1

- 2e/S = 0.2
- 2e/S = 0.4
- 2e/S = 0.6
- 2e/S = 0.8

Bending Moment (kN m/m)

θ (degrees)
(counter-clockwise from invert)
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.25, Reinforced Soil, N = 2

- 2e/S = 0.2
- 2e/S = 0.4
- 2e/S = 0.6
- 2e/S = 0.8

(crown)

(θ (degrees) counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.25, Reinforced Soil, N = 2

Bending Moment (kN m / m)

θ (degrees)
(counterclockwise from invert)
invert
crown
invert

- 2e/S = 0.2
- 2e/S = 0.4
- 2e/S = 0.6
- 2e/S = 0.8
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.25, Reinforced Soil, N = 3

Thrust (kN/m) (Negative = Compression)

θ (degrees)
(counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.25, Reinforced Soil, N = 3

Bending Moment (kN m / m)

θ (degrees)
(counter-clockwise from invert)
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, \( h/S = 0.30 \), Unreinforced Soil

Graph showing thrust in kN/m as a function of \( \theta \) (degrees) counter-clockwise from invert, with different line styles representing various values of \( 2e/S \) (0.2, 0.4, 0.6, 0.8). The graph indicates peaks and troughs corresponding to the invert and crown of the pipe.
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.30, Unreinforced Soil

Bending Moment (kN m/m)

θ (degrees)
(counter-clockwise from invert)

invert

Crown

192
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.30, Reinforced Soil, N = 1

Thrust (kN / m) (Negative = Compression)

\[ 0 \text{ (degrees)} \]
(counter-clockwise from invert)

- \( 2e/S = 0.2 \)
- \( 2e/S = 0.4 \)
- \( 2e/S = 0.6 \)
- \( 2e/S = 0.8 \)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, $h/S = 0.30$, Reinforced Soil, $N = 2$

![Graph depicting bending moment in a pipe wall versus perimeter location.](image-url)
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall
Eccentric Loading, h/S = 0.30, Reinforced Soil, N = 3
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.30, Reinforced Soil, N = 3
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.30, Reinforced Soil, N = 4

- 2e/S = 0.2
- 2e/S = 0.4
- 2e/S = 0.6
- 2e/S = 0.8

Thrust (kN/m) (Negative = Compression)

θ (degrees) (counter-clockwise from invert)
Axial Force (Thrust) in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, h/S = 0.30, Reinforced Soil, N = 5

Thrust (kN/m) (Negative = Compression)

θ (degrees) (counter-clockwise from invert)
Bending Moment in Pipe Wall vs Perimeter Location of Pipe Wall

Eccentric Loading, $h/S = 0.30$, Reinforced Soil, $N = 5$
Vita Auctoris

<table>
<thead>
<tr>
<th>Name:</th>
<th>André Bom</th>
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<tbody>
<tr>
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<td>Louisville, Kentucky, U.S.A.</td>
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<td>Year of birth:</td>
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