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Investigation of the Facing Response of Soil Nailed Excavations

Thesis submitted for the degree of Doctor of Philosophy
The University of Sydney

October 2004

by

Andrew de Ambrosis
Abstract

This thesis details work conducted to investigate the behaviour of facing elements in soil nailed excavations.

The body of research current at the time of writing is reviewed, with particular emphasis placed on the historical ‘evolution’ of the perceived role of the facing. A knowledge gap pertaining to the size of the loads experienced by the facing is identified and a program of work proposed to address the shortfall in understanding.

The development and validation of a three-dimensional finite element program, capable of directly calculating the response of facing elements to the construction of soil nailed excavations, is detailed. A full-scale experimental soil nailed wall, constructed as part of The French National Research Project Clouterre, is modeled using the program and comparisons between observed and calculated behaviour are presented and discussed in the context of other simulations of the same project.

A parametric study is conducted targeting factors that were, either shown by existing research to be of significance or revealed during the programming process to influence the calculations. In general the results confirm the current understanding regarding facing behaviour. Notwithstanding, it is shown that there is a strong theoretical basis for the argument that construction related issues can have a significant impact on the behaviour of the facing. In light of this, a number of possible indicators as to the impact of construction processes are discussed, and in conclusion a program of further research exploring these issues proposed.
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Initstr
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$A_C$  FHWA-SA-96-069R (1998) – Area of the base of the failure cone (Fig. 2.13).
$A_{GC}$  FHWA-SA-96-069R (1998) – Area of the nail grout column (Fig. 2.13).
$A_{HS}$  FHWA-SA-96-069R (1998) – Area of the body of the headed stud (Fig. 2.13).
$A_s$  Cross-sectional area of tensile reinforcement.
$\mathbf{a}$  Nodal displacement vector.
$B$  Displacement strain matrix.
$\sigma_u$  Undrained cohesive strength of the soil.
$D$  Incremental stress-strain matrix.
$D_{eo}$  Incremental stress-strain matrix corrected for elasto-plastic behaviour.
$E$  Young’s Modulus.
$E_b$  Young’s Modulus of elasticity of the tensile reinforcement.
$E_f$  Young’s Modulus of elasticity of the facing material.
$E_m$  Young’s Modulus of elasticity of soil obtained using a Menard Pressuremeter.
$E_n$  Young’s Modulus of elasticity of the tensile reinforcement.
$E_s$  Young’s Modulus of elasticity of the soil.
$FS$  Global Factor of Safety.
$FS_\phi$  Factor of Safety applied to the frictional component of soil shear strength.
$FS_c$  Factor of Safety applied to the soil cohesive strength.
$f'_c$  Characteristic compressive strength of concrete after 28 days.
$f_y$  Tensile yield stress of reinforcement.
$H$  Height of excavation.
$K_a$  Active earth pressure co-efficient.
$K_o$  Co-efficient of lateral earth pressure at rest.
$I$  Length of tensile reinforcement.
$M_{xx}$  Bending moment around the X axis (Fig. 5.7).
$M_{yy}$  Bending moment around the Y axis (Fig. 5.7).
$M_{v,peg}$  Vertical nominal unit moment resistance at the nail head.
M_{v,\text{pos}} \quad \text{Vertical nominal unit moment resistance at the mid-span location.}

N \quad \text{Shape functions.}

p \quad \text{Earth pressure acting at the facing.}

R_c \quad \text{Clouterre, (1991) - Capacity of nail in pure shear.}

R_n \quad \text{Clouterre, (1991) - Capacity of nail in pure tension.}

S \quad \text{Clouterre, (1991) - Maximum of } S_H \text{ and } S_V.

S_H \quad \text{Horizontal nail spacing.}

S_V \quad \text{Vertical nail spacing.}

T_c \quad \text{Clouterre, (1991) - Shear force induced in nail.}

T_f \quad \text{Structural tensile capacity of the reinforcement.}

T_{FN} \quad \text{FHWA-SA-96-069R (1998) - Strength of nail-facing connector.}

T_{\text{max}} \quad \text{Clouterre, (1991) - Maximum allowable nail tensile force.}

T_n \quad \text{Clouterre, (1991) - Tensile force induced in nail.}

T_N \quad \text{FHWA-SA-96-069R (1998) - Nail tendon tensile strength.}

T_o \quad \text{Clouterre, (1991) - Tensile force acting at the nail head.}

t \quad \text{Cross-sectional thickness of facing.}

\mathbf{t} \quad \text{Traction vector.}

V_N \quad \text{FHWA-SA-96-069R (1998) - Internal facing component of resistance.}

W \quad \text{Weight (force) of soil wedge.}

\gamma \quad \text{Bulk unit weight of material.}

\gamma \quad \text{Body force vector.}

\delta \quad \text{The angle of friction between the soil and the retaining wall.}

\nu \quad \text{Poisson's Ratio.}

\sigma \quad \text{Internal stress vector.}

\sigma_n \quad \text{Normal stress.}

\sigma_v \quad \text{Vertical stress.}

\phi \quad \text{Angle of internal friction of soil.}

\phi' \quad \text{Effective Angle of internal friction of soil.}

\psi \quad \text{Angle of soil dilation upon shearing.}
Chapter 1: Introduction.

1.0 Introduction

The temporary and permanent retaining of earth represents an integral part of a wide range of engineering projects. As such, it is an area that has seen continual innovation as emerging technologies and increased understanding allow the construction of increasingly efficient, cost effective and reliable support systems.

A brief introduction to various aspects of reinforced earth retaining systems is presented in this chapter. The development of a variation of reinforced earth, referred to as soil nailing, is outlined and the state-of-the-art for design and construction of this method is presented.

1.1 Reinforced earth-retaining structures

The term, “reinforced earth” refers to an earth retaining system, which seeks to stabilise an excavation through the placement of untensioned/passive inclusions (typically steel bars or geofabric sheets) throughout the soil adjacent to the face. Figure 1.1 shows a typical reinforced earth system. Three main elements are shown:

- The reinforcing elements: these are relatively rigid structural elements, typically steel strips, geofabric sheets or polymer grids which are placed throughout the fill immediately adjacent to the wall. The action of these inclusions is to create a relatively flexible monolithic ‘reinforced earth’ mass, which helps retain the adjacent soil.
• The facing panels: facing elements are structural members attached to the reinforcement via a pin or connector system. They are used in order to prevent sloughing and erosion of the soil between the inclusions. Numerous facing systems exist, (precast concrete, cast in place concrete, welded wire grids, gabions, geofabric sheets, steel panels, soil filled car tyres) with the most common being precast concrete panels.

• The ‘mechanically stabilised fill’: literally the soil/fill being reinforced by the inclusions. Generally, a well graded granular material with a low fines content is used. Whilst such good quality material may not always be required in terms of soil inclusion friction, ease of compaction and free draining characteristics associated with such granular materials often mean it is used.

![Diagram of Facing panels and Original surface level.](image)

Figure 1.1: Typical reinforced earth embankment.

The idea of increasing the strength of a soil by placing relatively inextensible passive inclusions within it is not a recent one. Evidence of this type of support system has been found in ancient Roman wharfs and fortifications. In fact, construction of this type can even be found in works from the 5th and 4th millennia BC, (Jones, 1996). That said, construction of this type from that era was invariably reliant on intuitive and empirical design.

The invention of the modern reinforced earth (RE) retaining system is commonly credited to French engineer Henri Vidal in 1963. Vidal built upon previous systems (such as the crib wall and Coyne’s multi anchored ladder wall) to develop the first technique that relied solely upon frictional forces developed along the entire length of
the reinforcement in order to create a mechanically stabilized earth block capable of retaining adjacent soil.

The first wall using Vidal’s system was built in France in 1965. Extensive research projects were conducted through the late 60’s and 70’s including a French project conducted through the Laboratoire Central des Ponts et Chaussées in connection with Henri Vidal (Barcot and Lareal, 1973 and Binquet and Carlier, 1973) and another UK project conducted by the U.K. Transport Road Research Laboratory (Boden et al, 1977 and Murray and Irwin, 1981). Schlosser (1990) singles out three major milestones that led to the widespread use and acceptance of RE technology:

- The use of galvanized steel strips and facing. Early research indicated fiberglass reinforced plastics, aluminum and stainless steel were inappropriate for use as long term reinforcing elements because of unacceptable corrosion when embedded in soil. Galvanised steel was proposed and is still widely used today.

- The development of standard precast concrete facing panels. These panels gave the RE retaining system an economical, relatively flexible, aesthetically pleasing facing element that was easily adapted to varying geometries.

- The development of ribbed strips. The use of ribbed strips was introduced in 1975 in order to utilise the beneficial effects of restrained dilatancy. Volume increases in the soil caused by shearing related to lateral displacements of the reinforcements results in increased normal forces and subsequently increased frictional forces between the soil and inclusion.

Understanding how a RE system acts to retain soil required a change in the recognised role of excavation support members. Pre-existing technology, (gravity retaining walls, cantilever walls, anchored walls.) relied upon a structure, commonly the facing, to collect/concentrate the forces induced by excavation. These collected forces, were then restrained by some external means, (the weight of the wall, moment in the embedded section, tension in the anchor.) In contrast, reinforced earth systems actively strengthen the soil mass adjacent to the excavation.

The present state of the art focuses on two not entirely independent analogies in order to explain the support mechanisms involved. Commonly accepted failure mechanisms
mirror these dual analogies through the division of considered failure mechanisms into two separate groups, namely internal modes of failure and external modes of failure. Figure 1.2 presents some of these commonly accepted failure mechanisms.

<table>
<thead>
<tr>
<th>External Modes of Failure</th>
<th>Internal Modes of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding</td>
<td>Pullout of resistant reinforcing</td>
</tr>
<tr>
<td>Slip</td>
<td>Breakage of reinforcing</td>
</tr>
<tr>
<td>Bearing</td>
<td>Pullout of active reinforcing and punching failure of facing</td>
</tr>
<tr>
<td>Rotation</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1.2: Commonly accepted failure mechanisms.

As can be seen, the external modes of failure consider the stability of an integral reinforced soil block acting under the influence of the adjacent unreinforced soil. Simple gravity retaining wall modes of failure apply (bearing, sliding, overturning and rotational slip). For these cases, the action of the reinforcement is simply considered to be the generation of a stable integral block of reinforced soil. The theoretical basis for this assumption can be found in the work of Bassett and Last (1978), who presented a general stress-strain theory (Figure 1.3) which illustrated how relatively inextensible horizontal bars/inclusions, would alter the stress-strain patterns in the reinforced zone. The effects of these changes are twofold:

- Firstly, they lead to a change in the preferred failure path. This phenomenon occurs, because the inclusions restrain deformation in the soil parallel to their
axis. This results in a change in orientation of the zero extension trajectories (α and β lines, Fig.1.3 (b)), and thus changes the orientation of potential slip or rupture planes. By retarding the formation of classical Mohr-Coulomb failure planes, the inclusions help form a stable reinforced earth block.

(a) Without reinforcing  (b) With reinforcing

Figure 1.3: Potential failure planes with and without reinforcing (Bassett and Last, 1978)

- Secondly, the restraining of deformation stops the reinforced soil rupturing. Figure 1.4 shows a Mohr-Coulomb yield surface for a cohesionless soil. Before excavation commences, the soil stress state is described by circle 1. The lateral stresses are related to the overburden stresses $\sigma_v$ by the at rest coefficient of lateral earth pressure $K_o$. In an unreinforced soil, the excavation of the adjoining soil effectively removes any lateral restraint. Consequently, the soil deforms laterally and the horizontal stress decreases to the active case (circle 2). This decrease in lateral stress causes yielding of the soil. In a reinforced soil, the excavation of the adjoining soil does not result in the same lateral strains. The relatively inextensible reinforcements act to restrain the soil deformation along their axes. This in turn retards the development of active lateral stresses in the soil, and stops the soil from yielding. In terms of the Mohr's circle, the inclusions have stopped the movement from circle 1 to circle 2, restricting the stress state to one in between the two.
Figure 1.4: Mohr-Coulomb Failure surface, with soil stress states before and after excavation.

In contrast, the internal modes of failure consider the stability of the reinforced soil acting under the influence of its own self weight. Schlosser (1971) showed that the earth mass behind a reinforced earth wall could be divided into two distinct zones, the active and resistant (Figure 1.5). The active zone represents the failed soil wedge, as this soil is trying to fall into the excavation. The soil in the resistant zone is simply the remaining stable soil. This soil is not trying to move into the excavation. The inclusions used in reinforced earth walls form a bridge between the two zones. Bars, anchored in the stable resistant zone, provide predominately tensile forces that act to support the soil in the active zone.

Figure 1.5: Location of Active and Resistant Zones.

Figure 1.2 depicts three modes of internal failure, namely:

- Pullout of resistant reinforcing. In this mode, the wall fails because the intact reinforcements are pulled out of the soil in the resistant zone.
• Breakage of reinforcing. Here the frictional forces developed in the reinforcing are greater than its structural capacity.

• Pullout of active reinforcing and failure of facing. In this instance, failure occurs when the intact reinforcements are pulled through the facing and the soil in the active zone.

It is important to note that whilst the internal modes of failure in Figure 1.2 depict retaining walls in which the specified failure mode has occurred to all the reinforcement in the wall, the probability of one failure mode occurring over another depends largely on the reinforcement embedment length in either the active or resistant zone. For a retaining wall with uniform reinforcement lengths, the length of embedment in either zone varies depending upon the depth within the fill. Consequently, it is likely that a real wall will display a combination of these failure modes. As such, the three internal failure modes depicted in Figure 1.2 should be considered possible for each layer of reinforcement in a wall.

![Diagram of Section A, Section C, and Section B with a shaded area indicating the shear zone.](image)

Figure 1.6: Conjugate reinforcing effects.

Likewise, whilst Figure 1.2 makes a clear definition between internal and external failure models, full-scale wall failures have suggested a combined internal/external failure needs to be considered (Schlosser, 1992). Such a case is illustrated in Figure 1.6. In the area where the reinforced zone and the active zone coincide (section A), the bars have facilitated the formation of a stable reinforced earth block. This block acts as a normal retaining wall. Where the potential failure plane cuts the reinforcements (section B), friction between the soil and the reinforcements develops tensile forces within the bars. These tensile forces act to restrain the active zone. Both
the internal and external support mechanisms (as described above) are utilized in such a situation.

Figure 1.7 shows the construction process for a reinforced earth wall built on sloping ground. For the depicted case, the construction process requires all the existing insitu soil in the reinforced zone to be excavated prior to construction of the remainder of the wall. The wall is then built incrementally from the base of this excavation. First, a layer of compacted fill is constructed to the height of the bottom reinforcement layer. The reinforcement is then laid out on the surface of the fill and attached to temporarily supported facing panels. This reinforcement is then buried under subsequent layers of fill and reinforcement, until the wall reaches the desired height.

![Diagram of construction process for typical RE wall.](image)

Figure 1.7: Construction process for typical RE wall.

The relative simplicity of this construction process has many advantages. The breaking down of the process into small layers and the lack of large components means large specialized equipment is not required and personnel do not need to be experienced craftsmen. In addition, the flexible nature of the system means that extensive foundations are not required to ensure tolerable deformations. This ease and flexibility of construction when accompanied by an economic advantage for walls over 4.5m in height (FHWA, 1990) have meant that reinforced earth retaining
Structures have been widely utilized and accepted. That said, the development of soil nailing (an alternative earth retaining system closely related to reinforced earth) stemmed from the need to overcome two of the major limitations of the RE system; the ‘bottom up’ construction method and the need to import suitable granular fill.

As has been mentioned, the ‘bottom up’ construction method means that in order to build a reinforced earth wall, it is often necessary to excavate insitu soil from the reinforced zone prior to construction of the wall and replace the insitu soil with a suitable free draining, high shear strength soil that is not susceptible to long term creep behavior (Figure 1.7). The potential economic disadvantages of this requirement are immediately obvious. Consider the two situations depicted in Figure 1.8. In case (a), the widening of a cutting necessitates the construction of a retaining wall. In case (b), a basement for a new building is to be constructed adjacent to an existing structure. For both cases, excavation of the reinforced zone prior to construction of the wall is going to be an issue of concern. Excess excavation and disposal of insitu soils as well temporary retainment of the face forming the back of the reinforced zone both restrict the suitability of reinforced earth in such situations.

![Diagram of hypothetical reinforced earth applications.](image)

Figure 1.8: Hypothetical reinforced earth applications.

The unsuitability of such a system is exacerbated if the works are to be carried out in soils that were suitable for reuse as mechanically stabilized fill. In the worst case, the process of excavation, stockpiling and eventual compaction could break down existing structure and cementation within the soil to such an extent that a soil, which had favorable strength properties insitu, is rendered unsuitable. These disadvantages
limited the scope of application of reinforced earth systems and in turn led to the development of soil nailed excavations.

In addition to this, there were also a number of technological factors, which led to the development of soil nailing, that need to be recognised. Around the time that soil nailing was being developed, a method of tunnel support called the New Austrian Tunnelling Method, NATM (Rabczewicz, 1964-65) was being increasingly utilized. This method uses reinforced shotcrete and rockbolting to create a flexible support system that is applied to unsupported tunnel sections shortly after excavation. The speed of construction of this system allows engineers to utilize the rock’s favourable short term stability characteristics. In soil nailing, the potential was recognized to develop a system which likewise utilised the favourable short term stability of an excavation and was thus able to provide an economical reinforced earth excavation support with a ‘top down’ construction process that uses the insitu soils. Whilst the original uses of NATM in the mid 60’s were in hard rock, by the late 60’s to early 70’s the technique was being successfully applied in highly weathered rock. This proved that passive grouted inclusions and reinforced shotcrete facings could be used to stabilize less competent ground. Without this technology and experience gained in applying NATM the development of soil nailing would have been delayed, if not postponed.

1.2 Soil Nailed support systems

Soil nailing is a term used to describe an excavation retaining system adapted from the reinforced earth retaining systems designed by Vidal in the late 60’s and 70’s and Rabczewicz’s New Austrian Tunneling Method NATM of the mid 60’s. The system was developed because it combined the benefits of a reinforced earth system with the advantages of a ‘top down’ construction process, whilst also utilizing the insitu soils.

The technique involves a stepped excavation sequence (Figure 1.9), with passive reinforcements being constructed into the newly excavated insitu soil prior to commencement of the subsequent excavation steps. Deformation of the soil induces predominantly tensile forces within this reinforcement, which act to both stabilise the
retained soil and prevent rupture of the reinforced soil. This results in the formation of an intact flexible monolithic reinforced soil retaining wall.

(a) Proposed excavation
(b) Excavation divided into steps
(c) Soil from first step excavated
(d) Reinforcement placed for current step
(e) Subsequent step excavated
(f) Subsequent reinforcement placed
(g) Cycle repeated
(h) Completed excavation

Figure 1.9: Soil nailed excavation construction sequence.

The following steps describe the process in detail:

i. Step one: Excavate a section of soil (approx. 1-2m depth). Initially this section of the face is unsupported and so the exact height and width of the layer excavated is determined by the short term stand up properties of the insitu soil. In difficult ground, stand up time can be extended through the use of slotted excavations. Figure 1.10 illustrates this process. As can be seen the technique employs the use of unexcavated slots to support adjacent excavated sections. The action of the unexcavated soil helps support the excavated sections until reinforcing is placed. Once the newly placed reinforcement has gained sufficient strength, the remaining soil is excavated. Now the newly reinforced adjacent slot provides the requisite excavation support. Such techniques are particularly useful for excavations through poorer strata in otherwise favorable conditions.
The necessity for the newly excavated face to be self supporting in the short term provides one of the major restrictions on what soil conditions are appropriate for this method of restraint. Clean sands and gravels as well as soft plastic clays are generally held to be unsuitable because they lack either, the capillary cohesion, the cementation or the general strength to be self supporting for the required period. Similarly, the existence of a water table above the base of an excavation can cause some concerns in this regard.

![Diagram showing steps of slotted excavation](image)

**Figure 1.10:** Use of slotted excavation in unfavorable soil conditions.

Another major restriction on appropriate ground conditions stems from the susceptibility of certain clay soils to strength loss upon ingress of water. Soil nailed excavations are typically considered inappropriate in such soils because of the subsequent reductions in soil to nail friction and associated creep displacements.

ii.  **Step two:** Install row of soil nails. Numerous methods exist for the installation of the nails; percussion, rotary drilling, flushing and vibration or pneumatic techniques, though these techniques fall into two general categories; driven nails or drilled and grouted nails. The chosen method of installation is generally a function of the encountered ground conditions, the design life of the structure (temporary or permanent), the nail dimensions and the contractor's preferences.

Driven nails (either pneumatic, vibratory or percussion) are typically reserved for relatively short (to approximately 6m) inclusions in dense gravels and sands. Difficulties incorporating corrosion protection (e.g.: a grout annulus) means that
driven nails are almost exclusively reserved for temporary support systems (design life up to two years, Gässler 1990). Driven nails are generally steel angles (50mm x 50mm x 5mm) or steel bars and tubes. Typical nail lengths for a vertical faced structure are 0.5 to 0.7 H, where H is the height of the excavation with nail spacing typically of the order of 1 to 2 nails per square metre (Recommendations Clouterre, 1991).

Drilled and grouted nails are the most widely utilized nail type. Generally speaking, drilled and grouted nails are utilized in situations requiring longer nails. Typical lengths for a vertical faced excavation are 0.8 to 1.2 times the height of the excavation. Nail spacings are also typically larger than those used for driven nails, with normal nail densities ranging from 2.5m² to 6m² per nail. It is important to note that for spacings greater than 6m² per nail, it is commonly held that the actions of the reinforcements will not lead to the creation of a monolithic reinforced soil block. As such, normal analysis techniques are most likely inappropriate for such nail densities (Recommendations Clouterre, 1991).

Holes for drilled and grouted nails are advanced using a wide range of technologies including; solid and hollow flight augers, vibropercussion and rotary drilling. The use of drilling mud is not recommended as difficulties flushing the mud can lead to caking of the hole surface. This results in reduced soil to inclusion friction. Similarly, water flushing of drill spoil can lead to softening of the adjacent soils in certain ground conditions. Casing can be utilized in collapsing soils.

Grouting is typically conducted from the bottom of the hole via a grout tube or hollow flight. This is done in order to ensure the risk of entrapped air/voids in the grout annulus is minimized. Nail centralisers are employed to ensure adequate cover for the steel bar.

Permanent support systems almost exclusively utilise drilled and grouted nails. This is because the construction process of drill and grouted nails easily incorporates the placement of suitable corrosion protection for the steel bars (e.g. grout) and minimizes damage to bar coatings (e.g. epoxy coating). Figure 1.11
shows a conventional double grout encased nail system that is typically used for corrosion protection in aggressive ground or for critical structures. As is shown, a corrugated plastic sheath pipe separates the two, grout annuli. The inner annulus is typically grouted prior to transportation to site. This allows strict control of the procedure and gives maximum confidence in the integrity of the innermost annulus. The outer annulus is placed insitu, with due care being required to ensure void minimization. In non-aggressive ground, an epoxy coating with a minimum thickness of 0.3mm and a minimum of 25mm grout cover are considered sufficient corrosion protection (FHWA-SA-96-069R).

![Diagram of double grout encased nail system]

**Figure: 1.11: Double grout encased soil nail.**

iii. Step three: Stabilise newly exposed face. Soil nailing was originally developed using a steel mesh or fibre reinforced sprayed concrete facing. The reason sprayed concrete was used, was because its speed of construction minimized the time the newly excavated face was left unsupported. Predominately aesthetic considerations have led to the development of alternative facing systems including precast and cast insitu concrete and other prefabricated facing panels. It is worth noting that these alternate systems almost exclusively include a thin provisional shotcrete layer that is applied soon after excavation of the face as means of providing adequate short term support by helping preserve the soil moisture and the subsequent apparent cohesion. This construction layer is then later incorporated into the final system.

Typical facing thicknesses are 100-250mm. A steel nail plate (typically 150 x 150 x 10mm or 200 x 200 x 15mm) is used to connect the facing to the nails (Gässler 1990).
iv. Repeat steps i-iii, until excavation has reached desired depth.

NB: Steps ii and iii are largely interchangeable. Differing nail installation methods sometimes dictate which is done first. Driven nails, may be placed in an exposed face, in order to avoid having to drive through the shotcrete, while it is often preferred to shotcrete before drilling, as the drilling process can disturb the exposed face. As has been mentioned, in relatively permeable uncemented soils, a thin layer of shotcrete is generally applied first in order to preserve the soil moisture and the subsequent apparent cohesion.

1.2.1 Distinguishing features of a soil nailed wall

The similarities between reinforced earth and soil nailing beg the question; ‘Can we treat a soil nailed wall as a reinforced earth wall’? In answering this, it is important to recognize that whilst the support offered by a soil nailed system is intrinsically linked to the reinforced earth support ethos (and as such there are a number a design similarities) the differences in construction sequences and reinforcing elements means that a number of important variations in the behavior of the two systems exist. This section outlines some critical differences recognized by the developers of soil nailing which in turn led to the development of a separate design method.

The first major difference relates to the stresses induced by construction. As has been mentioned, soil nailed and reinforced earth systems have different construction sequences. For a reinforced wall, construction begins at the base of the wall and continues up. This means that each layer of reinforcement carries stresses induced by the placement of the overlying layers. For a soil nailed wall this situation is reversed. The wall is constructed from the top down and as such, the stresses induced in a layer of reinforcement are the result of construction of the underlying layers. The most obvious manifestation of this difference is the characteristic displacement of the two systems. Figure 1.12 shows typical displacements for the two systems. As can be seen the maximum displacement for a reinforced earth wall occurs in the lower part of the wall. In contrast, soil nailed walls have maximum displacement at the top of the wall.
These differences reflect the different working stresses in the two systems upon completion of the wall.

![Diagram](image)

(a) Soil Nailed excavation.  
(b) Reinforced earth excavation.

Figure 1.12: Characteristic displacements.

Another aspect of the system’s respective construction processes which makes the two systems unique, is the near zero stiffness of the soil nailed excavations reinforcement at the time of placement. In soil nailing, the shotcrete facing and nail grout annulus are placed wet insitu, and harden whilst the cut is still deforming. For an excavation in a soil that deforms quickly the stresses in the reinforcements will only be induced by subsequent excavation steps (as the soil will have already completed a majority of the deformation associated with the present excavation step before the grout hardens sufficiently to begin accumulating appreciable stresses). Conversely, for an excavation in soil that deforms slowly, the grout may harden before the majority of the deformation associated with the present excavation step has occurred. This means stresses in the reinforcement will be determined by the deformation of the soil (due to the present step), left to occur after hardening of the grout, plus deformation induced through subsequent excavation steps. Essentially, the zero stress position for a layer of soil nailed reinforcement is a function of the time of placement, the speed of deformation of the excavation and the curing time of the concrete. This is not the case for reinforced earth where prefabricated panels and ungrouted strips are used. Again, this difference manifests itself in the characteristic displacements of the two systems. Because stresses in a soil nailed wall are a function of the time of placement, the speed of deformation of the excavation and the curing time of the concrete, then so are the final wall displacements. This dependence was
recognized early in the development technique, as is evidenced by the numerous warnings on the importance of strict construction control.

The relative bending stiffness of the inclusions used also provides an important point of difference between the two systems. The alignment of the reinforcements in both walls means that soil displacements occur both axially and normal to the reinforcement. In reinforced earth, the reinforcements are thin strips, meshes or sheets. The resistance they offer to soil displacements normal to the reinforcement is minimal. For a soil nailed excavation, the reinforcements are steel bars (typically 25mm to 35mm in diameter) encased in a grout annulus (typically 100mm to 300mm diameter). This inclusion has a significant moment capacity, and as such the opportunity to oppose normal as well as axial soil displacements exists. As will be shown later there is still substantial debate as to whether the ‘dowel effect’ of soil nails is significant enough to warrant its inclusion in design. That said, the possibility for its inclusion was recognised early on (Schlosser and Juran, 1979 highlight three reinforcement actions provided by soil nails; solely traction, solely shearing and a combination of both) and as such was one of the factors leading to separate design methods.

Finally, the reinforcement of in-situ soils meant that design methods for a soil nailed wall should be able to incorporate some cohesive strength. For reinforced earth systems, which are constructed in granular fill, soil cohesion could be disregarded.

These variations meant that the analysis techniques developed for reinforced earth needed to be reassessed with regard to their applicability to soil nailing. Section 1.2.2 follows the historical development of soil nailing, culminating in the present state-of-the-art for design.

1.2.2 Evolution of Soil Nailing

In Section 1.1 mention is made of the role the New Austrian Tunneling Method (NATM) played in developing the appropriate expertise for the construction techniques employed by soil nailing. This kinship is illustrated by the major role
contractors, experienced in the application of NATM in poor ground, played in the development of soil nailing. In fact, the driving forces behind the majority of the early applications of soil nailing as excavation support were contractors with such expertise. As such, the story of the development of the soil nailing technique is in some ways unique, in that it begins with reports on the performance of constructed systems and then flows into the major research projects aimed at verifying appropriate design techniques.

The following section presents some of the key early structures and follows the development of the design state-of-the-art through the major research studies, discussions and theories.

Temporary retention of a sloped excavation in Versailles, France 1972/73 (Rabejac and Toudic, 1974, Hovart and Rami 1975, Medio et al., 1983): The first recorded application of soil nailing is widely credited to a joint venture between the French contractor Bouygues and a specialist contractor Soletanche. They applied the technique to temporarily retain a 70° cut slope in moderately cohesive Fountainbleau sand as part of a railway widening project near Versailles in 1972/73 (the sand’s cohesive strength was the result of a small percentage of fines, some weak cementation and the soil’s capillary action). The height of the finished cut varied from 6m to 22m. Reinforcement consisted of drilled and grouted nails of either 4m or 6m length (placed at a density of 2 nails/m²) with a 50-80mm thick shotcrete construction facing. A reinforced concrete wall was cast over the shotcrete in order to provide long term support.

The Versailles wall was important for the development of soil nailing, not just because it was the first recorded application of the technique, but also because it was widely held to be an outstanding success. The system physically performed well, with no movement or local instability being observed and additionally allowed substantial construction timesavings. Building upon this, Bouygues successfully used the same technique for a number of slope and excavation retention systems soon after (Medio et al., 1983). The willingness of Bouygues to apply this technique and their widespread promotion of its success was instrumental in the early establishment of the process.
Excavation of the l’Espanade des Invalides metro rail station, Paris 1974 (Medio et al., 1983): The support of a 12m high excavation for a Paris metro rail station by Bouygues in 1974 whilst only one of a number of projects completed after the success of the Versailles wall is notable because it was the first soil nailed system to utilize driven nails. Six metre long steel tubes with internal and external diameters of 40mm and 49mm respectively were driven into the newly excavated face at densities of the order of 2 nails/m². This support system differed further from pre-existing systems in that it used nails of relatively small capacity and length placed at fairly high densities. This nailing technique came to be known as the ‘Hurpinoise’ method (literally the ‘method of Hurpin’) in deference to one of its main developers M. Hurpin.

Work at this site also illustrated the importance of strict adherence to the construction processes. Substantial surface cracks were observed after the contractor started excavation of an underlying layer before the nails for the current layer had been placed. Full failure was arrested by speedy placement of the missing reinforcement, however the movements highlighted the importance of assessing the stability of the excavation during each step of the excavation process.

“Bodenvernagelung” A German research project, 1975-1980 (Stocker, Körber, Gässler and Gudehus 1979, Gässler and Gudehus 1981, Gässler 1992): In 1975 a research and development project into soil nailed excavation support was conducted by the specialist contractor Bauer Schrobenhausen in collaboration with the Institute of Soil Mechanics and Rock Mechanics at the University of Karlsruhe. The program consisted of theoretical stability analyses conducted in conjunction with a series of monitored laboratory and full scale field structures. All test structures were pushed to failure through the use of surcharge loads (static or combined static and dynamic). Figure 1.13 shows the configurations of the seven full scale field tests. Structures were constructed in cohesive, non-cohesive and mixed layered soils. The aim of the research was to verify the applicability of the soil nailed technique to excavation support, identify probable failure mechanisms and subsequently formulate appropriate analysis techniques. Bodenvernagelung was significant in that it represented the first collaborative effort between commercial and public (partial funding was provided by
the German Federal Minister of Research and Technology) entities to properly characterize the behavior of soil nailed systems.

![Diagram of test structures](image)

Figure 1.13: Full scale test structures.

(a) curvi-linear rotational wedge failure.  
(b) bi-linear translational wedge failure.

Figure 1.14: Considered failure planes

The results of the research program led to the development of a limit equilibrium design method commonly referred to as the ‘German Method’ (Stocker and Riedinger, Gässler and Gudehus 1981). Based on observations of the test structures, Gässler and Gudehus presented a curvi-linear rotational wedge failure (Figure 1.14 (a)) that they believed to always be the most critical case. They then simplified this situation to a bi-linear translational wedge failure system (Figure 1.14 (b)) asserting that the difference between the two models was ‘negligible for the practically prevailing range of $q < \rho h$’ (where $q$ is the applied surcharge load). Later work by Stocker and Riedinger, 1990 mentioned that consideration of external failure modes of the reinforced soil mass (ie sliding, overturning and overall failure) was common

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practice amongst German consultants applying the German design method, as was consideration of the stability of all short term construction phases.

![Figure 1.15: Forces acting in German method bi-linear wedge failure](image)

Figure 1.15 shows the typical force system considered by the German design method. The shearing resistance of the soil (as defined by Mohr-Coulomb’s failure criterion) is assumed to be entirely mobilized along the potential failure surface ($K_1$, $K_2$, $K_{12}$ and $Q_1$, $Q_2$, $Q_{12}$). Nail forces are assumed to be purely tensile ($N_{3+4}$).

Initially, a lump factor of safety was defined as:

$$\eta = \text{dissipative work of the forces in the slip plane and the nail forces work of the external forces}$$

However, following additional work conducted at the University of Karlsruhe from 1980 to 1983 a system of partial safety factors was proposed for the design. Such a system was in keeping with the procedures outlined in Eurocode 1 (Gässler and Gudehus, 1983).

As with other limit equilibrium stability methods, the German Method requires that results for various failure surfaces be calculated in order to find the most critical. These calculations were greatly simplified by Gässler’s assertion that the minimum

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factor of safety could ‘generally be found by only varying $\delta_1$ and keeping other inclinations fixed at $\delta_2 = 45 + \phi/2$ and $\delta_{12} = 90^\circ$’.

Gässler’s assertion that a bi-linear failure surface could be assumed to be the most critical case has been the topic of some debate. Whilst it was agreed that the bi-linear surface had been observed in the test structures, it has been postulated that these results were likely influenced by the surcharge applied to the top of the walls (Recommendations Clouterre, 1991). Gässler himself recognized this and in later works (Gässler, 1987, 1988) placed provisos on the applicability of the bilinear surface, effectively limiting it to near vertical excavations ($\alpha \leq 10^\circ$) in cohesionless or near cohesionless soils subject to high surcharge far from the excavation edge.

Current feelings regarding the bi-linear surface are best summarized by Elias and Juran, 1991:

‘The assumed bi-linear surface is mainly based on a limited number of model tests where failure was caused by substantial surcharge loading ($q > \gamma h$). However, other full scale and model tests with surcharges placed closer to the face, with no surcharge or with traffic induced vibrations yielded a simple failure mechanism with only one nearly circular slip line being observed. The bilinear failure surface does not appear to be consistent with the observed behavior of soil nail retaining structures which are subjected mainly to self weight. In particular, stability analyses indicate that this bi-linear failure surface is generally not contained in the nailed soil mass and therefore yields an active zone (or potential failure wedge) which is substantially larger than that observed on actual structures.

Recent published data has shown that the bilinear failure mechanism is applicable only in cohesionless soils subject to high surcharges of limited extent with the slip circle mechanism being critical in all other cases (Gässler, 1988)’.
Excavations for the expansion of the Good Samaritan Hospital, Portland Oregon, 1976 and the University of California, Davis 1981 (ENR 1976, Shen et al. 1981a and 1981b, Shen et al. 1982): The use of soil nailing to support a vertical faced cut (ranging in depth from 10.7m to 13.7m) at the site of the Good Samaritan Hospital Portland is significant in that it is the first recorded application of this technique in the USA. The excavation was conducted in 1.5m high stages, with the reinforcing consisting of drilled and grouted nails (7-8.5m long) fixed to a 100mm thick wire mesh reinforced shotcrete facing. Reports regarding the systems performance (ENR, 1976) noted that the soil nailed construction process allowed a timesaving of 50 to 70% compared to conventional support systems. This allowed a cost reduction of approximately 30% for the construction of the final wall system. The importance for the development of soil nailing of such public accolades are obvious, but the Good Samaritan Hospital is significant also because it provided American academics with their first hands on experience and raw data from the technique.

A limited amount of field instrumentation was installed on the Good Samaritan Hospital site by researchers from the University of California Davis, as a way of compensating for local unfamiliarity with soil nailed systems. The information gathered on the site was insufficient to allow any wide ranging conclusions but it provided the impetus for a more comprehensive National Science Foundation funded study of the soil nail earth retention technique. This study included significant analytical analysis and centrifuge modeling and culminated in the construction of an extensively instrumented field prototype constructed on the campus of the University of California, Davis. Using the results of these studies, Shen et al. (1981) developed a limit equilibrium design procedure for the design of soil nailed excavations, which is commonly referred to as the Davis design method.

The Davis design method assumes the failure surface is best approximated by a parabolic curve passing through the toe of the excavation. Contours of factor of safety derived from finite element modeling in conjunction with contours of maximum shear strain observed in centrifuge modeling provided the basis for this assumption (Figures 1.16a and 1.16b). The method considers that only tensile forces are developed in the reinforcements, with these forces both providing a resisting force parallel to the failure plane and an increased normal force perpendicular to the failure plane (the...
increased normal force resulting in a subsequent increase in the shear forces developed). Distinction is made between cases where the failure surface extends beyond the reinforced soil block (case 1) and where it remains within the reinforced soil (case 2). Figure 1.17 shows the forces considered acting for a case 1 scenario.

![Figure 1.16](image)

Figure 1.16: Results of finite element analysis (a) Contours of shear stress (Shen et al., 1982) (b) Contours of factor of safety (Bang, Shen and Romstad, 1980).

The forces are resolved (through a limit equilibrium analysis) so that the total driving force is in equilibrium with the total resisting force. Separate factors of safety are applied to the soil shear strength (FS _φ_ ) and soil cohesion (FS _c_ ). The overall factor of safety being obtained when FS _φ_ = FS _c_ = FS.

![Figure 1.17](image)

Figure 1.17: Davis method – forces acting when the failure plane extends beyond the reinforcement zone. (Shen et al., 1982).

The method was updated by Bang in a series of later publications (Bang et al. 1992a, 1992b, Elias and Juran, 1991). The modified method allowed analysis of more complex soil and excavation geometries, as well as static and earthquake loading. Importantly however the modified method made a provision for the allocation of a specified nail-soil interface force per unit length (as would be indicated by a nail...
pullout test). This appears to have been incorporated in order to address one of the major criticisms of the original Davis design method, namely the determination of nail forces at failure.

In their original publications, Shen et al. (1981), asserted that the maximum restoring force able to be imparted by any nail could be calculated by taking the lesser of:

- The structural tensile capacity of the nail eg:
  \[ T_r = A_t f_y \]
- The shear force developed between the nail and the soil for that portion of the nail behind the failure surface (the effective length), where the maximum bond stress between the nail and the surrounding soil is prescribed by Coulomb’s failure criterion eg:
  \[ \tau_{ns} = c' + \sigma_n \tan \phi' \]

In a published discussion of Shen et al. 1981, Gássler outlined his own reservations regarding the validity of using a Coulomb failure criterion to determine the maximum bond stress between the nail and the surrounding soil. Gássler’s objections centred on the determination of appropriate normal stresses acting at the nail-soil boundary (\(\sigma_n\)). The Davis method asserted that \(\sigma_n\) could be assumed to be equal to the overburden pressure \(\gamma z\). Gássler argued that the effects of restrained dilatancy (Schlosser and Elias, 1978) could not be ignored, particularly in medium dense to dense granular soils. Further, the effects of nail installation generally meant that initial stress conditions were significantly altered in the ground surrounding the reinforcement members and as such, overburden pressures could not be assumed to be acting at the soil/nail interface. It is important to note, that whilst Gássler did not mention it in his discussion, the opinions he expressed were backed by numerous soil nail pullout test results (conducted during the Bodenvermächelung project) showing the ultimate nail force to be independent of depth, and independent work conducted by Schlosser (Schlosser et al. 1981).

Further to Gássler’s comments, it is worth noting that the Davis design method does not consider failure of the soil nails due to pullout of the reinforcement embedded in the active zone.

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"Clouterre" – The French national research project, 1986-1991. (Recommendations Clouterre 1991, Plumelle et al. 1989, Plumelle et al. 1990, Schlosser et al. 1992, Unterreiner et al. 1997, Benhamida et al. 1997): In 1986, a national research project into the soil nailed excavation support system was commenced in France. The project involved more than 21 organizations, with backgrounds ranging from government departments, research institutes, consulting and contracting firms. The general aim of the collaboration was to promote the use of soil nailing (particularly for medium and long term structures) by providing a well researched and practical set of recommendations and regulations for the design and construction of the method. In recommendations Clouterre, 1991, Schlosser identifies four central areas of research that were targeted:

- To better understand the behavior of soil nailed walls.
- To define the limitations of the process.
- To improve methods for designing structures.
- How to use soil nailing for long term structures.

An extensive work program was completed and included the results of:

- Three full scale instrumented model walls where failure was induced by distinct, deliberate mechanisms.
- Performance measurements from five, instrumented actual soil nailed walls.
- A detailed program of laboratory experimental studies investigating aspects of soil nail interaction, reinforced soil response and corrosion and durability of nails.
- Conducting, interpretation, accumulation and summary of soil nail pull-out test results.
- Theoretical and analytical analysis.
- Consultation with soil nailing community

The research project concluded with the publication of the French soil nailing specifications, ‘Recommendations Clouterre 1991’. America’s Federal Highways Administration (FHWA) sponsored an English translation of the recommendations (FHWA report no. FHWA-SA-93-026), noting its importance as a document 'that summarises the whole design and construction process, from geotechnical investigation to field quality control'.
In spite of the large amount of practical input and research work conducted as part of the project, the recommendations have not been without their share of controversy. In particular, differences have surfaced regarding the proposed design method. In the Recommendations Clouterre, Schlosser presents the ‘multi criteria’ design method as the appropriate design tool for soil nailed excavations. The multi criteria design technique is a limit equilibrium stability analysis, which attempts to account for both tensile and shear forces induced within the soil nails. It was first presented in its entirety in Blondeau et al., 1984, although earlier publications by Schlosser had detailed sections of the associated theory. Being a “classical” limit equilibrium method, the potential driving and restoring forces are calculated for a considered failure surface. In the case of the multi criteria method, a circular failure surface with a small radius of curvature that passes through the base of the facing is recommended (although analysis of irregular failure planes using methods such as the perturbation method are recognised as being legitimate for certain cases). The results for numerous different failure surface geometries are considered in order to find the most critical.

The main feature that distinguishes the multi criteria method from pre-existing design methods is the consideration of shear forces induced within the reinforcements. As is discussed earlier, the physical dimensions of soil nails are such that they possess some flexural rigidity. As such, when soil nails bridge a zone of soil shearing (as is the case in soil nailing), there exists the potential for bending moments and accompanying shear forces to be induced in the nails. Earlier methods considered the effects of shear forces induced in soil nails to be negligible, and concluded disregarding them would result in a slightly conservative, but greatly simplified calculation process. Schlosser et al. 1983 used theoretical computations, which considered the nail to be equivalent to an infinitely long pile intersecting a shear zone, to calculate the size of typical induced shear forces and the relevant displacements required to induce these forces. Based on the results of these calculations, Schlosser and his co-workers asserted that consideration of shear forces induced in soil nails was justified.
Figure 1.18: Multi Criteria Design Rule failure criterion, (after Recommendations Clouterre, 1991)

The inclusion of bending and shear forces induced in the reinforcement meant that more complicated possible failure mechanisms needed to be considered. In order to determine which of these competing failure modes was the most critical at any given time Schlosser and his co-workers developed a failure envelope for the soil nail and the immediately surrounding soil (Figure 1.18). As can be seen, this envelope is presented with respect to the axial force induced in the nail (T_n) and the shear force in the nail (T_c). Four separate failure criteria are presented to describe the possible failure modes of the soil nail system:

1. **Soil nail skin friction criterion**: failure occurs through nail pullout.

2. **Soil nail lateral pressure criterion**: failure occurs via a bearing pressure failure of the soil at the soil nail interface (the nail cuts through the soil).

3. **Soil nail plastification due to shear criterion**: failure occurs because the reinforcing in the nail becomes plastic at the point of zero moment due to the combination of shear and normal forces acting.

4. **Soil nail plastification due to bending criterion**: failure occurs because the reinforcing in the nail becomes plastic at the point of maximum moment due to the combination of normal force and bending moment acting.

Force combinations that lie within the envelope defined by these criteria are considered to be safe and as such will not result in failure. Those outside the envelope represent failure of the nail/soil system. The appropriate loading path for the soil nail
(i.e. the combination of shear and tensile forces that are acting in the nail at failure) is determined using the “maximum work rule”. Using knowledge of the geometry of the intersection of the soil nail and the failure surface, the point on the failure envelope is chosen so as to maximize the work of the force in the nail (Recommendations Clouterre 1991).

As has been mentioned earlier, two opposing schools of thought have developed regarding the inclusion of shear forces developed in soil nail reinforcement. In particular, the validity of two assumptions made by the multi criteria design method have been questioned. The first relates to the size of the shear force that will cause rupture in a reinforcement undergoing tension, and the second relates to what constitutes ‘failure’ of a soil nailed wall and the validity of the assumption that at this ‘failure’ both tensile and shear forces will be induced within the reinforcement.

These arguments for and against the inclusion of shear forces are dealt with in more detail in the section of chapter 1 entitled ‘Jewell and Pedley and the 1990 International Reinforced Soil Conference in Glasgow’. For the moment, we will simply note that in Figure 1.18, for a force combination $T_c, T_n$, where $T_c$ is greater than zero, the magnitude of the normal force $T_n$ at failure is always less than the pure tensile capacity of the reinforcing $R_n$. As such disregarding shear forces is not simply a matter of making a conservative assumption that disregards a small restoring force for the sake of computational efficiency. If Schlosser’s assumptions are valid, then disregarding shear forces could lead to an overestimate of the reinforcement’s tensile capacity. As Schlosser noted:

‘the effect of the bending stiffness can be either beneficial or unbeneﬁcial depending on the behaviour of the soil-inclusion system’ (Schlosser, 1991).

Pre-existing design methods utilized a classical ultimate limit state analysis in order to assess the wall's global stability. A certain failure mechanism is assumed for a wall then the total driving forces and total restoring forces are calculated and compared. This type of analysis contains the implicit assumption, that the soil and each layer of reinforcing will either achieve or maintain its maximum allowable load at the time of failure. For a soil nailed excavation there exists three major reasons why this may not be the case.

The first relates to issues of strain compatibility. Recommendations Clouterre reports that 'a nail's tensile strength, like its pull-out strength, is mobilized for very small displacements.' A soil's shear resistance can however require angular deformation, 'up to several percent' to develop. So depending on the reinforcement and soil characteristics, the possibility exists that the nail could reach failure before the soil shear resistances are fully developed. It should be noted that practical and experimental results have indicated that for most practical purposes ('mild steel reinforcements in sufficiently stiff soil', Recommendations Clouterre) soil shear stresses will be mobilized by the time tensile rupture of the nails occurs.

The second reason is not completely independent of the first and is more a complication of the strain compatibility issues caused by the wall's staged construction process. As is discussed in section 1.2.1, the staged aspect of a soil nailed wall's construction process means that determining exactly what strains each element of the nailed soil mass has undergone is relatively complex. Each layer of reinforcement is constructed at different times into/onto the deforming excavation. Nails towards the top of the excavation will have experienced strains associated with construction induced movements and support of temporary construction stages, as well as those required to stabilise the final excavation. As such, the reinforcements closer to the top of the excavation will have experienced greater strains and subsequently higher stresses than those at the base of the excavation.

As an illustration of this, consider Figures 1.19 (a) and (b). In 1.19(a), a soil nailed excavation is shown with two failure surfaces. Failure surface 1 represents the most critical failure surface at the intermediate construction stage, level A. Failure surface 2
represents the most critical failure surface at the final grade. Figure 1.19(b) shows a
typical stress strain curve for pullout of a soil nail. Consider the excavation at the
intermediate construction stage, level A. In order to stop failure of the wall at this
stage, a zone of shearing will develop along failure surface 1. This will induce tensile
stresses in nail 1. Importantly, the stresses in nail 1 will not be confined to the point
where it intersects with failure surface 1. In fact, if point B (the intersection of nail 1
and failure surface 2) is sufficiently close then appreciable tensile stresses will also be
induced here. In terms of Figure 1.19(b) the stresses in nail 1 at point B will have
moved from the origin to $\sigma_1$. Now when the excavation is continued to the final stage,
a zone of shearing develops along failure surface 2. This results in a strain “X” being
imposed on nails 1 and 2 at their points of intersection with failure surface 2. For nail
2, the strain “X” results in an increase in stress from the origin to $\sigma_2$. However, for
nail 1, strain “X” results in an increase from $\sigma_1$ to $\sigma_3$. Because of the wall’s staged
construction method nail 1 has experienced greater stresses than nail 2.

Figure 1.19: (a) hypothetical nailed slope. (b) hypothetical stress-strain curve for nail pullout.

The situation is further complicated when it is considered that the excavation will be
self supporting until some critical depth and that the critical failure surface will
intersect the nail at different locations depending on the excavation height. Because of
the wall’s staged construction method it cannot be assumed that each nail will reach a
failure point simultaneously. As has been shown, it is possible that some nails will
have experienced greater strains than others and as such these nails will be more
highly stressed.
The third reason is simply that the basic static equilibrium of the problem is such that reinforcements at different levels attract different forces.

The consequence of these factors is that the possibility exists for a progressive failure of a soil nailed wall. Overstressed nails may rupture or pull-out before the full capacity of the soil or the other nails has been reached. Such a situation cannot be considered using a classical limit equilibrium method.

Juran’s kinematical limit analysis design method can consider progressive failures caused by virtue of the problem’s statics, because it divides the nailed mass into slices parallel to the nails. This means each nail’s response can be separately calculated. It cannot account for failures due to strain incompatibility, though it can give an indication of nail loadings during intermediate stages, allowing an assessment of possible strains accumulated before the final stage.

![Diagram](image)

Figure 1.20: Juran et al’s Kinematical Design Method system of forces (Juran, 1990).

Unfortunately, solution of Juran’s proposed system is complex and requires a number of assumptions that have subsequently been called into question. In particular, the assertion that a unique failure surface (Figure 1.20), where shear and tensile forces will be at a maximum, can be defined using the angle of intersection with the surface above the cut, the angle of exit at the base of the cut and a log spiral curve has been publicly questioned (Recommendations Clouterre, 1991, Leshchinsky 1991) as has the size of the shear force induced within a soil nail (Jewell and Pedley, 1991).
Current feeling regarding the method is that in its present form it remains unproven and inappropriate for use as a design tool. These feelings are reflected in the most recent FHWA Manual for Design and Construction of Soil Nailed Walls, which has the following comments regarding the method:

'This method is theoretically and numerically complex, has not yet been presented in a form that can be easily understood or used by practicing engineers and has been challenged by others as containing questionable theoretical assumptions' (FHWA-SA-96-069R, 1998).

Schlosser, whilst more encouraging, also expressed his reservations:

'This design method was proposed (Juran et al., 1990) for designing soil nailed walls at serviceability limit state. The assumption is that the peak shear resistance of the soil is mobilized under service conditions along the maximum tension line, irrespective of the value of the wall’s global safety factor. This assumption, which is based on analysis of results from a few full-sized structures (Juran et al., 1988) needs to be justified, both theoretically and experimentally.' (Recommendations Clouterre, 1991).

Jewell and Pedley and the International Reinforced Soil Conference, Glasgow, 10-12 September 1990. (Jewell and Pedley 1990a, Jewell 1990, Jewell and Pedley 1991, Schlosser 1991, Jewell and Pedley 1992): In 1990, Jewell and Pedley published the first major criticisms of the inclusion of shear forces in soil nail stability calculations (Jewell and Pedley 1990a). The authors identified recently published works by Bridle, 1989 and Mitchell and Villet, 1987 as being the impetus for their paper, but went on to single out Schlosser’s multi criteria design method (Schlosser 1983) as the original source of the error. It is of note that the works of Bridle and Mitchell and Villet both misrepresent the multi criteria method as outlined by Schlosser. Bridle’s work, as he himself admitted, borrowed from Juran’s earlier work (Bridle1990a), whilst Mitchell and Villet over estimated the lateral capacity of the reinforcement by overlooking its failure due to combined bending and tension. Furthermore, throughout their critique of Schlosser’s work, Jewell and Pedley use Mitchell and Villet’s flawed interpretation of the multi criteria design method as representing Schlosser’s theory.
That said, in fairness to Jewell and Pedley it must be mentioned that Schlosser is noted as a contributor on the 1987 Mitchell and Villet report, a fact which was interpreted by Jewell and Pedley as indicating that he had abandoned the idea of failure due to combined bending and tension (Jewell and Pedley, 1991). Additionally, Schlosser's early publications on the multi criteria theory are general in nature and do not explicitly detail the full multi criteria design theory. These two facts provide a possible explanation for the misrepresentations and misunderstandings that followed.

This controversy aside, Jewell and Pedley 1990a provided a detailed and independent formulation for assessing the role of bending stiffness in a soil nailed excavation. They concluded that for typical reinforcement dimensions and for the types of soils typically supported using soil nailing, the size of the shear forces induced in the nails were sufficiently small as to be insignificant. In a paper Jewell presented at the 1990 International Reinforced Soil Conference in Glasgow he expanded on this, saying that a strain incompatibility exists between the mechanisms that induce tension and shear forces in the reinforcement, and as such it was 'unduly optimistic' to assume that both the limiting shear force and tensile force will be mobilized at failure. As evidence of this point, Jewell presents shear test data conducted by Pedley (Figure 1.21) and the results of a monitored full scale excavation conducted by Güessler as part of the 'Bodenvernagelung' project (Figure 1.22).

Figure 1.21: (a) overall shearing resistance and (b) maximum reinforcement forces measured in a large scale direct shear apparatus (after Pedley et al, 1990).

Figure 1.21(a) shows the relationship between shear stress and shear displacement for a reinforced soil with various reinforcement arrangements presented by Jewell (1990), (note that the solid line shows the response of an unreinforced soil). Figure 1.21(b)
shows the non-dimensionalised relationship between shear and axial forces developed in the reinforcement for the tests presented in Figure 1.21(a). As can be seen, initially for each test predominantly axial forces are generated in the reinforcement. Then, as the test progresses shear forces are predominantly induced. This confirms the disparity in the displacements required to generate each force. More importantly however is the shaded zone marked ‘failure for reinforced soil’ in Figure 1.21(b). This zone is ‘practically defined by the point of maximum overall shearing resistance’ as determined using the results presented in Figure 1.21(a) that is, it shows the point at which the maximum shear stress for the whole reinforced soil system was achieved. After this point, extra shear displacements resulted in reductions in the shear resistance of the reinforced soil. Note that for all the test results, the reinforced soil fails when only approximately half of the total available reinforcement shear resistance has been mobilized. This finding supports Jewell and Pedley’s assertion that inclusion of shear forces overestimates the restoring forces in a soil nailed excavation, because the wall will have failed before enough strain has occurred to induce appreciable nail shears.

As an aside, it is worth noting that for all the results in Figure 1.21(b) the total force provided by the reinforcing elements parallel to the plane of shearing increases after failure, but as Figure 1.21(a) shows that the total shear stress parallel to this plane is reducing (as is the definition of failure). This infers that the soil shear strength is reducing at a faster rate than the restoring forces due to the inclusion are increasing.

![Graphs](image1.jpg)

Figure 1.22: Measured (a) axial forces and (b) bending moments post failure (Jewell, 1990) and (c) bending moments pre and post failure (Gässler, 1990)
To show that the behaviour seen in Pedley’s shear test results also occurred in the field, Jewell presented some responses observed in full scale test structures by Gässler. Figures 1.22(a) and (b) show the axial forces and bending moments measured in a full scale excavation pushed to failure by a surcharge load. They are supplemented by Figure 1.22(c), which displays results of the same test presented by Gässler in the 1990 conference discussion. In the field trial shown, failure of the structure occurred under a surcharge of 150kN/m². During failure, the structure underwent 40mm to 50mm relative shear displacement along the failure zone (line AB). At the end of this shear displacement, the structure came to a new ‘post failure’ equilibrium with a reduced surcharge of 110kN/m² now acting. The results depicted in Figures 1.22 (a) and (b) were measured at this post failure equilibrium, while the results in Figure 1.22(c) show the measured bending moment in the nails both at failure and at the post failure equilibrium. In Figure 1.22(c) we can see that the nail bending moments were small at failure, it is only after the large shear strains were experienced that bending moments approaching the limiting case were recorded. Importantly, the new post failure equilibrium, with shear forces acting, was only maintained because of a reduced surcharge being applied to the system. Again, these results support the exclusion of potential shear resistance.

Further to this, we can see in Figures 1.22(b) and (c), a number of points of zero bending moment occur within the active wedge. Jewell asserted that this indicated that a number of shear zones had developed during the excavation process (lines CD and EF) and that the small size of the bending moments induced at these points indicated that the potential failure planes had been stabilised by predominantly tensile forces. This provides anecdotal evidence supporting the assertion that any shear resistance imparted by the soil nails is insignificant prior to failure of the wall.

Whilst Schlosser and his coworkers did not reply directly to these assertions, it is interesting to note the suggestions given in Recommendations Clouterre, 1991 regarding the assumption of a nail working in tension only. It says that this assumption 'can be justified [in certain cases] for the reasons given below.

- It corresponds to the way the nails work in the majority of structures in service. In fact, the mechanisms of skin friction interaction is preponderant in
soil nailed retaining structures and develops with small deformations, before the mechanism of lateral pressure, which is only observed in the immediate vicinity of the failure.

- For certain techniques (driven steel bars), the nails used in retaining structures have a section and inertia small enough, such that the results of calculations with the assumption of "tension only" differ slightly from those where shearing and bending have been taken into account.

Importantly however, and in defense of the systematic inclusion of shear forces, Schlosser qualifies the above statements by adding:

'On the theoretical level, when using classical limit equilibrium methods, there is not always a beneficial effect from taking account of bending and shearing, in fact as shown in the multicriteria principle, the value of the force at failure in pure tension $R_n$ can be significantly higher than the component at failure tension $T_n$ that results from the combination of tension and shearing. However, in methods based on yield design theories, taking bending and shearing into account would always be beneficial.

One must be careful, given the present knowledge, to consider that the "simplified" calculation provides systematically a conservative approach of conditions of stability.'

Essentially, Schlosser is saying that if a widely applicable system for the design of soil nailed structures is to be proposed, it should consider the effects of shear, because ignoring these effects can in some cases be unconservative. If your system incorporates the assumption of tension only, then the people using that system will need to be aware of the situations where this assumption is no longer conservative. This argument for the systematic inclusion of shear is however countered by the collective work of Jewell et al., which shows that the assumption that substantial proportions of the allowable shear and tensile forces will both be mobilized at failure is questionable. As such there exists the possibility that the combined restoring effect of shear and tension in the reinforcement and shear in the soil calculated using the multi criteria design method may overestimate the actual restoring effect at failure, because the large strains required to induce substantial shear forces will have not occurred before failure of the system.
Whilst the assumption of the nail working in tension only, is now the more widely used method for design of soil nail walls, it is worth noting that a consensus has not been found with regard to which of the above arguments is the most correct. Perhaps with a view of addressing this, Gässler, in the discussion of the 1992 Fukuoka conference, proposed that the name ‘soil nailing’ should be reserved for support systems that have the nails acting in tension, with shearing or bending as a possible secondary benefit. For cases where bending and shearing of the nails was appreciable, he proposed the term ‘dowel’ or soil dowelling. In this way, a ‘tension only’ design method for soil nails can be conservatively proposed, with a multi criteria type design method being reserved for the analysis of soil dowelling. By way of illustration, Figure 1.23 presents the three cases used by Gässler in this discussion. Case 1 and case 2 (a) are considered to be examples of soil nailing, case 2 (b) is considered to be an example of soil dowelling.

Figure 1.23: Examples of soil nailing (case 1 and case 2 (a)) and soil dowelling (case 2(b)). (Gässler, 1993)

Federal Highway Administration (FHWA) Demonstration Project – (DP 103), FHWA Tour for Geotechnology – Soil Nailing, 1993 (FHWA-PL-93-020), Manual for Design and Construction Monitoring of Soil Nail Wall, 1998 (FHWA-SA-96-069R) and Soil Nailing Field Inspectors Manual, 1994 (FHWA-SA-93-068): In 1992, the FHWA Demonstration Project (DP 103) ‘Design and construction Monitoring of soil nail walls’ was started by a collaboration of US government agencies and private firms. Its stated aim was to promote the use of soil nailed excavation support in the American transportation construction industry by providing ‘guidance for selecting, designing and specifying soil nailing at sites which
were technically suited and economically attractive for its application'. The project included: review of current practice, a coordinated research program, a series of monitored existing structures, instructional workshops and where appropriate, specific onsite technical assistance and design review.

Four major publications resulted from the project: FHWA Tour for Geotechnology – Soil Nailing, 1993 (FHWA-PL-93-020) which reviews European practice and summarises the findings of a scanning tour conducted through Europe in 1992, Manual for Design and Construction Monitoring of Soil Nail Wall, 1998 (FHWA-SA-96-069R) which summarises the findings of the project and presents in detail the recommended design and implementation procedures, Soil Nailing Field Inspectors Manual, 1994 (FHWA-SA-93-068) which provides a reference document for agents charged with inspecting and monitoring soil nail wall construction and Recommendations CLOUTERRE, English translation, 1993 (FHWA-SA-93-026) which is a translation of the findings of the French National Research Program, Clouterre.

Work started in 1992 with a scanning tour of Europe. This tour provided FHWA, State Highway and industry representatives with the opportunity to review current European soil nailing practices and subsequently formulate an appropriate research program. In a report summarizing the findings of the tour, Byrne et al. (1997) identified four major areas requiring ongoing research: a) 'development of a case history data bank', b) 'evaluation of current design computer codes', c) 'instrumentation and monitoring of in service walls' and d) 'establishment of an approach for evaluating the capacity (ultimate and serviceability limit states) of soil nail wall structural facings'. Whilst the project has undoubtedly resulted in an increased acceptance of soil nailed support in the US transportation industry, (information has been collected from at least 48 as part of DP 103) and provided comprehensive and practical guides for the application of the method, it is in this last area, the evaluation of facing capacity, that DP 103 has provided the most impact on the current state of the art.

The FHWA 'Manual for Design and Construction Monitoring of Soil Nail Walls' (FHWA-SA-96-069R) outlines the design method recommended by the project. In
essence it advocates a limit equilibrium design approach with a ‘tension only’ nail force criteria that is conceptually similar to much of the existing ‘tension only’ design methods (it also makes allowance for an ‘Earth Pressure design method’, though its limitations are noted). Where it adds to these pre-existing methods is its systemic consideration of the facing as a structural element that contributes to the global stability of the retaining system. This concept is explored in detail in Chapter two, but for now, Figure 1.24 introduces the concepts suggested by the method.

Figure 1.24: Typical nail strength diagram (after FHWA-SA-96-069R)

Figure 1.24 shows a typical ‘nail strength diagram’ recommended in FHWA-SA-96-069R. As can be seen, it graphically shows the allowable nail force as a function of position on the nail. As with other methods, ‘for any particular sliding wedge, the reinforcement contribution of the nail is a function of the location at which the associated slip surface intersects the nail.’ In zone A in the diagram, pullout failure of the reinforcement is critical. Consequently, the allowable nail contribution is simply the pullout resistance of that portion of the nail embedded beyond the slip surface. At the intersection of zone A and zone B, the pullout resistance is equal to the tensile capacity of the reinforcement. From here on (zone B) the allowable force is governed by the nail’s tensile capacity (T_N). In zone C the pullout capacity of the portion of the nail embedded in the active zone becomes critical. Importantly, in zone C the allowable force is equal to the force due to friction between the nail and the soil, plus the nail head facing connector strength (T_F). The idea that the connection between the facing and the nail head provides some resistance to nail pullout is in itself not particularly ground breaking, what is significant however is that FHWA-SA-96-069R
goes on to provide a rational system for evaluating the contribution of the facing to the nail head strength and thereby the global stability of the structure.

1.2.3 State-of-the-art

In this chapter, it has been shown that the mechanisms by which a soil nailed wall retains an excavation are relatively complex. In keeping with this, the present state of the art has been the result of evolutionary process. Initially, analysis of soil nailed walls relied heavily upon the known behaviour of similar, more familiar retention systems (reinforced earth). Deeper consideration of the support mechanisms involved led researchers to identify significant differences between soil nailing and other earth reinforcement systems. These differences provided the impetus for more detailed research aimed at formulating a design process specific to the soil nailed technique. As can be expected, this research continued the process of evolution, with subsequent researchers identifying weaknesses and improving upon the strengths of the pre-existing methods. Whilst it is true that this refinement process is still ongoing, the present state-of-the-art is such that when applied with a degree of engineering judgment, confidence can be had that the system’s behaviour can be reasonably predicted. Proof of this fact can be found in the array of competent structures designed using the present techniques.

In terms of systems for design, those presented in ‘Recommendations Clouterre’ and the FHWA ‘Manual for Design and Construction Monitoring of Soil Nail Walls’ both represent rational, easily applied systems that have been proven through test programs and wide spread use. As is discussed in the respective sections of this chapter, both methods have relative strengths and weaknesses. The FHWA manual presents a more comprehensive system of facing design that allows any structural capacity imparted by the facing to be considered as part of the wall’s global stability. On the other hand, its use of a ‘tension only’ nail strength criteria means it is not advisable for situations where appreciable shear and bending will be induced in the reinforcement. Clouterre’s multi criteria design method is capable of accounting for shear and bending in the reinforcement, but care must be taken when including any restoring effect due to shear. Strain incompatibilities between the soil reinforcement mechanisms typically
mean that the soil shear resistances and nail tensile resistances induced are post peak, when sufficient strains have occurred to generate the shears.

With respect to the current pool of knowledge regarding soil nailing, the following aspects of the above methods that are considered favourable in any applicable design method:

a. **Limit Equilibrium Analysis with a failure surface geometry appropriate to the insitu geology and load conditions:** Whilst limit equilibrium approaches do have problems (inability to predict progressive failure through rupture of overstressed nails – see the section of chapter 1 entitled ‘Jewell and Pedley and the 1990 International Reinforced Soil Conference in Glasgow’), their application to the design of soil nailed slopes has been proved through continued use for design of these structures. In terms of the failure surface geometry, early work relied on simple failure surface geometries (the German Method’s bilinear surface), however further research found such surfaces were too simplistic for most typical soil nailing applications. Circular or log spiral curves have been shown to most accurately reflect the case of a soil nailed wall in a homogeneous soil, loaded predominately by self weight, but as with all slope stability problems, engineering judgment should be used when deciding the most applicable mechanism in soils with a variable insitu soil profile.

b. **Determination of frictional resistance of the reinforcement through insitu pullout testing:** Extensive testing of installed soil nails has shown that the pullout resistance of soil nails is generally independent of the overburden pressure. This phenomenon is widely attributed to the effects of restrained dilatancy (Schlosser and Elias, 1978) and alterations caused to the initial ground stress conditions by the nail installation process. For design purposes a constant mean shear force per unit length (kN/m) is recommended. FHWA-SA-96-069R provides typical values depending upon the hole installation method, soil type and hole diameter. Recommendation Clouterre presents correlations with the limit pressuremeter pressure, depending upon the soil type and nail installation technique. Both design manuals stress that insitu testing should be conducted to confirm these preliminary design values.
c. Consideration of internal, external and mixed modes of failure as well as stability of intermediate construction stages: Figure 1.2 shows commonly accepted failure modes for soil nailed walls. In addition to these, mixed failure modes that intersect both reinforced and unreinforced zones need to be considered (Schlosser, 1992). Stability should be calculated for intermediate construction stages as well as final grade. FHWA-SA-96-069R recommends that the local short term face stability should be confirmed using field test cuts, as ‘this failure mode is not amenable to conventional stability analysis’.

d. Reduction of allowable ultimate tensile strength, where appreciable shear and bending is expected in the reinforcement (soil dowelling): Work by Jewell and Schlosser (detailed in the respective sections of this chapter), has shown that the tensile force at failure in a reinforcement undergoing appreciable shear and bending can be significantly reduced from the ultimate pure tensile strength of the reinforcement. In situations where significant shear forces are generated pre-failure (eg; soil dowelling), a failure envelope that accounts for the actions of combined loadings should be used (eg; multi criteria design method).

e. Consideration of contribution to global stability provided by the structural strength of the facing: European experience has found that facings built to meet typical constructability and durability constraints will provide sufficient strength so that failure of the facing is not a concern. Work conducted by the Americans as part of Development Project - DP 103, has provided a rational research based method for assessing the contribution of the facing to the nail head strength. This allows consideration of the facing design in global stability calculations.
Chapter 2: Facing Design.

2.0 Introduction

In chapter one, an introduction was given to various aspects of the development and design of soil nailed excavations. Three major components of any reinforced soil system were identified, namely: the reinforcing elements, the facing panels and the stabilised fill. Discussion regarding current design methods centered on the first and third components, the reinforcing elements (or soil nails) and how they interact with the surrounding soil. This chapter will focus on the design of the facing panels. It will be shown that a knowledge gap exists regarding actual facing responses. Typical facing systems are described and current empirical and theoretical models for the design of facing panels reviewed. Comparisons between the current design methods are made, with particular reference to the parameters that influence the facing response. Finally, a method for reducing the knowledge gap is proposed and detailed.

2.1 Facing Elements

As a consequence of the reliance of soil nail systems on short term stability, the construction techniques used have conventionally been chosen to minimize construction times. This has had particular impact on the type of facing used. As is mentioned in chapter one, sprayed concrete (shotcrete) has been intrinsically interconnected with soil nailed excavation support because of its speed and flexibility of construction. This is particularly the case for initial face protection or temporary construction support. That said, a number of reasons not the least of which is aesthetics, have led to the development of various alternate facing techniques (eg: cast
in place concrete, precast concrete and other structural and nonstructural prefabricated facing panels).

Figure 2.1: Typical reinforcement detail for double mesh reinforced shotcrete facing (FHWA-RD-89-198)

What constitutes a typical facing system for a soil nailed excavation, generally depends on the design life and intended function of the finished structure. For a temporary wall, the facing usually consists of mesh or fibre reinforced shotcrete with a thickness ranging from 50mm to 150mm depending predominately on the nail spacing. This facing is placed as soon as possible after excavation so as to maintain the soil moisture and avoid raveling or erosion (Bruce and Jewell, 1987). A steel plate (typically 150 x 150 x 10mm or 200 x 200 x 15mm) is used to connect the facing to the nail (Gäßler, 1990). Intermediate to long term structures will typically supplement this initial ‘construction’ layer with an additional facing layer. Numerous methods are used to construct this second layer.
Figure 2.1 shows a typical construction detail for a case where the construction layer is being supplemented by an additional mesh reinforced shotcrete cover. Such a situation is usually used in cases where aesthetics are not paramount (e.g., basement walls etc.), although sculptured and coloured shotcrete facings are sometimes used to simulate rock/textural finishes. Gassler (1990) describes such double wire meshed concrete facing systems as typical of those complying with German licensing restraints (total thickness of shell about 150mm to 200mm). For facings of this kind, both facing applications are typically placed prior to excavation of the next construction stage.

Figure 2.2: Typical reinforcement detail for cast in place concrete facing (FHWA-SA-96-069R)
Figure 2.2 shows a similar construction detail with a cast in place (CIP) mesh reinforced concrete facing overlying the shotcrete construction layer. This type of facing system has found favour amongst recent long term works in the USA because of the beneficial durability and aesthetic qualities of CIP concrete (FHWA-SA-96-069R, 1998). Note the shear lugs attached to the nail plate. These lugs are included to increase the facing’s resistance to punching shear.

CIP facings are typically built after the excavation has reached its final grade (the initial ‘construction’ facing having been builtincrementally as the excavation progressed). Constructability issues often dictate the thickness of the final CIP facing with typical total thicknesses being 200mm to 250mm (FHWA-SA-96069R, 1998).

Figures 2.3, 2.4, 2.5 and 2.6 show examples of prefabricated facing systems. Prefabricated panel systems are appealing because they provide durable, adaptable, architecturally pleasing facing systems that have the potential to reduce construction costs by ‘industrializing’ the construction process. Additionally, backfilling of the void between the temporary shotcrete layer and the prefabricated panel with gravel (Fig. 2.4) provides an easily constructed free draining wall (construction of adequate drainage can be an issue of concern with shotcrete faces).

Wide spread use of prefabricated panels for traditional reinforced earth systems meant that such systems became an obvious contender for adaptation to soil nailed walls. However a number of obstacles relating to the durability and load transfer mechanisms of the connections between the prefabricated panels and the construction facing/nail heads have meant that prefabricated systems have not been widely utilized.
for long term soil nail facings. That said, a number of commercial attachment/connection systems do exist, with some associated proprietary restrictions/patents applying. Figures 2.3 to 2.6 detail such systems. Prefabricated facings are also typically placed after the excavation has reached its final grade, with temporary support having been provided by the incrementally built initial shotcrete ‘construction’ facing.

2.2 Design of Facing Elements

In section 1.2.2, the evolution of the current state-of-the-art soil nail wall design method is presented in roughly chronological sequence. The main advances are discussed with regard to the knowledge base existing at the time. The aim of this method of presentation is to illustrate the rational development process that led to the current state-of-the-art. Section 2.2 repeats this process for the design of facing structures on soil nailed walls.

Before starting this process, it is worth noting that it is generally believed that, ‘our understanding of the magnitude of the face loadings developed in soil nailing applications is not as good as our knowledge of the maximum loads developed within the nails’ (FHWA-SA-96-069R). A possible explanation for this gap in the current knowledge is the belief that the facing is a non-critical element of the reinforcing system. Similar statements are common throughout the literature; eg: ‘the facing is not a major structural load-carrying element but rather ensures local stability of the soil between the reinforced layers and protects the ground from surface erosion and
weathering' (Kim et al., 1995) – (see section 2.2.1). This view of the role of facing has meant that assessment of facing response has typically been a secondary research consideration. To date, the amount of work published regarding the characterisation of facing response represents only a small proportion of the published literature concerning the design of soil nail walls.

2.2.1 Local Stability Analysis

In chapter 1, it is mentioned that one of the first models used to consider how the reinforcements in a soil nailed wall support an excavation was the stabilised earth block model. This model assumes that the action of the soil nails is simply to generate a stable, integral block of reinforced soil. This integral soil block resists the forces induced by the adjacent unreinforced soils as if it was a simple gravity retaining wall. In this analogy, the whole reinforced block of soil performs the role of a ‘traditional’ facing element (e.g. gravity retaining wall, sheet pile wall), providing a structural element that collects/combines driving and resisting forces. The action of the actual facing in the stabilised earth block model is simply to stop the soil between the nail heads unraveling and eroding the reinforced soil block. Essentially, the facing maintains local equilibrium around the nail heads and does not contribute to the system’s global stability.

Figure 2.7: Local failure wedge (Terrasol, 1983 after Mitchell and Villet, 1987)
This early view of the role of facing as a predominately non-structural element can be seen in some of the early facing design methods. Mitchell and Villet, 1987 present a simplified design method, which they credit to an internal Terrasol report (Terrasol, 1983). The failure wedge geometry shown in Figure 2.7 is used to calculate the earth pressure at the facing. \( S_V \) and \( S_H \) are the vertical and horizontal nail spacing. For this method, 'it is assumed that because of the arching effect above the considered section, the overburden pressure ... is transferred to the surrounding soil'. Thus the earth pressure at the facing is simply that caused by an active wedge forming between the nails, the overburden pressure from any overlying soil is discounted leaving:

\[
p = \frac{1}{2} \left( \gamma S_v \tan^2 \left( 45 - \frac{\phi}{2} \right) \right)
\]  

(2.1)

Here \( p \) is the earth pressure on the facing and \( \gamma \) and \( \phi \) are respectively, the unit weight and friction angle of the retained soil. It is assumed that this force is evenly distributed over the facing panel. Design of the reinforced shotcrete facing is then completed by 'considering each concrete layer as a beam or raft of width \( S_V \) (vertical spacing between the reinforcements) on simple supports formed by the reinforcements'.

Design of this type, despite its obvious simplifications, has persisted within the industry (Byrne et al., 1997 report that European practice during a 1992 scanning tour included 'designing the facing for an equivalent active load corresponding to a soil depth of one to two nail vertical spacings'). That said, it is worth noting that whilst Mitchell and Villet present this method as an appropriate design tool, in a separate section of their FHWA research report, they comment that 'it is probable that factors such as differences in construction techniques, installation methods used to insert the inclusions, facings and site conditions are the probable cause' of differences in facing responses observed in monitored structures. In essence, the authors are saying that whilst they recognise that the factors influencing the size of the earth pressures at the facing are numerous and complex, the simplified method of design they propose will be sufficient when applied within the context of typical applications of soil nailing technology.
2.2.2 Reduced Active Pressure Analysis

Some of the earliest work presenting measurements of actual facing responses came from the German Research Project ‘Bodenvernagelung’. Gässler and Gudehus (1981) present the results of earth pressure measurements conducted on full scale test structures (Figure 2.8). Load cells (Glötzl cells) were placed at the shotcrete-soil interface during the construction of the wall and measurements recorded before and during surcharge loading. Earth pressures recorded due to self weight are reported as being equal to 50% of the Coulomb earth pressure. In addition to these findings, Gässler and Gudehus summarize their observations of the facing responses of the full scale test structures and instrumented actual structures monitored during the Bodenvernagelung project, commenting that ‘Reductions of at least ca. 40% and 30% for self weight and surface portions of Coulomb earth pressure were observed again and again and can therefore be recommended for design’, and ‘shotcrete and nail heads never failed prior to global failure in the large scale tests, nor was any damage observed in the practical projects’.

Figure 2.8: Earth pressure measurements from instrumented full scale soil nailed wall (Gässler and Gudehus, 1981).

Long term monitoring conducted by the Germans showed similar results. Stocker and Riedinger (1990) present the results of long term observation of a cut instrumented as part of Bodenvernagelung. Again Glötzl cells were installed at the soil shotcrete interface during construction, however for this reported cut the results from these load cells were too erratic for sensible interpretation. Instead nail forces measured 0.2m
behind the face were used to give an indication of the earth pressures experienced by
the facing. Figure 2.9 shows the observed earth pressures upon completion of the
excavation and after 10 years of service. The authors report that the measured earth
pressure represents 50% to 65% of the calculated ‘rectangular pressure distribution’.

![Diagram of earth pressure](image)

Figure 2.9: Results of long term monitoring of an instrumented full scale soil nailed
wall (Stocker and Riedinger, 1990)

The design of facing elements for German structures ‘is specified in the government
license and must be followed. Typically, the face design pressure is taken as 75 to 85
percent of the corresponding Coulomb active earth loading applied as a uniform
pressure over the height of the wall face’ (Byrne et al., 1997). This is obviously an
adaptation of the observations of the Bodenverzahlgung project. The percentage
reduction of the Coulomb active earth pressure provided a simple design method
based on the empirical observation ‘that the earth pressure acting on the facing is
distinctly lower than the Coulomb’s earth pressure’ (Gässler, 1990). Interestingly,
Gässler (1990) also reveals that a number of minimum tolerances and ‘durability’ type
criteria are specified by the licenses (‘double wire meshed concrete facing with a
minimum of 3cm overlay’ for permanent structures), commenting that these
restrictions effectively provide a minimum facing design. The provision of minimum
designs is possibly a way of ensuring that the empirical relationships developed
during Bodenverzahlgung are not applied outside their intended application.

Chapter 2: Facing Design.
The German facing design method presents two major refinements to the stabilised earth block model. Firstly, it provides an empirically proven relationship for estimating the size of the earth pressures at the facing. Secondly, by considering the Coulomb active earth pressure for the final excavation height, it introduces the idea that the parameter H (total cut height) has an influence on the final earth pressure at the facing. Earth pressures calculated using the stabilised earth model are independent of the total cut height. This identification of cut height as a parameter influencing facing pressure is one of the first major challenges to the idea that the facing simply provides local support for the soil between the nails.

2.2.3 Reduced Maximum Nail Tension Analysis

From 1986 to 1991, the French National Research Project, Clouterre, conducted a research program that included: full scale test structures, monitored actual structures and theoretical investigations. Improvement of the existing knowledge regarding facing design was not a major aim of this project, consequently the ‘studies and instrumentation programs conducted for the Project CLOUTERRE did not permit an accurate investigation into the behaviour of wall facings’, (Recommendation Clouterre, 1991). That said, the issue of facing design was considered in the recommendations, and a number of significant steps were made towards improving the existing understanding of the facing’s behaviour.

As with previous work, a number of factors were identified as having an influence upon the size of the load on the facing; ‘the stiffness of the soil, rigidity of the facing, depth and spacing of nails’, however French experience indicated that ‘the most important of these (was) the spacing between the nails’. Observations of nail loads near the face of the test structures and consideration of the potential loading mechanisms formed the basis of a new facing design method detailed in the recommendations. Figure 2.10 presents the model considered by the recommendations to be the most analogous with the actual facing behaviour. As can be seen, this model is built around the idea that ‘Generally, the facing is subjected to tension $T_o$ of the nails at the head and to earth pressure $p_e(z)$ between the nails’. With the combination
of these forces resulting in the system behaving ‘more or less like a floor slab’, where
the nails act as column supports (Recommendations Clouterre, 1991).

Figure 2.10: Forces considered acting on segment of facing (after Recommendations
CLOUTERRE (1991)).

Simplification of the statics and assumption that the earth pressure \( p_e(z) \) is uniform led
to the following expression relating: the tensile force at the nail head \( T_o \) to the
uniform earth pressure \( p \) and the vertical and horizontal nail spacing \( S_V \) and \( S_H \)
respectively:

\[
T_o = p \times S_V \times S_H \tag{2.2}
\]

Empirical correlations between: \( T_o, T_{max} \) (the maximum allowable axial force in the
nail) and \( S \) (the maximum value of \( S_V \) and \( S_H \) expressed in meters) were proposed to
enable the calculation of \( T_o \) and thus the uniform earth pressure \( p \):

\[
T_o/T_{max} = 0.5 + 1/5(S - 0.5) \text{ when } 1 \leq S \leq 3m \tag{2.3}
\]
\[
T_o/T_{max} = 0.6 \text{ when } S \leq 1m \tag{2.4}
\]
\[
T_o/T_{max} = 1 \text{ when } S \geq 3m \tag{2.5}
\]

For punching shear calculations, ‘it is recommended to adopt \( T_o \) values such that
\( T_o/T_{max} = 1 \), whatever the layout arrangement’. This is done as a way of accounting
for ‘concentrations of earth pressure \( p \) on the facing around the nails’.
Once the appropriate earth pressures are calculated, the recommendations propose that the facing be designed as if it were a perpendicularly loaded slab, supported by concentrated loads at the nail heads. Different models of vertical continuity through the slab are proposed for different construction techniques and wall design lives. Figure 2.11 illustrates three examples. The facing is considered to be a continuous slab (Fig. 2.11 (a)) for short term walls where standard facing reinforcement overlap lengths are maintained between subsequent shotcrete applications. Hinges are considered at construction joints (Fig. 2.11 (b)) for medium to long term walls with sufficient facing reinforcement overlaps. Where there is insufficient continuity of reinforcement between construction stages, the facing is modelled as independent slab elements (Fig. 2.11 (c)).

Figure 2.11: Models of slab continuity (a) short term walls with suitable overlap (b) medium to long term walls with suitable overlap (c) insufficient overlap (Recommendations Clouterre, 1991)

The identification of the tension in the nail head \((T_o)\) as a possible indicator of the earth pressure at the facing represents an important refinement of the facing design process. Whilst pre-existing methods recognised that the loadings at the head were complex, the application of their design processes only considered loading on the facing to be due to the earth between the nails pushing out on the facing. In essence, the nail heads are considered to be stationary and the soil pushes out on the face. The consideration of tension in the nail head identifies an additional form of loading on the face, namely that caused by the nail head pulling the facing into the earth. The distinction between, the face loading being caused solely by the soil pushing onto the face (active case) and it being due to the nails and face being pulled into the soil (passive case) is very significant. Further, the idea that the facing may be helping the
nail head resist being pulled into the soil means that the facing is having some input to the global stability of the structure. As such, the view has changed from the facing being simply concerned with maintaining local stability between the nails to one where the facing is contributing to the overall stability of the structure. Additionally, the consideration of nail forces constitutes recognition that the structural connection between the nails and the face has some function with regard to the size of the load on the facing.

2.2.4 Nail Head Connector Strength Analysis

In 1992, America's Federal Highway Administration (FHWA) started a federally funded research program (Demonstration Project – DP 103) for the purpose of promoting the use of soil nailed excavation support in the American transportation construction industry. This project highlighted facing design as one area where substantial advancements could be made to the current knowledge. In their summary of a fact finding tour conducted as part of the DP 103, Byrne et al. (1997) reported that European practice ranged 'from entirely empirical, based on experience and without any formal design, to conventional strength limit state design based on estimated facing and connector loads'. In order to address this, the demonstration project conducted a program of full scale laboratory testing and finite element modelling. The results of this program form the basis of the facing design method presented in the FHWA Manual for Design and Construction Monitoring of Soil Nail Walls (FHWA-SA-96-069R, 1998).

The FHWA facing design method highlights three critical failure mechanisms; facing flexure, facing punching shear and head stud tensile fracture. Figure 2.12 illustrates these three failure modes. For each of these three critical failure mechanisms corresponding nail head strengths ($T_{FN}$) are calculated, with the lowest calculated strength indicating the critical failure mode. Other facing failure modes are mentioned, 'one-way shear of facing, flexure of connection bearing plate and shear of the connection bearing plate'. However, research conducted as part of DP 103 indicated that for typical facing systems 'these mechanisms are not critical and therefore are not checked in the design procedure' (FHWA-SA-96-069R, 1998).
For facing failure due to flexural loading of the face (Figure 2.12 (a)), the appropriate nail head strength \( T_{FN} \) is calculated using the following formula:

\[
T_{FN} = C_F (m_{v, \text{neg}} + m_{v, \text{pos}}) \left( \frac{S_H}{S_V} \right)
\]  

This formula was derived through the consideration of 'force and moment equilibrium of a typical interior facing panel loaded by the soil, with supports at the nail head locations and full plastic moment capacity developed at all applicable yield lines' (FHWA-SA-96-069R, 1998). \( m_{v, \text{neg}} \) and \( m_{v, \text{pos}} \) are the vertical nominal unit moment resistances at the nail head and mid-span locations respectively, \( S_H \) and \( S_V \) are the horizontal and vertical nail spacings and \( C_F \) is the facing pressure factor. The facing pressure factor was introduced as a way of considering variations in the allowable facing moment caused by 'the non-uniformity of the contact pressure between the facing and the subgrade' (FHWA-SA-96-069R, 1998). A table of recommended design facing pressure factors is presented for varying facing thicknesses and expected design life. The values presented are reported as being based on 'back analysis of case histories, full scale laboratory tests, calibrated finite element modelling, experience and judgement' (FHWA-SA-96-069R, 1998). The inclusion of design life as a relevant indicator of the degree of uniformity of the facing earth pressure results from differences in typical construction practice for temporary and permanent facings. Temporary facings typically include horizontal waler bars passing beneath the nail head bearing plate (Figure 2.2). The FHWA manual asserts that these
bars are required in order to ensure that the full plastic moment capacity is developed throughout the slab prior to failure (one of the base assumptions behind equation 2.6). For facings where these bars are not included (permanent facings), the more conservative approach of $C_F$ equal to 1 is adopted so that uncertainties associated with the development of full plastic moment can be moderated. In contrast, facing thickness is considered, because it gives an indication of the relative stiffness between the facing element and the soil.

A number of restrictions are placed on the applicability of equation 2.6. These limitations address issues relating to the ratio of horizontal to vertical spacing, the stiffness of the facing (as indicated by the reinforcement percentages) and the minimum reinforcement required in order to ensure a ductile failure. Any substantial variation away from the typical facing layouts (as outlined by these limits) results in the proposed empirically based formula no longer remaining suitable.

Calculation of the facing punching shear strength (Figure 2.12 (b)), is addressed for two nail head facing connection systems; the temporary 'bearing plate' type connection (Figure 2.13 (a)) and the permanent 'headed stud' type connection (Figure 2.13 (b)). For both systems, punching shear of the nail through the face is countered by two separate resistances: that afforded by the concrete in the facing and that contributed by the passive resistance of the soil against the base of the failed cone.

The resistance afforded by the concrete $V_N$ is calculated by considering the shear stress developed around the perimeter of a typical failure surface. Figure 2.13 shows the proposed failure surfaces for headed stud and bearing plate connections. The total force required to cause failure is calculated using the expression:

$$V_N = 0.33 \sqrt{F_c \text{ (MPa)}} (\pi)(D'C)(hc)$$

(2.7)

The relevant parameters are defined in Figure 2.13.

The resistance contributed by the soil pressure acting on the failure cone is calculated using the following empirical expression relating the punching shear strength ($T_{FN}$) to the facing shear strength ($V_N$), the area of the base of the failure cone ($A_C$), the area of
the nail grout column ($A_{GC}$), the horizontal and vertical nail spacing ($S_h$ and $S_v$) and the pressure factor for punching shear ($C_s$):

$$T_{FN} = V_N \left( \frac{1}{1 - C_s (A_c - A_{GC})/(S_v S_h - A_{GC})} \right) \quad (2.8)$$

The pressure factor for punching shear, $C_s$ is included as an attempt to account for non-uniformity of the earth pressure on the facing. As with $C_F$, the flexural pressure factor, design values are presented relative to facing thickness and design life.

Figure 2.13: (a) plate bearing connection (b) headed stud connection (FHWA-SA-96-069R, 1998)
Finally, the ultimate nail head strength relating to tensile failure of the headed studs welded to the steel plate (Figure 2.12 (c)) is calculated using the following simple ultimate tensile strength criterion:

$$T_{FN} = 4 A_{HS} F_U$$  \hspace{2cm} (2.9)

$A_{HS}$ is the cross-sectional area of the body of the headed stud, and $F_U$ is the ultimate tensile stress of a headed stud. Note that this formula assumes four studs are welded to each facing plate. Additionally, a number of stud dimensioning criteria are proposed in order to ensure bearing failure of the concrete beneath the stud heads does not occur.

The FHWA design method represents a substantial advance from the pre-existing systems for the design of facing support. In particular two main areas can be highlighted where significant improvements have been made to the existing knowledge.

The first is in the evaluation of the structural capacity of a proposed facing/nail head system. Previous methods generally shied away from specifying a suitable method for assessing facing strength, preferring instead to recommend a suitable model and refer to the appropriate concrete design codes. The FHWA design method provides a straightforward, research verified system for evaluating the structural capacity of the facing-nail head connector system, enabling more rigorous structural design of the facing. Additionally, the methods for evaluating structural capacity proposed by the FHWA method, allow the consideration a number of parameters whose effects, whilst recognised by pre-existing research, were not considered in their proposed design methods. A case to point is the provision of flexural and shear pressure factors ($C_F$ and $C_S$ respectively) in order to allow the assessment of the influence of non-uniformity of the earth pressure at the facing. Pre-existing methods acknowledged the existence of higher earth pressures at the nail heads but did not incorporate their effects into the design method. The reasoning behind this was that the assumption of uniformly distributed earth pressures was ‘conservative when compared with actual conditions (for) calculating bending in the facing’ (Recommendations Clouterre, 1991). Interestingly, according to the FHWA method, for a 100mm temporary facing, the inclusion of the effects of load concentration around the nails doubles the nominal
nail head strength ($T_{FN}$) associated with the flexural capacity of the facing. For such a case, the benefits of the refinements included in the FHWA method are obvious.

The other major advance made by the FHWA method, is the consideration of the effects of facing design on the global stability of the structure. By systematically including the strength of the facing in the allowable nail forces, the nail head connector strength method proposed by the FHWA explicitly includes the influence of the facing on the global stability of the soil nailed wall. This is a very significant point, because whilst techniques such as the French facing design method recognised the contribution the facing made to the global stability of the cut, the simplified design methods they proposed paid little deference to the idea. Consequently, these methods failed to highlight the accompanying notion that, the loading on the facing was the result of the nails pulling the facing into the soil as well as the soil pushing out on the facing. The FHWA method, by considering and attempting to characterise the influence of the interaction between the nails, soil and facing elements emphasizes the importance of this interaction as a driving factor generating earth pressures on the facing.

Figure 2.14: Normalised measured nail head loads (FHWA-SA-96-069R, 1998)
These advancements aside, it is important to note that whilst the assessment of facing strength is well considered by the FHWA design method, little contribution has been made to the current knowledge regarding the determination of expected loads at the facing. Figure 2.14 shows the results of the monitoring of full scale soil nail structures conducted as part of DP 103 and presented in FHWA-SA-96-069R. Nail head loads (as indicated by the extrapolation of nail loads to the facing) are presented normalised by the active earth pressure ($K_a \gamma H$) multiplied by the nail's tributary area ($S_v S_H$). Based upon these results and the published findings of Gässler et al. and Clouterre (as documented in previous sections), the FHWA 1998 design manual recommends two methods for determining the nail head loads that are likely to be experienced. That is, either assume that the ratio of the nail head load to the maximum nail load is equal to 0.5 or assume the nail head load is equal to $0.5 K_a \gamma HS_v S_H$.

Figure 2.15: Normalised maximum measured nail loads (FHWA-SA-96-069R, 1998)

As a measure of how rigorous these recommendations are, it is worth comparing the two methods to see whether they deliver consistent expected facing loads. If we assume that the nail head load is equal $0.5 K_a \gamma HS_v S_H$ as is recommended and we also

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assume that the ratio of the nail head load to the maximum nail load is equal to 0.5 (the alternate recommendation), then we find that the corresponding expected maximum nail load is equal to $K_{s}\gamma HS_{v}S_{H}$. Figure 2.15 presents the maximum nail loads (normalised with respect to $K_{s}\gamma HS_{v}S_{H}$) from the full scale soil nail structures monitored as part of DP 103 and presented in the FHWA 1998 manual. As can be seen the assertion that the maximum nail load is equal to $K_{s}\gamma HS_{v}S_{H}$ is hard to justify, in fact the manual itself asserts that ‘the mean normalised maximum nail load in the upper portion of the wall appears to lie in the range of 0.75 ($K_{s}\gamma HS_{v}S_{H}$)’.

Conversely, if the recommended mean maximum nail load of 0.75$K_{s}\gamma HS_{v}S_{H}$ is adopted and the recommended ratio of the nail head load to the maximum nail load of 0.5 is applied, then we find a nail head load of 0.375$K_{s}\gamma HS_{v}S_{H}$ should be expected. This is obviously at odds with the alternate recommended nail head load of 0.5$K_{s}\gamma HS_{v}S_{H}$. The manual is asserting that facing pressures corresponding to either 0.5$K_{s}\gamma HS_{v}S_{H}$ or 0.375$K_{s}\gamma HS_{v}S_{H}$ should be adopted, depending upon which method you use. This contradiction is a reflection of the degree of uncertainty associated with the determination of ‘at service’ facing loads.

2.2.5 Other and Ongoing Work

More recent work has been conducted by the French investigating working stresses in soil nail structures. This work is proceeding under the title Clouterre II. Unterreiner et al. (1997) present a 2D finite element simulation of the CEBTP (Centre Expérimental de Recherches et d’Etudes du Bâtiment et des Travaux Publics) test wall number one, pushed to failure as part of the Clouterre project. Comparisons of observed and calculated facing displacements and nail loads are presented as evidence of the program’s ability to model construction of the soil nailed wall. Earth pressures calculated by the program are presented for positions 0.5m, 1.0m and 3.0m from the face. The earth pressures calculated close to the face vary from about 25% of the active case to greater than the at rest ($K_{o}$) conditions. This leads Unterreiner et al. to comment that; 'The complexity of these results makes the design methods presently used for the facing look even more simple and rough'. At the time of writing the results of Clouterre II are as yet unpublished.
Additionally, attempts have been made to use reduced scale models to monitor earth pressures. Difficulties simulating the soil nail construction sequence have cast doubt on the effectiveness of such methods to accurately model the facing response. Raju et al. (1997), attempted to use Glötzl cells to record earth pressures at the facing of a model soil nailed wall. They were forced to rely on maximum nail forces to provide an indication of the facing loads, because ‘the results of the earth pressure cells were not reliable’ as they gave readings ‘about seven to eight times larger than the Rankine’s earth pressure at rest’. The model walls used by Raju et al. (1997) used pre-buried facing elements because of difficulties ensuring the short term stability of the temporarily unsupported cuts. Such a procedure is more analogous to a support system comprised of sheet piles tied to soil nails. Consequently, even if the maximum nail forces are giving an indication of the earth pressures at the facing, some doubt can be leveled at the legitimacy of using a pre-buried facing system to model the behaviour of facing in a soil nailed excavation.

Tei et al. (1998) had similar problems simulating the soil nail construction process. They conducted a series of 24 model centrifuge tests for the purpose ascertaining the influence of ‘wall slope, nail length, nail roughness, nail inclination, facing stiffness and facing roughness’ on the global stability, displacements and earth pressures at the facing. Because of difficulties placing the reinforcement whilst the centrifuge was in flight, they pre-buried the nails and the facing. Excavation was then simulated during the test, by draining flexible water reservoirs located in the excavated zone. The earth pressures they recorded gave a good correlation to the Coulomb active earth pressures for the equivalent actual walls. As is mentioned above, substantial work on full scale structures has shown that earth pressures of the order of 50% of the Coulomb active case are typical of soil nailed structures. Tei et al.’s results can probably be attributed to their failure to ensure that their models properly mirror the actual construction process.
2.3 Discussion of Current Facing Design Methods

Section 2.2 has shown the evolution of understanding that has led to the current state-of-the-art for facing design. Initially, design relied on the assertion that the effect of the reinforcements negated any overburden effects. Design of this type was based upon adaptation of the response of similar, better studied systems (e.g. RE wall) and limited actual observations. As the database of actual wall responses grew, design based on empirical observation emerged (e.g. the German design method). Design of this type identified new influences on facing response (e.g. excavation height), which in combination with ongoing research helped refine the theoretical models used to describe facing behaviour (Clouterre facing method). This process of refinement and characterisation of the theoretical models has continued with more recent publications and design methods (e.g. FHWA Manual for Design and Construction Monitoring of Soil Nail Walls, 1998).

As it stands, the current state of the art recognises the complexity of characterizing the behaviour of soil nail wall facings. Facing response is shown to be due to an interaction between the three major components of the soil nail system: the reinforcements or nails, the soil being reinforced and the facing panels. As such, factors that influence the behaviour any one of these components can affect the behaviour of all three. In fact, this complexity has been recognised from relatively early on in the development of soil nailing. Most of what has led to the current understanding has been the product of refinements to how we deal with this complexity.

Our present knowledge allows the ultimate capacity of a constructed system to be reasonably well predicted. The methods used account for the influence of the physical dimensions of the support system, the innate strength of the soil being reinforced and the influence of some simple soil structure interaction effects as they have been observed within the usual scope of application of the soil nailed method (e.g. the concentration of earth pressures around the nail head). The current knowledge does not however allow rigorous calculation of the size of the earth pressures likely to be developed at the facing. Whilst the influence of interaction between the reinforcing
components can be considered when calculating the ultimate capacity of the system, our present understanding does not permit these factors to be considered when evaluating the potential loading at the facing. In fact, it is true to say that a knowledge gap exists pertaining to the size of the earth pressures likely to be developed at the facing and what factors influence the development of these loads.

In this regard, it is worth noting that in section 2.2, results detailing the direct measurement of earth pressures at the facing are rare. In fact, it is only the work of Gässler et al. that presents direct measurements. Stocker and Riedinger, 1990 and Raju et al. 1997 both attempted to directly measure the earth pressures using Glötzl cells, but found the results nonsensical, forcing them to rely upon measured nail loads near the face. Further, work conducted by Clayton et al. (2000, 2002) exploring the validity of pressure cell measurements in sprayed concrete tunnel linings has cast doubt on the performance of pressure cells subjected to temperature and shrinkage stresses related to the strength development of early-age shotcrete.

Because of these difficulties, the majority of the existing information on actual facing loads has been interpreted from nails loads measured close to the face. Whilst relatively easy to obtain, nail loads can be 'difficult to assess in the vicinity of the facing, where bending effects (due to self weight of the facing) tend to be more significant' (FHWA-SA-96-069R). Additionally, traditional alternatives to full scale testing (model and centrifuge testing) have been unable to simulate the soil nailed wall construction process properly (Raju et al., 1997 and Tei et al., 1998). As was discussed earlier, simulations that use pre-buried facing elements should not be used to assess facing response because they do not model the stress relaxation that occurs in the excavated soil prior to the placement of the facing.

This difficulty in obtaining and modeling direct facing responses, is significant. In fact the FHWA Manual for Design and Construction Monitoring of Soil Nail Walls (FHWA-SA-96-069R) asserts that the main reason 'our understanding of the magnitude of the face loadings developed in soil nailing applications is not as good as our knowledge of the maximum loads developed within the nails', is 'that the quality of field monitoring data is poor'.
With this in mind, it is worth noting that recent attempts to predict facing behaviour have typically used finite element modeling to calculate facing responses (Clouterre II and FHWA DP 103, Untreriner et al. 1997). Such computational techniques are attractive to researchers because they allow the investigation to failure of numerous individual parameters, something that would be too costly using the more traditional techniques (full scale test modelling).

It should also be noted that whilst the methods presently used to predict the likely loading at the facing do not allow rigorous calculation, they have to date performed well in numerous actual and experimental excavations. Failure of a soil nailed wall because of inadequate facing elements is so rare as to be nonexistent. In fact durability and constructability issues often dictate the final facing design for permanent soil nail walls. This garners the question ‘Why do we need to know more’?

In answering this question, it is important to note the existence of provisos that limit the application of the available design methods. These restrictions limit the scope of application of the existing design methods so that the final design lies within the proven envelope of construction ‘norms’. They are required because the available design methods are not considered to be sufficiently rigorous to characterize facing behaviour outside of this envelope. Greater understanding of the mechanisms controlling the size and distribution of the face loads is required so that rigorous design can be conducted for applications of the soil nailed technique outside of the current practice.

2.4 Proposed Work

In its broadest terms, the aim of this research is to increase our understanding of the role played by shotcrete facing elements in a soil nailed excavation. More specifically, it is hoped that this work will identify which factors have the greatest influence on the stresses induced in the facing elements and then quantify and summarize the impact these major influences.
For this purpose, it is proposed that a 3D finite element program, capable of simulating construction of soil nailed excavations be developed and used to explore the influence of various parameters upon the working stress induced within the facing. As has been mentioned earlier, difficulties obtaining and modelling direct facing responses mean that finite element analysis is arguably the most appropriate means of conducting wide ranging parametric studies.

It is proposed that three separate elements, each corresponding to a separate reinforcing component, be used (a twenty noded elasto-plastic ‘soil’ element, an eight noded isoparametric Mindlin Shell ‘shotcrete’ element and a two noded beam ‘nail’ element). Inclusion of separate soil nail and facing elements will mean that induced moments and forces in the support elements can be directly calculated. Simulations of a full-scale experimental soil nailed wall conducted as part of The French National Research Project CLOUTERRE will be used in conjunction with a number theoretical and empirical observations in order to verify that the program can properly simulate the response of a soil nailed excavation.

Finally, a parametric study will be conducted in order to identify and quantify the major influences on facing behaviour.
Chapter 3: A three-dimensional program capable of simulating soil nailed excavations.

3.0 Introduction

Chapter three presents the three-dimensional (3D) finite element (FE) program, Td8auto, developed for the purpose of investigating the facing response of soil nailed excavations. Fortran code for the program is attached in Appendix B.

3.1 Simulation of soil nailed excavation

It is widely recognised that good numerical analysis requires an appropriate balance of simplicity and accuracy. Put simply, 'there is no benefit in using a very complex relationship to analyze a problem where the simplest representation ... would result in acceptable accuracy' (Duncan, 1996).

For this thesis, a 3D finite element program, capable of simulating construction of soil nailed excavations has been developed. In deference to the preceding paragraph, it is reasonable to ask why a 3D analysis has been considered necessary when 2D 'plane strain' models have previously been used to simulate soil nailed wall construction (Shen et al. 1981, Thompson and Miller 1990, Unterreiner et al. 1997, Benhamida et al. 1997).

Put simply, it was judged that the added complexity of a 3D simulation was justified because the primary use of this program would be an investigation of the facing
response. A 2D 'plane strain' model considers the discrete wall support points provided by the nails as being 'smeared' along the length of the wall. Effectively, the bar reinforcements are replaced with 'equivalent' sheets. This simulation is inappropriate when considering facing response, as bending in between supports along horizontal planes can not be modelled.

Additionally, a number of workers have highlighted problems associated with the simplification of situations where supports apply load on a wall at discrete points to 2D 'plane strain' conditions (Cardoso and Carreto, 1989 and Matos Fernandes, 1986). Whilst this work tends to concentrate more on situations where the actual conditions do not approximate plane strain conditions, it is interesting to note that the work of Matos Fernandes found discrepancies between 2D and 3D models of a strut supported diaphragm wall were greatest for earth pressures and bending moments at the wall.

As is mentioned in chapter 2, three types of element are utilised by the program; a twenty noded elasto-plastic 'soil' element, an eight noded isoparametric Mindlin Shell 'facing' element and a two noded beam 'nail' element. Inclusion of separate soil nail and shotcrete elements has meant that induced moments and forces in the support elements can be directly calculated.

The ability to directly model facing response distinguishes this program from a number of existing 3D finite element simulations of soil nailed support (Ho and Smith 1993, Smith and Su 1997, Zhang et al. 1999). That said, it is not unique in its use of shell elements for modeling soil nail facing. Briaud and Lim (1997) used the shell elements provided as part of the ABAQUS general finite element package (Hibbit, Karlson, Sorensen, Inc. 1992) to model the construction of a bridge abutment. Unfortunately, their calculated facing responses are not widely applicable, because of closely spaced piles (3.8 x diameter of the pile) situated at the excavation face.

Soil behaviour is characterized by an elasto-plastic (elastic – perfectly plastic) response curve. The Sloan and Booker (1986) modified Mohr-Coulomb yield surface is used to describe the failure criterion\(^1\), and a Von Mises' plastic potential surface is

\(^1\) See discussion in paragraph 2, p.77 relating to treatment of stresses exceeding failure criterion.
used to define strain orientation at failure. Use of Von Mises' plastic potential surface means that soil dilation is not considered. Additionally, joint elements have not been included and as such slip between the soil and reinforcing members is not modeled. As is mentioned later, the exclusion of soil dilation and joint elements, means that the results of analysis conducted using this program will be most applicable when considering working stresses and not limit values.

Excavation is simulated using Brown and Booker's (1985) virtual work solution. A skyline solver has been incorporated, in order to reduce the memory required to run the program. Versions, which allow analysis of Gibson soils and application of pre-loads to determine initial stresses, have also been produced.

3.2 Calculation process

Table 3.1 presents a flow chart depicting the program's calculation process. This chart reads from top to bottom, outlining the major steps in order of their implementation. As can be seen, two major programming loops exist within Td8auto, a load/excavation increment loop and an iteration loop. These loops allow calculation processes to be repeated so that (a) in the case of the load increment loop, a stepped excavation sequence can be considered and (b) in the case of the iteration loop, an iterative solution process can be implemented.

It is very important that a stepped excavation sequence be used when modeling soil nailed wall construction. In particular, it is essential that the simulation used should as much as is practicable reflect the actual excavation process. For our proposed program, there are two reasons why the process is necessary in soil nailed systems.

The first relates to the construction method used to erect soil nailed walls and is almost universally applicable to simulation of the method. As is mentioned previously, soil nail excavations are relatively unique, in that they are built from the top down with the reinforcement being constructed onto the newly excavated face. This process means that each layer of reinforcement has a unique zero stress position, the displacements that induce stress in the reinforcement are different from the total
Table 3.1: Flow chart depicting Td8auto programming sequence.
displacements and change for each layer of reinforcing. Without a stepped simulation process it would not be possible to properly model the influence of the incremental construction process. This phenomenon needs to be properly mirrored in any simulation for meaningful results.

The second reason that excavation needs to be simulated in steps representative of the actual construction process, relates to the decision to use an elasto-plastic relationship to describe the soil response. During a stepped excavation the soil will experience stress states that are different to its final stress state. Importantly, the stress states experienced by an element of soil during excavation can be more severe than its final stress state. The consequence of this is that elements which experience stresses higher than yield during an intermediate stage, will deform plastically. This plastic deformation, results in permanent strain, which would not have occurred if excavation were only simulated in one step, as the more severe stress state would not have been reached.

The load loop incorporated into td8auto means that excavation can be carried out in the required ‘stepped’ process. Without this, soil plasticity and incremental construction could not be properly modeled.

![Elasto-plastic soil response curve](image)

Figure 3.1: Elasto-plastic soil response curve.

The iteration loop, on the other hand, is necessary in order to obtain a solution in plastic soils. Figure 3.1, illustrates the perfectly elastic, perfectly plastic stress strain response used by Td8auto to characterise the soil behaviour. As can be seen, soil
behaves linearly elastic until the yield stress is reached, after which the soil behaves as if perfectly plastic. This response curve effectively limits the maximum stress in an element to the yield stress, as calculated using Sloan and Booker’s Mohr-Coulomb yield surface. Programming this method requires an iterative process. With continual checking of the newly calculated stress state, to ensure plastified elements are identified and corrected for. This process is dealt with in more detail, later in this chapter.

Td8auto starts by opening and reading from a data file. The data file is created automatically from responses to screen prompts, using a purpose built mesh generator. The information in the data file can be loosely grouped into two categories. The first consists of information which stays constant throughout the analysis or which sets up initial conditions for the mesh. This includes nodal coordinates, element connections, boundary conditions, soil properties and initial stresses. The second group contains information, which relates to a specific excavation step. This includes elements to be excavated and reinforcement details. Table 3.1, shows how ‘READ’ statements inside and outside of the excavation loop input this information appropriately.

After reading what reinforcement is being placed before the current excavation step is simulated, the program sets the zero stress positions for the newly added reinforcement. As is discussed earlier, it is important that the zero stress state for the reinforcement is recognised. In reality, the shotcrete and nail annulus, are placed wet onto the deforming excavation. All stresses within the reinforcement are induced by displacements after the reinforcements have hardened. The displaced shape of the excavation when the cement hardens thus becomes the zero stress position for that reinforcement. Only subsequent displacements of the excavation (and thus reinforcements) induce stress within that layer of reinforcement. For td8auto, this means that stresses within the reinforcement elements should not be calculated using total displacements, rather they need to be calculated using the displacements that have occurred since the reinforcement was installed. As such, the program stores the displacement of the reinforcement elements’ nodes when the reinforcement is first installed, and uses this as the zero stress position.
Once the reinforcement zero stress positions have been set, the program calculates the mesh stiffness matrix. The shell and bolt elements used in the program have been chosen so that they share nodes with the brick elements. As Figure 3.2 shows, a shell element shares the nodes of one face of a brick element, a bolt element, any two consecutive nodes. This means the process of building a soil nailed support system can be simulated by adding the stiffness of the required reinforcing elements onto the existing soil stiffness. Table 3.1 shows how this procedure has been programmed. First, a stiffness matrix is formed for the 20 node brick elements that have not been excavated. This stiffness matrix is calculated with respect to the global (XYZ) axes. Next the stiffness for each 8-node ‘shotcrete’ shell element is formed. The shell stiffness matrix is calculated with respect to convenient local axes. The local stiffness matrix is then transformed to the global axes system, (via multiplication with a transformation matrix) and then added to the appropriate entry in the 20-node soil stiffness matrix. This process is repeated for the bolt elements.

Figure 3.2: Elements used by Td8auto.

When the full stiffness matrix is calculated, it undergoes a Choleski reduction. The Choleski reduction (skyline solver) method used greatly reduces the size of the stiffness matrix, by only storing stiffness’s for nodal displacements that are free to deform. This process is discussed in greater detail in section 3.3.4.

After the stiffness matrix has been calculated and the size of the required storage space minimised, the excavation loads are calculated and added to the total load vector. As has been mentioned, the loads are determined, using Brown and Booker’s
(1985), virtual work solution. Brown and Booker found that by equating internal and external work produced by a virtual displacement, they could calculate the excavation force as follows:

\[
\Delta g = - \int_{V_i} (B^T D B) dV + \int_{V_i} (B^T \sigma_{i-1}) dV + \int_{V} (N^T \gamma) dV + \int_{S_i} (N^T t) ds
\]

(3.1)

Where \(B\) is the displacement strain matrix, \(D\) the incremental stress-strain matrix, \(N\) the shape functions, \(\sigma\) the internal stress vector, \(\gamma\) the body force vector, \(t\) is the traction vector and \(g\) is the nodal displacement vector. The required integrations are performed in a subroutine, attached to the program. Section 3.3.2, covers the calculation of excavation forces in greater detail.

At this point, the program has set up and minimised the stiffness matrix for the complete reinforcement system (unexcavated soil, shotcrete and nails), has calculated the incremental force vector, including forces induced by the current round of excavations and is ready to begin solving. As such, we now enter the iteration loop.

As has already been mentioned, an iterative process is required to solve our equation system because the program allows for plastic soil behaviour. For the first iteration of the first load increment the equations are solved assuming the soil behaves elastically. The new stress state is then calculated using these ‘elastic’ displacements. Plasticity is then checked for, using Sloan and Booker’s Mohr-Coulomb yield surface in conjunction with a Von Mises’ plastic potential surface. If a stress state sufficient to cause plastic failure is found at any of the Gauss points, corrections are made to the load vector to account for the plastic portion of the stiffness matrix, which was left out of the original ‘elastic’ stiffness matrix. The next iteration step finds new displacements using the ‘elastic’ stiffness matrix, and the corrected load vector. The new stress state is then calculated from these new displacements using a constitutive matrix that has been corrected to account for elasto-plastic behaviour \([D_{ep}]\). These new stresses are again checked for any newly formed plastic regions, and the iteration process repeated. The iteration loop will continue, until either a prescribed maximum number of iterations is reached, or the percentage change in two subsequent load
vectors reaches an input convergence criterion. For subsequent load steps, the first iteration is conducted using the $[D_{ep}]$ matrix constructed to model the plastic and elastic elements as they were calculated for the last iteration of the previous load step, i.e. for the first iteration of any load step, the only plastic elements are those that were previously plastic. This process is covered in more detail in section 3.3.1, however there is one pertinent point, which should be mentioned.

Stresses that exceed the failure criterion whilst the soils response is assumed to be elastic are not brought back to the failure surface. Figure 3.3, depicts a situation where a load step has caused an increase in stress from O to B. The load step has caused stresses to exceed the yield stresses by a value AB. For the following load steps, these stresses are forced to track parallel to the yield surface (BC) from the initial yield stress B. This means the stresses for subsequent load steps, still exceed the yield surface by the same distance AB. As such, it is important that the load/excavation steps are sufficiently small, so that increases in stress above the yield surface are not significant with respect to total stresses. This means any excavation needs to be carried out in reasonably small steps in plastic soils.

![Stress diagram](image)

Figure 3.3: Sloan and Booker (1986) modified Mohr-Coulomb yield surface

Once the iteration loop has completed its cycle the program calculates the new stress state and where appropriate, the subsequent stress resultants, (eg: moments in
shotcrete, forces in nail) for all the remaining elements. It then outputs information applicable to the just completed excavation step and restarts the load increment loop. This process continues until excavation is completed.

3.3 Program specifics

Td8auto, is an adaptation of an existing three-dimensional finite element program. Changes have been made in order to allow simulation of excavation, and direct calculation of reinforcement responses. Section 3.3 introduces the base program, outlining its pre-existing abilities and limitations. It then describes the programming changes made to the base program. Changes are discussed chronologically, so that arising problems and eventual solutions can be discussed in their actual context. As is mentioned earlier, the Fortran code for Td8auto and its accompanying subroutines can be viewed in the attached Appendix B.

3.3.1 Pre-existing program

As has been mentioned, t8dauto is an adaptation of an existing three-dimensional finite element program (Td). Whilst numerous changes were made to the base program, some of its features still remain within the skeleton of t8dauto. These features are discussed in section 3.3.1.

Td, the base program for t8dauto, is a general three-dimensional finite element program, written in FORTRAN77. Three types of element can be analysed with Td: an eight node brick element, a twenty node brick element or a thirty two node brick element. For t8dauto, it was only necessary to retain one brick element.

The eight node element, while providing good computational efficiency, was not considered to provide sufficient accuracy. Displacements within this element vary linearly between the nodes. This means the eight node element is not suitable for modelling curved or irregular boundaries. In contrast, the thirty two node element was discarded in the name of computational efficiency. The introduction of the extra
nodes associated with this element, causes significant increases in stiffness matrix bandwidth in addition to increasing the degrees of freedom associated with each element. The accuracy gained by using the thirty two node element instead of the twenty node element, was not considered sufficient to off-set the subsequent increases in required memory.

The twenty node element was chosen, because it provided the most convenient trade off between accuracy and computational efficiency. This 'brick' or 'soil' element, shown in Figure 3.4 with its parent element, is a three-dimensional isoparametric hexahedron. An isoparametric element is advantageous because, both the displacements and coordinates of all points within the element can be interpolated from the nodal coordinates using equations of the form:

\[ x = \sum_{i=1}^{20} N_i x_i \]  \hspace{1cm} (3.2)

\[ y = \sum_{i=1}^{20} N_i y_i \]  \hspace{1cm} (3.3)

\[ z = \sum_{i=1}^{20} N_i z_i \]  \hspace{1cm} (3.4)

Here \( x_i, y_i, z_i \) are the global coordinates of the twenty nodes making up the elements and \( N_{1\rightarrow20} \) are the interpolation or shape functions expressed in terms of the natural coordinate system (STU, Fig. 3.4). Formulations for \( N_{1\rightarrow20} \) are provided in Appendix A. Displacements are assumed to vary quadratically between element nodes, meaning the element is suitable for modeling curved element boundaries, and reduced (2x2x2) or full (3x3x3) integration orders can be assigned.

Figure 3.4: Twenty node 'brick' element.
All the brick elements used by the original program are assumed to have no rotational stiffness. As such, td only has the ability to solve for the three translational degrees of freedom ($\Delta x$, $\Delta y$, $\Delta z$). The shell and bolt elements used in td8auto have six degrees of freedom, three translational ($\Delta x$, $\Delta y$, $\Delta z$) and three rotational ($\theta_x$, $\theta_y$, $\theta_z$). Consequently, td8auto had to have the ability to solve for all six degrees of freedom. The programming of this process is dealt with in detail in sections 3.3.3 and 3.3.4.

Figure 3.5: Sloan and Booker (1986) yield surface in $\pi$-plane.

An elasto-plastic (elastic - perfectly plastic, Fig. 3.1) soil response curve is used by td. Sloan and Booker’s Mohr-Coulomb yield surface is used to describe the failure criterion, and a Von Mises’ plastic potential surface is used to define strain increment orientation at failure. This system has been retained in td8auto.

Sloan and Booker’s yield surface (Figure 3.5), is a revised Mohr-Coulomb surface. When plotted with respect to the principal effective stresses, the Mohr-Coulomb surface (Fig. 3.5) is an irregular hexagonal cone. The stress combinations described by the surface indicate the point at which failure occurs. Problems arise, when an unmodified Mohr-Coulomb surface is used as a failure or plastic potential surface.

The tangent of any point on the plastic potential surface describes the direction of strains upon yielding of the soil. For the points on the Mohr-Coulomb surface, where two planes intersect (point C, Fig.3.5), the tangent to the curve is not uniquely
defined. At this point, the stresses could be traveling perpendicular to either of the two intersecting planes. When stress state straddles the intersection point during iteration, then this lack of uniqueness can cause convergence problems. Sloan and Booker\(^2\) have modified the Mohr-Coulomb surface at these points of intersection by adding a transition surface, so that during the changeover from one plane to another, a unique plastic strain increment is maintained.

Von Mises' circular plastic potential surface (Fig. 3.5) has been used to define strain orientation at failure. One of the major disadvantages of using a non-associated flow rule is that soil dilation upon shearing cannot be modeled. Whilst the action of soil dilation has been shown to be of major importance in the mechanisms of soil-reinforcement interaction observed in actual structures, the exclusion of this soil behaviour from td8auto is not as significant, because the program assumes full adhesion between the soil and reinforcements. That said, the fact that Sloan and Booker's modified Mohr-Coulomb surface maintains uniqueness of the plastic strain increment direction means that an associated flow rule can be easily incorporated into the program. In fact, as part of the parametric studies conducted, a variant of td8auto was developed, which utilised the Sloan and Booker surface for both the yield surface and the plastic potential surface (see Section 5.3.10). This variant was then used to assess the impact of a non-associated flow rule on the calculated facing response.

Td uses the same iterative process as td8auto to calculate the plastic strains induced by each load increment. The iterative loop starts by assuming that only elements that have reached failure prior to the current iteration are plastic. Displacements and subsequent stresses are calculated for the soil, using the soil response appropriate for this degree of plastification. The resultant total stress state is then checked for any yielding. If yielding is found to occur, corrections are made to the load vector and D matrix before the next iteration step is commenced. The iteration loop continues until either a prescribed maximum number of iterations are completed, or the percentage change in two subsequent load vectors satisfies an input convergence criterion. The nature of the corrections made is outlined below.

\(^2\) The Sloan and Booker technique is one of several strategies available for avoiding instability at the corners of the Mohr-Coulomb failure surface.

*Chapter 3: A three-dimensional program capable of simulating soil nailed excavation.*
If a load step results in yielding of the soil, then we can consider the incremental strains induced by this load step ($\Delta \varepsilon_{ep}$) to consist of two parts: an elastic portion ($\Delta \varepsilon_e$), and a plastic portion ($\Delta \varepsilon_p$). This is shown in Equation 3.5

$$\Delta \varepsilon_{ep} = \Delta \varepsilon_e + \Delta \varepsilon_p \quad (3.5)$$

These incremental elasto-plastic strains $\Delta \varepsilon_{ep}$ are related to the induced stresses by an elasto-plastic constitutive matrix $[D_{ep}]$:

$$[\Delta \sigma] = [D_{ep}] [\Delta \varepsilon_{ep}] \quad (3.6)$$

It can be shown that $[D_{ep}]$ can be calculated, by using the elastic constitutive matrix $[D_e]$ and descriptions of the plastic strains $\Delta \varepsilon_p$ provided by the plastic potential surface $P(\sigma,m)$ and the elasto-plastic stresses $[\Delta \sigma]$ by the yield function $F(\sigma,k)$. This calculation gives the result:

$$[D_{ep}] = [D_e] - [D_c] \quad (3.7)$$

where $D_c = \frac{\alpha \beta}{\beta^2 A}$ and 

$$A = \left\{ \frac{\partial P(\sigma,m)}{\partial \sigma} \right\} \quad (3.8)$$

$$\alpha = [D_e] A \quad (3.9)$$

$$\beta = \left\{ \frac{\partial F(\sigma,k)}{\partial \sigma} \right\} [D_e] \quad (3.10)$$

If we now consider the stiffness matrix for the elasto-plastic soil $[K_{ep}]$, we can see how this can be broken into an elastic portion and a correctional portion as follows:

$$[K_{ep}] = \int B^T D_{ep} B dV \quad (3.11)$$

$$= \int B^T (D_e - D_c) B dV \quad (3.12)$$

As such our finite element equations can be written in the form:

$$[K_{ep}] d\delta = dF \quad (3.13)$$

Chapter 3: A three-dimensional program capable of simulating soil nailed excavation.
\[ \{ B^T D_e B dV - \int B^T D_e B dV d\bar{\delta} \} = dF \quad (3.14) \]

\[ \{ \int B^T D_i B dV d\bar{\delta} \} = dF + \{ \int B^T D_i B dV d\bar{\delta} \} \quad (3.15) \]

The elastic constitutive matrix \([D_e]\) is easily calculated. Similarly, it has been shown that plastic correction constitutive matrix \([D_i]\) can be calculated from the elastic constitutive matrix \([D_e]\) and the partial derivatives with respect to stress of the plastic potential and yield surfaces (Equations 3.8, 3.9 and 3.10). Thus it is possible to find the incremental displacements using Equation 3.15.

As has been mentioned, an iterative scheme is used to implement a solution: the correctional stiffness matrix is multiplied by the displacement increment from the previous iteration and added to the load vector at the start of each iteration. The equations are then solved using this new 'corrected' load vector. This process continues until one of the dual convergence criteria are satisfied.

### 3.3.2 Calculation of excavation forces

The simulation of excavation, using the finite element method follows the following simple analogy.

![Diagram](image)

Figure 3.6: Equivalence of unexcavated soil with loaded excavation boundary.

Figure 3.6, shows a body of soil, from which the shaded area (section B) is to be excavated. Prior to excavation, internal stresses inside and outside of the shaded region are in equilibrium, stresses in section A are matched by equal and opposite stresses in section B. The behaviour of the soil in section A, will be identical if the
soil in section B is replaced by forces $\tau$ which are equal to the previous internal stresses at the boundary of B. This equivalence is illustrated in Figure 3.6.

![Diagram showing the equivalence of forces](image)

and

![Diagram showing the addition of forces](image)

so

![Diagram showing the addition of forces](image)

Figure 3.7: addition of forces to simulate excavation of soil.

It follows that the removal of forces $\tau$ will be equivalent to the removal of the soil in section B. In td8auto, the removal of the force $\tau$ is achieved by applying an equal and opposite force $-\tau$ to the unexcavated soil structure. This process is shown graphically in Figure 3.7.

As is mentioned earlier, Brown and Booker (1985) showed that they could calculate the size of the force $\tau$ using the following relationship:

$$
\left( \int_{V_i} (B^T D_{ep} B) dV \right) \Delta a = - \int_{V_i} (B^T \sigma_{i-1}) dV + \int_{V_i} (N^T \gamma) dV + \int_{S_i} (N^T t) dS
$$

(3.16)

Here, $B$ is the displacement strain matrix, $D_{ep}$ the elasto-plastic constitutive matrix, $N$ the shape functions, $\sigma_{i-1}$ the internal stress vector relating to the previous excavation step, $\gamma$ the body force vector and $t$ the traction vector. In td8auto the final expression, relating to the traction vector, is not considered as the effects of any external tractions are incorporated into the initial stresses. The consequences of this are discussed further in chapter 4.
The implementation of this relationship is described below, whilst the Fortran code relating to the calculation of excavation forces is presented in the subroutine integ included in Appendix B.

**Excavation simulation procedure.**

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Figure 3.8: Excavation procedure.

Figure 3.8 shows a mesh of eight elements. Elements number 6,7 and 8 are to be excavated from the mesh one element at a time. Figure 3.8(b) shows the mesh with element 8 removed, Figure 3.8(c) with 7 and 8 removed, and Figure 3.8(d) with 6,7 and 8 removed. The process of simulating the removal of element 8 (moving from Fig. 3.8(a) to Fig. 3.8(b)) proceeds as follows:

1. The stiffness matrix, B matrix and shape functions are calculated for the mesh remaining after element 8 has been excavated (ie mesh of unexcavated elements depicted in Fig. 3.8(b)).

2. Equation 3.16 is solved for all the unexcavated elements (elements 1-7). In this solution, the $B^T, \gamma$ and $\gamma$ relate to the mesh with element 8 removed (Fig. 3.8(b)). However the internal stress vector $\sigma$ relates to those stresses within the soil before element 8 was removed (ie the internal stresses found in Fig. 3.8(a)).

3. The excavation forces calculated in step 2 are then added to the total load vector and used to solve the finite element equations:

$$[K][\delta] = [L]$$
These equations relate to the mesh with element 8 already excavated. So here $[K]$ is the stiffness matrix for the unexcavated elements (elements 1-7, Fig. 3.8(b)). The displacement vector $[\delta]$, which is solved for here, is then used to find the new stress state.

4. The process is repeated for the excavation of elements 7 and 6. Each time the stress state corresponding to the proceeding step is used to work out the excavation forces in the current step (i.e. stress state in Fig. 3.8(b) used to calculate forces imposed on the mesh in Fig. 3.8(c)). These forces are then imposed on the mesh remaining after the current excavation step is completed.

It is mentioned in step 2 that excavation forces are not only calculated at the boundary of the excavation. Rather they are calculated for all the elements remaining unexcavated at the end of the current step. This allows for a crosscheck of the calculation procedure.

Consider elements 3 and 7 in Figure 3.8(a). Stresses on either side of the boundary between elements 3 and 7 are in equilibrium, that is equal and opposite stresses act on either side of the boundary. Subsequently, when we simulate the excavation of element 8, equal and opposite excavation forces are calculated acting along the boundary of elements 3 and 7. The net effect is zero excavation forces along that boundary. The boundary excavation forces calculated due to stresses in element 7 negate the forces from stresses in element 3.

Now, if we consider the boundary between elements 7 and 8 in Figure 3.8(a). Here stresses along the boundary are in equilibrium before element 8 is excavated. However, when we simulate the excavation of element 8, we only calculate excavation forces due to stresses in unexcavated elements. No forces are calculated due to stresses in element 8. As such the excavation forces at the 7-8 boundary, due to stresses in element 7 are unopposed. In this way the appropriate excavation forces are applied to the excavation boundary.
3.3.3 Addition of reinforcing elements

Simulating a soil nailed wall’s incremental construction process, placed a number of stipulations on how the chosen reinforcing elements should interact with the existing brick elements. In order to simulate the construction of reinforcement members in between excavation steps, the program required a system that allows the reinforcement elements to be ‘turned’ on or off as needed. This system ensures that when the program starts each excavation step, it is able to recognise and consider the appropriate reinforcing for that step.

The choice of reinforcing element has a considerable impact upon how easily this process is performed. It was not considered sensible to be adding extra nodes to the mesh during the calculation process. To do so, would adversely affect the computational efficiency. This meant that the nodes for the reinforcing elements needed to be pre-existing in the original unexcavated mesh. This left two methods for incorporating the reinforcing, and subsequently two possible types of reinforcing element.

- The first are the solid type elements. These elements have nodes defining both the top and bottom surface. As such, they require their own element in the mesh. For solid type elements, the installation of reinforcement is simulated by changing the strength properties of the element from those of the surrounding soil, to those of the reinforcement (i.e: from soil to mesh/fibre reinforced shotcrete).

- The second type of reinforcement element considered, was the ‘mid-surface node’ type elements. These elements only have nodes describing the middle surface of the element. This means that if the reinforcement element is chosen so that it matches a portion of the solid ‘soil’ element, then a separate reinforcing element is not required in the mesh. Rather the reinforcing can simply share the appropriate nodes from the ‘soil’ element. For ‘mid-surface node’ type elements, installation of reinforcement is simulated by adding the reinforcement’s stiffness to the appropriate connections of the ‘soil’ element’s stiffness matrix. If we consider the case where a shell element is being added to a soil element. Before the shell element is added, the stiffness of the soil element is calculated from the soil stiffness parameters. After the reinforcement is added, both the soil’s stiffness and the reinforcing element’s stiffness are added into the global stiffness matrix.
For Td8auto, 'mid-surface node' type reinforcing elements were used. As Figure 3.2 shows, the facing element shares the nodes from one face of the brick 'soil' element and the bolt element shares the nodes for any two consecutive nodes. The adoption of these two, 'mid-surface node' type reinforcing elements greatly increased the computational efficiency of the program, by reducing the number of elements required and minimizing the stiffness matrix bandwidth. This is of particular importance in three dimensional modelling.

3.3.3.1 Addition of shell element

Td8auto uses an eight noded isoparametric Midlin shell element to model facing response. Figure 3.10 shows the facing element. The element has four Gauss points (a 2x2 integration scheme), as shown.

![Diagram of shell element with five degrees of freedom per node: three translational (\(\delta_x, \delta_y, \delta_z\)) and two rotational (\(\theta_x, \theta_y\)).](image)

Figure 3.10: 8 node isoparametric Midlin element, used to model facing.

The eight node shell element has five degrees of freedom per node, three translational and two rotational. These are shown in Figure 3.10. The missing rotational degree of freedom relates to in-plane rotations or drilling degrees of freedom. Much has been written regarding the development of shell elements with drilling degrees of freedom, Hughes and Brezi (1989), Sze et al. (1997) however at the time of writing, incompatibility between the existing 6 degree of freedom shell elements and the
twenty node ‘brick’ element used by the program have meant that drilling degrees of freedom have not been considered in the shell elements utilised by td8auto.

The exclusion of the in-plane rotations has restricted the scope of the program. Problems occur when the stiffness matrices for the shell element are transferred from local to global axes. Figure 3.11 shows two cases where a shell element is being added to the top face of a brick element. In Figure 3.11(a), the local axes for the shell are parallel to the global axes, for Figure 3.11(b) this is not the case.

(a) Local axis for in-plane rotation is parallel to a global axis.  
(b) Local axis for in-plane rotation is not parallel to any global axes.

Figure 3.11: Shell element with respect to local and global axes.

When the program calculates the stiffness contribution from the shell elements, it first calculates the shell element’s stiffness matrix, with respect to their local axes. This local stiffness is then transferred to the global axes, through multiplication with a transformation matrix. The in-plane rotations which are being considered, relate to the rotations about the U axis in Figure 3.11. In the local stiffness matrix, there are no entries relating to rotations about the U axis. In case 3.11(a), where the axes are parallel, this means that when the stiffness matrix is transformed to the global axes, there are no entries relating to the rotations about the Z axis, or more importantly there is a zero entry in the diagonal relating to this parameter. Because the zero entry is easily recognizable, the programmed procedure can deal with this situation and by subsequently assigning zero displacement to the parameter ensures that the drilling
degrees of freedom are not considered. In case 3.11(b), where the U axis is not parallel to the Z axis, the transformation of the shell elements stiffness matrix from local to global axes does not result in a zero entry in the diagonal. Instead some component of the other rotational stiffness is entered as the Z rotation stiffness. The relationship between the rotational components is now such that solution of the finite element equations is not always possible, and the program aborts.

In terms of practical restrictions on the program's application, this means that the local axes of the shell elements must remain parallel to the global axes. This ensures that the shell elements lack of in-plane rotational stiffness is recognised by the program and considered prior to solution. For vertically faced excavations this condition is easily met.

Programming of the calculation of the facing element stiffness matrices, transformation to global axes and addition to the mesh stiffness matrix is conducted by the subroutines: shotecr, prelim, dmat, bmat8, dxb and stif8, which are included in Appendix B. Formulations for shell element shape functions relative to the local coordinate axes are provided in Appendix A.

3.3.3.2 Addition of bolt element

T8auto uses a simple two noded beam element to model soil nail response. As presently programmed, only the axial strength of the beam element is considered. This simplification was considered justifiable because, as is discussed in chapter one, tensile forces predominate in soil nailed excavations prior to failure.

Figure 3.12 shows the bolt element. In its full form, the bolt element involves six degrees of freedom per node, three translational (\(\Delta x, \Delta y, \Delta z\)) and three rotational (\(\theta x, \theta y, \theta z\)), however as presently utilised by the program (axial strength only) just the three translational degrees of freedom are considered. As with the shell element, the bolt's stiffness is initially calculated with respect to convenient local axes and then transferred to the global axes via multiplication with a transformation matrix. The bolt
stiffness is then added to the appropriate entries in the 20 node ‘soil’ element stiffness matrix.

Three translational degrees of freedom per node (as used in td8auto).

Figure 3.12: Two noded bolt element.

Programming of the calculation of the bolt stiffness, transformation to global axes and addition to the mesh stiffness matrix is conducted by the subroutine sbolt, included in Appendix B.

3.3.4 Addition of Skyline solver

One of the great costs of three dimensional finite element analyses is reduced computational efficiency. Required memory and program run times are greatly increased when analysis changes from 2D to 3D. There are a number of reasons for these increases:

- The number of nodes required to define each element increases. As an example, consider the twenty node 3D ‘brick’ element used by td8auto, the 2D equivalent of this element is a eight node element.
- The number of elements required to define a problem increases. This is simply a result of increased complexity of the geometry that is being modeled.
- The bandwidth of the stiffness matrix increases. As a result of the extra number of nodes required to define an element and more importantly, the increased difference between the largest node number and smallest node number for a given
element, the size of the bandwidth (the width between the diagonal and ‘last’ non-
zero entry on a line of the stiffness matrix) increases dramatically for 3D analysis.

In the predecessors to td8auto, these factors were exacerbated by the addition of the
shell element. Prior to incorporation of the shell element, only the three translational
degrees of freedom were considered for each node. The addition of the five degree of
freedom shell element meant that each node had to have the ability to consider a total
of six degrees of freedom (three translational and three rotational). As such the
number of total variables that needed to be solved for doubled.

In an effort to increase the computational efficiency of the program and reduce the
memory required to run analyses, two changes were made to the way the program
stores the mesh stiffness matrix.

Firstly, entries corresponding to degrees of freedom that were either subject to
boundary conditions or not considered by the element they were associated with were
not stored in the stiffness matrix. In the case of the degrees of freedom subject to
imposed boundary conditions, entries in the stiffness matrix are not required, as the
solutions for these variables are known a priori (eg: $\delta_x = 0$ or $\theta_z = 0$). Alternatively,
the exclusion of degrees of freedom that are not being utilised by the associated
element, means that entries in the stiffness matrix relating to any degrees of freedom
that are not being used do not need to be saved (eg: the rotational degrees of freedom do
not need to be considered for the twenty node ‘brick’ element used to model the soil
response). Effectively, these two measures take the entries out of the stiffness matrix
for which the solution of the finite element equations are known prior to solving the
equations. To program this, a nodal freedom vector was introduced, which tracked the
status of all the possible degrees of freedom for a given mesh. This matrix was
initialised at the start of the run with the imposed boundary conditions, and then
updated at every load step to account for the changing reinforcement arrangements.

The second measure taken to improve the program’s efficiency was the incorporation
of a variable bandwidth stiffness matrix (Skyline solver). Prior to this change, the
program calculated the maximum bandwidth for the entire stiffness matrix. This
maximum bandwidth was then used for each degree of freedom (Figure 3.13). This
‘constant bandwidth’ procedure resulted in the storage of a lot of unnecessary data. To avoid this, the program was changed so that variable bandwidths could be analysed. The maximum bandwidth for each degree of freedom was calculated and then only the entries within that specific bandwidth are stored. This reduced the total number of entries stored in the stiffness matrix.

To achieve this, an extra vector is required to store the locations of each row in the stiffness matrix. The stiffness matrix is then stored as a one dimensional vector (as opposed to the previous two dimensional array) with each consecutive row of the stiffness matrix stored head to tail. In effect one vector stores the value of the stiffness entry and the second tells you where that value is located.

Again, the appropriate Fortran code can be found in Appendix B. The subroutine ‘stif3d’ assembles the mesh stiffness matrix, ‘fkdiag’ calculates the bandwidth for each degree of freedom and ‘free’ creates the vector matrix which tracks the status of each of the nodal freedoms.

Figure 3.13: Values stored by constant and variable bandwidth stiffness matrix
3.3.5 Variants of the program

Section 3.3.5 briefly details a number of variations and refinements made to the original finished td8auto. In essence, these changes were introduced in order to make the task of verifying the program and conducting a parametric study easier. Two major variations were made:

- The first allowed analysis of a Gibson type soil. The modifications made meant that soil properties could be varied linearly between two prescribed depths, with the program automatically calculating the appropriate soil properties at the average depth of each element. For complex mesh configurations, this refinement significantly reduced the time required to create input data for soils with linearly varying soil properties.

- The second introduced a loop at the start of the program, which enabled multiple input files to be analysed in each run. While the extra programming required to enact this feature was minimal, the ability to analyse multiple input files was of great benefit when conducting extensive parametric studies.

The loop allowing analysis of multiple input files is enabled at the start of the Fortran code for td8auto (Appendix B), whilst the code relating to the analysis of Gibson soils can be seen commented out at the end of the subroutine input.

3.3.6 Input data generators

Also included in Appendix B are an automatic mesh generator (gen3D) and a program that calculates the overburden pressure at each Gauss point of a mesh, for varying surface geometries (overb). Gen3D calculates node numbers, element connections and initial stresses for a simple 3D block mesh from either answers to screen prompts or an input file. The calculated values are output in a format suitable for use as an input file for td8auto. ‘Overb’ can calculate the overburden pressure at every Gauss point in a mesh in layered soils, even if the layer and surface geometry varies over the mesh. Again the purpose of these programs was to simplify the process of creating a suitable input file.
Chapter 4: Program Verification

4.0 Introduction

Chapter three introduced a three dimensional finite element program (td8auto) capable of simulating soil nail wall construction. The abilities and limitations of the program were discussed with reference to the underlying theory and base philosophy. Chapter four outlines the procedures used to validate td8auto. These checking procedures were aimed at establishing two main facts: (a) that the written code was correctly performing the required analyses and (b) that the theory applied by the program is sufficient for the simulation of working stresses in soil nailed excavations.

As with the actual programming, the verifications formed part of an evolutionary process. Checking procedures were ongoing throughout the programming process, generally targeting the section of td8auto added or modified at the time. In deference to this, chapter four presents the main proofing checks conducted in a roughly chronological order, showing how different techniques were used to validate each of the newly added features. Then, having established that the program is correctly conducting the considered analysis, the applicability of this process to the modeling of soil nailed excavations is considered. Comparisons of observed and calculated behaviour from the simulations of a full scale monitored excavation are presented to show that the applied theory can adequately characterize working stresses in a soil nailed excavation.
4.1 Validation of excavation

Chapter three describes how td8atuo was modified from an existing 3D general finite element program td. One of the first major changes made to the original program, was the addition of the ability to simulate excavation. Validation of this feature relied predominately upon known aspects of an excavation’s behaviour. Knowledge of the underlying theory being applied by the program, meant that for certain cases, the program’s predicted behaviour could be compared to theoretically expected behaviour.

In their paper outlining their virtual work equation for the calculation of excavation forces, Brown and Booker (1985) described the use of Ishihara’s postulate as a check of the simulation of excavation in linearly elastic soils. Put simply, the postulate says that ‘when the soil is linearly elastic, the results of excavation should be independent of the number of stages in the excavation process, and lack of such independence indicates an incorrect procedure’. Essentially, the behaviour of the soil is independent of strain history because all strain is reversible in a linear elastic soil.

Ishihara’s method provides a simple check of the simulation of excavation in linearly elastic soils. Analyses whose final states are identical should display identical behaviour, regardless of their stress history. In terms of the presented program, this means that an excavation in soil that does not yield, should give the same final results regardless of whether it is carried out in several steps or one step.

Also described in chapter three is how the programmed code calculates excavation forces for all unexcavated nodes. By calculating these forces at all of the remaining nodes, the program provides another avenue for crosschecking the calculation procedure.

If we consider two adjoining elements, then we know that the stresses at the intersecting boundary must be in equilibrium, stresses at the boundary due to one element have to be opposed by equal and opposite stresses in the other element. Consequently, when we calculate excavation forces along the intersecting boundary,
we should find that the forces due to one element are negated by equal and opposite forces from the adjacent element. If this is not the case then the forces are being incorrectly calculated. In terms of the proposed program, this means a simple check of the programming procedure can be performed by seeing whether the calculated excavation forces are solely restricted to the newly excavated boundary and those nodes with boundary conditions imposed.

Figure 4.1: Excavation simulation procedure

Further to this, the application of excavation forces should result in a stress free excavation boundary. Whether or not this stress free boundary is being produced can be easily verified by checking the excavation forces calculated during the following excavation step. Consider Figure 4.1. Here we have a mesh of eight elements, from which three elements are being consecutively removed. If we check excavation forces calculated to simulate the movement from (a) to (b) (ie: the removal of element eight) then we should find the only non zero excavation forces are on the newly freed boundaries of elements four and seven and at the mesh boundaries. The action of these forces should create a stress free boundary on the free edges of elements four and seven. Consequently when we check the excavation forces calculated to simulate the movement from (b) to (c) (ie: the removal of element seven) then we should see non zero forces at the excavation boundary with elements three and six, but not with element four. Likewise in going from (c) to (d) should have non zero excavation forces bounding elements two and five, but the forces at the excavation boundary of
elements four and three should be equal to zero, as the action of the previous excavation forces should have created zero stress conditions at these locations. This should be the case regardless of the whether soil yielding has occurred.

A final and relatively simple validation of the process used to simulate excavation of elements can be conducted by excavating an entire layer of elements from the top of a mesh. The size of the calculated excavation forces should be equivalent to the weight of the removed layer.

![Diagram of a simple square mesh](image)

**Figure 4.2:** Simple square mesh (triala.dat) used for initial program verification.

Figure 4.2 presents the mesh for the data file, triala.dat. This simple mesh was used to apply the checks listed above. Table 4.1 presents the soil parameters used for these analyses. Note that a very high cohesion has been used in order to ensure that the soil remains elastic. The boundary conditions imposed are as indicated in Figure 4.2. These conditions simulate the construction of an infinitely long, four metre wide trench (axial symmetry has been used to reduce the size of the required mesh).
Figure 4.3 compares the horizontal stresses induced along the plane X=3m, by an excavation conducted using triala.dat in either one step (Figure 4.3 (a)), or 3 steps (Figure 4.3 (b)). As is evident, the results of the two analyses are identical, confirming that the excavation simulation method used satisfies Ishihara’s postulate. Further, the excavation forces calculated in both cases conform to the limits outlined above. Effectively non-zero forces are calculated at all nodes except for those where: an adjoining element has been newly excavated or a boundary condition has been imposed. Additionally, the removal of the top layer of elements resulted in excavation forces equivalent to 1 metre of over burden being imposed at the new surface.

![Figure 4.3](image)

**Figure 4.3:** Horizontal stresses induced by excavation in elastic material conducted in one step (a), and three steps (b).

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</table>

Table 4.1: Input parameters for meshes, triala.dat and trialb.dat.
Figure 4.4 presents the results of analyses (location of plastified Gauss points and horizontal stresses) conducted using the mesh, trialb.dat. Again, the relevant soil parameters are presented in Table 4.1. Trialb.dat is identical to triala.dat, however in trialb.dat, the soil cohesion has been reduced so that soil plastification occurs. Whilst the presence of plastic behaviour now means that Ishihara's postulate no longer applies, the program's ability to simulate excavation in plastified soils can still be checked by confirming that non-zero excavation forces are only calculated at those nodes where: an adjoining element has been newly excavated or a boundary condition has been imposed. Again the analysis conformed to these expected behaviour.

![Diagram](img)

**Figure 4.4:** Results for trialb.dat, (a) Horizontal stresses after excavation (b) location of plastified Gauss points.

Additionally, the location of the plastified elements following excavation is also noted. Figure 4.4 shows their location relative to the mesh elements. As can be seen, the model is simulating the development of a shear zone running from the base of the excavation to a point on the ground surface about 1.5m to 2m behind the face. The development of such a shear zone, is as would be expected and further suggests that the excavation simulation is working correctly.

---

1 Note, actual stresses calculated along the excavated faces in Figure 4.3 and 4.4 were found to approach zero. This is not reflected in the plots due to simplifications in the contouring program.
4.2 Validation of shell element

The next major modification to td8auto was the addition of the eight noded shell element. Checking conducted at this time sought to confirm that: (a) the addition of the shell element did not impact upon the correctness of the excavation simulation and (b) the stiffness due to the shell was being correctly incorporated into the mesh stiffness. Section 4.2.1 presents the validation of the interaction of the shell element and calculation of excavation forces, whilst section 4.2.2 considers the correctness of the combined mesh.

4.2.1 Assessing the impact of the shell element on the calculation of excavation forces

Validation of compatibility between the shell element and the simulation of excavation followed the same procedures that were used to prove the simulation of excavation itself. Firstly, two analyses were conducted in linear elastic soils in order to confirm that Ishihara’s postulate was being satisfied. As before, the two models were identical except for the number of steps used to simulate excavation. Shell elements were placed along the cut face prior to excavation so as to ensure that the two models remained identical at all times. The results of this analyses proved that the addition of the shell element had not affected the uniqueness of the solution in linear elastic soils.

Similarly, inspection of the location of non zero excavation forces further supported the validity of the program by showing that: the action of the excavation forces was producing a stress free excavation boundary and where stresses were in equilibrium equal and opposite excavation forces were being calculated.

Whilst it could be said that this series of checks are reasonably trivial, it is interesting to note that they highlighted a significant problem in the original formulation for the program. In Chapter 3 the calculation of excavation forces using Brown and Booker’s
(1985) virtual work solution is described. As is mentioned, they showed that appropriate excavation forces could be calculated using the following equation:

\[
\left( \int_{V_i} (B^T DB) dV \right) \Delta \alpha = - \int_{S_i} \left( \frac{\delta}{\sigma_{i+1}} \right) dV + \int_{S_i} \left( N^T \gamma \right) dV + \int_{S_i} (N^T t) ds
\]

(4.1)

Of interest is the first term on the right hand side of the equation. This integral basically takes the stresses acting at a boundary before excavation, multiplies them by the transpose of the strain matrix \((B^T)\) for the mesh after the excavation and then sums the resultant over the volume. The implementation of this relatively simple process is complicated in t84auto by the addition of reinforcement elements during excavation. Brown and Booker's equation needs to be solved for all the elements in a mesh. This includes newly added reinforcement elements. Checking of non-zero excavation forces highlighted this need for further changes to the evaluation of excavation induced forces. Reference to appendix B shows how the subroutine integ1 uses Brown and Booker's equation for stresses in all three elements, the 20 noded 'brick' element, the 8 noded 'shell' element and the two noded 'bolt' element.

![Diagram](image)

Figure 4.5: Displacements inducing response in reinforcements. (a) no excavation (b) 1st step excavated no reinforcement (c) 1st step reinforced (d) 2nd step excavated.

Additionally, the checking of the location of non-zero excavation forces highlighted the need to properly define the displacements that induced stresses in each element. As has been mentioned earlier, the addition of new elements during the calculation
process, means that different elements can have different zero stress positions. As an example, we can consider the situation depicted in Figure 4.5. Here a soil nailed excavation is being constructed in a number of stages. As is the practice, the first step is excavated and the reinforcement built on the newly exposed face. Because the reinforcement is placed wet onto the deformed soil, the zero stress position for this layer of reinforcement is the deformed shape of the soil at the time the reinforcement hardens. Stresses in the reinforcement are only induced by subsequent displacement. In terms of Figure 4.5, this means that the stresses in the first layer of reinforcing should be determined using the displacements that have occurred since Figure 4.5 (c). Using total displacements (those that have occurred since Figure 4.5 (a)) to determine the stresses in the reinforcement will result in discrepancies between the stresses in shell element and those in the adjacent soil element.

As is mentioned earlier, stresses in adjacent elements that are not in equilibrium will result in the calculation of non-zero excavation forces at the intersecting boundary of the two elements. It is in this way that the checking of the location of all non-zero forces highlighted inconsistencies in the original formulation of td8auto.

4.2.2 Comparison of the raft and piled raft simulations conducted using td8auto

Having established that the added shell element was properly interacting with the excavation simulation, consideration was given to validating the process used to incorporate the shell element into the global stiffness matrix. For this purpose, comparisons were made between analyses of raft foundations conducted using td8auto and a number of established numerical techniques. The intent of this work was to confirm that the program could model situations were the shell ‘raft’ and the brick ‘soil’ had to interact. This would confirm that the program was properly formulating the shell element stiffness and adding this new stiffness to the existing mesh.

Figure 4.6 shows a problem involving a simple square raft acting under a small uniform load. Analyses of this theoretical raft were conducted using the raft analysis program, FEAR (Small, 2001) and the 3D finite element program developed by the author, td8auto. Sample results from these analyses are presented in Figures 4.7 and
4.8. Note that the four way symmetry of the problem has been utilised to reduce the required meshes and as such the results are presented for the top right hand quarter of the raft only.

![Diagram of raft foundation](image)

Four, 1m x 1m uniformly distributed loads:
\[ w = 10 \text{ kN/m}^2 \]

Axis of symmetry

20m deep layer of elastic 'soil':
\[ E = 20 \text{ MPa} \]
\[ v = 0.4 \]

Axis of symmetry

10m x 10m x 100mm raft foundation:
\[ E = 20000 \text{ MPa} \]
\[ v = 0.3 \]

Figure 4.6: Raft foundation loaded by four uniform column loads.

As can be seen, good agreement was found between the two methods. Contours of displacement and moment for the two methods are nearly identical\(^1\). Figures 4.9 and 4.10, present comparisons of calculated vertical displacements and moments along the X axis (M\(_{xx}\)) for section AA (Figure 4.6). Again very good agreement is found between the two methods.

The ability of td8auto to correctly calculate displacements and moments for this raft foundation problem indicates that the shell element is functioning properly. The interaction between the brick 'soil' elements and the shell 'facing' elements conforms to established norms of soil-structure interaction. It is significant that this be proved at this point because, as was discussed in chapter 2, very little information exists detailing actual facing responses. The fact that the program can correctly model better understood soil-structure interaction problems (such as the case of a raft foundation) indicates that it should likewise be able to model less well understood interaction problems (soil nail facing response).

\(^1\) Variations between the two analyses are likely attributable to minor disparities in the inherent assumptions of the two methods (e.g. FEAR assumes a perfectly smooth raft soil interface, whilst td8auto assumes a perfectly rough interface)
Figure 4.7: Contours of vertical displacement for the top right hand quarter of the raft, simulated using (a) FEAR and (b) Td8auto.
Figure 4.8: Contours of moment around the X axis $M_{xx}$, for the top right hand quarter of the raft, simulated using (a) FEAR and (b) Td8auto.
Figure 4.9: Vertical displacements calculated for section AA.

Figure 4.10: $M_{xx}$ for section AA.

Additionally, whilst not directly related to main thrust of this research, it is worth noting that the author and supervisor (Prof. J. C. Small) have had considerable success using td8auto to model the response of piled raft foundation systems. Small, Zhang and de Ambrosis (2000) presents comparisons of the results of analyses conducted using td8auto and a finite layer theory based program. Two cases were considered, a piled raft foundation subjected to vertical loads and the same raft subjected to
horizontal loads. Figures 4.11 and 4.12 present comparisons of normalised calculated moments for the two cases. Equation 4.2 defines these normalised values.

\[ l_M = \frac{M}{P} \] (4.2)

Here, M is the moment per unit length and P the total lateral or vertical load. As can be seen good agreement is found between the two methods.

![Comparison of normalised moments due to vertical loading](image1)

Figure 4.11: Comparison of normalised moments due to vertical loading

![Comparison of normalised moments due to horizontal loading](image2)

Figure 4.12: Comparison of normalised moments due to horizontal loading
4.3 Validation of bolt element

The final major modification to td8auto was the addition of the two noded bolt element. Again, checking centered on verifying that: (a) the addition of the bolt element did not impact upon the correctness of the excavation simulation and (b) the stiffness due to the bolt was being correctly incorporated into the mesh stiffness.

Again, validation of the compatibility between the bolt element and excavation simulation followed the procedures outlined in section 4.1. Ishihara’s postulate and inspection of the location of non-zero excavation forces were used to indicate the legitimacy of the simulation. Lessons learnt programming the addition of the shell element, meant that this process was relatively non eventful.

![Diagram](image)

Figure 4.13: Simple mesh used for mesh checking (a) with 5m long bolt element (b) with 5m long bolt element and shell elements.

Initial validation of the bolt element relied upon general aspects of the expected interaction between the three elements. This lower degree of verification was sufficient because, unlike the shell element, the response of soil nails in actual excavations has been widely monitored. As will be shown in section 4.4, a number of good benchmark cases exist describing the soil nail’s behaviour. As such, the bolt element’s ability to reproduce this behaviour will provide the ultimate verification of the element.
Figure 4.14: Contours of vertical displacement for section AA with (a) a stiff bolt (b) a relatively flexible bolt and (c) no bolt.

The general verifications conducted sought to confirm that the stiffer bolt element would both redistribute stresses throughout a less stiff ‘soil’ and provide an appropriate point restraint for a shell element. Figure 4.13 presents a very simple mesh used for this purpose. In Figure 4.13 (a) a point load is applied to the head of a bolt element embedded in the ‘soil’. Figure 4.14 presents contours of horizontal displacement along section AA of this mesh with: (a) a stiff bolt \((E_b/E_s = 1\times10^5)\), (b) a relatively flexible bolt \((E_b/E_s = 1\times10^3)\) and (c) without the bolt element. As can be seen, the action of the bolt element helps to distribute the point load throughout the remaining soil mass. Further, as the ratio of the bolt stiffness to soil stiffness \((E_b/E_s)\) increases, the displacements become more uniformly distributed throughout the soil. This confirms that the bolt element is acting to redistribute stresses throughout the soil and importantly the degree to which the stresses are being redistributed is proportional to the ratio of bolt to soil stiffness. As an aside, it is worth noting that nail load distributions similarly confirm the link between \(E_b/E_s\) and the extent of stress redistribution. Figure 4.15 presents axial loads induced in the bolt element by the point load with reference to the two ratios of nail to soil stiffness. As can be seen, the load is dissipated into the soil faster for the case where the nail stiffness is closer to
that of the soil ($E_s/E_s = 1 \times 10^3$). For the larger nail stiffness, the bolt load dissipates at
a slower rate and subsequently the point load becomes distributed over a larger area.
In terms of the contours of displacement presented in Figure 4.14 this behaviour
results in smaller total soil displacements spread over a larger area for the stiffer bolt.

![Diagram of bolt axial loads for stiff and flexible bolts](image1)

**Figure 4.15:** Bolt axial loads for stiff and relatively flexible bolt.

![Diagram of horizontal displacement](image2)

**Figure 4.16:** Horizontal displacement of facing at $x=1\text{m}$ and $y=0\text{m}$.
Figure 4.17: Mxx in shell elements due to uniform load at face.
Figure 4.13 (b) shows a distributed load being applied to shell elements on the free face of the simple mesh. Figure 4.16 presents horizontal displacements along the centreline of the facing (where x=1), without a centre bolt, with a very stiff centre bolt and with a relatively less stiff centre bolt. As can be seen by comparison of the facing displacements for the cases with and without a bolt, the placement of the bolt provides a point restraint for the shell elements lining the face. Additionally, comparison of the cases with bolts of varying stiffness show that the degree of restraint provided by the bolt is proportional to the stiffness of the bolt. This is as would be expected and thus suggests that the bolt element has been incorporated correctly into the mesh. Figure 4.17 presents the moments around the X axis (M_x) induced in the facing for the problem described in Figure 4.13 (b). As can be seen, the point restraint provided by the bolt is similarly mirrored in the facing moments. Again the bolt acts to restrain the movement of the shell and again, importantly, the extent of this restraint is shown to be proportional to the relative stiffness of the bolt.

4.4 Soil Nail Specific Analysis:

The preceding sections have detailed efforts made to ensure that td8auto is correctly conducting the required analyses. Expected theoretical behaviour and comparison with other recognized numerical analyses have established that the program is functioning properly. Section 4.4 now aims to investigate whether the proposed analysis method is appropriate for the simulation of working stresses in a soil nailed excavation.

Chapter three mentions a number of simplifications that have been made during the formulation of td8auto. Aspects of the actual behaviour of the structure have been simplified in the interest of preserving computational efficiency. These simplifications include: enforcement of zero slip between the reinforcements and the soil, a tension only reinforcement response, simple elastic – perfectly plastic soil behaviour and omission of the effects of soil dilatancy. It must be noted that research into soil nailed retaining systems has shown that the soil behaviour described by these excluded
mechanisms undoubtedly plays a role in influencing the behaviour of actual soil nailed excavations. In light of this, it is logical to ask: 'How can we have reasonable confidence in the results of analyses conducted using td8auto if the formulation does not consider a number of aspects of the structure's behaviour that has been observed in actual structures?'

Initial justification for the inclusion of these simplifications developed around two major arguments. Firstly, at the time of writing, machine capabilities were such that when considering such analyses, a choice needed to be made between a sophisticated 2D analysis and a more simplified 3D analysis. As is mentioned earlier, research does exist exploring this trade off. Cardoso and Carreto (1989) considered a soil nailed basement wall constructed in Portugal. Comparisons were made between observed behaviour and that predicted by an 'unsophisticated linear 3D model' and a 'more sophisticated' 2D model. Cardoso and Carreto concluded that for cases where the excavation geometry is such that plane strain conditions are not well met, then a simplified 3D analysis will better predict the system's behaviour than a more complex 2D model. In essence the 3D effects have more bearing on the system's response than the actions of the soil behaviour excluded in the simplified model. As is mentioned earlier, the author contends that facing response is an inherently 3D mechanism. To simplify the system to 2D would be to treat the reinforcements as horizontally continuous sheets not discrete bars. This would result in the analysis discounting the influence of bending horizontally between nails. Other authors have similarly recognised the intrinsically 3D nature of soil nailed excavations, Unterreiner et al. (1997) assert 'a soil nailed wall has a three dimensional geometry which cannot be modeled with a 2D model which assumes plane strain conditions'.

Further to this, difficulties determining appropriate input parameters for more complex models can negate the benefits of any additional numerical accuracy. Unterreiner et al (1997), highlight work conducted by Quaresma et al. (1993) studying soil-geotextile interfaces. They report that 'the use of complex constitutive models does not seem to improve the quality of the calculated displacements. Indeed, the accuracy which can be gained with an increasing number of constitutive parameters is lost by the uncertainties existing in these parameters'. In essence our ability to
characterize a soil’s behaviour prior to construction does not warrant the use of overly complex constitutive models.

The second major reason for using a less complex 3D analysis is that the program was only intended to be used to investigate the working stresses induced in the facing. Chapter one outlines how extensive research has concluded that 'the mechanism of skin friction interaction is preponderant in soil nailed retaining structures and develops with small deformations' (Recommendations Clouterre, 1991). This research suggests that for the majority of soil nailed structures, the aspects of the soil’s behaviour discounted by the enforcement of: zero slip between the reinforcements and the soil, a tension only reinforcement response and omission of the effects of soil dilatancy should not greatly influence the behaviour of the soil adjacent to the facing and as such the exclusion of these forms of soil behaviour should not greatly influence a working stress analysis of facing pressures experienced by soil nailed excavations.

In summary, the decision to use a simplified 3D model was made because it was believed that the 3D aspects of the system would have a greater influence on the system’s behaviour than the soil behaviour excluded by the 3D analysis.

That said, it is still reasonable to expect that the correctness of this assumption be explored. As a way of ascertaining the legitimacy of this preceding argument, section 4.4.3 presents the results of a simulation of a full scale monitored excavation conducted as part of the French National Research Program, Clouterre. Importantly, in order to provide an objective assessment of the program’s abilities, variables used in the simulation will be justified with respect to published testing conducted at the test wall site prior to excavation. It will be shown that analyses conducted using tdsauto can closely predict specific aspects of the observed behaviour when site characterisation is sufficiently comprehensive. Additionally however, some short comings of the presented analysis method, highlighted as a result of this simulation, will also be discussed.
4.4.1 First Experimental Wall Completed for CLOUTERRE - CEBTP Experiment No.1

In 1986, a national research project into soil nailed excavation support systems was commenced in France. Pivotal to this project, were three full scale monitored test walls conducted at the Centre Expérimental de Recherches et d'Etudes du Bâtiment et des Travaux Publics (CEBTP) facilities between 1986 and 1989. These three test walls provide an important benchmark describing the behaviour of full scale soil nailed retaining systems. Of interest to this thesis, are the observed displacements and nail loads recorded during the construction of the first test wall, CEBTP No.1.

The construction, monitoring and simulation of CEBTP No.1 have been described in a number of publications (Recommendation CLOUTERRE 1991, Plumelle et al 1989, Benhamida et al 1997, Plumelle et al. 1990, Unterreiner et al. 1997). The soil nailed wall was constructed in a 7m high, purpose built embankment at the CEBTP site (Figure 4.18). Two lateral walls covered by a greased double layer of polyethylene sheets were used at the ends of the embankment to ensure plane strain conditions were met.

Figure 4.18: Dimensions of CEBTP Wall No.1 conducted as part of Clouterre
Figure 4.18 shows how the soil nailed wall was constructed into the embankment in 1m high lifts. Nails were constructed out of aluminium tubes of varying lengths and cross sectional area (see Figure 4.18). They were placed approximately 10° off the horizontal using drill and grout techniques. Horizontal and vertical nail spacing was equal to 1.15m and 1m respectively. A 100mm thick, mesh reinforced shotcrete facing was used, with connection between the nails and facing being made via a bolted steel plate. Section 4.4 only considers construction of the wall to a height of 5m. This intermediate height has been chosen in order to negate the influence of considerable creep movements observed during a two month construction hiatus between phases five and six.

Construction induced responses were measured using extensive instrumentation installed on the wall and in the embankment. Facing displacements were recorded using survey marks attached to the facing during construction. Three inclinometers placed 2m, 4m and 8m behind the face, recorded movements of the soil mass (Figure 4.18) and strain gauges placed at 0.5m intervals along the length of the nail were used to measure tensile forces induced throughout the nails.

One of the major aspects that make these observations so important as a benchmark performance was the care taken to ensure that the soil strength parameters could be comprehensively characterised prior to construction of the wall. Unreiner et al. (1990) provides an excellent description of the efforts taken to characterise the site. Construction of the embankment was tightly controlled in order to ensure homogeneity and uniform density throughout the fill. The fill material used consisted of medium to dense Fontainebleau sand (relative density of 0.6). Fontainebleau sand has been widely studied and a large database of testing exists describing its behaviour. Insitu Menard Pressuremeter testing and laboratory triaxial tests complemented this database material allowing realistic Young’s Modulus and shear strength values to be determined. A small amount of fine material (about 5% equal to or less than silt size) was present in the sand, which resulted in the material being slightly cohesive. Comparisons of water content and percentage fines between material used in the embankment and that tested in previous projects allowed determination of an appropriate apparent cohesion.
4.4.2 Description of the model used to simulate construction of CEBTP wall No.1

Figure 4.19 presents the three dimensional finite element mesh used to simulate the construction of phases one to five of the monitored CEBTP wall No.1 described in section 4.4.1. As can be seen, the mesh utilises the plane strain conditions set up in the test wall (boundary conditions along the faces where x=0 and x=1.15 simulate an infinitely long embankment). This was done in order to reduce the size of the required mesh. That said, the nature of the mesh is such that the 3D aspects of the face loading and soil displacements between the discrete nail supports could still be considered. This was achieved by making the mesh one nail spacing wide (ie; the mesh covers 1.15m of the x direction) and applying boundary conditions which simulate this mesh continuing to positive and negative infinity on the x axis (Figure 4.20 shows the boundary conditions applied).

![3D finite element mesh used to model CEBTP Wall No.1](image)

Figure 4.19: 3D finite element mesh used to model CEBTP Wall No.1

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reported</th>
<th>Simulation</th>
<th>Method of determination</th>
</tr>
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<tbody>
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<td>36°-40°</td>
<td>38°</td>
<td>Triaxial tests</td>
</tr>
<tr>
<td>$c$</td>
<td>3-4 (kPa)</td>
<td>3 (kPa)</td>
<td>Triaxial test, water content and % fines</td>
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<td>$K_o$</td>
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<td>0.38</td>
<td>Jáky’s formula, $K_o=1-\sin\phi$</td>
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<td>$\gamma$</td>
<td>16.1 (kN/m$^3$)</td>
<td>16.1 (kN/m$^3$)</td>
<td>Compacted average unit weight</td>
</tr>
</tbody>
</table>

Table 4.2: Comparison of reported and simulated soil strength parameters.

Chapter 4: Program Verification.
The twenty noded ‘brick’ element introduced in chapter 3 is used to model the bulk of the embankment, with the 8 node shell element and the two node bolt elements being used to model the shotcrete facing and grouted nails respectively. The task of assigning appropriate soil strength parameters to these brick elements was greatly simplified by the comprehensive insitu and laboratory testing and controlled construction techniques applied as part of Clouterre. The nature of the work conducted to characterise the site meant that a high degree of confidence could be placed in the reported values (Unterreiner et al. 1997) and as such the simulation strived to mirror these reported values wherever possible. Table 4.2 shows comparisons between the soil parameters used in the simulation and those reported in the literature. Reinforcement parameters were similarly chosen to mirror reported values. Table 4.3 summarises the reinforcement stiffness parameters used in the simulation, again comparing them to values reported in Unterreiner et al. (1997).

Figure 4.20: Boundary Conditions applied to CEBTP Wall No.1 simulation.

Values of Young’s Modulus for the constructed fill embankment, were determined using the reported results of Menard Pressuremeter testing. Figure 4.21 presents a comparison between the measured and simulated Young’s Modulus as it varies with depth within the embankment. These values rely upon a ratio of $E_s/E_m$ (the ratio of the
soil modulus to the pressuremeter modulus) of 2.2. Unterreiner et al. (1997) report that compression triaxial testing indicated that the $E_s/E_m$ ratio varied between 2.2 and 2.8.

<table>
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<th>Reported</th>
<th>Simulation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
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<td>dia.</td>
<td>16 – 40 (mm)</td>
<td>16 – 40 (mm)</td>
</tr>
<tr>
<td>length</td>
<td>6 - 8 (m)</td>
<td>6 - 8 (m)</td>
</tr>
<tr>
<td>E</td>
<td>$70 \times 10^6$ (kPa)</td>
<td>$70 \times 10^6$ (kPa)</td>
</tr>
</tbody>
</table>

Table 4.3: Reinforcement stiffness parameters.

![Esoil Depth Diagram](image)

Figure 4.21: Comparison of measured and simulated $E_{soil}$ ($E_s/E_m = 2.2$)

Initial stresses in the embankment were modelled as geostatic. The soil unit weight of 16.1 kN/m$^3$ is as reported and the at rest coefficient of lateral earth pressure $K_o$ was calculated using Jáky’s formula for normally consolidated soils (1944).

$$K_o = 1 - \sin \phi$$

Excavation was simulated in 13 stages, with each stage corresponding to the removal of one layer of elements. Two methods exist for modelling the construction of the

---

2 Overburden stresses were calculated as unit weight of the soil multiplied by the vertical distance to the ground surface.
reinforcement using td8auto and it is worth considering the implications of each, as they have some bearing upon the applicability of any simulation of a soil nailed system.

As is mentioned in Chapter 1, the actual construction of a soil nailed wall is an incremental process. Excavation occurs in distinct layers. Each layer is dimensioned such that, when the layer is excavated, the newly exposed face is self supporting for the period it takes to construct the reinforcement for that layer. Excavation of subsequent layers occurs with the reinforcing for preceding layers already installed. Importantly, each construction lift utilises the favourable short term strength characteristics of the soil in order to remain stable. The idea is that the reinforcement for the current lift is placed and begins to accumulate stress before enough time has passed for the long term soil strength characteristics to start governing the behaviour of the excavation (that is before the benefits of any negative pore pressure or apparent cohesion have dissipated).

In the presented method of excavation simulation, the passage of time is only considered through the use of load/excavation steps. The excavation sequence is broken into a series of steps that mirror the actual excavation sequence as much as is practicable. Because we are interested in the long term response of the final system, soil strength parameters that model the long term behaviour of the soil are used in the analysis. No provision exists for considering the situation where favourable short term soil characteristics exist and are then reduced over time. This means that when an excavation step is performed, the response of the cut is as if the cut has been left long enough for the long term soil strength to govern the behaviour, the time dependent nature of the displacements is not being modeled. In terms of the reinforcements, this means that two possible methods exist for simulating their construction, either the reinforcements are considered to act before the current excavation step is started, or alternatively they are placed after all the displacements associated with the current step have occurred.

Figure 4.22 illustrates these two simulations. For case A, the reinforcement is placed after all the displacements for the current step have occurred. Note that for this case the stresses in the reinforcement are only induced by further excavation. For case B,
the reinforcement for the current step is added before the excavation has occurred. In effect no soil displacements occur before the reinforcement is constructed. In this case, the stresses in the reinforcements are induced by displacements associated with the current step as well those due to following steps.

As a check on the applicability of either simulation, it is worth noting that the actual behaviour of an excavation is unlikely to exactly equal either of these cases. Rather it is more likely to sit somewhere between these two extremes, with some but not all of the displacements associated with a particular step occurring before the reinforcement begins accumulating stress. As such, these two cases really represent the limits of the actual behaviour.

**Case A:** Reinforcement response is only due to the excavation of underlying layers.  
**Case B:** Reinforcement response is due to the excavation of both the current step and any underlying layers.

Figure 4.22: Excavation simulation methods.

That said, there are certain recognisable characteristics of any excavation, which will push the actual behaviour closer to either of these two limits. Most obviously, the type
of soil will affect which case is more applicable. Sands, being faster draining, would
typically be expected to display behaviour closer to case A. Clays on the other hand
take longer to dissipate negative pore pressures and as such are more likely to display
responses closer to case B. Construction related issues similarly have a huge influence
on which case is more applicable. Firstly, the simple matter of what delay has been
programmed between the excavation of the soil and the construction of the
reinforcements is important. Further, the use of slotted excavations in unfavorable soil
(Figure 1.10) or any such three dimensional support techniques (proximity to
temporary returns in the wall) will reduce the percentage of the final displacement that
will occur before the reinforcements are installed and as such push the soil response
closer to that simulated in case B. The nail installation method is also of significance.
Driven or fired nails start acting immediately, whilst drilled and grouted nails have to
wait for the grout annulus to harden before they can start accumulating appreciable
stress, consequently driven nails are more likely to respond in a manner similar to
case B than drilled and grouted nails. When simulating the construction of a soil
nailed wall, all of these features should be considered, so that the most correct
simulation method can be identified.

In terms of our simulation of CEBTP wall No.1, section 4.4.1 mentions that the
embankment fill consisted of slightly cohesive sand. Additionally, drilled and grouted
construction methods were used to install the nails. As is discussed above, these two
features indicate that Case A is most likely to represent the actual behaviour of the
wall. In keeping with this the construction of CEBTP wall No.1 was simulated by
assuming that all of the displacements associated with the current step occur before
the reinforcements for that step are placed.

4.4.3 Results of the simulation of CEBTP wall No.1

Section 4.4.3 presents the results of the simulation of CEBTP wall No.1 described in
section 4.4.2. Calculated displacements and nail loads are compared with the observed
wall response.
As has been mentioned in section 4.4.1, the CEBTP wall No.1 used three inclinometers located 2m, 4m and 8m behind the face and survey marks attached to the face to monitor excavation induced displacements. Unterreiner et al. (1997) present displacements recorded at the face and within the retained soil mass at two stages during the excavation, with the wall at a height of 3m and 5m. Figure 4.23 presents a comparison between these observed displacements and those calculated using td8auto. Figure 4.23 (a) presents observed and calculated displacements at both stages for the facing whilst Figures 4.23 (b)-(d) gives displacements at the three inclinometers 2m, 4m and 8m behind the face respectively.

The calculated displacements corresponding to the inclinometer readings are the total displacements calculated for a line of nodes situated the required distance behind the face of wall and in between two nails. As an example, the displacements presented in Figure 4.23 (b), are those calculated for a line of nodes 2m behind the face and half way between the soil nails at x=0m and x=0.575m. This position was chosen, because it most accurately mirrors the actual placement. Further, total displacements of these nodes have been reported. Common practice for inclinometers is to grout the casing in to a borehole drilled into the embankment, prior to construction of the wall. This means that the inclinometers are recording the total displacements that have occurred since installation and thus their recorded movements should be compared to total calculated displacements.

For the facing survey marks, the survey targets are attached to the newly constructed face. Any displacements that have occurred prior to the first survey readings of the target are discounted and become the ‘zero’ or initial reading for the facing. The recorded readings of facing displacement are simply the displacements that occur after this initial reading. In keeping with this, the calculated displacements presented in Figure 4.23 (a) are not the total calculated displacements, rather they are the displacements that have been induced by excavation of under lying layers, presented with respect to the last survey mark. The only exception to this is the calculated facing displacement corresponding to the ground surface (Z coord. = 12m). This value is the total soil displacement induced by excavation, and is in keeping with a survey mark constructed on the surface prior to excavation.
Figure 4.23 (a): Displacements at facing (b): Inclinometer 1 – 2m behind the facing (c): Inclinometer 2 – 4m behind the facing (d): Inclinometer 3 – 8m behind the facing.
As can be seen from Figure 4.23, there is very good agreement between the observed and calculated values at the facing and 1st inclinometer (2m behind face). Both the displaced shape and displacement magnitudes observed have been mirrored in the results of the simulation. Importantly this agreement is equally good for a wall height of 3m as it is for wall height of 5m.

For the 2nd inclinometer (4m behind the face), good agreement is found when the wall is at a height of 3m, however discrepancies exist between the observed and calculated displacements when the wall is at a height of 5m. As can be seen, the observed displacements appear to move away from the excavation between phases 3 and 5. This fact would suggest that some error has occurred with the inclinometer readings for displacements at phase 5. In fact, good agreement between the observed and calculated displaced shapes above a z-coordinate of 6m, suggests that an error has occurred for the readings below this point.

Finally, for the 3rd inclinometer, located 8m behind the face, a reasonable agreement is found between the observed and calculated displacements at the end of phases 3 and 5. Note should be made of the small magnitude of the displacements being measured (less than 1mm). As an example of the type of accuracy reasonably expected of an inclinometer, Sinco (Slope Indicator Company) Digitilt Inclinometer Manual (Model 50309-M indicator with 50325-M sensor) suggests that deflection errors over a 30 meter hole should not exceed ±7.5mm. For the reported readings, which are presented over a depth of 12m, this correlates to a acceptable error of ±3mm. In spite of these variations, there is reasonable agreement between the magnitudes of the displacements.

In summary, generally excellent agreement has been found between observed and predicted displacements for the CEBTP wall No.1. Some variation does exist between the observed and predicted displacements for inclinometers 2 and 3 (4m and 8m behind the facing respectively), however these irregularities can be reasonably accounted for as inconsistencies in the monitored results. Importantly, the agreement is best at and nearby the face.
Figure 4.24: Comparison of calculated and observed nail forces (a) Nail 1 (z coord. = 11.5m) (b) Nail 2 (z coord. = 10.5m) (c) Nail 3 (z coord. = 9.5m) (d) Nail 4 (z coord. = 8.5m).
Figure 4.24 presents calculated and observed forces in the nails for CEBTP wall No.1. Here tensile forces are positive. As has been mentioned earlier in this chapter, actual nail forces were measured using strain gauges placed at half meter intervals along the nail. These are compared to the tensile forces developed in the simple 2 noded bolt elements described in chapter 3.

As can be seen, the agreement between the magnitudes of the calculated and observed nail forces ranges from very good to fair. For nail 1 the correlation is very good, with the calculated values closely mirroring the observed. For nails 2 and 4 the correlation is not as good as that observed with nail 1. For both of these nails, the calculated nail force values over-predict the observed values. For nail 2, the calculated maximum nail force is 142% of the observed maximum nail force, whilst for nail 4 it is 123% of the observed maximum nail force. The worst correlation is found for nail 3. Here the calculated maximum nail force is approximately 175% of the observed maximum. Additionally, it is worth noting that for three of the four nails, the simulation has over-predicted the size of the maximum forces induced by excavation. Further, for nails 3 and 4 we see that in addition to over-predicting the maximum, the simulation has under-predicted the forces towards the end of the nail. It is possible that the exclusion of slip between the soil and reinforcement has caused this discrepancy. This mechanism of slip will have the effect of distributing stresses from regions of higher stress to regions of lower stress and as such an analysis, which discounts the mechanism of slip, will most likely over-predict the forces in areas of high stress and under-predict the forces in areas of low stress.

More encouraging is the program’s ability to correctly predict the shape of the induced forces along each nail. As can be seen, the calculated values in general provide a very good prediction of the location of the maximum nail load and the subsequent shape of the load distribution curve. This fact is quite significant. Schlosser (1971) introduced the idea of two distinct zones (the active and resistant zones, see Figure 1.5) forming in the earth mass behind a reinforced earth wall. In essence, these two zones distinguish between the unstable soil that is being retained by the reinforcing and the remaining stable embankment. The delineation between these two zones can be distinguished using the location of the maximum nail loads. Figure 4.25 illustrates this concept graphically. As can be seen, the relative
movements between the soil and the reinforcements are such that the frictional forces developed at the soil-reinforcement interface act towards the excavation in the active zone, and away from the excavation in the resistant zone. Consequently, the tensile forces developed in the reinforcement as a result of these frictional forces, are at a maximum at the boundary between the active and resistant zones. This means, that by correctly predicting the distribution of nail forces along the nail, the program is forming the same failure wedge geometry as was observed in the actual wall.

![Diagram](image.png)

Figure 4.25: Nail tensions induced in the proximity of the shear zone.

As an aside, it is useful to place these results in the context of other recently reported simulations of the same wall. The French researchers, Benhamida, Unterreiner and Schlosser present, in two separate papers (Unterreiner et al. (1997) and Benhamida et al. (1997)), two separate simulations of the CEBTP wall No.1. Both of these simulations were conducted using the same finite element code, CESAR-LCPC. Figures 4.26 and 4.27 present comparisons of the displacements and nail forces calculated using: the td8auto simulation presented in this thesis, the CESAR-LCPC simulation described in Unterreiner et al. (1997), the CESAR-LCPC simulation described in Benhamida et al. (1997) and those observed during the construction monitoring of CEBTP wall No.1.
Figure 4.26: Comparison of observed and calculated displacements (a) at the face (b) 2m behind face (c) 4m behind face (d) 8m behind face.
Figure 4.27: Comparison of calculated and observed nail forces (a) Nail 1 (z coord. = 11.5m) (b) Nail 2 (z coord. = 10.5m) (c) Nail 3 (z coord. = 9.5m) (d) Nail 4 (z coord. = 8.5m).
Before discussing the results from these three simulations, it is useful to first consider the differences between the three analyses.

The analysis method used by CESAR-LCPC differs from that used in td8auto in two major ways. Firstly, CESAR-LCPC performs a 2D analysis. Unterreiner et al. (1997) and Benhamida et al. (1997) describe how the authors use equivalence relationships to reduce the 3D soil nail installations to a 2D case. Reductions in: Young's Modulus, limit elastic stress and cross sectional area of the nails, are made in such a way as to preserve similarity between the 2D and 3D case. Secondly, CESAR-LCPC allows discontinuities in the displacements between the soil and the reinforcement elements. Simple interface elements which model linearly elastic, perfectly plastic slip behaviour are placed between the soil and reinforcement elements, allowing the effects of slip between the two elements to be approximated. Interestingly, the CESAR-LCPC analysis presented in Unterreiner et al. (1997) further augments this through the use of changed soil parameters in the elements adjacent to the facing. This is done in an attempt to simulate disturbance to the embankment caused during excavation of the face. In contrast, the simulation used by td8auto assumes full adhesion between the soil elements and both reinforcing elements (soil nail and facing).

<table>
<thead>
<tr>
<th></th>
<th>Unterreiner et al. (1997)</th>
<th>Benhamida et al. (1997)</th>
<th>Simulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_s$</td>
<td>20E03 (kPa) $E_s/E_m=2$</td>
<td>31E03 (kPa) $E_s/E_m=3$</td>
<td>varies - $E_s/E_m=2.2$</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>16.1 (kN/m$^3$)</td>
<td>16.6 (kN/m$^3$)</td>
<td>16.1 (kN/m$^3$)</td>
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<tr>
<td>$\nu$</td>
<td>0.33</td>
<td>0.39</td>
<td>0.33</td>
</tr>
<tr>
<td>$\phi$</td>
<td>38°</td>
<td>38°</td>
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<td>$\psi$</td>
<td>25°</td>
<td>27°</td>
<td>0</td>
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<tr>
<td>$c$</td>
<td>3 (kPa)</td>
<td>3 (kPa)</td>
<td>3 (kPa)</td>
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Table 4.4: Comparison of major input parameters for CEBTP Wall No.1 simulations.

Another significant difference between the three analyses relates to the interpretations of the pre-construction testing adopted for each simulation. Table 4.4 presents comparisons of the input parameters for the two simulations conducted using CESAR-
LCPC and the analysis conducted using td8auto. Whilst in general, it can be said that, all three analyses use very similar input parameters, there is some difference between the Young’s Modulus adopted in each simulation. The reason for this discrepancy lies in each simulation’s interpretation of the pressuremeter results. The model presented in Unterreiner et al. (1997) uses one ‘representative’ Young’s Modulus to describe the behaviour of the entire embankment. Importantly, this Young’s Modulus value was interpreted from the pressuremeter results using an assumed soil modulus to the pressuremeter modulus (E_s/E_m) ratio of 2. Benhamida et al. (1997), similarly use one ‘average’ Young’s Modulus to describe the behaviour of the embankment, however for their case they adopt a soil modulus to the pressuremeter modulus ratio (E_s/E_m) of 3. In the simulation conducted using td8auto the Young’s Modulus mirrors the pressuremeter readings by increasing with depth (Figure 4.21). Further, for td8auto, a soil modulus to the pressuremeter modulus ratio (E_s/E_m) of 2.2 has been adopted.

As a way of placing these varying assumptions in context, it is worth noting that in Unterreiner et al. (1997) it is reported that compression triaxial testing indicated that the ratio E_s/E_m varied between 2.2 and 2.8.

Having outlined the major differences between the analyses, we are now able to turn our attention to the results. Comparison of the observed and calculated displacements presented in Figure 4.26 reveals two major aspects of the results that are worthy of note. Firstly, significant differences exist between the two 2D simulations described in Unterreiner et al. (1997) and Benhamida et al. (1997) and secondly, in general, the displacements calculated using td8auto are in better agreement with the observed results than either of the results of the two CESAR-L CPC simulations.

In terms of the first point, the variations between the two CESAR-L CPC simulations, would seem to suggest that what were initially considered minor differences between the two models (the difference between adopting an E_s/E_m ratio of 2 rather than 3, when testing indicated a variation between 2.2 and 2.8 could be reasonably expected and the modeling of some soil disturbance caused by soil excavation close to the face) ended up having a significant impact upon the calculated displacement response of the excavation. This point has some bearing with regard to how well a soil needs to be characterized prior to excavation if numerical modeling is to be relied upon to give

Chapter 4: Program Verification.
displacement predictions. As has been mentioned, the CEBTP wall No.1 site underwent extensive testing to characterize the site, yet variations in the interpretations of these tests have resulted in significantly varied predicted wall responses. This highlights the importance of properly characterizing the site and further indicates the variation in predicted response that can be reasonably expected, even when extensive site testing has been conducted.

In terms of the second point, the fact that the displacements calculated using the program presented in this thesis, generally better predict the observed displacements seems to suggest that the advantages of more a complex characterisation of the soil-nail, soil-facing interface are outweighed by the errors introduced by either: reducing the 3D wall to a 2D simulation, assigning correct interface soil parameters or by using only one ‘average’ Young’s Modulus in a soil which displays increasing stiffness with depth. In fairness to Unterreiner et al. and Benhamida et al., it is important to note that even with the enforcement of an equivalence relationship, the 2D analysis can not model soil bulging between nails. This point is quite important. Put simply, the calculated 2D displacements are only an ‘average’ of more complex 3D displacements. This means it may not be fair to compare the displacements calculated by the 2D analysis solely to displacements observed at one location on the wall (e.g. in between the nails). If we recognise that the 2D analysis is providing us with average displacements then to ascertain the correctness of this analysis, we should be comparing it’s output to average observed behaviour. It is possible that a 3D analysis could better predict displacements observed between nails, even though the 2D analysis is providing a more accurate description of the ‘average’ behaviour of the wall.

That said, it should likewise be noted that the 2D analysis is not modeling the complex 3D nature of the wall’s response and as such, in instances where this 3D response has a large influence on the system’s behaviour (e.g. at the facing, 3D aspects of the soil displacements), a 2D analysis will not be sufficient. This point is significant, the main intention of this thesis is to explore the response of facings of soil nailed excavations. Comparisons between the observed displacements and the results of 2D and 3D simulations are supporting the original contention that a three
dimensional simulation is necessary in order to properly model soil response at and near the facing.

Continuing the comparison of recent simulations of CEBTP wall No.1, attention is drawn to Figure 4.27. Presented here is a comparison of the observed and predicted nail forces at phase 5 of the excavation. As can be seen, the quality of the predicted nail forces varies significantly between each simulation and between each nail. All three simulations provide a good prediction of nail force size and distribution for nail number 1 (the uppermost nail) and a reasonable prediction for nail 4 (the bottom nail). However for nails 2 and 3, a significant variation in the correctness of the three simulations exists.

In general, the results of Unterreiner et al.’s 2D analysis are in the best agreement and Benhamida et al.’s the worst agreement with the magnitude of the observed nail forces. For nails 2 and 3 Unterreiner et al. provide calculated maximum nail forces that are approximately 124% and 156% of the observed maximum nail force respectively. The td8auto simulation predicted values are approximately 142% and 175% of the observed maximum, whilst Benhamida et al. present forces approximately 182% and 213% of the observed maximum.

With regard to the nail force distribution (i.e. the shape of the nail force diagram), in general it can be said that the 3D simulation of td8auto best predicts the location of the maximum nail force, with Unterreiner et al. providing the worst correlation. As is discussed earlier, the ability to predict the location of the maximum nail force is significant, because the location of the maximum nail force gives an indication of the geometry of the failure wedge.

Again attention is drawn to the significant differences between the results of the two analyses conducted using the 2D CESAR-LCPC program. As was discussed previously, this degree of variation from two analyses conducted using the same computer code and soil characterisation testing is significant. As before, these discrepancies highlight the level of accuracy that should be expected from numerical predictions of soil nail wall behaviour. The difference between the two illustrates the level of variation that can be expected as a result of commonly accepted differences in interpretation of the soil testing. Further to this and of particular interest in this thesis,
the fact that the 3D td8auto simulation generally lies between the two 2D CESAR-LCPC simulations has particular bearing upon how important the modeling of the mechanisms of soil-reinforcement slip is to either of the two CESAR-LCPC simulations. Essentially, the td8auto simulation has provided a better prediction of the nail’s behaviour than that presented in Benhamida et al. (1997), even though an assumption of full adhesion between the soil and reinforcements is used. This is suggesting that for this case, how you interpret the soil testing (and thus assign soil mass and interface element properties) has more bearing upon the correctness of the simulation than the modeling of displacement discontinuities at the soil-reinforcement interface. This point harks back to a point made within Unterreiner et al. (1997) that was discussed earlier in this chapter. ‘The accuracy which can be gained with an increasing number of constitutive parameters is lost by the uncertainties existing in these parameters’. To be fair, it needs to be noted that these results are for an intermediate wall height (5m of a 7m deep excavation). The reinforcements that have been constructed are designed to retain the 7m deep excavation and so the factor of safety of the intermediate 5m wall is likely to be larger than that of the finished wall. Slip at the soil-reinforcement interface is likely to become more critical as the soil nail loads approach their ultimate values. As such it is possible that the variations between the two CESAR-LCPC simulations will become less significant when compared to the td8auto simulation, as the excavation progresses and the nail loads approach their ultimate values.

Notwithstanding, there is additional evidence supporting the idea that the simple interface elements presented in Unterreiner et al. (1997) and Benhamida et al. (1997) are not properly modeling the actual behaviour of the soil nail interface. Earlier in this section it was theorised that td8auto’s tendency to over predict nail forces in areas of high stress and under predict them in areas of low stress indicated that the programs enforcement of zero slip between the soil and reinforcements was introducing errors. But now we can see that simulations presented by Unterreiner et al. (1997) and Benhamida et al. (1997), which allow for differential displacement between the soil and the reinforcement, also display this behaviour. In fact, for the simulation presented by Benhamida et al. (1997), the calculated behaviour over-predicts the observed values in areas of high stress and under predicts them in areas of low stress to a greater extent than the td8auto simulation. This suggests that the interface
element is not redistributing these high stresses as it should. The fact that the results of Unterreiner et al. (1997) generally display this behaviour to a lesser extent than the results of td8auto, may suggest that through a refinement process, these researchers have been able to more closely, model the actual slip behaviour.

Another possible explanation is that the work of Unterreiner et al. (1997) has fully accounted for the effects of slip as allowed by their formulation and discrepancies with the observed behaviour are resulting from all three simulations containing another unidentified error. One possible source for this error could be incompatibility between the simulated and actual construction sequences. For all three of these numerical simulations when the reinforcements are numerically placed in the soil, the stiffness and strength characteristics of the reinforcements relate to the long term behaviour of the constructed nail. In actuality, the reinforcement consists of a steel tube, grouted into a borehole under hydrostatic pressures. In the early stages of its life, the behaviour of this reinforcement is controlled by the characteristics of the grout annulus. Immediately after placement, the reinforcement has zero stiffness. Then as the annulus hardens, the reinforcement becomes stiffer and stiffer. If excavation of subsequent stages continues before the grout annulus has fully cured, then the actual behaviour of the reinforcements will be less stiff than the long term strength characteristics of the nail dictate. The variations between the predicted and observed behaviour of CEBTP wall No.1 are consistent with the actual reinforcements being less stiff than the modelled ones and as such the variations between the three wall simulations and the actual observed behaviour could be the result of not modelling changes in the reinforcement stiffness with time.

Of course other possible sources of error do exist. Drill and grout nail construction techniques can lead to stress relief and physical disruption in the soil adjacent to the nail. This could result in this soil behaving in a less stiff manner than the surrounding embankment. Matters could be further complicated by the effects of restrained dilatancy upon shearing of the soil. This mechanism could lead to increases in confining pressure around the nail and result in relatively stiffer reinforcement response after shearing. This would result in a reinforcement behaviour that changed from less stiff than indicated by soil testing to more stiff, depending upon the degree of shearing experienced at the interface.
Finally, the last point regarding the differences between the calculated and observed nail responses presented in Figure 4.27 relates to the fact that all three numerical simulations overestimate the nail forces adjacent to the face. This could be quite significant for an investigation of facing response. As was discussed in Chapter 2, loads at the face are the result of an interaction between the three major components of a soil nail system, namely the reinforcements or nails, the facing and the soil. One possible facing load mechanism results from tensile forces in the nails pulling the facing at the nail head connection into the soil. If the program is overestimating the nail loads near the face, then it could similarly be overestimating the facing loads.

Interestingly, nail loads developed close to the face in full scale soil nail structures are quite well documented. Figure 2.14 presents normalised nail head loads reported in FHWA Manual for the Design and Construction Monitoring of Soil Nail Walls (FHWA-SA-96-069R, 1998). Based upon these and other results the FHWA recommend that a nail head service load \( (t_r) \) of \( 0.5K_S^gH_SH^b_S^b \) be adopted. Figure 4.28 presents calculations determining the FHWA recommended \( t_r \) for CEBTP Wall No.1 at a height of 5 meters. Inspection of Figure 4.27 shows that all four nails, the nail head loads predicted by the Unterreiner et al. simulation and the td8auto simulation are less than that recommended in FHWA-SA-96-069R. Further, for nails 1, 2 and 4 the td8auto simulation most accurately predicts the observed response, which indicates that it is unlikely that the mechanisms of soil-reinforcement slip or softening of the soil adjacent to the excavation (mechanisms which are considered by the Unterreiner et al. (1997) simulation) are causing this disparity. One possible cause of the lower observed nail tensile forces at the face is the self weight of the shotcrete. None of the three simulations consider the self weight of the shotcrete. In reality, the newly placed shotcrete facings hang off the soil nails, resulting in compressive forces being induced in the nail heads. As such the lower observed nail tensions may be the result of adding these compressive forces to higher tensile forces commensurate with those predicted by the simulations.
Soil:
\[ \phi = 38^\circ \]
\[ \gamma = 16.1 \, kN/m^3 \]
\[ S_H = 1.15 \, m \]
\[ S_V = 1 \, m \]

\[ K_a = \frac{1 - \sin \phi}{1 + \sin \phi} \]

\[ t_W = 0.5K_a\gamma HS_H S_V \]
\[ = 0.5 \times 0.238 \times 16.1 \times 5 \times 1.15 \times 1 \]
\[ = 0.5 \times 22.03 \]
\[ = 11 \, kN \]

Figure 4.28: Calculation of nail head service load (t_W) for CEBTP Wall No.1. using FHWA-SA-96-069R, 1998 recommendation.

What is apparent, from the comparison of these calculated and observed responses, is that the mechanisms of soil-reinforcement interaction are complex, and at present, our methods of analysis are inadequate to fully model the interaction. That said the three numerical simulations presented here have been able to provide a reasonable prediction of the general observed soil nail wall behaviour. Importantly, in the case presented here, the simple ‘full adhesion’ soil-reinforcement model used by td8auto can provide a prediction of soil-reinforcement behaviour that is at least the equal of other presently accepted analysis techniques.

In summary, the presented simulation has been able to accurately predict the displacement behaviour of soil mass, the failure wedge geometry and reasonably accurately predict the reinforcement response. At the very least the strength of the correlation between the calculated and observed wall behaviour verifies that program code is correctly applying the intended analysis and as such the results of analyses conducted using this program should be correct within the limitations of the theory used.
Chapter 5: Investigation of facing response

5.0 Introduction

In Chapter two, it is stated that the aim of this work, is to increase our understanding of the role played by shotcrete facing elements in soil nail earth excavation support systems. To this end, it was proposed that a 3D finite element program capable of simulating construction of a soil nail wall be developed and used to identify which factors have the greatest influence on the stresses induced in the facing.

Chapters three and four introduced a 3D FE program capable of simulating construction of a soil nail wall and detailed work conducted to verify that this program could adequately predict the response of a soil nailed wall. Following comparison between excavation responses observed in full scale monitored structures and that predicted by the program, it was concluded that the presented program could provide acceptable predictions of soil nail wall reinforcement response.

Having established the validity of the presented program, Chapter five presents the results of a parametric study conducted to quantify the influence of chosen parameters on the facing behaviour. In particular, parameters presently used in facing design or commonly held to have substantial influence, were targeted. Additionally, some factors relating to simplifications of the construction sequence and soil behaviour, applied by the numerical analyses are explored.
5.1 Parameters Analysed

For the results of any parametric study to have significant meaning, the choice of parameters must be well considered. When deciding which parameters were to be investigated, this thesis specifically targeted those soil characteristics that are presently used in facing design or are commonly held throughout the literature to have substantial influence. By doing this, we enable direct comparisons to be made between the results of the parametric study and the existing knowledge base.

5.1.1 Review of existing facing design methods:

In Chapter 2 the commonly accepted current facing design methods have been described. It is shown that the facing response is due to an interaction between the three major components of a soil nail system, the reinforcement or soil nails, the facing panels and the insitu soil. The complexity of the interaction between these three components has been widely recognised and as such the design methods generally represent a simplification of the actual behavior. Traditionally, the techniques for estimating the earth pressures at the facing typically use one of three simplifications:

- Assume the earth pressures at the face can be modelled using an equivalent pressure applied uniformly over the height of the wall. The size of this uniform pressure is assumed to be proportional to the total active earth pressure for the completed wall. Examples of such systems include the FHWA recommended method (FHWA-SA-96-069R, 1998), which recommends a nail head service load of \(0.5K_a\gamma H S_H S_V\) (here \(K_a\) is the Coulomb active earth pressure co-efficient) and the German method where 'the face design pressure is taken as 75 to 85 percent of the corresponding Coulomb active earth loading applied as a uniform pressure over the height of the wall face' (Byrne et al., 1997).

- Assume that the face earth pressure is proportional to the maximum nail load and apply this design load as a pressure, uniform over the nail tributary area \((S_H \times S_V)\). The French method, presented in Recommendations Clouterre (1991), is an example of this type of system. For the French case, the equations describing the
recommended proportionality vary depending upon the nail spacing (Equations 2.3 to 2.5).

- Assume that the effect of any overburden is negated by the action of the soil nails and design the facing to retain a Rankine active earth wedge forming between the nails. Mitchell and Villet (1987) present such a method, citing an internal Terrasol report (1983). Whilst not recommended by more recent publications, according to Byrne et al. (1997) 'designing the facing for an equivalent active load corresponding to a soil depth of one to two nail vertical spacings' has persisted in European design practice.

Using these three common simplifications, we can determine which parameters are currently considered to be of most influence by each system.

The Terrasol method presented by Mitchell and Villet (1987) is arguably the least theoretically rigorous of the three simplifications. Parameters considered by this method typically describe the basic soil strength and reinforcement geometry. As is mentioned above, for this method, loading on the face is assumed to be the result of a Rankine active wedge forming between the nails. As such, this model considers the soil unit weight ($\gamma$), the soil friction angle ($\phi$) and nail spacing when calculating the size of the face load (the first two parameters influence the size of the active force, whilst the nail spacing dictates the size of the failure wedge). Interestingly, the horizontal and vertical nail spacings ($S_H$, $S_V$) are also considered when calculating the facing response to the design load (the facing is designed as it were an equivalent raft or beam). This means that the effect of nail spacing is considered both when deciding the size of the design earth pressure on the face, and when calculating the reaction of the proposed face to this design earth pressure. As will be shown, this emphasis on nail spacing is in keeping with current understanding of the predominant face load mechanisms, FHWA-SA-96-069R (1998) notes that 'nail spacing plays a major role in the facing design' whilst Recommendations Clouterre, 1991 maintains that notwithstanding the large range of factors affecting the earth pressure at the facing, 'the most important of these is the spacing between the nails'. For the current purpose we need only note that this method considers the following soil parameters ($\gamma$, $\phi$, $S_H$ and $S_V$) and places particular emphasis upon the influence of the nail spacing ($S_H$, $S_V$).
Next, we consider the reduced active earth pressure methods (the German and FHWA Methods) for facing load estimation. Whilst the methods used to determine the design earth pressures for the German and FHWA methods are very similar (namely an empirically based blanket reduction in the Coulomb active earth pressure) the differing recommended techniques for determining the facing’s reaction to these loads, means that both of these systems eventually consider different factors when determining the appropriate facing design for a given excavation.

If we consider the German Method, the use of the Coulomb active earth pressure for the final excavation height means that the following parameters are considered when calculating the design pressure: the soil unit weight ($\gamma$), the soil friction angle ($\phi$), the soil cohesion ($c$), the soil-wall friction angle ($\delta$) and the total height of the excavation ($H$). As can be seen, the German method adds the parameters $c$, $\delta$ and $H$. The most notable of these additions, is the total excavation height ($H$). As was mentioned in chapter 2, pre-existing methods were independent of excavation height. Again, the facing is designed as if it were either a slab or beam, simply supported at the nail locations. This process introduces the parameters of horizontal and vertical nail spacing ($S_h, S_v$).

The FHWA Method, as is mentioned earlier, uses the same parameters as the German Method when determining the design load ($H, \gamma, \phi, \delta$), however when determining the flexural capacity of the facing it introduces a new parameter, the pressure factor for facing flexure ($C_f$). This factor effectively increases the flexural strength of the facing as the facing stiffness (as determined by the facing thickness) decreases. Importantly, the effects of this factor are separate from and apparently, contrary to, the normal interaction between the flexural strength of facing and the facing thickness. That is, the effects of this factor are considered in addition to the facing’s ‘traditional’ moment capacity calculations and act to increase the flexural capacity of the facing as the face thickness decreases. The text of FHWA-SA-96-069R (1998) explains that the pressure factor for facing flexure has been included to account for mechanisms of soil-facing interaction. In particular it targets the effects of non-uniformity in earth pressures acting at the facing. As the text explains, ‘As the facing flexural stiffness decreases
with respect to the soil subgrade reaction modulus, the pressure distribution behind the facing will become highly non-uniform, with large pressure concentrations occurring behind the nail heads'. Further they maintain that 'the non-uniformity of facing pressure distribution ... should be considered in design because the actual available nail head strength will generally be significantly larger than the strength that would be computed based on the conservative assumption of a uniform pressure distribution.' That is, the concentration of earth pressure at the nail head both acts to increase the soil resistance to a punching shear type failure (Fig 2.12 (b)) and decrease the loading on the facing away from the nail head, thereby reducing the possibility of flexural failure of the facing (Fig 2.12 (a)). In terms of identifying the appropriate factors for consideration in this parametric study, the introduction of the pressure factor for facing flexure is significant because it highlights facing stiffness, or more precisely the ratio of soil stiffness to facing stiffness as a parameter to be considered when determining the facing flexural loads.

Further to this, it is worth noting that the FHWA design manual for soil nailed walls (FHWA-SA-96-069R, 1998) comments that the magnitude of facing pressures developed in soil nail walls (as indicated by nail head tensile loads) 'depends on the timeliness of the nail installation, the ground stiffness characteristics, the nail tensile stiffness, the nail grout-ground interface stiffness and the facing stiffness'.

One aspect common to both of the reduced active earth pressure methods that is worth noting is that whilst the nail spacing is considered when calculating the moment induced by the design earth pressure, the actual design earth pressure itself is independent of the nail spacing. This is counter-intuitive for extreme cases. If we imagine a soil nail wall where nail spacings are extremely large, the effects of any arching between the nails are going to be minimal and as such earth pressures will revert to the active case. Similarly, if nail spacings are extremely small then earth pressures can be reasonably expected to be approaching zero.

As is mentioned earlier, the importance of nail spacing to facing design is discussed within the FHWA manual (FHWA-SA-96-069R, 1998) text. It explicitly makes the abovementioned point that the theoretical limits for the earth pressure on the facing vary from zero to the active earth pressure, dependant upon the nail spacing. This
independence between nail spacing and facing earth pressure, highlights the fact that these design methods have been conceived for use within the scope of common construction practice (FHWA-SA-96-069R, 1998 notes that 'unless the ground is extremely competent, vertical and horizontal soil nail spacings are generally within the range of 1.5 to 2 meters'). It should likewise be noted that both the German and FHWA methods have shown themselves to be reasonable for common soil nail wall applications. This may suggest that for a typical range of nail spacings, the effects of nail spacing on the earth pressures at the facing is less than the theoretical limits suggest. Either way, this discussion highlights nail spacing as a parameter requiring further investigation.

The last common simplification to be considered is typified by the French facing design method (Recommendations Clouterre, 1991), which assumes that the earth pressures at the facing are proportional to the maximum nail loads. As is discussed in Chapter 2, the French researchers considered simplified statics to equate the earth pressures at the face to the tensile nail loads at the head of the nail. They then proposed a series of empirically based equations describing the relationship between the loads at the nail head and the maximum nail loads (Equations 2.3 - 2.5). These equations are solely dependent upon the nail spacing used. This has been done because whilst the researchers recognised that the relationship between the nail head and maximum loads is dependent upon a 'number of parameters (the stiffness of the soil, rigidity of the facing, rigidity of the nails, depth and spacing of the nails)' their research indicated that 'the most important of these is the spacing between the nails' (Recommendations Clouterre, 1991).

By linking earth pressures to maximum nail load, the French Method links facing loads to a number of parameters not previously considered. As is discussed in Chapters 1 and 4, the determination of an appropriate theoretical model to describe the mechanisms of nail-soil friction is very complex. Current design is typically based on prior experience, augmented by insitu nail pullout tests. The existing research indicates that the typical soil strength parameters, the behaviour of the soil during shearing, the stiffness of the soil, the stiffness of the nail and even the nail installation method all play a significant role in determining the frictional behaviour of the nail-soil interface. Further, the frictional interaction is only one of a number of

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mechanisms determining the maximum force in a nail, as an example, the timeliness of construction is similarly important when determining the maximum nail load. The French facing design method, by linking facing loads to maximum nail loads, is in a way considering all of these parameters when apportioning design earth pressures at the face. Interestingly, the design earth pressure is assumed to act uniformly over the nail tributary area with the effects of facing stiffness not being explicitly considered. This said, it should likewise be noted that the text of Recommendations Clouterre, 1991 does state that the assumption of uniform facing pressure 'is conservative when compared with actual conditions when calculating bending in the facing' and it does explicitly mention the role of facing rigidity in determining facing pressures. Further, the effects of facing rigidity for typical construction practice are most likely built into the empirically based correlations between nail spacings and the ratio of nail head load to maximum nail load, so by not explicitly considering the ratio of soil to facing stiffness as a parameter, the French Method, while noting its contribution, essentially says that within the limits of usual practice this parameter is of secondary influence. Again, such discussion highlights the ratio of soil to facing stiffness as a parameter worthy of further investigation.

With respect to the importance placed upon nail spacing by the design method, we should note that as with the previously discussed methods, the facing is designed as if it were an equivalent slab or beam. So, as with the Terrasol method presented in Mitchell and Villet (1987), the French method places particular emphasis upon the nail spacing, with the nail spacing being considered both when calculating the size of the design earth pressure and when determining the effect of this uniform earth pressure on the facing panels.

In summary, in addition to the commonly considered soil strength and wall geometry parameters (H, γ, φ, δ, S_v and S_H) the French method links earth pressures at the face to a number of new parameters (nail and soil rigidity (E_n, E_s) and nail installation method). Further, it acknowledges the influence of a number of other mechanisms (i.e. facing rigidity and construction related issues such as timeliness of construction) and places particular emphasis on nail spacing by considering the parameter both
when determining design earth pressures and when calculating the response of the facing to these design pressures.

5.1.2 Scope of the parametric study:

Having reviewed the common facing design methods, it is now relatively easy to decide which parameters are most appropriate for consideration in this investigation. Table 5.1 presents the proposed scope of this parametric study. Review of the previous discussion shows that factors such as (γ, φ, H, Sᵥ and Sₜ) were necessary inclusions, if only for the fact that they are a common thread through current design methods. Similarly, the prevalence of discussion regarding the influence of nail, soil and facing stiffness (Eₘ, Eₙ and Eₜ) in the literature in addition to their rudimentary inclusion in the more recent design methods (French and FHWA) made these factors obvious inclusions.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Reason for consideration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil friction angle (φ)</td>
<td>Considered in all 3 facing design methods.</td>
</tr>
<tr>
<td>Soil unit weight (γ)</td>
<td>Considered in all 3 facing design methods.</td>
</tr>
<tr>
<td>Height of excavation (H)</td>
<td>Considered in more recent design methods.</td>
</tr>
<tr>
<td>Nail spacing (Sᵥ / Sₜ)</td>
<td>Current research considers a major influence, though varying degrees of consideration in current design methods.</td>
</tr>
<tr>
<td>Soil Young’s Modulus (Eₙ)</td>
<td>Highlighted by current research.</td>
</tr>
<tr>
<td>Nail Young’s Modulus (Eₙ)</td>
<td>Highlighted by current research.</td>
</tr>
<tr>
<td>Facing Young’s Modulus (Eₜ)</td>
<td>Highlighted by current research – inclusion in FHWA method.</td>
</tr>
<tr>
<td>Coefficient of lateral earth pressure at rest (Kₒ)</td>
<td>Major influence on calculated excavation forces in numerical simulations.</td>
</tr>
<tr>
<td>Nail layout pattern</td>
<td>Alters ‘slab/beam’ model of face.</td>
</tr>
<tr>
<td>Associated vs. non associated flow rule</td>
<td>Assess impact of modelling simplifications.</td>
</tr>
<tr>
<td>Construction simulation method</td>
<td>Assess impact of modelling simplifications.</td>
</tr>
</tbody>
</table>

Table 5.1: Proposed scope of parametric study.

In addition to these factors, a number of extra parameters have been included in the study. The nail layout pattern is a rudimentary design choice, which theoretically has
the potential to vary facing response. Changes to the layout pattern alter the geometry of the supports for the ‘slab/beam’ model used to design the face.

Discussion relating to the calculation of excavation induced forces shows that the size of the force induced by excavation is directly related to the stresses present in the soil prior to excavation. As such, the coefficient of lateral earth pressure at rest ($K_o$), theoretically has a large influence on the size of the forces the retaining system will need to resist and so potentially the size of the force experienced by the facing.

Finally, there are a number of issues relating to simplifications inherent in the program/modelling process that are worthy of further exploration. Discussion in chapter 4 highlighted potential errors being introduced by simplifications made to the excavation sequence during modelling. As is mentioned, the program can only model the extremes of the wall’s behaviour (Figure 4.22). The effect of these simplifications on the facing response is subsequently considered by the parametric study.

Similarly, Chapter 3 mentions the assumption of zero soil dilation inherent in the program’s use of a non-associated flow rule. In Chapter 4 the relative impact of this assumption was explored through the comparison of published soil wall simulations (using various soil/reinforcement interaction models) with simulations conducted using the presented program (Td8auto). It was concluded that the order of error being introduced by simplifications inherent to the numerical formulation was insignificant in comparison to the errors introduced by simplifications inherent to the modelling such a complex system (e.g. variability associated with the interpretation of soil characterisation testing was shown to negate the benefits of using a more complex numerical formulation). Notwithstanding, the following parametric study further explores the impact of the assumption of zero soil dilation, by comparing results from Td8auto with a program variant, which utilises an associated flow rule.

5.2 Parametric model analysed

Figure 5.1 presents the model considered by the parametric study. As can be seen a 10m wide, 6m high, infinitely long trench has been modelled. The insitu soils consist
of a uniform 12.5m thick strata, underlain by a relatively inflexible stratum. This is consistent with a uniform soil strata overlying bedrock. Figure 5.2 shows the finite element mesh used to model this excavation. An axis of symmetry in the center of the trench has been used to reduce the required mesh size. The boundary conditions imposed on the mesh are also shown in Figure 5.2.

![Diagram of trench](image)

Figure 5.1: Infinitely long trench considered by parametric study.

Before the parametric study was commenced, a number of analyses were run using this mesh with soil strength parameters and nail lengths typical of common soil nail practice (see Table 5.2). The results of these analyses were checked against commonly observed soil nail wall behaviour. The purpose of this checking procedure was to ensure that factors such as: the boundary conditions being imposed on the mesh and soil strata being modeled were not impeding the mesh’s ability to correctly reproduce typical soil nail wall behaviour.

Figure 5.3 presents nail loads, calculated using this ‘baseline’ problem. The results are shown relative to the wall geometry. Also shown is a line of maximum tension, presented in Recommendations Clouterre, 1991 as being typical of those observed in the monitored soil nail structures conducted as part of the French research program. As can be seen, the locations of the calculated maximum nail tensions show very good agreement with the Clouterre line of maximum tension. As has been discussed earlier, the line of maximum tension delineates between the active and resistant zones that have developed in the reinforced soil and as such the good correlation between the
calculated and observed lines of maximum tension indicates that the simulation is correctly modelling the failure wedge geometry.

Figure 5.2: Boundary conditions imposed on ‘baseline’ model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Adopted Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil friction angle (φ)</td>
<td>35°</td>
</tr>
<tr>
<td>Soil cohesion (c)</td>
<td>5 kPa</td>
</tr>
<tr>
<td>Soil unit weight (γ)</td>
<td>18 kN/m³</td>
</tr>
<tr>
<td>Coefficient lateral earth Pressure (Kₐ)</td>
<td>0.4</td>
</tr>
<tr>
<td>Soil Young’s Modulus (Eₘ)</td>
<td>60 MPa</td>
</tr>
<tr>
<td>Soil Poisson’s Ratio (ν)</td>
<td>0.45</td>
</tr>
<tr>
<td>Nail spacing (Sᵥ and Sₕ)</td>
<td>1 m x 1 m</td>
</tr>
<tr>
<td>Nail Young’s Modulus (Eₙ)</td>
<td>70 000 MPa</td>
</tr>
<tr>
<td>Nail length (l)</td>
<td>8 m</td>
</tr>
<tr>
<td>Nail x-sectional area (A)</td>
<td>250 mm²</td>
</tr>
<tr>
<td>Facing Young’s Modulus (Eₜ)</td>
<td>25 000 MPa</td>
</tr>
<tr>
<td>Facing Poisson’s Ratio (ν)</td>
<td>0.2</td>
</tr>
<tr>
<td>Facing thickness (t)</td>
<td>100 mm</td>
</tr>
</tbody>
</table>

Table 5.2: ‘Baseline case’ adopted soil strength parameters.
part of the FHWA's federally funded research program DP 103. Based upon these results, the FHWA Manual for the design of soil nail walls (FHWA-SA-96-069R, 1998) reports that the mean maximum nail load observed was equal to $0.75K_d\gamma H S_v S_H$. If we assume zero friction between the wall and the soil, then for our baseline wall, this relates to a nail load of approximately 22kN. The calculated maximum nail loads are equal to; 16.1kN, 20.4kN, 24.4kN, 25.7kN and 21.0kN running from the uppermost nail to the bottom nail. As can be seen these calculated values compare favourably with those typically expected in soil nail structures.

![Diagram](image)

**Figure 5.3:** Nail load distribution calculated using proposed mesh.

Continuing the interrogation of the baseline model results, Figure 5.4 presents ground displacements calculated using the model. Again, the results are shown relative to the wall geometry and the Clouterre recommended line of maximum nail tensions. The distribution of displacements in the reinforced soil mass show a good correlation to the expected failure wedge geometry. As can be seen, soil displacements are greatest in the expected failure wedge. Additionally, the magnitudes of the calculated displacements conform to the expected norms. Figure 5.5 presents a reproduction of the relationship between wall height and horizontal displacement at the top of the wall, detailed in Recommendations Clouterre, 1991. As can be seen the displacement of the top of wall, calculated using the baseline mesh, correlates well with the monitored field responses.

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Figure 5.4: Displacements calculated for proposed mesh.

Figure 5.5: Relationship between wall height (H) and horizontal displacement of the top of the wall (after Recommendations Clouterre, 1991)
Figure 5.6: Variation of displacement 1m behind the wall with respect to the maximum number of iterations.

The last of the major checks conducted on the baseline model, were aimed at exploring the nature of yielded elements within the mesh. Extensive plastification occurs during excavation and as such, the model was analysed using varying convergence criteria to ensure that an adequate solution was being reached. Figure 5.6 presents displacements 1m behind the wall for different specified maximum iterations. Using these results, a maximum iteration criterion of 20 iterations per excavation increment (provided the specified ‘change in load vector’ convergence criteria was not already met) was chosen as providing the best trade off between accuracy and computational efficiency.

Based upon the checks outlined above, the presented model was assumed to provide a reasonable description of expected soil nail behaviour and as such was adopted as the basic model around which the parametric study was founded.
5.3 Results of parametric study

Section 5.3 describes each parameter considered by the study and presents the calculated variation in facing moment for the considered parametric combinations.

The 6m high, 10m wide baseline model discussed in Section 5.2 is used for all models presented, with the appropriate parameter variations detailed in each respective section. Unless otherwise stated, the remaining soil parameters are kept the same as those used in the baseline model (Table 5.2).

Except for Section 5.3.11, where the effect of the construction simulation method is considered, the ‘Case A’ type construction simulation shown in Figure 4.22 is assumed for all models. As is explained in Section 4.4.2, this simulation method assumes that all of the displacements associated with the current excavation step occur before the reinforcements for that step are placed and as such is probably more applicable for low cohesion soils and drill and grout construction techniques. Notwithstanding, note is drawn to the discussion in Section 5.3.11.1 highlighting the sensitivity of the facing reaction to the above assumption.

Figure 5.7 presents a three dimensional sketch of the completed baseline wall with the accompanying coordinate axes and moment directions. Note that the soil nail location is offset by half of the horizontal spacing for each row of nails. This ‘offset’ nail layout pattern is used for most of the parametric models, with the only exception being the ‘vertical column’ nail pattern considered in Section 5.3.9.

Further, the nature of the ‘offset’ nail pattern is such that the moments generated in the facing will vary for consecutive columns of nails (e.g. between sections AA and BB in Fig 5.7). Figure 5.8 presents a typical 3D moment plot calculated for the shaded facing panel shown in Figure 5.7 with the relative locations of sections AA and BB shown.

The results of the parametric study (i.e. plots of facing moment) are presented along a line corresponding to Section AA in Figure 5.7. A 2D section has been chosen,
because whilst the 3D surface does more fully describe the facing moments, this 3D presentation format does impede comparison of results. Note that on Section AA soil nails are located at depths of 0.5m, 2.5m and 4.5m.

Figure 5.7: Completed baseline wall with the accompanying coordinate axes and moment directions
Figure 5.8: A typical 3D moment plot with the relative locations of sections AA and BB shown.
5.3.1 Soil friction angle (φ)

Throughout this thesis, Jaky's formula has been used for the estimation of the coefficient of lateral earth pressure (K₀):

\[ K₀ = 1 - \sin \phi \]

Mayne and Kulhawy (1982) show that this relationship is well supported by existing test data for normally consolidated soils. However they similarly show that the soil stress history is also a major factor in determining K₀ and as such, it is possible that naturally occurring soils with similar soil strengths exist at differing at rest conditions.

As a consequence of this dichotomy, when considering the potential impact of changes to the modelled soil friction angle, there exist arguments both for and against changing the soil friction angle independently from K₀. For some soils, the influence of φ on the 'at rest' conditions will constitute an important part of the soil friction angle's overall impact on the soil nailed excavation. Conversely, for other soils the 'at rest' conditions will be influenced by the soil's stress history rather than its friction angle. For such cases, it is important that each parameter that is capable of varying independently be assessed in isolation. This allows the relative importance of each factor to be considered.

When investigating the impact of the soil friction angle, this thesis has considered both the case where φ is independent of K₀ and the case where φ and K₀ are linked via Jaky's formula.

5.3.1.1 Soil friction angle varied independently of K₀

Table 5.3 presents the values of soil friction angle considered for this section. K₀ is held at the 'baseline' value of 0.4, (note Jaky's formula gives K₀ = 0.426 for φ=35°). For values of phi below 30°, extensive soil plastification was found to occur under initial stress conditions. This mass soil failure prior to excavation was most likely a direct result of keeping the co-efficient of lateral earth pressure (K₀) constant and varying the friction angle of a low cohesion material.
Figures 5.9 and 5.10 present the calculated variation in vertical (M_{yy}) and horizontal (M_{xx}) moment respectively, for the considered soil friction angles. Moments are shown relative to the depth below the surface and represent the final facing moments (i.e. those calculated at the end of construction).

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Soil Friction Angle - ( \phi ) (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>50</td>
</tr>
</tbody>
</table>

Table 5.3: Soil Friction Angles considered.

5.3.1.2 Some comments on the calculated effect of varying soil friction angle independently of \( K_o \)

Prior to discussing the results of the parametric study, let us consider what the existing design methods suggest will be the effect of varying the soil friction angle (\( \phi \)) independently of \( K_o \). Put simply, for almost all cases we expect to see the size of the moment induced in the facing to reduce as the friction angle increases.

For the reduced active earth pressure method and the negated overburden effect method this can be easily seen to be the case. Increased soil strength results in either a reduced active force or increased resisting force, both of which result in a reduced design earth pressure acting on the assumed ‘beam/slab’. For the French Method, where the design earth pressure is linked to the maximum nail force, the connection between the friction angle and predicted facing response is a little more complicated. If we consider the static equilibrium of the final excavation, increased soil strength will mean that a smaller ‘restoring’ force is required from the reinforcements. As such, increased soil strength will mean less total nail load and thus less total design earth pressure. It should however be noted that this may not be the case for individual nails. Increases in soil friction angle can be similarly associated with increased unit skin frictions. So if we have the case of a nail at its limit pullout force and if the nature of the excavation is such that following an increase in soil friction angle (with associated increase in nail skin friction) this nail is still at its limit pullout force, then the action of the increased soil friction strength will have been to increase the...
maximum force in the nail and consequently increase the design earth pressure at the facing. So, whilst the total design earth pressure at the face should be reduced, for the French Method there is the possibility that local areas of increased design pressure may exist.

If we consider the calculated results presented in Figures 5.9 and 5.10 we can see that in general, as expected an increase in the soil friction angle results in a decrease in the calculated facing bending moment. Interestingly, the degree to which this is true varies depending upon the proximity to the nails. If we consider the points at which the nails intersect the facing (depths of 0.5m, 2.5m and 4.5m), we can see that at these locations, the ‘increased soil friction angle - decreased facing bending moment’ relationship is always true. However, if we consider the points between the nails, the difference between the three friction angles is negligible with no obvious trend.

Figures 5.11 and 5.12 show this variance graphically. The calculated moments at a point 2.41m below the surface and 3.41m below the surface have been plotted relative to the respective soil friction angle used. These two points correspond to just adjacent to a nail (d=2.41m) and the midpoint between two nails (y=3.41m).
Figure 5.9: Calculated variation in vertical moment ($M_{yy}$).
Figure 5.10: Calculated variation in horizontal moment ($M_x$).

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Figure 5.11: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.12: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.

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5.3.1.3 Soil friction angle and $K_o$ varied interdependently

Table 5.4 presents the values of soil friction angle along with the corresponding values of co-efficient of lateral earth pressure ($K_o$) considered in this section. The relationship between $\phi$ and $K_o$ is as described by Jaky's Formula:

$$K_o = 1 - \sin \phi$$

Figures 5.13 and 5.14 present the calculated variation in vertical ($M_{xx}$) and horizontal ($M_{yy}$) moment respectively, for the considered models. As with the previous case, the moments presented in these figures are shown relative to the depth below the surface and represent the final facing moments (i.e. those calculated at the end of construction).

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Soil Friction Angle - $\phi$ (deg.)</th>
<th>Corresponding $K_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>10</td>
<td>0.826</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.658</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>0.357</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>0.234</td>
</tr>
</tbody>
</table>

Table 5.4: Soil Friction Angles and corresponding $K_o$ values considered.

5.3.1.4 Some comments on the calculated effect of varying soil friction angle and $K_o$ interdependently

The coefficient of earth pressure at rest is not specifically considered by any of the existing design methods and so, according to these methods, the relationship between the facing response and soil friction angle should be independent of $K_o$. Nevertheless theory regarding the calculation of excavation forces, shows that excavation induced forces are proportional to the stresses in the soil prior to excavation. Consequently, there is a theoretical basis to say that $K_o$ will affect the facing response.

Discussion in Section 5.3.1.2 shows that the size of the moment induced in the facing is expected to reduce as the friction angle increases. However, for the case presented herein, where the $K_o$ conditions are varied dependant upon the soil friction angle, the influence of changing $K_o$ will also need to be considered.
Table 5.4 shows that an increase in the φ angle results in lower lateral stresses during ‘at rest’ conditions. Lower lateral stresses should be expected to result in lower excavation forces and subsequently lower facing reaction. This means that the action of φ and K₀ are complementary. The action of decreasing the φ angle of the soil will not only increase the facing response as a result of decreased ‘resisting’ forces in the soil mass, but it will also increase the lateral stresses during ‘at rest’ conditions, which should further increase the facing response. As such, not only should we expect to see the calculated facing moments decreasing as φ increases, but we should also expect the results to show that the friction angle has a greater influence on the facing response than it did when K₀ was held constant (Section 5.3.1.2).

In order to aid comparison between the K₀ variable and K₀ constant cases, Figures 5.15 and 5.16 present the relationship between φ and Mᵧᵧ and Mₓₓ at two specific depths (d=2.41m and d=3.41m) for both K₀ cases.

If we first consider the case where φ and K₀ are dependent upon each other, we can see that as expected increasing the φ angle of the soil results in decreased facing response. Now if we compare the response of the variable K₀ and constant K₀ cases at a depth of 2.41m, it can be seen that for a change of φ between 40⁰ and 50⁰ the change in calculated moment is greater for the variable K₀ case. This is as expected. However, for a change of φ between 30⁰ and 40⁰ the change in calculated moment is of the same order for both cases. While this is not as initially expected, the result may be explained by the fact that for the K₀ constant case, K₀ is held at a value 0.4, which approximately correlates to a φ angle of 35⁰. The fact that the K₀ value for the ‘baseline’ case correlates to a φ angle in the middle of the considered interval (i.e. 30⁰ to 40⁰) may explain why the K₀ variable and K₀ constant cases display similar behaviour for this case.
Figure 5.13: Calculated variation in vertical moment ($M_{yy}$).

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Figure 5.14: Calculated variation in horizontal moment ($M_{xz}$).
Figure 5.15: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.16: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.

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5.3.2 Soil unit weight (γ)

Table 5.5 presents the values of soil unit weight (γ) considered by the parametric study. As can be seen, values ranging from 12 to 24 kN/m³ were considered. As a measure of how this range represents typical soils, it is worth noting that Recommendations Clouterre, 1991 presents soil unit weights ranging from about 17 to 22 kN/m³ as nominal values for some soils typically used for soil nailed retaining systems.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Soil unit weight – γ (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>24</td>
</tr>
</tbody>
</table>

Table 5.5: Soil unit weight considered.

Figures 5.17 and 5.18 present the calculated variation in vertical (M_y) and horizontal (M_x) moment respectively. Again, results are shown relative to the depth below surface and represent the final facing moments (i.e. those calculated at the end of construction).

5.3.2.1 Some comments on the calculated effect of varying soil unit weight

Again it is worth considering the expected reaction of the facing to an increase in soil unit weight prior to commenting on the calculated results. In summary, all three design methods should see an increased moment induced in the facing as a result of an increase in the soil unit weight, however the different simplifications made by each method means that each places difference emphasis upon the importance of γ.

The Terrosol Method, as is mentioned earlier, designs the facing to retain a Rankine active wedge forming between the nails. As such the design earth pressure is directly proportional to the soil unit weight. On the other hand, the reduced active earth pressure methods like the German or FHWA Methods use a reduced Coulomb Active earth pressure. The emphasis placed upon the soil unit weight is altered because in a
Coulomb analysis $\gamma$ contributes to both the driving force (through the mass of the failure wedge) and to the restoring force (through increased friction), with the ultimate impact of any change to the unit weight being essentially model dependent. The French Method is similarly affected by the fact that the soil unit weight contributes to both the driving and restoring forces. If we consider the static equilibrium of the final excavation, a change in the model unit weight $\gamma$ will likewise change the ‘restoring’ force required from the reinforcements and as such the total design earth pressure at the face, however as with the German and FHWA Methods, the final impact of any change will be model dependent.

If we consider Figures 5.17 and 5.18, we can see that the changes made to the soil unit weight have had limited effect upon the calculated facing moment and in general our expected relationship between $\gamma$ and moment is not uniformly met. In fact, the only major locations where our expected relationship can be seen are at the top and bottom nail for the horizontal moment and at the top nail for the vertical moment. Again, plots showing the calculated moments at two specific depths ($d=2.41m$ and $d=3.41m$) relative to considered soil unit weight can be used to show this graphically (Figures 5.19 and 5.20).
Figure 5.17: Calculated variation in vertical moment ($M_{yy}$).

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Figure 5.18: Calculated variation in horizontal moment ($M_{xx}$).

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Figure 5.19: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.20: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.
5.3.3 Excavation Height (H)

Table 5.6 presents the various excavation heights (H) considered by the parametric study. The different heights represent intermediate stages of the ‘baseline case’ wall.

Note that as a result of the ‘case A’ excavation sequence (see Figure 4.22) used for the study, no moments are presented for the excavation at 1m depth. As has been previously mentioned, the construction of the ‘baseline’ wall is simulated in 1m stages, with the reinforcement constructed at the end of each stage. The ‘case A’ simulation used assumes that all of the displacements associated with the current excavation step have occurred prior to installing the reinforcement. As such, there was no reinforcement response when the excavation was at a 1m depth. Reinforcement reaction only resulted from subsequent excavation.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Excavation Height – H (m).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>6</td>
</tr>
</tbody>
</table>

Table 5.6: Excavation Heights considered.

Figures 5.21 and 5.22 show the calculated vertical ($M_{yy}$) and horizontal ($M_{xx}$) facing moments respectively for the five cases shown above. Consistent with previous results, these plots show the response for the final excavation stage with respect to the depth below the surface.

5.3.3.1 Discussion relating to the height of excavation

The expected effect of varying the height of the excavation on the moment induced in the facing of a soil nailed excavation changes depending upon which of the current design methods you use. The majority of the design methods predict that facing moments should increase as the height of the excavation increases. The only exception to this is the Terrasol Method presented by Mitchell and Villet (1987). This
method assumes that as a result of 'the arching effect' the influence of any overburden can be ignored and the facing designed as though it were retaining a Rankine active wedge forming between the soil nails.

If we consider the calculated results shown in Figures 5.21 and 5.22 we can see that the moments behave as the majority of design methods would predict, that is the moment increases as the height of the excavation increases. It is however worth noting that this axiom is more noticeable at the nail heads than in the area between the nails. Again, plots showing the calculated moments at two specific depths relative to excavation height are presented to better show the relationship between the these parameters (Figures 5.23 and 5.24). However for this case the inspection points have been raised to depths of 0.49m and 1.41m as more information is available at these shallower depths.
Figure 5.21: Calculated variation in vertical moment ($M_{yy}$).
Figure 5.22: Calculated variation in horizontal moment ($M_{xx}$).
Figure 5.23: Calculated variation in $M_{yy}$ at depths 0.49m and 1.49m.

Figure 5.24: Calculated variation in $M_{xx}$ at depths 0.49m and 1.49m.

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5.3.4 Nail spacing (S)

Table 5.7 presents the nail spacings considered by the study. Only horizontal nail spacings have been varied. This has been done in order to separate the influence of the nail spacing from that of changes to the construction sequencing. Section 3.2 discusses the influence that changes to the excavation sequence have over the predicted behaviour of a soil nailed excavation. Alterations to the construction sequence vary both the zero stress positions for the reinforcements and the stress paths followed by the soil. If the vertical nail spacing were to be changed, then the size of each excavation step would likewise need to be altered, thus introducing a new variable. In short, by altering the excavation sequence it would be possible to get different reinforcement responses from otherwise identical excavations.

<table>
<thead>
<tr>
<th><strong>Excavation Simulation</strong></th>
<th><strong>Nail Spacing – $S_h$ (m)</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
</tr>
</tbody>
</table>

Table 5.7: Nail Spacing considered by study.

As a gauge of how the range of values presented in Table 5.7 compares to common practice it is worth noting that the FHWA Manual (FHWA-SA-96-069R, 1998) presents project summaries for some 48 soil nailed walls constructed in the US. For these 48 projects the horizontal spacing ranges from 0.6m to 2.4m.

Consistent with previous results, the calculated variation in vertical ($M_{yy}$) and horizontal ($M_{xx}$) moment for the final stage of excavation is shown with depth below surface in Figures 5.25 and 5.26 respectively.

5.3.4.1 Discussion relating to the nail spacing

Section 5.1.1 considers the emphasis placed by each of the current facing design methods on the influence of nail spacing. Whilst there are a number of differences
between the current methods, in essence they can be ‘boiled down’ to one major
distinction that is, whether or not nail spacing is considered as a variable for the
determination of the facing design earth pressure.

As was discussed in Section 5.1.1, all of the methods consider nail spacing, when
calculating the size of the moment induced by the design earth pressure (i.e. the nail
spacing becomes the distance between supports for the ‘slab/beam’ model) however
not all of the methods specifically consider the nail spacing when calculating that
design earth pressure. This line of distinction effectively changes the emphasis placed
upon nail spacing by each of the methods. Interestingly, except for the Terrasol
Method, all of the methods for determining design earth pressures at the face are
empirically based (regardless of whether or not they consider nail spacing). Figures
2.14 and 2.15 present the normalized nail head loads and normalized maximum nail
loads which form part of the basis of the design earth pressure calculation for the
FHWA Method. Inspection of this information shows that reasonable scatter is
inherent in this information and as such refinements such as nail spacing may not be
justified with the currently available information.

Nonetheless, it should also be noted that whilst the implementation of the various
methods may differ, there is general agreement amongst the published work that ‘nail
spacing plays a major role in the facing design’ (FHWA-SA-96-096R, 1998), with an
increase in the spacing leading to a increase in the facing response.

Inspection of Figures 5.25 and 5.26 shows the facing response calculated by this
parametric study agrees with the assertion that nail spacing has a major role in
determining the facing response. In particular the horizontal moment (M_{hx}) seems to
be influenced by the changes in nail spacing. This is not unexpected as it is the
horizontal spacing of the nails that is being varied. As with previous work, plots
showing the calculated moments at two specific depths (d=2.41m and d=3.41m)
relative to excavation height are presented to better show the relationship between
these parameters (Figures 5.27 and 5.28).
Figure 5.25: Calculated variation in vertical moment ($M_{yy}$).

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Figure 5.26: Calculated variation in horizontal moment ($M_{xx}$).

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Figure 5.27: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.28: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.
5.3.5 Soil stiffness - Young’s Modulus of the soil ($E_s$)

Table 5.8 presents the range of soil Young’s moduli considered by the study. Soil moduli values were chosen to reasonably represent those typically encountered by soil nailed wall applications. As an indication as to what magnitude and variability would be expected for a insitu soil typical of soil nailed excavations, note is drawn to Figure 4.21. Here the results of Menard Pressuremeter tests conducted as part of the French Research Project Clouterre are presented. It shows the Young’s modulus of the specially constructed embankment varying from approximately 15 to 35 MPa. As this testing was conducted in a man made fill embankment, it likely represents the lower end of the typically expected values, with the maximum 90 MPa being consistent with a dense sand (The University of Sydney, Centre for Geotechnical Research, Soil Mechanics Data Sheets present typical values of Young’s modulus for dense, normally-consolidated sands as varying from 40MPa to 84MPa and from 80MPa to 168MPa for a dense, over-consolidated sands).

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Young’s Modulus of the soil – $E_s$ (MPa)</th>
<th>$E_n/E_s$</th>
<th>$E_r/E_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>10</td>
<td>7000</td>
<td>2500</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>3500</td>
<td>1250</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>2333</td>
<td>833</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>1400</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>1000</td>
<td>357</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>778</td>
<td>278</td>
</tr>
</tbody>
</table>

Table 5.8: Considered Soil Modulus ($E_s$) and corresponding $E_n/E_s$ and $E_r/E_s$ ratio.

Note that in addition to presenting the Young’s modulus used, Table 5.8 also details the corresponding ratios of nail modulus to soil modulus ($E_n/E_s$) and facing modulus to soil modulus ($E_r/E_s$). These ratios have been included, as the difference in stiffness between the soil and reinforcing elements has been identified as one of the potential elements dictating the mechanisms of soil-reinforcement interaction.
Consistent with previous results, the calculated variation in vertical ($M_y$) and horizontal ($M_x$) moment for the final stage of excavation is shown relative to the depth below surface in Figures 5.29 and 5.30 respectively and plots showing the calculated moments at two specific depths ($d=2.41m$ and $d=3.41m$) relative to excavation height (Figures 5.31 and 5.32).

5.3.5.1 Discussion of the effects of varying the Young’s Modulus of the soil

Whilst the soil Young’s modulus is not specifically included in any of the current design methods, it (and perhaps more importantly its stiffness relative to the reinforcing elements) has been noted by the major research projects into soil nailed excavations (Recommendations Clouterre, 1991 and FHWA-SA-96-096R, 1998) as a contributor to the reinforcement response and as such some attempts have been made to include its influence (e.g. the FHWA facing flexure factor ($C_F$) or Recommendations Clouterre’s marrying to the maximum nail force).

In terms of discussing what the expected impact an increase in soil modulus would have on the facing response, it is useful to return to the analogy of a soil nailed excavation as a structure composed of three main components: the soil nails, the facing and the soil itself. As with any structure, we expect the stiffer elements to attract relatively more load. So by increasing the soil stiffness relative to the reinforcements, we expect that the soil will end up carrying a larger portion of the overall load and as such the reinforcements carry less of the load. The FHWA facing flexure factor ($C_F$) demonstrates this point, as the facing stiffness decreases (as represented by facing thickness) the factor acts to increase the flexural capacity of the facing (as shown by nominal nail head strength). This is not done because the facing has actually become stronger, but because it is held that less of the uniformly distributed design load actually makes it onto the facing. In terms of what we should expect when we look at our plots, this means an increase in soil stiffness will see a reduction in the calculated facing moment.

Inspection of Figures 5.29 to 5.32 shows this to be the case. An increase in soil modulus results in a decrease in the size of the moment induced in the facing in all areas of the facing.
Figure 5.29: Calculated variation in vertical moment ($M_{yy}$).
Figure 5.30: Calculated variation in horizontal moment ($M_{xx}$).
**Figure 5.31:** Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

**Figure 5.32:** Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.
5.3.6 Nail stiffness - Young’s Modulus of the nails (E_n)

Table 5.9 presents the range of soil nail Young’s moduli considered by the study. Values of soil nail moduli have been chosen to provide the same nail stiffness to soil stiffness ratios as used in Section 5.3.5. As is mentioned earlier, the baseline case uses a nail modulus of 70 GPa, which is in keeping with values reported as part of the French CEBTP wall No.1 Research Project detailed in Section 4.4.1.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Young’s Modulus of the nail – E_n (GPa)</th>
<th>Ratio E_n/E_s</th>
<th>Ratio E_f/E_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>420</td>
<td>7000</td>
<td>417</td>
</tr>
<tr>
<td></td>
<td>210</td>
<td>3500</td>
<td>417</td>
</tr>
<tr>
<td></td>
<td>140</td>
<td>2333</td>
<td>417</td>
</tr>
<tr>
<td></td>
<td>84</td>
<td>1400</td>
<td>417</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>1000</td>
<td>417</td>
</tr>
<tr>
<td></td>
<td>47</td>
<td>778</td>
<td>417</td>
</tr>
</tbody>
</table>

Table 5.9: Considered Nail Modulus (E_n) and corresponding E_n/E_s and E_f/E_s ratio.

As with previous work, the calculated variation in vertical (M_yy) and horizontal (M_xx) moment for the final stage of excavation is shown relative to the depth below surface in Figures 5.33 and 5.34 respectively and plots showing the calculated moments at two specific depths (d=2.41m and d=3.41m) relative to excavation height are included in Figures 5.35 and 5.36.

5.3.6.1 Discussion of the effects of varying the Young’s Modulus of the nails

Section 5.3.5.1 discusses how the current design methods attempt to include the relative stiffness of the major components of a soil nailed structure into their calculations for facing response. For the current section, in order to answer the question ‘what should we expect when we start changing the stiffness of the nails’ it is useful to reconsider the analogy of the retention system as a structure made up of: soil nails, facing elements and the retained soil. Again, we expect that the stiffer elements will attract relatively more load, so if we start to increase the stiffness of the nails we expect relatively less load on the facing. Unfortunately, the situation is not as
straight forward as this here. There is a structural connection between the nail and the facing and as such, increased force in the nail leads to increased loads on the facing. Perhaps the most obvious manifestation of this idea is in the French Design Method where the size of the design earth pressure at the face is determined using empirically factored maximum nail load. So the answer to our above question as to the potential effect of varying nail modulus is not easily reached, there are conflicting mechanisms potentially at work.

If we consider the results presented in Figures 5.33 to 5.36, the calculated behaviour seems to reflect this uncertainty. Firstly, the values of nail modulus chosen provide the same range of soil to nail modulus ratios as the values of soil modulus used in Section 5.3.5. However, the change in moment resulting from varying the nail modulus is considerably less than that due to varying the soil modulus. Secondly, whilst the patterns are not as clear as that for other parameters it seems that for the area near the nail heads an increase in nail stiffness results in an increase in facing moment. Interestingly, this pattern seems to be less evident away from the nail heads. This behaviour may be consistent with the two conflicting mechanisms discussed above. The increased nail stiffness may have drawn load away from the facing, but the increased nail load results in increased load in the facing, with the area closest to the nail head being most affected.
Figure 5.33: Calculated variation in vertical moment (Myy).
Figure 5.34: Calculated variation in horizontal moment ($M_{xx}$).
Figure 5.35: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.36: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.

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5.3.7 Facing stiffness - Young’s Modulus of the facing (Eᵢ)

Table 5.10 presents the range of facing Young’s Moduli considered by the study. As with the soil nail stiffness, values of facing Moduli were chosen to provide the same nail to soil ratios as used in Section 5.3.5. The baseline case used a facing modulus of 25 GPa, which was consistent with published work conducted for CEBTP wall No.1.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Young’s Modulus of the facing – Eᵢ (GPa)</th>
<th>Ratio Eᵢ/Eᵢ₀</th>
<th>Ratio Eᵢ/Eₛ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>150</td>
<td>1167</td>
<td>2500</td>
</tr>
<tr>
<td></td>
<td>75</td>
<td>1167</td>
<td>1250</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>1167</td>
<td>833</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>1167</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>21.4</td>
<td>1167</td>
<td>357</td>
</tr>
<tr>
<td></td>
<td>16.7</td>
<td>1167</td>
<td>278</td>
</tr>
</tbody>
</table>

Table 5.10: Considered Facing Modulus (Eᵢ) and corresponding Eᵢ₀/Eₛ and Eᵢ/Eₛ ratio.

As with previous work, the calculated variation in vertical (Mᵧᵧ) and horizontal (Mₓₓ) moment for the final stage of excavation is shown relative to the depth below surface in Figures 5.37 and 5.38 respectively and plots showing the calculated moments at two specific depths (d=2.41m and d=3.41m) relative to excavation height are included in Figures 5.39 and 5.40.

5.3.7.1 Discussion of the effects of varying the Young’s Modulus of the facing

Section 5.3.5.1 introduces the idea of a soil nailed wall as a composite material made up of: soil nails, facing elements and the retained soil. Using this analogy it was postulated that increasing the relative stiffness of any component of the composite would result in this element developing a larger share of any total load applied to the system. In Section 5.3.6.1, it was shown that this analogy could be overly simplistic, with the nature of how the system is loaded and the interaction between the elements needing to be better considered. In practical terms, the problem was not so much in the assertion that increasing the nail stiffness will result in larger loads on the nail, rather the oversimplification was in the assertion that an increase in the load carried
by the nail will mean that load in the facing is reduced. In fact the calculated response seemed to suggest that two mechanisms were at play, one causing an increase in load in the facing and the other a decrease. This said, the secondary interaction between the facing and the nails should be of less importance in this case as we are only interested in the reaction of the facing (i.e. it doesn’t matter whether an increase in the facing load causes an increase in the nail head load, because we are only interested in the facing response). In terms of predicting the expected response of the facing, it should be expected, that an increase in facing stiffness would result in increased moment in the facing.

As was mentioned in Section 5.3.5.1, there some indications that this is the case in the existing design methods. In particular, the FHWA facing flexure factor \( (C_f) \) demonstrates this point. As the facing stiffness decreases (as represented by facing thickness) the factor acts to increase the allowable load at the nail head. As was mentioned before, this is not done because the facing has actually become stronger rather it is because it is held that less of the uniformly distributed design load actually makes it onto the facing.

Again, inspection of the presented Figures (Fig 5.37 to 5.40) shows this to be true. An increase in facing modulus results in a increase in the size of the moment induced in the facing in all areas of the facing.
Figure 5.37: Calculated variation in vertical moment ($M_{yy}$).
Figure 5.38: Calculated variation in horizontal moment ($M_{xx}$).
Figure 5.39: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.40: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.
5.3.8 Coefficient of earth pressure at rest ($K_o$)

Table 5.11 presents the various coefficients of earth pressure at rest ($K_o$) considered by the parametric study. As is mentioned in Section 5.3.1, whilst Jaky's Formula describing the relationship between soil friction angle and insitu stress has been used extensively throughout this thesis, the decision has been made to present results with the soil friction angle ($\phi$) and $K_o$ varying, both independently and interdependently of each other. The following section presents results for the case where $K_o$ is varied independently of $\phi$.

Notwithstanding this, when assessing how the considered values relate compared to normally consolidated insitu stresses it is useful to note that Jaky's Formula gives values of $K_o$ equal to approximately 0.6 and 0.3 for soil friction angles of 25 degrees and 45 degrees respectively.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Coefficient of earth pressure at rest ($K_o$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
</tr>
</tbody>
</table>

Table 5.11: Coefficient of earth pressure at rest ($K_o$) considered.

Again, the calculated variation in vertical ($M_{yy}$) and horizontal ($M_{xx}$) moment for the final stage of excavation is shown relative to the depth below surface in Figures 5.41 and 5.42 respectively and plots showing the calculated moments at two specific depths ($d=2.41m$ and $d=3.41m$) relative to excavation height are included in Figures 5.43 and 5.44.

5.3.8.1 Discussion of the effects of varying the Coefficient of earth pressure at rest ($K_o$)

The coefficient of earth pressure at rest is not considered by any of the existing design methods. The factor was introduced into this parametric study, because theory surrounding the calculation of excavation induced forces showed that the size of the
force resulting from excavation is related to the stresses in the soil prior to excavation (see Section 3.3.2). As such, it was felt that the insitu stress had the potential to change the reinforcement response, particularly when a soil nailed excavation’s staged construction sequence is considered.

Predicting the facing response to an increasing coefficient of earth pressure at rest is relatively straightforward. The larger the initial insitu stresses, the greater the forces associated with excavation and so the larger the moment induced in the facing. Inspection of Figures 5.41 to 5.44 shows this to be the case, with increasing calculated moments resulting from increases to the initial insitu horizontal soil stresses.
Figure 5.41: Calculated variation in vertical moment ($M_{yy}$).
Figure 5.42: Calculated variation in horizontal moment ($M_{xx}$).
Figure 5.43: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.44: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.
5.3.9 Nail Pattern

Table 5.12 presents the two nail layout patterns considered by the study. It is interesting to note that the FHWA Soil Nail Manual (FHWA-SA-96-069R, 1998) says that the ‘offset pattern is especially recommended where it is anticipated that the excavation face may be marginally stable’ but also reports that common design practice (i.e. in cases where the excavation face is considered reasonably stable) for both Caltrans (75 soil nail walls designed at time of writing) and Schnabel Foundation Company (190 walls) is to use vertical nail columns as it aids placement of strip drains during construction.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Nail layout pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>Offset</td>
</tr>
<tr>
<td></td>
<td>Vertical columns</td>
</tr>
</tbody>
</table>

Table 5.12: Nail layout patterns considered.

As with previous work, the calculated variation in vertical (M_{yy}) and horizontal (M_{xx}) moment for the final stage of excavation is shown relative to the depth below the surface in Figures 5.45 and 5.46 respectively. However, for the plots showing the calculated moments at two specific depths relative to excavation height (Figures 5.47 and 5.48), the depths have been varied to coincide with locations where nails are located in both patterns (d=0.49m and d=2.41m).

5.3.9.1 Effects of varying the nail layout pattern

Inspection of Figures 5.45 to 5.48 shows that at the locations where both patterns have nail heads (d=0.5m, d=2.5m and d=4.5m) the offset nail pattern develops a larger moment in the facing. However, for the locations where only the vertical column pattern has nail heads, the moment in the offset pattern is less.
Figure 5.45: Calculated variation in vertical moment (M_y).
Figure 5.46: Calculated variation in horizontal moment ($M_{xx}$).
Figure 5.47: Calculated variation in $M_{yy}$ at depths 2.41m and 0.49m.

Figure 5.48: Calculated variation in $M_{xx}$ at depths 2.41m and 0.49m.
5.3.10 Effect of allowing and varying soil dilation during shearing

The assumption of zero soil dilation, which is inherent in the presented program's numerical formulation, represents one of the potential sources of error in the findings offered in this thesis. Whilst this issue was broached during the program validation (Chapter 4), the parametric study provided the opportunity to further investigate the impact of this assumption. To this end, a variant of the presented program was developed, which allowed the consideration of soil dilation during shearing. A copy of the altered subroutine PLAST is attached in Appendix C. This shows the programming changes required to use the Sloan and Booker yield surface for both the plastic potential surface and the failure surface.

Table 5.13 presents the various models considered. In addition to considering models with various angles of soil dilation (ψ), models with ψ set to zero have also been included for cases using both the Von Mises and the Sloan and Booker surfaces for the plastic potential surface. The inclusion of these two ψ=0 models, allows the impact of changing the plastic potential surface calculation method to be assessed independently from the influence of the changes to the soil's dilative behaviour. Parameters not detailed here are as reported for the 'baseline' case.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>angle of soil dilation (ψ)</th>
<th>Plastic potential surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>0</td>
<td>Von Mises</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>Sloan and Booker</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Sloan and Booker</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Sloan and Booker</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>Sloan and Booker</td>
</tr>
</tbody>
</table>

Table 5.13: Flow rule formulations considered.

For the above models, the calculated variation in vertical (M_{yy}) and horizontal (M_{xx}) moment for the final stage of excavation is shown relative to the depth below surface in Figures 5.49 and 5.50 respectively and plots showing the calculated moments at two specific depths (d=2.41m and d=3.41m) relative to excavation height are included in Figures 5.51 and 5.52.
5.3.10.1 Discussion of the effects of allowing and varying soil dilation during shearing

The above analyses essentially consider two situations. Firstly, the two $\psi=0$ models explore the effect of changing the plastic potential surface calculation scheme. In terms of the expected influence on the calculated results, the impact of changing the plastic potential surface should be minimal. Inspection of the results shows that this is generally the case. We do however note some divergence between the two solutions in the vicinity of the nail heads.

The second situation considered by the above analyses is the effect of changing the soil's dilatant behaviour. The existing facing design methods, do not explicitly consider $\psi$. In terms of predicting the influence of $\psi$ on the calculated results, we have to consider its expected influence on the calculated behaviour.

The concept of restrained dilatancy (Schlosser and Elias, 1978) was introduced in Chapter 1. In basic terms, this theory states that the dilation of a soil during shearing will result in increased frictional forces developing across a shear plane. This occurs because the action of the soil dilation and the restraint provided by the surrounding soil mass leads to increased normal forces being developed across the shear plane. Discussion in Section 5.3.2.1 shows that the existing facing design methods predict reduced facing moments as a result of increases to the soil's shearing resistance. This would suggest that an increased $\psi$ angle would result in a decreased moment induced in the facing.

Another way of coming to the same conclusion is to use the analogy of the soil nailed wall as a composite material comprising: the soil nails, the facing elements and the retained soil (Section 5.3.5.1). The action of increasing the $\psi$ angle should result in a stiffer soil response. This should in turn result in a decreased moment induced in the facing.

Notwithstanding, a potentially conflicting mechanism does exist. If we consider that the role of the facing is to retain a Rankine active earth wedge forming between the
nails (i.e. the Terrasol Method), then shearing occurring within this active soil wedge will result in soil dilation. This dilatant behaviour immediately adjacent to the facing, could potentially result in higher lateral stresses on the facing, which would result in increased induced moments.

Inspection of the results shows that increasing the modeled \( \psi \) angle does in fact result in increased induced moments in the facing. Whilst this suggests that the latter of the two suggested mechanisms is predominant, it is important to note that the changes to the modeled \( \psi \) angle only result in relatively minor changes to the facing response. This finding may support the idea that \( \psi \) is in fact influencing the facing via two conflicting mechanisms, with both mechanisms displaying the same order of influence over the modeled behaviour.

Either way these results support the assertion that the order of error being introduced by simplifications inherent to the programs numerical formulation was insignificant in comparison to the errors introduced by simplifications inherent to the modelling a complex system (i.e. variability associated with the interpretation of soil characterisation testing will typically negate the benefits a using a more complex numerical formulation than that presented).
Figure 5.49: Calculated variation in vertical moment ($M_{yy}$).
Figure 5.50: Calculated variation in horizontal moment (M_{xx}).
Figure 5.51: Calculated variation in $M_{yy}$ at depths 2.41m and 3.41m.

Figure 5.52: Calculated variation in $M_{xx}$ at depths 2.41m and 3.41m.
5.3.11 Excavation Simulation Method

Section 4.4.2 explains that as a result of inherent simplifications, the presented program is not able to model the time dependant nature of a soil nailed wall’s construction and response. This means there are two possible methods for simulating construction, either the reinforcements are considered to act before the current excavation step is started (case B), or alternatively they are placed after all the displacements associated with the current step have occurred (case A). Figure 4.22 shows the two simulation methods graphically.

In order to assess the potential errors introduced as a result of this simplification, the calculated facing response is compared for the baseline wall using both construction simulations. Table 5.14 shows the considered cases.

<table>
<thead>
<tr>
<th>Excavation Simulation</th>
<th>Wall model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case A</td>
<td>‘baseline’ case</td>
</tr>
<tr>
<td>Case B</td>
<td>‘baseline’ case</td>
</tr>
</tbody>
</table>

Table 5.14: Excavation simulation methods considered.

As with previous work, the calculated variation in vertical ($M_{yy}$) and horizontal ($M_{xx}$) moment for the final stage of excavation is shown relative to the depth below surface in Figures 5.53 and 5.54 respectively.

5.3.11.1 Discussion of the effect of varying the excavation simulation method

Inspection of Figures 5.53 and 5.54 shows that the excavation simulation model chosen has a large effect on the final moment induced in the wall. This is particularly evident for the vertical moment $M_{yy}$ where changing the simulation model effectively changes the sign of the moment induced in the facing. In order to understand why this occurs it is useful to consider the two excavation models and how the differing assumptions associated with each affects the displacements and as such the moments calculated by each case.
Figure 5.53: Calculated variation in vertical moment ($M_{yy}$).

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Figure 5.54: Calculated variation in horizontal moment (M_{yy}).
Let us start by considering the ‘zero stress’ positions of the facing for each simulation method. As is discussed in Chapter 3, in order to correctly simulate the construction of a soil nailed excavation, it is important that new elements (i.e. the reinforcing) can be added to the mesh during the calculation process. In reality, the reinforcements will be placed wet onto a deformed or deforming excavation. As such the zero stress position for any layer of reinforcement is the deformed shape of the soil at the time the reinforcement hardens. Stresses in the reinforcement are only induced by subsequent displacement. In terms of our two models, this means:

- For case A, it is only the excavation of underlying stages that induces stresses in the reinforcement.
- For case B, in addition to the displacements resulting from underlying stages, the displacements due to the excavation of the current stage are considered when calculating stresses in the reinforcement.

Using this knowledge, it is possible to calculate the actual displacements inducing the reinforcement response for each model. Figure 5.55 presents these displacements relative to the depth below surface. As can be seen, the displacements are divided into a series of steps. Each step relates to a different excavation stage. By comparing the shape of the facing for the two different models at the same step it can be seen that the curvature of the facing is different for each of the two models. For the case A model, the curve of each section runs out towards the base, whilst for the case B model the situation is reversed with the curve for each section running in towards the base. It is this difference in displaced profiles that results in the calculated facing responses being so different for the two excavation simulation methods (Figures 5.53 and 5.54).

Having shown that this difference in displacement is occurring, it is now worth considering the mechanisms that have led to this behaviour. Consider the simple two stage excavation shown in Figure 5.56. Figure 5.56 (b) shows the displaced profile after the first stage of excavation. Whilst the displaced profile shown here is simply a sketch, it is consistent with the calculated shape seen throughout the investigation. In particular, note is drawn to the fact that as a result of the restraint provided by the floor of the cut, the displacements for approximately the bottom third of the excavation reduce in size the closer you come to the base of the cut. Figure 5.56 (c)
Figure 5.55: Actual displacements inducing the reinforcement response for each model.

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shows the displaced profile after the second stage of excavation. This displaced shape is essentially a scaled up version of that shown for stage one. Note that the difference between the displaced profiles for stages one and two is greatest at the base of the stage one cut. In essence, the removal of the underlying stage has caused the bottom third of the ‘stage one section’ to displace more than the top two thirds. This has occurred because the excavation of the underlying stage has removed the restraint provided by the pre-existing floor. Considering the stage one section (i.e. the face exposed by the excavation of stage one), two different displacement mechanisms have been identified. The first occurs as a result of removing stage one and is characterised by the restraint rendered by the adjacent excavation floor. The second occurs as a result of the removal of the underlying stages and is characterised by the removal of the restraint due to the pre-existing floor.

![Diagram](image)

‘case A’ displacement inducing reinforcement response

‘case B’ displacement inducing reinforcement response

Figure 5.56: Simple two stage excavation.

As an aside, it is interesting to note that the progressive facing moments previously presented in Figures 5.21 to 5.24 (Section 5.3.3) support this model of soil displacement during excavation. The moments shown in these Figures are for a ‘case A’ simulation method. If the above displacement mechanisms are correct, then the moments shown here should be the result of the removal of the support provided by the excavation floor. Consider the moment induced at a depth of 2.5m. As can be seen, a moment is first induced in the facing at this location as a result of excavating from a depth of 3m to 4m. Subsequent excavation to depths of 5m and 6m similarly
results in extra facing response. Of interest here is the fact that as the base of the excavation moves away from the inspection point (at \( d = 2.41 \text{m} \)), the change in moment due to extra excavation decreases. In simple terms, the largest change in moment is due to the excavation of the stage immediately under our inspection point and continuing excavations result in decreasing additional moment. Such behaviour is consistent with the mechanism presented above. As the excavation proceeds past the inspection point, the restraint provided by the base of the excavation is of lessening importance at our inspection point and so the continuing excavation has a diminishing influence on the moment induced.

In addition to running the ‘baseline’ models with the two excavation simulation methods, the model combinations used for the parametric study have also been run using a ‘case B’ type excavation simulation. Figures 5.57 to 5.60 present a number of selected results. Shown, are the calculated vertical moments (\( M_{yy} \)) for both simulation methods relative to the depth below surface. As can be seen, not only does the chosen simulation method affect the size and sign of the moment induced, it also changes the degree of influence that various parameters have on the facing moment.

As an example, consider Figure 5.57. This figure shows that whilst the Young’s Modulus of the soil has a large impact on the facing moment for a ‘case A’ simulation method, it has no significant influence for a ‘case B’ type model. Further, the simulation method does not affect the influence of the parameters in the same way for all of the parameters. Consider Figure 5.58, this indicates that the facing stiffness has considerable influence on the facing moment, regardless of the simulation method. On the other hand, Figure 5.59 shows that for the ‘case A’ simulation method, the soil friction angle most effects the moment at the nail heads, whilst for the ‘case B’ method it has little influence on the moment at the nail heads, but seems to be effecting the moment mid-span.
Figure 5.57: Calculated vertical moments ($M_{yy}$) using both simulation methods relative to the depth below surface.

Chapter 5: Investigation of facing response.
Figure 5.58: Calculated vertical moments ($M_{yy}$) using both simulation methods relative to the depth below surface.

Chapter 5: Investigation of facing response.
Figure 5.59: Calculated vertical moments ($M_{yy}$) using both simulation methods relative to the depth below surface.

Chapter 5: Investigation of facing response.
Figure 5.60: Calculated vertical moments ($M_{yy}$) using both simulation methods relative to the depth below surface.

Chapter 5: Investigation of facing response.
Interestingly, knowledge of the differences between the two simulation methods may provide us with a means for explaining some of the variation between the calculated facing responses. Take for example the soil nail stiffness ($E_n$). Figure 5.60 shows that for a ‘case A’ simulation $E_n$ has a relatively small effect on the facing moment. However for the ‘case B’ simulation method, there is a strong correlation between the induced facing moment and $E_n$. As has been mentioned before, for a ‘case A’ simulation, the facing response is due to the excavation of underlying stages only whilst for a ‘case B’ simulation the facing response results from the displacements due to excavation of the current layer as well as those due to excavation of underlying layers. Now whilst the removal of the underlying stages is common to both simulation methods, it is important to note that there are different amounts of reinforcement in place when these underlying stages are being excavated. Consider a small two stage excavation. For a ‘case A’ simulation the response of the top layer of reinforcement is due to the displacements of the unreinforced second stage. For a ‘case B’ simulation, the removal of the underlying stages also induces part of the moment in the facing, but it is the response of the reinforced second stage that induces this portion of the facing reaction.

This point has been specifically made here, because if the same amount of reinforcement was in place for the two simulation methods when excavating underlying layers then it may have been possible to isolate the effect of excavating the ‘current’ layer using superposition. That is, case B contains the effect of the current and the underlying layers whilst case A contains only the underlying layers so case B minus case A gives us the effect of removing the current layer. The fact that different ‘reinforced soil systems’ (i.e. soil and reinforcement members) are involved for the removal of underlying layers with the different simulation models means that, when considering the calculated behaviour it is necessary to examine the effect of both the nature of the displacements inducing moment (i.e. is it mechanism one or mechanisms one and two) and the effect of the reinforcements on the behaviour of the excavation when the underlying layers are being removed.

In terms of the calculated behaviour for the various nail stiffness (Fig 5.60), this means that $E_n$ is either:

*Chapter 5: Investigation of facing response.*
a) having a significant influence on the facing moments due to the removal of the ‘current’ construction stage and less of an influence on the moments due to the removal of the underlying stages, or

b) having a significant influence on the facing moments only when the reinforcements are in place prior to the removal of each stage, or
c) a combination of the above two points.

In brief, it is either the individual or combined action of: the displacement mechanism generating the facing moment or having the reinforcement in place when the underlying layers are removed, that has resulted in $E_n$ having differing amounts of influence.

Using this means of interrogation for the calculated results for the nail stiffness (Figure 5.60) shows that the disparity in influence is not entirely unreasonable. That is, by inspection, it is reasonable that the stiffness of the nail should have less influence on the facing reaction for the model where the mechanism driving the predicted behaviour is the response of the unreinforced underlying face sections to excavation (case A) or visa versa, have more influence when it is the response of reinforced sections driving the facing behaviour.

Similarly, the same reasoning can be applied to the other parameters. Take for example the calculated variation between facing moment and soil stiffness ($E_s$). Figure 5.57 shows that for a ‘case A’ simulation $E_s$ has a strong influence on the induced facing moment, whilst for the ‘case B’ simulation method, there is basically no change in the induced facing moment resulting from the changes to $E_s$. Applying the above reasoning leads to the following conclusions:

a) Either the influence of $E_s$ on the facing moment due to removal of the underlying layers is completed negated by an equal but opposite influence on the facing moment the due to removal of the current layer, or

b) the action of the reinforcements negates the influence of $E_s$, or
c) a combination of the two.

Again inspection shows that neither of these explanations is completely unreasonable. However, the author notes that the above explanation is not rigorous and cannot definitively explain the calculated behaviour.
5.4 Discussion of parametric study results

In general, the factors considered by the detailed study can roughly be divided into two separate groups:

1. ‘Active wedge’ type parameters (γ, φ, H, $S_v$, and $S_{i1}$). These are typically associated with determining an active load with consideration given to nail spacing.

2. ‘Soil/reinforcement interaction’ type parameters ($E_n$, $E_s$, $E_f$, $K_o$ and construction related issues). These are predominately related to the interaction between the soil and the reinforcing elements.

Current facing design methods rely heavily on the ‘active wedge’ type parameters. Nevertheless, the findings of the more recent research projects (FHWA-SA-96-069R, 1998 and Recommendations Cloutere, 1991) have typically noted the ‘soil/reinforcement interaction’ type parameter’s role in determining reinforcement response within the written text. Further, the FHWA Manual has attempted to include $E_f$ (as indicated by facing thickness) through the use of a flexure pressure factor, $C_F$ (see section 2.2.4).

If we consider the calculated response of the ‘active wedge’ type parameters, it can be seen that their performance conforms to that predicted by the design methods. That is, the change in behaviour predicted by the design methods as a result of a parameter variation is typically mirrored by the calculated response. In particular, the calculated results correspond well with established thought, which maintains that the most critical of these parameters is the nail spacing (FHWA-SA-96-069R, 1998 and Recommendations Cloutere, 1991, see Section 5.1.1). Inspection of Figures 5.9 to 5.28 shows that horizontal nail spacing has the largest effect for the considered range of values, particularly when horizontal moment is considered.

Nonetheless, the considerable influence shown by the ‘soil/reinforcement interaction’ type parameters on the calculated facing response raises a number of issues with regard to why, with the exception of the FHWA ‘Flexure Pressure Factor $C_F$’,
current design methods do not specifically include such parameters within their facing design methods.

In answering this, it is important to note that the findings of the two most recent research projects into the soil nailed method (FHWA DP-103 and Clouterre) are united, in that they maintain the facing ground interaction depends on these ‘soil/reinforcement interaction’ type parameters. The FHWA Manual for design of soil nailed walls (FHWA-SA-96-069R, 1998) states that ‘the magnitude of the nail head tensile load depends on the timeliness of the nail installation, the ground stiffness characteristics, the nail tensile stiffness, the nail-grout ground interface stiffness and the facing stiffness’. The report on the findings of the French National Research Project (Recommendations Clouterre, 1991) states that ‘The values of $T_o/T_{max}$ depend on a certain number of parameters (the stiffness of the soil, rigidity of the facing, rigidity of the nails, depth and spacing of nails’). Notwithstanding this, the design methods are similarly united in that they are based predominately on the ‘active wedge’ type parameters. Recall, that the FWHA method calculates a design earth pressure at the facing based upon either a reduced active earth pressure or a percentage reduction in the maximum nail load, whilst the French Method presented in Recommendations Clouterre, 1991 uses a reduction in the maximum nail load dependant upon the nail spacing. Maximum nail loads are typically determined using a limit state analysis of the excavation, which is typically more dependant upon the ‘active wedge’ type parameters than the ‘soil/reinforcement interaction’ parameters.

A possible explanation for this disparity lies in the nature of the ‘soil/reinforcement interaction’ parameters. In terms of developing a system for design, the ‘active wedge’ type parameters are much more attractive because: they are basic soil parameters that most engineers will be comfortable with, they are typically easier to determine during the initial investigation stage, they are also likely to be more reliably determined prior to construction and finally they are required for the design of the soil nails and as such will need to be determined regardless of which design method is chosen. Further, it is important to note that the correlations between the ‘active wedge’ type parameters and earth pressure at the facing are empirically based. As such, the effect of any typical variation in the ‘soil/reinforcement interaction’ parameters is included either in the correlation itself, or in the scatter inherent in these.
correlations. With this in mind, it is prudent to consider Figures 2.14 and 2.15, which present normalized nail head load and maximum nail load with respect to the depth of excavation for up to eleven monitored walls. These results, in addition to the findings of previous research, form the basis of the FHWA method for determining design earth pressures at the facing. As can be seen, a large amount of scatter is evident, suggesting additional factors may be influencing the behaviour.

Conversely, the findings of the parametric study should be considered in conjunction with the widespread use of limit equilibrium methods for the analysis of soil nailed walls. Essentially, the applicability of limit equilibrium methods for the design of soil nailed walls has been proved through wide spread use. As a possible explanation for this disparity between the calculated and ‘empirical’ results, it is worth considering the role of slip between the soil nails and the soil. There will be many situations where the overloading of a layer of nails will not present a major problem, because it will simply result in additional slip between the nail and the soil. This is particularly the case for soils where the loss of strength due to sustained shearing is minimal. In this respect, the findings of the parametric study may simply support the current understanding that good practice should call for design of soil nails so that failure is of a ductile nature (e.g. slip between the nail and the soil) as opposed to a more brittle mechanism (rupture of the bar). Regardless, the role of the ‘soil/reinforcement interaction’ type parameters is worthy of mention if only to highlight the potential pitfalls inherent in the limit equilibrium design approach.

Whilst an aside from the main thrust of this document, it is also worth noting that the relationship shown between the ‘soil/reinforcement interaction’ type parameters and the reinforcement response seems to support the idea that a limit equilibrium approach to design of soil nails may not always be entirely appropriate. As is detailed in Chapter 1, the current recommended design methods for soil nailed walls use a limit equilibrium analysis to determine the soil nail design loads. Implicit in the use of a limit equilibrium analysis is the assumption that the displacements required for the soil to reach its limit state will not result in excessive stresses in the reinforcements. The forces in the reinforcements should be primarily as a result of providing the additional force required to maintain stability of the excavation, not unloading of the soil. The parametric study has shown that parameters associated with the stress-strain
behaviour of the soil and independent of the limit equilibrium strength of the soil (E_m, E_s, E_r and Ko) influence the facing response significantly. The inference here is that these parameters could similarly be having influence on the behaviour of the whole reinforcement system and as such a limit equilibrium approach may not be appropriate. Again, it is useful to consider the scatter evident in correlations between soil strength parameters and normalised measured maximum nail loads (Figure 2.15) in light of these concerns. If the maximum nail loads measured in reality were simply a result of the retaining of an active wedge then normalising these measured maximums using the ‘active wedge’ type parameters should give reasonably uniform results.

The final matter identified by the parametric study worthy of note, is the large influence the excavation simulation method used has on the calculated facing response. This issue has relevance beyond the simple assessment of errors introduced by modeling simplifications. In particular, it shows the impact construction related issues could have on the behaviour of a soil nailed wall. As an example consider the practice of using excavation slots to construct a soil nailed wall in unfavourable ground. Figure 1.10 shows this process graphically. As can be seen, the technique employs the use of unexcavated slots to support adjacent excavated sections. The action of the unexcavated soil helps support the excavated sections until reinforcing is placed. In terms of the model excavation simulation methods, the unexcavated slots act to reduce the percentage of the final displacement occurring before the reinforcements are installed. This pushes the soil response closer to that simulated in case B.

Many other construction related issues have the potential to change the degree of relief occurring before the reinforcements are installed and as such affect the facing response (staged construction of the wall’s width, the use of temporary berms, the choice of drill and grouted nails with shotcrete facing as opposed to driven nails with prefabricated facing panels). However, it is again important to note that the current design methods have essentially proved their applicability through continued, predominately problem free, use. There are a number of possible scenarios, which we can postulate could explain why the variation in facing response identified by the parametric study has not been reflected in problems with actual facings.

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The first relates to the simple fact that, unlike a reinforced concrete beam, the reinforcement for a shotcrete facing generally consists of a single layer of mesh or bars placed in the centre of the concrete section (Figures 2.1 and 2.2 show typical setouts as presented by the FHWA). This means that the capacity of the facing is basically the same for positive bending as it is for negative bending. Inspection of Figures 5.53, 5.54 and 5.57 to 5.60 shows that the two simulation cases yield moments of similar magnitudes but with opposing signs.

Another possible explanation exists in the fact that the two simulation methods describe the limits of the expected behaviour. It is likely that the actual behaviour of the facing elements sits between these two extremes, with some but not all of the displacements associated with a particular step occurring before the reinforcement begins accumulating stress. This means that the moment induced in the facing likely lies between the two plots shown in Figure 5.53 and 5.54.

Interestingly, there is some empirical evidence to support this idea. Again, note is drawn to Figures 2.14 and 2.15, presenting the normalised monitoring results of actual soil nailed excavations. One area of major difference between the two excavation simulation methods is the response of the final layer of reinforcement. By definition, the last layer of reinforcement for a case A simulation is redundant (it is only the excavation of underlying stages that induces stresses for this simulation case). So, by studying the monitored performance of the last layer of reinforcement it is possible to see which of the two simulation methods more accurately describes the actual behaviour. As can be seen, the final layer of reinforcement, whilst not redundant, does receive proportionally less load. In fact, the text accompanying Figures 2.14 and 2.15 in the FHWA Design Manual (FHWA-SA-96-069R, 1998) states that ‘within the lower one-quarter to one-third of the wall height, the maximum nail loads decrease significantly and appear in general to trend from the upper wall maximum values ... to a value of zero at the base of the wall. These observations are consistent with those reported by others’.

Now whilst this phenomenon has been previously explained through the action of soil arching, (reference paper ‘arch in soil arching’, Recommendations Clouterre, 1991) it is possible that part of this behaviour can be attributed to having a portion of the total
displacements occur before the reinforcement begins accumulating stresses. That is having a response in between those described by the two considered excavation simulation methods. This said, it should also be noted that a reasonable scatter exists in the monitored results and if points for individual projects are considered it can be seen that whilst some monitored walls have behaved similar to a ‘case A’ model, others are similar to a ‘case B’. As has been mentioned previously, this suggests that factors additional to those used for normalising the results are influencing the reinforcement response. It is possible that this scatter may be further refined if the construction related issues discussed above are taken into account.

5.5 Potential continuing research

Two aspects of the presented results arguably provide the most promising potential for continuing research. The first stems from the investigation of the excavation simulation methods and the two displacement mechanisms subsequently identified (Section 5.3.11.1, see Figures 5.55 and 5.56). In summary, it was shown that the calculated displacement of a section of excavation facing due to the removal of the ‘current’ stage of the excavation was considerably different from that due to excavation of the underlying layers.

Current facing design considers facing displacement to be similar to that of a floor slab, supported on columns (Figure 5.61 demonstrates this graphically). Figure 5.56 shows that the facing response calculated using the presented program is significantly different, with the nature of the restraint provided by the floor of the excavation interacting with the reinforcement installation process to dictate the final displacements inducing response in the facing.

There are a number of experimental and numerical investigations, which could be used to investigate exactly what deformation modes are loading the facing. Experimentally, it would be important to see if the behaviour predicted by the numerical model is actually occurring in the field. By measuring displacement on a facing panel at a number of locations (both at nail heads and locations in between), the actual displaced shape of the facing could be determined throughout the
excavation process. Numerically, it is still important to check that modeling simplifications have not impacted on the results. One such simplification that theoretically has the potential to impact the correctness of the presented results is the assumption of full adhesion between the facing and the soil. It is possible that the restraint provided by the floor of the excavation will have less impact on the facing if a joint element with low tensile strength was introduced between the facing and the soil. If this were the case then the calculated results should arguably conform more fully to the current understanding.

Figure 5.61: Assumed displaced profile of shotcrete facing.

The other aspect of the presented results worthy of continued research similarly stems from the investigation of the influence of the excavation simulation method. As was mentioned, the two excavation simulation methods considered by this thesis generally represent the limits of the possible behaviour. It was shown that the choice of which of the two limits used impacted heavily on not only the quantum of the facing response and also the relative importance of various parameters on determining this facing response. This inherent variability warrants further investigation, particularly if the two displacement modes discussed above are shown to actually be inducing facing response.
In terms of numerical methods there are a number of methods that could be used to investigate the response of excavation sequences lying between the two limits. Material 'smearing', where the 'partially' excavated elements are modeled using reduced stiffness materials, could be used to model stress relaxation prior to reinforcement installation. Experimentally, the collation of monitored results with particular reference to the behaviour of the last row of reinforcements would provide a valuable insight into what degree of deformation occurs prior to the reinforcements starting to accumulate stress. Further, if a range of behaviour is found to occur it would be useful to see if parameters such as soil type and reinforcement construction method could help predict the behaviour of the last row of reinforcement (i.e. which of the two limits is more appropriate). As is discussed in Section 5.4, there is a theoretical basis for such parameters affecting the amount of displacement prior to reinforcement installation and as such it would be worthwhile researching whether this is similarly reflected in the monitored results.

In summary, the work conducted as part of this thesis has shown that there is a strong theoretical basis for the argument that construction related issues have a significant impact on the behaviour of the facing of soil nailed excavations. Whilst this parameter has been given some mention in the current research as having an effect on the facing response, its influence is largely unquantified. The author proposes that assessing whether the degree of impact predicted by this thesis is actually occurring in the field and if so, providing some means of predicting the potential influence for a given structure form the basis of the next line of research opportunities.
Chapter 6: Conclusions

6.0 Conclusions

This thesis outlines work conducted to investigate the response of the facing elements in soil nailed excavations. Existing design methods are reviewed and a gap in the knowledge base identified with respect to design of the facing elements.

The development and validation of a three dimensional finite element program capable of directly modeling the construction of soil nailed excavations is presented with a view to utilizing this program to investigate the behaviour of the facing elements. Through a process of comparison between the published behaviour of monitored excavations and corresponding numerical simulations, it was found that the mechanisms of soil-reinforcement interaction dictating the behaviour of a soil nailed wall were complex and that presently available analysis methods needed to simplify this behaviour in order to maintain computational efficiency. Notwithstanding, it was also found that the presented formulation could predict the displacement behaviour of the soil mass, the failure wedge geometry and reasonably predict the reinforcement response of a monitored experimental wall using published parameters.

A parametric study was then undertaken targeting factors presently used in facing design or commonly held to have substantial influence on the facing behaviour. The calculated influence of these parameters was shown to conform well to that predicted by their respective design methods. In particular, the calculated results:

- Corresponded well to established thought, which maintains that the most critical of these parameters is nail spacing.
• Confirmed the importance of soil-reinforcement interaction in determining the reinforcement response (the soil stiffness and facing stiffness were found to significantly influence the induced facing response).

Additionally, the influence of a number of factors, not included in the current design methods but nonetheless still recognised by the current research as being of importance, was demonstrated. In particular, it was shown that construction related issues could potentially have a large influence on the facing behaviour, with the chosen construction simulation method being shown to effect not only the size and sense of the calculated facing response but also the relative importance of various parameters in determining this response.

In exploring the influence of construction related issues, two separate displacement mechanisms inducing response in the facing were identified and described. These calculated displacement mechanisms are characterised by the restraint provided by the floor of the excavation and do not conform to the currently accepted deformation behaviour of soil nail facing elements which typically considers the action of the soil nail head as paramount in determining the facing reaction.

This finding, regarding the importance of construction related issues and the associated displacement mechanisms, was discussed in light of the typically untroubled performance of actual soil nailed facing systems. A number of possible explanations for this disparity between the calculated and observed response were postulated and a program for continuing research proposed.

In summary the work included herein has:

• Summarised the body of research current at the time of writing regarding the design of soil nailed excavation, with particular emphasis being placed on the design of facing elements.

• Identified a knowledge gap with regards to the loads experienced by the facing elements.

• Developed a 3D finite element program capable of simulating the construction of a soil nailed excavation and directly outputting the reinforcement response.
• Verified the program, both in terms of its correct application of the intended programming theory and in its ability to reasonably predict the behaviour of actual monitored soil nailed walls.

• Investigated the calculated variation in facing moment as a result of changes to a series of selected parameters. This both confirmed that the behaviour predicted by the existing design methods was mirrored in the calculated results and showed that there was a strong theoretical basis for the argument that construction related issues have a significant impact on the behaviour of facing elements.

• Lastly the results were reviewed in light of simplifying assumptions made during the course of the work and a programme for continuing work proposed.
References


References


References


References


Appendix A

Interpolation Functions for 20 Node Brick Elements

Interpolation Functions for 8 Node Shell Elements
20 Node Brick Element - Formulation of Interpolation Functions (eg Potts and Zdravković):

Mid-side Nodes:

\[ N_9 = \frac{1}{4} \left( 1 - S^2 \right) \left( 1 - T \right) \left( 1 - U \right) \]
\[ N_{10} = \frac{1}{4} \left( 1 - T^2 \right) \left( 1 + S \right) \left( 1 - U \right) \]
\[ N_{11} = \frac{1}{4} \left( 1 - S^2 \right) \left( 1 + T \right) \left( 1 - U \right) \]
\[ N_{12} = \frac{1}{4} \left( 1 - T^2 \right) \left( 1 - S \right) \left( 1 - U \right) \]
\[ N_{13} = \frac{1}{4} \left( 1 - U^2 \right) \left( 1 - S \right) \left( 1 - T \right) \]
\[ N_{14} = \frac{1}{4} \left( 1 - U^2 \right) \left( 1 + S \right) \left( 1 - T \right) \]
\[ N_{15} = \frac{1}{4} \left( 1 - U^2 \right) \left( 1 + S \right) \left( 1 + T \right) \]
\[ N_{16} = \frac{1}{4} \left( 1 - U^2 \right) \left( 1 - S \right) \left( 1 + T \right) \]
\[ N_{17} = \frac{1}{4} \left( 1 - S^2 \right) \left( 1 - T \right) \left( 1 + U \right) \]
\[ N_{18} = \frac{1}{4} \left( 1 - T^2 \right) \left( 1 + S \right) \left( 1 + U \right) \]
\[ N_{19} = \frac{1}{4} \left( 1 - S^2 \right) \left( 1 + T \right) \left( 1 + U \right) \]
\[ N_{20} = \frac{1}{4} \left( 1 - T^2 \right) \left( 1 - S \right) \left( 1 + U \right) \]
Corner Nodes:

\[ N_1 = \frac{1}{8} (1 - S)(1 - T)(1 - U) - \frac{1}{2} (N_9 + N_{12} + N_{13}) \]

\[ N_2 = \frac{1}{8} (1 + S)(1 - T)(1 - U) - \frac{1}{2} (N_9 + N_{10} + N_{14}) \]

\[ N_3 = \frac{1}{8} (1 + S)(1 + T)(1 - U) - \frac{1}{2} (N_{10} + N_{11} + N_{15}) \]

\[ N_4 = \frac{1}{8} (1 - S)(1 + T)(1 - U) - \frac{1}{2} (N_{11} + N_{12} + N_{16}) \]

\[ N_5 = \frac{1}{8} (1 - S)(1 - T)(1 + U) - \frac{1}{2} (N_{13} + N_{17} + N_{20}) \]

\[ N_6 = \frac{1}{8} (1 + S)(1 - T)(1 + U) - \frac{1}{2} (N_{14} + N_{17} + N_{18}) \]

\[ N_7 = \frac{1}{8} (1 + S)(1 + T)(1 + U) - \frac{1}{2} (N_{15} + N_{18} + N_{19}) \]

\[ N_8 = \frac{1}{8} (1 - S)(1 + T)(1 + U) - \frac{1}{2} (N_{16} + N_{19} + N_{20}) \]
8 Node Shell Element - Formulation of Interpolation Functions (eg Potts and Zdravković):

\[ N_1 = \frac{1}{4} (1 - \xi)(1 - \eta)(-1 - \xi - \eta) \]

\[ N_2 = \frac{1}{2} (1 - \xi^2)(1 - \eta) \]

\[ N_3 = \frac{1}{4} (1 + \xi)(1 - \eta)(-1 + \xi - \eta) \]

\[ N_4 = \frac{1}{2} (1 + \xi)(1 - \eta^2) \]

\[ N_5 = \frac{1}{4} (1 + \xi)(1 + \eta)(-1 + \xi + \eta) \]

\[ N_6 = \frac{1}{2} (1 - \xi^2)(1 + \eta) \]

\[ N_7 = \frac{1}{4} (1 - \xi)(1 + \eta)(-1 - \xi + \eta) \]

\[ N_8 = \frac{1}{2} (1 - \xi)(1 - \eta^2) \]
Appendix B

Fortran Code for:

Td8auto
Integl
Sbolt
Bltlod
Shotcr
Gen3D
Initstr
Overb
C PROGRAM FOR ELASTO-PLASTIC THREE DIMENSIONAL SOIL ANALYSIS
C******************************************************
C
#1. IMPLICIT DOUBLE PRECISION USED THROUGHOUT
INCLUDING GRAVITY LOADING LINEAR, QUADRATIC
AND CUBIC SERENDIPITY ELEMENTS INCLUDED

#2. JMR ORIGINAL DATES FROM : 3 MAY 1982
MOST RECENT MODIFICATION : 8 JUL 1982
MOST RECENT PAPER TAPE : NONE 1982
LAST MODIFICATION : 1996

TD2.for is a newly modified version of td1.for.
This version now has six degrees of freedom for each
20 noded element. - modified 3/98.

TD4.for - another modified version, this one
incorporates
a skyline solver - which should increase
computational
efficiency.

td5.for - This version incorporates a simple beam
element
to simulate rock bolt support, and has increased
parameters
in order to handle large meshes.

Td6.for - This version removes the boundary condition
entries from the stiffness matrix, in an attempt to
reduce the memory required to run the program.

C******************************************************
C DICTIONARY OF VARIABLE NAMES
C
C PART 1 CONTRO
C
C NUMMOD : TOTAL NUMBER OF NODAL POINTS IN THE STRUCTURE
C NUMEL : TOTAL NUMBER OF ELEMENTS IN THE STRUCTURE
C NNODE : NUMBER OF NODES PER ELEMENT
C NDOFN : THE NUMBER OF DEGREES OF FREEDOM PER NODAL POINT
C NDIME : NUMBER OF COORDINATE COMPONENTS REQUIRED TO DEFINED
C EACH NODAL POINT
C NSTRE : NUMBER OF STRESS COMPONENTS AT ANY POINT
C NGAUS : NUMBER OF GAUSS RULE ADOPTED
C NPROP : THE NUMBER OF MATERIAL PARAMETERS REQUIRED TO
C DEFINE THE CHARACTERISTICS OF MATERIAL COMPLETELY
C NMATS : NUMBER OF MATERIAL SETS
C NVFIX : TOTAL NUMBER OF BOUNDARY POINTS
C NEVAB : NUMBER OF VARIABLES PER ELEMENT
C ICASE :
C NCASE : THE TOTAL NUMBER OF LOAD CASES TO BE SOLVED FOR
C
C PART 2 LGDATA
C
C COORD : COORDINATES OF NODAL POINTS
C PROPS : MATERIAL PROPERTIES FOR EACH MATERIAL SET
C PRESC : VALUES OF THE NORMAL AND TANGENTIAL LOAD INTENSITIES
C ELOAD : NODAL FORCES FOR EACH ELEMENT

Appendix B
program ETDSAl

IMPLICIT real*8 (A-H,O-Z)

CHARACTER*30 INFIL, OUTF, COMF, pltf, cont, plsf
CHARACTER CH*1
CHARACTER TITLE*80

DIMENSION ISUM(200), FLOAD(200), DEFLN(200),
* CNVG(200), IITER(200), numpos(150), nshmat(150)

COMMON / CONTRO / NUMNOD, NUMEL, NNODE, NDOF, NNDME, NSTRE,isc,
+ istel, istcod, NGAUS, NPROP, NMAT, NFX, NEVAR,
+ ICASE, NCASE

COMMON / LGDATA / COORD(9000, 3), PROPS(50, 10), PRES(200, 3),
+ ELOAD(1700, 96), NODES(1700, 32),
+ MATN(1700)

COMMON / WORK / ELCOD(3, 32), SHAPE(32), DERIV(3, 32), DMATX(6, 6),
+ CARTD(3, 32), DBMAT(6, 96), BMATX(6, 96),
+ POSGP(3), WEIGP(3), GPCOD(3),
+ NEROR(24), stpln(6000, 5)

COMMON / STRES/ EPS(6), STRF(1700, 6, 27), STRO(1700, 6, 27),
* DSTR(1700, 6, 27), IPEL(1700, 27)

COMMON / SUB5 / NOD, NOMNO(350), NOST, ISTRE(150), ISTR, iplcod,
+ plval
COMMON / PLAS / BI(54000), DDIS(54000), RESID(54000)
COMMON / BLOC / BX(54000), BXX(54000), TDIS(54000), X(96, 96),
+ stdis(1700, 48), btdis(500, 12)
COMMON / SOLV / AM(24000000), MAXBW
COMMON / SUB2 / ITEMP(54000), NT, IPLTT
COMMON / INNN /
MNLI, MAXIT, NNN, NLI0LD, TOTAL, FMULT, CONV, NDSUM, IDIR

common /excav/ iexel(150), istat(1700), nce, maxex
common /shotcr/ shprop(20, 3), ishot(1700), ishpso(1700),

Appendix B
ishmat(1700)
common / diag/ kdiag(54000),ldiag(54000)
common / bolt / bltnat(500), bltvar(500,4), nbolt(500,2),
+ numbolt, nbtemp
common /setesh/ numshot(150),nes

COMMON /SOLVL/ BB(54000), NVAR, nf(54000), nfvar,bb1(54000)
COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

PI = 3.14159265
C==== Open logical unit numbers..0 vdu...2 read...3 write...4 plot.
LJ0 = 0
LJ2 = 2
LJ3 = 3
LJ4 = 4
LJ5 = 5
LJ6 = 6

WRITE(LU0,/(A)) ' What is the control file name ?: '
READ(LU0,101) Cont
101 FORMAT(A)

OPEN(LJ5, FILE = cont, STATUS = 'UNKNOWN')
read(lu5,*)numrun

do 1034 ico=1,numrun

read(lu5,101)comf

DO 25 M = 1, 20
   CH = COMF(M:M)
   IF(CH.EQ. ' ') GO TO 26
   KCH = M + 3
   INFIL = COMF
   OUTF = COMF
   pltf = comf
   plsf = comf
   INFIL(M:KCH) = '.DAT'
   OUTF (M:KCH) = '.OUT'
   pltf (m:kch) = '.plt'
   plsf (m:kch) = '.pls'

OPEN(LU2, FILE = INFIL, STATUS = 'UNKNOWN')
OPEN(LU3, FILE = OUTF, STATUS = 'UNKNOWN')
OPEN(LU4, FILE = pltf, STATUS = 'UNKNOWN')
OPEN(LU6, FILE = plsf, STATUS = 'UNKNOWN')

READ (LU2,'(A)') TITLE
WRITE(LU3,'(A)') TITLE

C==== Call the input s/r

CALL INPUT
NGPTS = NGAUS*NGAUS*NGAUS

Appendix B
C==== Read load data and develop nodal loads
CALL LOAD3D

C==== Read in the initial stresses as found by gen3d.

    write(lu3,1004)
    write(lu3,1006)
1004 format(// ' Initial stresses in mesh '
    / ' ---------------------------------- ')
1006 format( ' Element ',3x,' Gauss Pt. ',5x,' x ',6x,' y ',6x,' z '
    / ' ---------------------------------- ')

    do 405 iel=1,numel
    do 1007 igpt=1,ngpts
        read(lu2,*) (str0(iel,j,igpt),j=1,3)
        do 1047 j=4,6
            str0(iel,j,igpt) = 0.0
        1047 continue
    write(lu3,1008) iel,igpt,(str0(iel,j,igpt),j=1,6)
1008 format(15,5x,15,7x,6f10.4)
1007 continue
405 continue

    do 5000 i = 1, numel
    do 6000 j = 1,nstreq
    do 7000 k = 1,8
        strf(i,j,k) = 0.0
        dstr(i,j,k) = 0.0
7000 CONTINUE
6000 CONTINUE
5000 CONTINUE

    do 1038 i=1,nvar
    bxo(i) = 0.0d0
    bx(i) = 0.0d0
    tdis(i) = 0.0d0
1038 continue

C****************** START LOAD INCREMENT LOOP ******************
DO 900 NLI = 1, MNLI
write(lu4,'(/i4)')nli
write(lu6,'(/i4)')nli

C==== Read in excavated elements - update istat

    read (lu2,*) nee
    write (lu3,'(/a)')'No. of elements excavated this load step: '
    write (lu3,'(i3)') nee
    if (nee .gt. maxex) then
        write (lu0,1002) maxex
        1002 format('Too many elements have been excavated'/
            *' Maximum allowed to be removed =','i4)
        stop
    endif

    if (nee .gt. 0) then
        read (lu2,*) (islex(i), i=1,nee)
        write (lu3,'(/a)') 'Elements to be excavated '

Appendix B
write (lu3,'(5i5)') (iexl(i), i=1,nee)
do 1003 i=1,nee
  k = iexl(i)
  irstat(k) = 0
  do 1003 j = 1,ngpts
  ipel(k,j) = 0
1003 continue
endif

C== Read in the shotcreting information.

read (lu2,*) nes
write (lu3,'(/a)') 'No of elements shotcreted before this excav.
',
write (lu3,'(i3)') nes
if(nes.gt.0) then
  read (lu2,*) nsm
  read (lu2,*) (numshot(i),numpos(i),nshmat(i),i=1,nes)
do 48 k=1,nsm
  read (lu2,*) i,(shprop(i,j),j=1,3)
48 continue
  write(lu3,'(/a)') ' Elements to be shotcreted & position code
    write(lu3,'(2i5)') (numshot(i),numpos(i),i=1,nes)
do 47 i=1,nes
  k = numshot(i)
  ishop(k) = 1
  ishpos(k) = numpos(i)
  ishmat(k) = nshmat(i)
47 continue
endif

C== Set Initial shape of shotcrete.
if(nes.gt.0) then
  call shotset
endif

C== Read in the rock bolting information.

read(lu2,*)iblt
write (lu3,'(/a)') 'No of rock bolts placed before this excav.'
write (lu3,'(i3)') iblt
if(iblt.gt.0) then
  nbtemp = numblt + 1
  numblt = numblt + iblt
  read (lu2,*) nrbm
  read (lu2,*)
  + (nbolt(i,1),nbolt(i,2),blmt(i),i=nbtemp,numblt)
do 49 k=1,nrbm
  read (lu2,*) i,(bltvar(i,j),j=1,4)
49 continue
write(lu3,'(/a)')
  + ' This step 1st & last bolt node, & material no. '
write(lu3,'(32i6)')
  + (nbolt(i,1),nbolt(i,2),blmt(i),i=nbtemp,numblt)
endif

C== Set the initial rock bolt positions.
if(iblt.gt.0) then

Appendix B
call boltset
endif

c==== Assemble the stiffness matrix
   call stif3d

c== Assemble shotcrete stiffness matrix
   call shotcr

c== Assemble rock bolt stiffness matrix.
   call sbolt

c== Test for and rectify non-zero diagonal terms.

   do 1001 i=1,nfvar
      k = kdiag(i)
      if(abs(am(k)).lt.1.0e-06) then
         am(k) = 1.0
      endif
   c   write(lu3,'(i7,e13.4)') i,am(k)
   1001 continue

c   if(nli.eq.4) then
   c   write(lu4,'(/a)') ' STIFFNESS MATRIX after'
   c   do 2101 j=1,nvar
   c      il = nf(j)
   c      if(il.eq.-1.or.il.eq.1) goto 2101
   c      if = kdiag(il)
   c      ig = ldiag(il)
   c      ih = il - ig + 1
   c      ij = kdiag(il-1)
   c      if(ij.gt.ih) goto 2101
   c      write(lu3,'(i5)') j
   c      WRITE (LU3,'(12e13.4)') (am(i), i=ih,if)
   c      write(lu4,'(15)') j
   c      WRITE (LU4,'(12e13.4)') (am(i), i=ih,if)
   c   2101 continue
   c   endif

c==== Perform Choleski reduction on stiffness matrix.
   call SPARIN

c==== Calculate excavation loads
   if(nel.gt.0) then
      Call integl
   endif

   WRITE(LU3,28)
   WRITE(LU0,28)
28 FORMAT('Iteration No. G/Pts. Convergence'/
*13X,'Plastic',7X,'Criterion %'/1X,38(IH-))

C*************** BEGIN ITERATION LOOP ***********************

   DO 830 NITER = 1, MAXIT
   ITT = NITER

   DO 410 I = 1, NVAR
   BXO(I) = BX(I)
410 BX(I) = 0.0

Appendix B
C---- Apply plastic correction to R.H.S.
   CALL PCOR(ml,itt)

C==== Add correction vector to incremental load vector
   DO 11 I = 1, NVAR
      11 BB(I) = BI(I) + BX(I)

   DO 46 i = 1, Numnod
      k = 6*i
      k2 = k - 1
      k3 = k - 2
      ki = k - 3
      l = k - 4
      m = k - 5
      DO 46 j = 1, 6
         il = [i5, 6el3.4] 1, bi(bm), bi(1), bi(k1), bi(k3), bi(k2),
         bi(k)
      46 CONTINUE

   DO 1039 i = 1, nvar
      bbl(i) = 0.
   1039 CONTINUE

C==== Change bb() so that it is compatible with am().
   DO 76 i = 1, nvar
      a = bb(i)
      ib = nf(i)
      IF(ib.eq.-1) GO TO 76
      bbl(ib) = a
   76 CONTINUE

C       write(lu3,’(a)’)’ BB vector after ’
C       write(lu3,’(6el3.4)’) (BBI(i),i=1,nfvar)

C==== Solve equations
   CALL SPABAC

C==== Change bbl() load vector back to bb() config.
   DO 79 i = 1, nvar
      ia = nf(i)
      IF(ia.eq.-1) THEN
         bb(i) = 0
      ELSE
         bb(i) = bbl(ia)
      ENDIF
   79 CONTINUE

C       write(lu3,’(a)’)’ made it to 1 ’
   DO 89 I = 1, NVAR
      DDIS(I) = BB(I)
   89 CONTINUE

C==== Calculate change in stress
   CALL STRE3D

C       write(lu3,’(a)’)’ D-Stress in gp1, elmts 1-8 ’
C       write(lu3,’(3el14.4)’) ((dstr(i,j,1),j=1,6),i=1,8)

Appendix B
DO 680 LL = 1, NUMEL
   if(istat(ll) .eq. 0) goto 680
   MAT = MATNO(LL)
   SSTR = PROPS(MAT,5)
   FH = PROPS(MAT,3)
   S = SIN(PH)

C==== Compute final stress
DO 681 NGP = 1, NGPTS
DO 640 M = 1, NSTRE
   640 STRF(LL,M,NGP) = STR0(LL,M,NGP) + DSTR(LL,M,NGP)
   c IF(NLI.EQ.1) GO TO 27

C==== Evaluate failure criterion to see if element is plastic.
   SX = -STRF(LL,1,NGP)
   SY = -STRF(LL,2,NGP)
   SZ = -STRF(LL,3,NGP)
   TXY = -STRF(LL,4,NGP)
   TYZ = -STRF(LL,5,NGP)
   TXZ = -STRF(LL,6,NGP)
   SM = (SX + SY + SZ)/3.0
   XX = (SX*SX + SY*SY + SZ*SZ - (SX*SY + SY*SZ + SZ*SX))/3.
   + TXY*TXY + TYZ*TYZ + TXZ*TXZ
   SB = SQRT(XX)
   IF(SB .EQ. 0.0) THEN
      WRITE(0,'(/A,E13.4)') ' SB is zero *STOP*!', SB
      STOP
   ENDIF
   X1 = 2.*SX - SY - SZ
   X2 = 2.*SY - SX - SZ
   X3 = 2.*SZ - SX - SY
   XJ3 = X1*X2*X3/27.0 - X1*TYZ*TYZ/3.0 - X2*TXZ*TXZ/3.0 -
         X3*TXY*TXY
   VAL = 0.
   IF(SB.GT.0.) THEN
      VAL = 3.*SQRT(3.)*XJ3/2./SB**3
      if(val.gt.1.0) val = 1.0
      if(val.lt.-1.0) val = -1.0
   ENDIF
   XLODE = ASIN(-VAL)/3.
   DEG = 180.*XLODE/PI
   S3 = SIN(3.*XLODE)
   C = COS(3.*XLODE)
   TEST = ABS(deg)

C==== Transition surfaces. Starting at 25 degrees Lode angle.
   IF(TEST.GT.25.) THEN
      SIGN = -1.
      IF(XLODE.LE.0.) SIGN = 1.
      AV = 1.432052 + 0.406942*SIGN*S
      BV = 0.544291*SIGN + 0.673903*S
      FUNC = SM*S + SB*(AV - BV*S3) - SSTR*COS(PH)
   ELSE
      C==== Planar part of Mohr-Coulomb surface
      CL = COS(XLODE)
      SL = SIN(XLODE)
      Q1 = CL - S*SL/SQRT(3.)
      FUNC = SM*S + SB*q1 - SSTR*COS(PH)
   ENDIF

C WRITE(LU3,*') 'Func',func
C write(lu3,*) ph,sstr,sx,sy,sz,txy,tyz,tzx

Appendix B
IF (FUNC .GE. 0.0) GO TO 660
IF (IPEL(LL,NGP) .GE. 0) GO TO 660
GO TO 681

C==== Bring stress back to yield surface when element first plastic.
C 660 CALL CORCTN(LL,NGP)
C 660 CONTINUE

C==== Recompute corrected stress increment.
IF (IPEL(LL,NGP) .EQ. 0) IPEL(LL,NGP) = NLI
DO 670 M = 1, NSTRE
670 DSTR(LL,M,NGP) = STRF(LL,M,NGP) - STR0(LL,M,NGP)
681 CONTINUE
680 CONTINUE

C==== Determine the number of plastic Gauss points
NEPL = 0
DO 740 I = 1, NUMEL
DO 740 NGP = 1, NGPTS
740 IF (IPEL(I,NGP) .GT. 0) NEPL = NEPL + 1

C==== Evaluate maximum change in correction vector
IF (NEPL.GT.0) THEN
SAB = 0.0
SDIF = 0.0
DO 78 I = 1, NVAR
ADIF = ABS(BX(I) - BXO(I))
IF(ADIF.GT.SDIF) SDIF = ADIF
AAB = ABS(BX(I))
IF(AAB .GT. SAB) SAB = AAB
78 CONTINUE
IF(SAB.GT.0.0) CONC1 = SDIF*100./SAB
ENDIF

WRITE (LU3,18) NITER,NEPL,CONC1
WRITE (LUO,18) NITER,NEPL,CONC1
18 FORMAT(I6,I12,E20.4)

IF(NITER.GT.1) THEN
IF(CONC1.LE.CNV) GO TO 27
ENDIF

C ***** END ITERATION LOOP ******************************

830 CONTINUE
27 CONTINUE

C==== Calculate new stress state
DO 850 I = 1, NUMEL
DO 850 J = 1, NSTRE
DO 850 NGP = 1, NGPTS
STR0(I,J,NGP) = STR0(I,J,NGP) + DSTR(I,J,NGP)
if(ISTAT(I).EQ.0) STR0(I,J,NGP) = 0.0
850 continue

C==== Calculate total displacements
DO 851 I = 1, NVAR
851 TDIS(I) = TDIS(I) + DDIS(I)
c==== Calculate the shotcrete moments & Stresses.
call bmom8

c==== Calculate the forces in teh bolts.
call blotd

c==== Write answers to plot file
do 673 i=1,numnod
   if(coord(i,ipicod) .eq. plval)then
      il= 6*i - 3
      i2= il - 1
      i3= il - 2
      write(lu4,'(i6,6e15.7)') i,coord(i,j),j=1,3,tdis(i3),
      tdis(i2),tdis(il)
   endif
   673 continue

C==== Write answers to output
WRITE (LU3,19)
19 FORMAT( A00,10H- )
   NODE Disp-x Disp-y Disp-z
   */3X,46(1H-)
   DO 880 I = 1, NOD
      K0 = NODNO(I)
      IF(K0 .EQ. 0) GO TO 880
      K3 = 6*K0 - 3
      K2 = K3 - 1
      K1 = K3 - 2
      WRITE(LU3,21) K0,TDIS(K1),TDIS(K2),TDIS(K3)
   880 CONTINUE

C *TDIS(J4),TDIS(J5),TDIS(J6),TDIS(J7),TDIS(J8)

C==== write stresses to plot file.
do 871 i = 1, isc
   k = stplt(i,1)
   ngp = stplt(i,2)
   SX = STR0(K,1,NGP)
   SY = STR0(K,2,NGP)
   SZ = STR0(K,3,NGP)
   TXY = STR0(K,4,NGP)
   TYZ = STR0(K,5,NGP)
   TZX = STR0(K,6,NGP)
   WRITE(LU4,'(2I6,9E14.4)') K,NGP,(stplt(i,j),j=3,5),SX,SY,SZ,
   + TXY, TYZ, TZX
   871 CONTINUE

WRITE(LU3,77)
77 FORMAT( A00,10H- )
   */3X,95(1H-)
   DO 881 I = 1, NOSTR
   K = ISTR5(I)
   DO 881 NGP = 1, NGPTS
      SX = STR0(K,1,NGP)
      SY = STR0(K,2,NGP)
      SZ = STR0(K,3,NGP)

Appendix B
TXY = STR0(K,4,NGP)
TYZ = STR0(K,5,NGP)
TZX = STR0(K,6,NGP)
WRITE(LU3,'(2I6,6E14.4)') K,NGP,SX,SY,SZ,TXY,TYZ,TZX
881 CONTINUE

IF(NEPL .NE. NEPL0) THEN
  WRITE (LU3,22)
22  FORMAT(1' ELEMENTS FAILED THIS LOAD STEP'/'
     *'----------------------- ----- -----'/
     *' Elem  G/p',8X,'Sx',12X,'Sy',11X,'Sz',
     *'12X,'Txy',13X,'Tyz',12X,'Tzx'/'
     *'  /3X,85(1H-)')
c
  WRITE (LU0,22)
  DO 890 K = 1, NUMEL
  DO 890 NGP = 1, NGPTS
  IF (IPEL(K,NGP) .EQ. NLI) THEN
    SX = STR0(K,1,NGP)
    SY = STR0(K,2,NGP)
    SZ = STR0(K,3,NGP)
    TXY = STR0(K,4,NGP)
    TYZ = STR0(K,5,NGP)
    TZX = STR0(K,6,NGP)
    WRITE(LU3,'(2I6,6E14.4)') K,NGP,SX,SY,SZ,TXY,TYZ,TZX
  ENDIF
 890 CONTINUE
ENDIF
NEPL0 = NEPL
C==== Create summary of output
ISUME(NLI) = NEPL
FL0AD(NLI) = TOTAL
nnn = 45
DEFLN(NLI) = TDIS(NNN)
CNVG(NLI) = CONCI
ITER(NLI) = ITT
900 CONTINUE
C ******************** END LOAD INCREMENT LOOP ********************
C==== Write summary of load-deflection behaviour
WRITE(LU0,302)
WRITE(LU3,302)
302 FORMAT(1' SUMMARY OF OUTPUT'/' -------- -- ------'/'
    *' Load  Multiple of Nominated Convergence',
    *' Number of No. Plastic'/' Step ',
    *'Applied Load  Deflection Criterion %  Iterations',
    *' Elements'/79(1H-))
WRITE (LU3,301) (I,FL0AD(I),DEFLN(I),CNVG(I),
    * ITER(I),ISUME(I),I = 1, MNLI)
WRITE (LU0,301) (I,FL0AD(I),DEFLN(I),CNVG(I),
    * ITER(I),ISUME(I),I = 1, MNLI)
301 FORMAT(I4,E15.4,E16.4,E14.4,I11,I13)
IF(IDIR.EQ.0) THEN
  WRITE(LU3,66) NDSUM
66  FORMAT(1' Summary for vertical displacement at node',I4)
ELSE
  WRITE(LU3,67) NDSUM
67  FORMAT(1' Summary for horizontal displacement at node',I4)
ENDIF

Appendix B
1034 continue

stop
end

C*******************************************************************************
C INPUT THE CONTROL DATA, GEOMETRIC DATA, BOUNDARY CONDITIONS
AND MATERIAL PROPERTIES
C*******************************************************************************

C SUBROUTINE INPUT

IMPLICIT real*8 (A-H,O-Z)
COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
+ istel,istcod,NGAUS,NPROP,NMATS,NVFIX,NEVAB,
+ ICASE,NCASE

COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3),
+ ELOAD(1700,96),NODES(1700,32),
+ MATNO(1700)

COMMON / WORK / ELCOD(3,32),SHAPE(32), DERIV(3,32),DMATX(6,6),
+ CARTD(3,32),DBMAT(6,96),BMA PX(6,96),
+ POSGP(3), WEIGP(3), GPCOD(3),
+ NEROR(24), stpl(6000,5)

COMMON / INNN / MNLI,MAXIT,NNN,NIOLD,TOTAL,FMULT,CONV,NDSUM,DIR
COMMON / SUB5 / NOD,NODNO(350),NOSTR,ISTRE(150),ISTRE,iplcod,
+ plval
COMMON / SUB9 / NU,NV,NW,IV(4500),IV(4500),IW(4500),
+ NUR,NVR,NWR,IVR(1700),IVR(1700),WIR(1700)

COMMON / STRES/ EPS(6),STRF(1700,6,27),STRO(1700,6,27),
+ STR(1700,6,27),IPEL(1700,27)
COMMON / SUB2 / ITTEMP(54000),NT,IPIT

common / shoptcr/ shprop(20,3),ishot(1700),ishpos(1700),
+ ishtmat(1700)
common / bolt / blmat(500), bltvar(500,4), nbolt(500,2),
+ numblt, nbtemp
common / excava/ iexle(150),istat(1700),nee,maxex

COMMON / SOLV1/ BB(54000), NVAR, rf(54000), nfb, bbl(54000)
COMMON / SOLV2/ NVARS, nen,in(8),sr(1700,8,4)

COMMON / UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

common / gamma / graden(50)

MAXNOD = 9000
MAXEL = 1700
MAXFIX = 2000
MAXBC = 4500
MAXOUT = 350
maxex = 150
PI = 3.14159265

C*** READ THE MASTER DATA CARD
READ (LU2,*), MNL1, MAXIT, CONV
READ (LU2,*), NUMNOD, NUMEL, NVFIX, NCASE, NNODE, NDOFN, NMATS,
+ NPROP, NGAUS, NDIME, NSTRE
NEVAB = NDOFN * NNODE
NVAR = NUMNOD*NDOFN

WRITE (LU3,905) NUMNOD, NUMEL, NVFIX, NCASE, NNODE, NDOFN, NMATS,
+ NPROP, NGAUS, NDIME, NSTRE, NEVAB
900 FORMAT (22I5)
905 FORMAT ( // ' Total no. of nodal points = ',I5 / 
+ ' Total no. of elements = ',I5 / 
+ ' No. of restrained nodes = ',I5 / 
+ ' No. of load cases = ',I5 / 
+ ' No. of nodes per element = ',I5 / 
+ ' Dgs of freedom per node = ',I5 / 
+ ' No. of different materials = ',I5 / 
+ ' No. of properties per matl = ',I5 / 
+ ' Order of Gaussian integr = ',I5 / 
+ ' No. of coord dimensions = ',I5 / 
+ ' No. of stress resultants = ',I5 / 
+ ' No. of ind vars per elem = ',I5 / )

IF ( NUMNOD .GT. MAXNOD ) WRITE (LU0,1001) MAXNOD
1001 FORMAT ( // ' TOO MANY NODES : MAX = ',I5 )
IF ( NUMEL .GT. MAXEL ) WRITE (LU0,1002) MAXEL
1002 FORMAT ( // ' TOO MANY ELEMENTS : MAX = ',I5 )
IF ( NVFIX .GT. MAXFIX ) WRITE (LU0,1003) MAXFIX
1003 FORMAT ( // ' TOO MANY RESTRAINTS : MAX = ',I5 )

C==== ZERO ALL NODAL COORDINATES BEFORE READING SOME OF THEM

DO 20 IPOIN = 1, NUMNOD
   DO 20 IDIME = 1, NDIME
   20  COORD(IPOIN,IDIME) = 0.0D0

C==== Initialise istat(1700), 0=excavated, 1=remaining
C==== and ishot(1700), 0=no shotcrete, 1=shotcrete
C==== and ipel(ll,ngp) - indicates whether element is plastic.
C==== sr(ll,8,4) - holds the starting stress resultants for
C==== shotcrete.
   DO 1000 i = 1, NUMEL
       istat(i) = 1
       ishot(i) = 0
   DO 1037 j = 1, 8
       ipel(i,j) = 0
   DO 1048 k = 1, 4
       sr(i,j,k) = 0.0D0
   1048 continue
   1037 continue
   1000 continue

C==== Initialise bolt numbers - numbbl = 0
C==== numbbl = 0

C==== READ SELECTED NODAL COORDINATES, FINISHING WITH THE LAST
C==== NUMBERED NODE

WRITE (LU3,920)
WRITE (LU3,925)
920 FORMAT ( // ' Nodal point coordinates ' 
+ ' /' ----- ----- ------------ ' )
30 READ (LU2,*) IPOIN,
  + (COORD(IPOIN,IDIME), IDIME = 1, NDIME)
390 FORMAT (I5,3F11.4)
  IF (IPOIN .NE. NUMNOD ) GO TO 30

DO 50 IPOIN = 1, NUMNOD
10 READ (LU3,935) IPOIN, (COORD(IPOIN,IDIME), IDIME = 1, NDIME)
50 WRITE (LU3,935) IPOIN, (COORD(IPOIN,IDIME), IDIME = 1, NDIME)
935 FORMAT (I5,2X,3F10.4)

C*** READ THE ELEMENT NODAL CONNECTIONS THE PROPERTY NUMBERS

WRITE (LU3,910)
910 FORMAT (1H0, '/',' Element', 38X, 'Node numbers', 51X, 'Material'&
  /118(1H-))

DO 100 IElem = 1, NUMEL
10 READ (LU2,*) NUMEL,
  + (NODES(NUMEL, INODE), INODE = 1, NNODE), MATNO(NUMEL)
  WRITE (LU3,915) NUMEL,
  + (NODES(NUMEL, INODE), INODE = 1, NNODE), MATNO(NUMEL)
101 CONTINUE

C**** Read where output is required

C**** NODNO(I) are the nodes at which displacements are output
READ (LU2,*) NOD
  IF (NOD.GT.MAXOUT) THEN
WRITE (LU0,56) MAXOUT
56 FORMAT (1H0, '/ Output required at too many points'/&
  /11H Maximum no. set at', 11I4)
STOP
ENDIF
READ (LU2,*) (NODNO(I), I=1, NOD)

C**** ISTRE(I) are the elements at which stresses are output
READ (LU2,*) NOSTR
  IF (NOSTR.GT.MAXOUT) THEN
WRITE (LU0,56) MAXOUT
STOP
ENDIF
READ (LU2,*) (ISTRE(I), I=1, NOSTR)

C---- Read the plane code and value for the plot file.
C-- 1= const.X, 2= const.Y, 3= const.Z
read(lu2,*) iplcod,plval

C--- Read the stress code and first element for plotting.
C== 1= const.X, 2= const.Y, 3= const.Z
read(lu2,*) istcod,istel

C*** READ THE FIXED VALUES

READ (LU2,*) NU
  IF (NU.GT.0) READ (LU2,*) (IU(I), I = 1, NU)
READ (LU2,*) NV
  IF (NV.GT.0) READ (LU2,*) (IV(I), I = 1, NV)

Appendix B
READ(LU2,*) NW
IF(NW.GT.0) READ(LU2,*) (IW(I), I = 1, NW)
READ(LU2,*) NUR
IF(NUR.GT.0) READ(LU2,*) (IUR(I), I = 1, NUR)
READ(LU2,*) NVR
IF(NVR.GT.0) READ(LU2,*) (IVR(I), I = 1, NVR)
READ(LU2,*) NWR
IF(NWR.GT.0) READ(LU2,*) (IWR(I), I = 1, NWR)

WRITE(LU3,54)
54 FORMAT(//' BOUNDARY CONDITIONS'// '---------' )
5 FORMAT(1015)
NT - NU + NV + NW + NUR + NVR + NWR

C==== X-direction
IF(NU.GT.MAXBC) GO TO 58
IF(NU.EQ.0) GO TO 8
WRITE(LU3,6)
6 FORMAT(//' X-Displacement Zero at Nodes...')
WRITE(LU3,5) (IU(I),I=1,NU)
DO 7 I = 1, NU
   ITEMP(I) = 6*IU(I) - 5
C      nbc = itemp(i)
C      nf(nbc) = -1
7 CONTINUE
8 CONTINUE

C==== Y-direction
IF(NV.EQ.0) GO TO 11
IF(NV.GT.MAXBC) GO TO 58
WRITE(LU3,9)
9 FORMAT(//' Y-Displacement Zero at Nodes...')
WRITE(LU3,5) (IV(I), I = 1, NV)
DO 10 I = 1, NV
   KI - I + NU
   ITEMP(KI) = 6*IV(I) - 4
C      nbc = itemp(ki)
C      nf(nbc) = -1
10 CONTINUE
11 CONTINUE

C==== Z-direction
IF(NW.EQ.0) GO TO 12
IF(NW.GT.MAXBC) GO TO 58
WRITE(LU3,14)
14 FORMAT(//' Z-Displacement Zero at Nodes...')
WRITE(LU3,5) (IW(I),I=1,NW)
DO 15 I = 1, NW
   KI = I + NU + NV
   ITEMP(KI) = 6*IW(I) - 3
C      nbc = itemp(ki)
C      nf(nbc) = -1
15 CONTINUE
12 CONTINUE

C==== X-rotation
IF(NUR.EQ.0) GO TO 112
IF(NUR.GT.MAXBC) GO TO 58
WRITE(LU3,114)
114 FORMAT(//' X-Rotation Zero at Nodes...')
WRITE(LU3,5) (IUR(I),I=1,NUR)
DO 115 I = 1, NUR
  KI = I + NU + NV + NW
  ITEMP(KI) = 6*IUR(I) - 2
115 continue
112 CONTINUE

C==== Y-Rotation
IF(NVR.GT.MAXBC) GO TO 58
IF(NVR.EQ.0) GO TO 18
WRITE(LU3,16)
16 FORMAT(//' Y-Rotation Zero at Nodes...'
WRITE(LU3,5) (IVR(I),I=1,NVR)
DO 17 I = 1, NVR
  KI = I + NU + NV + NW + NUR
  ITEMP(KI) = 6*IVR(I) - 1
17 CONTINUE
18 CONTINUE

C==== Z-Rotation
IF(NWR.EQ.0) GO TO 111
IF(NWR.GT.MAXBC) GO TO 58
WRITE(LU3,19)
19 FORMAT(//' Z-Rotation Zero at Nodes...'
WRITE(LU3,5) (IWR(I), I = 1, NWR)
DO 110 I = 1, NWR
  KI = I + NU + NV + NW + NUR + NVR
  ITEMP(KI) = 6*IWR(I)
110 CONTINUE
111 CONTINUE

C==== Check on exceeding boundary conditions allowed
GO TO 60
58 CONTINUE
WRITE(LU0,59) MAXBC
59 FORMAT(//' Maximum no. of boundary conditions exceeded'/
     ' +' Maximum no. allowed =',14)
STOP
60 CONTINUE

C*** READ THE AVAILABLE SELECTION OF ELEMENT PROPERTIES

WRITE (LU3,960)
960 FORMAT (// ' Material Properties '
     ' /' ------------------ '
WRITE (LU3,965)
965 FORMAT
     '(' No. Young''s Poisson''s Phi Psi Cu Gm '
     ' /' Modulus Ratio (radians)'/58(1-H-))

DO 100 IMATS = 1, NMATS
  READ(LU2,*) NUMAT, (PROPS(NUMAT,IPROP),IPROP=1,NPROP)
C 970 FORMAT(T5,7F8.3)
  PROPS(NUMAT,3) = PROPS(NUMAT,3)*PI/180
  PROPS(NUMAT,4) = PROPS(NUMAT,4)*PI/180
  WRITE(LU3,971) NUMAT, (PROPS(NUMAT,IPROP),IPROP=1,NPROP)
  GRADEN(NUMAT) = PROPS(NUMAT,6)
100 CONTINUE

C*** SET UP GAUSSIAN INTEGRATION CONSTANTS

Appendix B
CALL GAUSSQ
RETURN
END

******************************************************************************
GAUSS QUADRATURE ROUTINES
******************************************************************************

SUBROUTINE GAUSSQ

IMPLICIT real*8 (A-H,O-Z)

COMMON / CONTO / NUMNOD, NUMEL, NNODE, NDOFN, NDIME, NSTRE, isc,
+ istel, istcod, NGAUS, NPROP, NMATS, NVFIX, NEVAB,
+ ICASE, NCASE
COMMON / LGDATA / COORD(5000,3), PROPS(50,10), PRESC(200,3),
+ ELOAD(1700,96), NODES(1700,32),
+ MATNO(1700)
COMMON / WORK / ELCOD(3,32), SHAPE(32), DERIV(3,32), DBMATX(6,6),
+ CARTH(3,32), DBMAT(6,96), BMATX(6,96),
+ POSGP(3), WEIGP(3), GPCOD(3),
+ NEROR(24), stplt(6000,5)
COMMON / UNITS/ LU0, LUI, LUS, LU3, LU4, lu6

c WRITE (LU3,1000)
c 1000 FORMAT ( ' ENTERING S/R GAUSS' )

NGP = NGAUS + 1
NGS = (NGAUS+1)/2 + 1

GO TO (10,20,30,40,50), NGAUS

10 POSGP(1) = 0.000000000000000D0
WEIGP(1) = 2.000000000000000D0
GO TO 90

20 POSGP(1) = -0.577350269189626D0
WEIGP(1) = 1.000D0
GO TO 80

30 POSGP(1) = -0.774596669241483D0
WEIGP(1) = 0.555555555555555D0
POSQP(2) = 0.000000000000000D0
WEIGP(2) = 0.88888888888889D0
GO TO 80

40 POSGP(1) = -0.861136311594053D0
WEIGP(1) = 0.347854845137454D0
POSQP(2) = -0.339981043584856D0
WEIGP(2) = 0.652145154962546D0
GO TO 80

50 POSGP(1) = -0.906179845938664D0
WEIGP(1) = 0.23606885056189D0
POSQP(2) = -0.538469310105683D0
WEIGP(2) = 0.478628670499366D0
POSQP(3) = 0.000000000000000D0
WEIGP(3) = 0.56888888888889D0

Appendix B
80  DO 85 IG = NGS,NGAUS
    POSGP(IG) = -POSGP(NGP-IG)
    WEIGP(IG) = WEIGP(NGP-IG)
85  CONTINUE

90  CONTINUE

RETURN
END

C ****************************************************************************************************
C CALCULATES THE ELEMENT STIFFNESS AND STRESS MATRICES
C ****************************************************************************************************

SUBROUTINE STIF3D

IMPLICIT real*8 (A-H,O-Z)

COMMON / TEMP4 / ESTIF(96,96),LOC(96),NDEST(96)

COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDOFN,NDIMES,NSTRE,isc,
                   istel,istc,GDAUS, PROP,NMAT,NMATK,NEF,NEVAB,
                   ICASE, NCASE

COMMON / LGDATA / COORD(9000,3), PROPS(50,10), PESC(200,3),
                   ELOAD(1700,96), NODES(1700,32),
                   MATNO(1700)

COMMON / WORK / ELCOD(3,32), SHAPE(32), DERIV(3,32), DMATX(6,6),
                 CARTD(3,32), BMATX(6,96), BMATX(6,96),
                 POSGP(3), WEIGP(3), GCPG(3),
                 NERO(24), STPLT(6000,5)

COMMON / PROP / IP(96)
COMMON / SOLV / AM(240000000), MAXBW

COMMON / SOLVL/ BB(54000), NVAR, nfi(54000), nfi, nfi, bbl(54000)
common /excav/ ielex(150), istat(1700), nee, maxex
common / diag/ kdiag(54000), ldiag(54000)

COMMON / UNITS/ LU0, LU1, LU2, LU3, LU4, LU6

WRITE (LU0,1000)
1000 FORMAT (' ENTERING S/R STIF3D ')

C*** ZERO B MATRIX AND D MATRIX ONCE ONLY

DO 8 ISTRE = 1, NSTRE
    DO 5 IEVAB = 1, 60
        BMATX(ISTRE,IEVAB) = 0.000
5  CONTINUE

DO 8 JSTRE = 1, NSTRE
    DMATX(ISTRE,JSTRE) = 0.000
8  CONTINUE

c== Calculate the nodal freedoms - for this load step
   Call free

c== Initialise row lengths as 0.
do 13 i=1,nfvar
   kdiag(i) = 0
13 continue

C==== Calculate kdiag().
call fkdiag
   kh = kdiag(nfvar)
   write(97,'(a,el3.5)') ' kdiag(nfvar) ',kh

C==== Clear stiffness matrix
do 1009 i=1,kh
1009 am(i) = 0.0
isc = 0
ittest = 0
C***LOOP OVER EACH ELEMENT

DO 70 LL = 1, NUMEL
   if(istat(11) .eq. 0) goto 70
   LPROP = MATNO(LL)
   WRITE (LU0,1001) LL
1001 FORMAT ( ' FORMING STIFFNESS MATRIX FOR ELEMENT NO ',I3 )

C*** EVALUATE THE COORDINATES OF THE ELEMENT NODAL POINTS

DO 10 INODE = 1, NNODE
   LNODE = NODES(LL,INODE)
   DO 10 IDIME = 1, NDIME
10   ELCOD(IDIME,INODE) = COORD(LNODE,IDIME)

C==== Set up position matrix.
DO 55 I = 1, NNODE
   K = NODES(LL,I)
   nvn1 = 6 - 1
   jj = 0
   do 55 j = 3, nvn1
      l = 3*i - jj
      m = 6*k - j
      jj = jj + 1
55   ip(l) = m

C==== Evaluate the D matrix
CALL DMAT(LPROP)

C*** INITIALISE THE ELEMENT STIFFNESS MATRIX

DO 20 IEVAB = 1,60
DO 20 JEVAB = 1,60
20   ESTIF(IEVAB,JEVAB) = 0.0
LG = 0
C*** ENTER LOOPS FOR AREA NUMERICAL INTEGRATION

DO 50 IG = 1, NGAUS
   DO 50 JK = 1, NGAUS
   DO 50 KG = 1, NGAUS
   LG = LG +1
50   EXISP = POSGP(IG)
   ETASP = POSGP(JG)

Appendix B
EZESP = POSGP(KG)

C       write(real(13.3e13.4)' ) lg,exisp,etasp,ezesp

C*** COMPUTE THE SHAPE FUNCTIONS AT THE SAMPLING POINTS
C       AND THE ELEMENT VOLUME
C
CALL SF3R2(Exisp,Etasp,EZESP)
CALL JACOB3(LL,DJACB,LG)
DVOLU = DJACB * WEIGP(IG) * WEIGP(JG) * WEIGP(KG)

C== Set up the stress output gauss points.
  if(ll.eq.istel.and.test.eq.0)then
      conscod = gpcod(istcod)
      test = 1
    endif

difst = gpcod(istcod) - conscod
if(difst .le. 1.0e-8 .and. difst .ge. -1.0e-8)then
    isc=isc+1
    stpl(st,1)=ll
    stpl(st,2)=lg
    stpl(st,3)=gpcod(1)
    stpl(st,4)=gpcod(2)
    stpl(st,5)=gpcod(3)
  endif

C*** EVALUATE THE B AND DB MATRICES FOR THE ELEMENT
C
CALL BMAT3D
CALL DBE

C*** CALCULATE THE ELEMENT STIFFNESS MATRIX
C
DO 30 IEVAB = 1, 60
DO 30 JEVAB = IEVAB, 60
DO 30 ISTRE = 1, NSTRE
30 ESTIF(IEVAB,IEVAB) = ESTIF(IEVAB,JEVAB) +
    BMATX(ISTRE,IEVAB) * DBMAT(ISTRE,JEVAB) * DVOLU
50 CONTINUE

C*** CONSTRUCT THE LOWER TRIANGLE OF THE STIFFNESS MATRIX
C
DO 60 IEVAB = 1, 60
DO 60 JEVAB = IEVAB, 60
60 ESTIF(IEVAB,IEVAB) = ESTIF(IEVAB,JEVAB)

DO 90 I = 1, 60
II = IP(I)
i = nf(ii)
if(i.eq.-1) goto 90
DO 80 J = 1, 60
JJ = IP(J)
ja = nf(jj)
if(ja.eq.-1) goto 80
KK = ia - ja
IF (KK.LT.0) GO TO 80
IPS = kdiag(ia) - kk
AM(IPS) = AM(IPS) + ESTIF(I,J)
80 CONTINUE

Appendix B
CONTINUE
+ 11,(am(i), i=1,5000)
2100 FORMAT ('STIFFNESS MATRIX for ',i3 // (6D12.4) )

70 CONTINUE
RETURN
END

***************************************************************
* Routine to find the maximum bandwidth for each 
* freedom.                                            *
***************************************************************

subroutine fkdiag

implicit real*8 (a-h,o-z)

COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIMN,NSTRE,isc,
+ istel,istcod,NGAUS,NPROP,NMAFS,NVFIX,NEVAB,
+ ICASE,NCASE
COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3),
+ ELOAD(1700,96),NODES(1700,32),
+ MATNO(1700)
common /excav/ ielex(150),istat(1700),nee,maxex
common / prop / ip(96)
common / shotcr/ shprop(20,3),ishot1(1700),ishpos1(1700),
+ ishmat(1700)
common / diag/ kdiag(54000),ldiag(54000)
COMMON / SOLV1/ BB(54000), NVAR, nf(54000),nfvar,bbl(54000)
COMMON / UNITS/ LU0, LU1, LU2, LU3, LU4, lu6
common / bolt / bitmat(500), bltvar(500,4), nbolt(500,2),
+ numblt, nbtemp

DIMENSION is(48), in(8), ib(12)

***LOOP OVER EACH ELEMENT

DO 70 LL = 1, NUMEL
   if(istat(ll) .eq. 0) goto 70

*** For solid elements.

C== Set up position matrix.

DO 55 I = 1, NNODE
   K = NODES(LL,I)
   nvn1 = 6 - 1
   jj = 0
   do 55 jj = 3, nvn1
   do 55 jj = 3, nvn1
   l = 3*i - jj
   m = 6*k - jj
   jj = jj + 1
55 ip(l) = m

do 20 i=1,60
iwpl = 0

Appendix B
\begin{verbatim}
na = ip(i)
if(nf(na).eq.-1) goto 20
do 30 j=1,60
   nb = ip(j)
   if(nf(nb).eq.-1) goto 30
   im = nf(na) - nf(nb) + 1
   if(im.gt.iwpl) iwpl = im
30 continue
   k = ip(i)
   kl = nf(k)
   if(iwpl.gt.kdiag(kl).and.kl.ne.-1) kdiag(kl)=iwpl
20 continue

c== For shotcrete elements.
   if(ishot(ll).eq.0) goto 70

c== Read the side of the element to be shotcreted

c== 0= on top, 1=right side
   if(ishpos(ll).eq.0) then
      in(1) = NODES(ll,13)
in(2) = NODES(ll,14)
in(3) = NODES(ll,15)
in(4) = NODES(ll,16)
in(5) = NODES(ll,17)
in(6) = NODES(ll,18)
in(7) = NODES(ll,19)
in(8) = NODES(ll,20)
   else if(ishpos(ll).eq.1) then
      in(1) = NODES(ll,3)
in(2) = NODES(ll,4)
in(3) = NODES(ll,5)
in(4) = NODES(ll,11)
in(5) = NODES(ll,17)
in(6) = NODES(ll,16)
in(7) = NODES(ll,15)
in(8) = NODES(ll,10)
   else if(ishpos(ll).eq.2) then
      in(1) = NODES(ll,1)
in(2) = NODES(ll,2)
in(3) = NODES(ll,3)
in(4) = NODES(ll,10)
in(5) = NODES(ll,15)
in(6) = NODES(ll,14)
in(7) = NODES(ll,13)
in(8) = NODES(ll,9)
   else
      write(lu0,'(/a)') ' Error assigning shotcrete position ' stop
endif

c== Set shotcrete position matrix.
i=0
   do 110 i = 1,8
      do 110 j = 1,6
         ii=ii+1
      110 is(ii) = (6*in(i)) - (6-j)
\end{verbatim}

Appendix B
do 90 i=1,48
   iwpl = 0
   na = is(i)
   if(nf(na).eq.-1) goto 90
   do 100 j=1,48
      nb = is(j)
      if(nf(nb).eq.-1) goto 100
      im = nf(na) - nf(nb) + 1
      if(im.gt.iwpl) iwpl = im
   100 continue
   k = is(i)
   kl = nf(k)
   if(iwpl.gt.kdiag(kl).and.kl.ne.-1) kdiag(kl)=iwpl
90  continue
70  continue

C==== Consider the rock bolt entries.
   do 120 ll-1,numblt

C===== Set up the bolt position matrix.
   ii=0
   do 130 i=1,2
      do 130 j=1,6
         ii=ii+1
      130 ib(ii) = (6*nbolt(ll,i)) - (6-j)
   do 140 i=1,12
      iwpl = 0
      na = ib(i)
      if(nf(na).eq.-1) goto 140
      do 150 j=1,12
         nb = ib(j)
         if(nf(nb).eq.-1) goto 150
         im = nf(na) - nf(nb) + 1
         if(im.gt.iwpl) iwpl = im
      150 continue
      k = ib(i)
      kl = nf(k)
      if(iwpl.gt.kdiag(kl).and.kl.ne.-1) kdiag(kl)=iwpl
140  continue
120  continue

C=== Write row lengths to ldiag()
   do 80 i = 1,nfvar
      80 ldiag(i) = kdiag(i)
      kdiag(1) = 1
      do 40 i=2,nfvar
         40 kdiag(i) = kdiag(i) + kdiag(i-1)

c   write(lu3,'((a))' ' kdiag()' 
c   write(lu3,'(10i8)') (kdiag(i),i=1,nfvar)
   return
end

Appendix B
**ROUTINE FOR ELASTIC D MATRIX**

```
SUBROUTINE DMAT(LPROP)

IMPLICIT real*8 (A-H,O-Z)

COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
+ istel,istcod,NGAUS,NPROP,NMATS,NVFIX,NEVAB,
+ ICASE,NCASE

COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3),
+ ELOAD(1700,96),NODES(1700,32),
+ MATNO(1700)

COMMON / WORK / ELCOD(3,32),SHAPE(32), DERIV(3,32),DMATX(6,6),
+ CARTD(3,32), DBMAT(6,96), BMATX(6,96),
+ POSGP(3), WEIGP(3), GECOD(3),
+ NEROR(24), stpltt(6000,5)

COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

YOUNG = PROPS(LPROP,1)
POISS = PROPS(LPROP,2)
TEMP1 = YOUNG / ( (1.0D0 + POISS) * (1.0D0 - 2.0D0*POISS) )
TEMP2 = (1.0D0 - POISS) * TEMP1
TEMP3 = POISS * TEMP1
TEMP4 = 0.5D0 * YOUNG / (1.0D0 + POISS)

DO 88 I = 1, NSTRE
   DO 88 J = 1, NSTRE
     DMATX(I, J) = 0.0
   88 CONTINUE

   DO 20 I = 1, 3
     DMATX(I,1) = TEMP2
     DMATX(I+3, I+3) = TEMP4
   20 CONTINUE

   DO 30 I = 1, 3
     II = 1 + I/3
     JJ = 2 + I/2
     DMATX(II,JJ) = TEMP3
     DMATX(JJ,II) = TEMP3
   30 CONTINUE

RETURN
END
```

**CALCULATES SHAPE FUNCTIONS AND THEIR DERIVATIVES FOR THREE DIMENSIONAL QUADRATIC ELEMENTS**

```
SUBROUTINE SF3R2(S,T,U)

IMPLICIT real*8 (A-H,O-Z)

DIMENSION CN(20,3), PROD(3), X(3)
```
COMMON / CONTRO / NUMNOD, NUMEL, NNODE, NDORF, NDIME, NSTRE, isc,
+ istel, istcod, NGAUS, NPROP, NMATS, NVFIX, NEVAB,
+ ICASE, NCASE

COMMON / LGDATA / COORD(9000,3), PROPS(50,10), FRESID(200,3),
+ ELOAD(1700,96), NODES(1700,32),
+ MATNO(1700)

COMMON / WORK / ELCOD(3,32), SHAPE(32), DERIV(3,32), DMATX(6,6),
+ CARTD(3,32), DBMAT(6,96), BMATX(6,96),
+ POSGP(3), WEGP(3), GMCOD(3),
+ NEROR(24), stptl(6000,5)

COMMON / UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

C** FIRST VALUES OF S AT NODES

DATA CS / -1., 0., +1., +1., +1., 0., -1., -1.,
  + -1., +1., +1., -1.,
  + -1., 0., +1., +1., 0., -1., -1.,

C** NOW T VALUES AT NODES

  + -1., -1., -1., 0., +1., +1., 0.,
  + -1., +1., -1., -1.,
  + -1., -1., 0., +1., +1., 0.,

C** NOW U VALUES AT NODES

  + -1., -1., -1., -1., -1., -1.,
  + 0., 0., 0., 0.,
  + +1., +1., +1., +1., +1., +1., +1. /

X(1) = S
X(2) = T
X(3) = U
IN = 0

20 IN = IN + 1

C** CORNER NODE FIRST

SS = S * CN(IN,1)
TT = T * CN(IN,2)
UU = U * CN(IN,3)
SSP = SS + 1.0DO
TTP = TT + 1.0DO
UUP = UU + 1.0DO

SHAPE(IN) = 0.125DO * SSP * TTP * UUP *
+ (SS + TT + UU - 2.0DO)
DERIV(1,IN) = 0.125DO * CN(IN,1) * TTP * UUP *
+ (2.0DO*SS + TT + UU - 1.0DO)
DERIV(2,IN) = 0.125DO * CN(IN,2) * SSP * UUP *
+ (SS + 2.0DO*TT + UU - 1.0DO)
DERIV(3,IN) = 0.125DO * CN(IN,3) * SSP * TTP *
+ (SS + TT + 2.0DO*UU - 1.0DO)

C** NOW MIDSIDE NODE

30 IN = IN + 1

Appendix B
PROD(1) = DABS( CN(IN,2) * CN(IN,3) )
PROD(2) = DABS( CN(IN,3) * CN(IN,1) )
PROD(3) = DABS( CN(IN,1) * CN(IN,2) )
SSP = 1.0D0 + S * CN(IN,1)
TTP = 1.0D0 + T * CN(IN,2)
UUP = 1.0D0 + U * CN(IN,3)

TEMP1 = 0.25D0 * SSP * TTP * UUP
TEMP2 = 1.0D0 - S*S*PROD(1) - T*T*PROD(2) - U*U*PROD(3)
SHAPE(IN) = TEMP1 * TEMP2
DO 40 ID = 1,3
   DERIV(ID,IN) = TEMP1 * ( TEMP2 * CN(IN,ID) / 
                        ( 1.0D0 + CN(IN,ID)*X(ID) ) 
                        - 2.0D0 * PROD(ID) * X(ID) )
40 CONTINUE

IF ( IN .EQ. NNODE ) RETURN
IF ( (2*IN - 19) / 4 ) 20,30,20
END

C *** SUBROUTINE JACOB3(IELEM,DJACB,LG) ***
IMPLICIT REAL*8 (A-H,O-Z)
DIMENSION XJ(3,3),XI(3,3)

COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc, + istel,istcod,NGAUS,NPROP,NMATS,NVFIX,NEVAB, + ICASE,NCASE

COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3), + ELOAD(1700,96),NODES(1700,32), + MATNO(1700)

COMMON / WORK / ELCOD(3,32),SHAPE(32), DERIV(3,32), BMATX(6,6), + CARID(3,32),BMAT(6,96),BMATX(6,96), + POSGP(3), WEIGP(3), GPCOD(3), + NEROR(24), split(6000,5)

COMMON /UNITS/ LU0, LUI, LU2, LU3, LU4, lu6

C*** CALCULATE COORDINATES OF SAMPLING POINT

DO 10 IDIME = 1,NDIME
   GPCOD(IDIME) = 0.0D0
DO 10 INODE = 1,NNODE
   GPCOD(IDIME) = GPCOD(IDIME) + + ELCOD(IDIME,INODE) * SHAPE(INODE)
10 CONTINUE

C*** SET UP JACOBIAN MATRIX

DO 20 IDIME = 1,NDIME
DO 20 JDIME = 1,NDIME

Appendix B
XJ(IDIME,JDIME) = 0.0D0
DO 20 INODE = 1,NNODE
C write(1u3,1005) deriv(idime,inode),elcod(jdime,inode)
c 1005 format (/ ' deriv ',e10.4 ,'/ elcod ',e10.4)
XJ(IDIME,JDIME) = XJ(IDIME,JDIME) +
+ DERIV(IDIME,INODE) * ELCOD(IDIME,INODE)
20 CONTINUE
C CALCULATE DETERMINANT AND INVERSE OF JACOBIAN
DJACB = XJ(1,1) * ( XJ(2,2)*XJ(3,3) - XJ(2,3)*XJ(3,2) )
+ - XJ(1,2) * ( XJ(2,1)*XJ(3,3) - XJ(2,3)*XJ(3,1) )
+ + XJ(1,3) * ( XJ(2,1)*XJ(3,2) - XJ(2,2)*XJ(3,1) )
IF( DJACB .GT. 0.0D0 ) GO TO 30
WRITE (LU3,900) IELEM,djacb
WRITE (LU0,900) IELEM,djacb
900 FORMAT ( // 2X,'PROGRAM HALT IN JACOB3' /
+ 2X,'ZERO OR NEGATIVE AREA IN ELEMENT NUMBER',I5 /
+ 2X,'D Jacob value of;',e15.4)
STOP
30 XI(1,1) = ( XJ(2,2)*XJ(3,3) - XJ(2,3)*XJ(3,2) ) / DJACB
XI(1,2) = ( XJ(1,3)*XJ(3,2) - XJ(1,2)*XJ(3,3) ) / DJACB
XI(1,3) = ( XJ(1,2)*XJ(2,3) - XJ(1,3)*XJ(2,2) ) / DJACB
XI(2,1) = ( XJ(2,3)*XJ(3,1) - XJ(2,1)*XJ(3,3) ) / DJACB
XI(2,2) = ( XJ(1,1)*XJ(3,3) - XJ(1,3)*XJ(3,1) ) / DJACB
XI(2,3) = ( XJ(1,2)*XJ(1,3) - XJ(1,1)*XJ(2,3) ) / DJACB
XI(3,1) = ( XJ(2,1)*XJ(3,1) - XJ(2,3)*XJ(3,2) ) / DJACB
XI(3,2) = ( XJ(1,2)*XJ(3,1) - XJ(1,1)*XJ(3,2) ) / DJACB
XI(3,3) = ( XJ(1,1)*XJ(2,2) - XJ(1,2)*XJ(2,1) ) / DJACB
C*** CALCULATE CARTESIAN DERIVATIVES
DO 40 IDIME = 1,NDIME
DO 40 INODE = 1,NNODE
CARTD(IDIME,INODE) = 0.0D0
DO 40 JDIME = 1,NDIME
CARTD(IDIME,INODE) = CARTD(IDIME,INODE) +
+ XI(IDIME,JDIME) * DERIV(JDIME,INODE)
40 CONTINUE
RETURN
END
C ***************************************************************
C THE SUBROUTINE CALCULATES THE STRAIN MATRIX B
C ***************************************************************
SUBROUTINE BMAT3D
IMPLICIT real*8 (A-H,O-Z)
COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
+ istel,istcod,NGAUS,NPROP,NMATS,NVFIX,NEVAB,
+ ICASE,NCASE
COMMON / LGDATA / COORD(9900,3),PROPS(50,10),PRES(200,3),
+ ELOAD(1700,96),NODES(1700,32),

Appendix B
MATNO(1700)

COMMON / WORK / ELCOD(3,32), SHAPE(32), DERIV(3,32), DMATX(6,6),
+ CARTD(3,32), DBMAT(6,96), BMATX(6,96),
+ POSGP(3), WEIGP(3), GPCOD(3),
+ NEROR(24), stpl(6000,5)

COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

III = 0
DO 10 IN = 1, NNODE
  I = III + 1
  II = I + 1
  III = II + 1
  BMATX(1,I) = CARTD(1,IN)
  BMATX(2,II) = CARTD(2,IN)
  BMATX(3,III) = CARTD(3,IN)
  BMATX(4,I) = CARTD(2,IN)
  BMATX(4,II) = CARTD(1,IN)
  BMATX(5,II) = CARTD(3,IN)
  BMATX(5,III) = CARTD(2,IN)
  BMATX(6,I) = CARTD(3,IN)
  BMATX(6,III) = CARTD(1,IN)
10 CONTINUE

RETURN
END

C **********************************************
C CALCULATES D X B
C **********************************************

SUBROUTINE DBE

IMPLICIT real*8 (A-H,O-Z)

COMMON / CONTRO / NUMNOD, NUME1, NNODE, NDOFN, NDIME, NSTRE, isc,
+ istel, istcod, NGAUS, NPROP, NMATS, NVFIX, NEVAB,
+ ICASE, NCASE

COMMON / LGDATA / COORD(9000,3), PROPS(50,10), PRESC(200,3),
+ ELOAD(1700,96), NODES(1700,32),
+ MATNO(1700)

COMMON / WORK / ELCOD(3,32), SHAPE(32), DERIV(3,32), DMATX(6,6),
+ CARTD(3,32), DBMAT(6,96), BMATX(6,96),
+ POSGP(3), WEIGP(3), GPCOD(3),
+ NEROR(24), stpl(6000,5)

COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

DO 10 ISTRE = 1, NSTRE
DO 10 IEVAB = 1, 60
  DBMAT(ISTRE,IEVAB) = 0.0
DO 10 JSTRE = 1, NSTRE
  DBMAT(ISTRE,IEVAB) = DBMAT(ISTRE,IEVAB) +
    DMATX(ISTRE,JSTRE) * BMATX(JSTRE,IEVAB)
10 CONTINUE

RETURN
END

Appendix B
C *************************************************************
C EQUIVALENT NODAL LOADS FOR THREE DIMENSIONAL ELEMENTS NODAL
C POINT LOADS, DISTRIBUTED FACE STRESSES, AND GRAVITATIONAL
C BODY FORCES INCLUDED
C *************************************************************

SUBROUTINE LOAD3D

IMPLICIT real*8 (A-H,O-Z)

DIMENSION POINT(6),corner(4),center(4)

INTEGER TITLE(36)

COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
 + istel,istcod,NGAUS,NPROP,NMAT,SNV,F,NEVAB,
 + ICASE,NCASE

COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3),
 + ELOAD(1700,96),NODES(1700,32),
 + MATNO(1700)

COMMON / WORK / ELCOD(3,32),SHAPE(32), DERIV(3,32),DMATX(6,6),
 + CARTD(3,32),DMATX(6,96),
 + POSGP(3), WEIGP(3), GPCOD(3),
 + NEROR(24), stplt(6000,5)

common /gamma / graden(50)
common /SOLV / AM(24000000), MAXBW
common /SOLV2 / BB(54000), NVAR, nf(54000), nivar,bbi(54000)
common /FLAS / BI(54000), DDII(54000), RESID(54000)
common /PROP / IP(96)
common /UNITS/ LUO, LUI, LU2, LU3, LU4, lu6

DO 10 I = 1, NVAR
   bi(i) = 0.0d0
10  BB(I) = 0.0d0

C*** READ EACH LOAD TYPE SEPARATELY, EACH BEGINNING WITH
C AN INTEGER TO DEFINE THE NUMBER OF LOADS OF THAT TYPE

READ (LU2,*) NPLOD
910 FORMAT (415 )
WRITE (LU3,911) NPLOD
911 FORMAT ( /' No. of Point Loads =', I4)

IF (NPLOD. EQ. 0) GO TO 500

WRITE (LU3, 912)
912 FORMAT ( /' Node X-Load Y-Load Z-Load'
 + ' 0x 0y 0z'
 + '/' )
C*** READ NODAL POINT LOADS

DO 51 IPLOD = 1, NPLOD
   READ(LU2,*) LODPT, (POINT(I DOFN), I DOFN=1,NDOFN)
915 FORMAT (I5,3F8.3)
   WRITE (LU3,916) LODPT, (POINT(I DOFN), I DOFN=1,NDOFN)
916 FORMAT ( I5,1X,6E13.4)

Appendix B
DO 50 IDOFN = 1, NDOFN
   K = (LORDT - 1) * NDOFN + IDOFN
50   BI(K) = BI(K) + POINT(IDOFN)
51 CONTINUE

500 CONTINUE

C*** DISTRIBUTED EDGE LOADS

READ (LU2,*) NEDGE
WRITE (LU3,925) NEDGE
925 FORMAT (/' No of Loaded Edges = ', I4)
600 CONTINUE

C*** DISTRIBUTED FACE STRESSES

READ(LU2,*) NFACER
WRITE (LU3,935) NFACER
935 FORMAT (/' No of Loaded Faces = ', I4)
IF ( NFACER .EQ. 0 ) GO TO 700

WRITE (LU3,940)
940 FORMAT (/' Element Pressure Direction & face code' + + '--------------------------------------------------')
c NODFA = 2 + NNODE / 3
do 30 i = 1, nfacer
C==== Read uniform pressure, loaded element and
c==== direction (0='X'dir., 1='Y'dir., 2='Z'dir.)
c==== face id ( 1='X-Y'face highest Z, 2='X-Z'face low Y)
c==== read(lu2,*) ll,q,id,iface
write(lu3,'(i5,e16.4,i2,7x,i2)') ll,q,id, iface
c==== Set the face variables.
   if(idface.eq.1) then
      corner(1)=13
      corner(2)=15
      corner(3)=17
      corner(4)=19
      center(1)=14
      center(2)=16
      center(3)=18
      center(4)=20
      iacor=2
      ibcor=1
   elseif(idface.eq.2) then
      corner(1)=1
      corner(2)=3
      corner(3)=15
      corner(4)=13
      center(1)=2
      center(2)=10
      center(3)=14
      center(4)=9
      iacor=3
      ibcor=1
   else
      write(lu0,'(/a)') ' Error assigning face to be loaded'
      stop
   endif

Appendix B
C==== Apply load to upper face only
    ik=corner(3)
    il=corner(2)
    im=corner(1)
    k = nodes(il,ik)
    l = nodes(il,il)
    m = nodes(il,im)
    ay = abs(coord(k,iacor) - coord(l,iacor))
    be = abs(coord(l,ibcor) - coord(m,ibcor))
    q1 = -q*ay*be/12.
    q2 = q*ay*be/3.

    do 6 j = 1,4
       ij=corner(j)
       k = nodes(il,ij)
       if(id.eq.0) kk = 6*k - 5
       if(id.eq.1) kk = 6*k - 4
       if(id.eq.2) kk = 6*k - 3
       bi(kk) = bi(kk) + q1
     6 continue
    do 7 j = 1,4
       ij=center(j)
       k = nodes(il,ij)
       if(id.eq.0) kk = 6*k - 5
       if(id.eq.1) kk = 6*k - 4
       if(id.eq.2) kk = 6*k - 3
       bi(kk) = bi(kk) + q2
     7 continue
    30 continue

700 CONTINUE

C*** GRAVITY LOADING SECTION

READ (LU2,910) NGRAV
WRITE (LU3,914) NGRAV
914 FORMAT (/'No of Gravity Loads = ',I4)
   IF (NGRAV.EQ.0) GO TO 800

C*** READ GRAVITY ANGLES ANTICLOCK REL TO -Z AXIS

READ (LU2,960) THETAX,THETAY
960 FORMAT (2D15.7)
WRITE (LU3,965) THETAX,THETAY
965 FORMAT (/'GRAVITY ANGLES REL TO -Z AXIS'/
     'THETA X Z = ',F10.3/
     'THETA Y Z = ',F10.3)

THETAX = THETAX / 57.295779514D0
THETAY = THETAY / 57.295779514D0
SINEX = DSIN(THETAX)
COSEX = D COS(THETAX)
SINEY = DSIN(THETAY)
COSEY = D COS(THETAY)
WRITE (LU3,970)
970 FORMAT (/'MATERIAL GRAVITATIONAL DENSITIES',
     'FORCE PER UNIT VOLUME', 'MATERIAL DENSITY')

C*** READ MATERIAL DENSITIES

Appendix B
DO 55 IMATS = 1,NMATS
READ (LU2,927) MATL,GRADEN(MATL)
927 FORMAT (I5, E15.7)
WRITE (LU3,975) MATL,GRADEN(MATL)
975 FORMAT (I7, 1PD15.7)
55 CONTINUE

C*** LOOP OVER EACH ELEMENT
DO 290 IELEM = 1,NUMEL

C*** SET UP PRELIMINARY CONSTANTS
LPROP = MATNO(IELEM)
RHOG = GRADEN(LPROP)
IF ( RHOG .EQ. 0.0 ) GO TO 290
GXCOM = RHOG * SINEX
GYCOM = RHOG * SINEY
GZCOM = -RHOG * ( COSEX + COSEY ) / 2.0DO

C*** COORDINATES OF THE ELEMENT NODAL POINTS
DO 210 INODE = 1,NNODE
LNODE = IABS(NODES(IELEM,INODE))
DO 210 IDIME = 1,NDIME
210 ELCOD(IDIME,INODE) = COORD(LNODE,IDIME)

C*** ENTER LOOPS FOR AREA INTEGRATION
DO 240 IGAUS = 1,NGAUS
EXISP = POSGP(IGAUS)
DO 240 JGAUS = 1,NGAUS
ETASP = POSGP(JGAUS)
DO 240 KGAUS = 1,NGAUS
EZESP = POSGP(KGAUS)

C*** COMPUTER THE SHAPE FUNCTIONS AT THE SAMPLING POINTS
C AND THE ELEMENT VOLUME
CALL SF3R2(EXISP,ETASP,EZESP)
LGASP=1
CALL JACOB3(IELEM,DJACB,LGASP)
DVOLU = DJACB * WEIGP(IGAUS) * WEIGP(JGAUS) * WEIGP(KGAUS)

C** CALCULATE NODAL EQUIVALENT LOADS AND ASSOCIATE WITH ELEMENT
DO 230 INODE = 1,NNODE
IEVAB1 = ( INODE - 1 ) * NDOPN + 1
IEVAB2 = IEVAB1 + 1
IEVAB3 = IEVAB2 + 1

+ ELOAD(IELEM,IEVAB1) = ELOAD(IELEM,IEVAB1) +
+ GXCOM * SHAPE(INODE) * DVOLU
+ ELOAD(IELEM,IEVAB2) = ELOAD(IELEM,IEVAB2) +
+ GYCOM * SHAPE(INODE) * DVOLU
230 ELOAD(IELEM,IEVAB3) = ELOAD(IELEM,IEVAB3) +
+ GZCOM * SHAPE(INODE) * DVOLU
240 CONTINUE
290 CONTINUE

Appendix B
800 CONTINUE

C*** TEMPERATURE LOADING OMITTED

C     WRITE (LU3,980)
C  980 FORMAT ( '// 5X, ' NODAL FORCES FOR EACH ELEMENT ' )

C     DO 390 IELEM = 1, NUMEL
C  390 WRITE (LU3,985) IELEM, (ELOAD(IELEM,IEVAB), IEVAB=1, NEVAB)
C  985 FORMAT ( 1X, 14.5X, 12D9.2 / (10X, 12D9.2) )
     RETURN
END

C ******************************************************************************
C S/R S/R S/R
C ******************************************************************************

SUBROUTINE STRE3D

IMPLICIT real*8 (A-H, O-Z)

COMMON / CONTO / NUMNOD, NUMEL, NNODE, NDOFN, NDIME, NSTRE, isc,
+ istel, istcod, NGAUS, NPROP, NMATS, NVFIX, NEVAB,
+ ICASE, NCASE

COMMON / LGDATA / COORD(9000, 3), PROPS(50, 10), PRESC(200, 3),
+ ELOAD(1700, 96), NODES(1700, 32),
+ MATNO(1700)

COMMON / WORK / ELCOD(3, 32), SHAPE(32), DERIV(3, 32), DMATX(6, 6),
+ CARTD(3, 32), DEMAT(6, 96), BMATX(6, 96),
+ POSGP(3), WEIGP(3), GPCOD(3),
+ NEROR(24), stplt(6000, 5)

COMMON / STRES/ EPS(6), STRF(1700, 6, 27), STR0(1700, 6, 27),
* DSTR(1700, 6, 27), IPEL(1700, 27)

COMMON / PROP / IP(196)
COMMON / SOLV1 / BB(54000), NVAR, nf(54000), nfv, bbl(54000)
COMMON / PLAS / BI(54000), DDIS(54000), RESID(54000)

COMMON / UNITS/ LUN, LUI, LUX, LUN, LUI, lu6

WRITE (LU0, 1000)
1000 FORMAT ( ' ENTERING S/R STRE3D ' )

C==== LOOP OVER EACH ELEMENT

DO 60 LL = 1, NUMEL
    LPROP = MATNO(LL)
    WRITE (LUN, 910) LL
C  910 FORMAT ( / 5X, 'ELEMENT NO. ', I5 )

C*** EVALUATE THE COORDINATES OF THE ELEMENT NODAL POINTS

DO 10 INODE = 1, NNODE
    LNODE = NODES(LL, INODE)
    DO 10 IDIME = 1, NDIME
    ELCOD(IDIME, INODE) = COORD(LNODE, IDIME)
    10 C

Appendix B
C== Set up position matrix.
   DO 55 I = 1, NNODE
      K = NODES(LL,I)
      nvnl = 6 - 1
      jj = 0
   DO 55 j = 3, nvnl
      l = 3*i - jj
      m = 6*i - j
      jj = jj + 1
   55 ip(l) = m

C*** ENTER LOOPS OVER THE SAMPLING POINTS
   NGP = 0
   DO 50 IGAUS = 1, NGAUS
   DO 50 JGAUS = 1, NGAUS
   DO 50 KGAUS = 1, NGAUS

C*** SET UP B AND D MATRICES FOR THIS GAUSS POINT
   NGP = NGP + 1
   EXISP = POSGP(IGAUS)
   ETASP = POSGP(JGAUS)
   EZEESP = POSGP(KGAUS)

C*** COMPUTE THE SHAPE FUNCTIONS AT THE SAMPLING POINTS
   CALL SF3R2(EXISP,ETASP,EZEESP)
   CALL JACOB3(LL,DJACB,NGP)

C*** EVALUATE THE B AND DB MATRICES FOR THE ELEMENT
   CALL BMAT3D

C== Set up the D matrix.

   CALL DMAT(IPROP)
   IF(IPEL(LL,NGP) .GT. 0) CALL PLAST(LL,NGP)

C*** COMPUTE THE CARTESIAN STRESS COMPONENTS AT THE SAMPLING POINTS

C== Multiply B*(Nodal deflections) to get strains EPS().
   DO 2 I = 1, NSTRE
      SUM = 0.0
   DO 1 J = 1, 60
      JJ = IP(J)
      1 SUM = SUM + BMATX(I,J)*DDIS(JJ)
   2 EPS(I) = SUM
   IF(ll .eq. 1) then
      write(lu3,'(a,i5)') ' strain matrix for ',ll
      write(lu3,'(6e13.7)') (eps(i),i-1,6)
   end if

C== Multiply D*(strains) to get stress changes DSTR().
   DO 4 I = 1, NSTRE
      SUM = 0.0
   DO 3 J = 1, NSTRE
      3 SUM = SUM + DMATX(I,J)*EPS(J)
   4 DSTR(LL,I,NGP) = -SUM

C*** OUTPUT THE STRESSES

Appendix B
50 CONTINUE
60 CONTINUE

RETURN
END

C==============================================================
C Subroutine to perform Choleski reduction of the variable
C bandwidth stiffness matrix, stored as a vector.
C==============================================================
C
C subroutine SPARIN

implicit real*8 (a-h,o-z)

common / diag/ kdiag(54000),ldiag(54000)
COMMON /SOLV1/ BB(54000), NVAR, nf(54000), nfvar,bbl(54000)
COMMON /SOLV / AM(24000000), MAXBW
COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

am(1) = sqrt(am(1))
do 10 i=2,nfvar
  ki = kdiag(i) - i
  L = kdiag(i-1) - ki + 1
  do 20 j=I,i
    x = am(ki+j)
    kj = kdiag(j) - j
    j1=j-1
    if(abs(j1).lt.1.0E-11) goto 20
    Lbar = kdiag(j-1) - kj + 1
    Lbar = max0(L,Lbar)
    ll=1bar-j
    if(abs(ll).lt.1.0E-11) goto 20
    m = j - l
    do 30 k=1bar,m
      x = x - am(ki+k)*am(kj+k)
    c write(lu3,'(a,2e13.7)') ' kj+j,am(kj+j) ', kj+j,am(kj+j)
  20 am(ki+j) = x/am(kj+j)
    if(abs(x).lt.1.0e-08) then
      write(lu3,'(a,2e13.4)') 'i, x in sparin ',i,x
      x = abs(x)
    endif
    10 am(ki+i) = sqrt(x)

return
end

C==============================================================
C Subroutine to perform the Choleski back-substitution
C on the variable bandwidth stiffness matrix.
C==============================================================
C
C subroutine SPABAC

implicit real*8 (a-h,o-z)

common / diag/ kdiag(54000),ldiag(54000)
COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

Appendix B
COMMON /SOLVI/ BB(54000), NVAR, nf(54000), nfvar,bbl(54000)
COMMON /SOLV/ /AM(240000000), MAXB

bbl(1) = bbl(1)/am(l)

do 10 i = 2,nfvar
   ki = kdiag(i) - i
   L = kdiag(i-1) - ki + 1
   x = bbl(i)
   if(L.eq.i) goto 10
   m = i - 1
   do 20 j = L,m
20      x = x - am(ki+j)*bbl(j)
10      bbl(i) = x/am(ki+i)
   do 30 it=2,nfvar
      i = nfvar + 2 - it
      ki = kdiag(i) - i
      x = bbl(i)/am(ki+i)
      bbl(i) = x
      L = kdiag(i-1) - ki + 1
      if(L.eq.i) goto 30
      m = i - 1
   do 40 k=1,L
40      bbl(k) = bbl(k) - x*am(ki+k)
30      continue
   bbl(1) = bbl(1)/am(l)

return
end

C==========================================
C Apply plastic correction to Right Hand Side
C==========================================

== SUBROUTINE PCOR(lli,itt) ==
IMPLICIT real*8 (A-H,O-Z)
DIMENSION TEMP(6,96)
COMMON /PLA/ BET(6), A(6), COR(96,96)
COMMON / TEMP4/ ESTIF(96,96), LOCEL(96), NDEST(96)
COMMON / CONTRO/ NUMNOD, NUMEL, NNODE, NDOFN, NDIME, NSTRE, isc,
                 istel, istcod, NGASU, NPRP, NMATS, NFIX, NEVAB,
                 ICASE, NCASE
COMMON / LGDATA/ COORD(9000,3), PROPS(50,10), PRESC(200,3),
                  ELOAD(1700,96), NODES(1700,32),
                  MATNO(1700)
COMMON / WORK/ ELCOD(3,32), SHAPE(32), DERIV(3,32), DMATX(6,6),
               CARTD(3,32), BMAT(6,96), BMATX(6,96),
               PCOSGP(3), WEIGP(3), GPCOD(3),
               NEROR(24), stplt(6000,5)
COMMON /STRES/ EPS(6), STRF(1700,6,27), STRB(1700,6,27),
               DSTR(1700,6,27), IPEL(1700,27)

Appendix B
COMMON / PROP / IP(96)
COMMON / PLAS / BI(54000), DDIS(54000), RESID(54000)
COMMON / BLOC / BX(54000), BSO(54000), TDIS(54000), XS(96, 96), +
                   stdis(1700, 48), btdis(500, 12)
COMMON / SOLV / AM(240000000), MAXEW
COMMON / SOLV1/ BB(54000), NVAR, nf(54000), nfvar,blb(54000)

common /excaiv/ ielex(150), istat(1700), nne, maxex
COMMON /INN/ 
MNLI, MAXIT, NNN, NLIOOLD, TOTAL, FMULT, CONV, NDSUM, IDIR
COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6
NGPTS = NGAUS*NGAUS*NGAUS

C==== Loop over each element
DO 480 LL = 1, NUMEL
   if(istat(ll) .eq. 0) go to 480
   LPROP = MATNO(LLU)
C==== Find no. of Gauss points that are plastic
   NG = 0
   DO 116 I = 1, NGPTS
      116 NG = NG + TPEL(LL, I)
   IF(NG .EQ. 0) GO TO 480
   DO 6 I = 1, 60
      DO 6 J = 1, 60
      6 XS(I, J) = 0.0
C WRITE (LU0,1000)
C 1000 FORMAT ( ' ENTERING S/R STIF3D ' )
C==== ZERO B MATRIX AND D MATRIX ONCE ONLY
DO 8 I = 1, NSTRE
   DO 5 J = 1, 60
      BMATX(I, J) = 0.0
5   CONTINUE
8 CONTINUE
C==== EVALUATE THE COORDINATES OF THE ELEMENT NODAL POINTS
DO 10 I = 1, NNODE
   LNODE = NODES(LL, I)
   DO 10 J = 1, NDIME
   10 ELCOD(J, I) = COORD(LNODE, J)
C==== Set up position matrix.
DO 55 I = 1, NNODE
   K = NODES(LL, I)
   nvn1 = 6 - 1
   jj = 0
   DO 55 J = 3, nvn1
      L = 3*I - jj
      M = 5*K - j
      JJ = JJ + 1
      55 IPT(J) = M
C==== Clear the element plastic correction matrix
C   DO 20 I = 1, NEVAB
C   DO 20 J = 1, NEVAB
C 20   COR(I, J) = 0.0

LG = 0

C==== Enter loops for area numerical integration

   DO 50 IG = 1, NGAUS
   DO 50 JG = 1, NGAUS
   DO 50 KG = 1, NGAUS
   LG = LG + 1
   IF(IE(L, LG) .EQ. 0) GO TO 50
   EXISP = POSGP(IG)
   ETASP = POSGP(JG)
   EZESP = POSGP(KG)

C==== Compute the shape functions at the Gauss points
C   and the element volume

   CALL SF3R2(EXISP, ETASP, EZESP)

   CALL JACOB3(LL, DJACB, LG)

C==== Evaluate the B matrix at the Gauss point
   CALL BMAT3D

C==== Evaluate the D matrix
   CALL DMAT(LPROP)

C==== Evaluate COR matrix for the Gauss point -
C==== COR is the plastic correction matrix
C   IF(IE(L, LG) .GT. 0) CALL PLAST(LL, LG)

   IF(IE(L, LG) .GT. 0 ) then
      IF(nli .eq. 1 .or. nli .eq. mnli) then
         write(1,16) (215,3e13.4) ll,lg,gpcod(1),gpcod(2),gpcod(3)
      endif
   endif

   IF(lg.eq.1) then
      WRITE(LU3,*) ' COR() after call to plast'
      WRITE(LU3, '(6E13.4)') ((COR(I,J),J=1,6),I=1,6)
   endif

      DO 440 I = 1, NSTRE
      DO 440 J = 1, 60
         SUM = 0.0
      DO 430 K = 1, NSTRE
         SUM = SUM + COR(I,K)*BMATX(K,J)
      430   TEMP(I,J) = SUM
      DO 433 I = 1, 60
      DO 433 J = 1, 60
         SUM = 0.0
      DO 444 K = 1, NSTRE
         SUM = SUM + BMATX(K,I)*TEMP(K,J)
      444   XS(I,J) = XS(I,J) + SUM*DVOLU

Appendix B
50 CONTINUE

C==== Set up correction matrix in vector BB()
DO 460 I = 1, 60
   II = IP(I)
   SUM = 0.0
   DO 450 J = 1, 60
      JJ = IP(J)
      SUM = SUM + XS(I,J)*DDIS(JJ)
450   BX(II) = BX(II) + SUM
460 CONTINUE

RETURN
END

C=================================================================================
+=
C Routine to set up the plastic correction matrix COR().
C=================================================================================
+=
SUBROUTINE PLAST(LL,NGP)

IMPLICIT REAL*8 (A-H,O-Z)
DIMENSION AL(6),BP(6),SAV(6),V1(6),V2(6),V3(6)

COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
                  istel,istcod,NGAUS,NPROP,NMATS,NVFIX,NEVAB,
                  ICASE,NCASE

COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3),
                  ELOAD(1700,96),NODES(1700,32),
                  MATNO(1700)

COMMON / WORK / ELCOD(32),SHAPE(32), DERIV(32,32),DMATX(6,6),
                  CARTD(32),DBMAT(6,96),BMATX(6,96),
                  POSGP(3), WEIGP(3), GPCOD(3),
                  MEROR(24), stplt(6000,5)

COMMON / PROP / IP(96)
COMMON / STRES/ EPS(6),STRF(1700,6,27),STRO(1700,6,27),
              DSTR(1700,6,27),IPEL(1700,27)

COMMON / PLA / BET(6), A(6), COR(96,96)
COMMON /UNITS/ LU0,LU1,LU2,LU3,LU4,lu6

PI = 3.14159265

C==== Compute average stress over increment
DO 1 J = 1, NSTRE
1  SAV(J) = STRO(LL,J,NGP) + DSTR(LL,J,NGP)/2.

MAT = MATNO(LL)
PH = PROPS(MAT,3)
S  = SIN(PH)
C  C = COS(PH)
C  FS = PROPS(MAT,4)
C  S1 = SIN(FS)
C  SSTR = PROPS(MAT,5)

C==== Three dimensional failure surface.
C==== Sloane's Mohr-Coulomb with rounded corners

Appendix B
C==== Needs compression -ve. Order of stresses shown below.
SX = -SAV(1)
SY = -SAV(2)
SZ = -SAV(3)
TXY = -SAV(4)
TYZ = -SAV(5)
TZX = -SAV(6)
SM = (SX + SY + SZ)/3.
XX = (SX*SX + SY*SY + SZ*SZ - (SX*SY + SY*SZ + SZ*SX))/3.
  + TXY*TXY + TYZ*TYZ + TZX*TZX
SB = SQRT(XX)
IF(SB .EQ. 0.0) THEN
  WRITE(0,'(/A,E13.4)') ' SB is zero *STOP*'
END
STOP
X1 = 2.*SX - SY - SZ
X2 = 2.*SY - SX - SZ
X3 = 2.*SZ - SX - SY
XJ3 = X1*X2*X3/27. - X1*TYZ*TYZ/3. - X2*TZX*TZX/3. -
X3*TXY*TXY
VAL = 0.
IF(SB.GT.0.) THEN
  VAL = 3.*SQRT(3.)*XJ3/2./SB**3
  if(val.gt.1.0) val = 1.0
  if(val.lt.-1.0) val = -1.0
END
XLODE = ASIN(-VAL)/3.
DEG = 180.*XLODE/PI
S3 = SIN(3.*XLODE)
C3 = COS(3.*XLODE)
TEST = ABS(deg)
C==== Transition surfaces. Starting at 25 degrees Lode angle.
IF( (TEST.GT.25.)) THEN
  SIGN = -1.
  IF(XLODE.GE.0.) SIGN = 1.
  AV = 1.432052 + 0.406942*SIGN*S
  BV = 0.544291*SIGN + 0.673903*S
  CON = (2.*BV*S3 + AV)*0.5/SB
  CON1 = 3.*SQRT(3.)*BV/XX/2.
ELSE
C==== Planar part of Mohr-Coulomb surface
  cl = cos(xlode)
  sl = sin(xlode)
  PL = -S3/SB/C3
  P2 = -SQRT(3./2./SB**3/C3
  Q1 = cl - s*sl/sqrt(3.)
  Q2 = -sl - s*cl/sqrt(3.)
  CON = (SB*Q2*P1 + Q1)*0.5/SB
  CON1 = SB*Q2*P2
END
C==== Differentiation wrt sigma bar
V1(1) = CON*X1/3.
V1(2) = CON*X2/3.
V1(3) = CON*X3/3.
V1(4) = CON*2.*TXY
V1(5) = CON*2.*TYZ
V1(6) = CON*2.*TZX
C==== Differentiation wrt J3
V2(1) = CON1*(2.*SY*SZ/3. + SX*SX/3. - SM*SM

Appendix B
\[ -2.0 \cdot TYZ \cdot TYZ / 3.0 + TZX \cdot TZX / 3.0 + TXY \cdot TXY / 3.0 \]
\[ V2(2) = CON1 \cdot 2.0 \cdot SZ \cdot SX / 3.0 + SY \cdot SY / 3.0 - SM \cdot SM \]
\[ -2.0 \cdot TZX \cdot TZX / 3.0 + TYZ \cdot TYZ / 3.0 + TXY \cdot TXY / 3.0 \]
\[ V2(3) = CON1 \cdot 2.0 \cdot SX \cdot SY / 3.0 + SZ \cdot SZ / 3.0 - SM \cdot SM \]
\[ -2.0 \cdot TXY \cdot TXY / 3.0 + TYZ \cdot TYZ / 3.0 + TZX \cdot TZX / 3.0 \]
\[ V2(4) = CON1 \cdot 2.0 \cdot TYZ \cdot TZX - 2.0 \cdot X3 \cdot TXY / 3.0 \]
\[ V2(5) = CON1 \cdot 2.0 \cdot TXY \cdot TZX - 2.0 \cdot X1 \cdot TYZ / 3.0 \]
\[ V2(6) = CON1 \cdot 2.0 \cdot TXY \cdot TYZ - 2.0 \cdot X2 \cdot TZX / 3.0 \]

C----- Differentiation wrt J1

\[ V3(1) = S / 3.0 \]
\[ V3(2) = S / 3.0 \]
\[ V3(3) = S / 3.0 \]
\[ V3(4) = 0.0 \]
\[ V3(5) = 0.0 \]
\[ V3(6) = 0.0 \]
\[ BP(1) = -(V1(1) + V2(1) + V3(1)) \]
\[ BP(2) = -(V1(2) + V2(2) + V3(2)) \]
\[ BP(3) = -(V1(3) + V2(3) + V3(3)) \]
\[ BP(4) = -(V1(4) + V2(4) + V3(4)) \]
\[ BP(5) = -(V1(5) + V2(5) + V3(5)) \]
\[ BP(6) = -(V1(6) + V2(6) + V3(6)) \]

C----- Von Mises

\[ XX = SX \cdot SX + SY \cdot SY + SZ \cdot SZ - SX \cdot SY - SY \cdot SZ - SZ \cdot SX \]
\[ + 3.0 \cdot TXY \cdot TXY + 3.0 \cdot TYZ \cdot TYZ + 3.0 \cdot TZX \cdot TZX \]
\[ RX = SQRT(XX) \]
\[ CON = 1.0 / (2.0 \cdot RX) \]
\[ A(1) = -CON \cdot 2.0 \cdot SX - SY - SZ \]
\[ A(2) = -CON \cdot 2.0 \cdot SY - SX - SZ \]
\[ A(3) = -CON \cdot 2.0 \cdot SZ - SX - SY \]
\[ A(4) = -CON \cdot 6.0 \cdot TXY \]
\[ A(5) = -CON \cdot 6.0 \cdot TYZ \]
\[ A(6) = -CON \cdot 6.0 \cdot TZX \]

C   WRITE(LU3,*) 'A and BP'
C   WRITE(LU3, '(6e13.4)') (a(i), i=1,NSTRE)
C   WRITE(LU3, '(6e13.4)') (BP(i), i=1,NSTRE)

DO 20 I = 1, NSTRE
   SUM = 0.0
   DO 10 J = 1, NSTRE
      10 SUM = SUM + DMATX(I,J) * A(J)
   20 AL(I) = SUM

   DO 40 I = 1, NSTRE
      SUM = 0.0
   DO 30 J = 1, NSTRE
      30 SUM = SUM + DMATX(I,J) * BP(J)
   40 BET(I) = SUM

   SUM = 0.0
   DO 50 I = 1, NSTRE
      SUM = SUM + BET(I) * A(I)
   50 SUM = SUM

   DO 60 I = 1, NSTRE
      DO 60 J = 1, NSTRE
         60 COR(I,J) = AL(I) * BET(J) / SUM

   DO 70 I = 1, NSTRE
      DO 70 J = 1, NSTRE
         70 DMATX(I,J) = DMATX(I,J) - COR(I,J)

Appendix B
C    WRITE(LU3,*) ' COR('  
C    WRITE(LU3, '(6E13.4)') ((COR(I,J), J=1,6), I=1,6)  
RETURN  
END  
C==================================================================================================  
C Subroutine to print out displacements and moments. 8 noded element.  
C==================================================================================================  
SUBROUTINE BMOM8  
IMPLICIT REAL*8 (A-H,O-Z)  
COMMON /ELST/ ES(48,48), B(8,48), CCD(6), WEI(6), NVN  
COMMON /SHAPEF/ H(4,8), P(4,2,8)  
COMMON /BLOK2/ PJ(8), ST(48,48), D(8,8), DB(8,48), XY(2,8),  
*      AJAC(2,2), TR(3,3)  
COMMON /LGDATA/ COORD(9000,3), PROPS(50,10), PRES(200,3),  
  +      ELOAD(1700,96), NODES(1700,32),  
  +      MATNO(1700)  
COMMON /CONTRO/ NUMNOD, NUMEL, NNODE, NDOFN, NDIME, NSTRE, isc,  
  +      istel, istcod, NGAUS, NPROP, NMATS, NFIX, NEVAB,  
  +      ICASE, NCASE  
COMMON /BLOC/ BX(54000), BXB(54000), TDIS(54000), XS(96,96),  
  +      stdis(1700,48), btdis(500,12)  
common /shtrc/ shprop (20,3), ishot (1700), ishpis (1700),  
  +      ishmat (1700)  
COMMON /UNITS/ LU0, LUI, LU0, LUI, LU3, LU4, lu6  
COMMON /SOLV2/ NVARS, nni, in(8), sr(1700,8,4)  
COMMON /PROPS/ EXR, EYR, PNUXR, PNUYR, thic  
  
dimension is(48)  
  
WRITE(LU3,110)  
WRITE(LU3,120)  
  
do 101 ll = 1, numel  
if (ishot (ll).eq.0) goto 101  

C== Set the shotcrete properties for this element.  
     mat = ishmat (ll)  
     exr = shprop (mat,1)  
     pnuxr = shprop (mat,2)  
     thic = shprop (mat,3)  

C== Read the side of the element to be shotcreted  
C==  0= on top, 1-right side  
    if (ishpos (ll).eq.0) then  
      in(1) = NODES (11,13)  
      in(2) = NODES (11,14)  
      in(3) = NODES (11,15)  
      in(4) = NODES (11,16)  
      in(5) = NODES (11,17)  
      in(6) = NODES (11,18)  
      in(7) = NODES (11,19)  
      in(8) = NODES (11,20)  
    else if (ishpos (ll).eq.1) then  
      in(1) = NODES (11,3)  
      in(2) = NODES (11,4)  
  
Appendix B
iN(3) = NODES(11,5)
iN(4) = NODES(11,11)
iN(5) = NODES(11,17)
iN(6) = NODES(11,16)
iN(7) = NODES(11,15)
iN(8) = NODES(11,10)

else if(ishpos(ll).eq.2) then
  iN(1) = NODES(11,1)
iN(2) = NODES(11,2)
iN(3) = NODES(11,3)
iN(4) = NODES(11,10)
iN(5) = NODES(11,15)
iN(6) = NODES(11,14)
iN(7) = NODES(11,13)
iN(8) = NODES(11,9)

else
  write(lu0,'(a)') ' Error assigning shotcrete position '
  stop
endif

C== zero B matrix
   do 23 i=1,8
       do 23 j=1,48
         23 b(i,j) = 0.
   call dmats

C== Set shotcrete position matrix.
   ii=0
   DO 20 I = 1,8
       DO 20 J = 1,6
           ii=ii+1
           20 is(ii) = (6*jn(i)) - (6-j)

C----- Go through the 4 Gauss points
   DO 100 N = 1, 4
     call bmat8(N,DET)
     CALL DxB

     DO 60 I = 1, 8
       SUM  = 0.0
       DO 70 K = 1, 48
         KK   = is(K)
         sdis = tdis(kk) - stdis(ll,kk)
         70 SUM = SUM + DB(I,K) * sdis
       60   SR(ll,i,n) = SUM

     PRINT(110, '(214,8E12.4)') LL, N, (SR(ll,i,n), i=1,8)

100 continue
101 continue

110 FORMAT(*'// STRESS RESULTANTS - FORCES AND MOMENTS PER UNIT WIDTH')
120 FORMAT(*'Elmt GP No. Nxx Nyy Nxy',
          *'Mxx Myy Mxy Qxx Qyy'/*)
RETURN
END

= = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = =

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Routine to calculate the nodal freedom vector

= = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = = =

subroutine free

IMPLICIT real*8 (A-H,O-Z)

common /shotcr/ shprop(20,3),ishot(1700),ishpos(1700),
+ ishmat(1700)
COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
+ istel,istcod,NGAUS,NPROP,NUMATS,NVFIX,NEVAB,
+ ICASE,NCASE
COMMON /SOLVI1/ BB(54000),NVAR,nf(54000),nfvar,bbl(54000)
COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3),
+ ELOAD(1700,96),NODES(1700,32),
+ MATNO(1700)
common /excav/ ielex(150),istat(1700),nee,maxex
COMMON /SUB2 / ITEMP(54000),NT,IPLT
COMMON /UNITS/ LU0, L01, LU2, LU3, LU4, Lu6

dimension in(48),ip(120)

c== Initialise Nodal Freedom vector.
do 75 i=1,nvar
   nf(i) = -1
75 continue

do 10 ll=1,numel

c== Place zero entries for all non-excavated elements.
   if(istat(ll).eq.0) goto 10

C==== Set up position matrix.
   DO 55 I = 1, NNODE
       K = NODES(LL,I)
       jj = 0
       do 55 j = 3,5
           l = 3*i - jj
           m = 6*k - j
           jj = jj + 1
55   ip(l) = m

   do 23 i=1,60
      ia = ip(i)
      nf(ia) = 0
23 continue

C== Loop over shotcrete elements.
   if(ishot(ll).eq.0) goto 60

C== Read the side of the element to be shotcreted
C== 0 = on top, 1 = right side

Appendix B
if(ishpos(ll).eq.0) then
  in(1) = NODES(ll,13)
  in(2) = NODES(ll,14)
  in(3) = NODES(ll,15)
  in(4) = NODES(ll,16)
  in(5) = NODES(ll,17)
  in(6) = NODES(ll,18)
  in(7) = NODES(ll,19)
  in(8) = NODES(ll,20)
else if(ishpos(ll).eq.1) then
  in(1) = NODES(ll,3)
  in(2) = NODES(ll,4)
  in(3) = NODES(ll,5)
  in(4) = NODES(ll,11)
  in(5) = NODES(ll,17)
  in(6) = NODES(ll,16)
  in(7) = NODES(ll,15)
  in(8) = NODES(ll,10)
else if(ishpos(ll).eq.2) then
  in(1) = NODES(ll,1)
  in(2) = NODES(ll,2)
  in(3) = NODES(ll,3)
  in(4) = NODES(ll,10)
  in(5) = NODES(ll,15)
  in(6) = NODES(ll,14)
  in(7) = NODES(ll,13)
  in(8) = NODES(ll,9)
else
  write(iu0,'(/a)') ' Error assigning shotcrete position '
  stop
endif
c=== Set up nf().
  do 20 i=1,8
    do 20 j=4,6
      ii=(6*in(i)) - (6-j)
      nf(ii) = 0
  20 continue
  60 continue
  10 continue
c=== Apply boundary conditions to nf().
  do 70 ll=1,nt
    k=itemp(ll)
    if(nf(k).eq.0) then
      nf(k)=-1
    endif
  70 continue
c=== Set the non-negative numbers in nf()
  n=1
  do 30 i=1,nvar
    if(nf(i).eq.-1) goto 30
    nf(i) = n
    n=n+1
  30 continue
30 continue

    nfvar = n - 1

    write(lu3,'(/a)') ' final nf() '
    write(lu3,'(e13.7)') nfvar

    c    write(lu3,'(/a)') ' node free nf() '    
    c    k=0
    c    do 40 i=1,numnod
    c    do 40 j=1,ndofn
    c    k= k+1
    c    write(lu3,'(4i8)') i,j,nf(k),k
    c    40 continue

    c    write(lu3,'(/a,i8)') ' nfvar ',nfvar

    return
end

==============================================================================

===

===

Routine to set the initial shotcrete layer positions.

===

subroutine shotset

implicit real*8 (a-h,o-z)

COMMON / LGDATA / COORD(9000,3), PROPS(50,10), PRES(200,3),
 + ELOAD(1700,96), NODES(1700,32),
 + NATNO(1700)
COMMON / BLOC / EX(54000), BXO(54000), TDIS(54000), XS(96,96),
 + stdis(1700,48), btdis(500,12)
common / shotcr/ shprop(20,3), ishot(1700), ishpos(1700),
 + ishmat(1700)
common / setsh/ numshot(150), nes
COMMON / SOLV1/ BB(54000), NVAR, nf(54000), nfvar, bbl(54000)
COMMON / UNITS/ LU0, LUL, LU2, LU3, LU4, lu6

dimension in(8), is(48)

    write(lu3,'(a)') ' entering shotset '

    c=== Loop over the new shotcrete elements.

    do 100 il = 1, nes
    ll = numshot(ll)

    c=== Read the side of the element to be shotcreted

    c== 0= on top, 1=right side

    if(ishpos(ll).eq.0) then
    in(1) = NODES(ll,13)
    in(2) = NODES(ll,14)
    in(3) = NODES(ll,15)
    in(4) = NODES(ll,16)
    in(5) = NODES(ll,17)
    in(6) = NODES(ll,18)

Appendix B
in(7) = NODES(11,19)
in(8) = NODES(11,20)

else if(ishpos(11).eq.1) then
  in(1) = NODES(11,3)
in(2) = NODES(11,4)
in(3) = NODES(11,5)
in(4) = NODES(11,11)
in(5) = NODES(11,17)
in(6) = NODES(11,16)
in(7) = NODES(11,15)
in(8) = NODES(11,10)

else if(ishpos(11).eq.2) then
  in(1) = NODES(11,1)
in(2) = NODES(11,2)
in(3) = NODES(11,3)
in(4) = NODES(11,10)
in(5) = NODES(11,15)
in(6) = NODES(11,14)
in(7) = NODES(11,13)
in(8) = NODES(11,9)

else
  write(lu0,'(/a)') ' Error assigning shotcrete position ' stop
endif

c== Set shotcrete position matrix.
i = 0
DO 20 I = 1,8
DO 20 J = 1,6
  ii = ii + 1
20  is(ii) = (6*in(i)) - (6-j)

c== Set the initial position stdis() at the previous displacements.
do 30 i=1,48
  kk = is(i)
  stdis(11,i) = tdis(kk)
30 continue

100 continue

return
end

Appendix B
common / bolt / bltmat(500), bltvar(500,4), nbolt(500,2),
            numblt, nbtemp
COMMON /UNITS/ LU0, LU1, LU2, LU3, LU4, lu6

dimension ib(12)

    do 100 ll = nbtemp,numblt

*===== Set up the bolt position matrix.

    ii=0
    do 10 i=1,2
      do 10 j=1,6
         ii=ii+1
      10 ib(ii) = (6*nbolt(ll,i)) - (6-j)

*===== Set the displacements.

    do 20 i=1,12
       kk = ib(i)
       btdis(ll,i) = tdis(kk)
    20 continue

100 continue

    return

end
SUBROUTINE INTEGL

implicit real*8 (a-h,o-z)
COMMON /SOLVI/ BB(42000),NVAR,rf(42000),nvar,bbl(42000)
common /prop / ip(96)
COMMON /UNITS/ LU0,LU1,LU2,LU3,LU4
COMMON / WORK / ELCOD(3,32),SHAPE(32), DERIV(3,32),DMATX(6,6),
+ CARTD(3,32),DBMAT(6,96),BMATX(6,96),
+ POSGP(3), WEIGP(3), GPCOD(3),
+ NEROR(24), stplt(6000,5)
COMMON /JNTS / NJEL,NJN,JMTL(350),NMS(350,6),CCHJ(10),SKM(10),
+ SNM(10),PHJ(10),PSJ(10),ROT(12,12),DJ(2,2),
+ SF(2,12),IFJ(350,3),SJMNT(350,6),DSJNT(350,6),
+ sjnt(350,6)
common /lgdata/ coord(7000,3),props(50,10),presc(200,3),
+ eload(1500,96),nodes(1500,32),matno(1500)
COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
+ istel,istcod,NGAUS,NPROP,NNATS,NVFIX,NEVAB,
+ ICASE,NCASE
COMMON /STRES/ eps(6),strf(1500,6,27),STRO(1500,6,27),
+ dstr(1500,6,27),ipel(1500,27)
COMMON /PLAS/ BI(42000),ddis(42000),resid(42000),binit(42000),
+ initbi,inbsw
common /excav/ ielex(150),istat(1500),nee,maxex
common /gamma/ graden(50)
COMMON /SOLV2/ NVAR2,nen,ni(8),sx(1500,8,4)
COMMON /ELST/ ES(48,48),B(8,48),COD(6),WEI(6),NVN
COMMON /BLOK2/ PJ(8), ST(48,48), D(8,8), DB(8,48), XY(2,8),
+ AJAC(2,2), TR(3,3)
COMMON /PROPS/ EXR, EYR, PNUXR, PNURYR, thic
common /shopt/ shprop(20,3),ishot(1500),ishpos(1500),
+ ishm(1500)
COMMON /BLOC/ BX(42000), BKO(42000), TDIS(42000), XSN(96,96),
+ stdis(1500,48),bdtdis(500,12)
common /bolt/ blmat(500), bltvx(500,4), nbolt(500,2),
+ numbl, ntemp
common /bltf/ btes(12,12), temptm(3,3), trans(12,12),
+ blgs(12,12)

DIMENSION is(48),ib(12),sbum(12,12)

write(lu0,'(/a)') ' Entering intag'

DO 20 I = 1, nvar
20 bi(I) = 0.0

xsl = 1.
xwl = 1.

call prelim

C== Calculate the residual for the solid elements

Appendix B
DO 1000 LL = 1, NUMEL
IF(ISTAT(LL) .EQ. 0) GO TO 1000
MTL = matno(LL)
GM = graden(mtl)

C==== set up elcod for each element

do 1006 inode = 1,nnode
   inode = nodes(ll,inode)
do 1006 idime = 1,ndime
1006 elcod(idime,inode) = coord(inode,idime)

C==== Set up position matrix.
DO 55 I = 1, NNODE
   K = NODES(LL,I)
   nvnl = 6 - 1
   jj = 0
   do 55 j = 3, nvnl
      l = 3*I - jj
      m = 6*K - j
      jj = jj + 1
      55 ip(l) = m

C==== Begin numerical integration of BT*sigma.
IGPNO = 0
DO 10 IG = 1, ngaus
   S = posgp(IG)
   WT1 = Weigp(IG)
   DO 10 JG = 1, ngaus
      T = posgp(JG)
      WT2 = weigp(JG)
do 10 kg - 1, ngaus
   v = posgp(kg)
   wt3 = weigp(kg)
   IGPNO = IGPNO + 1

C==== Set up shape functions at Gauss point.
CALL SF3r2(s,t,v)
call jacob3(ll,djacb,igpno)
dvolu = djacb*wt1*wt2*wt3
c    if(11.eq.1 .or. 11.eq.2) then
c      write(lu3,'(a,i5)') ' dvolu for ',ll
c      write(lu3,'(e13.4)') dvolu
c    endif

C==== Set up B( ) matrix.
CALL BMAT3d
c    if(ll.eq.1) then
c      write(lu3,'(/a,i3)') ' Bmatx from tdl ', ll
c      write(lu3,'(3e15.7)') ((bmatx(k,j),k=1,6),j=1,60)
c    endif

DO 3 I = 1, 60
   KP = IP(I)
   SUM = 0.0
   DO 4 K = 1, nstre
      4 SUM = SUM + Bmatx(K,I)*STRO(LL,K,IGPNO)
3 bi(KP) = bi(KP) + SUM*dvolu*xs1
DO 8 I = 1, nnnode
  L = 3*1
  KP = IP(L)
8 bi(KP) = bi(KP) - SHAPE(I)*GM*dvolu*xw1
10 CONTINUE

c== Now consider the shotcrete elements.

  if(ishot(ll).eq.0) goto 1000

c== Set the shotcrete properties for this element.

  mat = ishmat(ll)
  exr = shprop(mat,1)
  pnuxr = shprop(mat,2)
  thic = shprop(mat,3)

c== Read the side of the element to be shotcreted

  c== 0= on top, 1=right side

  if(ishpos(ll).eq.0) then
    iN(1) = NODES(ll,13)
    iN(2) = NODES(ll,14)
    iN(3) = NODES(ll,15)
    iN(4) = NODES(ll,16)
    iN(5) = NODES(ll,17)
    iN(6) = NODES(ll,18)
    iN(7) = NODES(ll,19)
    iN(8) = NODES(ll,20)
  else if(ishpos(ll).eq.1) then
    iN(1) = NODES(ll,3)
    iN(2) = NODES(ll,4)
    iN(3) = NODES(ll,5)
    iN(4) = NODES(ll,11)
    iN(5) = NODES(ll,17)
    iN(6) = NODES(ll,16)
    iN(7) = NODES(ll,15)
    iN(8) = NODES(ll,10)
  else if(ishpos(ll).eq.2) then
    iN(1) = NODES(ll,1)
    iN(2) = NODES(ll,2)
    iN(3) = NODES(ll,3)
    iN(4) = NODES(ll,10)
    iN(5) = NODES(ll,15)
    iN(6) = NODES(ll,14)
    iN(7) = NODES(ll,13)
    iN(8) = NODES(ll,9)
  else
    write(lu0,'(/a)') ' Error assigning shotcrete position ' stop
endif

c== Set shotcrete position matrix.

  ii=0
  DO 21 I = 1, 8
  DO 21 J = 1, 6
  ii=ii+1

Appendix B
21 is(ii) - (6*in(i)) - (6-j)

c=== Set up the variables for transformation & zero appropriate matrices.

   DO 12 I = 1, 8
12   PJ(I) = 0.
   DO 11 I = 1, 8
   DO 11 J = 1, 48
11   B(I,J) = 0.

call dmats

c=== Begin numerical integration of BT*moment

   ignore = 0
   do 22 ig=1,2
   do 22 jg=1,2
   ignore = ignore + 1

   call bmats(igno, det)

   do 23 i=1,48
      kpi = is(i)
      sum = 0.0
   do 24 k=1,8
24      sum = sum + B(k,i)*sr(ll,k,igno)
23   bi(kpi) = bi(kpi) - sum*det

   22 continue

1000 CONTINUE

   c=== Now consider the rock bolt elements.

   do 1001 l1 = 1, numblt
   mat = bltmat(l1)
   Esb = bltvar(mat, 1)
   area = bltvar(mat, 2)
   Iz = bltvar(mat, 3)
   Iy = bltvar(mat, 4)

   n1 = nbolt(l1, 1)
   n2 = nbolt(l1, 2)

   c=== determine bolt variables.

   dx = coord(n2, 1) - coord(n1, 1)
   dy = coord(n2, 2) - coord(n1, 2)
   dz = coord(n2, 3) - coord(n1, 3)

   b1 = sqrt((dx)**2 + (dy)**2 + (dz)**2)

   c==== Set up the bolt position matrix.

   ii=0
   do 91 i=1,2
   do 91 j=1,6
   ii=ii+1

Appendix B
\[ \text{ib}(i) = (6 \times \text{nbolt}(i)) - (6-j) \]

c== Set up stiffness matrix for a truss element.
c==== initialise the stiffness matrix.
   do 26 i=1,12
   do 26 j=1,12
     bltes(i,j) = 0.
   26 continue
   bltes(1,1) = (Esb*area)/bl
   bltes(1,7) = -1*(Esb*area)/bl
   bltes(7,1) = -1*(Esb*area)/bl
   bltes(7,7) = (Esb*area)/bl
   c write(lu3,'(a)') ' bltes()
   c write(lu3,'(6e13.7)') ((bltes(i,j), j=1,12), i=1,12)

c== Set up transformation matrix for the bolt -
c==== X-axis runs down bolt and alpha=0.

c== initialise temptm() and trans()
   do 31 i=1,3
   do 31 j=1,3
     temptm(i,j) = 0.
   31 continue
   do 61 i=1,12
   do 61 j=1,12
     trans(i,j) = 0.
   61 continue

c== Set up the required values.
   Cx = dx/bl
   Cy = dy/bl
   Cz = dz/bl

c== Test to see if the X axis is vertical
   if(dx.eq.0.and.dz.eq.0) then
     temptm(1,2) = Cy
     temptm(2,1) = -1*Cy
     temptm(3,3) = 1
   else
     C22 = sqrt((Cx*Cx)+(Cz*Cz))
     C21 = (-1*Cx*Cy)/C22
     C23 = (-1*Cy*Cz)/C22
     C31 = (-1*Cz)/C22
     C33 = (Cz)/C22
     temptm(1,1) = Cx
     temptm(1,2) = Cy
     temptm(1,3) = Cz
     temptm(2,1) = C21
     temptm(2,2) = C22
     temptm(2,3) = C23

Appendix B
```plaintext
temptm(3,1) = C31
temptm(3,3) = C33
endif

c    write(lu3,'(a)') ' 3x3 transformation matrix '  
c    write(lu3,'(3e15.7)') ((temptm(i,j),j=1,3),i=1,3)

c=== Compute Final Transformation matrix.
    do 41 k=0,9,3
    do 51 i=1,3
       ii = i + k
    do 51 j=1,3
       jj = j + k
       trans(ii,jj) = temptm(i,j)
      51 continue
    41 continue

c=== Find global stiffness matrix.
  c====== [K] = [trans]T*[Km]*[trans]
    do 71 i=1,12
    do 71 j=1,12
       sbum(i,j) = 0.
    do 71 k=1,12
       sbum(i,j) = sbum(i,j) + bltes(i,k)*trans(k,j)
      71 continue
    do 81 i=1,12
    do 81 j=1,12
       bltgs(i,j) = 0.
    do 81 k=1,12
       bltgs(i,j) = bltgs(i,j) + trans(k,i)*sbum(k,j)
      81 continue

c    write(lu3,'(a)') ' bltgs() '
  c    write(lu3,'(6e13.7)') ((bltgs(i,j),j=1,12),i=1,12)

c=== Calculate the forces to retain equilibrium.
    do 27 i=1,12
    kk=ib(i)
    sum=0.0
    do 28 k=1,12
    kp=ib(k)
    bdis = tdis(kp) - btdis(11,k)
   28    sum = sum + bltgs(i,k)*bdis
    27    bi(kk) = bi(kk) - sum
   1001 continue

c=== Exclude forces due to initial loading.
    if(inbsw .eq. 0) goto 139
    do 189 i=1,nvar
       bi(i) = bi(i) + binit(i)
   189 continue
   139 continue

c    write(lu3,'(a)') ' Non-zero Excavation forces '
```

Appendix B
do 46 i = 1, numnod
   k = 6*i
   k2 = k - 1
   k3 = k - 2
   k1 = k - 3
   l = k - 4
   m = k - 5
write(lu3,'(i5,6f13.4)') i,bi(m),bi(l),bi(kl),bi(k3),bi(k2),
   + bi(k)
46 continue

write(lu3,'(a)') ' leaving integl'

RETURN
END
Subroutine sbolt, sets up a simple rock bolt element which only considers the axial strength of the bolt.

subroutine sbolt

implicit real*8 (a-h,o-z)

COMMON / LGDATA / COORD(9000,3), PROPS(50,10), PRESC(200,3),
+                    ELOAD(1700,96), NODES(1700,32),
+                    MATNO(1700)
common / diag/ kdiag(54000), ldiag(54000)
COMMON / SOLV / AM(24000000), MAXBW
COMMONE/SOLV1/ BB(54000), NVAR, nf(54000), nftvar, bbl(54000)
COMMON / UNITS/ LU0, LU1, LU2, LU3, LU4, lu6
common / bolt / bitmat(500), bltvar(500,4), nbolt(500,2),
+                    numbtl, nubtemp
common / bltst/ bltes(12,12), temptm(3,3), trans(12,12),
+                    bltgs(12,12)

dimension sum(12,12), ib(12)

write(lu3,'(/a)') ' Rock bolt Properties '
write(lu3,'(a)') ' Nodes      Es      Area      Iz      Iy      '
write(lu3,'(a)') '-----------------------------------------'

c== Loop over elements.

do 10 ll=1,nublt
mat = bitmat(ll)
Es = bltvar(mat,1)
area = bltvar(mat,2)
Iz = bltvar(mat,3)
Iy = bltvar(mat,4)

nl=nbolt(ll,1)
n2=nbolt(ll,2)

write(lu3,'(2i6,4e13.5)') nl,n2,Es,area,Iz,Iy

c== determine bolt variables.

dx = coord(n2,1) - coord(n1,1)
dy = coord(n2,2) - coord(n1,2)
dz = coord(n2,3) - coord(n1,3)

bl = sqrt( (dx)**2 + (dy)**2 + (dz)**2 )

c==== Set up the bolt position matrix.

ii=0
do 90 i=1,2
do 90 j=1,6
ii=ii+1
90 ib(ii) = (6*nbolt(ll,i)) - (6-j)

Appendix B
c==== Set up stiffness matrix for a truss element.
c==== set up the stiffness matrix.
    do 20 i=1,12
        do 20 j=1,12
            bltes(i,j) = 0.
        20 continue
        bltes(1,1) = (Es*area)/bl
        bltes(1,7) = -1*((Es*area)/bl)
        bltes(7,1) = -1*((Es*area)/bl)
        bltes(7,7) = (Es*area)/bl
    c    write(lu3,'(a)') ' bltes() '
    c    write(lu3,'(6e13.7)') ((bltes(i,j),j=1,12),i=1,12)

c==== Set up transformation matrix for the bolt -
c==== X-axis runs down bolt and alpha=0.
c==== initialise temptm() and trans()
    do 30 i=1,3
        do 30 j=1,3
            temptm(i,j) = 0.
        30 continue
    do 60 i=1,12
        do 60 j=1,12
            trans(i,j) = 0.
        60 continue

c==== Set up the required values.
    Cx = dx/bl
    Cy = dy/bl
    Cz = dz/bl

c==== Test to see if the X axis is vertical
    if(dx.eq.0.and.dz.eq.0) then
        temptm(1,2) = Cy
        temptm(2,1) = -1*Cy
        temptm(3,3) = 1
    else
        C22 = sqrt((Cx*Cx)+(Cz*Cz))
        C21 = (-1*Cx*Cy)/C22
        C23 = (-1*Cy*Cz)/C22
        C31 = (-1*Cz)/C22
        C33 = (Cx)/C22
        temptm(1,1) = Cx
        temptm(1,2) = Cy
        temptm(1,3) = Cz
        temptm(2,1) = C21
        temptm(2,2) = C22
        temptm(2,3) = C23
        temptm(3,1) = C31
        temptm(3,3) = C33

Appendix B
endif

write(lu3,'(a)') ' 3x3 transformation matrix'
write(lu3,'(3e15.7)') ((temtm(i,j),j=1,3),i=1,3)

c=== Compute Final Transformation matrix.
do 40 i=0,9,3
do 50 i=1,3
   ii = i + k
   do 50 j=1,3
      jj = j + k
      trans(ii,jj) = temtm(i,j)
   50 continue
do 40 continue

c=== Find global stiffness matrix.
c===== [K] = [trans]T*[Km]*[trans]
do 70 i=1,12
do 70 j=1,12
   sum(i,j) = 0.
do 70 k=1,12
   sum(i,j) = sum(i,j) + bltes(i,k)*trans(k,j)
70 continue
do 80 i=1,12
do 80 j=1,12
   bltgs(i,j) = 0.
do 80 k=1,12
   bltgs(i,j) = bltgs(i,j) + trans(k,i)*sum(k,j)
80 continue

write(lu3,'(a)') ' bltgs()
c write(lu3,'(6e13.7)') ((bltgs(i,j),j=1,12),i=1,12)

c=== Add the rock bolt stiffness to the mesh stiffness matrix.
do 100 i=1,12
   ii=ib(i)
   ia=nf(ii)
   if(ia.eq.-1) goto 100
   do 110 j=1,12
      jj=ib(j)
      ja=nf(jj)
      if(ja.eq.-1) goto 110
      kk = ia - ja
      if(kk.lt.0) goto 110
      ips = kdiag(ia) - kk
      am(ips) = am(ips) + bltgs(i,j)
110 continue
100 continue

return
end

Appendix B
Subroutine bltload - calculates the force in the bolt.

```
subroutine bltload

implicit real*8 (a-h,o-z)

COMMON / LGDATA / COORD(9000,3), PROPS(50,10), PRES(200,3),
+ ELOAD(1700,96), NODES(1700,32),
+ MATNO(1700)
COMMON / BLOC / RX(54000), BXO(54000), TDIS(54000), XS(96,96),
+ stdis(1700,48), btdis(500,12)
COMMON / UNITS/ L100, L1, L12, L02, L03, L04, L16
common / bolt / bltmat(500), bltvar(500,4), nbolt(500,2),
+ numbtl, nbtemp
common / bltst/ bltes(12,12), temptm(3,3), trans(12,12),
+ bltg(12,12)

dimension ib(12), edis(12), bltfor(500)

if(numblt.gt.0)write(lu3,104)
104 format ( // ' Blt no. bit. nodes local displ. force'
* /'------------------------------------------'

c== Loop over elements.
    do 10 ll=1,numblt
    mat = bltmat(ll)
    Es = bltvar(mat,1)
    area = bltvar(mat,2)

    n1=nbolt(ll,1)
    n2=nbolt(ll,2)

c== determine bolt variables.
    dx = coord(n2,1) - coord(n1,1)
    dy = coord(n2,2) - coord(n1,2)
    dz = coord(n2,3) - coord(n1,3)

    bl = sqrt( (dx)**2 + (dy)**2 + (dz)**2 )

c===== Set up the bolt position matrix.
    ii=0
    do 90 i=1,2
    do 90 j=1,6
    ii=ii+1
    90 ib(ii) = (6*nbolt(ll,i)) - (6-j)

c=== Set up transformation matrix for the bolt -
c===== X-axis runs down bolt and alpha=0.

c--- initialse temptm() and trans()
    do 30 i=1,3
    do 30 j=1,3

Appendix B
temptm(i,j) = 0.
30 continue
  do 60 i=1,12
    do 60 j=1,12
      trans(i,j) = 0.
    60 continue
  c--- Set up the required values.
    Cx = dx/bl
    Cy = dy/bl
    Cz = dz/bl
  c--- Test to see if the X axis is vertical
    if(dx.eq.0.and.dz.eq.0) then
      temptm(1,2) = Cy
      temptm(2,1) = -1*Cy
      temptm(3,3) = 1
    else
      C22 = sqrt((Cx*Cx)+(Cz*Cz))
      C21 = (-1*Cx*Cy)/C22
      C23 = (-1*Cy*Cz)/C22
      C31 = (-1*Cz)/C22
      C33 = Cx/C22
      temptm(1,1) = Cx
      temptm(1,2) = Cy
      temptm(1,3) = Cz
      temptm(2,1) = C21
      temptm(2,2) = C22
      temptm(2,3) = C23
      temptm(3,1) = C31
      temptm(3,3) = C33
    endif
  c    write(1u3,'(a)') ' 3x3 transformation matrix '
  c    write(1u3,'(3e15.7)') ((temptm(i,j),j=1,3),i=1,3)
  c--- Compute Final Transformation matrix.
    do 40 k=0,9,3
      do 50 i=1,3
        ii = i + k
        do 50 j=1,3
          jj = j + k
          trans(ii,jj) = temptm(i,j)
        50 continue
      40 continue
  c--- Compute the local element displacement parallel to the bar.
  c--- [trans]*[dis]
    do 101 i=1,12
      sum = 0.0
      do 102 k=1,12
      102 continue
  Appendix B
kk = ib(k)
bdis = tdis(kk) - btdis(ll,k)
102 sum = sum + trans(i,k)*bdis
101 edis(i) = sum

c=== Calculate the forces.
u1 = edis(1)
u2 = edis(7)

bltfor(ll) = ((u2-u1)*Es*area)/bl

c=== Calculate the mid points for plotting
pxmid = coord(n2,1) +((coord(n2,1)-coord(n1,1))/2)
pymid = coord(n2,2) +((coord(n2,2)-coord(n1,2))/2)
pzmid = coord(n2,3) +((coord(n2,3)-coord(n1,3))/2)

c=== Print out the results.

write(lu3,103) ll,n1,n2,u1,u2,bltfor(ll)
write(lu4,'(i4,4e15.7)') ll,pxmid,pymid,pzmid,bltfor(ll)
103 format (i4,5x,i5,4x,i5,3x,e15.7,2x,e15.7,2x,f11.4)

10 continue

return
end
C subroutine to set up shotcrete layer
contribution to the stiffness matrix.
modified for td2.for

C
C
C subroutine shotcr

IMPLICIT REAL*8 (A-H,O-Z)

PARAMETER ( MAXBWS = 180, MAXEL = 460, MAXNOD = 540 )
PARAMETER ( MAXTH = 50, MAXBC = 100, MAXPL = 300, MAXUL = 50,
* MAXMO = 30, MAXMX = 50, MAXMY = 50 )
PARAMETER ( MAXEQ = MAXEL+3, MAXE1 = MAXEQ+1, MAXVAR = 6*MAXNOD,
* NBCND = 6*MAXBC, maxam = maxvar*maxbws )

CHARACTER BAKSP*1

COMMON / CONTRO / NUMNOD, NUMEL, NODES, NDOPN, NDIME, NSTRF, insc,
  istel, istcod, NGAMU, NPROP, NMATS, NVFIX, NEVAB,
  ICASE, NCASE
COMMON / LGDATA / COORD(9000,3), PROPS(50,10), PRES(200,3),
  ELOAD(1700,96), NODES(1700,32),
  MATNO(1700)
COMMON / BLOK2 / PJ(8), ST(48,48), D(8,8), DB(8,48), XY(2,8),
  AJAC(2,2), TR(3,3)
COMMON / SOLV / AM(240000000), MAXBW
COMMON / SOLV1 / BB(54000), NVAR, nf(54000), ntv,bbl(54000)
COMMON / SOLV2 / NVARS, ntv, in(8), sr(1700,8,4)
COMMON / PROPS / EXR, EYR, PNUXR, PNURY, thic
COMMON / ELST / ES(48,48), B(8,48), COD(6), W(50), NVN
COMMON / UNITS / U0, U1, U2, U3, U4, u6
common / shotcr/ shprop(20,3),ishot(1700),ishpos(1700),
  ishmat(1700)
common / diag/ kdiag(54000),ldiag(54000)

DIMENSION is(48)

BAKSP = CHAR(8)
IBAK = 3
nvars = 48

C===== Open logical unit numbers. [0=vdu], [2=read], [3=write],
[4=plot],
C===== [6=external mesh file].
C===== [20=external interaction matrix].
LU0 = 0
LU2 = 2
LU3 = 3
LU4 = 4

NEN = 8
NVN = 48
call prelim

WRITE(LU0,'(/A)') ' Beginning assembly of shotcrete stiffness

Appendix B
' matrix'
WRITE(LU0,'((//A))') ' Assembling shotcrete element '
write(lu3,57)
57 format('/ Properties of shotcreted elements '/'
'------------------------------------------'/
' 'Elem.',4x,'mat1',6x,' E ',8x,'Pois.',6x,'thic')

C==== ASSEMBLE STIFFNESS MATRIX

    do 100 ll = 1,numel
        if(ishot(ll).eq.0) goto 100
        WRITE(LU0,'("*",I3)') LL
        DO 36 IB = 1, IBAK
            36 WRITE(LU0,'("*", A)') BAKSP

C== Set the shotcrete properties for this element.
    mat = ishmat(ll)
    exr = sprop(mat,1)
    pnuxr = sprop(mat,2)
    thic = sprop(mat,3)

C== Write the shotcrete properties to .out

     write(lu3,'(2i6,3e13.4)') ll,mat,exr,pnuxr,thic

C== Read the side of the element to be shotcreted
C== 0= on top, 1=right side

    if(ishpos(ll).eq.0) then
    IN(1) = NODES(ll,13)
    IN(2) = NODES(ll,14)
    IN(3) = NODES(ll,15)
    IN(4) = NODES(ll,16)
    IN(5) = NODES(ll,17)
    IN(6) = NODES(ll,18)
    IN(7) = NODES(ll,19)
    IN(8) = NODES(ll,20)

    else if(ishpos(ll).eq.1) then
    IN(1) = NODES(ll,3)
    IN(2) = NODES(ll,4)
    IN(3) = NODES(ll,5)
    IN(4) = NODES(ll,11)
    IN(5) = NODES(ll,17)
    IN(6) = NODES(ll,16)
    IN(7) = NODES(ll,15)
    IN(8) = NODES(ll,10)

    else if(ishpos(ll).eq.2) then
    IN(1) = NODES(ll,1)
    IN(2) = NODES(ll,2)
    IN(3) = NODES(ll,3)
    IN(4) = NODES(ll,10)
    IN(5) = NODES(ll,15)
    IN(6) = NODES(ll,14)
    IN(7) = NODES(ll,13)
    IN(8) = NODES(ll,9)

    else
    write(lu0,'(/a)') ' Error assigning shotcrete position '
    stop

Appendix B
endif

c== Set shotcrete position matrix.
i=0
DO 20 I = 1,8
DO 20 J = 1,6
ii=ii+1
20 is(ii) = (6*in(i)) - (6-j)

C==== SET UP ELEMENT STIFFNESS MATRIX
CALL STIF8

c write(1u3,'(a,i5)') ' shotcrete stiffness matrix for ',i1
write(1u3,'(6e13.4)') ((es(i,j),j=1,14),i=1,14)

DO 15 I = 1,14
II = is(I)
i = nf(ii)
if(ii.eq.-1) goto 15
DO 16 J = 1,14
JJ = is(J)
ja = nf(jj)
if(jj.eq.-1) goto 16
KK = ii - ja
IF(KK.LT.0) GO TO 16
ips = kdiag(iii) - kk
AM(ips) = AM(ips) + ES(I,J)
16 continue
15 CONTINUE
100 continue

WRITE(LUC,'(/A)') ' Stiffness matrix set up'

return
end

C=============================================================================
==
C Subroutine to set up shape functions. 8 noded element
C=============================================================================
==
SUBROUTINE PRELIM
IMPLICIT REAL*8 (A-H,O-Z)

COMMON /SHAPEF/ H(4,8), P(4,2,8)
COMMON /UNITS/ LUC, LUI, LDU, LU3, LUI4, LU6

DIMENSION RS(2,8), GS(2)

DATA RS/-1.,-1., 0.,-1., 1.,-1., 1.,0.,1., l., 1., 0., 1.,-1., 1.,-1., 1.,0./
DATA GS/ .5773503, .5773503 /

N = 0
DO 30 I = 1,2
DO 30 J = 1,2
R = GS(I)
S = GS(J)
N = N + 1
30 continue

DO 10 K = 1,7,2

Appendix B
R0 = RS(1,K)*R
S0 = RS(2,K)*S
H(N,K) = (1. + R0) * (1. + S0) *(R0 + R0 + S0 - 1.)/4.
P(N,1,K) = RS(1,K)* (1. + S0) *(R0 + R0 + S0)/4.
10 P(N,2,K) = RS(2,K)* (1. + R0) *(R0 + R0 + S0 + S0)/4.

DO 20 K = 2, 6, 4
S0 = RS(2,K)*S
H(N,K) = (1. - R*R) *(1. + S0)/2.
P(N,1,K) = -R*(1. + S0)
20 P(N,2,K) = RS(2,K)* (1. - R*R)/2.

DO 30 K = 4, 8, 4
R0 = RS(1,K)*R
H(N,K) = (1. + R0) *(1. - S*S)/2.
P(N,1,K) = RS(1,K)* (1. - S*S)/2.
30 P(N,2,K) = -S*(1. + R0)

RETURN
END

C-------------------------------------------------------------------------
C Subroutine to form D and ROTATION matrices. 8 noded element.
C-------------------------------------------------------------------------

SUBROUTINE DMATS
IMPLICIT REAL*8 (A-H,O-Z)

COMMON / LGDATA / COORD(9000,3), PROPS(50,10), PRES(200,3),
+ ELOAD(1700,96), NODES(1700,32),
+ MATNO(1700)
COMMON /SHAPE/ H(4,8), P(4,2,8)
COMMON /SOLV2/ NVARS, nen, in(8), sr(1700,8,4)
COMMON /PROPS / EXR, EYR, PNUXR, PNYR, thic
COMMON /ELST / ES(48,48), B(8,48), WEI(6), NVN
COMMON /BLOK2 / PJ(8), ST(48,48), D(8,8), DE(8,48), XY(2,8),
* AJAC(2,2), TR(3,3)
COMMON /UNITS/ LU0, LUI, LU2, LU3, LU4, LU6

YM = EXR
PR = PNUXR

D(1,1) = YM*THIC/(1. - PR*PR)
D(2,1) = D(1,1)*PR
D(1,2) = D(2,1)
D(2,2) = D(1,1)
D(3,3) = D(1,1) *(1. - PR)/2.
D(4,4) = D(1,1)*THIC*THIC/12.
D(5,4) = D(4,4)*PR
D(4,5) = D(5,4)
D(5,5) = D(4,4)
D(6,6) = D(4,4) * (1. - PR)/2.
D(7,7) = D(3,3)/1.2
D(8,8) = D(7,7)

N1 = iN(1)
N3 = iN(3)
N5 = iN(5)
TR(1,1) = coord(N3,1) - coord(N1,1)
TR(1,2) = coord(N3,2) - coord(N1,2)
\[
\begin{align*}
TR(1,3) &= \text{coord}(N3,3) - \text{coord}(N1,3) \\
TR(2,1) &= \text{coord}(N5,1) - \text{coord}(N1,1) \\
TR(2,2) &= \text{coord}(N5,2) - \text{coord}(N1,2) \\
TR(2,3) &= \text{coord}(N5,3) - \text{coord}(N1,3) \\
C &= \sqrt{TR(1,1)^2 + TR(1,2)^2 + TR(1,3)^2 - TR(1,1) - TR(1,2)^2 - TR(1,3) - TR(2,1)^2 + TR(2,2)^2 + TR(2,3) - TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2 - TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2 - TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2} \\
C &= \sqrt{TR(1,2)^2 + TR(1,3)^2 + TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2 - TR(1,2)^2 - TR(1,3) - TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2 - TR(1,2)^2 - TR(1,3) - TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2} \\
C &= \sqrt{TR(1,3)^2 + TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2 - TR(1,3)^2 - TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2 - TR(1,3)^2 - TR(2,1)^2 + TR(2,2)^2 + TR(2,3)^2} \\
\end{align*}
\]

DO 25 I = 1, 8
JT = I
DO 25 J = 1, 2
C=TR(J,1)*\text{coord}(JT,1)+TR(J,2)*\text{coord}(JT,2)+TR(J,3)*\text{coord}(JT,3)
XY(J,I) = C
25 CONTINUE

RETURN
END

---

Subroutine to form the B matrix.
---

subroutine bmat8(n,det)
implicit real*8 (a-h,o-z)

COMMON /SHAPEF/ H(4,8), P(4,2,8)
COMMON /PROPS / EXR, EYR, PNUXR, PNUYR, thic
COMMON /ELST / ES(48,48), B(8,48), COD(6), WEL(6), NVN
COMMON /BLOK2/ FJ(8), ST(48,48), D(8,8), DB(8,48), XY(2,8),
                     AJAC(2,2), TR(3,3)
COMMON /UNITS/ L00, L01, L02, L03, L04, l06

C==== Form Jacobian = derivatives of shape functions * coords
DO 30 I = 1, 2
DO 30 J = 1, 2
SUM = 0.
DO 20 K = 1, 8
20 SUM = SUM + P(N,I,K)*XY(J,K)
30 AJAC(I,J) = SUM

DET = AJAC(1,1)*AJAC(2,2) - AJAC(2,1)*AJAC(1,2)

DO 40 K = 1, 8
C==== Cartesian derivatives
FX = ( AJAC(2,2)*P(N,1,K) - AJAC(1,2)*P(N,2,K) )/DET
FY = ( -AJAC(2,1)*P(N,1,K) + AJAC(1,1)*P(N,2,K) )/DET

Appendix B
DO 35 L = 1, 3
L1 = (K-1)*6 + L
L4 = L1 + 3
B(1,L1) = Fx*TR(1,L)
B(2,L1) = Fy*TR(2,L)
B(3,L1) = Fx*TR(2,L) + Fy*TR(1,L)
B(7,L1) = -Fx*TR(3,L)
B(8,L1) = -Fy*TR(3,L)
B(4,L4) = -Fx*TR(2,L)
B(5,L4) = Fy*TR(1,L)
B(6,L4) = Fx*TR(1,L) - Fy*TR(2,L)
B(7,L4) = -H(N,K)*TR(2,L)
35 B(8,L4) = H(N,K)*TR(1,L)
40 PJ(K) = PJ(K) + H(N,K) * DET

return
end

C===============================================================================
==
C Subroutine to form the product DB at a gauss point. 8 noded element.
C===============================================================================
==
SUBROUTINE DxB
IMPLICIT REAL*8 (A-H,O-Z)

COMMON /ELST/ ES(48,48),B(8,48),COD(6),WEI(6),NVN
COMMON /BLOK2/ PJ(8), ST(48,48), D(8,8), DB(8,48), XY(2,8),
              AJAC(2,2), TR(3,3)
COMMON /UNITS/ LU0, LUI, LUZ, LU3, LUI4, LU6

DO 60 I = 1, 8
DO 60 J = 1, 48
     SUM = 0.
DO 50 K = 1, 8
50    SUM = SUM + D(I,K)*B(K,J)
60    DB(I,J) = SUM

RETURN
END

C===============================================================================
==
C Subroutine to form element stiffness matrix. 8 noded element.
C===============================================================================
==
SUBROUTINE STIF8
IMPLICIT REAL*8 (A-H,O-Z)

COMMON /SHAPEF/ R(4,8), P(4,2,8)
COMMON /PROPS/ EXR, EYR, PNUXR, PNUYR, thic
COMMON /ELST/ ES(48,48), B(8,48), COD(6), WEI(6), NVN
COMMON /BLOK2/ PJ(8), ST(48,48), D(8,8), DB(8,48), XY(2,8),
              AJAC(2,2), TR(3,3)
COMMON /UNITS/ LU0, LUI, LUZ, LU3, LUI4, LU6
COMMON /SOLV2/ NVARS, nen, in(8), sr(1700,8,4)

DO 10 I = 1, 8
10    PJ(I) = 0.
DO 11 I = 1, 48

Appendix B
DO 11 J = 1, 48
11 ES(I,J) = 0.0
   DO 12 I = 1, 8
      DO 12 J = 1, 48
   12 B(I,J) = 0.
      do 13 i=1,8
      do 13 j=1,8
13 d(i,j) = 0.

CALL DMATS

c==== Go through Gauss points and integrate stiffness matrix
N = 0
   DO 80 LX = 1, 2
      DO 80 LY = 1, 2
         N = N + 1
   80 call bmat8(n, det)
   c    write(lu3,'(a,2i5)') ' det in shotcr, ll, igno ',ll,n
   c    write(lu3,'(9e15.7)') ((b(k,i),k-1,8),i-1,48)
   c    write(lu3,'(e15.7)') det
      CALL DxB

   DO 80 I = 1, 48
      DO 80 J = 1, 48
         SUM = 0.
            DO 70 K = 1, 8
               70 SUM = SUM + B(K,I)*DB(K,J)
   80 ES(i,j) = ES(i,j) + SUM*DET

RETURN
END

Appendix B
C=============
C PROGRAM GEN3D
C--------------
C A program, which generates a three dimensional block mesh from
C either answers to screen prompts, or an input file. This is
C then formatted into an input file for the 3D finite element
C program TD1.for - initial stresses are also calculated.
C Modified from genjnt.for - 3/97
C=================================

CHARACTER*20 INFILE, IOUT
CHARACTER*80 TITLE
CHARACTER*3 ELEM, ISTRESS, STYPE, IPRESS, IJOINT

COMMON /ALPH / ELEM, IJOINT
COMMON /ISO1 / NXC, XCOORD(100), YCOORD(100), zcoord(100),
+    NBOT(100), NTOP(100), nzr, nyc
COMMON /JJJ / NCEL, NODE(100), NUMS(350,6), SJNT(350,6)
COMMON /ONE / NODES(5000,20), STR0(5000,3,8)
COMMON /TWO / X(50000), Y(50000), z(50000), POS0(5000), NUMEL,
+    numnod, nep, nea, nez, matno(5000)
COMMON /THREE/ ISTRESS, STYPE, IPRESS
COMMON /FOUR / NPL(30), GMA(30), OK(30), OKL(30), YSTOR(30),
+    NC0ORD(30,50), NUMLAY, ZWATER, GWATER
common /gauss / posgp(3), weigp(3), ngaus, onit(20,5)
COMMON /UNITS/ LU, LU0, LU1, LU2, LU3, LU4

DIMENSION IUX(3000), iuy(3000), IV(3000), notemp(5000), props(20,5)
dimension face(5000,2), point(1500,4), nodno(100), istre(100)
dimension iexc(50,50), nee(50), ztcord(100), ytcord(100)

C==== Set maximum array sizes...Note: Arrays are set for GENFE and
GENPLT.
MAXJNT = 350
MAXEL = 5000
maxex = 50
MAXNOD = 50000
MAXLDS = 50
MAXBCS = 3000
MAXCE = 100
MAXZ = 100
MAXY = 100
MAXLAY = 20

C==== Set logical unit numbers
C L00 = Visual display unit
C L01 = Input file
C L02 = Output file
C L03 = File containing data for checking.
C L04 = Echo data input file.
LU0 = 0
LU1 = 5
LU2 = 6
LU3 = 7
LU4 = 8

C==== Set variables for 20 noded elements.
ngpts = 8
nen = 20

Appendix B
numlod = 1  
ndofn = 6  
ncodi = 3  
numprop = 6  
nstres = 6  

WRITE(LU0,'(A)') ' Is input from screen=1, or from file=2 ? '  
READ(LU0,'*') INPT  
IF(INPT.EQ.1) THEN  
  LU=LU0  
  OPEN(LU4,FILE='GENR8.DAT',STATUS='UNKNOWN')  
ELSE  
  LU=LU1  
ENDIF  

C==== Open input-output files  
IF(INPT.EQ.2) THEN  
  WRITE(LU0,'(A)') ' What is the name of your input file ? '  
  READ(LU0,101) INFIL  
  FORMAT(A)  
  OPEN(LU1,FILE=INFIL,STATUS='UNKNOWN')  
ENDIF  

WRITE(LU0,'(A)') ' What is the name of your output file ? '  
READ(LU0,101) IOUT  
OPEN(LU2,FILE=IOUT,STATUS='UNKNOWN')  

C  
OPEN(LU3,FILE='GENR8.CHK',STATUS='NEW')  

IF(LU.EQ.LU0) WRITE(LU,'(a/)')  
* ' Give a one line title... a maximum of 80 characters.'  
READ(LU,101) TITLE  
IF(LU.EQ.LU0) WRITE(LU4,101) TITLE  

IF(LU.EQ.LU0) WRITE(LU,'(a/)')  
* ' Do you wish to have the (xypl)ane, or (xzpl)ane horozontal ?  
  read(lu,101) ipla  
  if(ipla.eq.'XZPL'.or.ipla.eq.'xzpl') then  
    iaxis = 1  
  else  
    iaxis = 0  
  endif  

IF(LU.EQ.LU0) WRITE(LU,'(a/)')  
* ' What number would you like the first node to be. ? '  
  read(lu,*) ilnod  

IF(LU.EQ.LU0) WRITE(LU,'(a/)')  
* ' What number would you like the first element to be. ? '  
  read(lu,*) ilel  

C==== Read NZR = No. of nodal rows (Z direction)  
C==== NXC = No. of nodal columns wide (X direction)  
C==== NYC = No. of nodal columns deep (Y direction)  
IF(LU.EQ.LU0) WRITE(LU,'(a/)')  
* ' How many rows of nodes? (Z direction) '  
  READ(LU,* ) NTRZR  
IF(NTRZR.GT.MAXZR) THEN  
  WRITE(LU0,'(A,I4)')  
  * ' Number of nodal rows exceeds maximum allowable of : ',MAXZR
STOP
ENDIF
IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' How many columns of nodes wide is the mesh? (X direction) '
READ(LU,*) NXC
IF(NXC.GT.MAXXC) THEN
    WRITE(LU0,'(/a,i4/)')
    * ' Number of nodal columns in the X direction exceeds maximum
    * allowable of : ',MAXXC
    STOP
ENDIF
IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' How many columns of nodes (Y direction) ? '
READ(LU,*) NTYC
if (ntyc.gt.maxyc) then
    write(lu0,'(/a,i4/)')
    * ' Number of nodal columns in the Y direction exceeds maximum
    * allowable of : ',maxyc
    stop
endif

C==== Read coordinates for base nodes

IF(LU.EQ.LU0) WRITE(LU,'(/a/)') ' Give Z-coords for each row.'
READ(LU,*)(ZTCORD(I),I=1,NTZR)
IF(LU.EQ.LU0) WRITE(LU4,200)(ZTCORD(I),I=1,NTZR)
IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' Give x-coordinate of nodal rows. '
READ(LU,*)(XCOORD(I),I=1,NXC)
IF(LU.EQ.LU0) WRITE(LU4,200)(XCOORD(I),I=1,NXC)
if(lu.eq.lu0) write(lu,'(/a/)')
* ' Give y-coordinate of nodal rows. '
read(lu,*)(YCOORD(i),i=1,NTYC)
if(lu.eq.lu0) write(lu4,200) (ycoord(i),i=1,ntyc)

if(iaxis.eq.1) then
    nzr = ntyc
    nyc = ntzr
    do 455 i = 1,ntyc
        zcoord(i) = ytcord(i)
    455 continue
    do 465 i = 1,ntzr
        ycoord(i) = -ztcord(i)
    465 continue
else
    nzr = ntzr
    nyc = ntyc
    do 475 i=1,ntyc
        ycoord(i) = ytcord(i)
    475 continue
    do 485 i=1,ntzr
        zcoord(i) = ztcord(i)
    485 continue
endif

write(lu2,'(/a/)') ' made it to 1. '
WRITE(LU2,'(/a/)') ' zcoord '
WRITE(LU2,200)(ZCOORD(I),I=1,NZR)
WRITE(LU2,'(/a/)') ' ycoord '
WRITE(LU2,200)(YCOORD(I),I=1,NYC)
WRITE(LU2,'(/a/)') ' xcoord '

Appendix B
WRITE(LU2,200) (XCOORD(I),I=1,NXC)

C==== Calculate the required variables.

nil = (2*nzr) - 2
nci = (3*nzr) - 1
nlbn = (nci*(nxc-1)) + 1
nyp = ((4*nzr*nxc) - nxc) - nzr
numnod = (nyc*nyp) - (nzr*nxc)
if(numnod.gt.maxnod) then
   write(lu0,'(/a,i6,a/a,i5/)
* ' This mesh has ',numnod,' nodes',
* ' This exceeds the maximum of ',maxnod
   stop
endif
lnn = numnod - nil
nfin = (((3*nzr*nxc)-nxc)-nzr)+1
lnl = (numnod - nyp) + 1
lnbp = nyp - (nzr - 1)

r = 1

C==== Set up nodal coordinates of nodes along main columns.

do 15 i=1,nnn,nyp
   m = 0
   n = n+1
   do 25 j=i,nnn,nci
      m = m+1
      x(j) = xcoord(m)
      z(j) = zcoord(1)
      y(j) - ycoord(n)
      s = j + 2
      nm = j + nil
      do 105 k=s,nm,2
         r = r+1
         x(k) = xcoord(m)
         y(k) = ycoord(n)
         z(k) = zcoord(r)
      105 continue
   25 continue
   nlbn = nlbn + nyp
   15 continue

C==== Calculate the z coords for the intermitent nodes

n = 0
nlbn = (nci*(nxc-1)) + 1
a = 0
b = 0
do 115 i = 1,nnn,nyp
   m = 0
   n - n + 1
   do 125 j = i,nnb,nci
      m = m + 1
      s = j + 1
      nm = j + nil
      do 135 k = s,nm,2
         x(k) = xcoord(m)

Appendix B
y(k) = ycoord(n)
a = a + 1
b = a + 1
z1 = zcoord(a)
z2 = zcoord(b)
ztemp = ((z2-z1)/2) + z1
z(k) = ztemp

135 continue
a = 0
b = 0
125 continue
nlbn = nlbn + nyp
115 continue

C==== Set up coordinates of nodes in-between y columns

r = 1
m = 0
do 35 i=nfin,nlin,nyp
  a = a + 1
  b = a + 1
  y1 = ycoord(a)
y2 = ycoord(b)
ytemp = ((y2-y1)/2) + y1
do 45 j = i,lbnp,nzr
  m = m + 1
  x(j) = xcoord(m)
z(j) = zcoord(l)
y(j) = ytemp
  s = j + 1
  nm = j + (nzr-1)
do 55 k = s,nm
  r = r + 1
  x(k) = xcoord(m)
z(k) = zcoord(r)
y(k) = ytemp
55 continue
r = 1
45 continue
lbnp = lbnp + nyp
m = 0
35 continue

C==== Write coordinates to intermitent x columns.

C==== First calculate the variables required.

nfbn = 2*nzr
nbn = nummod - (nci-1)
nlbin = (((3*nzr*nxc)-nxc)-nzr) - (nci-1)
r = 1
n = 0

do 75 i = nfbn,nbn,nyp
  m = 0
  n = n+1
  aa = 0
  bb = 0
do 85 j = i,nlbin,nci
  aa = aa + 1
75 continue
85 continue

Appendix B
\[ bb = aa + 1 \]
\[ x1 = xcoord(aa) \]
\[ x2 = xcoord(bb) \]
\[ xtemp = ((x2-x1)/2 + x1 \]
\[ x(j) = xtemp \]
\[ y(j) = ycoord(n) \]
\[ z(j) = zcoord(l) \]
\[ s = j + 1 \]
\[ nm = j + (nzn - 1) \]
\[ do 95 k = s,nm \]
\[ r = r + 1 \]
\[ x(k) = xtemp \]
\[ y(k) = ycoord(n) \]
\[ z(k) = zcoord(r) \]
\[ 95 continue \]
\[ r = 1 \]
\[ 85 continue \]
\[ nlbin = nlbin + nyp \]
\[ 75 continue \]
\[
\begin{align*}
\text{c & write(lu2,'(/a)') ' node no.  x  y  z '}
c & do 65 i=1,nummod 
\text{c & write(lu2,'(i4,3e15.7)') i, x(i), y(i), z(i) } 
c & 65 continue
\end{align*}
\]

\[ C===== Calculate which nodes constitute the elements. \]
\[ C===== First consider the first column of elements \]
\[ n = 1 \]
\[ nm = 0 \]
\[ nbnte = (2*nzn) - 3 \]
\[ nap = (((3*nzn*nxc)-nxc)-nzn) \]
\[ numel = (nzn-1)*(nxc-1)*(nyc-1) \]
\[ if(numel.gt.maxel) then 
\text{write(lu0,'(/a,i5/)') '}
\text{ ' The number of elements exceeds the maximum of, ', maxel }
\text{ stop endif } 
\]
\[ do 145 i = 1,nbnte,2 \]
\[ \text{nodes(n,1) = i } \]
\[ \text{nodes(n,9) = i + 1 } \]
\[ \text{nodes(n,13) = i + 2 } \]
\[ \text{ij = i + nyp } \]
\[ \text{nodes(n,7) = ij } \]
\[ \text{nodes(n,12) = ij + 1 } \]
\[ \text{nodes(n,19) = ij + 2 } \]
\[ k = i + ((2*nzn)-1) - nm \]
\[ \text{nodes(n,2) = k } \]
\[ \text{nodes(n,14) = k + 1 } \]
\[ kj = k + nyp \]
\[ \text{nodes(n,6) = kj } \]
\[ \text{nodes(n,18) = kj + 1 } \]
\[ l = i + ((3*nzn) - 1) \]
\[ \text{nodes(n,3) = l } \]
\[ \text{nodes(n,10) = l + 1 } \]
\[ \text{nodes(n,15) = l + 2 } \]
\[ lj = l + nyp \]
\[ \text{nodes(n,5) = lj } \]
nodes(n,11) = lj + 1
nodes(n,17) = lj + 2

m = i + nap - nm
nodes(n,8) = m
nodes(n,20) = m + 1
mj = m + nzr
nodes(n,4) = mj
nodes(n,16) = mj + 1

nm = nm + 1
n = n + 1
145 continue

C==== Now to calculate the nodes for the elements across (x-dir.)
C==== the mesh.

nlbin = (((3*nzr*nxc)-nxc)-nzr) - (nci-1)
nfic = 3*nzr
nliec = nlbin - ((2*nzr) - 1)
ze = nzr - 1
 nbnte = nbnte + nci

do 155 i = nfiec,nliec,nci
do 165 j = i,nbnte,2
 nr = n - nez
 do 175 k = 1,20
  nxi = nodes(nr,k)
  if (k.eq.8 .or. k.eq.4 .or. k.eq.16 .or. k.eq.20) then
    nei = nzr
  else
    nei = nci
  endif
  nxi = nxi + nei
  nodes(n,k) = nxi
175 continue
 n = n + 1
165 continue
 nbnte = nbnte + nci
155 continue

C==== Now to calculate the element nodes for the elements
C==== lying in the z-dir.

nyp1 = nyp + 1
nfnle = numnod - (((7*nzr*nxc)-(2*nxc)-(2*nzr)) - 1 )
nep = (nzr - 1)*(nxc - 1)

do 185 i = nyp1,nfnle,nyp
do 195 j = 1,nep
 nr = n - nep
 do 205 k = 1,20
  nxi = nodes(nr,k)
  nxi = nxi + nyp
  nodes(n,k) = nxi
205 continue
 n = n + 1
195 continue
185 continue

cwrite(1u2,'(/a)') ' elements and nodal points '

Appendix B
C==== Read some of the required input parameters.

    IF(LU.EQ.LU0) WRITE(LU,'(/a,/a)')
    + ' Give the number of load increments (ie: excavation steps),
    ',
    + ' iterations, and convergence. '
    read(lu,*) mm1, maxit, conv
    if(mm1.gt.maxlds) then
    write(lu,'(/a,/a,i4)') ' The number of load increments must ',
    * ' be less than ', maxlds
    stop
    endif

C==== Read the material layer information

    IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
    + ' Enter number of material layers. '
    READ(LU,*) NUMLAY
    IF(NUMLAY.GT.MAXLAY) THEN
    WRITE(LU0,'(/a,i3/)')
    * ' Number of materials exceeds maximum allowable of : ', MAXLAY
    STOP
    ENDIF
    IF(LU.EQ.LU0) WRITE(LU4,100) NUMLAY
    IF(NUMLAY.LT.1) THEN
    WRITE(LU0, '(/a/)') ' There must be at least 1 material
    layer'
    STOP
    ENDIF
    if(numlay.eq.1) then
    do 245 k=1,numel
    matno(k)=1
    245 continue
    if(lu.eq.lu0) write(lu,'(/a,i3/)')
    + ' Enter the: E , Poisson, phi, psi, Cu, for material... ',1
    read(lu,*) (props(1,iprop),iprop=1,5)
    go to 255
    endif
    DO 225 I=1,NUMLAY
    IF(LU.EQ.LU0) WRITE(LU,'(/a,i3/)')
    * ' Give the number of elements defining layer... ',I
    READ(LU,*) NPLi
    if(lu.eq.lu0) write(lu,'(/a/)')
    + ' Give the numbers of those elements. '
    read(lu,*) (notemp(j), j=1,npli)
    do 235 k=1,npli
    1 = notemp(k)
    matno(1) = i
    235 continue
    if(lu.eq.lu0) write(lu,'(/a,i3/)')
    + ' Enter the: E , Poisson, phi, psi, Cu, for material... ',1
    read(lu,*) (props(i,iprop),iprop=1,5)
    225 CONTINUE
    255 continue
    c    WRITE(LU2,*) (matno(k),k=1,numel)

C==== Set up initial stress state and material properties
    CALL INITSTR(NEN,NGPTS)

C==== Read loading data
IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' How many nodal point loads are there ?'
READ(LU,*1) npload
if(npload.eq.0) go to 295
do 285 iplod = 1,npload
IF(LU.EQ.LU0) WRITE(LU,'(/a,i3/)')
* ' Give the node no. & X, Y, Z loads for point load, ' , iplod
read(lu,*) (point(iplod,j),j=1,4)
285 continue
295 continue

IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' How many edge loads are there ?'
read(lu,*) nedge

IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' How many loaded faces are there ?'
read(lu,*) nface
if(nface.eq.0) go to 305
do 315 iface = 1,nface
IF(LU.EQ.LU0) WRITE(LU,'(/a,a,i3/)')
* ' Give the element number and uniform pressure for ',
* ' face load, ' , iface
read(lu,*) (face(iface,j),j=1,2)
315 continue
305 continue

IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' Do you wish to use (gravity or (nogravity ?'
read(lu,101) iweeb
if(iweeb.eq.'NOGR'.or.iweeb.eq.'nogr') go to 325
igrav = 1
IF(LU.EQ.LU0) WRITE(LU,'(/a,a/)')
* ' Give the angles gravity makes with the -Z axis relative to ',
* ' the XZ plane and the YZ plane; theta X, & theta Y. '
read(lu,)(2d15.7)) thetax,thetay
go to 445
325 continue
igrav = 0
445 continue

C==== Read boundary condition data

IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' Give the total number of restrained nodes. '
read(lu,*) nvfix
if(nvfix.eq.0) go to 275
C==== Test for boundary condition format.

IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
* ' Enter the boundary conditions (pern)ode or by (plan)e ' 
read(lu,101) ibocon
if(ibocon.eq.'pern'.or.ibocon.eq.'PERN') go to 326
IF(LU.EQ.LU0) WRITE(LU,'(/a,a/)')
* ' give the no. of planes along which you would like all ',
* ' nodes fixed in the x direction - write -1 for no fixed. ' 
read(lu,*) nxbc

Appendix B
if(nxbc.eq.-1) goto 329

nux = 0
do 331 j=1,nxbc
   IF(LU.EQ.LU0) WRITE(LU,'(/a,14,/a/)')
   * ' Give the x coord of plane ',j,
   * ' on which all nodes will be fixed in the x direction. '
   read(lu,*).xIBC
   do 328 i=1,numnod
      if(x(i).ne.xIBC) goto 328
      nux = nux + 1
   IF(NUX.GT.MAXBCS) THEN
      WRITE(LU0,'(/A,13/)')
      *' Number of x direction restraints exceeds allowable maximum
   of: '
   * ,MAXBCS
   STOP
   ENDiF
328 continue
331 continue
329 continue
   IF(LU.EQ.LU0) WRITE(LU,'(/a/a/)')
   *' give the no. of planes along which you would like all ',
   *' nodes fixed in the y direction - write -1 for no fixed. '
   read(lu,*).nyBC
   if(nyBC.eq.-1) goto 332
   nuy = 0
do 333 j=1,nyBC
   IF(LU.EQ.LU0) WRITE(LU,'(/a,14,/a/)')
   *' Give the y coord of plane ',j,
   *' on which all nodes will be fixed in the y direction. '
   read(lu,*).yBC
   do 334 i=1,numnod
      if(y(i).ne.yBC) goto 334
      nuy = nuy + 1
   IF(NUY.GT.MAXBCS) THEN
      WRITE(LU0,'(/A,13/)')
      *' Number of y direction restraints exceeds allowable maximum
   of: '
   * ,MAXBCS
   STOP
   ENDiF
334 continue
333 continue
332 continue
   IF(LU.EQ.LU0) WRITE(LU,'(/a/a/)')
   *' give the no. of planes along which you would like all ',
   *' nodes fixed in the z direction - write -1 for no fixed. '
   read(lu,*).nzBC
   if(nzBC.eq.-1) goto 336
   nv = 0
do 337 j=1,nzBC
   IF(LU.EQ.LU0) WRITE(LU,'(/a,14,/a/)')
   *' Give the z coord of plane ',j,
   *' on which all nodes will be fixed in the z direction. '
337 continue
336 continue
335 continue

Appendix B
read(lu,*) zbc
   do 338 i=1,numnod
      if(z(i).ne.zbc) goto 338
      nv = nv + 1
   if(nv.gt.maxbcs) then
      write(luo,'(/a,13)')
      ' Number of vertical restraints exceeds allowable maximum of:
      ','maxbcs
      stop
   endif
   iv(nv) = i
338 continue
336 continue
goto 327
326 continue
   if(lu.eq.luo) write(lu,'(/a/)')
   ' Give no. of nodes fixed in the x direction.'
   read(lu,*) nux
   if(nux.gt.maxbcs) then
      write(luo,'(/a,13)')
      ' Number of x direction restraints exceeds allowable maximum
   of:
   ','maxbcs
   stop
   endif
   if(nux.gt.0) then
      if(lu.eq.luo) write(lu,'(/a/)')
      ' Give the numbers of these nodes.'
      read(lu,*) (iu(x(i)),i=1,nux)
   endif
   if(lu.eq.luo) write(lu,'(/a/)')
   ' Give no. of nodes fixed in the y direction.'
   read(lu,*) nuy
   if(nuy.gt.maxbcs) then
      write(luo,'(/a,13)')
      ' Number of y direction restraints exceeds allowable maximum
   of:
   ','maxbcs
   stop
   endif
   if(nuy.gt.0) then
      if(lu.eq.luo) write(lu,'(/a/)')
      ' Give the numbers of these nodes.'
      read(lu,*) (iu(y(i)),i=1,nuy)
   endif
   if(lu.eq.luo) write(lu,'(/a/)')
   ' Give no. of nodes fixed vertically, (z dir.).'
   read(lu,*) nv
   if(nv.gt.maxbcs) then
      write(luo,'(/a,13)')
      ' Number of vertical restraints exceeds allowable maximum of:
   ','maxbcs
   stop
   endif
   if(nv.gt.0) then

Appendix B
IF(LU.EQ.LU0) WRITE(LU,'(a)')
  * ' Give the numbers of these nodes.'
  READ(LU,*) (IV(I),I=1,NV)
ENDIF

275 continue
327 continue

C==== Find result summary information.

IF(LU.EQ.LU0) WRITE(LU,'(a)')
  * ' Give no. of nodes at which output is required. '
  read(lu,*) nod
IF(LU.EQ.LU0) WRITE(LU,'(a)')
  * ' Give the node numbers of these nodes. '
  read(lu,*) (nodno(i),i=1,nod)
IF(LU.EQ.LU0) WRITE(LU,'(a)')
  * ' Give the number of elements at which stress output is
    wanted.'
  read(lu,*) nostr
IF(LU.EQ.LU0) WRITE(LU,'(a)')
  * ' Give the element numbers of these elements. '
  read(lu,*) (istre(i),i=1,nostr)

C==== Find excavation information.

do 405 i=1,mnli
  if(lu.eq.lu0) write(lu,'(a,i4)')
  * ' How many elements are to be removed in step. ',i
  read(lu,100) nee(i)
  neen = nee(i)
  if(nee.gt.maxex) then
    write(lu0,'(a,/a,i5)') ' Number of excavated elements exceeds
    ',
    * ' the maximum allowable of ',maxex
    stop
  endif
  if(nee.eq.0) go to 425
  if(lu.eq.lu0) write(lu,'(a,i4)')
  * ' What are the numbers of the elements to be removed in step
    ',i
  read(lu,100) (iexc(i,j),j=1,nee)
405 continue
425 continue

C==== Write mesh to file
WRITE(LU2,101) TITLE
WRITE(LU2,'(2i4,e15.7)') mnli,maxit,conv
WRITE(LU2,100) numnod,numel,nvfix,numlod,nen,ndofn,numlay,
  * numprop,ngaus,ncodi,nstres
  do 335 i = 1,numnod
    i2 = i + ilnod - 1
    if(laisis.eq.1) then
      write(lu2,'(i6,3e15.7)') i2,x(i),z(i),y(i)
    else
      write(lu2,'(i6,3e15.7)') i2,x(i),y(i),z(i)
    endif
335 continue
  do 215 j=1,20
    nodes(i,j) = nodes(i,j) + (ilnod - 1)
215 continue

Appendix B
1215 continue
215 continue
   do 345 i = 1, numel
    i2 = i + iel1 - 1
    write(lu2,100) i2, (nodes(i,j), j=1, 20), matno(i)
345 continue
   write(lu2,100) nod
   write(lu2,100) (nodno(i), i=1, nod)
   write(lu2,100) nostr
   write(lu2,100) (istre(i), i=1, nostr)
    do 1216 i = 1, nux
    iux(i) = iux(i) + (i1nod - 1)
1216 continue
   write(lu2,100) nux
   write(lu2,100) (iux(i), i=1, nux)
    do 1217 i = 1, nuy
    iuy(i) = iuy(i) + (i2nod - 1)
1217 continue
   write(lu2,100) nuy
   write(lu2,100) (iuy(i), i=1, nuy)
    do 1218 i = 1, nv
    iv(i) = iv(i) + (i1nod - 1)
1218 continue
   write(lu2,100) nv
   write(lu2,100) (iv(i), i=1, nv)
    do 355 i = 1, numlay
   write(lu2,'(i4,6e15.7)') i, (props(i,iprop), iprop=1,5), onit(i,1)
355 continue
   write(lu2,100) nplod
    do 365 i = 1, nplod
   write(lu2,'(i5,3e15.7)') (point(i,j), j=1, 4)
365 continue
   write(lu2,100) ndedge
   write(lu2,100) nface
    do 375 i = 1, nface
   write(lu2,'(i5,6e15.7)') (face(i,j), j=1, 2)
375 continue
   write(lu2,100) igrav
    if (igrav.eq.0) go to 385
   write(lu2,'(2e15.7)') thetax, thetay
    do 395 i = 1, numlay
   write(lu2,'(i5,6e15.7)') i, onit(i,1)
395 continue
   do 385 i = 1, numel
    if (iaxis.eq.1) then
     do 495 k = 1, ngpts
    write(lu2,'(3e15.7)') str0(i,1,k), str0(i,3,k), str0(i,2,k)
495 continue
    else
     write(lu2,'(3e15.7)') ((str0(i,j,k), j=1, 3), k=1, ngpts)
end if
435 continue
   do 415 i = 1, mnull
    write(lu2,100) nee(i)
    neen = nee(i)
    write(lu2,100) (iexec(i,j), j=1, neen)
415 continue
100 FORMAT(22I6)
200 FORMAT(4E15.7)

Appendix B
300 FORMAT(2I4,E15.7)
303 FORMAT(3I4,E15.7)
C==== Close files
     CLOSE(LU1)
     CLOSE(LU2)
     C
     CLOSE(LU3)
     END

C==================================
==
C Routine to generate initial stresses and material types for
general
C shaped regions which may be defined by a series of straight lines.
C==================================
==
SUBROUTINE INITSTR(NEN,NGPTS)
    CHARACTER*3 ISTRESS,STYPE,IPRESS,ELEMT,JIJOIN
    COMMON /ALPH / ELEMT,JIJOIN
    COMMON /JJJ / NJEL,NODE(100),NUMS(350,6),SJNT(350,6)
    COMMON /ONE / NODES(5000,20),STR0(5000,3,8)
    COMMON /TWO / X(50000),Y(50000),Z(50000),PB0(50000),NUMEL,
                  numnod,nnem,nez,matno(5000)
    COMMON /THREE / ISTRESS,STYPE,IPRESS
    COMMON /FOUR / NPL(30),GMA(30),OK(30),OKL(30),YSTOR(30),
                  NCOORD(30,50),NUMLAY,ZWATER,GWATER
    COMMON /UNITS / L0,L00,L1,L11,L2,L3,L4
    COMMON /SHF / SHAPE(20),DERIV(3,20),ELC0D(3,20),CARTD(3,20)
    common /gauss / posgp(3),weigp(3),ngaus,oinit(20,5)
    DIMENSION COJ(3),XT(8),YT(8),ZT(8),CYT(5000)

COJ(1)=.774596 666
COJ(2)=.0
COJ(3)=.774596 666

C==== Determine if initial stress set up is required,
C==== Read STRESS or NOSTRESS
    IF(LU.EQ.LU0) WRITE(LU,'(/A/)') ' (STRESS)ess or (NOSTRESS) set
    READ(LU,101) ISTRESS
    IF(LU.EQ.LU0) WRITE(LU4,101) ISTRESS
    IF(ISTRESS.EQ.'NOS'.OR.ISTRESS.EQ.'nos') RETURN

C==== Set up the gauss point coordinates for the elements.
    if(lu.eq.lu0) write(lu,'(/A/)') ' What order of Gaussian int. '
    read(lu,'(i3)') ngaus
    call gaussq

C==== Set up material layers by reading surface profiles of layers.
    101 FORMAT(A)
    100 FORMAT(2I4)
    200 FORMAT(4E15.7)
    STYPE='NUL'
    IPRESS='NOP'
C==== Read EFF or TOT to choose effective or total stress set up.
    IF(LU.EQ.LU0) WRITE(LU,'(/A/)') ' (EFF)ective or (TOT)al stress

Appendix B
READ(LU,101) STYPE
IF(LU.EQ.LU0) WRITE(LU4,101) STYPE
C==== Read PORE or NOPORE to select initial pore pressure.
IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
   * ' Initial pore (PRE)s sure or (NOP)ressure ?'
READ(LU,101) IPRESS
IF(LU.EQ.LU0) WRITE(LU4,101) IPRESS
C==== Read unit weight, Kx, Ky for each layer
   do 106 i = 1,nulay
      IF(LU.EQ.LU0) WRITE(LU,'(/a,I3/)')
      * ' Give unit wt., Kx, kly, Ky, kly, for layer... ',i
      READ(LU,*)(onit(i,j),j=1,5)
      IF(LU.EQ.LU0) WRITE(LU4,200) (GMA(I),OK(I),OKL(I),I=1,NULAY)
   c   106 continue
   IF(LU.EQ.LU0) WRITE(LU,'(/a/)')
      * ' Give coord. of water level, unit wt. water.'
      READ(LU,*),ZWATER,GWATER
      IF(LU.EQ.LU0) WRITE(LU4,200) ZWATER,GWATER
   C   WRITE(LU3,61)
   61 FORMAT(//' INITIAL STRESS DATA'//-------------------//')
   C   * Layer unit weight'7X'Ko'12X'Kl'/IX,48(LH-)
   C   WRITE(LU3,87) (I,GMA(I),OK(I),OKL(I),I=1,NULAY)
   87 FORMAT(16,3E14.4)
   C   WRITE(LU3,62) ZWATER,GWATER
   62 FORMAT(//' Y-coordinate of water table =',E13.4/
   C   * ' Unit weight of water =',E13.4)
C==== Calculate the overburden from each element.
   nea = nez - 1
   nfelp = numel - nea
   do 105 i=1,nea
      n=i
   c   do 102 j=1,nfelp,nez
d   do 104 ij=1,nea
      nte = n + ij
      mat = matno(nte)
      nb = nodes(nte,1)
      nt = nodes(nte,13)
      zdif = z(nt) - z(nb)
      gmal = onit(mat,1)
      ovb = zdif*gmal
      ovt(n) = ovt(n) + ovb
   c   104 continue
   n = n + nez
   102 continue
   nea = nea - 1
   105 continue
   write(lu2,'(/8el5.7/)') (ovt(i),i=1,numel)
   DO 85 LL=1,NUMEL
   85 N1=NODES(LL,1)
   N2=NODES(LL,2)
   N3=NODES(LL,3)
   XT(1)=(X(N1)+X(N2)+X(N3))/3.
   YT(1)=(Y(N1)+Y(N2)+Y(N3))/3.
   ZT(1)=(Z(N1)+Z(N2)+Z(N3))/3.
   DO 5 K=1,20
   ND=NODES(LL,K)
ELCOD(1,K)=X(ND)
ELCOD(2,K)=Y(ND)
elcod(3,k)=z(nd)
NGP=0
DO 10 L=1,ngaus
S=posgp(L)
DO 10 M=1,ngaus
T=posgp(M)
do 10 n=1,ngaus
v=posgp(n)
NGP=NGP+1
c write(lu2,'(a)') ' s,t,v ' c write(lu2,'(3e15.7)') s,t,v CALL SF3r2(S,T,v)
XG=0.0
YG=0.0
zg=0.0
DO 20 K=1,NEN
zg=zg+shape(k)*elcod(3,k)
YG=YG+SHAPE(K)*ELCOD(2,K)
20 XG=XG+SHAPE(K)*ELCOD(1,K)
z(t) = zg
XT(NGP) = XG
10 YT(NGP) = YG
DO 888 NGP=1,NGPTS
XBAR=XT(NGP)
YBAR=YT(NGP)
zbar = zt(ngp)
WPRESS=0.0
IF(STYPE.EQ.'EFF'.OR.STYPE.EQ.'eff') THEN
  WPRESS = (ZWATER-ZBAR)*GWATER
  IF(WPRESS.LT.0.0) WPRESS=0.0
ENDIF

C==== Calculate the extra total stress due to water above ground.
  zmax = z(nummod)
dl = zwater - zmax
svert = 0.0
if(dl .gt. 0.0) svert = dl*gwaler

C==== Calculate the extra total stress due to the current element.
  mat = matno(ll)
gmal = onit(mat,1)
ntn = nodes(ll,13)
ztot = z(ntn)
ove = (ztot - zbar)*gmal
sts = cvt(ll) + ove
svert = svert + sts

88 CONTINUE
STRO(LL,3,NGP)=SVERT-WPRESS
STRO(LL,1,NGP)=onit(mat,2)*STRO(LL,3,NGP)+onit(mat,3)
STRO(LL,2,NGP)=onit(mat,4)*STRO(LL,3,NGP)+onit(mat,5)
888 CONTINUE
85 CONTINUE
c write(lu2,'(a)') ' made it to 3d ' c do 265 lj = 1,numel
c write(lu2,'(a)') ' str0(numel,-,1) ' 

Appendix B
write(lu2,'(3e15.7)') (str0(lj,ik,1), ik=1,3)
continue

DO 985 LJ=1,NJEL
C===> If a joint element, compute centroid differently.
   N1=NUMS(LJ,1)
   N2=NUMS(LJ,2)
   N3=NUMS(LJ,3)
   IF(N3.EQ.0) THEN
      DY=Y(N2)-Y(N1)
      XBAR=(X(N1)+X(N2))/2.
      YBAR=(Y(N1)+Y(N2))/2.
      YTJ=YBAR+COJ(2)*DY/2.
      YBJ=YBAR+COJ(1)*DY/2.
   ELSE
      DY=Y(N3)-Y(N1)
      XBAR=(X(N1)+X(N3))/2.
      YBAR=(Y(N1)+Y(N3))/2.
      YTJ=YBAR+COJ(3)*DY/2.
      YMJ=YBAR
      YBJ=YBAR+COJ(1)*DY/2.
   ENDIF

WPRESS=0.0
IF(STYPE.EQ.'EFF'.OR.STYPE.EQ.'eff') THEN
   WPRESS=(ZWATER-YBAR)*GWATER
   IF(WPRESS.LT.0.0) WPRESS=0.0
ENDIF

DO 937 I=1,NUMLAY
   K=NPL(I)-1
DO 936 J=1,K
   L1=NCORD(I,J)
   L2=NCORD(I,J+1)
   X1=X(L1)
   X2=X(L2)
   IF(XBAR.GE.X1.AND.XBAR.LT.X2) THEN
      GRAD=(Y(L2)-Y(L1))/(X2-X1)
      YSTOR(I)=Y(L1)+GRAD*(XBAR-X1)
   ENDIF
936 CONTINUE
937 CONTINUE

C===> Calculate extra total stress due to water above ground level
   D1=ZWATER-YSTOR(1)
   SVMID=0.0
   SVTOP=0.0
   SVBOT=0.0
   IF(D1.GT.0.0) SVERT=D1*GWATER

DO 989 K=1,NUMLAY
   LAYR=K
   STS=(YSTOR(K)-YSTOR(K+1))*GMA(K)
   IF(YBAR.GE.YSTOR(K+1)) THEN
      STOP=(YSTOR(K)-YTJ)*GMA(K)
      SMID=(YSTOR(K)-YMJ)*GMA(K)
      SBOT=(YSTOR(K)-YBJ)*GMA(K)
      SVTOP=SVTOP+STOP
      SVMID=SVMID+SMID
      SVBOT=SVBOT+SBOT
   GO TO 988

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ENDIF
SVTOP=SVTOP+STS
SVMID=SVMID+STS
989 SVBOT=SVBOT+STS
988 CONTINUE
   IF(DY.EQ.0.0) THEN
C==== Horizontal joint
   SJNT(LJ,1)=0.0
   SJNT(LJ,2)=SVBOT+OKL(LAYR)
   SJNT(LJ,3)=0.0
   SJNT(LJ,4)=SVMID+OKL(LAYR)
   SJNT(LJ,5)=0.0
   SJNT(LJ,6)=SVBOT+OKL(LAYR)
ELSE
C==== Vertical joint
   SJNT(LJ,1)=0.0
   SJNT(LJ,2)=OK(LAYR)*SVBOT+OKL(LAYR)
   SJNT(LJ,3)=0.0
   SJNT(LJ,4)=OK(LAYR)*SVMID+OKL(LAYR)
   IF(NEN.EQ.6) THEN
      SJNT(LJ,4)=OK(LAYR)*SVMID+OKL(LAYR)
      SJNT(LJ,5)=0.0
      SJNT(LJ,6)=OK(LAYR)*SVBOT+OKL(LAYR)
   ENDIF
ENDIF
985 CONTINUE
C==== Set up initial pore water pressures if required
   IF(IPRESS.EQ.'POR'.OR.IPRESS.EQ.'pore') THEN
      DO 170 I=1,NUMNOD
         DEP=ZWATER+Y(I)
         WPRESS=0.0
         IF(DEP.GE.0.0) WPRESS=DEP*GWATER
      170 PP0(I)=WPRESS
   ENDIF
RETURN
END
C  **********************************************************************
C  CALCULATES SHAPE FUNCTIONS AND THEIR DERIVATIVES
C  FOR THREE DIMENSIONAL QUADRATIC ELEMENTS
C  **********************************************************************

SUBROUTINE SF3R2(S,T,U)

DIMENSION CN(20,3),PROD(3),X(3)

common /shp / shape(20),deriv(3,20),elcod(3,20),cartd(3,20)

COMMON / UNITS / lu,L00,LU1,LU2,LU3,LU4

C   write(lu2,'(/a/)') ' entering sf3r2 '

C*** FIRST VALUES OF S AT NODES

   DATA CN / -1.,0.,+1.,+1.,+1.,0.,-1.,-1.,
   + -1.,+1.,+1.,-1.,
   + -1.,0.,+1.,+1.,+1.,0.,-1.,-1.,

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C*** NOW T VALUES AT NODES
+ -1.,-1.,-1.,0.,+1.,+1.,+1.,0.,
+ -1.,-1.,+1.,+1.,
+ -1.,-1.,-1.,0.,+1.,+1.,+1.,0.,

C*** NOW U VALUES AT NODES
+ -1.,-1.,-1.,-1.,-1.,-1.,-1.,-1.,
+ 0.,0.,0.,0.,
+ +1.,+1.,+1.,+1.,+1.,+1.,+1.,+1.,

X(1) = S
X(2) = T
X(3) = U
IN = 0

20 IN = IN + 1

C*** CORNER NODE FIRST

SS = S * CN(IN,1)
TT = T * CN(IN,2)
UU = U * CN(IN,3)
SSP = SS + 1.0DD0
TTP = TT + 1.0DD0
UUP = UU + 1.0DD0

SHAPE(IN) = 0.125DD0 * SSP * TTP * UUP *
+ (SS + TT + UU - 2.0DD0)
DERIV(1,IN) = 0.125DD0 * CN(IN,1) * TTP * UUP *
+ (2.0DD0 * SS + TT + UU - 1.0DD0)
DERIV(2,IN) = 0.125DD0 * CN(IN,2) * SSP * UUP *
+ (SS + 2.0DD0 * TT + UU - 1.0DD0)
DERIV(3,IN) = 0.125DD0 * CN(IN,3) * SSP * TTP *
+ (SS + TT + 2.0DD0 * UU - 1.0DD0)

C*** NOW MIDSIDE NODE

30 IN = IN + 1

PROD(1) = DABS( CN(IN,2) * CN(IN,3) )
PROD(2) = DABS( CN(IN,3) * CN(IN,1) )
PROD(3) = DABS( CN(IN,1) * CN(IN,2) )
SSP = 1.0DD0 + S * CN(IN,1)
TTP = 1.0DD0 + T * CN(IN,2)
UUP = 1.0DD0 + U * CN(IN,3)

TEMP1 = 0.25DD0 * SSP * TTP * UUP
TEMP2 = 1.0DD0 - S*S*PROD(1) - T*T*PROD(2) - U*U*PROD(3)
SHAPE(IN) = TEMP1 * TEMP2
DO 40 ID = 1,3
DERIV(ID,IN) = TEMP1 * ( TEMP2 * CN(IN,ID) / 
+ ( 1.0DD0 + CN(IN,ID)*X(ID) )
+ - 2.0DD0 * PROD(ID) * X(ID) )
40 CONTINUE

IF ( IN .EQ. 20 ) RETURN
IF ( (2*IN - 19) / 4 ) 20,30,20
END

Appendix B
C **********************************************************************
C GAUSS QUADRATURE ROUTINES
C **********************************************************************

SUBROUTINE GAUSSQ

COMMON /gaua/, posgp(3), weigp(3), ngaus, onit(20,5)

COMMON / units / lu, LUO, LU1, LU2, LU3, LU4

C WRITE (LU0,1000)
C 1000 FORMAT (' ENTERING S/R GAUSSQ '

NGP = NGAUS + 1
NGS = (NGAUS+1)/2 + 1

GO TO (10,20,30,40,50), NGAUS

10 POSGP(1) = 0.0000000000000000D0
    WEIGP(1) = 2.0000000000000000D0
    GO TO 90

20 POSGP(1) = -0.577350269189626D0
    WEIGP(1) = 1.000D0
    GO TO 80

30 POSGP(1) = -0.774596669241483D0
    WEIGP(1) = 0.555555555555556D0
    POSGP(2) = 0.0000000000000000D0
    WEIGP(2) = 0.888888888888889D0
    GO TO 80

40 POSGP(1) = -0.861136311594053D0
    WEIGP(1) = 0.347854845137454D0
    POSGP(2) = -0.339981043584856D0
    WEIGP(2) = 0.652145154862546D0
    GO TO 80

50 POSGP(1) = -0.906179845938664D0
    WEIGP(1) = 0.236926885056189D0
    POSGP(2) = -0.538469310105683D0
    WEIGP(2) = 0.478628670499366D0
    POSGP(3) = 0.0000000000000000D0
    WEIGP(3) = 0.569888888888889D0

80 DO 85 IG = NGS, NGAUS
    POSGP(IG) = -POSGP(NGP-IG)
    WEIGP(IG) = WEIGP(NGP-IG)

85 CONTINUE

90 CONTINUE

RETURN
END

Appendix B
Calculates the overburden pressure of problem with specified top surface.

IMPLICIT real*8 (A-H,O-Z)

CHARACTER*20 INFIL,IOUT,IPLT
CHARACTER*4 ihoz

dimension poin(20,2),pmat(20),x(10000),y(10000),z(10000),
+ nodes(2000,20),matno(2000),xt(8),yt(8),zt(8),str0(2000,3,8)

COMMON / units / LU0,LU1,LU2,LU3,LU4
common /gauss / posgp(3),weigp(3),onit(20,5),ngaus
COMMON /HSP / SHAPE(20),DERIV(3,20),ELCOD(3,20),CARTID(3,20)

maxp0n = 20
maxmat = 19
maxnod = 10000
maxel = 2000

C==== Open logical unit numbers...0 vdu...2 read...3 write...4 plot.
LU0 = 0
LU2 = 2
LU3 = 3
LU4 = 4

C==== Open input-output files
WRITE(LU0,'(/A)') ' What is the name of your input file ?'
READ(LU0,101) INFIL
101 FORMAT(A)
OPEN(LU2,FILE=INFIL,STATUS='UNKNOWN')

WRITE(LU0,'(/A)') ' What is the name of your output file ?'
READ(LU0,101) IOUT
OPEN(LU3,FILE=IOUT,STATUS='UNKNOWN')

WRITE(LU0,'(/A)') ' What is the name of your plot file ?'
READ(LU0,101) IPLT
OPEN(LU4,FILE=IPLT,STATUS='UNKNOWN')

C==== Start gathering surface information.
write(lu0,'(a)') ' How many points define the top surface. '
read(lu0,*) ipon
if(ipon.gt.maxpon) then
write(lu0,'(a,i3)') ' Too many definition points, max=',maxpon
stop
endif
write(lu3,'(/a,i3)') ' No. of points defining top surface=',ipon
ipol = ipon - 1

write(lu3,'(/a)') ' No. horz. vert. '
write(lu3,'(a)') '-----------------------'
do 10 i=1,ipon
write(lu0,'(a,i4/)') ' input the horz. & vert. coords of pt ',i

Appendix B
read(1u0,*), poin(i,1), poin(i,2)
write(1u3,'((i4,2x,f12.4,2x,f12.4))')i,poin(i,1), poin(i,2)
10 continue

write(1u0,'(a)') ' How many material layers exist? '
read(1u0,*) imat
if(imat.gt.maxmat) then
write(1u0,'(a,i3)') ' Too many material layers, max=',maxmat
stop
endif
imat1 = imat + 1
write(1u3,'(/a,i3)') ' No. of points defining materials=', imat1

write(1u3,'(a)') ' No. height. '
write(1u3,'(a)') '-------------------'

do 21 i=1,imat1
write(1u0,'(a,i4/)') ' input the height of material pt ',',i
read(1u0,*) pmat(i)
write(1u3,'((i4,2x,f12.4))')i, pmat(i)
21 continue

c====== Read the coord data from the input file.
read(1u2,*), numnod, numel

write(1u3,'(/a,i5)') ' The no. of nodes = ', numnod
write(1u3,'(a,i5)') ' The no. of elements = ', numel

if(numnod.gt.maxnod) then
write(1u0,'(a)') ' Too many nodes for this program. '
stop
endif

if(numel.gt.maxel) then
write(1u0,'(a)') ' Too many elements for this program. '
stop
endif

WRITE (LU3,920)
WRITE (LU3,925)
920 FORMAT (/ ' Nodal point coordinates ' + / ' ------ ------ ------ ')
925 FORMAT (/ ' Node', 7X, 'X', 9X, 'Y', 9X, 'Z'/ + ' ------ ------ ------ ')

do 20 i=1,numnod
read(1u2,*) ii, x(ii), y(ii), z(ii)
write(1u3,'((i5,3f11.4))') ii, x(ii), y(ii), z(ii)
20 continue

WRITE (LU3,910)
910 FORMAT (/ ' Element', 38X, 'Node numbers', 51X, 'Material' + / '118(1H-)')

DO 102 IELEM = 1,NUMEL
READ (LU2,*) IEL,
+ (NODES(IEL,INODE), INODE = 1, 20), MATNO(IEL)
WRITE (LU3,915) iel,
+ (NODES(iel,INODE), INODE = 1, 20), MATNO(iel)
915 FORMAT (1X,I5,2X,2I15 )
102 CONTINUE

c==== Initialise the stress matrix.
do 71 i=1,numel
do 72 j=1,8
do 73 k=1,3
    str0(i,k,j) = 0.
73 continue
72 continue
71 continue

C==== Set up the gauss point coordinates for the elements.
write(lu6,'(/a/)') ' What order of Gaussian int. '
    read(lu6,'(i3)') ngaus
call gaussq

C==== Read unit weight, Kx, Ky for each layer
do 106 i = 1,imat
    WRITE(LU0,'(/a,I3/)')
    * ' Give unit wt., Kx, klx, Ky, kly, for layer...' ,i
    READ(LU0,*) (onit(i,j),j=1,5)
106 continue

    write(lu3,'(/a/)') ' Mat. gama xx klx ky kly '
    do 41 i = 1,imat
        write(lu3,'(i4,5f13.4)')i,(onit(i,j),j=1,5)
41 continue

C==== Work out if X or Y is the horozontal coord. 
write(lu0,'(a)') ' (xhor)ozontal or (yhor)ozontal '
    read(lu0, '(a)') ihoz

    write(lu3,'(/a/)') ' El no. gpt. dl d2 hdist zsurf '

DO 85 LL=1,NUMEL
N1=NODES(LL,1)
N2=NODES(LL,2)
N3=NODES(LL,3)
XT(1)=(X(N1)+X(N2)+X(N3))/3.
YT(1)=(Y(N1)+Y(N2)+Y(N3))/3.
zT(1)=(z(n1)+z(n2)+z(n3))/3.
DO 5 K=1,20
    ND=NODES(LL,K)
    ELCOD(1,K)=X(ND)
    ELCOD(2,K)=Y(ND)
5 elcod(3,K)=Z(ND)
NGP=0
DO 107 L=1,ngaus
    S=posgp(L)
DO 107 M=1,ngaus
    T=posgp(M)
    do 107 n=1,ngaus
        v=posgp(n)
        NGP=NGP+1
    c    write(lu2,'(/a/)') ' s,t,v '
    c    write(lu2,'(3e15.7)') s,t,v
    CALL SF3r2(S,T,v)
    XG=0.0
    YG=0.0
    ZG=0.0
    DO 207 K=1,20

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zg = zg + shape(k) * elcod(3, k) 
YG = YG + SHAPE(K) * ELCOD(2, K) 
207 XG = XG + SHAPE(K) * ELCOD(1, K) 
zf (ngp) = zg 
X(NGP) = XG 
107 Y(NGP) = YG 

DO 888 NGP = 1, 8 
XBAR = X(NGP) 
YBAR = Y(NGP) 
zbar = zf (ngp) 
hdist = 0. 

if (ihoz .eq. 'XHOR' .or. ihoz .eq. 'XHOR') then 
hdist = xbar 
else 
hdist = ybar 
endif 
zsurf = 0. 

do 12 ip = 1, ipol 
ip2 = ip + 1 
if (hdist .ge. poin(ip, 1) .and. hdist .le. poin(ip2, 1)) then 
diff = poin(ip2, 2) - poin(ip, 2) 
endif 
diff2 = poin(ip2, 1) - poin(ip, 1) 
if (diff2 .lt. le-09 .and. diff2 .gt. -le-09) then 
write (lu0, '(a)') ' A surface element is vertical ' 
stop 
elseif (diff1 .lt. le-09 .and. diff1 .gt. -le-09) then 
zsurf = poin(ip, 2) 
else 
zsurf = ((diff1/diff2) * (hdist - poin(ip, 1))) + poin(ip, 2) 
endif 
endif 
12 continue 

write (lu3, '(2i6,5f11.4)') ll, ngp, diff1, diff2, hdist, zsurf 

c=== We now have the surface height; zsurf, and the gauss pt. 
c=== height; zbar. 

do 13 im = 1, imat 
  i1 = im 
  i2 = im + 1 
if (zsurf .le. pmat(i2)) then 
goto 13 
elseif (zbar .le. pmat(i2) .and. zsurf .lt. pmat(i1)) then 
str0(1L, 3, ngp) = str0(1L, 3, ngp) + (zsurf - pmat(i2)) * onit(im, 1) 
STRO(1L, 1, NGP) = onit(im, 2) * STRO(1L, 3, NGP) + onit(im, 3) 
STRO(1L, 2, NGP) = onit(im, 4) * STRO(1L, 3, NGP) + onit(im, 5) 
write (lu3, '(a)') ' made it to 1 ' 
elseif (zbar .le. pmat(i2) .and. zsurf .ge. pmat(i1)) then 
str0(1L, 3, ngp) = str0(1L, 3, ngp) + (pmat(i1) - pmat(i2)) * onit(im, 1) 
STRO(1L, 1, NGP) = onit(im, 2) * STRO(1L, 3, NGP) + onit(im, 3) 
STRO(1L, 2, NGP) = onit(im, 4) * STRO(1L, 3, NGP) + onit(im, 5) 
write (lu3, '(a)') ' made it to 2 ' 
elseif (zbar .gt. pmat(i2)) .and. zbar .lt. pmat(i1) + .and. zsurf .ge. pmat(i1) then 

Appendix B
str0(ll,3,ngp) = str0(ll,3,ngp) + (pmat(i1)-zbar)*onit(im,1)
STRO(LL,1,NGP)=onit(im,2)*STRO(LL,3,NGP)+onit(im,3)
STRO(LL,2,NGP)=onit(im,4)*STRO(LL,3,NGP)+onit(im,5)
write(lu3,'(a)') ' made it to 3'
else if(zbar.gt.pmat(i2) .and. zbar.lt.pmat(i1)
 + .and. zsurf .lt. pmat(i1) )then
  str0(ll,3,ngp) = str0(ll,3,ngp) + (zsurf-zbar)*onit(im,1)
STRO(LL,1,NGP)=onit(im,2)*STRO(LL,3,NGP)+onit(im,3)
STRO(LL,2,NGP)=onit(im,4)*STRO(LL,3,NGP)+onit(im,5)
write(lu3,'(a)') ' made it to 4'
endif
13 continue

write(lu4,'(4e15.7)') hdist,zbar,zsurf,str0(ll,3,ngp)

888 CONTINUE
85 CONTINUE

do 14 i=1,numel
write(lu3,'(3e15.7)') ((str0(i,j,k),j=1,3),k=1,8)
14 continue

C==== Close files
CLOSE(LU2)
CLOSE(LU3)
CLOSE(LU4)

END

C ***********************************************
C CALCULATES SHAPE FUNCTIONS AND THEIR DERIVATIVES
C FOR THREE DIMENSIONAL QUADRATIC ELEMENTS
C ***********************************************

SUBROUTINE SF3R2(S,T,U)
IMPLICIT real*8 (A-H,O-Z)

DIMENSION CN(20,3),PROD(3),X(3)

COMMON / units / LU0,LU1,LU2,LU3,LU4
COMMON /SHP / SHAPE(20),DERIV(3,20),ELCOD(3,20),CARTD(3,20)

c write(lu2,'(/a/)') ' entering sf3r2 ' 

C*** FIRST VALUES OF S AT NODES 
   DATA CN / -1.,0.,+1.,+1.,+1.,0.,-1.,-1., 
 + -1.,+1.,-1.,-1., 
 + -1.,0.,+1.,+1.,+1.,0.,-1.,-1., 

C*** NOW T VALUES AT NODES
 + -1.,-1.,-1.,0.,+1.,+1.,0.,
 + -1.,-1.,+1.,+1.,
 + -1.,-1.,-1.,0.,+1.,+1.,0.,

C*** NOW U VALUES AT NODES
 + -1.,-1.,-1.,-1.,-1.,-1.,-1.,-1.,
+ 0.,0.,0.,0.,
+ +1.,+1.,+1.,+1.,+1.,+1.,+1.,+1.
/ 
X(1) = S
X(2) = T
X(3) = U
IN = 0

20 IN = IN + 1

C*** CORNER NODE FIRST

SS = S * CN(IN,1)
TT = T * CN(IN,2)
UU = U * CN(IN,3)
SSP = SS + 1.0D0
TTP = TT + 1.0D0
UUP = UU + 1.0D0

SHAPED(IN) = 0.125D0 * SSP * TTP * UUP *
+ (SS + TT + UU - 2.0D0)
DERIV(1,IN) = 0.125D0 * CN(IN,1) * TTP * UUP *
+ (2.0D0*SS + TT + UU - 1.0D0)
DERIV(2,IN) = 0.125D0 * CN(IN,2) * SSP * UUP *
+ (SS + 2.0D0*TT + UU - 1.0D0)
DERIV(3,IN) = 0.125D0 * CN(IN,3) * SSP * TTP *
+ (SS + TT + 2.0D0*UU - 1.0D0)

C*** NOW MIDSIDE NODE

30 IN = IN + 1
PROD(1) = DABS( CN(IN,2) * CN(IN,3) )
PROD(2) = DABS( CN(IN,3) * CN(IN,1) )
PROD(3) = DABS( CN(IN,1) * CN(IN,2) )
SSP = 1.0D0 + S * CN(IN,1)
TTP = 1.0D0 + T * CN(IN,2)
UUP = 1.0D0 + U * CN(IN,3)

TEMP1 = 0.25D0 * SSP * TTP * UUP
TEMP2 = 1.0D0 - S*S*PROD(1) - T*T*PROD(2) - U*U*PROD(3)
SHAPE(IN) = TEMP1 * TEMP2
DO 40 ID = 1,3
DERIV(ID,IN) = TEMP1 * ( TEMP2 * CN(IN,ID) / 
+ ( 1.0D0 + CN(IN,ID)*X(ID) ) 
+ - 2.0D0 * PROD(ID) * X(ID) )
40 CONTINUE

IF ( IN .EQ. 20 ) RETURN
IF ( (2*IN - 19) / 4 ) 20,30,20
END

C **********************************************************************
C GAUSS QUADRATURE ROUTINES
C **********************************************************************

SUBROUTINE GAUSSQ

IMPLICIT real*8 (A-H,O-Z)

Appendix B
COMMON / units / LU0,LU1,LU2,LU3,LU4
common /gauss / posgp(3),weigp(3),enit(20,5),ngaus

WRITE (LU0,1000)
C 1000 FORMAT ( ' ENTERING S/R GAUSSQ ' )

NGP = NGAUS + 1
NGS = (NGAUS+1)/2 + 1

GO TO (10,20,30,40,50),NGAUS

10 POSGP(1) = 0.000000000000000D0
   WEIGP(1) = 2.000000000000000D0
   GO TO 90

20 POSGP(1) = -0.577350269189626D0
   WEIGP(1) = 1.00D0
   GO TO 80

30 POSGP(1) = -0.774596669241483D0
   WEIGP(1) = 0.555555555555556D0
   POSGP(2) = 0.000000000000000D0
   WEIGP(2) = 0.888888888888889D0
   GO TO 80

40 POSGP(1) = -0.861136311594053D0
   WEIGP(1) = 0.347854845137454D0
   POSGP(2) = -0.339981043584856D0
   WEIGP(2) = 0.652145154862546D0
   GO TO 80

50 POSGP(1) = -0.906179845938664D0
   WEIGP(1) = 0.236926885056189D0
   POSGP(2) = -0.538469310105683D0
   WEIGP(2) = 0.478628670499366D0
   POSGP(3) = 0.000000000000000D0
   WEIGP(3) = 0.568888888888889D0

80 DO 85 IG = NGS,NGAUS
   POSGP(IG) = -POSGP(NGP-IG)
   WEIGP(IG) = WEIGP(NGP-IG)
85 CONTINUE

90 CONTINUE

RETURN
END
Appendix C

Plast (with Sloan and Booker's modified Mohr-Coulomb plastic potential surface)
C Routine to set up the plastic correction matrix COR().
SUBROUTINE PLAST(LL,NGP)

IMPLICIT REAL*8 (A-H,O-Z)
DIMENSION AL(6),EP(6),SAV(6),V1(6),V2(6),V3(6)

COMMON / CONTRO / NUMNOD,NUMEL,NNODE,NDOFN,NDIME,NSTRE,isc,
  istel,istcod,NGAUS,NPROP,NMATS,NVFIX,NEVAE,
  ICASE,NCASE

COMMON / LGDATA / COORD(9000,3),PROPS(50,10),PRES(200,3),
  + ELOAD(1700,96),NODER(1700,32),
  + MATNO(1700)

COMMON / WORK / ELCOD(3,32),SHAPE(32),DERIV(3,32),DMAIX(6,6),
  + CARTD(3,32),DBMAT(6,96),BMATX(6,96),
  + POSGP(3),WEIGP(3),GPCODE(3),
  + NEROR(24),stplt(6000,5)

COMMON / PROP / IP(96)
COMMON / STRES/ EPS(6),STRF(1700,6,27),STRO(1700,6,27),
  + DSTR(1700,6,27),IPEL(1700,27)

COMMON / PLA / BET(6),A(6),COR(96,96)
COMMON /UNITS/ LUO,LU1,LU2,LU3,LU4,lu6

PT = 3.14159265

C== Compute average stress over increment
DO 1 J = 1, NSTRE
  1 MAT = MATNO(LL)
  PH = PROPS(MAT,3)
  S = SIN(PH)
  C = COS(PH)
  PS = PROPS(MAT,4)
  S1 = SIN(PS)
  C = SSTR = PROPS(MAT,5)

C== Three dimensional failure surface.
C== Sloane's Mohr-Coulomb with rounded corners
C== Needs compression -ve. Order of stresses shown below.
  SX = -SAV(1)
  SY = -SAV(2)
  SZ = -SAV(3)
  TXY = -SAV(4)
  TYZ = -SAV(5)
  TZX = -SAV(6)
  SM = (SX + SY + SZ)/3.
  XX = (SX*SX + SY*SY + SZ*SZ - (SX*SY + SY*SZ + SZ*SX))/3.
  + TXY*TXY + TYZ*TYZ + TZX*TZX
  SB = SQRT(XX)
  IF(SB .EQ. 0.0) THEN
    WRITE(0,'((/A,E13.4))') ' SB is zero *STOP*', SB
    STOP
  ENDIF
  X1 = 2.*SX - SY - SZ
  X2 = 2.*SY - SX - SZ
  X3 = 2.*SZ - SX - SY

Appendix C

VAL = 0.
IF(SB.GT.0.) THEN
   VAL = 3.*SQRT(3.)*XJ3/2./SB**3
IF(val.gt.1.0) val = 1.0
IF(val.lt.-1.) val = -1.0
ENDIF
XLODE = ASIN(-VAL)/3.
DEG = 180.*XLODE/P1
S3 = SIN(3.*XLODE)
C3 = COS(3.*XLODE)
TEST = ABS(DEG)

C==== Transition surfaces. Starting at 25 degrees Lode angle.
IF(TEST.GT.25.) THEN
   SIGN = -1.
   IF(XLODE.GE.0.) SIGN = 1.
   AV = 1.432052 + 0.406942*SIGN*S
   BV = 0.544291*SIGN + 0.673903*S
   CON = (2.*BV*S3 + AV)*0.5/SB
   CON1 = 3.*SQRT(3.)*BV/XX/2.
ELSE
C==== Planar part of Mohr-Coulomb surface
   c1 = cos(xlode)
   s1 = sin(xlode)
   P1 = -S3/SB/C3
   P2 = -SQRT(3.)/2./SB**3/C3
   q1 = c1 - s*s1/sqrt(3.)
   q2 = -s1 - s*c1/sqrt(3.)
   CON = (SB*Q2*P1 + Q1)*0.5/SB
   CON1 = SB*Q2*P2
ENDIF

C==== Differentiation wrt sigma bar
V1(1) = CON*X1/3.
V1(2) = CON*X2/3.
V1(3) = -CON*X3/3.
V1(4) = -CON2*TYX
V1(5) = CON2*TYZ
V1(6) = CON2*TXZ

C==== Differentiation wrt J3
V2(1) = CON1*(2.*SY*SY/3. + SX*SX/3. - SM*SM
   * 2.*TYZ*TYZ/3. + TXZ*TXZ/3. + TXY*TXY/3.)
V2(2) = CON1*(2.*SZ*SZ/3. + SY*SY/3. - SM*SM
   * 2.*TZX*TZX/3. + TYZ*TYZ/3. + TXY*TXY/3.)
V2(3) = CON1*(2.*SX*SX/3. + SZ*SZ/3. - SM*SM
   * 2.*TXZ*TXZ/3. + TZY*TZY/3. + TXZ*TXZ/3.)
V2(4) = CON1*(2.*TYZ*TZX - 2.*X3*TXZ/3.)
V2(5) = CON1*(2.*TYX*TZX - 2.*X1*TYZ/3.)
V2(6) = CON1*(2.*TXY*TZX - 2.*X2*TXZ/3.)

C==== Differentiation wrt J1
V3(1) = S/3.
V3(2) = S/3.
V3(3) = S/3.
V3(4) = 0.
V3(5) = 0.
V3(6) = 0.
BP(1) = -(V1(1) + V2(1) + V3(1))
BP(2) = -(V1(2) + V2(2) + V3(2))
BP(3) = -(V1(3) + V2(3) + V3(3))

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BP(4) = -(V1(4) + V2(4) + V3(4))
BP(5) = -(V1(5) + V2(5) + V3(5))
BP(6) = -(V1(6) + V2(6) + V3(6))

C==== For A() matrix - Three dimensional failure surface.
C==== Sloane's Mohr-Coulomb with rounded corners
C==== Needs compression -ve. Order of stresses shown below.
VAL = 0.
IF(SB.GT.0.) THEN
  VAL = 3. * SQRT(3.) * XJ3/2. / SB**3
  IF (VAL.GT.1.0) VAL = 1.0
  IF (VAL.LT.-1.0) VAL = -1.0
ENDIF
XLODE = ASIN(-VAL) / 3.
DEG = 180. * XLODE / PI
S3 = SIN(3. * XLODE)
C3 = COS(3. * XLODE)
TEST = ABS(DEG)

C==== Transition surfaces. Starting at 25 degrees Lode angle.
IF (TEST.GT.25.) THEN
  SIGN = -1.
  IF (XLODE.GE.0.0) SIGN = 1.
  AV = 1.432052 + 0.406942 * SIGN * S1
  BV = 0.544291 * SIGN + 0.673903 * S1
  CON = (2. * BV * S3 + AV) * 0.5 / SB
  CON1 = 3. * SQRT(3.) * BV / XX / 2.
ELSE
C==== Planar part of Mohr-Coulomb surface
  c1 = cos(xlode)
  s1 = sin(xlode)
  p1 = -S3/SB/C3
  p2 = -SQRT(3.) / 2. / SB**3 / C3
  q1 = c1 - s1 * s1 / sqrt(3.)
  q2 = s1 - s1 * c1 / sqrt(3.)
  CON = (SB * Q2 * P1 + Q1) * 0.5 / SB
  CON1 = SB * Q2 * P2
ENDIF

C==== Differentiation wrt sigma bar
  V1(1) = CON * X1 / 3.
  V1(2) = CON * X2 / 3.
  V1(3) = CON * X3 / 3.
  V1(4) = CON * 2. * TXY
  V1(5) = CON * 2. * TYZ
  V1(6) = CON * 2. * TXZ

C==== Differentiation wrt J3
  V2(1) = CON1 * (2. * SY * SZ / 3. + SX * SX / 3. - SM * SM)
  * -2. * TYZ * TYZ / 3. + TZX * TZX / 3. + TXY * TXY / 3.)
  V2(2) = CON1 * (2. * SX * SX / 3. + SY * SY / 3. - SM * SM)
  * -2. * TXZ * TXZ / 3. + TYZ * TYZ / 3. + TYX * TYX / 3.)
  V2(3) = CON1 * (2. * SX * SY / 3. + SZ * SZ / 3. - SM * SM)
  * -2. * TXY * TXY / 3. + TYZ * TYZ / 3. + TXZ * TXZ / 3.)
  V2(4) = CON1 * (2. * TYZ * TXZ - 2. * X3 * TYX / 3.)
  V2(6) = CON1 * (2. * TXY * TYZ - 2. * X2 * TXZ / 3.)

C==== Differentiation wrt J1
  V3(1) = S1 / 3.
  V3(2) = S1 / 3.
  V3(3) = S1 / 3.
  V3(4) = 0.

Appendix C
V3(5) = 0.
V3(6) = 0.
C
a(1) = -(V1(1) + V2(1) + V3(1))
a(2) = -(V1(2) + V2(2) + V3(2))
a(3) = -(V1(3) + V2(3) + V3(3))
a(4) = -(V1(4) + V2(4) + V3(4))
a(5) = -(V1(5) + V2(5) + V3(5))
a(6) = -(V1(6) + V2(6) + V3(6))

C
CON = 1./(2.*RX)
C
A(2) = bp(2)
C
A(3) = bp(3)
C
A(4) = bp(4)
C
A(5) = bp(5)
C
A(6) = bp(6)

C
WRITE(LU3,*) ' A and BP'
C
write(lu3,'(6e13.4)') (a(i), i=1,NSTRE)
C
write(lu3,'(6e13.4)') (bp(i), i=1,NSTRE)

DO 20 I = 1, NSTRE
    SUM = 0.0
    DO 10 J = 1, NSTRE
        SUM = SUM + DMATX(I,J)*A(J)
    10 SUM = SUM
        AL(I) = SUM
    DO 40 I = 1, NSTRE
        SUM = 0.0
        DO 30 J = 1, NSTRE
            SUM = SUM + DMATX(I,J)*BP(J)
        30 SUM = SUM
            BET(I) = SUM
        DO 50 I = 1, NSTRE
            SUM = 0.0
            DO 40 J = 1, NSTRE
                COR(I,J) = AL(I)*BET(J)/SUM
            40 COR(I,J) = DMATX(I,J) - COR(I,J)
    50 SUM = SUM

Appendix C