MODELING OF AN UNDERGROUND MINE BACKFILL BARRICADE

by

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A thesis submitted in conformity with the requirements for the degree of Masters of Applied Science
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Abstract

In this thesis finite element analyses were performed to investigate the behavior of fill fences installed in underground mines to retain Cemented Paste Backfill (CPB) pressure. For this purpose, two fill fences installed and tested in the Cayeli mine in Turkey were modeled using a 2-D nonlinear finite element analysis program, Augustus-2, and a 3-D nonlinear finite element analysis program VecTor4, and the results were compared with measured field data. Different models were employed representing the material properties, boundary conditions, reinforcement ratio, and geometric properties, and it was found that boundary conditions (stiffness of surrounding rocks) has the highest influence on the pressure capacity of the fence among the other factors. The accuracy of the Augustus-2 program was investigated by modeling and comparing the analytical response with test results of 12 axially restrained beams tested by Su et al. (2009).
Acknowledgments

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Chapter 1
Introduction

1 Introduction

1.1 Cemented Paste Backfill

Cemented paste backfill (CPB) is a material used by many modern mines throughout the world as a technique to dispose of mine wastes. It is an alternative to other methods such as conventional surface mine waste disposal and has been used since the 1980’s. In this method the waste products (tailings) are mixed with binders (3-7% Portland cement) and then transported back to the underground mine to backfill the excavated stope. Binder contents generate mechanical strength in the CPB so enough support can be provided to the surrounding stopes, hence enhancing the mining process.

Cemented paste backfill method has two main advantages. The first advantage is that it has reduced environmental impact. The waste products are transported underground, thus the chance of acidic effluent generation is very low. The second advantage is that it can reduce operational cost. Backfilling open stopes reduce the amount of waste materials needed to be sent to the disposal tailing facilities. In addition to this, when CPB develops strength it can potentially provide enough support to the adjacent stopes, thus the mining process can be accelerated and operation costs reduced (Belm et al., 2000).

Although this method was found to be a very efficient technique, the lack of good understanding of the CPB behavior results in conservative design strategies. For instance, in order to prevent the failure of reinforced concrete wall (fill fence) due to the applied pressure by cemented paste backfill in the mine, stopes are poured in multiple stages. This delay can be prevented by having a good understanding about the fill fence response and capacity under hydrostatic loading. Thus, a series of tests have been conducted in William, Kidd and Cayeli mines by Professors M.W.
Grabinsky and W. Bawden to investigate the CPB behavior. In this research, the response of the fill fences installed in the Cayeli mine was investigated and results are compared to the finite element analysis response.

1.2 Fill Fence

In order to isolate the open stope from the adjacent stopes for backfilling process, the fill fence is constructed and installed at the bottom of the excavated stope, at the entrance of the horizontal tunnel that connects the stopes to resist the hydrostatic pressure resulting from cemented paste backfill (See Figure 1.1). A Good understanding of the structural behavior and response of the fill fence can help in designing a safe and cost effective fence and can also accelerate the filling process and minimize cost. These are generally, constructed out of reinforced concrete, and placed as shotcrete.

Figure 1-1 A: cross section view of a stope, B: plan view of a stope

(Thompson et al., 2010)
The Fill fences installed in underground mines and surrounded by adjacent rocks consist of four main components (shown in Figure 1.2): 1- Rebar grouted into the rock, 2- Wooden frame installed to provide a backing of shotcrete, 3- Horizontal and vertical rebar placed at mid depth of the fence, 4- Shotcrete.

Figure 1-2 Fill Fence

(Thompson et al., 2010)
The behavior of the fill fence structures used to retain the CPB is similar to conventional two-way reinforced concrete slabs subjected to uniformly distributed load but heavily restrained around the edges. When these slabs resist uniform load, the side exposed to the uniform load will resist in plane compression and the other side will go into tension. Since the tension is resisted only by the reinforcement but the compression is resisted by the reinforcement and the concrete, the tensile strain will be much higher than the compressive strain. The average strain will thus be tensile and this indicates that conventional slabs would experience a net elongation due to the applied load. In labs this is accommodated by roller bearings, for example. For fill fences however, this net elongation cannot occur due to the restraint provided by the surrounding rocks. By restraining these elongation effects an axial compression will be developed and these membrane forces will increase the shear and moment capacity, probably by a large amount. This effect can be called membrane action.

Figure 1.3 shows this effect for a rigid barrier with a single crack, note that the fence must push into the surrounding rock to produce a failure mechanism. These fill fences can be expected to be potentially dramatically stronger than conventional two-way slabs and where serviceability concerns are not important, as in underground applications, this strength enhancement can be considered in design.

Figure 1-3 Compressive membrane action
Counteracting this membrane effect, there are other behaviors that should be considered by designers. Geometric nonlinear effects, in the form of the P-δ effect will reduce the beneficial effect of the compression if displacements are large enough. Shrinkage effects will also produce a permanent gap between the fence and the surrounding rocks requiring larger displacements to engage the effect. Their effect will not be considered in this thesis though they can be important particularly for members with low reinforcement bars.

1.3 Finite element analysis methods

There are generally two types of analyses that can model an underground fill fence: 2-D modeling and 3-D modeling. In a 2-D model, a 1 m wide slice of the fence is modeled ignoring 2-way slab action and the fence is treated as a one way beam. In 3-D modeling the full geometry of the fill fence is considered. In both cases one of the dimensions models the out-of-plane displacement. The 2-D model is relatively simple and allows analyses to be solved quickly on an inexpensive computer but can produce less accurate results. The more advanced model should produce a more accurate result but at a lower speed and increased technical knowledge to operate. There are also two possibilities for each analysis: linear and nonlinear. The advantage of a nonlinear finite element analysis over a linear one is that it can capture the effect of membrane action; hence the fill fence behavior, load capacity and failure mode can be predicted more accurately. The current state of the art for building design is to do 3D linear analyses. For fill fences, it may be possible to do 3D nonlinear analyses in a research environment but it should not be difficult to do 2D nonlinear analyses in a practical way.

To perform a nonlinear analysis, a constitutive theory is needed for cracked reinforced concrete. In this thesis the Modified Compression Field Theory (MCFT) will be used (Vecchio and Collins, 1986). The MCFT was developed in the 1980’s and is able to predict the behavior of reinforced concrete elements subjected to in-plane shear and normal stresses. Equilibrium, compatibility and stress-strain relationships are based on average stress and average strain. Additional checks are made to ensure that conditions locally at a crack do not control in terms of steel yielding and sliding along a crack. The model assumes that cracked reinforced concrete can be treated as a new material with its own stress-strain characteristics.
This research will focus on analyzing a published restrained beam tests and an underground mine backfill fence. The structural analysis will be based on 2-D nonlinear finite element programs Augustus-2 and Response-2000 developed at the University of Toronto by Professor E.C Bentz, and three dimensional finite element program VecTor4, developed at the University of Toronto by Professor F.J. Vecchio. Both programs are based on the Modified Compression Field Theory (MCFT) (Vecchio and Collins, 1986).

1.4 Research Highlights

Chapter 2 - Represents field work result of two fill fences installed and tested in the Cayeli mine in Turkey. The fill fences construction process as well as their Pressure-displacement response is discussed and shown in this chapter.

Chapter 3- Investigate the load-deflection behavior of 12 axially restrained specimens tested in China by Su, Tian, and Xiaosheng (2009). to evaluate the influence of the compressive membrane action on the beams load carrying capacity.

Chapter 4- The twelve specimens summarized in chapter 3 were modeled using Augustus-2 program, load deformation analysis was conducted for each beam and the results are compared to the actual test data.

Chapter 5- The fill fence installed and backfilled in the Cayeli mine was modeled using a 3-D finite nonlinear element analysis program VECTOR4 and a 2-D nonlinear finite element program Augustus-2 and results are compared to the filed data.

Chapter 6- Conclusion and Recommendations
Chapter 2

Cayeli Mine

2 Cayeli Bakir Isletmeleri Mine

2.1 Introduction

Cayeli Bakir Isletmeleri A.S. (CBI) is an underground mine producing copper and zinc. It was founded in 1994 in northeastern Turkey and is expected to operate until 2017. This mine includes two test stopes called 685N20 and 715N22. The total applied pressure and displacement were measured on the fill fences used to retain the cemented paste backfill and results are presented in this thesis.

Based on the report provided by Thompson et al. (2010), Stope 685N20 was filled with CPB having 8.5% binder content for the first 8 m height and 6.5% binder for the rest of the stope. Stope 715N22 was filled with CPB having 6.5% binder content. Thus the effect of using different binder contents and different rise rate on the total applied pressure on the fill fence has been investigated as part of the research project and results are summarized in Table 2.1.

<table>
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<th>Stope</th>
<th>Binder Content (%)</th>
<th>Rise rate(cm/hr)</th>
<th>Fence Peak Pressure(kPa)</th>
<th>Displacement(mm)</th>
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<tr>
<td>685N20</td>
<td>6.5 (0-8 m height)</td>
<td>8.5 (8-15 m height)</td>
<td>23</td>
<td>46</td>
</tr>
<tr>
<td>715N22</td>
<td>8.5</td>
<td>35</td>
<td>100</td>
<td>22</td>
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</table>
2.2 Fill Fence

This section presents the experimental details of the fence constructed and tested at Cayeli mine. Geometric properties, material properties, instruments and boundary conditions of each fence and their effect on the fence pressure-displacement are presented.

2.2.1 685N20 Fill Fence

2.2.1.1 Fence Construction

The fill fence installation process started by grouting 20 mm diameter rebar to the surrounding rock at 1000 mm spacing, so the fill fence was restrained from any in-plane and out-of-plane displacement along its four edges. The first construction step was followed by placing a wooden frame to provide a backing for the shotcrete and placing two layers of 28 mm diameter rebars perpendicular to each other at the mid thickness of the finished fence as is shown in Figure 2.1. Spacing of the horizontal reinforcement was 400 mm and spacing of the vertical reinforcement was 300 mm as shown in Figure 2.2a and 2.2b. The last stage of the construction process was shotcreting the fill fence. The constructed fill fence had a constant thickness of 300 mm.

Figure 2-1 Fill Fence Components (Thompson et al., 2010)
Figure 2-2a Fill Fence Construction

(Thompson et al., 2010)

Fig 2.2b Fill Fence Shotcreting Process

(Thompson et al., 2010)
2.2.1.2 Fence instruments

Three total earth pressure cells (TEPC), were placed at the center axis of the fence along with displacement transducers at different heights of the fence to measure the total applied horizontal pressure and displacement of the fence (See Figure 2.3a). The top cell was placed at 4.2 m from the bottom, the middle TEPC was placed at the height of 2.4 m and the lowest cell was placed at 1.2 m (Figure 2.3b-2.3c). In addition to TEPC, five cages were places at different heights and different horizontal distances from the stope to investigate the loading mechanics of the paste. The first two cages (Cage 1 and 2) were placed at the height of 1.8 m having a horizontal spacing of 3.5 m, cage 3 was placed at a height of 2 m and horizontal distance of 10 m from the fence, cage 4 was placed at the height of 6 m and cage 5 placed at the height of 10 m but the same horizontal distance as cage 3 from the fence (See Figure 2.3d).

Figure 2-3a A: TEPC, B: Piezometers, C: Electrical Conductivity Probe, D: Heat dissipative sensor

(Thompson et al., 2010)
Figure 2.3b a: Instrument cage, b: Protective Cage

(Thompson et al., 2010)

Figure 2.3c Photograph of Fence Installed at 685N20 with approximate location of displacement transducers

(Thompson et al., 2010)
Figure 2.3d Fill Fence Instrumentation Plan

(Thompson et al., 2010)
2.2.1.3 Fill Fence Pressure, Displacement and Cracking

In order to design a safe and cost effective fill fence, a good understanding of the fence behavior under different loading and boundary conditions, as well as different geometric and material properties is required, so any unexpected failure can be prevented. For this purpose the pressure-deflection profile of 685N20 fence (Figure 2.4) was investigated and presented in this section.

Figure 2.4a shows the short term (2 day) behavior of the displacement and pressure response of the fill fence installed at 685 level. This fence was filled using a cemented paste backfill with 8.5% binder content. The initial plan was to fill the fence until a peak pressure of 100 kPa, but since the measured pressure never reached that value, the stope was poured continuously. The total applied pressure increased linearly until it reached the first peak value of 55 kPa at the top-mid, 42 kPa at the mid height and 35 kPa at the low height. The applied pressure dropped after it reached the initial peak value of 52 kPa and was followed by an increase perhaps due to hydration and expansion of the CPB.

The fence deflection was monitored and reported by placing six potentiometers at height of 1.4 m and 2.8 m above the base of the slope. They have been positioned at the same intervals and are symmetric about the vertical center axis as shown in Figure 2.5. The short term time-displacement response of fence 685 is shown in Figure 2.4 a. The long term response is presented in Figure 2.4b and results are summarized in Table 2.2. Based on the fieldwork report provided by Thompson et al. (2010), the maximum displacement of 6.5 mm happened at the mid height of the fence after 2 days. The maximum deflection increased by 20% and reached 7.8 mm at day five.

Table 2-2 Comparison of short term and long term displacement of fence 685N20

<table>
<thead>
<tr>
<th>Channel</th>
<th>Bottom-Left</th>
<th>Top-Left</th>
<th>Bottom-Mid</th>
<th>Top-Mid</th>
<th>Bottom-Right</th>
<th>Top-Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement(mm) at the First day</td>
<td>-3.9</td>
<td>-4.6</td>
<td>-5.1</td>
<td>-6.5</td>
<td>-3.5</td>
<td>-4.9</td>
</tr>
<tr>
<td>Displacement(mm) at the Third day</td>
<td>-4.6</td>
<td>-5.6</td>
<td>-6.0</td>
<td>-7.8</td>
<td>-4.2</td>
<td>-5.8</td>
</tr>
</tbody>
</table>
Displacement key: B: bottom, T: Top, R: right, L: Left, M: Middle

Figure 2.4a Short term Time-Deflection and Pressure Response of Fence 685N20

(Thompson et al., 2010)

Displacement key: B: Bottom, T:Top, R:right,L:Left,M:Middle

Figure 2.4b Long term Time-Deflection and Pressure Response of Fence 685N20

(Thompson et al., 2010)
Cracks observed post backfilling are marked in white paint and are shown in Figure 2.6. The cracks widths were smaller than the width of a box cutter blade and were mainly focused in the center of the fence. Cracks located in the center do not appear to be structurally induced though the ones located at the bottom-center and shown with letter “A” may be due to applied pressure and positive moment if the floor connection was insecure at this location.
2.2.2 715N22 Fill Fence

2.2.2.1 Fence Geometry, Construction and instrumentation

This fence is placed in the stope located between the 715 and 700 level (shown in Figure 2.7). The fence had a long span of 8.5 m with a height 4.5 m. The construction process was the same as the 685N20 fence. Total earth pressure cells were placed at three different locations along the height of the fence. One was placed at height of 1.1 m from the bottom, the mid one was placed at 2.2 m, and the top cell was place at 3.3 m. To measure the fence displacement, 10 displacement sensors were placed at critical locations (Figure 2.8).

Figure 2-7 Test stope located on 715 level

(Thompson et al., 2010)
Figure 2-8 Map of displacement transducers and total earth pressure cells

(Thompson et al., 2010)
2.2.2.2 Fill Fence Pressure, Displacement and Cracking

The total pressure applied to the fill located on stope 715 in Cayeli is shown in Figure 2.9. The initial plan was to backfill the stop with a rate of 35 cm/hr for the first 8 m, but since the pressure on the fence reached 100 kPa, and large deflection was observed, the backfilling process was stopped for two days. Cracks were observed, some of which were wide enough to fit a screwdriver.

The short and long term deflections of the fill fence measured by displacement transducers placed on the fence and are presented in Table 2.3. Short term deflection was measure on the first day and the long term was measured on the day seven. The maximum deflection of 22 mm occurred at the center of the fence.

<table>
<thead>
<tr>
<th>Stope 715N22</th>
<th>Bottom-Left</th>
<th>Top-Left</th>
<th>Center</th>
<th>Top-Mid</th>
<th>Bottom-Right</th>
<th>Top-Right</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement at the 1\textsuperscript{st} day (mm)</td>
<td>-19</td>
<td>-16</td>
<td>-21</td>
<td>-18</td>
<td>-11</td>
<td>-13</td>
</tr>
<tr>
<td>Displacement at the 7\textsuperscript{th} day (mm)</td>
<td>-20</td>
<td>-18</td>
<td>-22</td>
<td>-19</td>
<td>-13</td>
<td>-14</td>
</tr>
</tbody>
</table>
Figure 2.9a short term Time-Pressure profile of fence 715N22

(Thompson et al., 2010)

Figure 2.9b Long term Time-Pressure profile of fence 715N22

(Thompson et al., 2010)
Cracks observed post backfilling are marked with white painted and are shown in Figure 2.10. Based on the picture provided in the Cayeli mine report, vertical cracks located at the bottom-center and horizontal cracks located at the middle-center have developed due to two way action. That is, bending due to the uniform pressure behind the fence has put the visible face into flexural tension and these cracks are visible. Negative moment cracks, if any, would be on the other side of the fence and these would not be visible.

Figure 2-10 Crack Pattern of Fence 715 N 22

(Thompson et al., 2010)
Chapter 3
Axially Restrained Beams

3 Axially Restrained Beams

3.1 Introduction

The purpose of this chapter is to summarize the details and results of the tests that were done in China by Su, Tian, and Xiaosheng (2009). This research consisted of 12 specimens to investigate the load carrying capacity of reinforced concrete beams restrained against axial deformation. The specific objectives of the experiment were to evaluate the influence of the compressive membrane action on the load carrying capacity and the effect of loading rate on the ultimate strength of beams.

The first set of specimens, A-series, and included six reinforced concrete beams having identical geometric properties, but varying longitudinal and transverse reinforcement ratios. The second series of specimens (B-series) consisted of three beams with an identical cross sectional area but with a longer span than the A-series beams. They also had varying flexural and transverse reinforcement ratios. The third series of specimens (C-series) had the same span as the A-series specimens, but had smaller cross sectional area. The flexural and transverse reinforcement ratio was the same in C-series beams. The geometrical properties, material properties and the test results are presented in the following sections.
3.2 Specimen Properties

3.2.1 A-Series Specimen Properties

3.2.1.1 Introduction

The first set of specimens (A-series) consisted of six beams, A1 to A6. All the specimens had an identical clear span length of 1225 mm. They all had a rectangular cross section with an overall width of 150 mm and height of 300 mm. The beams elevation view and plan view are shown in Figure 3.1.

![Plan View](image)

![Elevation View](image)

Figure 3-1 Elevation view and plan of A-Series specimens

(Su et al., 2009)

The purpose of these tests was to investigate the effect of the longitudinal reinforcement ratio on the load carrying capacity, horizontal reaction and vertical displacement of the beams. For this reason specimens were reinforced with 12 mm and 14 mm diameter bars placed at two layers in the top and bottom of the cross section. In addition to the longitudinal reinforcement, transverse reinforcement bars having cross sectional area of 50 mm$^2$ were placed in the beam so that a shear failure could be prevented.
The concrete clear cover was specified as 20 mm for all the specimens. The geometric properties of the beams are summarized in Table 3.1 and the cross sections are shown in Figure 3.2.

Table 3-1 A-Specimen properties

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>b × h</th>
<th>1_n</th>
<th>l_n/l</th>
<th>f_{cu}</th>
<th>Top reinforcement(ratio)</th>
<th>Bottom reinforcement(ratio)</th>
<th>Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>150×300</td>
<td>1225</td>
<td>4.08</td>
<td>32.3</td>
<td>2ϕ 12 mm (.55%)</td>
<td>2ϕ 12 mm (.55%)</td>
<td>ϕ 8 at 100 mm</td>
</tr>
<tr>
<td>A2</td>
<td>150×300</td>
<td>1225</td>
<td>4.08</td>
<td>35.3</td>
<td>3ϕ 12 mm (.83%)</td>
<td>3ϕ 12 mm (.83%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
<tr>
<td>A3</td>
<td>150×300</td>
<td>1225</td>
<td>4.08</td>
<td>39</td>
<td>3ϕ 14 mm (1.13%)</td>
<td>3ϕ 14 mm (1.13%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
<tr>
<td>A4</td>
<td>150×300</td>
<td>1225</td>
<td>4.08</td>
<td>28.8</td>
<td>2ϕ 12 mm (.55%)</td>
<td>1ϕ 14 mm (.38%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
<tr>
<td>A5</td>
<td>150×300</td>
<td>1225</td>
<td>4.08</td>
<td>33.1</td>
<td>3ϕ 12 mm (.83%)</td>
<td>2ϕ 12 mm (.55%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
<tr>
<td>A6</td>
<td>150×300</td>
<td>1225</td>
<td>4.08</td>
<td>35.8</td>
<td>3ϕ 14 mm (1.13%)</td>
<td>2ϕ 14 mm (.75%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
</tbody>
</table>
Figure 3-2 Reinforcement details of A-Series Specimens

(From Response-2000)
3.2.1.2 Concrete

The concrete compressive strength, $f'_c$, used to build A-series beams was determined by testing three 150 mm by 150 mm cubes filled from the batch that was used to cast the specimens. The cube compressive strength varied from 32.3 MPa to 35.8 MPa. Because the programs used to perform the analyses use cylinder strength, a reduction factor of 0.8 was used to convert the cube compressive to cylinder compressive strength. The aggregate size was not specified in the report so it was assumed to be 10 mm.

3.2.1.3 Reinforcement

A-Series specimens consisted of three beams with symmetrical reinforcement ratio and another three with asymmetrical reinforcement ratio. Beams A3 and A6 were reinforced using bars with larger diameter compared to other four beams. Longitudinal reinforcement bars were placed in accordance to ACI 318-05 code requirements for development length. In addition to longitudinal reinforcement, shear reinforcement bars having diameter of 8 mm were used to prevent any premature failure due to shear. The yielding strength, $f_y$, ultimate strength, $f_u$, and ultimate strain, $\varepsilon_{\text{rupture}}$, of the reinforcement bars are shown in Table 3.2.

Table 3-2 Steel Properties of A-Series Beams

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>Area (mm²)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$\varepsilon_{\text{rupture}}$ (10⁻³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>50</td>
<td>290</td>
<td>455</td>
<td>330</td>
</tr>
<tr>
<td>12</td>
<td>113</td>
<td>350</td>
<td>540</td>
<td>260</td>
</tr>
<tr>
<td>14</td>
<td>154</td>
<td>340</td>
<td>535</td>
<td>270</td>
</tr>
</tbody>
</table>
3.2.2 B-Series Specimen Properties

3.2.2.1 Introduction

The B-series specimens consisted of three beams, B1, B2, and B3. The cross sectional area of specimens was the same as A-series beams but they had a longer span so the effect of beam to span ratio on the flexural capacity was examined. The clear span of beam B1 was 1975 mm and the clear span of beam B2 and B3 was 2725 mm. All the specimens had a rectangular cross section with an overall width of 150 mm and height of 300 mm. The beams elevation view and plan view are shown in Figure 3.3.

![Plan View](image1)

**Figure** Figure 3.3 Elevation and plan view of B-Series specimens

(Su et al., 2009)

The B-Series specimens all used 14 mm diameter longitudinal bars placed in top and bottom of the beams. B1 and B2 beams had symmetrical reinforcement ratio and B3 had asymmetrical reinforcement ratio. The specimens properties are summarized in Table 3.3 and the cross sections are shown in Figure 3.4.
Table 3-3 B-Series Specimen properties

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>b × h (mm)</th>
<th>$l_n$ (mm)</th>
<th>$l_n / h$</th>
<th>$f_{cu}$ (MPa)</th>
<th>Top reinforcement (ratio)</th>
<th>Bottom reinforcement (ratio)</th>
<th>Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>150×300</td>
<td>1975</td>
<td>6.58</td>
<td>23.2</td>
<td>3φ 14 mm (1.13%)</td>
<td>3φ 14 mm (1.13%)</td>
<td>φ 8 at 100 mm</td>
</tr>
<tr>
<td>B2</td>
<td>150×300</td>
<td>2725</td>
<td>9.08</td>
<td>24.1</td>
<td>3φ 14 mm (1.13%)</td>
<td>3φ 14 mm (1.13%)</td>
<td>φ 8 at 120 mm</td>
</tr>
<tr>
<td>B3</td>
<td>150×300</td>
<td>2725</td>
<td>9.08</td>
<td>26.4</td>
<td>3φ 14 mm (1.13%)</td>
<td>2φ 14 mm (.75%)</td>
<td>φ 8 at 120 mm</td>
</tr>
</tbody>
</table>

**Figure 3-4 Reinforcement details of B-Series Specimens**

(From Response-2000)
3.2.2.2 Concrete

The compressive strength of the B-series specimens was less than A-series beams so the effect of the beam axial stiffness on the membrane action could also be examined. The compressive strength varied from 23.2 MPa to 26.4 MPa, determined by testing and averaging the strength of three 150 mm by 150 mm cubes (See Table 3.3).

3.2.2.3 Reinforcement

The yielding strength, $f_y$, ultimate strength, $f_u$, and ultimate strain, $\varepsilon_{\text{rupture}}$, of the reinforcement bars placed in B-Series specimens are shown in Table 3.2. The clear cover for all the specimens was specified to be 20 mm.
3.2.3 C-Series Specimen Properties

3.2.3.1 Introduction

The C-series specimens were constructed and tested to investigate the effect of the loading rate on development of membrane action in horizontally restrained beams. For this reason three beams having identical geometry, longitudinal and transverse reinforcement ratio were subjected to center point loading in displacement control mode. These specimens had the same clear span length of 1225 mm similar to A-Series beams. The overall width of the specimens was 100 mm and they all had a total height of 200 mm (See Figure 3.5).

![Plan View](Su et al., 2009)

![Elevation View](Su et al., 2009)

Figure 3-5 Elevation view and plan of C-Series specimens

All the beams had a symmetrical reinforcement ratio of 1.13%. The top reinforcement layer consisted of three bars having 12 mm diameter and bottom layer was consisted of two bars having a 14 mm diameter. Stirrups were placed at 80 mm along the span. The Specimen reinforcement arrangement is summarized in Figure 3.6 and the material and geometrical properties are shown in Table 3.4.
Table 3-4 C-Series Specimen properties

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>$b \times h$ mm$^2$</th>
<th>$l_n$ mm</th>
<th>$l_n/h$</th>
<th>$f_{cu}$ MPa</th>
<th>Top reinforcement (ratio)</th>
<th>Bottom reinforcement (ratio)</th>
<th>Stirrups</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>100×200</td>
<td>1225</td>
<td>6.12</td>
<td>19.9</td>
<td>3ϕ12 mm (1.3%)</td>
<td>2ϕ14 mm (1.3%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
<tr>
<td>C2</td>
<td>100×200</td>
<td>1225</td>
<td>6.12</td>
<td>21</td>
<td>3ϕ12 mm (1.3%)</td>
<td>2ϕ14 mm (1.3%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
<tr>
<td>C3</td>
<td>100×200</td>
<td>1225</td>
<td>6.12</td>
<td>20.4</td>
<td>3ϕ12 mm (1.3%)</td>
<td>2ϕ14 mm (1.3%)</td>
<td>ϕ 8 at 80 mm</td>
</tr>
</tbody>
</table>

Figure 3-6 Reinforcement details of C-Series Specimens

(From Response-2000)
3.2.3.2 Concrete

The C-series specimens were made by using a ready mix design having compressive strengths of 19.9 to 20.4 MPa. The strength of the concrete was determined by testing 150 mm by 150 mm cubes filled from the batch that was used to cast the specimens.

3.2.3.3 Reinforcement

The yielding strength, $f_y$, ultimate strength, $f_u$, and ultimate strain, $\varepsilon_{\text{rupture}}$, of the reinforcement bars placed in C-series specimens are shown in Table 3.2. The Clear cover was specified to be 20 mm.
3.3 Test Program

3.3.1 Test Set-Up

The paper reports that the support conditions were arranged so the beams ends were allowed to displace one millimeter after imposing 1000 kN axial compression to the horizontal struts placed at each end of the beams (See Figure 3.7). The experimental boundary condition also provided a rotational stiffness of 17500 kNm/rad. In order to prevent the twisting of the specimens during the test, two bearing rollers were placed on each side of the beam at location of the applied load.

Figure 3-7 Test Support Conditions

(Su et al., 2009)
3.3.2 Testing Procedure and Measurements

All the specimens were subjected to center-point loading in displacement-control mode. A and B series beams were subjected to the constant loading rate of 5 mm/min until the failure of beams. C-series specimens were subjected to three different loading rates of 0.2, 2 and 20 mm/min. During loading of the specimens, in addition to the applied load $P$, measured by using load cells placed within the actuator, the horizontal reaction $N$ and vertical reaction $F$ were also measured so the applied moments at different location of each specimen could be determined. Details of the test set up and support conditions are shown in Figure 3.8.

![Figure 3-8 Test loading conditions and measurements](Su et al., 2009)
3.4 Discussion of Test Result

This section focuses on the behavior and response of the specimens discussed in the previous section. The load-deflection response of each beam is presented. Test results are summarized in Table 3.5 showing the cracking load $P_{cr}$, yielding load at supports, $P_y$, peak load, $P_u$, displacement at peak load, $\delta_u$, horizontal reaction at peak load, $N_u$, maximum compressive horizontal reaction, $N_{max}$ and displacement at maximum compressive horizontal reaction, $\delta_{max}$.

Table 3-5 Test Results

<table>
<thead>
<tr>
<th>Beam</th>
<th>$P_{cr}$ (kN)</th>
<th>$P_y$ (kN)</th>
<th>$P_u$ (kN)</th>
<th>$\delta_u$ (mm)</th>
<th>$N_u$ (kN)</th>
<th>$N_{max}$ (kN)</th>
<th>$\delta_{max}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>26</td>
<td>117</td>
<td>168</td>
<td>48</td>
<td>281</td>
<td>388</td>
<td>87.8</td>
</tr>
<tr>
<td>A2</td>
<td>30</td>
<td>148</td>
<td>221</td>
<td>56.4</td>
<td>318</td>
<td>324</td>
<td>59.3</td>
</tr>
<tr>
<td>A3</td>
<td>29</td>
<td>152</td>
<td>246</td>
<td>76.4</td>
<td>296</td>
<td>305</td>
<td>87.8</td>
</tr>
<tr>
<td>A4</td>
<td>24</td>
<td>82.3</td>
<td>147</td>
<td>65</td>
<td>309</td>
<td>344</td>
<td>93.5</td>
</tr>
<tr>
<td>A5</td>
<td>29</td>
<td>129</td>
<td>198</td>
<td>70.7</td>
<td>340</td>
<td>393</td>
<td>108</td>
</tr>
<tr>
<td>A6</td>
<td>27</td>
<td>153</td>
<td>226</td>
<td>69.2</td>
<td>177</td>
<td>191</td>
<td>89.1</td>
</tr>
<tr>
<td>B1</td>
<td>13</td>
<td>105</td>
<td>125</td>
<td>100</td>
<td>211</td>
<td>225</td>
<td>146</td>
</tr>
<tr>
<td>B2</td>
<td>10</td>
<td>73.2</td>
<td>82.9</td>
<td>102</td>
<td>190</td>
<td>210</td>
<td>125</td>
</tr>
<tr>
<td>B3</td>
<td>9.9</td>
<td>65.0</td>
<td>74.7</td>
<td>85.5</td>
<td>172</td>
<td>210</td>
<td>150</td>
</tr>
<tr>
<td>C1</td>
<td>8.0</td>
<td>48.2</td>
<td>60.9</td>
<td>33.7</td>
<td>91.6</td>
<td>108</td>
<td>62.5</td>
</tr>
<tr>
<td>C2</td>
<td>9.1</td>
<td>-</td>
<td>64.9</td>
<td>33.5</td>
<td>96.4</td>
<td>117</td>
<td>65.4</td>
</tr>
<tr>
<td>C3</td>
<td>10.2</td>
<td>-</td>
<td>68.6</td>
<td>28.7</td>
<td>108</td>
<td>134</td>
<td>60.0</td>
</tr>
</tbody>
</table>
3.4.1 A-Series Specimens

Figures 3.9 and 3.11 show the measured vertical load versus displacement responses. The tested specimens exhibited a linear load-displacement response at initial stage, after cracks and before yielding of reinforcement. Yielding of bottom reinforcement at midspan followed after yielding of top reinforcement at support, resulting in noticeable decrease in the stiffness of the load versus deflection responses. As the applied load increased, beams failed through rupture of the bottom reinforcement at midspan.

Figures 3.10 and 3.12 show the measured horizontal reaction versus displacement responses. The axial compression reaction was negligible at the initial stage, but increased notably after flexural cracking occurred indicating the influence of membrane action. The maximum axial compression force was measured when concrete crushed at the midspan.

Based on the test results, comparing beam A1 to A4, A2 to A5 and A3 to A6, all the beams exhibited the similar behavior but the load carrying capacity of the beams having less bottom reinforcement ratio was lower, the imposed horizontal force and deflection at midspan was higher.
Figure 3-9 Vertical Load-Displacement Responses for A1, A2 and A3

(Su et al., 2009)

Figure 3-10 Horizontal Reaction-Displacement Responses for A1, A2 and A3

(Su et al., 2009)
Figure 3-11 Vertical Load-Displacement Responses for A4, A5 and A6

(Su et al., 2009)

Figure 3-12 Horizontal Reaction-Displacement Responses for A4, A5, and A6

(Su et al., 2009)
3.4.2 B-Series Specimens

The B-series specimens were tested to investigate the effect of span to depth ratio on the beams load carrying capacity (See Table 3.3). Load-deflection responses of beams, subjected to center deflection are shown in Figure 3.13. The ultimate load capacity of beam B1 with a span length of 1975 mm was 125 kN which was almost 1.5 times the load capacity of the beam B2 having a long span of 2725 m.

The horizontal reaction of B-series beams are shown in Figure 3.14. Based on the test results, short span and low reinforcement ratio contribute to higher load imposed on boundaries. For instance, the horizontal reaction imposed by B1 was 211 kN but in case of B2 it was 190 kN. Comparing B2 and B3, maximum axial load imposed by B2 was 190 KN but the maximum applied by B3 having less longitudinal reinforcement ratio was 172 kN.

The influence of the span length on load carrying capacity, horizontal reaction and center displacement were investigated by comparing beams A3, B1 and B2 and results are presented in Figures 3.15 and 3.16. All three cases had an identical and symmetric top and bottom longitudinal reinforcement ratio of 1.13% but varying span length. Span length of the beam A3 was 1225 m and the span length of beams B1 and B2 was 1725 mm, so the clear span of beam was almost half of the B1 and B2 beams. Based on the test results, decreasing the span length by almost 50% resulted in increasing the load carrying capacity by 160%, increasing the horizontal reaction by 40% and decreasing the deflection at ultimate load by 108%.
Figure 3-13 Vertical Load-Displacement Responses for B-Series Specimens

(Su et al., 2009)

Figure 3-14 Horizontal Reaction-Displacement Responses for B-Series Specimens

(Su et al., 2009)
Figure 3-15 Vertical Load-Displacement Responses for B1, B2 and A3 Series Specimens

(Su et al., 2009)

Figure 3-16 Horizontal Reaction-Displacement Responses for B1, B2 and A3 Series Specimens

(Su et al., 2009)
3.4.3 C-Series Specimens

The C-series beams were tested to investigate the effect of the loading rate on specimen behavior. For this purpose, beams were tested by applying three different loading rates and the results are shown in Figures 2.17 and 2.18. All the specimens had identical geometric properties and reinforcement ratio. The ultimate load carrying capacity of the beam C3 subjected to highest loading rate was 10% higher than C1 and 6.5% higher than beam C2. Maximum horizontal reaction at ultimate load in beam C1 was 91.6 kN, in C2 was 96.4 kN and in C3 was 108 kN, so increasing the loading rate resulted in increased horizontal reaction. For instance, comparing beams C1 and C3, horizontal force imposed to boundaries in beam C3 was 18% higher than C1. Deflection at ultimate load was almost the same in all three cases. Hence it can be concluded that loading rate had a small effect on the beams and thus can be expected to have a small effect on fill fences.
Figure 3-17 Vertical Load-Displacement Responses for C-Series Specimens

(Su et al., 2009)

Figure 3-18 Horizontal Reaction-Displacement Responses for C-Series Specimens

(Su et al., 2009)
Chapter 4  
Numerical Modeling of the Specimens

4 Introduction

4.1 Introduction

The twelve specimens described in chapter 3 were modeled for finite element analyses. Twelve finite element models were used representing the beams, depending on their clear span, depth, height and reinforcement ratio. The purpose of the modeling was to obtain the theoretical load versus displacement response of the beams.

4.2 Augustus-2

For the finite element analysis, program Augustus-2 was used. This program is a two-dimensional finite element program developed at the University of Toronto by Professor E. C. Bentz based on a fibre model for axial load and moment and the simplified Modified Compression Field Theory (MCFT) method, which implemented in CSA A23.3-04, code for moment behavior.

This program models the shear response, taking into account the nonlinear behavior of reinforced concrete in axial load, shear and flexure, requires a reduced number of elements and considers the member elongation due to bending. These are some of the advantages of this program compared to conventional linear finite element programs.

This program reads the input data from the job file, which is a text file, and Response-2000 input files. Response-2000 also written by Professor E.C. Bentz, allows for sectional analysis of reinforced concrete members subjected to axial load and moment, and feeds these behaviors into Augustus-2. The job file allows the user to input data relating to the structure mesh, truss elements, boundary conditions and applied load and displacement. The user can use Response-2000 to define the geometry of the cross section, material properties and location of reinforcement. This program is also able to retrieve the output files and plot various kinds of graphs and section profiles.
The elements used in this program consist of four nodes, a shear panel in the middle, and for purpose of understanding behavior, can be thought of as having four truss bars around the shear panel as shown in Figure 4.1. The nodes can be displaced in two perpendicular direction, x and y, and accordingly have two degree of freedom. For the axial and flexural response the Response-2000 based elements element can be considered as if there is a truss bar between nodes 2 and 4 and 1 and 3 which can only carry axial load and be displaced along its own axis.

Figure 4-1 Augustus-2 Element Components
The uniaxial tension and compression cases are shown in Figures 4.2A and 4.2B. When the element is in tension, the top and bottom truss bars elongate and when it is in compression, top and bottom bars shorten.

Elements in pure positive and negative bending moments are shown in Figures 4.3A and 4.3B. In positive moment, top truss bar shorten and bottom truss bar elongate. Converse is true for negative moment.
The element in pure shear is shown in Figure 4.4.

![Figure 4-4 Element in positive shear](From Yeung, 2008)

The way that the deformations shown in Figures 4.2 to 4.4 are converted to forces are explained in the thesis written by Yeung (2008).
4.3 Numerical Modeling of Specimens

The specimens described in chapter 3, were modeled using the Augustus-2 program. Twelve finite element meshes were used to represent the specimens, depending on their geometrical properties and reinforcement ratios. Three regions having different transverse reinforcement configuration were defined modeling these specimens (See Figure 4.5). Region 1, representing the short columns at the edges where the rotational and axial restraints were applied, was modeled using a compressive and tensile strength of 1000 MPa, so any crack formation could be prevented. Region 2 is located along the clear span having different arrangement of transverse reinforcement. Shear reinforcement bars were spaced more closely near the support and applied load so the formation of diagonal cracks and as a result the premature failure due to shear can be prevented. The transverse reinforcement ratio in this region was .75% which is three times higher than the rest of the clear span. The length of this region was selected to be about 0.9\(d_y = 0.9d\). Region 3 was defined using the same material properties and reinforcement ratio as the tested beams. A total number of 56 rectangular elements in total, two elements in region A with \(\Delta x = 150\) and 50 elements in Region 2 and 3, having \(\Delta x = 24.5\) mm in region B and C, and five truss elements were defined to model the support column, clear span and axial and rotational restraints of the beams. Truss bars A and B were used to model the rotational stiffness and truss bars C, D and E were defined to model the axial stiffness. Detailed information of the material regions are shown in Figure 4.5 and the model details are summarized in Table 4.1.

![Figure 4-5 Material regions for A-Series Specimens](From Augustus-2)
Table 4-1 Finite Element Analysis Details for A, B and C-Series specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of Nodes</th>
<th>Number of Elements</th>
<th>Number of Rectangular Elements</th>
<th>Number of Truss Elements</th>
<th>Number of Concrete Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>A, B and C-Series</td>
<td>112</td>
<td>57</td>
<td>52</td>
<td>5</td>
<td>3</td>
</tr>
</tbody>
</table>

### 4.3.1 Restraint Conditions

Restraint conditions consisted of a pinned support at the supports (nodes 1, 3 and 4) to limit the vertical and lateral movements, pinned roller support at the bottom of the side column (node 2) to limit the vertical movement and pinned roller support at the mid span of the beam (nodes 5 and 6) to limit the lateral movement (See Figure 4.6).

![Figure 4-6 Finite Element representation of support conditions for A, B and C-series specimens](From Augustus-2)
4.3.2 Axial and Rotational Stiffness

The axial stiffness of the tested beams was reported to be 1000 kN/mm. This stiffness was implemented to the model by defining two diagonal truss elements, C and D, having a very high stiffness and one axial truss element, E. The stiffness was set by, placing pinned support at node 1 to limit the horizontal and vertical and roller support at nodes 2 to 5 to limit the vertical movements. 1 mm axial displacement was imposed at the nodes 5 and 6 to set the axial stiffness of truss E. Using this value, however resulted in a very high load carrying capacity compared to the test result. The axial stiffness of element E until the results matched well. Future research will be needed to determine why the optimum value was so different from the published value.

Figure 4-7 Representation of support conditions employed to find the axial stiffness of element E

(From Augustus-2)

The rotational stiffness, k, was reported to be 17500 kN.m/rad and this was implemented to the model by placing to truss bars, A and B, and a roller pinned at support column (See Figure 4.8). Knowing the stiffness of the bars, E, length of the support column, 2 a, and length of the truss bars, L, the cross sectional area, A, of the bars can be calculated using the following equations. Detailed information are shown in Table 4.2.
Figure 4-8 Truss bars used to define the axial and rotational stiffness

\[
F = \theta . a \cdot \frac{AE}{L}
\]

\[
k. \theta = M \quad \Rightarrow \quad k = \frac{2a^2AE}{L}
\]

\[
M = F . 2a
\]

Table 4-2 Truss bars properties of A, B, and C-Series models

<table>
<thead>
<tr>
<th>Truss Bar</th>
<th>Area (mm²)</th>
<th>E (MPa)</th>
<th>L (mm)</th>
<th>( f_y ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>584</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B</td>
<td>584</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>C</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>D</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>E</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A1</td>
<td>130.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>A2</td>
<td>130.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>A3</td>
<td>112.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>A4</td>
<td>112.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>A5</td>
<td>148.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>A6</td>
<td>60.0</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B1</td>
<td>117</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B2</td>
<td>61.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B3</td>
<td>61.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>C1</td>
<td>61.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>C2</td>
<td>61.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>C3</td>
<td>61.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
</tbody>
</table>
4.3.3 Material Properties

The cube compressive strength of the concrete reported in the paper was converted to cylinder compressive strength using a reduction factor of 0.8. The maximum aggregate size was not specified in the report so it was assumed to be 19 mm. To avoid underestimating the flexural ductility after yield, the effect of confinement on stress strain relationship of the concrete in flexural compression was modified in Region 2 (See Figure 4.5). In this region the concrete had its post-peak response replaced with a stress Plato out to strain of 100 mm/m (See Figure 4.9A). Figure 4.9B describes the stress strain curve used in Region 2 and Region3 of beam A1.

![Concrete Stress-strain curve](image)

Figure 4-9 A: Concrete Stress-strain curve used in Region 2, B: Concrete Stress-strain curve used in Region 3
4.4 Analytical Results

4.4.1 A-Series Specimens

This section represents the analytical results of the A-series specimens. The observed vertical load and horizontal reaction versus midspan deflection of each specimen is compared to Augustus-2 analytical results and given in Figures 4.10 to 4.21. The theoretical response had an initial stiffness similar to the observed response. Flexural cracking were predicted to occur at the midspan and at the interface of the beam and support; First yielding occurred at the bottom reinforcement at the midspan due to positive moment and then at the top reinforcement moment at the support due to negative bending. Cushing of concrete at the support was predicted to cause a significant drop in the load capacity of the beam; the same behavior was observed in the test. Flexural failure was predicted to occur at the midspan which was the same as the test observation.

Horizontal reaction versus midspan deflection was not predicted as well. The paper reported the axial stiffness of 1000 kN/mm. There are two interpretations of this, however one is that applying a 1000 kN axial load at the midspan will cause 1 mm axial displacement, alternatively the author may have meant, if the load was applied to the support itself but not through the beam, the support would move 1 mm. The analyses were conducted based on the first assumption. Trial and error was used to determine the axial stiffness bet match the load deformation curve. Cracking load, $P_{cr}$, yielding load at support, $P_y$, Ultimate load capacity, $P_{ult}$, horizontal reaction at the ultimate load, $N_u$, and maximum horizontal reaction, $N_{max}$, were compared and results are shown in Table 4.3.
Table 4-3 A-Series Beams Response Comparison

<table>
<thead>
<tr>
<th>Beam Response</th>
<th>Axial Stiffness (kN/mm)</th>
<th>Rotational Stiffness (kN.m/rad)</th>
<th>$P_{cr}$ (kN)</th>
<th>$P_y$ at support (kN)</th>
<th>$P_u$ (kN)</th>
<th>$N_u$ (kN)</th>
<th>$N_{max}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 Test</td>
<td>1000</td>
<td>17500</td>
<td>26</td>
<td>117</td>
<td>168</td>
<td>281</td>
<td>388</td>
</tr>
<tr>
<td>A1 Augustus-2</td>
<td>87</td>
<td>17500</td>
<td>29</td>
<td>138.4</td>
<td>187</td>
<td>365</td>
<td>365</td>
</tr>
<tr>
<td>A1 Test/Pred.</td>
<td>11.5</td>
<td>1.0</td>
<td>.89</td>
<td>.84</td>
<td>.89</td>
<td>.77</td>
<td>1.06</td>
</tr>
<tr>
<td>A2 Test</td>
<td>1000</td>
<td>17500</td>
<td>30</td>
<td>221</td>
<td>221</td>
<td>318</td>
<td>324</td>
</tr>
<tr>
<td>A2 Augustus-2</td>
<td>87</td>
<td>17500</td>
<td>37.4</td>
<td>165</td>
<td>233</td>
<td>419</td>
<td>365</td>
</tr>
<tr>
<td>A2 Test/Pred.</td>
<td>11.5</td>
<td>1.0</td>
<td>.80</td>
<td>1.33</td>
<td>.95</td>
<td>.76</td>
<td>.89</td>
</tr>
<tr>
<td>A3 Test</td>
<td>1000</td>
<td>17500</td>
<td>29</td>
<td>246</td>
<td>246</td>
<td>296</td>
<td>305</td>
</tr>
<tr>
<td>A3 Augustus-2</td>
<td>75</td>
<td>17500</td>
<td>37</td>
<td>230</td>
<td>284</td>
<td>540</td>
<td>540</td>
</tr>
<tr>
<td>A3 Test/Pred.</td>
<td>13.3</td>
<td>1.0</td>
<td>.78</td>
<td>1.07</td>
<td>.87</td>
<td>.55</td>
<td>.56</td>
</tr>
<tr>
<td>A4 Test</td>
<td>1000</td>
<td>17500</td>
<td>24</td>
<td>147</td>
<td>168</td>
<td>309</td>
<td>344</td>
</tr>
<tr>
<td>A4 Augustus-2</td>
<td>75</td>
<td>17500</td>
<td>27.8</td>
<td>111</td>
<td>159</td>
<td>385</td>
<td>385</td>
</tr>
<tr>
<td>A4 Test/Pred.</td>
<td>13.3</td>
<td>1.0</td>
<td>.86</td>
<td>1.32</td>
<td>1.06</td>
<td>.80</td>
<td>.89</td>
</tr>
<tr>
<td>A5 Test</td>
<td>1000</td>
<td>17500</td>
<td>29</td>
<td>198</td>
<td>168</td>
<td>340</td>
<td>393</td>
</tr>
<tr>
<td>A5 Augustus-2</td>
<td>99</td>
<td>17500</td>
<td>33.8</td>
<td>156</td>
<td>214</td>
<td>469</td>
<td>469</td>
</tr>
<tr>
<td>A5 Test/Pred.</td>
<td>10.1</td>
<td>1.0</td>
<td>.86</td>
<td>1.27</td>
<td>.78</td>
<td>.72</td>
<td>.84</td>
</tr>
<tr>
<td>A6 Test</td>
<td>1000</td>
<td>17500</td>
<td>27</td>
<td>226</td>
<td>226</td>
<td>177</td>
<td>191</td>
</tr>
<tr>
<td>A6 Augustus-2</td>
<td>40</td>
<td>17500</td>
<td>38.4</td>
<td>199</td>
<td>250</td>
<td>460</td>
<td>460</td>
</tr>
<tr>
<td>A6 Test/Pred.</td>
<td>25</td>
<td>1.0</td>
<td>.70</td>
<td>1.13</td>
<td>.90</td>
<td>.28</td>
<td>.42</td>
</tr>
<tr>
<td>Average Ratio</td>
<td>14.1</td>
<td>1</td>
<td>.81</td>
<td>1.16</td>
<td>.91</td>
<td>.65</td>
<td>.78</td>
</tr>
<tr>
<td>Coefficient of Variation (%)</td>
<td>38.75</td>
<td>0</td>
<td>8.56</td>
<td>16.26</td>
<td>10.21</td>
<td>30.98</td>
<td>30.69</td>
</tr>
</tbody>
</table>
Figure 4-10 A1: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-11 A1: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
**Figure 4-12 A2: Vertical load-Displacement prediction of Augustus-2 vs. Test Result**

**Figure 4-13 A2: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result**
Figure 4-14 A3: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-15 A3: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
Figure 4-16 A4: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-17 A4: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
Figure 4-18 A5: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-19 A5: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
Figure 4-20 A6: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-21 A6: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
4.4.2 B-Series Specimens

In this section the theoretical response of B-series specimens is compared to the actual test results. Vertical load and horizontal reaction versus midspan deflections are shown in Figures 2.22 to 2.27. The overall behavior of the beams was the same as A-series specimens, but due to the longer clear span beam response was more gradual. Results are shown in table 4.4.

Table 4-4 B-Series Beams Response Comparison

<table>
<thead>
<tr>
<th>Beam Response</th>
<th>Axial Stiffness (kN/mm)</th>
<th>Rotational Stiffness (kN.m/rad)</th>
<th>P&lt;sub&gt;cr&lt;/sub&gt; (kN)</th>
<th>P&lt;sub&gt;y at support&lt;/sub&gt; (kN)</th>
<th>P&lt;sub&gt;u&lt;/sub&gt; (kN)</th>
<th>N&lt;sub&gt;y&lt;/sub&gt; (kN)</th>
<th>N&lt;sub&gt;max&lt;/sub&gt; (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1 Test</td>
<td>1000</td>
<td>17500</td>
<td>13</td>
<td>105</td>
<td>125</td>
<td>211</td>
<td>255</td>
</tr>
<tr>
<td>Augustus-2</td>
<td>78</td>
<td>17500</td>
<td>14</td>
<td>77.2</td>
<td>145</td>
<td>292.7</td>
<td>320</td>
</tr>
<tr>
<td>Test/Pred.</td>
<td>21.27</td>
<td>1</td>
<td>.93</td>
<td>1.36</td>
<td>.86</td>
<td>.72</td>
<td>.80</td>
</tr>
<tr>
<td>B2 Test</td>
<td>1000</td>
<td>17500</td>
<td>10</td>
<td>73.2</td>
<td>83</td>
<td>211</td>
<td>225</td>
</tr>
<tr>
<td>Augustus-2</td>
<td>41</td>
<td>17500</td>
<td>12</td>
<td>51.8</td>
<td>109</td>
<td>319</td>
<td>325</td>
</tr>
<tr>
<td>Test/Pred.</td>
<td>24.39</td>
<td>1</td>
<td>.83</td>
<td>1.41</td>
<td>.76</td>
<td>.66</td>
<td>.69</td>
</tr>
<tr>
<td>B3 Test</td>
<td>1000</td>
<td>17500</td>
<td>9.9</td>
<td>65</td>
<td>74.7</td>
<td>281</td>
<td>388</td>
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<tr>
<td>Augustus-2</td>
<td>41</td>
<td>17500</td>
<td>14</td>
<td>41.8</td>
<td>98.6</td>
<td>316</td>
<td>316</td>
</tr>
<tr>
<td>Test/Pred.</td>
<td>24.39</td>
<td>1</td>
<td>.71</td>
<td>1.56</td>
<td>.76</td>
<td>.89</td>
<td>1.23</td>
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<tr>
<td>Average</td>
<td>23.35</td>
<td>1</td>
<td>.82</td>
<td>1.44</td>
<td>.79</td>
<td>.76</td>
<td>.91</td>
</tr>
<tr>
<td>Coefficient of Variation (%)</td>
<td>7.71</td>
<td>0</td>
<td>13.37</td>
<td>7.21</td>
<td>7.27</td>
<td>15.77</td>
<td>31.45</td>
</tr>
</tbody>
</table>
**Figure 4-22 B16: Vertical load-Displacement prediction of Augustus-2 vs. Test Result**

**Figure 4-23 B1: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result**
Figure 4-24 B2: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-25 B2: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
Figure 4-26 B3: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-27 B3: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
4.4.3 C-Series Specimens

The C-series specimens subjected to different loading rates, but since time has no effect on the predicted response of the beams, so only one analysis was conducted and compared to C1. The theoretical response had an initial stiffness equivalent to the observed response. Flexural cracking predicted to occur at the midpan and at the interface of the beam and support; First yielding occurred at the bottom reinforcement at the midspan due to positive moment and then at the top reinforcement moment at the support due to negative bending. Crushing of concrete at the support predicted to cause a significant drop in the load capacity of the beam; same behavior was observed in the test. Flexural failure was predicted to occur at the midspan which was the same as the test result. The theoretical Responses using Augustus-2 were compared to the actual test and are shown in Figures 4.28 and 4.29. Results are found to be predicted reasonably well and are summarized in Table 4.5.

Table 4-5 C-Series Beams Response Comparison

<table>
<thead>
<tr>
<th>Beam Response</th>
<th>Axial Stiffness (KN/mm)</th>
<th>Rotational Stiffness (KN-m/rad)</th>
<th>P_cr (KN)</th>
<th>P_y at support (KN)</th>
<th>P_u (KN)</th>
<th>N_max at P_u (KN)</th>
<th>N_max (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1 Test</td>
<td>1000</td>
<td>17500</td>
<td>8.0</td>
<td>48.2</td>
<td>60.9</td>
<td>91.6</td>
<td>388</td>
</tr>
<tr>
<td>Augustus2</td>
<td>41.00</td>
<td>17500</td>
<td>10.0</td>
<td>41.40</td>
<td>73.20</td>
<td>146</td>
<td>365</td>
</tr>
<tr>
<td>Ratio</td>
<td>24.39</td>
<td>1.0</td>
<td>0.8</td>
<td>1.16</td>
<td>0.83</td>
<td>0.63</td>
<td>1.06</td>
</tr>
<tr>
<td>C2 Test/Pred.</td>
<td>1000</td>
<td>17500</td>
<td>9.10</td>
<td>-</td>
<td>64.9</td>
<td>96.4</td>
<td>324</td>
</tr>
<tr>
<td>Augustus2</td>
<td>41.00</td>
<td>17317</td>
<td>10.0</td>
<td>41.40</td>
<td>73.20</td>
<td>1.77</td>
<td>365</td>
</tr>
<tr>
<td>Test/Pred.</td>
<td>24.39</td>
<td>1.01</td>
<td>0.91</td>
<td>-</td>
<td>0.89</td>
<td>54.46</td>
<td>0.89</td>
</tr>
<tr>
<td>C3 Test</td>
<td>1000</td>
<td>17500</td>
<td>10.20</td>
<td>-</td>
<td>68.60</td>
<td>108</td>
<td>305</td>
</tr>
<tr>
<td>Augustus2</td>
<td>41.00</td>
<td>17200</td>
<td>10.0</td>
<td>41.40</td>
<td>73.20</td>
<td>163</td>
<td>540</td>
</tr>
<tr>
<td>Test/Pred.</td>
<td>24.39</td>
<td>1.02</td>
<td>1.02</td>
<td>-</td>
<td>0.94</td>
<td>0.66</td>
<td>0.56</td>
</tr>
<tr>
<td>Average Test/Pred.</td>
<td>24.39</td>
<td>1.01</td>
<td>0.91</td>
<td>-</td>
<td>0.86</td>
<td>18.58</td>
<td>0.83</td>
</tr>
<tr>
<td>Coefficient of Variation (%)</td>
<td>0</td>
<td>.87</td>
<td>12.09</td>
<td>-</td>
<td>5.94</td>
<td>167.19</td>
<td>30.13</td>
</tr>
</tbody>
</table>
Figure 4-28 C1: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-29 C1: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
Figure 4-30 C2 Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-31 C2: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
Figure 4-32 C3: Vertical load-Displacement prediction of Augustus-2 vs. Test Result

Figure 4-33 C3: Horizontal reaction-Displacement prediction of Augustus-2 vs. Test Result
4.4.4 Sensitivity of the analysis

To investigate the influence of the axial and rotational stiffness on the response of specimens, the same model for A1 having different horizontal support and rotational stiffnesses were used. Shown in Figure 4.30 is the predicted load versus midspan deflection. The initial stiffness was seen to be equivalent. After cracking, however, the predicted response of the beam with stronger axial support was stiffer. In all the cases, first yielding occurred at the bottom reinforcement bars at the midspan and followed by yielding of the top reinforcement bars at the support. The vertical load carrying capacity dropped by crushing of concrete at the support and final failure occurred due to fracture of the bottom reinforcement at the midspan. However, the center deflection of beam decreased notably as more axial support was provided. The cracking load, $P_{cr}$, yielding load, $P_y$, ultimate load capacity, $P_u$, and center deflection at the ultimate load, $\delta_u$, are shown in Table 4.6.

![A1 Graph](image)

Figure 4-34 Vertical load-Displacement response of beam having different horizontal support condition

<table>
<thead>
<tr>
<th>Axial Stiffness (kN/mm)</th>
<th>$P_{cr}$ (kN)</th>
<th>$P_y$ at support (kN)</th>
<th>$P_u$ (kN)</th>
<th>$\delta_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td>32.2</td>
<td>61.2</td>
<td>164.6</td>
<td>99.8</td>
</tr>
<tr>
<td>78</td>
<td>32.6</td>
<td>67.8</td>
<td>177.8</td>
<td>40.4</td>
</tr>
<tr>
<td>212</td>
<td>33.6</td>
<td>78.8</td>
<td>186.1</td>
<td>63.1</td>
</tr>
</tbody>
</table>
Detailed information of the truss bars are shown in Table 4.7.

Table 4-7 Truss bars properties of models having various axial stiffness

<table>
<thead>
<tr>
<th>Truss Bar</th>
<th>Area (mm$^2$)</th>
<th>E (MPa)</th>
<th>L (mm)</th>
<th>$f_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>584</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B</td>
<td>584</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>C</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>D</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>E</td>
<td>1</td>
<td>318</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>E</td>
<td>2</td>
<td>117</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>E</td>
<td>3</td>
<td>31.5</td>
<td>300</td>
<td>600</td>
</tr>
</tbody>
</table>

The influence of the rotational stiffness on the predicted load versus midspan deflection response of one of the tested specimens, A1, was investigated by defining truss bars, A and B (Shown in Figure 4.5), having a rotational stiffness of 35000 kN.m/rad, 17500 kN.m/rad (base case) and 4375 kN.m/rad. Shown in Figure 4.31 is the predicted response of the model. The initial stiffness was seen to be equivalent. After cracking, in the first two cases, first yielding occurred at the bottom reinforcement bars at the midspan and followed by yielding of the top reinforcement bars at the support. The vertical load carrying capacity dropped by crushing of concrete at the support and final failure occurred due to fracture of the bottom reinforcement at the midspan. However, in the third case having rotational stiffness of 4375 kN.m/rad, first yielding occurred at the bottom reinforcement bars at the midspan. Failure occurred at the support due to shear. The cracking load, $P_{cr}$, yielding load, $P_y$, ultimate load capacity, $P_u$, and center deflection at the ultimate load, $\delta_{tu}$, are shown in Table 4.8.
Figure 4-35 Load-Displacement response of A1 having different rotational stiffness

<table>
<thead>
<tr>
<th>Rotational Stiffness (kN.m/rad)</th>
<th>$P_{cr}$ (kN)</th>
<th>$P_Y$ at support (kN)</th>
<th>$P_u$ (kN)</th>
<th>$\delta_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4375</td>
<td>22.2</td>
<td>141.2</td>
<td>144.4</td>
<td>64.1</td>
</tr>
<tr>
<td>17500</td>
<td>32.6</td>
<td>67.6</td>
<td>177.8</td>
<td>63.1</td>
</tr>
<tr>
<td>52500</td>
<td>33.4</td>
<td>65.2</td>
<td>177.4</td>
<td>62.1</td>
</tr>
</tbody>
</table>

Detailed information of truss bar properties are shown in Table 4.9.

Table 4-9 Detailed information of truss bar properties of the beams having various axial stiffness

<table>
<thead>
<tr>
<th>Truss Bar</th>
<th>Area (mm$^2$)</th>
<th>E (MPa)</th>
<th>L (mm)</th>
<th>$f_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>145.8</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>2</td>
<td>583.3</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>3</td>
<td>1733.3</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>145.8</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>2</td>
<td>583.3</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>3</td>
<td>1733.3</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>C</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>D</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>E</td>
<td>130.5</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
</tbody>
</table>
Chapter 5
Analytical Modeling of Fill Fences

5 Analytical Modeling of Fill Fences

5.1 Introduction

To obtain an estimated pressure capacity of the tested fill fences installed and backfilled in the Cayeli mine, the fence shown in Figure 5.1 was analyzed using Augustus-2 program, considering a 1 m slab strip along the height of the fence, and a 3-D finite element program, VecTor4 considering the full geometry of the fence. In this chapter the theoretical load versus deflection response of the fences were predicted and compared to the field data.

Figure 5-1 Fill Fence installed in Cayeli mine
(Thompson et al., 2010)
5.2 Fill Fence Properties

Shown in Figure 5.2 is the elevation view of the tested fill fences. Both fences had an identical material properties and restraint conditions but different geometry. The long span, \( l \), height, \( h \), thickness, \( t \), and steel bars arrangements are shown in Table 5.1. The material properties and aggregate size of the structures were not specified in the report, so the shotcrete was assumed to have a normal compressive strength, \( f'_c \), of 20 MPa, and an aggregate size of 19 mm. Shear reinforcement was not provided.

![Figure 5-2 Fill fence geometry](image)

(Thompson et al., 2010)

<table>
<thead>
<tr>
<th>Fence</th>
<th>( l ) (mm)</th>
<th>( h ) (mm)</th>
<th>( t ) (mm)</th>
<th>( f'_c ) (MPa)</th>
<th>Horizontal Steel bars</th>
<th>Vertical Steel bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>685N20</td>
<td>8500</td>
<td>5600</td>
<td>300</td>
<td>20</td>
<td>19- ( \phi ) 28</td>
<td>19- ( \phi ) 28</td>
</tr>
<tr>
<td>715N22</td>
<td>8500</td>
<td>4500</td>
<td>300</td>
<td>20</td>
<td>19- ( \phi ) 28</td>
<td>21- ( \phi ) 28</td>
</tr>
</tbody>
</table>
Shown in Figure 5.3 is the reinforcement arrangement of the fences. Although, the force distribution along the short span direction is significantly higher, it’s noticed that the reinforcement ratio provided in the short span was less than the reinforcement ratio of the long span. In addition to the steel bars used to reinforced the fences, grouted bars having a nominal diameter of 33 mm and spacing of 1000 mm were placed at the boundaries to prevent the slab from punching through. Reinforcement properties, yielding strength, $f_y$, ultimate strength, $f_u$, and the modulus of elasticity, $E_s$, of the steel bars as used in the analysis are summarized Table 5.2.

![Figure 5-3 Steel bar arrangement and Restraint conditions](image)

**Table 5-2 Steel Properties of fill fences**

<table>
<thead>
<tr>
<th>Bar size</th>
<th>Nominal Diameter (mm)</th>
<th>Area (mm²)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$E_s$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$ 28</td>
<td>28</td>
<td>616</td>
<td>350</td>
<td>540</td>
<td>200000</td>
</tr>
<tr>
<td>$\phi$ 33</td>
<td>33</td>
<td>855</td>
<td>350</td>
<td>540</td>
<td>200000</td>
</tr>
</tbody>
</table>
5.3 Augustus-2 Model

5.3.1 Geometric and Material Properties
Theoretical predictions of the responses of the fill fences installed at Cayeli mine were made using the Augustus-2 program. The analysis was conducted by modeling a 1 m wide slab strip along the short span of the fill fence shown in Figure 5.4. Two models were used to analyze the fill fence constructed in stopes 685N20 and 715N22. Models had the same thickness, material properties and reinforcement ratios but different lengths.

Figure 5-4 Fill fence slab Strip along the short span
Shown in Figure 5.5 is the model developed using Augustus-2 program. Due to symmetry of the slab strip only half of the slab was modeled. One reinforced concrete region was used representing the slab strip. To prevent the any failure due to shear, transverse reinforcement bars placed at 10 mm spacing. The concrete stress-strain response was based on the Popovics model but was changed in a way that axial strain at maximum axial strength was multiplied by 50 so premature crushing of concrete could be prevented (Shown in Figure 4.9). The maximum aggregate size of the shotcrete was not reported so it was assumed to be 19 mm. The cross section view of the slab strip is shown in Figure 5.6 (All the calculations were performed for 1 m length of the span). The length, L, thickness, t, width, w, concrete compressive strength, \( f_c' \), longitudinal reinforcement ratio along the length, \( \rho_s \), used to define the model were the same as the fill fence properties and are shown in Table 5.1.

<table>
<thead>
<tr>
<th>Stope</th>
<th>L (mm)</th>
<th>w (mm)</th>
<th>t (mm)</th>
<th>( f_c' ) (MPa)</th>
<th>( \rho_s ) (%)</th>
<th>Stirrups (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>685N20</td>
<td>5600</td>
<td>1000</td>
<td>300</td>
<td>20</td>
<td>.51</td>
<td>5.0</td>
</tr>
<tr>
<td>715N22</td>
<td>4500</td>
<td>1000</td>
<td>300</td>
<td>20</td>
<td>.51</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Figure 5-5 Slab Strip along the short span of the fill fence
(From Augustus-2)
5.3.2 Restrained and Loading Conditions

To investigate the sensitivity of the model on the boundary conditions, two models were defined and results are compared. In this section Model A was fixed end, so the lateral and vertical movement was limited in one end, but since only half of the slab was modeled the other side was free for vertical but limited for lateral movement. The second model (B) was defined in chapter 4, by placing five truss bars at the slab end to limit the axial and rotational stiffness of the beam. Two truss bars, A and B, were defined to limit the rotational stiffness and three truss bars, C, D and E were defined to limit the axial stiffness, models are shown in Figure 5.7. The model was loaded by applying uniformly distributed load along the span until failure (See Figure 5.8).

Table 5-4 Truss bar Properties of models B-1 and B-2

<table>
<thead>
<tr>
<th>Truss Bar</th>
<th>Area (mm²)</th>
<th>E (MPa)</th>
<th>L (mm)</th>
<th>fy (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>1000</td>
<td>2,000,000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B-2</td>
<td>2.0</td>
<td>2.0</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>1000</td>
<td>2,000,000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B-2</td>
<td>2.0</td>
<td>2.0</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>C</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>D</td>
<td>1000</td>
<td>200000</td>
<td>250</td>
<td>600</td>
</tr>
<tr>
<td>E</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>318</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
<tr>
<td>B-2</td>
<td>117</td>
<td>200000</td>
<td>300</td>
<td>600</td>
</tr>
</tbody>
</table>
Figure 5-7 A: Fully restraint Slab strip, B: Partially Restraint Slab strip

Figure 5-8 Loading Condition
Shown in Figure 5.9 is the predicted response of the slab strip, models A and B, subjected to the same loading but different support conditions: Fixed end (A), Stiff truss Bars (B-1), and flexible truss bars (B-2) to represent the strength of a conventional slab analysis.

Figure 5-9 Theoretical prediction for the slab strip response having different boundary conditions

Based on the theoretical prediction of Augustus-2, models A and B-1 demonstrated greater strength compared to B-2. The enhanced response was due to development of compressive membrane action. In model B-1 the compressive axial force at ultimate load was 1712 kN, however, in model B-1, conversely the axial compressive force was only 140 kN prior to failure. For model B-1, first yielding at the midspan predicted to occur at the pressure of 31 kPa. For model B-2, first yielding the interface of the slab and support predicted to occur at 166 kPa. B-1 was predicted to fail at the ultimate pressure of 209 kPa and B-2 failed at the ultimate pressure of 64 kPa.
5.4 Vector 4

VecTor4 is a 3-D nonlinear finite element analysis program for reinforced concrete shell elements. This program was developed at the University of Toronto by Professor M.A. Polak, supervised by Professor F.J. Vecchio and refined by Professor F.J. Vecchio’s current Ph.D student, Trevor Hrynyk. Modeling of reinforced concrete behavior is done based on the formulation of the Modified Compression Field Theory (MCFT) (Collins and Vecchio, 1986). The quadrilateral elements adopted in this program are called Heterosis elements. These elements have nine nodes and 42 degrees of freedom; eight nodes located on the sides and one at the center. The eight side nodes have five degrees of freedom, three for translation and two for rotation. The ninth node has two degrees of freedom for rotation (see Figure 5.10). Details of the formulation are provided by Polak and Vecchio (1993).

This program reads the input data from three different types of files, 1- job file, 2- structure file, and 3- load case files. The output files can be read by a post processor called Janus developed by Professor F. J. Vecchio.

![Figure 5-10 Heterosis Element](M.A. Polak, 1993)
5.4.1 VecTor4 Slab Strip Model

In order to validate the accuracy of the VecTor4 program, a slab strip taken along the short span of the fence is modeled and analyzed using VecTor4 and the theoretical prediction of the load versus deflection response was compared to Augustus-2 response. The model is shown in Figure 5.11. The geometry, material properties, boundary and loading conditions were identical to the Augustus-2 model and are shown in Table 5.4. The 3-D finite element mesh details are shown in Table 5.5.

Figure 5-11 Slab Strip Model Using VecTor4
Table 5-5 Finite Element mesh Detail of slab strip

<table>
<thead>
<tr>
<th>Model</th>
<th>No. of Nodes</th>
<th>No. of Rectangular elements</th>
<th>No. of nodes with prescribed d.o.f.</th>
<th>No. of reinforced concrete material types</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cayeli Slab Strip</td>
<td>187</td>
<td>40</td>
<td>22</td>
<td>1</td>
</tr>
</tbody>
</table>

Slab strip length, l, width, w, thickness, t, compressive strength, $f'_c$, longitudinal reinforcement ratio, $\rho_s$, and stirrups ratio are shown in Table 5.5.

Table 5-6 Model Properties

<table>
<thead>
<tr>
<th>Fence Location</th>
<th>l (mm)</th>
<th>W (mm)</th>
<th>t (mm)</th>
<th>$f'_c$ (MPa)</th>
<th>$\rho_s$ (%)</th>
<th>Stirrups (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>685N20</td>
<td>5600</td>
<td>1000</td>
<td>300</td>
<td>20</td>
<td>.51</td>
<td>5.0</td>
</tr>
<tr>
<td>715N22</td>
<td>4500</td>
<td>1000</td>
<td>300</td>
<td>20</td>
<td>.51</td>
<td>5.0</td>
</tr>
</tbody>
</table>
5.4.2 VecTor4 Fill fence Model

Theoretical prediction of the response of the full geometry of the fill fences was made using the 3-D nonlinear finite element analysis program VecTor4. The purpose of this work was to obtain the pressure versus displacement response of the fill fence installed at Cayeli mine. For this reason, finite element models were developed representing two fill fences constructed and tested in the mine and results are compared to the actual fill fence response.

The geometry, material properties and restraints conditions of the analytical models of the fill fences installed and tested in stopes 685N20 and 715N22 at the Cayeli mine (Thompson et al., 2010) were based on the provided report from the field. The fence constructed in the stope 685N20 had a same long span, width, material properties and reinforcement ratio compared to the fence installed at the stope 715N22. However, the height was different. Shown in Figure 5.12 is a picture of the tested fill fence.

![Tested fill fence](image)

Figure 5-12 Tested fill fence

(Thompson et al., 2010)
5.4.2.1 Finite Element Mesh

Shown in Figure 5.13 is the model defined to predict the response of the fill fences using VecTor4 program. Due to symmetry, only half of the structure was modeled and only one reinforced concrete region was defined. Steel bar arrangements, compressive strength, reinforcement properties and boundary conditions were specified based on the actual fill fence properties explained in section 5.2.

![Finite Element Mesh for the Cayeli Fence](From VecTor4)

The model consisted of 40 rectangular elements, having same type of material, 187 nodal points and 52 nodes with prescribed degree of freedom.
5.4.2.2 Restraint and Loading Conditions

Restraint conditions consisted of a pinned supports along the edges to prevent the punching failure. Since only half of the structure was modeled so pinned rollers were placed along the height at the midspan allowing the fence to deflect (See Figure 5.14).

The model was subjected to uniformly distributed load up to the failure.
5.4.2.3 Sensitivity of the analyses

The sensitivity of the theoretical response of the fill fence was investigated assuming various material properties, reinforcement ratio, and restraint conditions. Results are presented in this section.

5.4.2.4 Boundary Condition

This section represents the predicted response of the VecTor4 analyses conducted to investigate the effect of the boundary conditions on the fence pressure capacity. For this purpose, the following two support conditions were employed: 1- the rotational stiffness was assumed to be infinity along the edges ($\theta_x=0, \theta_y=0$), so the rotation around the boundaries was prevented, 2- rotational stiffness was set to zero, ($\theta_x\neq0$, and $\theta_y\neq0$). Based on the results, boundary condition has high influence on the fence pressure capacity. Shown in Figure 5.15 are the predicted behaviors of the models.

![Diagram](image)

Figure 5-15 Analytical response of fill fence having different reinforcement ratio

The cracking pressure, $\sigma_{cr}$, displacement at the first crack occurs, $\delta_{cr}$, initial crack width $w_{cr}$, ultimate pressure, $\sigma_{u}$, displacement at the ultimate pressure, $\delta_{u}$, and the maximum crack width $w_{u}$ are summarized in Table 5.6.
Table 5-7  Influence of boundary conditions on fence response

<table>
<thead>
<tr>
<th>Model</th>
<th>$\sigma_{cr}$ (kPa)</th>
<th>$\delta_{cr}$ (mm)</th>
<th>$w_{cr}$ (mm)</th>
<th>$\sigma_u$ (kPa)</th>
<th>$\delta_u$ (mm)</th>
<th>$w_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No rotation</td>
<td>20</td>
<td>1.02</td>
<td>.04</td>
<td>200</td>
<td>68.15</td>
<td>5.30</td>
</tr>
<tr>
<td>Free to rotate</td>
<td>20</td>
<td>2.0</td>
<td>.03</td>
<td>60</td>
<td>30.0</td>
<td>1.54</td>
</tr>
<tr>
<td>Ratio</td>
<td>1.0</td>
<td>.51</td>
<td>1.33</td>
<td>3.33</td>
<td>2.27</td>
<td>3.44</td>
</tr>
</tbody>
</table>
5.4.2.5 Material Properties

In order to investigate the effect of the shotcrete compressive strength, $f'_c$, on the pressure versus displacement behavior of the fill fence two models were defined having different compressive strength. The models were allowed to rotate along the edges. Based on the results, the model with higher compressive strength exhibited stiffer behavior. Using a higher compressive strength delayed the crushing of the concrete and as a result the final failure. The pressure versus displacement responses are shown in Figure 5.16.

![Graph showing pressure versus displacement](image)

**Figure 5-16** Analytical response of fill fence with different restraint conditions
The cracking pressure, $\sigma_{cr}$, displacement when the first crack occurs, $\delta_{cr}$, initial crack width $w_{cr}$, ultimate pressure, $\sigma_u$, displacement at the ultimate pressure, $\delta_u$, and the maximum crack width $w_u$ are summarized in Table 5.7.

Table 5-8 Influence of material properties on fence response

<table>
<thead>
<tr>
<th>$f'_c$ (MPa)</th>
<th>$\sigma_{cr}$ (kPa)</th>
<th>$\delta_{cr}$ (mm)</th>
<th>$w_{cr}$ (mm)</th>
<th>$\sigma_u$ (kPa)</th>
<th>$\delta_u$ (mm)</th>
<th>$w_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>20</td>
<td>.98</td>
<td>.03</td>
<td>90</td>
<td>33.48</td>
<td>3.83</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
<td>2.0</td>
<td>.03</td>
<td>60</td>
<td>30.0</td>
<td>1.54</td>
</tr>
<tr>
<td>Ratio</td>
<td>1</td>
<td>.49</td>
<td>1.0</td>
<td>1.5</td>
<td>1.12</td>
<td>2.49</td>
</tr>
</tbody>
</table>
5.4.2.6 Reinforcement Ratio

This section focuses on the effect of the reinforcement ratio, $\rho_s$, on the theoretical pressure versus displacement response of the fill fence subjected to uniformly distributed pressure. For this purpose analyses were conducted based on two models having no longitudinal reinforcement and 0.6% longitudinal reinforcement ratio and results are shown in Figure 5.17.

![Graph showing analytical response of fixed fill fence having different reinforcement ratio](image)

**Figure 5-17** Analytical response of fixed fill fence having different reinforcement ratio

Based on the analytical response, if the surrounding rocks provide a stiff support around the fill fence, the influence of the reinforcement ratio will not affect the fence behavior significantly.

The cracking pressure, $\sigma_{cr}$, displacement at the cracking pressure, $\delta_{cr}$, initial crack width, $w_{cr}$, ultimate pressure capacity, $\sigma_{u}$, deflection ay the ultimate pressure, $\delta_u$, and the crack width at the ultimate pressure, $w_u$, are summarized in Table 5.8.
Table 5-9 Influence of longitudinal reinforcement ratio on fence response

<table>
<thead>
<tr>
<th>$\rho_s$ (%)</th>
<th>$f'_c$ (MPa)</th>
<th>$\sigma_{cr}$ (kPa)</th>
<th>$\delta_{cr}$ (mm)</th>
<th>$w_{cr}$ (mm)</th>
<th>$\sigma_w$ (kPa)</th>
<th>$\delta_w$ (mm)</th>
<th>$w_w$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>.68</td>
<td>20</td>
<td>20</td>
<td>1.02</td>
<td>.03</td>
<td>200</td>
<td>68.15</td>
<td>5.30</td>
</tr>
<tr>
<td>0</td>
<td>20</td>
<td>20</td>
<td>1.02</td>
<td>.03</td>
<td>190</td>
<td>53.09</td>
<td>4.67</td>
</tr>
<tr>
<td>Ratio</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.05</td>
<td>1.28</td>
<td>1.13</td>
</tr>
</tbody>
</table>
5.5 Influence of Geometric Properties

Shown in Figure 5.18 is the pressure versus deflection behavior of the analytical models employed to investigate the effect of the aspect ratio on the fence load carrying capacity. Additionally the analytical responses of slab strip along the short span of the fence predicted by Augustus-2 and VecTor4 are shown in the same Figure so comparison can be made between the slab strip and the two-way slab behavior. The purpose of this work was to compare the pressure versus displacement behavior of the slab strip to two-way slab and also to compare the predicted response of Augustus-2 and VecTor4 programs (See Figure 5.19). The concrete compressive strength was assumed to be 20 MPa in all the cases. The model was not based on any of the actual fences but a sample model.

![Figure 5-18 Analytical response of fully restrained two-way slabs having different aspect ratios and one-way slab strip](image-url)
The ultimate pressure capacity, $\sigma_{ult}$, and the maximum deflection prior to failure $\delta_{ult}$, of two-way slabs having various aspect ratios, and slab strips are compared and results are shown in Table 5.9.

Table 5-10 Analytical response of two-way slabs having different aspect ratios and one-way slab strip

<table>
<thead>
<tr>
<th>Two-way slab</th>
<th>Long Span (mm)</th>
<th>Short Span (mm)</th>
<th>Ratio</th>
<th>Thickness (mm)</th>
<th>$\sigma_{ult}$ (kPa)</th>
<th>$\delta_{ult}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5000</td>
<td>5000</td>
<td>1</td>
<td>1000</td>
<td>258</td>
<td>129.4</td>
</tr>
<tr>
<td>2</td>
<td>10000</td>
<td>5000</td>
<td>2</td>
<td>1000</td>
<td>237</td>
<td>124.4</td>
</tr>
<tr>
<td>3</td>
<td>20000</td>
<td>5000</td>
<td>4</td>
<td>1000</td>
<td>177</td>
<td>102.7</td>
</tr>
<tr>
<td>4</td>
<td>40000</td>
<td>5000</td>
<td>8</td>
<td>1000</td>
<td>159</td>
<td>93.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Slab Strip</th>
<th>Length (mm)</th>
<th>Width(mm)</th>
<th>Thickness(mm)</th>
<th>$\sigma_{ult}$ (KPa)</th>
<th>$\delta_{ult}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VecTor4</td>
<td>5000</td>
<td>300</td>
<td>1000</td>
<td>153</td>
<td>66.3</td>
</tr>
<tr>
<td>Augustus-2</td>
<td>5000</td>
<td>300</td>
<td>1000</td>
<td>98.4</td>
<td>51.6</td>
</tr>
</tbody>
</table>
5.6 Analytical Results

5.6.1 Slab Strip Model

This section presents the predicted response of the pressure versus center deflection of the fill fences installed in Cayeli mine using slab strip model (see section 5.2). Although both Augustus-2 and VecTor4 programs are able to analyze the slab strip model, due to the high influence of the rotational stiffness on the fence response and since VecTor4 model doesn’t consider various rotational stiffnesses, in this section the analyses were conducted using only Augustus-2 program. The actual stiffness of the surrounding rocks were not provides so the axial and rotational stiffness of the model were defined by trial and error to get the closest prediction to the actual fence behavior. The predicted responses are shown in Figure 5.20.

![Figure 5-20 Analytical response of fences installed at Cayeli mine](image)
The analytical results indicate that the fence installed at stope 715N22 had a higher ultimate pressure capacity. In both cases, first yielding of reinforcement occurs at the midspan and the final failure predicted to occur due to crushing of concrete at the midspan. The yielding pressure, \( \sigma_y \), ultimate pressure, \( \sigma_u \), and deflection at ultimate pressure, \( \delta_u \), are shown in Table 5.10.

Table 5-11 Analytical results of slab strip using Augustus-2

<table>
<thead>
<tr>
<th>Fence</th>
<th>Axial Stiffness (kN/mm)</th>
<th>Rotational Stiffness (kN.m/rad)</th>
<th>( \sigma_y ) (kPa)</th>
<th>( \sigma_u ) (kPa)</th>
<th>( \delta_u ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>685N20</td>
<td>105</td>
<td>26100</td>
<td>82.4</td>
<td>132.42</td>
<td>51.63</td>
</tr>
<tr>
<td>715N22</td>
<td>100</td>
<td>30,000</td>
<td>101</td>
<td>126.35</td>
<td>38.98</td>
</tr>
</tbody>
</table>

The modulus of elasticity, \( E \), length, \( L \), yielding strength, \( f_y \), ultimate strength, \( f_u \), of the truss bars used to define the axial and rotational stiffness of the model (see Figure 5.7B) are summarized in table 5.11.

Table 5-12 Augustus2 Vs. VecTor4 results comparison

<table>
<thead>
<tr>
<th>Fence</th>
<th>Truss Bar</th>
<th>A (mm(^2))</th>
<th>E (MPa)</th>
<th>( f_y ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>685N20</td>
<td>A</td>
<td>870</td>
<td>200000</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>870</td>
<td>200000</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>1000</td>
<td>200000</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>1000</td>
<td>200000</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>158</td>
<td>200000</td>
<td>350</td>
</tr>
<tr>
<td>715N22</td>
<td>A</td>
<td>480</td>
<td>200000</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>480</td>
<td>200000</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>1000</td>
<td>200000</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>1000</td>
<td>200000</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>150</td>
<td>200000</td>
<td>350</td>
</tr>
</tbody>
</table>
5.6.2 Full Geometry Model

This section represents the predicted pressure versus displacement response of the two fences installed at stopes 685N20 and 715N22 having an identical material, reinforcement ratio and long span, but different height, using VecTor4 program.

Based on the results found in the sections 5.2.5.4 and 5.2.5.5, restraint conditions and material properties have a significant effect on the pressure capacity of the fence. However, none of these properties were specified in the field report provided by Thomson, 2010. Thus, models were developed by assuming two rotational conditions: 1- zero rotational stiffness, 2- Infiniti rotational stiffness around the fence. The accuracy of the predicted response of the fences was predicted by defining shotcretes having different compressive strength.

Shown in Figure 5.21 are the predicted responses of the fences assuming infinity rotational stiffness around the fence. The fence installed at stope 700N20 with a longer height, had a higher pressure capacity. In both models, First cracking predicted to appear at the midspan, interface of the fence and surrounding rocks due to negative bending moment (element # 5 in Figure 5-13). This crack was occurred at the not visible side of the fill fence. First Yielding of the reinforcement bars was predicted to occur at the same location (Element # 5) and final failure occurred due to crushing of concrete and yielding of reinforcement due to negative bending moment at the midspan along the top edge of the fence. The center of the fence (Element # 25) had the maximum deflection prior to flexural failure.

Shown in figures 5.21 and 5.22 are the predicted responses of the fences assuming zero and infinity rotational stiffness around the edges. This model predicted the failure to occur at lower applied pressure at the midspan due to crushing of concrete.
Figure 5-21 Analytical response of fill fences using VecTor4 assuming infinite rotational stiffness

Figure 5-22 Analytical response of fill fences using VecTor4 assuming zero rotational stiffness
The cracking pressure, $\sigma_{cr}$, displacement at the cracking pressure, $\delta_{cr}$, initial crack width, $w_{cr}$, ultimate pressure capacity, $\sigma_u$, deflection at the ultimate pressure, $\delta_u$, and the crack width at the ultimate pressure, $w_u$, are summarized in Tables 5.11 and 5.12.

Table 5-13 Analytical result of fill fences using VecTor4 assuming infinity rotational stiffness

<table>
<thead>
<tr>
<th>Fence</th>
<th>$\sigma_{cr}$ (kPa)</th>
<th>$\delta_{cr}$ (mm)</th>
<th>$w_{cr}$ (mm)</th>
<th>$\sigma_u$ (kPa)</th>
<th>$\delta_u$ (mm)</th>
<th>$w_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>685N20</td>
<td>15</td>
<td>.32</td>
<td>.04</td>
<td>165</td>
<td>16</td>
<td>1.13</td>
</tr>
<tr>
<td>700N22</td>
<td>12</td>
<td>.24</td>
<td>.04</td>
<td>201</td>
<td>21</td>
<td>3.93</td>
</tr>
</tbody>
</table>

Table 5-14 Analytical result of fill fences using VecTor4 assuming zero rotational stiffness

<table>
<thead>
<tr>
<th>Fence</th>
<th>$f'_{c}$ (MPa)</th>
<th>$\sigma_{cr}$ (kPa)</th>
<th>$\delta_{cr}$ (mm)</th>
<th>$w_{cr}$ (mm)</th>
<th>$\sigma_u$ (kPa)</th>
<th>$\delta_u$ (mm)</th>
<th>$w_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>685N20</td>
<td>60</td>
<td>10</td>
<td>.61</td>
<td>.024</td>
<td>80</td>
<td>30.91</td>
<td>3.86</td>
</tr>
<tr>
<td>700N22</td>
<td>60</td>
<td>12</td>
<td>.24</td>
<td>.04</td>
<td>201</td>
<td>21</td>
<td>3.93</td>
</tr>
</tbody>
</table>
5.7  Comparison of Analytical Results and Actual Test Data

In this section the actual pressure versus deflection behavior of two fences installed and tested at stopes 715N22 and 685N15 of Cayeli mine are compared to the theoretical response predicted by Augustus-2 and Vector4 program.

5.7.1  Fence 715N22

The actual and predicted pressure versus displacement response of the fill fence installed and tested in the stop 715N22 of the Cayeli mine is shown in Figure 5.23. The backfilling process of fence was stopped at the applied pressure of 100 kPa, at that stage the maximum deflection at the center of the fence was measured to be 22 mm.

![Figure 5-23 Actual test Vs. Predicted response of Fence 715N22](image-url)
The initial stiffness of the predicted response of both programs was very similar to the stiffness of the actual test response. The overall prediction of Augustus-2 was more accurate and much closer to the fence behavior. This could be due to the restraint conditions defined in the Augustus-2 models. In VecTor4 model, it wasn’t possible to define the stiffness of the surrounding rocks so the best answer was found by assuming shotcretes having different compressive strength. However, in Augustus-2 model, the stiffness of the boundaries could be defined by changing the rotational and axial stiffness of the truss bars attached to the beam end.

Both programs predicted the maximum displacement to occur at center of the fence. Vector-4 predicted the final failure to occur due to crushing of concrete at the support, however, Augustus-2 predicted the final failure to occur due to crushing of concrete at the midspan.

The measured displacement at 100 kPa applied pressure, $\delta_{100}$, crack width at this pressure, $w_{100}$, ultimate pressure capacity, $\sigma_u$, deflection at the ultimate pressure, $\delta_u$, and the crack width at the ultimate pressure, $w_u$, are compared to Augustus-2 and VecTor4 predictions (zero rotational stiffness) and results are summarized in Table 5.14 and 5.15.

<table>
<thead>
<tr>
<th>Field Data</th>
<th>Augustus-2</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_{100}$ (mm)</td>
<td>22</td>
<td>22.6</td>
</tr>
<tr>
<td>$w_{100}$ (mm)</td>
<td>12</td>
<td>-</td>
</tr>
<tr>
<td>$\sigma_u$ (kPa)</td>
<td>-</td>
<td>126.34</td>
</tr>
<tr>
<td>$\delta_u$ (mm)</td>
<td>-</td>
<td>38.98</td>
</tr>
</tbody>
</table>

Table 5-16 Comparison between actual test result and VecTor4 prediction

<table>
<thead>
<tr>
<th>Field Data</th>
<th>VecTor4</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_{100}$ (mm)</td>
<td>22</td>
<td>20.69</td>
</tr>
<tr>
<td>$w_{100}$ (mm)</td>
<td>5</td>
<td>3.86</td>
</tr>
<tr>
<td>$\sigma_u$ (kPa)</td>
<td>-</td>
<td>110</td>
</tr>
<tr>
<td>$\delta_u$ (mm)</td>
<td>-</td>
<td>33.60</td>
</tr>
</tbody>
</table>
5.7.2 Fence 685N20

The actual and predicted pressure versus displacement response of the fill fence installed and tested in the stop 685N20 of the Cayeli mine is shown in Figure 5.24. The backfilling process of fence was stopped at the applied pressure of 50 kPa, at that stage the maximum deflection at the center of the fence was measured to be 7.8 mm.

Figure 5-24 Actual test Vs. Predicted response of Fence 685N20
The measured displacement at the 50 kPa applied pressure, $\delta_{50}$, crack width at measured pressure, $w_{50}$, ultimate pressure capacity, $\sigma_u$, deflection at the ultimate pressure, $\delta_u$, and the crack width at the ultimate pressure, $w_u$ of Augustus-2 and VecTor4 (zero rotational stiffness) are compared to Augustus-2 and VecTor4 (zero rotational stiffness) predictions and results are summarized in Table 5.16 and 5.17.

Table 5-17 Comparison between actual test result and Augustus-2 prediction

<table>
<thead>
<tr>
<th>715N22</th>
<th>$\delta_{50}$ (mm)</th>
<th>$w_{50}$ (mm)</th>
<th>$\sigma_u$ (kPa)</th>
<th>$\delta_u$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field Data</td>
<td>7.78</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Augustus-2</td>
<td>7.71</td>
<td>-</td>
<td>132.42</td>
<td>52.63</td>
</tr>
<tr>
<td>Ratio</td>
<td>1.16</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5-18 Comparison between actual test result and VecTor4 prediction

<table>
<thead>
<tr>
<th>715N22</th>
<th>$\delta_{50}$ (mm)</th>
<th>$w_{50}$ (mm)</th>
<th>$\sigma_u$ (kPa)</th>
<th>$\delta_u$ (mm)</th>
</tr>
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<tr>
<td>Field Data</td>
<td>7.78</td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>VecTor4</td>
<td>7.69</td>
<td>5.46</td>
<td>80</td>
<td>30.89</td>
</tr>
<tr>
<td>Ratio</td>
<td>.98</td>
<td>.91</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Chapter 6
Conclusions and Recommendations

6 Conclusions and Recommendation

6.1 Conclusions

The method called Cemented Paste Backfill is a new method for use in mining. Mine wastes (tailings) are mixed with binder content and water and then delivered underground to fill voids, thus enough support can be provided to the adjacent stopes resulted in accelerating of the mining process. This method is called Cemented Paste Backfill (CPB). In order to isolate the excavated stope for backfilling process, a fill fence is installed at the bottom of the open stopes to retain the hydrostatic pressure resulting from CPB. Hence, a good understanding of the fill fence structural behavior can help increasing the backfilling rate.

The field work report of two fences constructed, installed and tested in the Cayeli mine in Turkey was studied. Since these fences were installed in an underground mine, the longitudinal elongation due to the applied pressure can be prevented by the surrounding rocks resulting in the development of compressive membrane action. The influence of membrane action on the load carrying capacity of reinforced concrete beams was investigated by studying the behavior of 12 axially restrained beams tested in China by Su et al. 2009. The tested beams were modeled and analyzed using a 2-D nonlinear finite element program Augustus-2, and the analytical responses were compared to the actual results. The theoretical behavior of the models was very similar to the actual response of the beam. The average ratio of the ultimate load capacity of the tested beam to the predicted load capacity of the models was 0.87. The tested fill fences in the Cayeli mine were modeled using the Augustus-2 program. Additionally, a 3-D model was developed using a nonlinear finite element program VecTor4. Theoretical responses of the fill fences were compared to the field data.

The experimental program by Su et al. 2009, showed that axially restrained beams developed high axial compressive (membrane) forces and this resulted in a significant increase in the specimen
load carrying capacity. The test also indicated that increasing the flexural reinforcement ratio and span to height ratio decreases the influence of the compressive membrane action. However, the effect of loading rate can be neglected. Based on the analytical results, it was found that axial and rotational stiffnesses have a high influence on the load carrying capacity of the beams. Comparing the test results and analytical responses of the specimens, the required value for the axial stiffness of the tested specimens was found to be much lower than published. However, the rotational stiffnesses matched well. The effect of compressive membrane action on the pressure capacity of the fill fence was found to be of significance. A fill fence subjected to high axial and rotational stiffnesses can have a pressure capacity 3.3 times higher than a fence with negligible axial and rotational stiffnesses. The geometric properties were also found to have a significant effect on the pressure capacity of the fill fence. Based on the 3-D analyses conducted with VecTor4, the pressure capacity of the fill fence can be increased by designing a fence with a lower aspect ratio. Comparing the analytical results of the fill fence and the field data, it was noticed that the fence installed in stope 685N20 had a safety factor of 2.6, however the fence installed at stope 715N22 may have had a safety factor of only 1.2.

6.2 Recommendations

This section presents recommendations to improve the analytical response of the axially restrained beams discussed in Chapter 4, and fill fences installed in underground mines discussed in Chapter 5.

Although the Augustus-2 program was able to capture the effect of the membrane action and was able to predict the specimen behavior fairly accurately, after the ultimate load was reached, a sudden drop occurred at the load-deflection predictions, and this may be due to the P-δ effect. This effect also consumes a percentage of the load capacity so not considering this effect may result in overestimating the ultimate load capacity. Thus the current program can be improved by considering this effect.
Shrinkage effects were not considered in the analyses. However, shrinkage of the fill fence produces a permanent gap between the fence and the surrounding rocks and as a result larger displacement will be required to engage the effect of membrane action. Thus it is recommended to consider this effect for future research.

The stiffness of the surrounding rocks has a high influence on the fence pressure capacity. However, no measurements have been done to determine how stiff they are. Thus it is highly recommended to consider this factor for future work.

In this research flexural failures were assumed to be dominant. Further investigation is recommended to find if there is also a possibility of shear failure.

Designing the underground fill fence based on conventional slab theory doesn’t consider the secondary actions and geometry nonlinearities which can be very conservative. Thus it may be more cost effective to design the fences based on more advanced methods.

The current models of the fill fences can be improved by modeling of more complex shapes. For instance building these fences in an arch shape likely can significantly increase the pressure capacity.

In this research the cemented paste pressure was assumed to be uniformly distributed. However, the pressure profile of the cemented paste is more complex, so investigation on the applied pressure profile is recommended.
References


Appendices

Appendix 1: Input Data for Numerical Analyses with Augustus-2

Job file is provided for the axially restrained beam A-1:

Element Input File V2.0

//
// This is an Augustus II input file.
//

Analysis Definition

title=Axially Restrained beam-A1 // title
date=2010/02/12 // Date
doneby=Sina Ghazi // Doneby
path =C:\Documents and Settings\Sina\Desktop\Sina Ghazi\Research-Up Dated\response\units=METRIC // units (METRIC / US)
End

//
// This list is node#, X coord (mm|in), y coord (mm|in), [ #_nodes d_node_#, d_x, d_y ] [ #_lines d_node_# d_x d_y ]
//

Node List
1 0 0 3 2 150 0 2 1 0 300
7 324.5 0 48 2 24.5 0 2 1 0 300
103 1500.5 0 2 2 34.5 0 2 1 0 300
107 -200 150
108 -500 150
109 0 -200
restrained nodes: node #, restrained X (0 | 1 | 2) , restrained Y (0 | 1 |2) [condX] [CondY] [ #_nodes d_node_#]

restraint codes: 0 = unrestrained, 1 = fully restrained, 2 = conditionally restrained DOF.

Restraint List
3  0 1
108 1 1
109 1 1
110 1 1
105 1 0
106 1 0
End

// ELEMENT INCIDENT LIST

//

// element #, node 1, node2, node 3, node 4, Response-2000_File [ #_elements, d_elem_#, d_node_#] [ #_lines d_elem_#, d_node_#]

Incident Beam
1 1 2 3 4 "REGION1.rsp" 2 1 2
3 5 6 7 8 "REGION2.rsp" 10 1 2
13 25 26 27 28 "REGION3.rsp" 28 1 2
41 81 82 83 84 "REGION2.rsp" 10 1 2

End

//
// Wall Incident list

// element #, node1, node2, node 3, node 4, Membrane-2000_File [ #_elements, d_elem #, d_node #] [#_lines d_elem #, d_node #]

//

Incident Wall

End

//

// element #, node1, node2, Response_2000_File[:model#] [ #_elements, d_elem #, d_node #]

//

Incident Truss

53 107 1 TrussC.rsp STEEL

54 107 2 TrussD.rsp STEEL

55 108 107 TrussE.rsp STEEL

56 109 1 TrussA.rsp STEEL

57 110 5 TrussB.rsp STEEL

End

Loads List

8 0 -.02 0
10 0 -.02 0
12 0 -.02 0
14 0 -.02 0
16 0 -.02 0
18 0 -.02 0
20 0 -.02 0
22 0 -.02 0
24 0 -.02 0
26 0 -.02 0
28 0 -.02 0
86 0 -0.02 0
88 0 -0.02 0
90 0 -0.02 0
92 0 -0.02 0
94 0 -0.02 0
96 0 -0.02 0
98 0 -0.02 0
100 0 -0.02 0
102 0 -0.02 0
104 0 -0.02 0
106 0 -0.02 0
End
Job file is provided for Fill fence 715N22

Element Input File V2.0

//

// This is an Augustus II input file.

//

Analysis Definition

title=Fill Fence 715N22 // title
date=2010/11/12 // Date
doneby=Sina Ghazi // Doneby

path=C:\Documents and Settings\Sina\Desktop\Sina Ghazi\Research-Up Dated\response\units=METRIC // units (METRIC/US)

End

//

// This list is node#, X coord (mm|in), y coord (mm|in), [ #_nodes d_node_#, d_x, d_y ] [ #_lines d_node_#
d_x d_y ]

//

Node List

1 0 0 3 2 150 0 2 1 0 300
7 324.5 0 48 2 90.0 0 2 1 0 300
103 1500.5 0 2 2 90.0 0 2 1 0 300
107 -200 150
108 -500 150
110 0 -200

End

//
restrained nodes: node #, restrained X (0 | 1 | 2) , restrained Y (0 | 1 |2) [condX] [CondY] [ #_nodes d_node_#]

restraint codes: 0 = unrestrained, 1 = fully restrained, 2 = conditionally restrained DOF.

Restraint List

3 0 1
108 1 1
109 1 1
110 1 1
105 1 0
106 1 0
End

// ELEMENT INCIDENT LIST

Incident Beam

1 1 2 3 4 "Support.rsp" 2 1 2
3 5 6 7 8 "Fence685.rsp" 50 1 2
End

// Wall Incident list

Incident Wall
End
// element #, node1, node2, Response_2000_File[:model#] [ #_elements, d_elem_, d_node_#]

Incident Truss

53 107 1 TrussC.rsp STEEL
54 107 2 TrussD.rsp STEEL
55 108 107 TrussE.rsp STEEL
56 109 1 TrussA.rsp STEEL
57 110 5 TrussB.rsp STEEL

End

Loads List
8 0 -.02 0
10 0 -.02 0
12 0 -.02 0
14 0 -.02 0
16 0 -.02 0
18 0 -.02 0
20 0 -.02 0
22 0 -.02 0
24 0 -.02 0
26 0 -.02 0
28 0 -.02 0
30 0 -.02 0
32 0 -.02 0
34 0 -.02 0
36 0 -.02 0
38 0 -.02 0
40 0 -.02 0
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<tr>
<th>42 0</th>
<th>.02 0</th>
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<tr>
<td>44 0</td>
<td>.02 0</td>
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<tr>
<td>46 0</td>
<td>.02 0</td>
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<tr>
<td>48 0</td>
<td>.02 0</td>
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<td>.02 0</td>
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<td>52 0</td>
<td>.02 0</td>
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<td>.02 0</td>
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<td>56 0</td>
<td>.02 0</td>
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<tr>
<td>58 0</td>
<td>.02 0</td>
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<tr>
<td>60 0</td>
<td>.02 0</td>
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<td>.02 0</td>
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<td>72 0</td>
<td>.02 0</td>
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<td>74 0</td>
<td>.02 0</td>
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<td>76 0</td>
<td>.02 0</td>
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<td>78 0</td>
<td>.02 0</td>
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<td>90 0</td>
<td>.02 0</td>
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<tr>
<td>92 0</td>
<td>.02 0</td>
</tr>
<tr>
<td>94 0</td>
<td>.02 0</td>
</tr>
<tr>
<td>96 0</td>
<td>.02 0</td>
</tr>
</tbody>
</table>
98 0 .02 0
100 0 .02 0
102 0 .02 0
104 0 .02 0
106 0 .02 0
End
Appendix 2: Response-2000 file to define geometric and material properties for Augustus2 analysis of Beam A1 (Region3)

<table>
<thead>
<tr>
<th>Geometric Properties</th>
<th>Gross Conc.</th>
<th>Trans (h=3.42)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (mm$^2$) x 10^3</td>
<td>45.0</td>
<td>48.4</td>
</tr>
<tr>
<td>Inertia (mm$^4$) x 10^6</td>
<td>337.5</td>
<td>394.2</td>
</tr>
<tr>
<td>$y_1$ (mm)</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>$y_2$ (mm)</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>$S_1$(mm$^3$) x 10^3</td>
<td>2250.0</td>
<td>2537.7</td>
</tr>
<tr>
<td>$S_2$(mm$^3$) x 10^3</td>
<td>2250.0</td>
<td>2537.7</td>
</tr>
</tbody>
</table>

Crack Spacing

2 x dist x 0.1 dp/p

Loading (N.M.V. + dN.dM.dV)

0.0,0.0,0.0, 0.0,0.1,0.0

Concrete

$\mu_c = 25.8$ MPa
$a_1 = 18$ mm
$h = 1.05$ MPa (auto)
$\varepsilon_{c1} = 1.91$ mm/mm

Rebar

$\upsilon = 540$ MPa

Axially restrained frame beam

sina 2009/1
Appendix 3: Input Data for Numerical Analyses with VecTor-4

Job file, load case, and structure file are provided for Fell fences installed in stopes 6585N20.

* * * * * * * * * * * * *
* VecTor *
* JOB DATA *
* * * * * * * * * * * * *

Job Title (30 char max) : two way slab
Job File Name (8 char max) : T1
Date (30 char max) : June 03/10

STRUCTURE DATA

----------
Structure Type : 4
File Name (8 char max) : Struc

LOADING DATA

--------
No. of Load Stages : 999
Starting Load Stage No. : 1
Load Series ID (5 char max) : T1

<table>
<thead>
<tr>
<th>Load File Name</th>
<th>Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case (8 char. max.)</td>
<td>Initial</td>
</tr>
<tr>
<td>1 LOAD1</td>
<td>0.000000</td>
</tr>
<tr>
<td>2 NULL</td>
<td>0.000000</td>
</tr>
<tr>
<td>3 NULL</td>
<td>0.000000</td>
</tr>
</tbody>
</table>
ANALYSIS PARAMETERS

-------------------
Analysis Mode                       (1,3,4) : 1
Seed File Name                 (8 char max) : NULL
Convergence Limit                    (>1.0) : 1.000010
Averaging Factor                     (<1.0) : 0.50
Maximum No. of Iterations                   : 100
Convergence Criteria                  (1-3) : 1
Results File Type                     (1-4) : 2
Result Output Format                  (1-3) : 4

MATERIAL BEHAVIOUR MODELS

-------------------------
Concrete Compression Base Curve       (0-3) : 2
Concrete Compression Post-Peak         (0-3) : 1
Concrete Compression Softening         (0-8) : 1
Concrete Tension Stiffening           (0-5) : 1
Concrete Tension Softening            (0-3) : 1
Concrete Tension Splitting            (1-2) : 0
Concrete Confined Strength            (0-2) : 1
Concrete Dilatation                   (0-1) : 0
Concrete Cracking Criterion           (0-4) : 0
Concrete Crack Slip Check             (0-2) : 1
Concrete Crack Width Check             (0-2) : 0
Concrete Bond or Adhesion              (0-4) : 0
Concrete Creep and Relaxation (0-1) : 1
Concrete Hysteresis (0-3) : 1
Reinforcement Hysteresis (0-3) : 1
Reinforcement Dowel Action (0-1) : 0
Reinforcement Buckling (0-1) : 0
Element Strain Histories (0-1) : 1
Element Slip Distortions (0-4) : 0
Strain Rate Effects (0-1) : 1
Structural Damping (0-1) : 0
Geometric Nonlinearity (0-1) : 1
Crack Allocation Process (0-1) : 1

ANALYSIS PARAMETERS:

--------------

1. INTEGRATION SCHEME:
   1. Selective (Bending - 3 ; Shear - 2)
   2. Full (Bending - 3 ; Shear - 3)
   3. Reduced (Bending - 2 ; Shear - 2)

2. CONVERGENCE CRITERIA:
   1. Displacements - Weighted Average
   2. Displacements - Maximum Value
      3. Secant Moduli - RMS

3. RESULTS FILE TYPE:
   1. ASCII and Binary Files
2. ASCII Files Only
3. Binary Files Only
4. Last Load Stage Only

4. RESULTS OUTPUT FORMAT

Print concrete strains and stresses of:
1. All layers at all gauss points
2. Top and bottom layers at all gauss points
3. All layers for central gauss point only

MATERIAL BEHAVIOUR MODELS:

-----------------------------

1. OUT-OF-PLANE SHEAR:
   0. Shear Not Considered
   1. Considered - Uniform Shear Strain

2. GEOMETRIC NONLINEARITY:
   0. Neglect Effect
   1. Consider Effect

3. LOAD HISTORY:
   0. Neglect Previous Loading Effects
   1. Consider Previous Loading Effects

4. CONCRETE COMPRESSION BASE CURVE:
   0. Linear Elastic
   1. Normal (Parabolic)
   2. High Strength
5. CONCRETE COMPRESSION SOFTENING:

   0. Neglect Effect
   1. Vecchio-Collins 1982 Model
   2. Vecchio-Collins 1986 Model
   3. Vecchio 1992-A Model
   4. Vecchio 1992-B Model

6. CONCRETE TENSION STIFFENING:

   0. Neglect Effect
   1. Vecchio-Collins 1982 Model
   2. Collins-Mitchell Model
   3. Izumo, Maekawa et al Model

7. CONCRETE TENSION SOFTENING:

   0. Neglect Effect
   1. Linear Softening
   2. Linear Softening + 10% Residual Stress
   3. 10% Residual Stress Only
   4. Yamamoto Model (Base Curve Only)
   5. Yamamoto Model (Strain Hardening Considered)

8. CONCRETE STRENGTH ENHANCEMENT:

   0. Biaxial Effects Neglected
   1. Biaxial Effects Considered

9. CONCRETE PLASTIC STRAINS:

   0. Neglect Effects
1. Consider Effects

10. CONCRETE CRACK SLIP CHECK:
   0. Omit Check
   1. Perform Check

11. REINFORCEMENT STRESS RESPONSE:
   0. Linear Elastic
   1. Elastic-Plastic
   2. Strain Hardening Considered
   3. Bauschinger Effects Considered

12. SPLITTING FAILURE CRITERIA:
   0. Neglect Effects
   1. Consider Effects

NOTES:
------
1. This program uses metric units only.

2. Do not insert or delete any line.

* * * * * * * * * * * * *
* STRUCTURE          *
* DATA               *
* Version 1.0        *
* * * * * * * * * * * * *
**STRUCTURAL PARAMETERS**

Structure title (30 char. max.) : two way slab
Structure file name (8 char. max.) : T1
No. of reinforced concrete material types : 2
No. of steel material types : 0
No. of RC elements : 40
No. of truss elements : 0
No. of nodal points : 187
No. of nodes with prescribed d.o.f. : 52

Aggregate Type (1-2) : 1
Creep Coefficient (> 0) : 0.0
Concrete Shrinkage (-me) : 0.0

**MATERIAL SPECIFICATIONS**

**GENERAL**

---

<table>
<thead>
<tr>
<th>MAT</th>
<th>T</th>
<th>OS</th>
<th>CONC</th>
<th>REIN</th>
<th>DIA-Z</th>
<th>RHOZ</th>
<th>Fyz</th>
<th>Fuz</th>
<th>Esz</th>
<th>eshz</th>
<th>euz</th>
<th>Agg</th>
<th>Sx</th>
<th>Sy</th>
<th>Sz</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>300.0</td>
<td>8</td>
<td>2</td>
<td>8.0</td>
<td>0.0</td>
<td>290.0</td>
<td>455</td>
<td>190250.0</td>
<td>3.2</td>
<td>330.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>300.0</td>
<td>8</td>
<td>2</td>
<td>8.0</td>
<td>0.0</td>
<td>290.0</td>
<td>455</td>
<td>190250.0</td>
<td>3.2</td>
<td>330.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

/  

**CONCRETE**

---
MAT  fc  f't  Ec  e'c  Mu  Cc  kc  Density
TYP  (MPa)  (MPa)  (MPa)  (mm/m)  (/C)  (mm^2/hr)  (kg/m^3)
1  60.  0.  0.  0.150  12.E-6  4320.  2400.0
2  60.  0.  0.  0.150  12.E-6  4320.  2400.0
/

REINFORCEMENT

-------------
MAT  REF  DIR  d  DIA  As  Fy  Fu  Es  esh  eu  Cs  Dep
TYP  (1-6)  (deg)  (mm)  (mm)  (mm^2/m)  (MPa)  (MPa)  (MPa)  (mm/m)  (mm/m)  (/C)  (mm/m)
1  1  0.0  150  28.  2052.  350.  540  200000.  80.  260.  0.  0
1  1  90.0  150  28.  1539.  350.  540  200000.  80.  260.  0.  0
2  1  0.0  150  28.  2052.  350.  540  200000.  80.  260.  0.  0
2  1  90.0  150  28.  1539.  350.  540  200000.  80.  260.  0.  0
/

(B) STEEL

--------

<NOTE:> TO BE USED FOR TRUSS ELEMENTS ONLY
MAT  REF  OS  DIA  As  Fy  Fu  Es  esh  eu  Cs  Dep
TYP  (1-6)  (mm)  (mm)  (mm^2)  (MPa)  (MPa)  (MPa)  (mm/m)  (mm/m)  (/C)  (mm/m)
/

ELEMENT INCIDENCES

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

(A) HETEROSIS ELEMENTS

--------------

<<<<<< FORMAT >>>>>  (counterclockwise direction)

ELMT  INC1  INC2  INC3  INC4  INC5  INC6  INC7  INC8  <INC9>  [#ELMT d(ELMT) d(INC1) d(INC4)] x2 /
1  1  2  3  14  25  24  23  12  13  5  1  2  2  8  5  22  22 /
(B) TRUSS ELEMENTS

ELMT INC1 INC2 [ #ELMT d(ELMT) d(INC)] [ #ELMT d(ELMT) d(INC)]

MATERIAL TYPE ASSIGNMENT

MAT. TYPE   ELMT   [ #ELMT d(ELMT)]
1             1       19      1       
2             20        1      1       
1             21        4      1      
2             25        1      1 
1             26       15      1       

COORDINATES

TOP or C/L   <BOT>

NODE TYPE   X   Y   Z   <X   Y   Z> [ #NODE d(NODE) d(X) d(Y) d(Z)]
1             2        0.0  0.0           11  1  425.0. 0. 17 11 0. 350.0 0.  / 

NODAL RESTRAINTS AND PRESCRIBED D.O.F.
<<<<<< FORMAT >>>>> (units = mm, degrees)

NODE  DX-R DY-R DZ-R R1-R R2-R DX-P DY-P DZ-P R1-P R2-P  [#NODES d(NODE)] /
1  1  0  1  0  0  0  0  0  0  10  1 /
12  0  1  1  0  0  0  0  0  0  16  11 /
178 1  0  1  0  0  0  0  0  0  9  1 /
22  1  1  0  1  0  0  0  0  0  15  11 /
11  1  1  1  0  0  0  0  0  0  2  176 /
/

<NOTES:>

Smeared Reinforcement:
   d - Distance from the top of the element to the centroid of the
       reinforcement layer.

Truss Elements:
   OS - Element offset measured from nodal location (typically middle layer).
       Negative OS is toward element bottom surface.

REF - Reinforcement Types (smeared & truss):
   1 - Ductile Steel Reinforcement (tension+compression)
   2 - Prestressing Steel (tension+compression)
   3 - Ductile Steel Reinforcement (tension only)
   4 - Ductile Steel Reinforcement (compression only)
   5 - Linear Elastic Truss (tension only)
   6 - Linear Elastic Truss (compression only)

Element incidences:
   <INC9> - Only required when nine noded element is used.
Coordinates:

  TYPE - 1 - Top and Bottom coordinates of the node are provided.
  2 - Centre Line coordinates of the node are provided.

Restrained D.O.F.:

  0 - Unrestrained degree of freedom
  1 - Restrained degree of freedom

(1) DO NOT INSERT OR DELETE ANY LINE.

   EXCEPTION: INSERTION OF LINES IN THE SPACE PROVIDED FOR INPUT OF
   DATA. IN THIS CASE, LEAVE LINE WITH SLASH AFTER LAST DATA LINE.

(2) BLANK SPACES SHOULD BE USED TO SEPARATE DATA WITHIN A DATA LINE.

(3) ELEMENT INCIDENCE NUMBER 9 (i.e. <INC9>) TO BE IGNORED WHEN 8 NODED
    SERENDIPITY ELEMENT USED.

(4) DIMENSIONED FOR: 50 ELEMENTS, 200 NODES, 100 RESTRAINED NODES,
    16 CONCRETE LAYERS, 6 REINFORCEMENT LAYERS, 30 MATERIALS, 20 LAYER
    PATTERNS, MAXIMUM FRONTWIDTH OF 100.

* * * * * * * * * * * * *
* LOAD CASE *
* DATA *
* Version 1.0 *
**LOAD CASE PARAMETERS**

Structure title (30 char. max.) : two way slab
Load case title (30 char. max.) : DELTA
Load case file name (8 char. max.) : OA3

No. of elements with a u.d.l. : 40
No. of elements with hydrostatic loads : 0
No. of elements with temperature loads : 0
No. of nodes with concentrated loads : 0
No. of nodes with imposed displacements : 0
No. of elements with body loads : 0
No. of nodes with lumped mass assignment : 0
No. of nodes with impulse loads : 0
No. of ground acceleration data values : 0

**UNIFORMLY DISTRIBUTED LOADS**

<<<<< FORMAT >>>>>

ELMT LOAD MAG.(MPa) [#ELMT d(ELMT)] x2 /
1 -.001 40 1/
/

**HYDROSTATIC LOADS**

***************
ELMT   LOAD MAG.(MPa)   Z(mm)   [#ELMT d(ELMT)] x2 /

TEMPERATURE LOADS
*************************

ELMT FINAL BOT. INIT. BOT FINAL TOP INIT. TOP ELAPSED [#ELMT d(ELMT)] x2 /

TEMP. TEMP. TEMP. TEMP. TIME (hr.)

CONCENTRATED LOADS
************************

NODE FX FY FZ MXZ MYZ [#NODE d(NODE) d(FX) d(FY) d(FZ) d(MXZ) d(MYZ)] x2 /

(kN) (kN) (kN) (kNm) (kNm)

PRESCRIBED NODE DISPLACEMENTS
*******************************

NODE DOF DISPL  [#NODE d(NODE)]

(1-5) (mm|deg)

GRAVITATIONAL LOADS
************************

<<<<< FORMAT >>>>>
ELMT  GX  GY  GZ  GAMMA  [#ELMT  d(ELMT)] x2 / 
/

ADDITIONAL LUMPED MASSES
*********************************

<NOTE:> UNITS: kg, m/s, m/s^2

<<<<<< FORMAT >>>>>

/

IMPULSE, BLAST AND IMPACT FORCES
*********************************

<NOTE:> UNITS: Sec, kN

<<<<<< FORMAT >>>>>

NODE DOF T1  T2  T3  T4  F1  F2  F3  F4  [ #NODE d(NODE) ] / 
/

GROUND ACCELERATION
*******************

<NOTE:> UNITS: Sec, G

<<<<<< FORMAT >>>>>

TIME  ACC-X ACC-Y ACC-Z / 
/

<NOTES>

LOAD MAG.:  

Either the value of the udl, or, the maximum value of the  
hydrostatic load.
Z:

The coordinate of zero pressure for hydrostatic loading.

(1) DO NOT INSERT OR DELETE ANY LINE.

EXCEPTION: INSERTION OF LINES IN THE SPACE PROVIDED FOR INPUT OF DATA. IN THIS CASE, LEAVE LINE WITH SLASH AFTER LAST DATA LINE.

(2) BLANK SPACES SHOULD BE USED TO SEPARATE DATA WITHIN A DATA LINE.