An Investigation of the Shear Design of a Reinforced Concrete Box Structure

by

Sasha Kuzmanovic

A thesis submitted in conformity with the requirements for the degree of Master of Applied Science Graduate Department of Civil Engineering University of Toronto

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Abstract

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Recent research has shown that size effects reduce the shear strength of large beams with no shear reinforcement and with low amounts of flexural reinforcement. It was suspected that these size effects may be important for the thick one-way slabs of the underground box structures used in subway construction.

To investigate this hypothesis a 45% scale model of a slice of a Toronto Transit Commission box unit was subjected to uniform loading.

The specimen failed in shear at only 55% of its flexural capacity. The failure shear stress of the 45MPa concrete was only 0.85MPa. Thus, it was found that the size effect in shear for this one-way slab was very important. Also, it was proven that the ACI Code expressions as well as the AASHTO Culvert Method are unconservative for this case. Only, the shear design methods of the CSA Code provided accurate estimates of shear capacity. Thus, the need for better shear strength expressions was emphasized. In the meantime, it is recommended that one-way slabs with a thickness greater than 300mm should not be excluded from the provisions which require minimum shear reinforcement if the shear stress exceeds one half of the shear strength.
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Notation

\( a = \) distance from support to applied load
\( A_s = \) area of reinforcement in tension zone
\( A_p = \) area of tendons in tension zone
\( b = \) design width usually taken as 1 m (AASHTO Code)
\( b_w = \) minimum effective web width within depth \( d \)
\( d = \) distance from extreme compression fiber to centroid of longitudinal tension reinforcement
\( d_v = \) distance measured perpendicular to the neutral axis between the resultants of the tensile and compressive forces due to flexure
\( f'_c = \) specified compressive strength of concrete
\( f_{po} = \) stress in prestressing tendons when stress in the surrounding concrete is zero
\( E_s = \) modulus of elasticity of reinforcement
\( E_p = \) modulus of elasticity of tendons
\( L = \) distance between inflection points
\( M_f = \) factored moment at section (CSA-94 Code)
\( M_a = \) factored moment at section (ACI Code)
\( N_f = \) factored axial load normal to the cross section occurring simultaneously with \( V_f \)
\( s_z = \) spacing of longitudinal cracks in web of member
\( V_f = \) factored shear force at section (CSA-94 Code)
\( V_u = \) factored shear force at section
\( x = \) distance from midspan of slab to section
\( \beta = \) factor accounting for shear resistance of cracked concrete
\( \rho_w = \) \( A_v/b_wd \)
\( \theta = \) angle of inclination of diagonal compressive stresses to the longitudinal axis of the member
\( \lambda = \) factor to account for low density concrete
\( \phi_c = \) resistance factor for concrete
1. Introduction

1.1 GENERAL

Underground boxes are frequently used in the design of subway running structures where their function is to provide sufficient space to operate trains while resisting vertical and lateral pressures due to earth, hydrostatic and live loads. They are also utilized underneath highways, facilitating drainage flow or traffic. They may consist of single or multiple cells and their size varies.

Generally, highway box structures or culverts are used instead of bridges when spans measure up to 12m. Culverts can be cast in place or set on site as precast units.

Most subway running structures consist of double cell box units with cell widths of approximately 4m. The thickness of the walls and slabs of these structures often exceeds 600mm. Figure 1.1 shows two out of three cells of a box structure located south of Downsview Station.

In order to obtain flexible subway operations, which would permit trains to change direction and switch from one cell to the other, cross over sections are occasionally built. At these locations, two or more cells merge together into one wide cell, its width exceeding 10m, while the thickness of the walls and slabs ranges up to 2.4m.

Subway box structures are built according to a cut and cover method (see Figure 1.2). A large trench with vertical walls, supported by steel piles, horizontal steel struts and tiebacks, is excavated to build the subway line. Once excavation is completed, box structures are cast in place on a mud slab (a layer of low strength concrete). Since this method requires only a narrow space above the proposed line, it is frequently used in populated areas.

However, subway lines must be located relatively close to the ground level or bored tunnels could be considered as offering a more feasible solution. Thus, the earth cover usually ranges up to 12m (40ft). If a box is designed with an earth cover in excess of 2.4m (8 ft), it is reasonable to assume that concentric loads applied at ground level would spread through the earth
Figure 1.1 - TTC Box Structure on the Spadina Line

Figure 1.2 - Cut and Cover Method
and the top of the box would be subjected to equivalent, uniformly distributed loads. Lateral hydrostatic and earth pressures are to be considered, especially in the design of the walls. They cause not only bending of the walls but also compression in slabs, which enhances shear strength. It would be hard to ensure that walls would be subjected to lateral pressure throughout the entire life of these structures. Thus, designers should also check the capacity of the slabs, assuming that the structure would be subjected only to vertical loads.

Even though, they are known to be an economical replacement for short span bridges, a considerable number of these structures, which are either used in subways or as culverts underneath highways, demand a much better understanding of all aspects of their behavior.

1.2 REASONS FOR INVESTIGATION

1.2.1 SIZE EFFECTS

Early research on the shear strength of beams, with or without web reinforcement, was based on the test results obtained from small specimens whose depth often did not exceed 250 to 300mm.

A major impact on the shear design of large beams was the failure of the Wilkins Air Force Depot warehouse in Ohio in 1955. The partial collapse was caused by shear failure that occurred at three bays of six span frames. Each frame consisted of 914mm deep beams with a low reinforcing ratio and no shear reinforcement. Those beams spanned approximately 20.3m while supporting roof block purlins and gypsum slabs. Expansion joints were provided near the centerline of each frame, but most of them were seized. Entire frames were cast during single working days. This contributed to considerable shrinkage. Prior to the failure a very wide shear crack (about 40mm wide!) was discovered in the interior span beam. Even though this beam was shored, the shear failure of beams at adjacent spans could not be prevented. After testing one-third size specimens, the Portland Cement Association concluded that the failure was mainly due to shrinkage, which contributed to axial tension forces in the beams, having a disadvantageous effect on shear strength. Lack of knowledge regarding detrimental size effects on shear strength could not contradict the unconservative nature of current codes. However, this failure led to additional research on large beams without shear reinforcement.

Leonhardt and Walther initially tested eight beams with the same geometry and reinforcing ratio while the type of the bars was varied. It was found that improved bonding
properties increased their corresponding shear strengths. After additional testing of two series of beams with different depths, while maintaining either the same reinforcing ratio or the same diameter bars, they concluded that the decrease in shear strength in the large beams was mainly due to the increase in the bar diameters which had poorer bonding properties.

Analyzing the current equation of the ACI code, Kani\textsuperscript{1} tested four series of beams. The height of beams ranged from 150mm to 1220mm between the series. The reinforcing ratio of 2.8\% and the concrete compressive strength of 26MPa was kept constant while the \textit{a/d} ratio varied within each series. Kani reported that the unit shear strength decreased significantly when the depth of beam increases. A reduction of 40\% in the failure shear stress was recorded in the 1220mm beams in comparison to the 150mm beams. Kani also pointed out that the crack lengths and geometry were slightly different according as the depth of the beams changed.

Taylor\textsuperscript{4} tested fifteen beams in a series whose depth varied from 250mm to 1000mm. In addition, he varied the maximum aggregate size while the reinforcing ratio was kept constant at 1.35\%. A reduction of shear strength was observed but it was less dramatic when the maximum aggregate size was properly scaled. Taylor considered that the strength of Leonhardt's large beams was partially reduced due to reduced dowel effects when the bars were placed in two layers and due to a reduced aggregate interlocking strength since the maximum aggregate size remained constant in all tested beams. Following Kani, Taylor noted and recommended a reduction in shear strength by up to 40\% when large beams were in question.

A number of tests proved that the shear strength was also dependent on the reinforcing ratio. For instance, Rajagopolan and Ferguson\textsuperscript{5} tested ten and analyzed 27 other beams with reinforcing ratios from 0.25\% to 1.73\%. They concluded that the shear strength would be significantly reduced if the reinforcing ratio were to be dropped below 1\%.

Recent research by Shioya et al\textsuperscript{6} reinforced Kani's conclusion concerning size effects while emphasizing the importance of the aggregate size and the low reinforcing ratio. Their beams had a constant reinforcing ratio of 0.4\% and concrete a compressive strength of 23.5Mpa while the effective depth of the beams varied from 100mm to 3000mm. The maximum aggregate size ranged from 1mm to 25mm. It was found that the shear strength was reduced as the maximum aggregate size decreased. The same effects were observed even in the beams whose depth was in excess of 1000mm. Their tests indicate that the shear strength of large beams depends on the aggregate interlocking strength, which is significantly reduced when the critical shear crack widens.
Code provisions, which require beams with a minimum shear reinforcement when the shear is more than 50% of the shear capacity, were primarily developed to overcome the unexpected loads and the unwanted brittle nature of members with no web reinforcement subjected to shear. The same provisions would perhaps have prevented the shear failure of the large beams at Wilkins Air Force Depot warehouse. However slabs, footings and joists are excluded from these provisions because of their multi load sharing capacities, which permit the load to be shared between weaker and stronger areas.

In addition, slabs are one of the few structural elements that might have very low reinforcing ratios and no shear reinforcement. When they are used in the design of underground box structures their thickness extends to limits where they are clearly classified as large beams. Therefore, the shear design of slabs in underground box structures might be unconservative.

Concern about the shear capacity of thick concrete slabs was expressed in regard to the box structures for the proposed Toronto Transit Commission’s Eglinton Extension Line. Slabs with a thickness of up to 1400mm were classified as large beams. Providing minimum shear reinforcement to both top and bottom slabs eliminated any uncertainty regarding the possibility of shear failures. However, it was questioned whether the additional shear reinforcement was really required or the original design with no ties would have been adequate.

1.2.2 LOCATION OF THE CRITICAL SHEAR SECTION WHEN UNIFORM LOADING IS APPLIED

Since tests with applied uniform loading are more complex and expensive, most of the research conducted was based on beams subjected to single or double concentric loads acting at distance $a$ away from the supports. From Kani’s graph (Figure 1.3) it can be observed that due to arching the shear strength increases significantly when a concentrated force is shifted towards the supports and the $a/d$ ratio is less than about 2.5.

Concrete codes, which express shear strength only in terms of concrete compressive strength (for instance, the ACI Code 7), assume that the critical shear section would be near the support where the maximum shear force is expected. Specifically, the critical section is determined to be the distance $d$ from the support and any load near to the support would be transferred by a compressive $45^\circ$ strut.
Figure 1.3 - Shear Stress at Failure versus $a/d$ (Adopted from Kani)
Following the General Method of the Canadian Code, it becomes clear that the moment increases longitudinal strains and widens the flexural-shear cracks. This consequently leads to reduction of aggregate interlock strength and the total shear strength of the member. Since maximum moments and shear forces usually do not occur at the same place, it becomes harder to pinpoint the location of critical shear sections. Based on tests concerning simply supported beams subjected to uniform loading, it was found that shear failures often occurred at a distance of 0.2L from supports where both the shear forces and moments were significant. This explains why certain codes require the shear strength to be checked not only near the supports but also at every 10th point of the span.

If Kani's experiment was so conclusive when concentrated loads were considered, could the same analogy be used in the case of uniform loading? Is the critical section located at distance d from the support or is it appropriate to check the shear capacity at every 10th point of the span? The entire concept becomes even more complex when additional negative moments are introduced at the ends of the beams. For instance, the ends of slabs in box structures are subjected to negative moments. To answer this and other questions it was decided to build and test one box structure.

The building, testing and analyzing of a concrete box structure would help to better understand the detrimental size effects. It would indicate the unconservative nature of shear strength expressions in some current North American codes. The adequacy of additional shear reinforcement in the design of the TTC's underground boxes could therefore be more easily addressed. In addition, the testing of a box shaped specimen with uniform loading would help to determine the expected critical shear sections. Moreover, the non-linear behavior of the box structure after the cracking of concrete would permit a better understanding of the nature of these structures under monotonic loading.

1.3 HOW TO TEST THE BOX STRUCTURE?

One of the single cell box structures, which had to be built at the cross over section of the proposed Eglinton West Subway Extension Line, was selected for testing. Due to its size the box was scaled down to 45% and only one 960mm thick slice was built and tested. To ease building and testing of the specimen, some minor modifications had to be made.

The diameters of the reinforcing bars and the maximum aggregate size were also scaled down while the reinforcing ratio was maintained the same as it was in the original design. The
minimum yield strength of rebars and concrete compressive strength were chosen to match corresponding properties specified on the original drawings. The main difference was in the shear reinforcement that was omitted in the specimen modeling.

Only vertical uniform loading was considered in the test and was simulated by seven point loading applied perpendicular to the exterior face of each slab. This was accomplished by seven sets of double jacks, which acted against the exterior faces of both slabs. Figure 1.4 pictures the steps involved in the realization of the experimental program pertaining to real underground conditions.

For more details about specimen geometry, the material properties, and loading or instrumentation please refer to the next chapter.

1.4 CURRENT CODES

In addition to the experiment, the shear strength of the slabs in the specimen was determined by current North American codes. The following section summarizes the shear strength expressions and corresponding clauses presented in the ACI, AASHTO, and OHBDC codes

1.4.1 ACI-95 Code

When members are subjected to shear and flexure only, one can use the traditional expression 11-4 to determine shear strength.

\[ V_c = 0.167 \sqrt{f_c b_w d} \quad \text{[mm and MPa]} \]

A slightly different expression 11-5 for shear strength is sensitive not only to concrete compressive strength but also to the reinforcing ratio and the shear-moment ratio

\[ V_c = \left( 0.16 \sqrt{f_c} + 17 \rho_w \frac{V_u d}{M_u} \right) b_w d \leq 0.29 \sqrt{f_c b_w d} \quad \text{[mm and MPa]} \]

While \[ \frac{V_u d}{M_u} \leq 1 \]

When a simple supported beam is subjected to two point loading.
\[ \frac{V_u d}{M_u} = \frac{d}{a} \leq 1 \quad \text{or} \quad d \leq a \]

which implies that the critical section should be located at a minimal distance \( d \) from the support. When \( V_u > 0.5\phi V_c \), Clause 11.5.5.1 requires minimum shear reinforcement in all flexural members in accordance with clause 11.5.5.3 and equation 11-13.

\[ A_v = 0.35 \frac{b_w s}{f_v} \quad \text{[mm and MPa]} \]

Slabs, footings and joists are excluded from this clause.

1.4.2 AASHTO-94 Code\(^8\) – Design for Shear in Slabs of Box Culverts

In Clause 5.14.5.3 shear strength is expressed as

\[ V_c = \left( 0.178 \sqrt{f_c} + 32 \frac{A_v V_d}{bd M} \right) bd \leq 0.332 \sqrt{f_c} b_w d \quad \text{[mm and MPa]} \]

but \( V_c \geq 0.25 \sqrt{f_c} b \ d \) for single cell culverts with slabs monolithic with walls

1.4.3 CSA-94 Code\(^9\) – Simplified Method 11.3

Clause 11.3.5.2 states that the shear strength of members with an effective depth greater than 300mm and no shear reinforcement should be calculated according to equation 11-7.

\[ V_c = \left( \frac{260}{1000 + d} \right) \lambda \phi_c \sqrt{f_c} b_w d \geq 0.10 \lambda \phi_c \sqrt{f_c} b_w d \]

Even though the expression is simple and easy to use, the size effects are still accounted for to some extent through the effective depth \( d \) expressed in terms that are enclosed in the brackets. It should be noted that the coefficients 260 and 0.10 in these CSA expressions were calibrated to account for the low value of \( \phi_c \) used in this code (\( \phi_c=0.6 \)). When using these expressions to compare with experimental results \( \phi_c \) is taken as 1.0 but the coefficients are taken as 217 and 0.084.
1.4.4 ASSHTO-94, CSA-94, OHBDC-91\textsuperscript{10}. General Method

The general method was based on Modified Compression Field Theory. It recognizes size effects in large beams with no shear reinforcement and was accepted in the last 10 years by several North American Standards. Small variations, mainly related to notations were noted between the codes.

The following equations are found in the CSA-94 Code.

The shear strength of reinforced concrete beams with no shear reinforcement is calculated using the equation

\[ V_c = \beta \sqrt{f'_c b_s d_t} \]

where \( \beta \) is determined from either tables or figures and is based on the values of axial strain \( \varepsilon_t \) and crack spacing \( s_c \).

Longitudinal strain is calculated from the equation

\[ \varepsilon_t = \frac{0.5(N_f + V_f \cot \theta) + M_f / d_v - A_p f_p}{E \varepsilon_A + E_p A_p} \]

while crack spacing shall be considered either as the effective depth or the maximum distance between the layers of crack control reinforcement, whichever is smaller.

Equations reviewed in this chapter were used in predicting the shear strength of the specimen's slabs and the relevant calculations are presented in Chapter 4.
2. Description of Experimental Program

2.1 SPECIMEN GEOMETRY AND REINFORCEMENT LAYOUT

To ease building and to make testing possible, the specimen had to be scaled down and modified. Chapter 2.1 describes the governing conditions in choosing the specimen size and all the modifications that were implemented. Specimen geometry, the choice of principal reinforcement and rebar layout were also described in this chapter.

2.1.1 SPECIMEN SIZE AND SHAPE

The scale factor of 45% was governed by crane lifting capacity, the maximum capacity of the test loading assembly and the volume of concrete that could be delivered to the laboratory in one mixer truck.

In order to strip the formwork and utilize the bottom face of the specimen during testing, the specimen had to be lifted clear of the concrete floor by at least 0.4m by a 10 tonne crane. If the specimen was to be carefully elevated in small increments by lifting only one side at a time, the maximum weight of the specimen had not to be more than 210kN.

In addition to the weight limitation, the shear and flexural capacity of the slabs governed the size of the specimen. It was crucial to ensure that testing could reach the failure stage. Therefore, the maximum loading that could be applied by a series of jacks during testing had to be greater than the predicted failure load if the specimen was assumed to fail in either shear or flexure.

In addition to symmetrical loading, geometry and reinforcement, the consistency of the concrete mix was one of the key issues in order to obtain symmetrical behavior of the specimen. The only way to achieve this consistency was to deliver ready mix concrete in one truck. The maximum volume of concrete that could be transported in the mixer truck was about 9m$^3$. It was safe to assume that the volume of the specimen had to be less than 8.5m$^3$ if 5% of the concrete was accounted for in forms of miscellaneous losses.
There was a prior intention to preserve the cross-sectional shape of the originally designed structural unit. However, modifications had to be made mainly to simplify construction, testing and analysis of the specimen. Nevertheless, they were minor and they did not have a significant impact on the test and the results. The original box (Figure 2.1) had roof crown sloping 1.8% down from the centerline of the structure towards the exterior faces of the walls. This way, water was easily drained from the roof to both sides of the structure. The crown was eliminated and the average thickness of the roof slab was used in modeling. Therefore the original top slab thickness that varied from 1400 to 1460 mm was scaled down to a uniform thickness of 650 mm. The thickness of the invert slab remained uniform but was also scaled down to 630 mm.

Likewise, the thickness of both walls were scaled down from 750 to 340 mm. Reinforced concrete 45° haunches and catwalks were modeled at the wall interfaces with the slabs as they were originally designed. The purpose of the catwalks was not only to stiffen the structure's bottom corners, but also to provide maintenance crews with a safe place to stand and yield to oncoming trains.

In addition, fourteen φ100mm circular holes were formed in the slabs at 783mm spacing. Seven holes in the roof slab were aligned with corresponding holes in the invert slab in order to accommodate steel tension rods during testing. Tension rods were part of the test loading assembly and their function is described latter in this chapter. A concrete outline of the specimen is shown in Figure 2.2.

2.1.2 REINFORCEMENT

The building of the actual cast-in-place box structures can be divided into three stages, which are represented in Figure 2.3. First, the invert slab and catwalks are cast on top of a mud slab. Second, reinforcement is placed in the walls and spliced above catwalks with exposed embedded rebars in the invert slab. Also, some of the rebars are extended to the top slab. The casting of the walls up to the bottom of the haunches completes this stage. At the last stage, reinforcement is placed and spliced in the slab above the walls. The concrete is cast and the haunches and the roof slab are formed. In order to achieve proper moment and shear transfer between walls and slabs, special attention is given to the construction joints located at the bottom and top of the walls. Since the test specimen was cast horizontally, there was no need for construction joints and they were therefore eliminated. However, splicing of the reinforcement was maintained to closely model the original box unit.
Figure 2.1 – Original TTC Box Unit
Figure 2.2 – Specimen Outline
In order to study the shear response of a large structural element with no shear reinforcement, stirrups were intentionally omitted from the roof and invert slab. The model thus represented the original design which did not contain stirrups. Considering that the walls were not expected to experience significant shear forces, stirrups were also eliminated from the walls in order to simplify construction. Therefore, the test specimen contained only principal and temperature reinforcement, which is shown in Figure 2.4.

The principal and temperature reinforcement used in the test specimen was scaled down in bar size and spacing while the reinforcement ratios were maintained approximately in the same way as corresponding ratios in the original TTC design. Bars were placed in ten strips. Spacing between the strips was 92 mm. An exception was made between strips 5 and 6 where the spacing was increased to 132 mm to accommodate 100mm PVC tubes. These tubes were used to form tension-rod openings in both slabs. The number, size and length of bars varied not only between the strips, but also within each strip. Even though splices were scaled to 45% of their original length: most of the reinforcing details were still in accordance with the CSA code requirements. The principal reinforcement in walls and slabs consisted of 15M and 20M bars, while US #3 rebars were placed in haunches and catwalks.
Figure 2.4 – Specimen Reinforcing Layout
According to the TTC drawing, single and twin 35M rebars, alternated at 150mm spacing, were placed along the tension faces of both slabs where maximum moments were expected. This reinforcement was modeled with fourteen 20M rebars. Ten rebars were located in each strip and an additional 4 rebars were added to every third location. Originally, the exterior top slab reinforcement consisted of 30M rebars spaced at 300mm. This reinforcement was mainly used to ease the installation of shear ties and was modeled with four 15M rebars placed at every third strip. Likewise, 35M rebars at the exterior layer of the invert slab were modeled with four 20M rebars.

Since the specimen had vertical axes of symmetry, both walls had the same geometrical and reinforcement layout. Interior 25M bars spaced at 300 mm were provided to secure the wall ties in place and to resist any moment that might be the result of lateral earth pressure applied to the exterior faces of the walls. They are modeled with four 15M bars placed at every third strip. More reinforcing bars were supplied in walls near the exterior face since the negative slab moments at each corner were transferred to the walls. Therefore, L-shaped and straight 30M and 35M rebars were added to the exterior layer of both walls. These rebars were substituted with 20M bars in the test specimen.

During construction of the specimen, haunches and catwalks were reinforced with four US#3 bars, placed at every third strip.

The proper choice of temperature reinforcement and its spacing is important in the control of shrinkage cracks. Since rebars embedded in the bottom slab restrain the lower portion of the walls from shrinking, vertical shrinkage cracks may develop in the walls. By the same analogy, similar cracks are expected in the roof slab and haunches, since the slab would be partially prevented from shrinking by the walls. Shrinkage cracks may also develop through the entire section when structural units are very long. In the original design, temperature reinforcement was placed on the interior and exterior faces of both slabs and the walls. Slab 25M bars at 300 mm spacing were modeled with 15M bars at every 250 mm. Similarly, 10M bars at 275 mm, used in the walls, were modeled with 15M bars at 250 mm spacing.

Since box structures are exposed to harsh underground conditions, a sufficient concrete cover is required to extend their lives. The following clear concrete covers were accepted in the design of the TTC box unit.
DESCRIPTION OF EXPERIMENTAL PROGRAM

<table>
<thead>
<tr>
<th>Location</th>
<th>Clear cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>from top beam exterior face to principal reinforcement</td>
<td>75mm</td>
</tr>
<tr>
<td>from top beam exterior face to stirrups</td>
<td>60mm</td>
</tr>
<tr>
<td>from top beam interior face to principal reinforcement</td>
<td>50mm</td>
</tr>
<tr>
<td>from wall and invert slab exterior face to principal reinforcement</td>
<td>100mm</td>
</tr>
<tr>
<td>from wall and invert slab exterior face to stirrups</td>
<td>70mm</td>
</tr>
<tr>
<td>from wall and invert slab interior face to principal reinforcement</td>
<td>50mm</td>
</tr>
</tbody>
</table>

Except for the concrete cover at the exterior face of the top beam, which was increased by 20 mm to account for the roof crown, the concrete covers were scaled and they are shown below:

<table>
<thead>
<tr>
<th>Location</th>
<th>Clear cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>from top beam exterior face to principal reinforcement</td>
<td>55mm</td>
</tr>
<tr>
<td>from top beam interior face to principal reinforcement</td>
<td>25mm</td>
</tr>
<tr>
<td>from wall or invert slab exterior face to principal reinforcement</td>
<td>50mm</td>
</tr>
<tr>
<td>from wall or invert slab interior face to principal reinforcement</td>
<td>25mm</td>
</tr>
</tbody>
</table>

2.2 BUILDING OF SPECIMEN

The specimen was constructed within a period of four months in the following sequence.

- All principal and temperature reinforcing bars were measured, cut and bent in accordance with pre-defined lengths.
- Selected bars were strain-gauged. Ninety-six strain gauges were attached to the principal reinforcement and this step was completed within four weeks. Additional information regarding the type and location of strain gauges can be found later in this chapter.
- The base of the formwork had to be built before reinforcement was assembled in the cage. Two layers of 19mm thick plywood were secured together with screws at 300mm spacing. Top plywood was used to outline the geometry of the specimen and to partially support the formwork plywood walls. Considering that many long and intricately shaped specimens exhibited shrinkage cracks after casting, some improvements were made to the formwork. To allow lateral movement due to shrinkage, the formwork was designed in such a way that the wall panels could be quickly and easily stripped. Moreover, the friction between specimen and the floor was minimized through a thin film of grease wrapped by two sheets of plastic and placed between plywood panels below the specimen. Thus shrinkage cracks were
eliminated by allowing the specimen and top sheets of plywood to float on a thin layer of grease.

- The reinforcing cage was built on top of the formwork base. Reinforcement was assembled in smaller segments (corners, and wall and slab panels). These segments were connected together and tied with additional rebars. The reinforcing bars positioned at the catwalks and haunches were added at a later stage to the cage. Wood templates were used to accurately control spacing between bars, while plastic chairs were used to position the bars against the formwork. Lifting hooks were bent from 25M bars into a U-shape and placed at midspan and at each end of both slabs. Once the cage had been completed, all strain gauge wires were neatly bundled and protected by duct tape. Photos of the cage are shown in Figures 7.5 to 2.12.

- 100mm diameter PVC tubes were positioned in the top and bottom slabs at 783mm spacing before the rest of the formwork was constructed.

Formwork walls were built using 19mm thick plywood panels, which were supported by steel frames. The plan view of the formwork is shown in Figure 2.13. To ease stripping of the formwork after the specimen was cast, a thin layer of oil was applied to the interior formwork surfaces that had sustained contact with concrete. Steel frames consisting of C- and I-shaped horizontal beams were connected to triangular steel soldiers by structural bolts. Soldiers were screwed to the base of the formwork while the upper portion was strengthened by 5mm steel ties. Ties were also installed through PVC tubes. Figures 2.14 to 2.17 picture the assembled formwork prior to casting.

- Concrete was delivered in one mixer truck and placed into the formwork in three layers. Workability of the concrete was enhanced by a plasticizer and two vibrators (25mm and 50mm in diameter). Standard 150mm cylinders were cast at the same time and left to cure beside the specimen. The top exposed concrete surface was finished one hour after casting and the whole specimen was covered with wet burlap and plastic for seven days. The steel framing and interior vertical formwork walls were stripped two days after casting. Floating on a thin layer of grease allowed the specimen to shrink and the corresponding cracks were successfully avoided. The rest of the formwork was removed seven days after casting. The casting of the specimen is shown in Figures 2.18 and 2.19.

- Six small concrete pedestals had to be built. They supported the elevated specimen at the haunches, catwalks, and midspans of the slabs. Once the specimen had been lifted, two plain rubber pads were centered on top of each pedestal and positioned underneath the specimen.
Figure 2.5 – Reinforcing Cage – Looking South

Figure 2.6 – Reinforcing Cage – Looking West
Figure 2.7 – Reinforcing Cage – Exterior Reinforcing Layer of Roof Slab

Figure 2.8 – Reinforcing Cage – Interior Reinforcing Layer of Roof Slab
Figure 2.9 – Reinforcing Cage - Exterior Reinforcing Layer of East Wall

Figure 2.10 – Reinforcing Cage - Interior Reinforcing Layer of West Wall
Figure 2.11 – Reinforcing Cage – Roof/wall Connection at Top Corner

Figure 2.12 – Reinforcing Cage – Invert Slab Adjacent to Catwalk
Figure 2.13 – Formwork – Plan View
Figure 2.14 – Formwork – Looking South

Figure 2.15 – Formwork – Looking West
Figure 2.18 - Casting of Specimen

Figure 2.19 - Casting of Specimen – Vibration of Concrete
These pads had to support the specimen weight and to accommodate any lateral displacement of the specimen during testing.

- PVC tubes were pulled out of the hardened concrete leaving fourteen tension-rod holes in the slabs. The specimen was painted white and prepared for the instrumentation and the test set up which is described in the following sections of this chapter.

2.3 MATERIAL PROPERTIES

Space limitations, the magnitude of the applied loads and construction costs are some of the governing factors in selecting the type of structure and the appropriate construction materials. Generally, underground box structures do not have strict space constraints. In most cases, interaction is only with utilities, which can be easily relocated. Also, the underground boxes usually span less than the bridges or buildings and consequently their factored loads are considerably lower. However, the scope of the work involved in building the subway box structures is usually significant. Therefore, the cost of construction usually dictates the method of construction and the materials to be used. Thus, box structures are usually designed as single or twin cell rigid frame structures with thick slabs and walls built with normal strength concrete and standard widely available reinforcing bars. To minimize construction costs, the use of post tensioned, prestressed or high strength concrete is generally avoided.

2.3.1 CONCRETE

In order to closely model the chosen TTC box structure, it was decided to build the specimen with the concrete properties similar to those specified in the original design. Therefore, a 35MPa 28-day uniaxial compressive strength of concrete and a 10mm maximum aggregate size were chosen. Since the supplier had a tendency to provide concrete with somewhat higher strength than requested, “25MPa” concrete was ordered and cast. The standard 150mm cylinders were cast and cured beside the specimen in a similar condition. They were tested under uniaxial compression at various times and the results are shown in Figure 2.20. Stress-strain curves of five concrete cylinders were obtained two days after the specimen had been tested and they are shown in Figure 2.21.

To improve the workability of fresh concrete, the superplasticizer was added prior to casting, until a slump of 100mm was obtained.
Figure 2.20 - Concrete Stress-Strain Curves Two Days After the Test

Figure 2.21 - Increase of Concrete Uniaxial Compressive Strength over Time
2.3.2 STEEL

Principal reinforcement was modeled with three different bar types. Their stress-strain properties are listed in Table 2.1 and shown in Figure 2.22. The yield stress of 400MPa specified in the TTC design is somewhat lower than the actual yield stresses obtained from the bars used in the specimen. This should be considered in the design of specimen bar splices, the computer modeling of the members and the prediction of the shear strength of slabs. All the bars exhibited distinct yield plateaus and strain hardening.

2.4 LOADING SCHEME

Seven parallel concentrated forces, which were applied at the exterior faces of each slab at 783mm spacing, simulated vertical uniform loading as accounted for in the original design (see Figure 2.23).

Each force was applied by a set of two 50-ton single-acting jacks. The base of each jack acted against the top surface of the roof slab while the jack head reacted against the invert slab through the assembly of cross beams, spherical bearings, steel tension rods and load cells (Figure 2.24). In order to successfully simulate uniform loading, the jack loads had to be of the same intensity at all seven locations.

The bottom of each jack was positioned on a 38mm thick steel base plate. The machine recessing of these plates provided an accurate alignment of the jacks. Each plate was attached to the roof slab with four anchor bolts that carried not only the weight of the plate but also the weight of the jacks, a cross beam and a tension rod. The base plates were also used to partially distribute the applied load from the jacks to the specimen preventing localized crushing of the concrete contact surface.

The roof slab cross-beams consisted of two 38mm (1.5in) thick trapezoidal steel plates connected by three 51mm (2in) thick plates, which were recessed to accommodate jack heads and tension rods. Thus, each jack was precisely aligned between the base plate and the cross-beam.

The main function of the cross-beams was to transfer reaction forces from the jack heads to the tension rods. In the design of the cross-beams additional consideration had to be given to their
Figure 2.22 - Reinforcement Stress-Strain Curves

Table 2.1 - Reinforcing Steel Properties

<table>
<thead>
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<th>Size</th>
<th>Area</th>
<th>$f_y$</th>
<th>$f_u$</th>
<th>$A_o$</th>
<th>$A_m$</th>
<th>$E_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>US #3</td>
<td>70.9</td>
<td>508</td>
<td>778</td>
<td>2.4</td>
<td>8.75</td>
<td>8000</td>
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<tr>
<td>M20</td>
<td>300</td>
<td>492</td>
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</tr>
<tr>
<td>M15</td>
<td>200</td>
<td>484</td>
<td>646</td>
<td>2.3</td>
<td>18.4</td>
<td>3410</td>
</tr>
</tbody>
</table>

* - Initial slope of strain-hardening curve
Figure 2.24 – One Set of Loading Assembly
stability, because the jack heads and the tension rods were aligned such that the corresponding forces acted along the same plane.

Tension rods were used to transfer the reaction forces between opposite beams. High strength 44mm-diameter steel bars were fabricated into fourteen short rods. These rods were coupled together with seven load cells, and fabricated out of 63mm-diameter high strength bars. Each side of the tension rods was threaded for coupling with load cells and in-house manufactured bolts.

To prevent the tension rods from bending, spherical bearings were placed between the rod ends and the cross-beams. By allowing rotation at the bearings, hinges were created at each end of the tension rods. Therefore, each rod functioned as a truss member transferring only the tension force between the roof and invert slab.

Jack reaction forces were transferred through tension rods to C-shaped cross-beams. These beams spread the reaction force to the exterior face of the invert slab. Each C-shaped beam was bolted to the invert slab with two anchor bolts. Spacing between the crossbeams was maintained at the same level at both slabs. By setting all tension rods so that they were parallel to the centerline of the specimen, applied loads were also parallel, equally spaced and perpendicular to the exterior faces of the slabs.

The test loading assembly was designed to fail the specimen either in shear or flexure and was tested up to a load of 800kN/m2. Jack loads were controlled by the load maintainer and all fourteen jacks had a similar friction coefficient. Therefore, the jacks produced loads with almost the same intensity at various oil pressures. All hardware used in the test setup was fabricated with high accuracy. Figures 2.25 and 2.26 picture the loading assembly. Shop drawings of cross-beams, load cells, spherical bearings and other hardware are presented in Appendix A.

2.5 INSTRUMENTATION

Prior to testing, the specimen was equipped with instrumentation, which was used to monitor and record applied loading, deflections and strains. A number of electronic strain gauges, Linear Variable Differential Transformers (LVDTs) and Zurich targets and gauges were used to obtain the data. In addition to 35mm photos, the entire test was recorded on videotape.
Figure 2.25 – Loading Assembly – Roof Slab Cross-beams

Figure 2.26 – Loading Assembly – One Set of Jacks, Cross Beams and Tension Rods
2.5.1 REFERENCE SCHEME

To easily understand the terminology related to different specimen components and their faces, a reference scheme was developed and is shown in Figure 2.27. The specimen was set in a horizontal position and oriented in the same direction as it was built. The top surface and soffit of the specimen outlined the cross-sectional shape of the modeled structure. The roof slab faced north while the invert slab was oriented towards the south. The west and east walls were named according to their geographical orientation. West and east haunches and catwalks were referenced in the same fashion as the walls, and they were located at the interfaces of the walls and the slabs.

2.5.2 LOAD CELLS

 Loads applied to the specimen were monitored at two different locations. The oil pressure supplied to the jacks was checked by a pressure transducer installed adjacent to a load maintainer. This transducer was used to monitor pressure at the main line, which supplied oil simultaneously to all the jacks.

To simulate uniform loading, it was necessary to ensure that all jack-induced loads had the same intensity. It was not practical to monitor the pressure at each individual jack. Also, jack loads could only be estimated because of the friction that developed around the jack heads at low oil pressures. Therefore, seven load cells were coupled to the tension rods, which transferred reactive forces between the roof and invert slabs. Each cell was equipped with four electronic strain gauges and corresponding strains were averaged to eliminate any bending effects. Knowing strains, modulus of elasticity and cross-section areas it was easy to find the corresponding reactive tension forces being transferred by the rods from the top to the bottom slab. Prior to testing, load cells were calibrated to directly display tension forces. One of seven load cells is shown in Figure 2.28.

2.5.3 STRAIN GAUGES

In order to analyze the behavior of the specimen and capture key points such as the cracking of the concrete, the yielding of the reinforcement or failure load, it was important to obtain a continuous record of strains at selected reinforcing bars. A number of 5mm long electronic strain gauges were attached to the principal reinforcement located in the walls and slabs. After gluing the gauges to the bars, they were protected and wired to a data acquisition system. Fifty-nine out of the eighty-four gauges were placed west of the specimen centerline.
Figure 2.27 - Reference Scheme

Figure 2.28 - Load Cell
The goal was to capture the strains near the shear failure plane regardless of where it may occur. Since it was hard to predict at which corner the specimen would fail, an additional 25 gauges were positioned at the east end of the specimen. These gauges were also used to check if there was any discrepancy in symmetrical behavior due to the cracking of concrete or the possible imperfections in the specimen geometry or loading. Particular locations had two or three gauges usually bonded on the rebars in the first, fourth or tenth strip. By doubling and tripling the gauges, it was ensured that all strategically selected strains were recorded even if one of the gauges were to fail. The exact position of each strain gauge is shown in Figure 2.29.

The main concentration of gauges was on rebars of both slabs adjacent to the interior and exterior faces at locations of expected maximum moment and shear failure. Some of the gauges were positioned along slab principal reinforcement to capture the strain pattern along the bars. For example, one of the rebars at the interior face of the top slab had nine strain gauges bonded along its entire length. Twenty-nine and twenty-seven strain gauges were placed respectively on reinforcing bars of the roof and invert slabs.

A similar scheme was used in strain gauging of the principal wall reinforcement. The gauges were bonded along rebars to observe the strains in the walls between the haunches and the catwalks.

Reinforcing bars at the haunches and the catwalks were used mainly for detailing purposes since these areas were assumed to be in compression when the specimen was subjected to uniform loading. Therefore, there were no strain gauges in either the haunches or the catwalks.

2.5.4 LVDTs

Since strain gauges could only capture strains along reinforcing bars, Linear Variable Differential Transformers (LVDTs) were used to record a continuous record of average concrete surface strains and deflections. Forty-one LVDTs were used and their gauge lengths ranged from 350 to 4800mm.

Four LVDTs were installed to record the total deflections between the opposite interior faces of the slabs and the walls. Eleven gauges were mounted on the soffit of the specimen to capture diagonal concrete strains at locations of possible shear failure. Other LVDTs were attached to the interior and exterior faces of the slabs and the walls at locations of expected maximum moment or at critical shear sections. None of the gauges were mounted on the top surface of the specimen since they would obstruct the space required for a Zurich recording. Figure 2.30 shows the LVDTs layout.
Figure 2.29 - Layout of Strain Gauges
2.5.5 ZURICH TARGETS

In terms of strains, neither strain gauges nor LVDTs could cover the entire specimen. A grid had to be laid down on the top surface and the distance between the grid points was measured at different load stages. Zurich targets (disk-shaped aluminum buttons) were glued to concrete surfaces at each grid point. They were designed to facilitate fast and accurate distance recording with mobile transducers called Zurich gauges. By recording initial and final lengths, it was relatively simple to find the corresponding surface strains.

Zurich targets were arranged in square 200x200mm and 300x300mm grids. Five hundred and eight readings were recorded between one hundred and seventy-eight targets just prior to testing and at each of the five consecutive load stages. The layout of the Zurich targets is shown in Figure 2.31.

2.5.6 VIDEO AND 35mm CAMERAS

Two video cameras were positioned north and south of the specimen and they recorded the overall behavior of the specimen during the entire period of testing. In addition, color photos of the specimen, crack patterns and the test set-up were recorded by two 35mm cameras. Also, all stages of construction, including finished formwork, the steel cage and casting were recorded on 35mm color film.
Figure 2.31 - Zurich Targets
3. Test Observations and Results

All records obtained by the strain gauges and LVDTs were collected at specific loading points as well as at regular time intervals. Data was sent to a data acquisition system, organized in sets and stored in one computer text file. This chapter summarizes observations based on the stress strain curves obtained by LVDTs and strain gauges. Strains obtained from Zurich readings and crack patterns were collected at 5 loading stages.

3.1 LOADING

The specimen was subjected to monotonic loading simulated by fourteen jacks until the point of failure. Equivalent uniform loading \( q \), used in this chapter, was obtained from reactive forces at seven tension rods.

Reactive forces calculated from recorded strains at seven load cells were averaged and presented in the form of equivalent uniform vertical loading that would be applied to specimens in real underground conditions. The difference in recorded loads at load cells was less than \( \pm 2.5\% \) at any stage of the test and less than \( \pm 1.5\% \) just prior to failure. The test was completed in two days and the load history is shown in Figure 3.1.

Seven load stages were set up at particular loading points in order to obtain the necessary Zurich readings, take photos and observe crack patterns. At each stage, the load was reduced on average by approximately 11% to ensure the safety of technical personnel. After the completion of all tasks at one load stage, the specimen was loaded until the next load stage was reached. The first three load stages were set up on the first day and the remaining stages were completed on the second day.
In order to minimize concrete creep effects and avoid test-related hazards, the specimen was unloaded to 25.5 kN/m² overnight between two consecutive test days. At the beginning of the second day, the load was brought back to the maximum level recorded at the third stage, and then cycled from 198.8 kN/m² to 29.5 kN/m² two times. Because it was known that cracks should be exhibited on the soffit and the top surface of the specimen, load cycling was done in the order to break the thick layer of white paint that partially covered the crack pattern. Since strain gauges and LVDTs did not record any significant changes of strains, all cycling related to data was omitted in order to maintain the clarity of the presented graphs.

Once failure was reached, the applied load was quickly removed to preserve the specimen from further damage, and then after photographs had been taken it was reloaded to find the post failure capacity.
3.2 TEST OBSERVATIONS

The specimen was loaded until shear failure occurred in the roof slab, adjacent to the west haunch, at a load of 347 kN/m². Overall behavior of the specimen prior to failure can be observed in Figure 3.24, which depicts the load-deflection curve recorded by LVDTs between opposite interior faces of the slabs.

Detailed observations are presented in the following text in chronological order as loading was applied. They are grouped within the seven load stages, such that any changes noted prior to or at the load stage are presented together.

The reference scheme described in 2.5.1 is used. Roof slab was oriented to the North, invert slab to the South while east and west walls were named according to their geographic orientation. The mid-height of the specimen refers to the horizontal plane through the tension rods.

3.2.1 LOAD STAGE 1

The initial localized cracks at mid-height of the roof and invert slabs were captured by strain gauges located at the midspan. It can be observed from Figures 3.2 and 3.10 that these cracks occurred at about 50kN/m². When the load was increased to 73kN/m², cracking was more evident, not only at the mid-height but also near the top and bottom gauge in the roof slab. Compressive strains recorded along exterior reinforcing bars at the same section were less sensitive in capturing initial cracking, but it can be observed that the slope of the load strain curves, as shown in Figure 3.2, changes at a load of 73kN/m².

The load was reduced from 73kN/m² to 65kN/m² and the specimen was visually inspected. However, signs of cracking were not found on concrete surfaces. Photographs of the specimen at this load stage are presented in Figures 3.53, 3.54, 3.65, 3.69 and 3.77.

Zurich readings were obtained at reduced loads and the corresponding average strains are shown in Figures 3.40 and 3.41. A maximum average tensile strain of 190µs was recorded by the Zurich gauge along the interior edge of the invert slab above the second tension rod from the East Side. A high compressive strain at the same section but along the exterior edge indicated additional cracking of the invert slab approximately 3m east of the midspan. However, relatively low strains, which are shown in Figure 3.13, were captured along the bottom reinforcing bar at
the same location. Perhaps cracking occurred between the tension-rod opening and the top surface of the invert slab. A maximum roof slab tensile strain of 98.8 με was recorded at midspan where maximum moments were expected. Surface wall strains were slightly lower than strains at slabs reaching a maximum of 70με along exterior faces. The maximum wall compressive strain was detected at the interface of the west wall and the catwalk.

In addition to Zurich readings and the visual inspection of the specimen, this load stage was used to ensure that all instrumentation and loading equipment were functioning properly prior to further testing.

3.2.2 LOAD STAGE II

When the specimen was subjected to a load of 101kN/m², the initial crack at midspan of the invert slab propagated towards the top surface. Through an additional load of 5kN/m² this crack extended downwards to the specimen soffit (see Figure 3.10). Compressive strains recorded along the bars at the same section indicated a less pronounced cracking at the same load (see Figure 3.10). Strain gauges and diagonal LVDTs, which were located in slabs approximately 1m west and east of midspan, recorded new cracking as the load was further increased from 107kN/m² to 132kN/m². This is shown in Figures 3.3, 3.11, 3.34 and 3.35. Cracking of the slabs at midspan was also captured by LVDTs mounted on interior faces. Even LVDTs on exterior faces at the same sections captured the cracking of the concrete as increments of compressive strains slightly increased at a load of 101kN/m². Load-strain curves recorded by LVDTs on interior and exterior faces of slabs at midspan are shown in Figures 3.26 to 3.29.

Wall cracking was detected at 107kN/m². Looking at Figures 3.16, 3.17, 3.20 and 3.21, it is obvious that cracking occurred in both walls adjacent to haunches and catwalks. The load-strain curve in Figure 3.18, captured by the strain gauge in the east wall, was also used to detect new cracks. Wall cracking was less evident in interior faces, but could still be observed along reinforcing bars, and these load-strain curves are shown in Figures 3.16 to 3.21.

LVDT SI4, which was mounted on the exterior face of the east wall detected cracking when the load was increased from 107kN/m² to 132kN/m². This is shown in Figure 3.36. The same figure also depicts the stress-strain curve obtained by the LVDT on the tension face of the west wall. It can be observed that cracking at this location occurred just prior to Load Stage III. However, LVDTs on interior faces of both walls detected cracking at 107 kN/m² (see Figure 3.37).
Overall softening of the specimen due to cracking can be observed in Figures 3.24 and 3.25, which display load-deflection curves of slabs and walls.

Even though strain gauges and LVDTs captured cracking at various locations, cracks were not visible at a loading of 121kN/m². Perhaps, the crack pattern, which was developed on the tension faces of the slabs and walls, was still covered with layers of white paint. Thus, photos were not taken at this load stage.

Average strains obtained by Zurich gauges at a reduced load of 121kN/m² are shown in Figures 3.42 and 3.43. A maximum tensile strain of 424µs was recorded at the interior face of the roof slab west of the center tension rod. High tensile and compressive strains along interior and exterior edges above three center tension rods indicate the location of flexural cracks in the roof slab.

Likewise, high strains recorded on the top surface of the invert slab above three tension rods indicated flexural cracks. High strain was also noted above the second tension rod from the east side and at the same location maximum strains were captured at the previous load stage.

Based on strains along the exterior faces of both walls, which ranged up to 236 µs, it was obvious that the cracking of the walls occurred prior to this load stage.

Since the average diagonal strains obtained from the Zurich readings were much smaller than the strains at the midspan of the slabs, cracks up to this load stage exhibited only a flexural nature.

3.2.3 LOAD STAGE III

When load increased to more than 131kN/m², the formation of new significant cracks in the slabs has not been recorded. However, further softening of slabs was mainly due to the formation of fine cracks and the opening of existing cracks.

At a load of 161kN/m², new cracking could be noted in the exterior face of the west wall at strain gauge W4B, which load-strain curve is shown in Figure 3.18. Cracking was also observed in Figure 3.19, which depicts load strain curves recorded by strain gauges near exterior faces of the walls. An initial crack in the west wall adjacent to the haunch propagated downward to the specimen soffit at 178.6kN/m².

The specimen was subjected to a maximum load of 201kN/m² before the third load stage was set.

Prior to visual inspection, the load was cycled twice to break the layer of surface paint and uncover hidden cracks. Since the soffit was cast against a smooth plywood surface, it was
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easier to locate the crack on the soffit than on the top of the specimen. A series of cracks were exhibited on the tension faces of slabs and walls (see Figures 3.57 and 3.58). They were evenly spaced and perpendicular to the neutral axis. The crack pattern observed on the soffit of the specimen is shown in Figure 3.50.

Narrow vertical cracks were exhibited on interior tension faces of both slabs and they could be seen in Figures 3.66 and 3.70. They were spaced at approximately 150mm and their widths ranged from 0.05 mm at quarter points to 0.15mm at the midspan. The spacing of cracks was increased up to 300mm at the soffit. A few soffit cracks propagated up to 400mm through the depth of the slabs. The flexural nature of the cracks was observed at the same time. Only one slightly inclined crack was noted on the soffit of the roof slab. This crack was located approximately 900mm west of the midspan.

Figures 3.74 and 3.78 show cracks in walls at approximately 175mm spacing. Cracks were 0.05 to 0.10mm wide and located on the exterior faces where the wall thickness was 340mm. Cracks were not observed in haunches or catwalks at this load stage.

Figures 3.44 and 3.45 show the Zurich average strains at 189kN/m². A maximum strain of 1034 μs was recorded along the interior face of the roof slab above the first tension rod west of the midspan. High Zurich strains at both slabs confirm visual observations regarding the extensive cracking, as well as findings from strain gauges and LVDTs. A maximum compressive strain of ~422 μs was detected at the interface of the west wall with the catwalk. A high compressive strain at this location was caused by an abrupt change of wall thickness, which resulted in stress concentration at the interior corner.

3.2.4 LOAD STAGE IV

When loading was increased from 201kN/m² to 274kN/m², the interior faces of the roof and the invert slab exhibited cracking up to approximately 300mm from the haunches and catwalks. Figures 3.5 and 3.6 display softening of the load-strain curves of reinforcing bars near the haunches. The softening was mainly due to the formation of fine cracks and opening of existing cracks. At 225kN/m² diagonal LVDTs adjacent to the haunches captured the development of the new cracks (see Figure 3.32). The cracking of the invert slab near the catwalks at the same loading was recorded by strain gauges S10B and S14B and their load-strain curves are shown in Figures 3.12 and 3.13. Since the stiffness of the slabs was reduced due to additional cracking, the moment was redistributed to the walls. Thus, the corresponding strain
increments captured near the midspan of both slabs were also reduced. This can be observed in Figures 3.2 and 3.3, which depict load-strain curves recorded by gauges N12B, N13B and N14B.

The new cracks were different in their orientation from previously developed cracks. Figure 3.51 pictures cracks in a specimen soffit at a reduced load of 251 kN/m². Newly developed cracks, which were located in the roof slab near the haunches were partially inclined and classified as flexure-shear cracks. They propagated up to 500mm towards the exterior face.

A number of fine cracks found on the interior faces of both slabs reduced the crack spacing while the maximum crack width remained 0.15mm. Photos of the roof and invert slab are shown in Figures 3.59, 3.60, 3.67 and 3.71.

New cracks were also found in the soffit of both walls. All wall cracks were flexural and they propagated up to 150mm towards the interior faces. Most of the fine cracks in the exterior faces of both walls were 0.05mm wide with one crack in each wall that was 0.1mm wide. The cracking of the walls is shown in Figures 3.75 and 3.79.

In addition, the formation of flexural-shear cracks was captured by Zurich gauges and the corresponding average concrete surface strains are shown in Figures 3.46 and 3.47. A maximum shear strain of 519 με was noted at the second grid east of the west haunch at same location where the flexural-shear crack was formed. Figure 3.38 shows the load-strain curves obtained by Zurich readings. A significant increase in shear strains was also noted at the east haunch and catwalk and can be seen in Figure 3.39.

Zurich strains along the interior edges of the roof and invert slabs were increased up to 1585 με while the maximum strain along tension faces of both walls was 1032 με.

### 3.2.5 LOAD STAGE V

As the load was increased new shear cracks were formed in the roof slab at the east and west haunch. At a load of 303kN/m² Figures 3.7, 3.9 and 3.30 depict sudden slope changes of the load-strain curves captured along a reinforcing bar in the roof slab at the west haunch. Similar observations were made at the east haunch when an additional load of 3kN/m² was applied (see Figures 3.8, 3.9 and 3.30). At the load of 339kN/m², reinforcing rebars at the west haunch started to rapidly elongate. In addition, diagonal LVDTs D2 and D3 recorded opening of shear cracks at the same location (see Figure 3.32). Shear cracks in the invert slab were captured by LVDTs D5 and D8, which load-strain curves are shown in Figure 3.33. Widening of the cracks followed by excessive elongation of the reinforcing bars in the invert slab is shown in Figures 3.14, 3.15 and 3.31. It was evident at this stage that the specimen was close to failure and the load stage V was
set at 342kN/m². To prevent further opening of the shear cracks the load was first reduced to 323kN/m². Since elongation of the rebars and the crack opening continued, the load was further reduced to 302kN/m².

One of two cracks in the roof slab adjacent to the west haunch was observed at the previous load stage. With additional loading, this crack propagated diagonally up to almost the full depth of the roof slab (see Figure 3.52). A second, 0.35mm wide crack propagated diagonally from the haunch/slab interface towards the existing flexural-shear crack. Combined together these two cracks formed an inclined critical shear plane.

A similar crack pattern was noted in the roof slab at the east haunch. A diagonal 0.3mm wide crack intersected the flexure-shear crack, creating a full depth shear plane.

Significant shear cracking was also noted in the invert slab near the east catwalk. However, the maximum crack width measured at this quadrant was only 0.1mm.

The number and width of flexural cracks increased further at this load stage. The maximum width of the cracks in the roof and invert slab at midspan was 0.2mm (see Figures 3.61 and 3.72). Additional cracking was noted in both walls while some of the wall cracks opened up to 0.2mm.

Figures 3.48 and 3.49 display concrete surface Zurich strains. Shear strain at the second grid east of the west haunch increased from 519 µs to 1407 µs. A considerable increase of shear strains was also noted near the east haunch. Maximum tensile strains at the roof and the invert slabs were 1814 µs and 1990 µs, respectively. Tensile and compressive Zurich strains also increased at both walls, ranging from -775 µs at the interior face of the west wall adjacent to the catwalk and up to 1467 µs at the exterior face of the east wall.

### 3.2.6 LOAD STAGE VI

Once the specimen was subjected to a previous maximum load of 342 kN/m², large strains were detected at all four quadrants. Excessive elongation of rebars and the widening of cracks at the west haunch were observed just prior to shear failure at 347kN/m². The failure was sudden and was followed by a dull sound. To ensure the safety of personnel and protect instrumentation, the specimen was quickly unloaded.

Failure occurred in the roof slab adjacent to the west haunch (see Figures 3.62 and 3.63). Photos of the failure section are illustrated in Figures 3.81 to 3.84. The critical shear crack described at the previous load stage opened up. The aggregate interlock strength was reduced until the additional shear could not be carried over the gap. At the same time the crack also
propagated along the exterior reinforcement up to approximately 650mm west of the midspan. At the other side of the failure plane the concrete was extensively cracked as a result of dowel action. A similar crack pattern was observed on the slab soffit. The exterior face exhibited two vertical cracks between the second and third jack set from the west side. Two wide vertical cracks were also noted on the interior face of the roof slab at the interface with the west haunch.

Maximum total deflection of slabs prior to failure was 22mm (Figure 3.24). LVDTs spanning between the walls recorded that the maximum deflection between walls was 12.8mm (Figure 3.25). Photos presented in Figures 3.68, 3.73, 3.76 and 3.80 picture cracks in the slabs and the walls.

3.2.7 LOAD STAGE VII

It appears that the specimen could still carry a considerable load. Thus, the specimen was reloaded in the post-failure stage. Only the final load was recorded through the data acquisition system and the necessary photos were obtained.

As the load was increased, the wide shear crack opened and slipped over the already established failure plane at a load of 247kN/m². Spalled concrete and exposed rebars were noted on the exterior face of the roof slab. Exterior 15M bars were bent as a result of the dowel action. Concrete was extensively cracked at the haunch/roof slab interface. Figures 3.64 and 3.85 to 3.88 show photos of the shear failure crack and the associated spalls and cracks in the roof slab.

3.2.8 ADDITIONAL REMARKS

Data collected from LVDTs and strain gauges was represented by twenty point curves and are summarized in Appendix B.

Most of the photographs indicate approximate applied loading, which was measured by a pressure transducer at various load stages. Load cells recorded more accurate applied loading than that shown on the photos.
Figure 3.2 - Roof Slab Reinforcement Strains At Midspan

Figure 3.3 - Roof Slab Reinforcement Strains Near Midspan
Figure 3.4 - Roof Slab Reinforcement Strains

Figure 3.5 - Roof Slab Reinforcement Strains Near West Haunch
Figure 3.6 - Roof Slab Reinforcement Strains Near East Haunch

Figure 3.7 - Roof Slab Reinforcement Strains at West Haunch
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Figure 3.8 - Roof Slab Reinforcement Strains at East Haunch

Figure 3.9 - Roof Slab Reinforcement Strains at Haunches
Figure 3.10 - Invert Slab Reinforcement Strains At Midspan

Figure 3.11 - Invert Slab Reinforcement Strains Near Midspan
Figure 3.12 - Invert Slab Reinforcement Strains Near West Catwalk

Figure 3.13 - Invert Slab Reinforcement Strains Near East Catwalk
Figure 3.14 - Invert Slab Reinforcement Strains at Catwalks

Figure 3.15 - Invert Slab Reinforcement Strains at Catwalks
Figure 3.16 - West Wall Reinforcement Strains at the Haunch

Figure 3.17 - East Wall Reinforcement Strains at the Haunch
Figure 3.18 - Wall Reinforcement Strains Near the Haunches

Figure 3.19 - Wall Reinforcement Strains Near the Catwalks
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Figure 3.20 - West Wall Reinforcement Strains at the Catwalk

Figure 3.21 - East Wall Reinforcement Strains at the Catwalk
Figure 3.22 - West Wall Reinforcement Strains at the Haunch

Figure 3.23 - West Wall Reinforcement Strains at the Catwalk
Figure 3.24 - Total Deflections Between Interior Opposite Faces of the Slabs

Figure 3.25 - Total Deflections Between Interior Opposite Faces of the Walls
Figure 3.26 - Roof Slab Concrete Strains at the Midspan

Figure 3.27 - Roof Slab Concrete Strains at the Midspan
Figure 3.28 - Invert Slab Concrete Strains at the Midspan

Figure 3.29 - Invert Slab Concrete Strains at the Midspan
Figure 3.30 - Roof Slab Concrete Strains at the Haunches

Figure 3.31 - Invert Slab Concrete Strains at the Catwalks
Figure 3.32 - Roof Slab Concrete Strains - Diagonals

Figure 3.33 - Invert Slab Concrete Strains - Diagonals
Figure 3.34 - Roof Slab Concrete Strains - Diagonals

Figure 3.35 - Invert Slab Concrete Strains - Diagonals
Figure 3.36 - Wall Concrete Strains - Exterior Face

Figure 3.37 - Wall Concrete Strains - Interior Face
Figure 3.38 - Roof Slab Shear Strains by Zurich Gauge

Figure 3.39 - Invert Slab Shear Strains by Zurich Gauge
strains in \( \mu \)s

Figure 3.40 - Zurich Strains Parallel and Normal to Neutral Axes at \( q = 73kN/m^2 \)

strains in \( \mu \)s

Figure 3.41 - Diagonal Zurich Strains at \( q = 73kN/m^2 \)
Figure 3.42 - Zurich Strains Parallel and Normal to Neutral Axes at $q = 132\text{kN/m}^2$

Figure 3.43 - Diagonal Zurich Strains at $q = 132\text{kN/m}^2$
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Figure 3.44 - Zurich Strains Parallel and Normal to Neutral Axes at q = 201kN/m²

Figure 3.45 - Diagonal Zurich Strains at q = 201kN/m²
TEST OBSERVATIONS AND RESULTS

Figure 3.46 - Zurich Strains Parallel and Normal to Neutral Axes at $q = 27.4kN/m^2$

Figure 3.47 - Diagonal Zurich Strains at $q = 27.4kN/m^2$
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strains in μs

Figure 3.48 - Zurich Strains Parallel and Normal to Neutral Axes at q = 342kN/m²

strains in μs

Figure 3.49 - Diagonal Zurich Strains at q = 342kN/m²
Figure 3.52 - Crack Pattern on Soffit of Specimen at \( q = 342 \text{kN/m}^2 \)
Figure 3.53 - Initial Setup - $q = 0 \text{kN/m}^2$

Figure 3.54 - Initial Setup - $q = 0 \text{kN/m}^2$
Figure 3.55 – Load Stage I - $q = 73 \text{kN/m}^2$

Figure 3.56 – Load Stage I - $q = 73 \text{kN/m}^2$
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Figure 3.57 – Load Stage III - $q = 201 \text{kN/m}^2$

Figure 3.58 – Load Stage III - $q = 201 \text{kN/m}^2$
Figure 3.59 – Load Stage IV - $q = 274 \text{kN/m}^2$

Figure 3.60 – Load Stage IV - $q = 274 \text{kN/m}^2$
Figure 3.61 - Load Stage V - \( q = 342 \text{kN/m}^2 \)

Figure 3.62 - Load Stage VI - \( q = 347 \text{kN/m}^2 \)
Figure 3.63 – Load Stage VI - $q = 347 \text{kN/m}^2$

Figure 3.64 – Load Stage VII – Post-Failure Stage
Figure 3.65 – Load Stage I - $q = 73 \text{kN/m}^2$ - Roof Slab

Figure 3.66 – Load Stage III - $q = 201 \text{kN/m}^2$ - Roof Slab
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Figure 3.67 - Load Stage IV - $q = 274 \text{kN/m}^2$ - Roof Slab

Figure 3.68 - Load Stage VI - $q = 347 \text{kN/m}^2$ - Roof Slab
Figure 3.69 - Load Stage I - \( q = 73 \text{ kN/m}^2 \) - Invert Slab

Figure 3.70 - Load Stage III - \( q = 201 \text{ kN/m}^2 \) - Invert Slab
Figure 3.71 - Load Stage IV - $q = 274 \text{ kN/m}^2$ - Invert Slab

Figure 3.72 - Load Stage V - $q = 342 \text{ kN/m}^2$ - Invert Slab
Figure 3.73 - Load Stage VI - $q = 347 \text{kN/m}^2$ - Invert Slab

Figure 3.74 - Load Stage III - $q = 201 \text{kN/m}^2$ - East Wall
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Figure 3.75 - Load Stage IV - $q = 274 \text{ kN/m}^2$ - East Wall

Figure 3.76 - Load Stage VI - Failure Load $q = 347 \text{ kN/m}^2$ - East Wall
Figure 3.77 – Load Stage I – $q = 73 \text{kN/m}^2$ - West Wall

Figure 3.78 – Load Stage III – $q = 201 \text{kN/m}^2$ - West Wall
Figure 3.79 - Load Stage IV - $q = 274 \text{kN/m}^2$ - West Wall

Figure 3.80 - Load Stage VI - Failure Load $q = 347 \text{kN/m}^2$ - West Wall
Figure 3.81 - East End of Roof Slab - Top Surface - Failure Load $q = 347 \text{kN/m}^2$

Figure 3.82 - East End of Roof Slab - Bottom Surface - Failure Load $q = 347 \text{kN/m}^2$
Figure 3.83 – East End of Roof Slab – Interior Face – Failure Load $q = 347 \text{kN/m}^2$

Figure 3.84 – East End of Roof Slab – Exterior Face – Failure Load $q = 347 \text{kN/m}^2$
Figure 3.85 - *East End of Roof Slab* - Top Surface - Post-Failure Load

Figure 3.86 - *East End of Roof Slab* - Bottom Surface - Post-Failure Load
Figure 3.87 - East End of Roof Slab - Exterior Face - Post-Failure Load

Figure 3.88 - East End of Roof Slab - Interior Face - Post-Failure Load
4. Discussion of Test Results

This chapter describes computer modeling by a non-linear frame analysis program TEMPEST. The shear strength of the specimen is calculated using different North American Codes. The test results are also examined. Lastly, test and computer results as well as the code estimates are compared and discussed.

4.1 COMPUTER MODELING BY TEMPEST

A non-linear frame analysis program, developed at the University of Toronto by Prof. Frank Vecchio was used to find the flexural capacity of the specimen, the member forces, the displacements and the particular strains. The program utilizes an iterative procedure since there is no direct way to find the forces and displacements when members consist of materials that exhibit non-linear behavior. The stiffness of each member is calculated by bisection-analysis and the stiffness method is applied at each iterative step. The load is applied in increments and the program created the result file at each load step.

4.1.1 MODEL

Figure 4.1 illustrates the computer model used to run the program. The second order effects, the softening and cracking of concrete, and the yielding of steel bars were accounted for while the shear effects were ignored. Each member was modeled using eleven to sixteen concrete layers and one to four reinforcing layers. For example, member 1 located in the roof slab left of the midspan consisted of thirteen 50mm thick concrete layers, and top and bottom steel layers which represented 15M and 20M reinforcement (see Figure 4.1). Occasionally, two reinforcing
DISCUSSION OF TEST RESULTS

Figure 4.1 - Computer Model of Specimen Used to Run Program TEMPEST

Figure 4.2 - Moment Diagram and Deflected Shape at $q=344\text{kN/m}^2$
layers were modeled at the same member depth to properly represent the splicing of different reinforcing bars.

Since the specimen and the applied loading were symmetrical, it was only necessary to model one side of the specimen. The other side was substituted with proper supports at the midspan, which allowed vertical deflections while rotations and horizontal displacements were restrained.

An additional support was introduced at the bottom left corner, which restrained infinite displacement of the model in the y direction. The left half of the specimen was modeled with 33 members defined between 34 joints.

The uniform loading was applied on the roof and the invert slab members. Concentric forces, which replaced uniform loading left of the wall centerline, created moments when they were shifted from their original location to the corner joints.

The three input files are printed in Appendix C. The first file defines the range of load steps and modes in which the TEMPEST program was executed. The second file describes the structural components of the model, including geometry and material properties. The last file defines the loading pattern being applied to the model. The TEMPEST program could be executed only if all three files were input properly.

4.1.2 RESULTS

Figure 4.2 depicts a moment diagram and the deformed shape of the left side of the specimen, subjected to a uniform loading of 344kN/m². From the load-deflection curve presented in Figure 4.3, it can be observed that flexural failure of the specimen was predicted to occur at 635 kN/m² and that this flexural failure would be associated with very large deflections (over 200mm). Yielding of reinforcement was noted first at the midspan of the roof slab at 490 kN/m². The invert slab reinforcing bars yielded at 510 kN/m² while the yielding of the exterior wall reinforcing bars occurred after reaching a load of 541 kN/m². Figure 4.4 displays load-strain curves along reinforcing bars at the midspan of both slabs and at the exterior layer of the wall. In addition, cracking occurred in the roof and in the invert slab at 62.5 kN/m² and 72.9 kN/m², respectively, while signs of wall cracking were noted at 83.3 kN/m².
Discussion of Test Results

Figure 4.3 - Total Deflection of both slabs

Figure 4.4 - Reinforcement Strains at Midspan of Slabs and at Wall


4.2 CODE PREDICTIONS

The shear strength of the roof and invert slab was predicted by different North American Codes. The distribution of member forces was obtained from the TEMPEST results. Once the moments at member joints, modeled to run TEMPEST are known, it is easy to find moments anywhere on the slabs or the walls. However, moments would be obtained even more easily if locations of inflection points were known and the roof and invert slabs of the box structure were treated as simply supported beams with supports at inflection points. The maximum moment at midspan \( M_{\text{TEMPEST}} \), found by the TEMPEST program, was \( pL^2/8 \), where \( L \) was the distance between the inflection points (see Figure 4.5). Therefore, the distance from the centerline of the specimen to the inflection point \( L/2 \) would be:

\[
\frac{L}{2} = \sqrt{\frac{2M_{\text{TEMPEST}}}{p}}
\]

Figure 4.6 shows how the location of inflection points moved at the first cracking of the concrete and near yielding of the reinforcement. In the range from 290kN/m\(^2\) to 450kN/m\(^2\), where shear failure is expected to occur, the inflection points stayed essentially fixed. Thus, for the purpose of finding the shear failure load, which was assumed to be in the same range, it was reasonable to assume that the inflection points in the roof slab and the invert slab were located 2184mm and 2086mm, respectively, from the midspan of each slab. Bearing this in mind, moments in the roof and invert slab could be expressed in the following terms:

\[
V = xp \\
M = p\left(\frac{L^2}{8} - \frac{x^2}{2}\right) \\
\frac{V}{M} = \frac{x}{\frac{L^2}{8} - \frac{x^2}{2}}
\]

where: \( p \) is the uniform loading per unit width

\( V \) and \( M \) are the shear forces and moments at distance \( x \) from the midspan while

\( L \) was the distance between inflection points.
Figure 4.5 - Moment Diagram and Inflection Points

Figure 4.6 - Location of Inflection Points
4.2.1 ACI-95 CODE

Roof Slab-Clause 11.3.1.1

According to clause 11.2.3.1 the critical shear section was located at distance $d$ from the support. In this case, critical sections would be located at distance $d$ from the interface of the haunch and the roof slab (see Figure 4.7).

Using expression 11-4 of ACI Code

$$V_c = 0.167 \sqrt{f'_c b_w d} = 0.167 \sqrt{45 \times 960 \times 615} = 661kN$$

since critical section was 1515mm away from midspan

$$V_c = 1515 \times p = 661kN$$

Thus

$$\begin{align*}
V_c &= 661kN \\
p &= 437kN / m \\
q &= \frac{p}{b} = \frac{437}{0.960} = 455kN / m^2
\end{align*}$$

Invert slab-Clause 11.3.1.1

Critical sections were located at a distance 595mm inward of catwalks or 1465mm on each side of the midspan. Therefore:

$$\begin{align*}
V_c &= 0.167 \sqrt{f'_c b_w d} = 0.167 \sqrt{45 \times 960 \times 595} = 640kN \\
p &= \frac{640}{1.465} = 437kN / m \\
q &= \frac{437}{0.960} = 455kN / m^2
\end{align*}$$

By Clause 11.3.1.1, ACI-95 Code predicts that both the roof and the invert slab would fail in shear when the load was $q=455kN/m$. 

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Figure 4.7 - Critical Sections by ACI Code
Roof Slab-Clause 11.3.2.1

The shear-moment ratio, expressed in term of the distance to the centerline, was previously derived such that:

\[
\frac{V}{M} = \frac{x}{L^2 - \frac{x^2}{2}} = \frac{1515}{(2184 \times 2)^2 - \frac{1515^2}{8}} = \frac{1}{817}
\]

By the clause 11.3.2.1, the expression for shear strength was slightly modified while the critical sections remained at the same locations.

\[
V_c = \left(0.16\sqrt{f_c} + 17 \frac{V_u}{M_u}\right) b_w d = \left(0.16\sqrt{45} + 17 \frac{3375}{960 \times 615} \cdot \frac{615}{817}\right) 960 \times 615 = 677kN
\]

but must be less than

\[
0.29\sqrt{f_c} b_w d = 0.29\sqrt{45} \times 960 \times 615 = 1149kN
\]

Thus

\[
\begin{align*}
V_c &= 677kN \\
p &= \frac{677}{1.515} = 447kN/m \\
q &= \frac{447}{0.960} = 465kN/m^2
\end{align*}
\]

Invert Slab-Clause 11.3.2.1

\[
\frac{V}{M} = \frac{x}{L^2 - \frac{x^2}{2}} = \frac{1465}{(2086 \times 2)^2 - \frac{1465^2}{8}} = \frac{1}{753}
\]

\[
V_c = \left(0.16\sqrt{f_c} + 17 \frac{V_u}{M_u}\right) b_w d = \left(0.16\sqrt{45} + 17 \frac{3852}{960 \times 595} \cdot \frac{595}{753}\right) 960 \times 595 = 665kN
\]

which is less than

\[
0.29\sqrt{f_c} b_w d = 0.29\sqrt{45} \times 960 \times 595 = 1111kN
\]
Therefore

\[
\begin{align*}
V_c &= 665kN \\
p &= \frac{665}{1.465} = 454kN/m \\
qu &= \frac{454}{0.960} = 473kN/m^2
\end{align*}
\]

Thus, this expression estimated the shear failure of the roof slab at \(q = 465kN/m^2\).

### 4.2.2 AASHTO CODE

**Roof Slab-Clause 5.14.5.3**

Following the AASHTO expression, similar to ACI Clause 11.3.2.1, the estimated shear strength of the roof slab was

\[
V_c = \left(0.178\sqrt{f_c} + 32 \frac{A_s}{bd} \frac{Vd}{M}\right)bd = \left(0.178\sqrt{45} + 32 \frac{3375}{960 \times 615} \frac{615}{817}\right)960 \times 615 = 786kN
\]

It had to be less than

\[
0.332 \sqrt{f_c} b_w d = 0.332 \sqrt{45} \times 960 \times 615 = 1315kN
\]

but not less than

\[
0.25 \sqrt{f_c} b_w d = 0.25 \sqrt{45} \times 960 \times 615 = 990kN
\]

Thus

\[
\begin{align*}
V &= 990kN \\
p &= \frac{990}{1.515} = 653kN/m \\
qu &= \frac{653}{0.960} = 681kN/m^2
\end{align*}
\]

**Invert Slab-Clause 5.14.5.3**

The shear force-moment ratio at the critical section was determined earlier as

\[
\frac{V}{M} = \frac{1}{753}
\]
Therefore the shear strength was

\[ V_c = \left( 0.178 \sqrt{f_c'} + 32 \frac{A}{bd} \right) bd = \left( 0.178 \sqrt{45} + 32 \frac{3852}{960 \times 595} \frac{595}{753} \right) 960 \times 595 = 779 kN \]

and \( V_c \) must be at least

\[ 0.25 \sqrt{f_c'} b d = 0.25 \sqrt{45} \times 960 \times 595 = 958 kN \]

Thus:

\[
\begin{align*}
V_c &= 958 kN \\
p &= \frac{958}{1.465} = 654 kN / m \\
q &= \frac{654}{0.96} = 681 kN / m^2
\end{align*}
\]

AASHTO Code-Clause 5.14.5.3 predicted that the specimen would fail in flexure since the shear failure load is higher than the flexural failure load obtained by TEMPEST.

4.2.3 CSA-94

Roof Slab-Clause 11.3.5.2

Expression had to be modified to eliminate safety factors. such that shear strength of roof slab is

\[ V_c = \frac{0.167}{0.20} \left( \frac{260}{1000 + d} \right) \sqrt{f_c} b_w d = \frac{0.167}{0.20} \left( \frac{260}{1000 + 615} \right) \times \sqrt{45} \times 960 \times 615 = 532 kN \]

Where: \( d = 61.5 \text{mm} \)
\( b_w = 960 \text{mm} \)
\( f_c = 45 \text{Mpa} \)

The shear strength of the roof slab would not be less than

\[ \frac{0.167}{0.20} 0.10 \sqrt{f_c} b_w d = \frac{0.167}{0.20} 0.10 \sqrt{45} \times 960 \times 615 = 331 kN \]
Therefore

\[ V_c = 532\, kN \]
\[ p = \frac{532}{1.515} = 351\, kN/m \]
\[ q = \frac{351}{0.96} = 366\, kN/m^2 \]

**Invert Slab-Clause 11.3.5.2**

\[ V_c = \frac{0.167}{0.20} \left( \frac{260}{1000 + d} \right) \sqrt{f_c' b_w d} = \frac{0.167}{0.20} \left( \frac{260}{1000 + 595} \right) \times \sqrt{45} \times 960 \times 595 = 522\, kN \]

but must be greater than

\[ \frac{0.167}{0.20} \times 0.10 \sqrt{f_c' b_w d} = \frac{0.167}{0.20} \times 0.10 \sqrt{45} \times 960 \times 595 = 320\, kN \]

Thus

\[ V_c = 522\, kN \]
\[ p = \frac{522}{1.465} = 356\, kN/m \]
\[ q = \frac{356}{0.96} = 371\, kN/m^2 \]

By Clause 11.3.5.2 of the CSA Code, specimen was expected to fail in shear of the roof slab at 366kN/m².

**4.2.4 GENERAL METHOD (CSA-94, ASSHTO-94, OHBDC-91)**

**Roof slab**

The effective shear depth could be calculated either as 0.9d or as a distance between compression and tension resultant forces due to flexure. From Figure 4.8

\[ d_e = 593\, mm \]
Figure 4.8 - Critical Sections by General Method
Thus, the critical section was located 593 mm away from the haunch or 1537 mm from the midspan and the corresponding shear and moment could be calculated from previously determined equations.

\[ V = xp = 1.537p \]

\[ M = p \left( \frac{L^2}{8} - \frac{x^2}{2} \right) = p \left( \frac{(2.184 \times 2)^2}{8} - \frac{1.537^2}{2} \right) = 1.204p \]

\[ \frac{M}{V} = \frac{1.204p}{1.537p} = 0.783 \]

The crack spacing parameter \( s_z \) was to be taken as \( d_c \), or the maximum spacing between the crack control reinforcement, whichever is smaller. Therefore, \( s_z \) was taken as 552 mm, which was the distance between the top and bottom layer of reinforcement (see Figure 4.8).

Equations for shear capacity and longitudinal strains could be simplified to

\[ V_c = \beta \sqrt{f_c b_w d_v} = \beta \sqrt{45 \times 960 \times 593} = 3819 \beta \]

\[ \varepsilon_v = \frac{0.5(N_f + V_f \cot \theta) + M_f / d_v - A_p f_{p_l}}{E_v A_v + E_p A_p} \]

where values of \( \varepsilon_v \) and \( \beta \) are to be determined from Figure 11-2 or Table 11-2 in the CSA-94 Code or equivalent Figures and Tables in the AASHTO-94 and OHBDC-91 Codes.

If longitudinal strain was assumed to be \( \varepsilon_v = 1.3 \times 10^{-3} \Rightarrow \beta = 0.134 \) and \( \theta = 50.7^\circ \)

Then:

\[ V_c = 3819 \times 0.133 = 512 kN \]

\[ M = 0.783 \times 512 = 401 kNm \]

\[ \varepsilon_v = \frac{0.5V_f \cot \theta + M_f / d_v}{E_v A_v} = \frac{0.5 \times 512 \times 10^{-3} \times \cot 50.7^\circ + 401 \times 10^6 / 593}{200 \times 10^3 \times 3325 \times 10^{-6}} = 1.33 \times 10^{-3} \]

Since assumed longitudinal strain was less than the calculated strain, an additional step was required.

For \( \varepsilon_v = 1.32 \times 10^{-3} \Rightarrow \beta = 0.133 \) and \( \theta = 50.8^\circ \)
DISCUSSION OF TEST RESULTS

Calculated longitudinal strain was the same as the assumed strain, therefore

\[
\varepsilon_t = \frac{0.5V_t \cot \theta + M_t}{E_t A_t} = \frac{0.5 \times 508 \times 10^3 \times \cot 50.7^\circ + 398 \times 10^6}{200 \times 10^3 \times 3325 \times 10^{-6}} = 1.32 \times 10^{-3}
\]

Three additional sections at distances 0.2L, 0.4L and 0.6L from the centerline were checked. Also, a section where four 20M bars developed full yield stress, was analyzed. Results are summarized in Table 4.1 and illustrated in Figure 4.9. From this Figure it can be noted that the shear failure would occur at distance \(d_i\) from the haunch at 344kN/m². Table 4.1 also shows the maximum shear resistance if all fourteen 20M bars would be terminated at the ends of the roof slab.

Invert Slab

The shear capacity of the invert slab was determined in a similar fashion. From figure 4.8 it could be seen that

\(d_i = 570\text{mm}\)
\(s_i = 535\text{mm}\)

Therefore, the critical section was located 570mm away from the catwalk or 1490mm away from the midspan.

The shear and moment at this location were:

\[V = xp = 1.490p\]
\[M = p \left( \frac{L^2}{8} - \frac{x^2}{2} \right) = p \left( \frac{(2.086\times2)^3}{8} - \frac{1.49^2}{2} \right) = 1.066p\]

\[\frac{M}{V} = \frac{1.066p}{1.490p} = 0.715m\]

The expression for shear capacity was simplified to
**Discussion of Test Results**

<table>
<thead>
<tr>
<th>location</th>
<th>0.1L</th>
<th>0.2L</th>
<th>(L) from cut off</th>
<th>0.3L</th>
<th>(d_v) from haunch</th>
<th><strong>(d_v) from haunch</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>(x[mm])</td>
<td>437</td>
<td>874</td>
<td>1152</td>
<td>1311</td>
<td>1537</td>
<td>1543</td>
</tr>
<tr>
<td>(A_d[mm^2])</td>
<td>4200</td>
<td>4200</td>
<td>4200</td>
<td>3839</td>
<td>3325</td>
<td>4200</td>
</tr>
<tr>
<td>(d_x[mm])</td>
<td>587</td>
<td>587</td>
<td>587</td>
<td>589</td>
<td>593</td>
<td>587</td>
</tr>
<tr>
<td>(s_y[mm])</td>
<td>552</td>
<td>552</td>
<td>552</td>
<td>552</td>
<td>552</td>
<td>552</td>
</tr>
<tr>
<td>(V_p/m)</td>
<td>0.437</td>
<td>0.874</td>
<td>1.152</td>
<td>1.311</td>
<td>1.537</td>
<td>1.543</td>
</tr>
<tr>
<td>(M_p/m^2)</td>
<td>2.289</td>
<td>2.003</td>
<td>1.721</td>
<td>1.526</td>
<td>1.204</td>
<td>1.195</td>
</tr>
<tr>
<td>(M_V/m)</td>
<td>5.239</td>
<td>2.292</td>
<td>1.494</td>
<td>1.164</td>
<td>0.783</td>
<td>0.774</td>
</tr>
<tr>
<td>(e_y[10^{-3}])</td>
<td>&gt;2.0</td>
<td>2.005</td>
<td>1.59</td>
<td>1.47</td>
<td>1.32</td>
<td>1.13</td>
</tr>
<tr>
<td>(\beta)</td>
<td>0.105</td>
<td>0.120</td>
<td>0.125</td>
<td>0.133</td>
<td>0.143</td>
<td>0.143</td>
</tr>
<tr>
<td>(\theta(\degree))</td>
<td>55°</td>
<td>55.0</td>
<td>52.5</td>
<td>51.7</td>
<td>50.8</td>
<td>49.6</td>
</tr>
<tr>
<td>(V[kN])</td>
<td>215</td>
<td>397</td>
<td>455</td>
<td>475</td>
<td>508</td>
<td>539</td>
</tr>
<tr>
<td>(M[kN/m])</td>
<td>1125</td>
<td>909</td>
<td>680</td>
<td>553</td>
<td>398</td>
<td>418</td>
</tr>
<tr>
<td>(q[kN/m^2])</td>
<td>512</td>
<td>473</td>
<td>411</td>
<td>377</td>
<td>344</td>
<td>364</td>
</tr>
</tbody>
</table>

- - yielding of longitudinal reinforcement
** - case when all 14M20 bars are terminated at the ends of the slab
\(x\) - distance measured from the midspan of the slab to the analyzed section
\(P\) - linear uniform loading (\(q \times 0.96\))
\(\beta\) and \(\theta\) are obtained from Table 11-2 in CSA-94 Code

**Table 4.1 - General Method - Load Capacities at Different Sections in the Roof Slab**

![Figure 4.9 - Predicted Shear Strength and Critical Section at the Roof Slab](image)
\[ V_c = \beta \sqrt{f_c b_w \sigma_v} = \beta \sqrt{45 \times 960 \times 570} = 3671\beta \]

For \( \varepsilon_c = 1.2 \times 10^{-3} \Rightarrow \beta = 0.140 \) and \( \theta = 49.8^\circ \)

\[ V_c = 3671 \times 0.140 = 514kN \]
\[ M = 0.715 \times 514 = 367kNm \]
\[ \varepsilon_v = \frac{0.5V_f \cot \theta + M_v / d_v}{E_A} = \frac{0.5 \times 514 \times 10^3 \times \cot 49.8^\circ + 367 \times 10^6 / 570}{200 \times 10^3 \times 3795 \times 10^{-6}} = 1.13 \times 10^{-3} \]

Since the calculated longitudinal strain was smaller than the assumed one, further calculations were not required, but one more step was executed to obtain a better estimate.

For \( \varepsilon_c = 1.16 \times 10^{-3} \Rightarrow \beta = 0.141 \) and \( \theta = 49.5^\circ \)

\[ V_c = 3671 \times 0.142 = 521kN \]
\[ M = 0.715 \times 521 = 373kNm \]
\[ \varepsilon_v = \frac{0.5V_f \cot \theta + M_v / d_v}{E_A} = \frac{0.5 \times 521 \times 10^3 \times \cot 49.5^\circ + 373 \times 10^6 / 570}{200 \times 10^3 \times 3795 \times 10^{-6}} = 1.16 \times 10^{-3} \]

Therefore

\[
\begin{array}{|c|}
\hline
V_c = 521kN \\
M = 373kNm \\
p = \frac{521}{1.490} = 350kN/m \\
q = \frac{350}{0.960} = 364kN/m^2 \\
\hline
\end{array}
\]

The shear-moment capacities were obtained at five locations and the results were summarized in Table 4.2. Figure 4.10 illustrates that the critical section was at a distance \( d_i \) from the catwalk. The table also contains the results of the invert slab with all bars terminating over the walls.

According to General Method specimen would fail in the roof slab in shear near the haunch at a load of 344kN/m².
**DISCUSSION OF TEST RESULTS**

<table>
<thead>
<tr>
<th>location</th>
<th>0.1L</th>
<th>0.2L</th>
<th>0.3L</th>
<th>( l_p ) from cut off</th>
<th>( d_e ) from catwalk</th>
<th>&quot;( d_e ) from catwalk&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_{[\text{mm}] } )</td>
<td>417</td>
<td>834</td>
<td>1251</td>
<td>1312</td>
<td>1490</td>
<td>1493</td>
</tr>
<tr>
<td>( A_{[\text{mm}^2]} )</td>
<td>4200</td>
<td>4200</td>
<td>4200</td>
<td>4200</td>
<td>3795</td>
<td>4200</td>
</tr>
<tr>
<td>( d_{[\text{mm}] } )</td>
<td>567</td>
<td>567</td>
<td>567</td>
<td>567</td>
<td>570</td>
<td>557</td>
</tr>
<tr>
<td>( s_{[\text{mm}]} )</td>
<td>535</td>
<td>535</td>
<td>535</td>
<td>535</td>
<td>535</td>
<td>535</td>
</tr>
<tr>
<td>( V_p [\text{m}] )</td>
<td>0.417</td>
<td>0.834</td>
<td>1.251</td>
<td>1.312</td>
<td>1.49</td>
<td>1.493</td>
</tr>
<tr>
<td>( M_p [\text{m}^3] )</td>
<td>2.089</td>
<td>1.828</td>
<td>1.393</td>
<td>1.315</td>
<td>1.068</td>
<td>1.061</td>
</tr>
<tr>
<td>( W_p [\text{m}] )</td>
<td>5.009</td>
<td>2.192</td>
<td>1.114</td>
<td>1.002</td>
<td>0.715</td>
<td>0.711</td>
</tr>
<tr>
<td>( c_{[0-10]} )</td>
<td>&gt;2</td>
<td>1.97</td>
<td>1.36</td>
<td>1.29</td>
<td>1.16</td>
<td>1.08</td>
</tr>
<tr>
<td>( \beta )</td>
<td>*</td>
<td>0.107</td>
<td>0.132</td>
<td>0.136</td>
<td>0.142</td>
<td>0.146</td>
</tr>
<tr>
<td>( \alpha (\text{°}) )</td>
<td>55°</td>
<td>54.5</td>
<td>50.8</td>
<td>50.3</td>
<td>49.5</td>
<td>49.1</td>
</tr>
<tr>
<td>( V_p [\text{kN}] )</td>
<td>217</td>
<td>391</td>
<td>482</td>
<td>495</td>
<td>521</td>
<td>535</td>
</tr>
<tr>
<td>( M_p [\text{kJ} \cdot \text{m}] )</td>
<td>1086</td>
<td>858</td>
<td>537</td>
<td>496</td>
<td>373</td>
<td>380</td>
</tr>
<tr>
<td>( q_p [\text{kN} / \text{m}^2] )</td>
<td>541</td>
<td>489</td>
<td>401</td>
<td>393</td>
<td>364</td>
<td>373</td>
</tr>
</tbody>
</table>

- **-** yielding of longitudinal reinforcement
- **"** case when all 14M20 bars are terminated at the ends of the slab
- \( x \) - distance measured from the midspan of the slab to the analyzed section
- \( p \) - linear uniform loading \( (= q x 0.96) \)
- \( \beta \) and \( \alpha \) values are obtained from Table 11.2 in CSA-94 Code

**Table 4.2 - General Method - Load Capacities at Different Sections in the Invert Slab**

![Diagram](image)

Figure 4.10 - *Predicted Shear Strength and Critical Section at the Invert Slab*
4.3 DISCUSSION OF TEST RESULTS

Through careful observation of the test results presented in chapter 3, it can be seen that the specimen progressed through 3 distinct stages. The first stage ranged up to the loading of 107kN/m² in which the specimen exhibited an elastic nature. Localized cracks were noted only at holes in the slabs. Reinforcing strains captured at the midspan of both slabs are shown in Figures 3.2 and 3.10. It can be seen that cracking occurred first at the mid-height of the specimen. Figure 4.11 illustrates the stress concentration in the disturbed region around the roof slab tension hole at midspan. An increase of strains due to this stress concentration was captured by the strain gauge N13M. Thus, localized vertical cracks were developed at the holes. Since these cracks did not influence the overall behavior of the specimen, their influence was neglected in further discussion.

![Graph showing strains along reinforcing bar](image)

Figure 4.11 – *Increase of strain due to stress concentration around tension-rod hole*

At a load of 107kN/m², cracking becomes evident. From graphs presented in the previous chapter it can be seen that cracking was more pronounced in slabs than in the walls. The stiffness of the cracked slabs and walls was changed at the same time such that the ratio of slab and wall stiffness was reduced. Thus, the moment was redistributed to the walls and the inflection points were shifted towards the centerline. For instance, Figure 3.7 pictures strains observed at strain gauges near west inflection point of the roof slab. Strain gauges along the exterior and interior face indicate no cracking at this location. Therefore, the variation of slopes
DISCUSSION OF TEST RESULTS

and sharp points at the load-strain curves were mainly due to the moment redistribution between slabs and walls which caused the shifting of the inflection point.

Shear cracking and failure characterized the final stage. First, shear cracks were detected by diagonal LVDTs at 225kN/m². Shear strains obtained at critical sections of the slabs are shown in Figure 4.12. Based on load-strain curves, the same graph indicates that both sides of the roof slab were close to failure near the haunches. In terms of shear strains, Zurich readings were consistent with values obtained by LVDTs (see Figure 4.13). Considering that LVDTs were mounted on the specimens bottom surface while Zurich readings were obtained from the top surface, the specimen behaved uniformly throughout its height.

The observed failure was typical of a shear failure of large beams. After formation of a critical shear crack, aggregate interlocking contributed significantly to the total shear strength. As the shear crack widened the aggregate interlock strength was reduced until slippage occurred over the crack. It is evident that longitudinal strains and corresponding crack widths depend directly on the reinforcing ratio. Since the reinforcing ratio was only 0.56%, it definitely contributed to an early shear failure.

Based on the figures presented in the previous chapter, it can be concluded that only a small amount of the load was required for a specimen to fail in shear from the time when significant shear cracks were first noted. LVDTs, which recorded total deflections between slabs and walls did not indicate that the specimen was near failure. Flexural cracking of the slabs and walls caused much more softening of the specimen than did the shear cracking just prior to failure. Figures 4.14 to 4.17 illustrate strains along reinforcing bars in the roof and invert slab. None of the principal reinforcing bars in the slabs reached their yield strains. The specimen was carefully inspected just prior to failure. Only one 0.35mm wide diagonal crack was found on the bottom surface of the roof slab at the west haunch. This crack could be seen because a slice of the box unit was being tested. Interior faces, which would be accessible for inspection in the case of full-scale structures, did not exhibit any evidence of the shear cracks at the critical section. Therefore, the full size box structure would not exhibit any signs of structural distress prior to sudden failure.
DISCUSSION OF TEST RESULTS

**Figure 4.12 - Shear Strain by LVDTs**

**Figure 4.13 - Shear Strain by Zürichs and LVDTs**
DISCUSSION OF TEST RESULTS

Figure 4.14 - Top Slab Reinforcement Strains

Figure 4.15 - Top Slab Reinforcement Strains
DISCUSSION OF TEST RESULTS

Figure 4.16 - Invert Slab Reinforcement Strains

Figure 4.17 - Invert Slab Reinforcement Strains
4.4 COMPARISON OF TEST RESULTS AND CODE PREDICTIONS

The shear strengths of the slabs were estimated by different codes, and the corresponding capacities are compared in the following section.

Since the TEMPEST program was executed with the shear mode switched off, it was found that the specimen would fail in flexure at a load of 635 kN/m² by yielding of the top and bottom slabs. The computer model estimated a stiffer response than that displayed by the specimen. TEMPEST adequately simulated a softening of the model due to the cracking of the concrete. The load at which cracking appeared was almost the same as the cracking load that was recorded during testing. However, cracking of the walls and slabs was more pronounced.

The traditional ACI Code expression estimated that the invert and the roof slab would fail in shear at the same time. Thus, shear failure was to occur at a distance d either from the haunches or the catwalks. The second expression, which is described in Clause 11.3.2.1 predicted that shear failure was to occur in the roof slab at a distance d from the haunches. The actual failure load of the specimen was more than 30% lower than ultimate capacity calculated using both expressions.

Unusually high shear strengths of both slabs were estimated by the AASHTO Culvert Design Method, which is described in clause 5.14.5.3. Thus, the specimen was expected to fail in flexure rather than in shear. According to the AASHTO Culvert Design Method shear reinforcement would not increase the load capacity of the specimen.

The Simplified Method in the CSA Code estimated lower values of shear strength than previous codes, mainly because this method accounts for size effects. The predicted shear capacity was 5% higher than the actual capacity of the specimen. Also, the location of shear failure was correctly predicted by the Simplified Method of the CSA Code.

Only the General Method predicted that the specimen would fail at a load less than 350 kN/m². Moreover, the estimated shear capacity of the roof slab was only 1% less than the actual capacity of the specimen. In addition, the General Method indicated that the invert slab had a slightly higher shear capacity, which was confirmed by test observations that the invert slab was also close to failure. The General Method proved to be sensitive not only with regard to the depth of the slabs but also with regard to the reinforcing ratio, which at the critical section was only 0.56%. Table 4.3 and Figure 4.18 summarize the predictions according to different North American Codes.
In addition, it was found that minor alterations of the detailing could influence the shear strength of the slabs. If the 4M20 bars were extended to the ends of the slabs, the reinforcing ratio would increase as well as the shear strength of the slabs and the corresponding load capacity. Thus, according to the General Method, this minor alteration would increase the shear strength of the roof slab by 5.8% and the invert slab by 2.5%. The amount of the additional cost would be negligible in terms of this shear strength enhancement. However, this alteration would still not eliminate the abrupt and brittle nature of the shear failure of the large flexural members with no shear reinforcement.

<table>
<thead>
<tr>
<th>Method</th>
<th>Estimated Shear Capacity [kN/m²]</th>
<th>Location of critical section</th>
<th>Ratio of estimated failure load to actual failure load</th>
<th>Ratio of estimated failure load to calculated flexural failure load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural</td>
<td>635</td>
<td>at midspan of both slabs</td>
<td>183%</td>
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Table 4.3 – Summary of Predicted Load Capacities of Specimen
Figure 4.18 - *Code Predictions and Observed Shear Failure Loads*
5. Conclusions and Recommendations

5.1 CONCLUSIONS

After the testing of the specimen, it was more than obvious that the shear strength of the roof and the invert slabs were heavily dependent on their size. More particularly, the maximum shear stress at the critical section at failure was only 0.85MPa, which was significantly lower than the shear strengths of tested slabs and beams whose depth ranged up to 300mm. Figure 5.1 summarizes the test results of several series of large beams with different reinforcing ratios and

Figure 5.1 – Influence of Member Size on Shear Strength (adopted from CSA-94 Code)
CONCLUSIONS AND RECOMMENDATIONS

aggregate sizes. The shear strength of the box structure is also illustrated in the same figure. The figure shows that the shear stress at failure is significantly reduced when the depth of beams is increased. The test of the box structure confirms this pattern. Therefore, size effects are significant in slabs with no shear reinforcement.

The answer to the question as to why the specimen had a shear strength less than Kani’s beams of similar size lies in the fact that the roof slab had only a 0.56% reinforcing ratio and a 10mm maximum aggregate size. Low reinforcing ratios contribute to the increase of longitudinal strains and the corresponding crack widths. Further, the widening of the cracks combined with the reduction of the aggregate size reduced the aggregate interlocking strength, which mainly contributed to the total shear strength of the roof slab.

The capacity of the specimen was predicted by several North American Codes. Clauses 11.3.1.1 and 11.3.2.1 of the ACI-95 Code were proven to be unconservative when large beams with low reinforcing ratios were considered. The ASSHTO-94 Culvert Method was found to be very unconservative. The CSA-94 Simplified Method accounts for the depth variations of the flexural members, but predicted slightly higher shear capacity than the actual capacity of specimen. The General Method, which is included in the AASHTO-94, CSA-94 and OHBDC-91 Codes, adequately addressed the size effects and the related variations of the reinforcing ratio.

The failure of the specimen was sudden with no visible warning. Shear cracks were found on the top and bottom surfaces of the specimen. However, these sides are accessible for inspection only in laboratory specimens where a slice of the modeled structure is tested. Inspection of the real box units is limited to observations of the interior faces of the slabs and the walls. These faces exhibited only flexural cracks at the midspan of the roof and the invert slab. Therefore, the original TTC box unit would not exhibit significant signs of distress which may indicate that the structure was close to shear failure through the slabs.

Based on the testing of the model TTC box unit it is obvious that the shear design of slabs may not be adequate unless shear reinforcement is introduced. Since the model was scaled down to 45%, analyzed and tested, the full-scale box structure would have a shear strength that was even a lower proportion of its flexural capacity.

The specimen failed at the west haunch. However, this does not mean that the critical section is always located in slabs at a distance d from the haunch or catwalk when the box structure is subjected to uniform loading. Most tests have been performed on beams with a constant reinforcing ratio along their entire length. Thus the beams would usually fail at 0.2L
from the support where significant shear forces and moments were encountered. In the case of a box structure where the reinforcing ratio is not constant, the location of the critical section depends also on the reinforcement layout. Thus, the critical section is at a location where both the shear forces and the longitudinal strains are significant, considering that longitudinal strains depend not only on member forces but also on the reinforcement ratios.

Running in an off-shear mode, the non-linear frame analysis program TEMPEST predicted reasonably well the flexural behavior of the specimen.

5.2 RECOMMENDATIONS

Even though thousands of beams have been tested in shear, very little research has been done on large beams and slabs with reinforcing ratios of less than 1%. To better understand how sensitive these large flexural members are, in terms of shear strength, to aggregate size and low reinforcing ratios, further research is strongly recommended.

The unconservative nature of the shear strength expressions in the ACI-95 and the AASHTO-94 Codes should be corrected by the introduction of better expressions which would be sensitive not only to the compressive strength of concrete and the moment to shear ratio but also to the depth of flexural members, the reinforcing ratio and the maximum aggregate size.

Even though provisions requiring minimum shear reinforcement were developed to account for unexpected member forces, they mainly eliminated the potential failures of large beams due to the detrimental size effects on shear strength. However these provisions exclude slabs and footings. Considering the graph in Figure 5.1, it can be concluded that the ACI Code is adequate when the thickness of the beams and the slabs is less than 300mm. Therefore, to eliminate shear failures of slabs classified as large beams, the exclusion in these provisions should be limited only to the slabs whose depth does not exceed 300mm. This limitation should be introduced until better expressions for shear strength are available.

In the case of continuous beams or box structures where the reinforcing ratio is not constant throughout the length, the critical shear section could be located not only at a distance $d$ from the supports where the maximum shear forces are expected, but also at any location where the reinforcing ratio is significantly reduced. Thus, using the General Method, the shear capacity should be checked near the supports as well as at the locations of cut off bars. Additionally, shear capacity should be checked systematically throughout the entire length of the members in order to
ensure that failure will not occur at locations where both shear forces and moments are significant.

Since the specimen failed at 55% of its flexural capacity, minimum shear reinforcement would enhance the shear strength of the roof and the invert slabs. Also additional shear reinforcement would enable this type of structure to fail in a less brittle fashion, exhibiting signs of structural distress prior to failure.

In the absence of shear reinforcement, it is recommended to at least extend all bars to the ends of the slabs. This would increase the reinforcing ratio and the corresponding shear strength near the haunches and the catwalks. However, this minor alteration would not eliminate the brittle nature of the shear failure when structural members with no ties or stirrups are considered.
References


7. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary ACI 318R-95," American Concrete Institute, Detroit, 1995, 369pp.


Appendix A

Loading Assembly – Shop Drawings
GE75SX Trust Bearing

3a

90° at each level

D=1/5" dia.
p=UNC 8/4

NOTE: All other corners filleted.
Appendix B

Strain Gauge and LVDTs Data
|        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |        |
Appendix C

TEMPEST Input Files
TEMPEST III
***********

JOB DATA FILE
**************

NOTES: Notes that relate to the input of job data can be found at the ---- end of this file.

Job File Name (8 char. max.) : UNITH
Date (30 char. max.) : May 29, 95

STRUCTURE DATA
----------
File Name (8 char. max.) : UNITH

LOADING DATA
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Data File Series Name : UNITH
No. of Load Stages : 75
Starting Load Stage No. : 1

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Maximum No. of Iterations : 150
Averaging Factor ( 0 to 1 ) : 0.500000

Initial Axial Stiffness Factor : 1.000000
Initial Flexural Stiffness Factor : 1.000000

Section Analysis Mode (1 - 6) : 1
Shear Analysis Mode (0 - 4) : 0
Tension Stiffening (0 - 1) : 1
Strain Compatibility (0 - 1) : 1
Geometric Nonlinearity (0 - 1) : 1
Reinf Strain Hardening (0 - 1) : 0
Temperature Softening (0 - 1) : 0
Concrete Creep Effects (0 - 1) : N/A
Concrete Shrinkage Effects (0 - 1) : N/A
Load History (0 - 1) : N/A

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NOTES:

1. IN CHOOSING THE SECTION ANALYSIS MODE, THE FOLLOWING OPTIONS ARE AVAILABLE:

   1. Nonlinear Section Analysis - Deformation Controlled
   2. Nonlinear Section Analysis - Force Controlled
   3. Effective Stiffness - Branson's Formula
   4. Cracked/Uncracked - ACI 349
   5. Uncracked (Ig)
   6. Fully Cracked (Icr)

2. IN CHOOSING THE METHOD OF SHEAR ANALYSIS, THE FOLLOWING OPTIONS ARE AVAILABLE:

   0. Shear Not Considered
   1. Multi-Layer Analysis - Uniform Shear Flow
   2. Multi-Layer Analysis - Uniform Shear Strain
   3. Single Layer Analysis ( @ mid-depth)
   4. Modified Single Layer Analysis

3. WHEN SELECTING ANALYSIS PARAMETERS WHICH ASK FOR A 0 OR 1, THE FOLLOWING RULE APPLIES:

   0 - Neglect Effect
   1 - Consider Effect

4. WHEN SELECTING OUTPUT DATA, THIS WILL INCLUDE RESULTS SUCH AS CONCRETE STRAINS AND STRESSES, SHEAR REINFORCEMENT STRAINS AND STRESSES, AND MEMBER SECTION STRAINS AND STRESSES. THE INPUT PARAMETERS FOR THIS OPTION ARE AS FOLLOWS:

   Member : The member number of the desired member for which the strains and stresses will be shown.
   Node   : The end of the member for which the stresses and strains will be shown.
   Avg    : 0 if average values for member section stresses/strains are not required.
            1 if average values for member section stresses/strains are required.
   Crk    : 0 if values for member section stresses/strains are not required at a crack location.
            1 if values for member section stresses/strains are required at a crack location.

5. THIS PROGRAM USES METRIC UNITS ONLY.
### TEMPEST INPUT FILES

---

**STRUCTURE DATA FILE**

---

Note: Notes that relate to the input of structure data can be found at the end of this file.

---

#### GENERAL PARAMETERS

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- **Structure Title**: (30 char. max.) : UNITH
- **Structure File Name**: (8 char. max.) : UNITH
- **No. of Members**: (50 max.) : 32
- **No. of Member Types**: (97 max.) : 24
- **No. of Joints**: (45 max.) : 33
- **No. of Support Joints**: : 3
- **No. of Support Restraints**: : 5

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#### STRUCTURE PARAMETERS

---

**(A) Joint Coordinates**

---

<NOTE:> Coordinate units in mm

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3 2058 2855 /
4 1787 2855 /
5 1546 2855 /
6 1290 2855 /
7 1004 2855 /
8 680 2855 /
9 440 2855 /
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19 0 1208 /
20 0 1004 /
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22 0 315 /
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TEMPEST INPUT FILES

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<td>1</td>
<td>35.00</td>
<td>3000.0</td>
<td>492.0</td>
<td>492.0</td>
<td>0.000</td>
</tr>
<tr>
<td>23</td>
<td>2</td>
<td>35.00</td>
<td>1200.0</td>
<td>404.0</td>
<td>404.0</td>
<td>0.000</td>
</tr>
<tr>
<td>23</td>
<td>3</td>
<td>570.00</td>
<td>3000.0</td>
<td>135.0</td>
<td>135.0</td>
<td>0.000</td>
</tr>
<tr>
<td>23</td>
<td>4</td>
<td>570.00</td>
<td>1200.0</td>
<td>492.0</td>
<td>492.0</td>
<td>0.000</td>
</tr>
<tr>
<td>24</td>
<td>1</td>
<td>35.00</td>
<td>4200.0</td>
<td>492.0</td>
<td>492.0</td>
<td>0.000</td>
</tr>
<tr>
<td>24</td>
<td>2</td>
<td>570.00</td>
<td>1200.0</td>
<td>492.0</td>
<td>492.0</td>
<td>0.000</td>
</tr>
</tbody>
</table>

\/

NOTES:  

C - 7
1. DO NOT INSERT OR DELETE ANY LINE. EXCEPTION: INSERTION OF LINES IN THE SPACE PROVIDED FOR OUTPUT OF DATA. IN THIS CASE, LEAVE LINE WITH SLASH AFTER LAST DATA LINE.

2. PROGRAM IS DIMENSIONED FOR: 45 JOINTS, 50 MEMBERS, 10 MEMBER TYPES, 35 LOADS PER TYPE, 18 CONCRETE LAYERS, AND 10 REBAR POSITIONS.

3. IN REFERENCE TO THE INPUT OF THE SUPPORT RESTRAINT LIST:

   0 = Not Restrained
   1 = Restrained

4. THIS PROGRAM USES METRIC UNITS ONLY.

5. DIFFERENT MEMBERS, MEMBER TYPES, AND JOINTS SHALL BE NUMBERED AS 1, 2, 3, ..., ETC.

6. STEEL LAYERS SHALL BE NUMBERED AS 1, 2, 3, ..., ETC., FROM THE TOP THE MEMBER SECTION.

7. TABS, BLANKS, OR COMMAS CAN BE USED TO SEPARATE DATA. DO NOT ENCLOSE ANY DATA IN QUOTATION MARKS.
TEMPEST III
*********

LOAD DATA FILE
=============

Notes: Notes that relate to the input of load data may be found at the
----- end of this file.

GENERAL LOAD CONDITIONS
************************

Load Case ID (15 char. max.) : UNITH
Load Case Data File (8 char. max.) : UNITH
Load Factored (0 - 1) : 1
Time Factored (0 - 1) : 0

No. of Loaded Joints : 2
No. of Members w/ End Action Loads : 3
No. of Members w/ Concentrated Loads : 0
No. of Members w/ Distributed Loads : 20
No. of Members w/ Temperature Loads : 0

Reference Temperature (deg. C) : 26.0

LOAD PARAMETERS
***************

Joint Loads
------------

<NOTE:> UNITS: kN, kN-m
<<<<<< FORMAT >>>>>
NODE Fx Fy Mz [ #nodes d(node) d(Fx) d(Fy) d(Mz) ]<-- (2)/
11 0.0 -1.7 0.1445 /
23 0.0 1.7 -0.1445 /
/

Member End Actions
-------------------

<NOTE:> UNITS: kN, kN-m
<<<<<< FORMAT >>>>>
M AF1 SF1 BM1 AF2 SF2 BM2 [ #M d(M) ]<-- (2)/
/

Concentrated Loads
--------------------

<NOTE:> UNITS: kN, kN-m, m
<<<<<< FORMAT >>>>>
M Fx Fy Mz x/L [ #M d(M) d(Fx) d(Fy) d(Mz) ]<-- (2)/
/

Uniformly Distributed Loads
-----------------------------

<NOTE:> UNITS: kN/m, m
<<<<< FORMAT >>>>>
M W a/L b/L [ #M d(M) d(W) ]<-- (2)/
1 -10 0 1 10 1 0 /
23 10 0 1 10 1 0 /

Temperature Loads
------------------

<NOTE:>
UNITS:  Deg.C, hrs
<<<<<< FORMAT >>>>>
M T1' T2' T1 T2 TIME [ #M d(M) ]<-- (2)/

NOTES:
-----

1. DO NOT INSERT OR DELETE ANY LINE. EXCEPTION: INSERTION OF LINES IN THE SPACE PROVIDED FOR THE INPUT OF THE DATA. IN THIS CASE, LEAVE LINE WITH A SLASH AFTER THE LAST DATA LINE.

2. THIS PROGRAM USES METRIC UNITS ONLY.

3. MOMENTS ARE COUNTERCLOCKWISE POSITIVE.

4. WHEN INPUTTING DATA RELATING TO TEMPERATURE LOADING, THE FOLLOWING VARIABLES ARE REQUIRED:

   T1' = Initial Top Temperature
   T2' = Initial Bottom Temperature
   T1  = Final Top Temperature
   T2  = Final Bottom Temperature

   Note: All temperatures are differentials with respect to the reference temperature.

5. TABS, BLANKS, OR COMMAS CAN BE USED TO SEPARATE DATA. DO NOT ENCLOSE ANY DATA IN QUOTATION MARKS.

6. INFORMATION ABOUT LOAD/TIME FACTORED OPTIONS ARE AS FOLLOWS:

   Load Factored = 0 --> Do not multiply loads by load factors.
   Load Factored = 1 --> Multiply loads by load factors.
   Time Factored = 0 --> Do not multiply loads by time factors.
   Time Factored = 1 --> Multiply loads by load factors.