Size Effect and the Influence of Longitudinal Reinforcement on the Shear Response of Large Reinforced Concrete Members

By

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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

To investigate the influence of the longitudinal reinforcement ratio as well as the presence of minimum transverse reinforcement on the shear properties of very large reinforced concrete members, a total of four 2-meter deep spans with varying amount of longitudinal and transverse reinforcement were constructed and tested.

Consistency of the Modified Compression Field Theory in predicting shear strength of the tested members is observed throughout this research project.

A simplified relationship relating the longitudinal reinforcement ratio with the overall member shear strength is then proposed and compared with the experimental observations. Good correlation between the predictions and test observations is observed.
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1. Introduction:

1.1 Background:

The advances of modern construction technology enable large, complex structures to be built. From the deep footings of super-high skyscrapers to the thick roofs of deep tunnels, many publicly important large structures rely on deep slab-like components to resist large forces. While very important, these components often contain no shear reinforcement or very little shear reinforcement. Because of this they are susceptible to the so-called size effect in shear which refers to the fact that for such members the shear stress required to cause failure decreases as the size of the member increases.

The ACI 318-99 shear provisions, which were formulated some 35 years ago and have remained essentially unchanged since then, ignore this important effect. According to this code, no shear reinforcement is necessary if the factored shear force \( V_u \) is lower than the shear strength provided by the concrete \( \phi V_c \) for slabs and footings and for beams with total depths not greater than half the width of the web, for more narrow beams minimum shear reinforcement must be provided if the factored shear force is more than one-half of the shear strength provided by the concrete.

Unfortunately, the expression for the shear strength \( V_c \), because it neglects the size effect, overestimates the strength of large members. In some recent tests of large beams the ACI shear provisions overestimated the shear capacity by factors of about 2.
This situation raises very serious public safety concerns, as has already been pointed out in a paper published in the ACI structural journal entitled “How Safe are our Large, Lightly Reinforced Concrete Beams, Slabs, and Footings?”

To examine the above-related situations closely and to provide further evidence for the physical existence of the size effect and the extent of its influence, a series of large-sized structural members have been tested in the structural laboratory of the University of Toronto.

Specimens with depths of one to two meters that represent a large but still practical structural member size were carefully detailed, constructed and tested. The parameters examined, besides the member size, were the longitudinal reinforcement ratio, shear reinforcement ratio, longitudinal reinforcement distribution over the depth of the member, shear reinforcement spacing and the concrete strength. Many test results were then obtained, which as a whole, provide a good support for the veracity of the underlining analytical model — namely, the MCFT as well as its predicted size effect. The largest such tests were those conducted recently by Yoichi Yoshida as part of his Master of Applied Science thesis. These so-called YB beams were 2-meter deep by 12-meter long specimens constructed and tested to represent a vertical slice (300mm wide) cut from a deep foundation footing. The reinforcing details are such that they can reflect the conventional engineering practice in reinforcing such deep concrete footings. The experimental results were compared in Yoshida’s thesis to the predictions from both of the ACI shear provisions and the CSA shear provisions as well as those from the non-linear program approach (Response 2000) that is based on the MCFT. The result had show that the ACI shear provisions, neglecting the size effect, in some situations, may result in a very large overestimation of the actual shear strength of such members.

To direct investigations more closely into how the reinforcing details may influence the size effect and when the ACI shear formulation is most vulnerable to it, two such members (SB series) were designed to facilitate a quantified study on this aspect. SB2012 and SB2003, bear a geometric similarity to the beams of the YB
series, but contained different amount of longitudinal reinforcement from a low of 0.36% to a high of 1.52%.

Below are the several illustrative cases where concerns for the size effect shall be exerted.

(1). The one-way slabs near some of the stations of the new extensions of the Toronto subway, shown in Fig. 1-1, are 1.4m thick and contain only 0.5 percent of flexural reinforcement. Traditionally, such members are constructed with no stirrups, and are sized by using the ACI shear strength equation.

![Figure 1-1. Typical Single-Cell Box Structure for Toronto Subway (Adapted from Collins and Kuchma)](image-url)
(2). Structural transfer girders with depths limited by story height considerations, sometimes have a width exceeding 2 times their depth, and hence are exempted from the need to provide minimum shear reinforcement if the shear exceeds $0.5\phi V_c$ according to the latest ACI shear design provisions.
1.2 Research Objectives:

As introduced in the background section, the specific purposes for this research are as follows:

1. Compare the experimental results as obtained from this and previous tests with the extant ACI 99, AASHTO LRFD Bridge Design Specifications and CSA-A23.3. Provide further evidence to verify the validity of the MCFT in its explanation of the size effect.

2. Propose revised simple design equation as an extension to the extant CSA formulation accounting for the influence of longitudinal reinforcement in the shear design of beams and slabs without or with less than the minimum ACI required shear reinforcement.
2. Review of Theoretical Background

2.1. Models:

Two analytical models, which try to explain the observed size effect in shear are briefly reviewed here. They are:

1. Fracture mechanics model (Single crack/crack band)
2. Smeared model (Smeared material and smeared cracks)
   - For instance, the MCFT

2.1.1 Fracture Mechanics Type Model

The fracture mechanics approach, although it can provide a reasonable prediction of brittle shear failure, is largely limited to the phenomena where failure is due to the propagation of a single critical diagonal crack with the remaining body of the beam staying the elastic state. The presence of reinforcement, and its yielding, aside from providing an additional ductile mechanism, which carries part of the load, has the effect of spreading the fracture process zone. Therefore, reinforced concrete is generally less susceptible to fracture effects and the accompanying size effects of fracture mechanics type. This limits the applicability of the fracture mechanics model in its explanation of the size effect in reinforced concrete members. Examples are the deep beams reinforced with middle layer skin reinforcements, where the size effect will not be governed by the size of the member, rather, it is strongly influenced by the vertical spacing between these longitudinally distributed skin reinforcements, as already been shown in many tests - i.e. in reference 9.

2.1.2. Smeared Model --- MCFT

There are two kinds of finite element modeling for cracked reinforced concrete: the microscopic models and the macroscopic models (ASCE/ACI, 1993). Microscopic models are based on the stress-strain relations of plain concrete and steel, focusing on their interactions through bond slipping along the reinforcing bars and through shear-sliding along cracked surfaces.
“The microscopic modeling was found to be unsuitable for large structures, because the strong and complicated interaction between concrete and steel can be reproduced neither by the simple superposition of the material laws of plain concrete and steel, nor by the introduction of bond and shear interface elements.” To overcome this difficulty, the macroscopic models were developed. This kind of modeling (smeared crack conception) is based on the average stress-strain relationships of concrete and steel where bond slipping and shear sliding are implicitly included.

The constitutive laws of concrete and steel were then obtained directly from tests of reinforced concrete panels subject to biaxial loading.

The Modified Compression Field Theory (MCFT), based on this conception, was developed by Vecchio and Collins about 20 years ago at the University of Toronto. By employing equations of equilibrium, compatibility and experimentally developed constitutive relations (figure 2-1), the theory is implemented into a non-linear sectional analysis program: Response 2000 and many non-linear finite element analysis programs: Tril98, 99, Vector2. Its ability in providing consistent reliable prediction in both the load-displacement history and the ultimate load capacity have long been observed. Simplified methods based on this model were incorporated into many design codes, they are Ontario Highway Bridge Design Code (OHBDC), the Canadian Standards Association Concrete Design Code (CSA-A23.3-94), and the AASHTO LRFD specifications.
**Equilibrium:**

- Average Stresses:
  \[ \rho_x f_{xav} = f_x + r \cot \theta - f_1 \]
  \[ \rho_y f_{yav} = f_y + r \tan \theta - f_1 \]
  \[ f_2 = r \left( \tan \theta + \cot \theta \right) - f_2 \]

- Stresses at Cracks:
  \[ \rho_x f_{xcor} = f_x + r \cot \theta + r_{cd} \cot \theta \]
  \[ \rho_y f_{ycor} = f_y + r \tan \theta - r_{cd} \tan \theta \]

**Geometric Conditions:**

- Average Strains:
  \[ \varepsilon_x = \frac{(e_1 \tan \theta + e_3)}{(1 + \tan \theta)} \]
  \[ \varepsilon_y = \frac{(e_1 + e_2 \tan \theta)}{(1 + \tan \theta)} \]
  \[ \gamma_{xy} = 2 \left( \frac{(e_2 - e_3)}{\tan \theta} \right) \]
  \[ \tan \theta = \frac{(e_2 - e_3)}{(e_1 - e_2)} \]

- Crack Widths:
  \[ w = \frac{f_x}{E_I} \epsilon_1 \quad \text{where} \]
  \[ \epsilon_{\theta} = \frac{1}{r} \left( \frac{\sin \theta}{\frac{r_x}{r_y}} \frac{\cos \theta}{\frac{r_y}{r_x}} \right) \]

**Allowable Shear Stress on Crack**

\[ v_{cd} \leq \frac{0.18 \sqrt{f'_2}}{0.31 + \frac{24 w}{\pi + 16}} \]

---

**Figure 2-1** A Summary of the Relationships Used in the Modified Compression Field Theory

(Adopted from Collins and Bentz)
For lightly reinforced members the MCFT model predicts shear failure when the average tensile stress in the cracked concrete can not be transferred across the cracks by the shear interlock at the crack surfaces. Since the MCFT is a fully rotating crack model, the principle compressive stress is supposed to be coincide with the crack direction, resulting in a zero average shear stress. Hence, the average principal tensile stress transmission mechanism is directly related to the local steel stress and the local shear stress in the concrete at the location of cracks. The maximum capacity for the locally increased shear stress at the crack location depends on the aggregate interlock mechanism and hence is a function of the width of the crack and the size of the aggregate, Equation 2-1.

\[

\nu_{ci} = \frac{2.16 \sqrt{f'_c}}{0.3 + \frac{24w}{a + 0.63}} \quad \text{psi and inch units \ --- \ --- \ --- equ. 2-1}
\]

\[

\nu_{ci} = \frac{0.18 \sqrt{f'_c}}{0.3 + \frac{24w}{a + 16}} \quad \text{MPa and mm units}
\]

Where

\[

w = \varepsilon_1 s m \theta \quad s_m \theta = \frac{1}{\sin \theta + \cos \theta}
\]

\[

s_m \theta = \frac{1}{s_m x + s_m y}
\]

The MCFT checks the magnitude of these local stresses, if they exceed the limiting value given above, a reduction is then made to the magnitude of the post-cracking average tensile stresses that can be sustained, equation 2-2.

\[

f_1 = \nu_{ci} \tan \theta + \frac{A_v}{s b_w} (f_w - f_v) \quad \text{--- equ.2-2}
\]
The MCFT simplifying assumption that the direction of principle compressive stresses in the concrete coincides with the direction of principle compressive strains means that the MCFT doesn’t make allowance for shear slip along the cracks. The effect of this slip has been accounted for in a new model developed by Vecchio called the Disturbed Stress Field Model. This model is an extension to the MCFT, and treats the relations between the concrete tensile stress and the local shear stresses at the crack in a more complicated but more accurate manner. It links the average tensile stress transmission to an average reinforcement stress increment. The locally increased steel stresses will contribute to the local shear stress at the location of crack in the concrete. This introduces a slip deformation which, upon superimposing onto the originally calculated strains, results in an angle deviation between the principal strain and stress directions. See equation 2-3, 2-4 and figure 2-2.

\[
\delta_s^a = \frac{V_{ei}}{1.8w^{-0.8} + (0.234w^{-0.707} - 0.20) \times f_{cc}} \quad \text{equ. 2-3}
\]

\[
\gamma_s^a = \frac{\delta_s^a}{s} \quad \text{equ. 2-4}
\]

Where,

s - Crack spacing

The contribution of this slip deformation to the overall principal tensile strain was separated from the original principal tensile strain, making it possible to reduce the Cs factor in the compression strength reduction factor for concrete from 1.00 to 0.55, see equation 2-5 and Figure 2-3.

\[
\beta_d = \frac{1}{1 + C_s C_d} \leq 1.0 \quad \text{equ. 2-5}
\]

Where, \( C_d = 0.27(\frac{\varepsilon_{c1}}{\varepsilon_{c2}} - 0.37) \)

\( \beta_d \): Compression strength reduction factor for concrete
Figure 2-2 Illustration of the Slip Deformation

(Adapted from Vecchio, F.J. Reference 5.)
Under extreme cases, such as unbalanced reinforcing scheme in the X and Y directions, this revised version of MCFT is believed to be able to yield closer predictions to the test results. (Vecchio, Disturbed stress field model for reinforced concrete: Formulation)

The above-related revisions to the MCFT have been implemented into Vector2, a nonlinear finite element program which will be used together with program Response2000 in the next few chapters of this thesis.

2.2. Analytical Programs:

2.2.1. Program Response 2000

Response 2000 is a non-linear sectional analysis program, developed by Evan Bentz as a part of his Ph.D. research project under the supervision of Professor M. P. Collins. Its friendly interface and graphic in-time-feed-back greatly simplify the use of this program. Hence, a consistent result is easily achieved among different users.
Upon executing a "member response", the program first carries out a series of sectional analyses with different combinations among shear and moments to decide on the shear vs. moment interaction envelope. Then depending on the geometrical configurations of the beam and the corresponding loading conditions, it can give predictions as to the crack pattern, load-displacement history, shear strain distribution, curvature distribution along the span and deflection along the span. The following figures (figure 2-4. Figure 2-5) give a typical overview of how the program may look like during its execution.

**Figure 2-4 Input information feed backs in Response 2000**
Figure 2-5 A typical output of the response 2000

The program, although a 2D program based on MCFT, is also capable of performing calculations on circular, T-shapes and a varieties of other non-uniform section shapes, this feature considerably widens its applicability into many other areas.

The program also provides different levels of user-program interaction, making it easier to control. The material properties, such as stress-strain relations, can be easily monitored upon input. The output, likewise, shown graphically, can be easily understood, hence making the spotting of the final failure mechanism a quite straightforward task.

2.2.2. Program Trix98, 99 and Vector2.

2.2.2.1 Brief Introduction:

Program Trix98, 99 and Vector2, which have been written by Professor F.J. Vecchio, are non-linear finite element programs.

The non-linear response of the concrete and steel makes the stiffness matrix a function that depends, amongst other things, on the loading level. Hence a total incremental method is utilized. The total incremental method, when used in predicting the response of cracked reinforced concrete, is generally found to be superior to the incremental method.

The number of desired loading stages is defined in the job file, where most of the controlling information is entered. The magnitude of load stage increment can be entered in the load file, which has an extension file name of "l2d" (for Trix, it is "ldr"). For each of the loading steps, the program will do as many iterations as needed to ensure some certain convergence criteria is achieved. The convergence criteria, listed also in the job file, can be easily adjusted to fit to the particular purpose.

The secant module method is used in generating the element stiffness matrixes. The idea is roughly illustrated in figure 2-6.
The final stage is reached when the global stiffness matrix is found to be not positive definite any more. The apparent reason is that so many elements have been so degraded (the determinant approaches zero) that the number of independent relations in the whole system is never larger than the degree of freedoms introduced by the assembly of nodals. A single definite solution is hence impossible.

2.2.2.2 Constitutive Model

Modified compression field theory and the above-mentioned stress disturbed field theory are incorporated into the stress-strain field determinations at the end of each iterations. After the nodal displacements are obtained, the MCFT will then be used to define the related strain, stress fields in order that the next iteration/loading stage can be continuously carried out.

Lists of optional models that are available in Vector2 for choosing are attached in appendix A.
2.2.2.3. Available Element Types:

Bi-linear rectangular elements and constant strain triangles are the current available element types.
3. Review of Relevant Code Provisions and Past Work at the University of Toronto

The previous chapter dealt mainly with a general review of the model and analytical programs based on it. The following few sections will review the relevant code provisions in the CSA-A23.3 and ACI 318-99 as well as the relevant previous work done at the University of Toronto.

3.1 Code Provision Review
3.1.1 ACI 318-99

The ACI shear provision attributes the shear resistance of a reinforced concrete member to the concrete \( V_{ct} \) as well as transverse reinforcement \( V_s \).

The concrete contribution, generated from large amount of laboratory size tests, is the flexure-shear cracking load of a reinforced concrete member. The 1999 ACI Code gives the following expression for \( V_c \):

\[
V_c = (1.9 \sqrt{f_c'} + 2500 \rho_w \frac{V \times d}{M})b_w d \leq 3.5 \sqrt{f_c'} b_w d \quad \text{psi units}
\]

or

\[
V_c = (0.16 \sqrt{f_c'} + 17 \rho_w \frac{V \times d}{M})b_w d \leq 0.29 \sqrt{f_c'} b_w d \quad \text{MPa units}
\]

The code also permits a simple formulation depending only on \( f_c' \):

\[
V_c = 2 \sqrt{f_c'} b_w d \quad \text{psi units}
\]

or

\[
V_c = 0.167 \sqrt{f_c'} b_w d \quad \text{MPa units}
\]
The contribution from the transverse reinforcement, on the other hand, is estimated according to Mörsch’s 45 fixed angle truss model.

\[ V_c = \frac{A_y f_y d}{s} \]

ACI 318-99 requires a minimum amount of shear reinforcement if the shear force exceeds 0.5\(V_c\) except for those situations mentioned in the introduction section of this thesis. This minimum amount of needed shear reinforcement is:

\[ A_v = 50 \frac{b_w s}{f_y} \quad \text{psi units} \]

\[ A_v = 0.33 \frac{b_w s}{f_y} \quad \text{MPa units} \]

When shear reinforcement is provided, the spacing has to be less than 0.5d or 24 in. (610 mm) whichever is the smaller.

In the absence of any transverse reinforcement, the shear will be sustained exclusively by the concrete contribution.

3.1.2 CSA-A23.3-94

The modified compression filed theory can be implemented in different ways with varying levels of complexity, from a full, nonlinear finite-element analysis (Trix, Vector), to a multi-layer sectional analysis that accounts for the variation of crack width over the section (Response 2000), to the simplest case where only the crack width at the level of the flexural tension reinforcement is considered. The last case, which is most suitable for hand design, is the General Shear Design Method or Beta Method, which was adapted by CSA-A23.3 in its shear design provisions.
In the general shear design method, shear strength comes from both the concrete and the transverse shear reinforcement. The expression is as follows:

\[ V = V_c + V_s = \beta \sqrt{f_c b w d_v} + \frac{A_y f_y d_v \cot \theta}{s} \quad [\text{mm and MPa}] \]

The \( \beta \), a function of crack width, nominal shear stress, crack orientation and longitudinal strains, considers the shear force transfer mechanism, equilibrium conditions while treats cracked concrete as a new material which has its distinctive properties in a biaxial stress fields. Hence, the \( \beta \) value is a comprehensive parameter, capable of predicting the shear contribution from the concrete in a more accurate manner.

Since different parameters comes into play in members with transverse reinforcement and those without, separate tables and graphs for \( \theta \) and \( \beta \) are prepared for the design as listed in Table 3-1 and 3-2 and Figure 3-1, 3-2.
Table 3-1 Values of $\theta$ and $\beta$, for Members with at Least Minimum Web Reinforcement.

<table>
<thead>
<tr>
<th>$\nu/f'c$</th>
<th>$\leq 0.000$</th>
<th>$0.00025$</th>
<th>$0.0005$</th>
<th>$0.00075$</th>
<th>$0.0010$</th>
<th>$0.0015$</th>
<th>$0.0020$</th>
<th>$0.0025$</th>
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<td>$\leq 0.050$</td>
<td>$\beta$</td>
<td>0.405</td>
<td>0.290</td>
<td>0.208</td>
<td>0.197</td>
<td>0.185</td>
<td>0.162</td>
<td>0.143</td>
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<tr>
<td>$\theta$</td>
<td>27.0°</td>
<td>28.5°</td>
<td>29.0°</td>
<td>33.0°</td>
<td>36.0°</td>
<td>41.0°</td>
<td>43.0°</td>
<td>45.0°</td>
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<tr>
<td>$\leq 0.075$</td>
<td>$\beta$</td>
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<td>0.250</td>
<td>0.205</td>
<td>0.194</td>
<td>0.179</td>
<td>0.158</td>
<td>0.137</td>
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<tr>
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<td>27.5°</td>
<td>30.0°</td>
<td>33.5°</td>
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<td>40.0°</td>
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Figure 3-1 Values of $\theta$ and $\beta$, for Members with at Least Minimum Web Reinforcement.
Table 3-2 Values of $\theta$ and $\beta$, for Members without Web Reinforcement

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</thead>
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<td>$\leq 125$</td>
<td>$\beta$</td>
<td>0.406</td>
<td>0.309</td>
<td>0.263</td>
<td>0.214</td>
<td>0.183</td>
</tr>
<tr>
<td></td>
<td>$\theta$</td>
<td>27°</td>
<td>29°</td>
<td>32°</td>
<td>34°</td>
<td>36°</td>
</tr>
<tr>
<td>$\leq 250$</td>
<td>$\beta$</td>
<td>0.384</td>
<td>0.283</td>
<td>0.235</td>
<td>0.183</td>
<td>0.156</td>
</tr>
<tr>
<td></td>
<td>$\theta$</td>
<td>30°</td>
<td>34°</td>
<td>37°</td>
<td>41°</td>
<td>43°</td>
</tr>
<tr>
<td>$\leq 500$</td>
<td>$\beta$</td>
<td>0.359</td>
<td>0.248</td>
<td>0.201</td>
<td>0.153</td>
<td>0.127</td>
</tr>
<tr>
<td></td>
<td>$\theta$</td>
<td>34°</td>
<td>39°</td>
<td>43°</td>
<td>48°</td>
<td>51°</td>
</tr>
<tr>
<td>$\leq 1000$</td>
<td>$\beta$</td>
<td>0.335</td>
<td>0.212</td>
<td>0.163</td>
<td>0.118</td>
<td>0.095</td>
</tr>
<tr>
<td></td>
<td>$\theta$</td>
<td>37°</td>
<td>45°</td>
<td>51°</td>
<td>56°</td>
<td>60°</td>
</tr>
<tr>
<td>$\leq 2000$</td>
<td>$\beta$</td>
<td>0.306</td>
<td>0.171</td>
<td>0.126</td>
<td>0.084</td>
<td>0.064</td>
</tr>
<tr>
<td></td>
<td>$\theta$</td>
<td>41°</td>
<td>53°</td>
<td>59°</td>
<td>66°</td>
<td>69°</td>
</tr>
</tbody>
</table>

Figure 3-2 Values of $\theta$ and $\beta$, for Members without Web Reinforcement
Where $s_z$ is defined in Figure 3-3 and is taken as 0.9d for members that have only concentrated reinforcement near the flexural tension face, or as the maximum distance between the layers of longitudinal reinforcement if the member contains intermediate layers of crack control reinforcement.

Figure 3-3. Values of Crack Spacing Parameter $s_z$. 

(i) Member with stirrups

(ii) Member without stirrups and with concentrated longitudinal reinforcement

(iii) Member without stirrups but with well distributed longitudinal reinforcement
For aggregate size other than 25mm, the following equation can be used for calculating the equivalent crack spacing as:

\[ s_{se} = s_e \left( \frac{35}{a + 16} \right) \quad [\text{mm}] \]

Another fact that is worth noting is that the most straightforward size effect is observed for members without shear reinforcement which contain only small amounts of longitudinal reinforcement, where the shear resistance is directly dependent on the shear transfer capacity of the cracks. The larger the specimen, the bigger the crack spacing, the wider the crack opening and hence the aggregate interlock mechanism at the cracks becomes weaker. This directly reduces the member’s shear resistance. In all the other situations where members are heavily reinforced, the size effect is somewhat mitigated. All these facts are also neatly treated in the \( \beta \)-method, as can be clearly seen from the descriptions in the coming chapters.

3.2 Limitation in Using the Sectional Analysis Approach

To investigate general elasto-plastic problems, a stage-wise solution technique must be followed. That is to say stress and strain solutions are formed by following the whole detailed loading history of the structural member. However, if we are only interested in obtaining a maximum load capacity of the structure under consideration, the so-called upper and lower bound limit theorems can be utilized.

Non-linear finite element methods, in searching for the final solutions, follow the whole load history of the structural member and hence use a stage-wise solution procedure. The sectional approach and the strut and tie model, by assuming the final failure mechanism explicitly, are typically based on limit theorems.
The upper bound theorem, by employing the virtual work principal on all the permissible inner velocity fields present in the structure, states that the structure will collapse if there is any compatible pattern of plastic deformation for which external work exceeds internal strain energy dissipation. Lower bound theorem, on the other hand, by examining the equilibrium condition, states that if an equilibrium distribution of stress can be found which balances the applied load, the structure will not collapse.

By utilizing both of the upper and lower bound theorems in judging the maximum loading capacity of the related structure, the correct solution is found to lie where the two theorems give the same prediction.

Similarly to the solution of a elasto-plastic problem, the sectional analysis and the strut and tie model, both by presuming a failure mechanism — either of a sectional type where \( V_{\text{applied}} = V_{\text{concrete}} + V_{\text{reinforcement}} \) or a limit-state balanced truss formed by cracked compression concrete struts and reinforcement tension ties, are of the lower bound approach where an equilibrium condition is first assumed and its limiting value taken as the solution.

Hence, as what lower limit bound theorem postulates, whichever of the two methods can provide a higher prediction of the load carrying capacity for the calculated member, that answer will be deemed to be favorable as it goes closer to the correct answer.

The Beta method adapted in the CSA shear provisions and Program Response 2000—a non-linear sectional analysis program, are both based on lower bound limit theorem, hence must observe the span to depth ratio limit as what Figure 3-4 illustrated. That is the sectional models cease to be accurate when the applied load is closer than about 2.5 times the beam depth to the support.
3.3 Review of the Relevant Past Work in the University of Toronto

3.3.1 Past Experimental Work

In order to verify the predictions from the MCFT about how various factors can affect the shear capacity in large, lightly reinforced flexural members, a series of experiments were conducted at the University of Toronto. These experimental studies were summarized by Collins and Kuchma. Experiments discussed included 22 specimens tested with a central point load on a simple span.

Among these 22 beams were 8 beams that are particularly relevant to the current study. The properties of these 8 beams have been summarized in Table 3-3 along with the properties of the 4 sections tested by Yoshida. As can be seen these beams contained from 0.74% to 1.01% of longitudinal reinforcement and were tested with a shear span to depth ratio of about 2.9. The depth of the beams ranged from 125mm to 2000mm. Seven of the beams did not contain transverse reinforcement.
while five contained about the minimum amount of stirrups specified by ACI. The properties of these members are listed in Table 3-3.

3.3.2 Conclusions from Past Experimental Work

Conclusions relevant to this research project as deduced from the above series of tests are summarized as follows:

1. Tests of lightly reinforced specimens with no transverse reinforcement exhibited a significant size effect.
2. The amount of longitudinal reinforcement had a significant influence on the member's shear capacity as will be examined again in Chapter 7.
3. Providing a small amount of transverse reinforcement resulted in significant gains in both of the member reserve strength and the ductility.
4. The simple modifications to the current ACI shear design equation, proposed by Collins and Kuchma will result in a more consistent level of safety across the possible range of concrete strengths and member sizes.
5. The shear design methods based on MCFT, known as “the General Method” or “Beta Method” are capable of providing consistent reliable predictions in all the situations examined.

3.3.3 Review of the Proposed Simple Modifications to the Current ACI

This section will give a brief review of the proposed simple modifications to the current ACI shear design formulation. The equation discussed here later will be extended to include the effect of longitudinal reinforcement ratio.
1. For members containing less than the specified minimum quantity of stirrups, $V_c$ should be calculated by Eq. (1) with the parameters and limitations as defined in Figure 3-5.

2. The specified minimum quantity of stirrups should be given by Eq. (2).

3. Members should contain at least the minimum quantity of stirrups if $V_u$ exceeds $\phi V_c$, where $V_c$ is given by Eq. (1).

4. The traditional expression for $V_c$, Eq. (3), should be used in determining the shear strength of members with more than minimum stirrups.

$$V_c = \frac{245}{1275 + s_e} \sqrt{f'_c b_w d} \quad \text{(mm and MPa units)} \quad \text{(1)}$$

$$A_{\text{min}} = \frac{\sqrt{f'_c}}{12 f_y} b_w s \quad \text{(MPa units)} \quad \text{(2)}$$

$$V_c = \frac{2}{12} \sqrt{f'_c b_w d} \quad \text{(MPa units)} \quad \text{(3)}$$
### Table 3-3

<table>
<thead>
<tr>
<th>Name</th>
<th>$h$</th>
<th>$d$</th>
<th>$b$ (mm)</th>
<th>$f'_{ce}$ (MPa)</th>
<th>$\rho$ (%)</th>
<th>$\rho f_{ce}$</th>
<th>$S_{mx}$ (mm)</th>
<th>$a$ (mm)</th>
<th>$a/d$</th>
<th>Test $\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BN100</td>
<td>1000</td>
<td>925</td>
<td>300</td>
<td>37.2</td>
<td>0.76</td>
<td>~</td>
<td>1121</td>
<td>10</td>
<td>2.92</td>
<td>0.113</td>
</tr>
<tr>
<td>B100</td>
<td>1000</td>
<td>925</td>
<td>300</td>
<td>36</td>
<td>1.01</td>
<td>~</td>
<td>1120</td>
<td>10</td>
<td>2.92</td>
<td>0.135</td>
</tr>
<tr>
<td>B100L-R</td>
<td>1000</td>
<td>925</td>
<td>300</td>
<td>39</td>
<td>1.01</td>
<td>~</td>
<td>1120</td>
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<td>0.136</td>
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<tr>
<td>BN12.5</td>
<td>125</td>
<td>110</td>
<td>300</td>
<td>37.2</td>
<td>0.91</td>
<td>~</td>
<td>133</td>
<td>10</td>
<td>3.07</td>
<td>0.199</td>
</tr>
<tr>
<td>BN25</td>
<td>250</td>
<td>225</td>
<td>300</td>
<td>37.2</td>
<td>0.89</td>
<td>~</td>
<td>273</td>
<td>10</td>
<td>3.00</td>
<td>0.177</td>
</tr>
<tr>
<td>BN50</td>
<td>500</td>
<td>450</td>
<td>300</td>
<td>37.2</td>
<td>0.81</td>
<td>~</td>
<td>545</td>
<td>10</td>
<td>3.00</td>
<td>0.160</td>
</tr>
<tr>
<td>BM100</td>
<td>1000</td>
<td>925</td>
<td>300</td>
<td>47</td>
<td>0.76</td>
<td>0.402</td>
<td>1121</td>
<td>10</td>
<td>2.92</td>
<td>0.180</td>
</tr>
<tr>
<td>BM100D</td>
<td>1000</td>
<td>925</td>
<td>300</td>
<td>47</td>
<td>1.05</td>
<td>0.402</td>
<td>229</td>
<td>10</td>
<td>2.92</td>
<td>0.242</td>
</tr>
<tr>
<td>YB2000/0</td>
<td>2000</td>
<td>1890</td>
<td>300</td>
<td>35.4/31.8</td>
<td>0.74</td>
<td>0</td>
<td>2450</td>
<td>10</td>
<td>2.86</td>
<td>0.078</td>
</tr>
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<td>YB2000/4</td>
<td>2000</td>
<td>1890</td>
<td>300</td>
<td>35.4/31.8</td>
<td>0.74</td>
<td>0.33</td>
<td>2450</td>
<td>10</td>
<td>2.86</td>
<td>0.21</td>
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<td>YB2000/6</td>
<td>2000</td>
<td>1890</td>
<td>300</td>
<td>37.3/34.5</td>
<td>0.74</td>
<td>0.33</td>
<td>2450</td>
<td>10</td>
<td>2.86</td>
<td>0.16</td>
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<td>2000</td>
<td>1890</td>
<td>300</td>
<td>37.7/35.0</td>
<td>0.74</td>
<td>0.37</td>
<td>2450</td>
<td>10</td>
<td>2.86</td>
<td>0.138</td>
</tr>
</tbody>
</table>

**Figure 3-5 Suggested Modifications to ACI Shear Design Equation**  
(Adapted from Collins and Kuchma)
4. Experimental Test Program

4.1 YB Series Tested by Yoshida

The current study is intended to complement the earlier work of Yoshida by examining the influence of longitudinal reinforcement ratio on the shear response of large members.

The four sections of the two large beams tested by Yoshida are described in Table 4-1. As can be seen all four sections contained 0.74% of longitudinal reinforcement. One section YB2000/0 contained no transverse reinforcement while the other three sections contained about the minimum amount of transverse reinforcement specified by the ACI code. These three specimens differed in the spacing and size of the T-headed bars used to provide this minimum transverse reinforcement.

The test results from the four sections tested by Yoshida are summarized in Table 4-2. Note that the section without transverse reinforcement failed at a shear of 255 kN while the section containing US #6 T-headed bars at 1350 mm spacing failed at a shear of 550 kN.
Table 4-1. Properties for YB Series Member

<table>
<thead>
<tr>
<th>Specimen</th>
<th>YB2000/0/9</th>
<th>YB2000/4/6</th>
</tr>
</thead>
<tbody>
<tr>
<td>YB2000/0</td>
<td>East End</td>
<td>West End</td>
</tr>
<tr>
<td>YB2000/6</td>
<td>East End</td>
<td>West End</td>
</tr>
<tr>
<td>Width x Height</td>
<td></td>
<td></td>
</tr>
<tr>
<td>300 x 2000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Cross Sectional Dimensions

<table>
<thead>
<tr>
<th>Shear span</th>
<th>Width x Height</th>
<th>Width x Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>a (mm)</td>
<td>5400</td>
<td>5400</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shear span ratio</th>
<th>a/d</th>
<th>a/d</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.86</td>
<td>2.86</td>
</tr>
</tbody>
</table>

Longitudinal Reinforcement

<table>
<thead>
<tr>
<th>Top</th>
<th>3-20M (A=900mm²)</th>
<th>3-20M (A=900mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>d' (mm)</td>
<td>70</td>
<td>70</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bottom</th>
<th>6-30M (A=4200mm²)</th>
<th>6-30M (A=4200mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>d (mm)</td>
<td>1890</td>
<td>1890</td>
</tr>
<tr>
<td>ρ bottom (%)</td>
<td>0.74</td>
<td>0.74</td>
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</tbody>
</table>

Transverse Reinforcement

<table>
<thead>
<tr>
<th>Designation</th>
<th>~</th>
<th>US #9 (A=900mm²)</th>
<th>US #6 (A=900mm²)</th>
<th>US #4 (A=900mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spacing s (mm)</td>
<td>~</td>
<td>2700</td>
<td>1350</td>
<td>590</td>
</tr>
<tr>
<td>Yield Strength (Mpa)</td>
<td>~</td>
<td>470</td>
<td>465</td>
<td>468</td>
</tr>
<tr>
<td>Shear Reinforcement index (f_y A_y / b_w s)</td>
<td>~</td>
<td>0.37</td>
<td>0.33</td>
<td>0.33</td>
</tr>
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</table>
Table 4-2. Test Results for Members of YB Series

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Overall depth (mm)</th>
<th>Width (mm)</th>
<th>$d_v$ (mm)</th>
<th>$f'_c$ (MPa)</th>
<th>$A_x$ (mm²)</th>
<th>$\rho_x$ (%)</th>
<th>$s$ (mm)</th>
<th>$\rho_y$ (MPa)</th>
<th>$a/d$</th>
<th>$P_{exp}$ (kN)</th>
<th>$V_{exp}$ (kN)</th>
<th>$\delta_{exp(max)}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>YB2000/0</td>
<td>2000</td>
<td>300</td>
<td>1890</td>
<td>35.4/31.8</td>
<td>4200</td>
<td>0.74</td>
<td>~</td>
<td>~</td>
<td>2.86</td>
<td>461</td>
<td>255</td>
<td>8.0</td>
</tr>
<tr>
<td>YB2000/4</td>
<td>2000</td>
<td>300</td>
<td>1890</td>
<td>35.4/31.8</td>
<td>4200</td>
<td>0.74</td>
<td>590</td>
<td>0.33</td>
<td>2.86</td>
<td>1300</td>
<td>674</td>
<td>~</td>
</tr>
<tr>
<td>YB2000/6</td>
<td>2000</td>
<td>300</td>
<td>1890</td>
<td>37.3/34.5</td>
<td>4200</td>
<td>0.74</td>
<td>1350</td>
<td>0.33</td>
<td>2.86</td>
<td>1053</td>
<td>550</td>
<td>34</td>
</tr>
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<td>300</td>
<td>1890</td>
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<td>4200</td>
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<td>2700</td>
<td>0.37</td>
<td>2.86</td>
<td>897</td>
<td>472</td>
<td>27</td>
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</tbody>
</table>
4.2 Specimen Details:

The four sections of the two large beams tested in this investigation are described in Figure 4-1. Figure 4-2 and Table 4-3. While Yoshida's two beams both contained 6-No. 30 bars as longitudinal reinforcement, one beam, SB2012, of the current study contained 12 - No. 30 bars while the other, SB2003, contained only 3- No. 30 bars. For both beams one end contained no transverse reinforcement while the other end contained US #6 T-headed bars at 1350 mm spacing.

The small cage shown in Figure 4-2 which was placed under the point of load application at mid-span was intended to delay the destruction of the flexural compression zone under the load so that after the weaker half of the beam had failed in shear repairs could be more easily made.
Figure 4-2. View of the Compression Zone Reinforcing Detail
Table 4-3 Details of the SB Series

<table>
<thead>
<tr>
<th>Specimen</th>
<th>SB2003/0</th>
<th>SB2003/6</th>
<th>SB2012/0</th>
<th>SB2012/6</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Width x Height</strong></td>
<td>300x2000</td>
<td>300x2000</td>
<td>300x2000</td>
<td>300x2000</td>
</tr>
<tr>
<td><strong>Shear span</strong></td>
<td>5400</td>
<td>5400</td>
<td>5400</td>
<td>5400</td>
</tr>
<tr>
<td><strong>Shear span to depth Ratio (a/d)</strong></td>
<td>2.81</td>
<td>2.81</td>
<td>2.92</td>
<td>2.92</td>
</tr>
</tbody>
</table>

**Longitudinal reinforcement**

| Top | Note: second layer compressive reinforcement locates locally on the top of the mid-span, with a total length of 1 m, 0.5 m for each span. See figure 4-1 for stirrup details. |
| Layer 1 | 3 20M | 3 20M | 3 20M | 3 20M |
| Layer 2 (local) | 2 20M | 2 20M | 2 20M | 2 20M |
| Stirrups (local) | #3 @100mm (within 1000 mm) | #3 @100mm (within 1000 mm) | #3 @100mm (within 1000 mm) | #3 @100mm (within 1000 mm) |
| Top rebar center to beam top | 70 | 70 | 70 | 70 |
| Bottom rebar center to beam bottom | 75 | 75 | 75 | 75 |
| Bottom | 3 30M | 3 30M | 12 30M in 3 layers | 12 30M in 3 layers |
| Bottom reinforcement ratio | 0.36% | 0.36% | 1.52% | 1.52% |

**Transverse reinforcement**

| T-headed bars | ~ | US #6 | ~ | US #6 |
| Spacing | ~ | 1350 | ~ | 1350 |

Note: all units are in mm.
The dimensions of the specimens were 22m in length, 2m in depth, and 300mm in width. The size of the specimens was chosen so that the capacity of the laboratory space and its lifting equipment was fully extended. Moreover, as has been mentioned before, this is the second in a series of tests, hence, a geometric similarity with the previous set of tested specimen was pursued in order to maintain a maximum degree of uniformity among all tests.

The two beams are named separately as SB2003 and SB2012, 2 as the first digit represents the depth of the member, 3 and 12 are the number of the bottom longitudinal rebars. Each member has two half spans that are differently reinforced in terms of their transverse reinforcement. Hence, in SB2003/0, “/0” refers to one half span of the member which contains no transverse reinforcement, while in SB2003/6, “/6” refers to the other half span which contains US #6 T-headed bars as shear reinforcement. Likewise, SB2012/0 and SB2012/6 contains no shear reinforcement in one half span and #6 reinforcement in the other. The spacing of the T-headed bars remains the same in all cases—at 1350mm, which gives a shear reinforcement index of 0.34MPa, \((\frac{A_{svy}}{bw}) = 0.338\text{MPa}\) about the minimum amount required by the ACI 318-99 shear provisions (0.33MPa).

4.3 Material Properties
4.3.1 Concrete Properties

As in the previous tests, Dufferin Concrete provided the concrete for this project. The cylinders were cast simultaneously at the pouring. Since the concrete was shipped to site by two separate trucks, two separate batches of cylinders were prepared and designated respectively batch #1 and batch #2, the aggregate size for both batches is 10mm.

In order to maintain equivalent curing conditions between the cast specimen and the cylinders, the side forms were kept on the beam for 3 weeks. When the forms were dissembled, the cylinders were taken out of the containers at the same time. The curing of the concrete was achieved by spreading wet burlaps on top of the two members and the cylinders, which were then sprinkled regularly everyday for one week since the casting.
The development of the concrete compressive strength was recorded by testing 3 cylinders from each batch every 7 days. The 5000kN capacity MTS machine was used to obtain the full compressive stress-strain response at the day of testing, while the 7, 14, 21, 28 day peak strengths were recorded by using a simple load-control tester in the concrete laboratory.

The recorded curves for cylinder strength gain vs. day for each of the two concrete batches are shown in Figure 4-3 and Table 4-4.

![Concrete Strength vs. Days](image)

Figure 4-3 Strength Development of the Concrete
Table 4-4 Concrete Properties

<table>
<thead>
<tr>
<th>Beam</th>
<th>SB2012/0&amp;6</th>
<th>SB2003/0&amp;6</th>
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</thead>
<tbody>
<tr>
<td>Specimen</td>
<td>SB2012/0</td>
<td>SB2012/6</td>
</tr>
<tr>
<td>Cast Date</td>
<td>June. 22</td>
<td>June. 22</td>
</tr>
<tr>
<td>Test Date</td>
<td>Aug. 2</td>
<td>Aug. 3</td>
</tr>
<tr>
<td>Age at Test Date</td>
<td>41</td>
<td>42</td>
</tr>
<tr>
<td>$f_c$ on test date (MPa)</td>
<td>Batch #1</td>
<td>28.7</td>
</tr>
<tr>
<td></td>
<td>Batch #2</td>
<td>26.2</td>
</tr>
</tbody>
</table>

4.3.2 Steel Properties

The stress-strain relationships of the rebars and T-headed bars used in this project were obtained by coupon tests using a MTS machine (three coupons cut from the bottom rebars and from the T-headed bars, for a total of 6 tests). The properties as obtained in the tests are listed in Table 4-5.

The fractured surface of all the 3 tested coupons for the longitudinal rebars indicate a somewhat brittle-ductile failure feature with fracture actually extending from a small but noticeable defect locating near the circumference of the rebar. The necking is not that usually seem predominantly noticeable upon the final failure. Necking and the so-called cup and cone phenomena, on the other hand, were observed in the broken coupons for the US #6 rebars.

The fractured coupon surface of the tested rebars and T-heads are shown in Figure 4.4.
Table 4-5 Steel Properties

<table>
<thead>
<tr>
<th>Rebar Size</th>
<th>Nominal Diameter (mm)</th>
<th>Cross-sectional Area (mm$^2$)</th>
<th>Yield Stress (MPa)</th>
<th>Ultimate Stress (MPa)</th>
<th>Strain hardening Strain $\varepsilon_{sh}$ (mm/m)</th>
<th>Rupture Strain $\varepsilon_{src}$ (mm/m)</th>
<th>Young's Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>US #6</td>
<td>19.1</td>
<td>284</td>
<td>483</td>
<td>615</td>
<td>9</td>
<td>273</td>
<td>200000</td>
</tr>
<tr>
<td>M 20</td>
<td>19.5</td>
<td>300</td>
<td>433</td>
<td>638</td>
<td>14.4</td>
<td>148</td>
<td>200000</td>
</tr>
<tr>
<td>M 30</td>
<td>30</td>
<td>700</td>
<td>436</td>
<td>700</td>
<td>8</td>
<td>183</td>
<td>200000</td>
</tr>
</tbody>
</table>

Figure 4-4 Typical Fracture Surfaces of No.30 (30M) and US #6 bars
4.4 Specimen Construction

4.4.1 Formwork Set Up

The form-work was designed and constructed by Yoshida in his tests for investigating the size effect of the large, lightly reinforced structural members. The form-work was thus available for this project.

As shown in Figure 4-5 the form-work is composed of two sets of side forms and one set of middle forms with the two specimens sharing the same middle form. The sides are propped up before casting by an array of two pairs of bracers at the waist level of the side forms to provide supports to stabilize the whole assembled form-work system. Linkages between the form-work is achieved by simply inserting steel rods at regular intervals longitudinally along the forms.

In order to maintain a constant 300mm inner-form spacing, wood spacers were also used, spanning through the 3 rails of form-works on the top, it forces the spacing between the form-works to be exactly at 300mm. Together with the middle level steel rods and the bottom base planks, the spacing inside of the form-works is consistently maintained everywhere.

Two layers of steel channels were mounted around the whole form-work system to act as a peripheral girdle to efficiently wrap up all the sides and ends. Taking large pressures during the casting, these steel channels are the key components that ensure the concreting can be safely and efficiently carried out for these two very large beams.

The threaded steel rods, arranged with the steel channels, are maneuvered into two separate layers, namely, top layer and middle layer. The top layer rods, fastened onto top level steel channels by nuts at both of its ends, are also used to hang the top layer compression rebar cages into position; the middle layer, as already mentioned early, is used to provide middle level inter-form linkage.

The middle level steel rods are sheathed with PVC tubes to eliminate the bond to the concrete and to permit the threaded rods to be reused.

Figure 4-5 provides a cross section view of the form-work.
4.4.2 Assemblage of the Rebar Cages

SB2012 contained twelve of longitudinal reinforcing bars, many of which had a number of strain gauges. In order to keep these rebars in position during casting, they were tied into a unit using #3 bars in the pattern described in Figure 4-6.
The middle 2 rebars in each layer of the cage were purposely loosened until all the T heads were inserted. Then, all the steels were bonded together tightly by double cross steel wires. Strain gauge tails were tightened underneath the longitudinal rebars in order to achieve some level of protection against the forces from the concrete during casting.

4.4.3 Concrete Casting and Curing

The concrete was cast on June 22, 2000. With a total volume of 14.4 m³, the casting of the two specimens require two nine cubic meter ready-mix trucks.

The concrete was transported from the trucks on the laboratory ramp to the forms by a 0.9 m³ concrete bucket. Casting was performed by filling up both sides of the forms simultaneously in order to prevent concrete pressure differentials inside the forms. Hence when the first truck load was placed, the concrete level on both sides of the middle form was about the same, leaving the upper half of the forms to be filled by the second truck. Note this
casting scheme puts the first batch of concrete, designated as #1, in the lower half of the specimens, with the second batch, #2, on the top half.

On the casting day, a preset fixed working platform was set up at the south-east side of the form-work (the forms were oriented on an east-west axis), covering 6 meters at that end, it provided a fixed access to the top of the form. Access to the top on the west part of the form-work was made possible by installing scaffolding, sitting on wheels and fully maneuverable, this soldier scaffolding was convenient in providing a mobile platform so that concrete buckets could be easily guided during the casting procedure.

Due to the closely spaced bottom reinforcing bars which were in 3 layers, about 315 mm high up from the form bottom, a small diameter (about 35mm) vibrator was chosen for vibrating the concrete during casting. With a length of 3 meters this vibrator could reach the bottom concrete around the rebars and hence ensure a consistent quality for the cast concrete.

The whole casting procedure began at 10:30am in the morning, lasted for 5 hours and ended at 3:30pm when the top concrete was smoothed down and wet burlap spread on. The moisture was kept in the concrete by an extra layer of plastic film that wrapped around the wetted burlap and was sealed tightly onto the side form-work.

4.5 Test Rig Details

As in the previous tests by Yoshida a three point loading scheme was selected for these tests. Right underneath the loading head of the testing machine, a pivot type roller was installed on top of a rigid thick steel plate (300mm x 300mm x 40mm in dimension) that sits on top of the member at the middle of the span. The pivot, used to give better compliance to cater for any possible rotations during the testing, ensures the load under the loading pad is distributed evenly without introducing any form of second order forces. Likewise, at the supporting points, rollers of a similar type were used. Centered at 5400mm from the Baldwin loading point, the rollers sit on steel pedestals and support the beam through a thin layer of fast drying mortar which acts as a buffer zone, ensuring a maximum contact between the specimen's bottom surface and the reaction plates of the rollers.
The bolts in the supporting pivots were removed prior to the testing to allow for a full roller support at the beam-ends. Since both of the beam-ends are supported on such rollers, the horizontal free extension of the beam is ensured. A view of a roller support is given in Figure 4-7.

Figure 4-7. Roller Support
(Note the bolts shown in the picture were removed just before the beginning of the test.)
4.5.1 Instrumentation

Figure 4-8 shows a fully instrumented specimen just before test.

Figure 4-8. Fully Prepared Specimen
4.5.2 Electronic Gauges

Electric resistance strain gauges, of 10-mm gage length, were applied to the longitudinal rebars and the T-headed bars to give measurements of local strain conditions in this reinforcement.

One longitudinal reinforcing bar in each layer of reinforcing was equipped with 10 such gauges. Applied on both sides of any single location, the possibility of losing all gauge readings simultaneously due to some unexpected circumstances is then minimized, moreover, the electronic noise may easily be averaged out by comparing the corresponding other readings. The positioning of these strain gauges and their designation system are illustrated in Figure 4-9. As can be seen the longitudinal bars were gauged at mid-span, at the inside edge of the bearing plate and at about the quarter points of the span. For the T-headed shear reinforcement gauges were placed at mid-height of the beam and at a location a little above the flexural tension reinforcement.
Figure 4-9 Gauge Distribution and Designation System
Surface strains were measured on the south side of the test member using so-called Zurich demountable mechanical strain gages with electronic feed-outs. Strain readings were taken in the longitudinal, transverse and two diagonal directions, at each load stage, from targets placed on a 300-mm grid over most of the surface between the supports. The grids tapered off towards the supports, to cover the zone of potential cracking. See Figure 4-10 and 4-11.

Figure 4-10 Arrangement of Zurich Targets
Figure 4-11. Demountable Mechanical Strain Gauges for Obtaining Zurich Reading
Linear variable differential transducers (LVDTs) arranged in large 45° crosses, were mounted on the north side of the member. Covering the whole member span, they provided a continuous monitoring of average shear strain conditions. See Figure 4-12.

![LVDT surface grids](1900x1900)

Figure 4-12 Diagonally Arranged Surface LVDTs

The arrangement of the vertical LVDTs under the specimen's bottom surface is illustrated in Figure 4-13.

![Vertical LVDT arrangement](1900x1900)

Figure 4-13 Vertical LVDT Arrangement
4.6. Testing Procedures and Post Failure Reinforcement

The load was applied by a large Baldwin universal testing machine. The whole expected failure load was divided into a number of load stages. The criteria for choosing a proper magnitude for each of them was based largely on the on-site observed behavior of the beam under test and the analytical predictions for the ultimate load from either Vector2 or Response2000. A total of between five and eight load stages were employed.

At each load stage, crack patterns were marked and the width of significant cracks were measured using a crack-width comparator. In addition the displacements of Zurich targets were read.

During the loading stages when readings were being taken, the applied load was reduced by about 10% both as a safety precaution and to hold the deformations about constant.

After the weaker span (the span without transverse reinforcement) failed, the failed part was tied together by several sets of external Dywidag high-strength bars which were clamped by a pair of steel box sections on both the top and bottom of the member.

The idea is illustrated in figure 4-14. The top picture shows the newly patched fast-harden concrete after the span failed in shear, the compressive cylinder strength of this new paved concrete reached 40MPa in just 3 days. The strengthened span with the external reinforcement shown on the bottom picture was then strong enough to carry the load until the second half of the beam reached its failure load.
Figure 4-14. Post Failure Reinforcement
5. Test Observations

5.1 Overview of the Test Results

SB2012/0 was tested 42 days after the concrete was cast. The purpose of this test was to examine how the shear behavior of such large beams will be influenced by the presence of a relatively large amount of longitudinal reinforcement. The two spans, SB2012/0 and SB2012/6 were detailed to contain the same amount of flexural reinforcement at 1.52% but with a varying amount of shear reinforcement - zero for SB2012/0 and the traditional ACI minimum amount of shear reinforcement ($A_vf_y/b_wS = 0.33\text{Mpa}$) for SB2012/6.

The test on SB2012/0 began at 10:30am in the morning, load was gradually applied. Due to the excellent crack control of the four layers of 30M bars on the bottom, the flexural cracks developed very slowly and concentrated in a limited area under the mid-span load. Diagonal cracks were first observed in the west end of the beam, the end which contains shear reinforcement, at an applied machine load of about 620kN. No such cracks appeared in the east end, the end with no shear reinforcement, until just prior to failure at 758 kN, when a critical diagonal crack opened without warning. The mid-span deflection at this load was 13 mm.

The test on SB2012/6 followed after the failed SB2012/0 span was reinforced with 5 sets of external Dywidag bars. This span behaved in a rather ductile manner as compared with SB2012/0 with new cracks developing and previous cracks extending steadily. With crushing of the top concrete compression zone adjacent to the loading pad, the member failed at a machine load of 1225 kN. The accompanying mid-span deflection was observed to be 28 mm.

It can be seen from the load-deflection curves given in Figure 5-1 that the addition of even a minimum amount of shear reinforcement considerably enhances the strength and deformation capability of the member.

The two tests on SB2003 were conducted two weeks later. The span with zero shear reinforcement (SB2003/0) was tested first. As expected a very brittle behavior was observed with failure happening very abruptly and totally destroying the structural integrity of the member. The span could only resist a central load of 399 kN with a mid
span deflection of 14mm. Once again, significant diagonal cracks were first observed in the west half of the span – the side with shear reinforcement.

One week later, after a major repair was completed, the test on SB2003/6 was carried out. At a central load somewhat higher than 500kN, the longitudinal rebars at the mid-span began to yield as indicated by the wide mid-span flexural cracks (2mm) and the rebar strain gauge readings, the load – deflection curve began to flatten. At the final load stage when central Baldwin load approached 652kN the T-head bar that crossed the primary diagonal shear crack began to yield. Failure occurred by a sudden widening of the primary shear crack at a load of 652 kN and a mid-span deflection of 66mm.

Table 4.7 compares the load – deflection responses of the four SB specimens and the comparable YB specimens – YB2000/0 and YN2000/6. It can be seen that the post crack stiffness in SB2012 are higher than those of SB2003 with the YB series in the middle, due to the difference in the amount of flexural reinforcement.

Figure 5.1 Comparison of Load-Deflection Curves of the Six Specimens
5.2 Detailed observations of SB2012/0

The observations made during the test are given below.

**Load stage 1**
- Central load = 150 kN
- Displacement = 0.6 mm

Observation:
Load is linearly picking up without any crack.

**Load stage 2**
- Central load = 250 kN changed to 300 kN.
- Displacement = 1.5 mm

Observation:
No crack appears yet, prediction of the load at first cracking was about 200 kN.

**Load stage 3**
- Central load = 350 kN
- Displacement = 2.0 mm

Observation:
Two cracks that are symmetric to the loading point appear in both of the spans, typical flexural cracks, perpendicular to the bottom edge of the beam with an approximate height of 700 mm, both appears to be in the order of 0.1 mm in width.

**Load stage 4**
- Central load = 450 kN
- Displacement = 4.4 mm

Observation:
Cracks are symmetrically expanding with respect to the centerline of the beam. All are flexural type. No typical flexure shear crack appears yet.

**Load stage 5**
- Central load = 520 kN
- Displacement = 5.9 mm

Observation:
Cracks begin to develop unsymmetrically with most of the new cracks appearing in the west span that contains T-headed bars spaced at 1.35 m. The east span, which was predicted to fail by this stage, still doesn't show diagonal cracks except for some already existing cracks that begin to widen, but still remains in the range of 0.05 to 0.1 mm.
Load Stage 6:
Central load = 620 kN
Displacement = 7.7 mm

Observation:
More small diagonal cracks opened up in the west span with average width going up to 0.1 to 0.3 mm. For the east side, no remarkable shear cracks happen yet.

Load Stage 7:
Central load = 720 kN
Displacement = 10.4 mm

Observation:
Cracks are still asymmetrically expanding on the west side of the span, the east side has nothing new comparing to the previous stage. Careful checks are then performed to make sure the roller supports underneath the beam are not jammed. As indicated from the strain gauge readings from T heads in the west span, if not for the presence of those T-heads, the west span would fail in this stage. Readings from those strain gauges have been organized into charts in Appendix C.

West side Zurich readings are obtained at this stage for fear of any possible unexpected failure of that part of the beam.

Load Stage 8:
Central load = 758 kN
Displacement = 13.0 mm

Observation:
A suddenly critical diagonal shear crack happens in the east span, which is not a simple extension from any of the existing flexural cracks. Rather, it appears beyond the outer most flexural cracks, stemming around 3 meters from the east support, sharply directed to the loading point indicating a major shear failure is pending. It appears all of a sudden. --- A typical critical diagonal shear crack. At the onset it reached a width of 3mm.

The test was then halted in order to preserve the top compression concrete from being seriously damaged which, if is the case, may affect the test on the other span.
Figure 5-1 Final Loading Stage for SB2012/0
5.3 Specimen SB2012/6

Load stage 1:
Central load = 300 kN
Displacement = 8.9 mm

Observation:
Load is first zeroed, then raise to 300 kN, no new cracks appear during the operation.

Load stage 2:
Central load = 600 kN
Displacement = 12.5 mm

Observation:
East side primary crack is widening, maximum in the middle hits 4 mm. No new cracks appear. (East side has been reinforced with Dywidag bars, spaced at about 1200 mm).

Load stage 3:
Central load = 800 kN
Displacement = 16.5 mm

Observation:
New cracks begin to develop on top of the older ones, particularly, one of the primary diagonal cracks which suddenly widened to 5 mm, showing a sign of shear failure. Cracks closer to the bottom edge of the beam are progressively developing, turning and go horizontally, forming splitting lines along the top level longitudinal rebars on both sides of the beam.

Load stage 4:
Central load = 1000 kN
Displacement = 20.56 mm

Observation:
P-Δ curve shows a decaying trend, concrete adjacent to the loading pad underneath the Baldwin loading head begin to split introducing a nearly horizontal crack spread out parallel to the top surface of the member. The top 3 M20 compressive rebars and the intercepted T head are still holding the two splitting parts firmly together. Primary shear cracks are still developing, widening and rotating, but at a slower pace.
Load Stage 5:
Central load = 1152 kN
Displacement = 24.42 mm

Observation:
Local compressive concrete near the loading pad begins to fail, P-Δ curve is curving down, showing a trend for leveling off.

Load Stage 6
Central load = 1225 kN
Displacement = 28 mm

Observation:
Locally failed concrete compressive zone is spreading which then interferes with the primary shear cracks, connected at first and then separated to form a large horizontal crack running from immediately beside the Baldwin loading pad to 1.2 meters west. Compressive reinforcing bars are buckled and tossed upward beyond 500 mm from the loading point where the second layer of compression reinforcement terminates. The T-headed bar holds down the compressive rebars from being pushed up at a position 675 mm from the member’s central line. Compression reinforcement can be seen visually through the extremely large 12 mm horizontal cracks, the beam is at the brink of failure. See figure 5-2.

The crack patterns for SB2012/0 and SB2012/6 are followed at the end of this section.
Figure 5-2. Splitting of the Top Compressive Concrete
Figure 5-3. Final Loading Stage for SB2012/6
Specimen: SB2012/0  Load Stage: 1,2  Applied Load: 150, 300 kN

Crack Pattern

Specimen: SB2012/0  Load Stage: 1,2  Applied Load: 150, 300 kN

Crack Width
Specimen: SB2012/0  Load Stage: 5  Applied Load: 520 kN

Crack Pattern

Specimen: SB2012/0  Load Stage: 5  Applied Load: 520 kN

Crack Pattern
Specimen: SB2012/0
Load Stage: 6
Applied Load: 620 kN
Crack Pattern

Specimen: SB2012/0
Load Stage: 6
Applied Load: 620 kN
Crack Pattern
Specimen: SB2012/0  Load Stage: 7  Applied Load: 720 kN

Crack Pattern

Specimen: SB2012/0  Load Stage: 7  Applied Load: 720 kN

Crack Width
Specimen: SB2012/0  Load Stage: Final  Applied Load: 760 kN

Crack Pattern

Specimen: SB2012/0  Load Stage: Final  Applied Load: 760 kN

Crack Pattern
Specimen: SB2012/6      Load Stage: 2      Applied Load: 600 kN

Crack Pattern

Specimen: SB2012/6      Load Stage: 2      Applied Load: 600 kN

Crack Pattern
Specimen: SB2012/6    Load Stage: 3    Applied Load: 800 kN

Crack Pattern

Specimen: SB2012/6    Load Stage: 3    Applied Load: 800 kN

Crack Pattern
Specimen: SB2012/6  Load Stage: 4  Applied Load: 1000 kN

Crack Pattern

Specimen: SB2012/6  Load Stage: 4  Applied Load: 1000 kN

Crack Width
Specimen: SB2012/6  Load Stage: 5  Applied Load: 1150 kN

Crack Pattern

Specimen: SB2012/6  Load Stage: 5  Applied Load: 1150 kN

Crack Width
5.4 Specimen SB2003/0

Load stage 1
Central load = 133 kN
Displacement = 0.8 mm

Observation:
Load is linearly picking up without any cracks.

Load stage 2
Central load = 220 kN
Displacement = 1.65 mm

Observation:
Several flexural cracks appear, with one of them rising up about 1.2m directly to the loading point. All the cracks are near the mid-span, starting from the beam bottom.

Load stage 3
Central load = 281 kN
Displacement = 5.8 mm

Observation:
Many cracks appear near mid-span in this loading stage. Three major cracks already extend to a considerable height with the biggest crack width reaching 0.55mm --- quite large for this early stage. All cracks are of mid-span flexure type.

Load stage 4
Central load = 325 kN
Displacement = 7.8 mm

Observation:
The biggest crack in the mid-span region develops to be a major flexural crack. The width at the bottom end of this crack is about 0.9 mm, and is still visually widening with the increasing load.

Load stage 5
Central load = 375 kN
Displacement = 10.4 mm

Observation:
The cracks are consistently widening and spreading, the farthest cracks have gone beyond where the second stirrup is positioned---about 2000mm afar from the loading center on both spans.
Load stage Final
Central load = 399 kN
Displacement = 14.0 mm

Observation:
A large crack suddenly appears, when it appears, the whole of SB2003 falls into parts. The crack doesn't seem to develop from any of the existing one, and happens beyond the outmost crack.

A typical brittle shear failure

Figure 5-4 Final Loading Stage for SB2003/0
5.5 Specimen SB2003/6

Load stage 1:
Central load = 350 kN
Displacement = 10.1 mm

Observation:
Reload to 350kN, the existing cracks in this span are stable during the operation.

Load stage 2:
Central load = 500 kN
Displacement = 14.8 mm

Observation:
Many new diagonal cracks appear actively with the older ones progressively extending and widening, the mid-span flexural crack which happened in the testing of the SB2003/0, widened to about 2.5 mm. The longitudinal steel strain at this position approaches 2183-micrometers indicating that the M30 bars are just yielding.

Load stage 3:
Central load = 550 kN
Displacement = 21 mm

Observation:
Dial-gauge readings from the member’s west end start to increase rapidly indicating a considerable increase in the length of the beam. Primary diagonal shear crack has widened to 2mm. The two sets of major cracks, one located in the middle span, of a flexural type, with a width of 2.5mm, another goes diagonally, of a typical shear crack, about 2 mm width, are indicating two dominant but competing different mechanisms that may introduce the final failure to the testing beam. The E3X gauge reading goes up with the increasing load from 2183ue to 2300ue.

Load stage 4:
Central load = 596 kN
Displacement = 28.5 mm

Observation:
The longest loading stage, loads are vibrating around the average value but the whole trend is still keeping up.
E3X strain gauge reading suddenly jumps from 2442 ue to 5650 ue at 595 kN and then drops rapidly to 2445 ue, there it keeps for a while and then quickly lowers to 774 ue when the applied load goes from 593 to 599 kN.

Cracks are progressively widening. Both of the two major cracks have widened to a perilous 4mm width in this stage.

**Load stage 5**

Central load = 642 kN  
Displacement = 52.5 mm

Observation:  
M30 bar in the middle span (E3X) remains at a lower value about 742 ue over the whole of this loading stage while the gauge beside (E4X) it remains relatively stable at 1700ue. T heads at several locations stay stable at about 1500 ue.

**Load stage Final**

Central load = 652 kN  
Displacement = 66 mm

Observation:  
The load is still creeping up with only the diagonal primary crack keeping widening at this time. A very loud sound is suddenly heard. The beam finally failed by a rapid opening of the primary diagonal crack. The load drops to about 300 kN in a moment, however seriously damaged, the beam hasn’t totally lose its integrity.

The specimen at this stage is believed to fail. The test is hence terminated. The overall failure mode, as already being predicted by Response2000, is a post flexure shear failure. This can be seen by the very large mid-span deflection that upon failure is about 66mm.

Crack patterns for SB2003/0 and SB2003/6 are followed at the end of this section.
Figure 5-5 Final Loading Stage for SB2003/6
Specimen: SB2003/0
Crack Pattern
Load Stage: 1
Applied Load: 133 kN

Specimen: SB2003/0
Crack Width
Load Stage: 1
Applied Load: 133 kN
Specimen: SB2003/0  Load Stage: 2  Applied Load: 220 kN

Crack Pattern

Specimen: SB2003/0  Load Stage: 2  Applied Load: 220 kN

Crack Width
Specimen: SB2003/0
Load Stage: 3
Applied Load: 281 kN

Crack Pattern

Specimen: SB2003/0
Load Stage: 3
Applied Load: 281 kN

Crack Width
Specimen: SB2003/0
Crack Pattern

Load Stage: 4

Applied Load: 325 kN

Specimen: SB2003/0
Crack Width
Specimen: SB2003/0
Crack Pattern
Load Stage: 5

Applied Load: 375 kN

Specimen: SB2003/0
Crack Width
Load Stage: 5
Specimen: SB2003/6  Load Stage: 2  Applied Load: 500 kN

Crack Pattern

Specimen: SB2003/6  Load Stage: 2  Applied Load: 500 kN

Crack Width
Specimen: SB2003/6    Load Stage: 3    Applied Load: 550 kN

Crack Pattern

Specimen: SB2003/6    Load Stage: 3    Applied Load: 550 kN

Crack Width
Specimen: SB2003/6    Load Stage: 4    Applied Load: 596 kN

Crack Pattern

Specimen: SB2003/6    Load Stage: 4    Applied Load: 596 kN

Crack Width
Specimen: SB2003/6  Load Stage: 5  Applied Load: 642 kN

Crack Pattern

Specimen: SB2003/6  Load Stage: 5  Applied Load: 642 kN

Crack Pattern
6. Analysis of Results


Theoretical predictions of the applied load vs. middle span deflection together with the experimental results for SB2003/0/6 and SB2012/0/6 are compared in the following tables and figures. (figure 6-1 to 6-4, table 6-1 to 6-3).

*Load vs. displacement (SB2003/0)*

![Load vs. displacement (SB2003/0)]

*Figure 6-1*

*Load vs. displacement (SB2003/6)*

![Load vs. displacement (SB2003/6)]

*Figure 6-2.*
Load vs. displacement (SB2012/0)

Figure 6-3

Load vs. displacement (SB2012/6)

Figure 6-4
Finite Element program Vector2 was used in the analysis. The procedures are as follows.

Firstly, a 40x10 grid mesh was generated in a half-span simplified calculation model which considerably cuts down the calculation time and tends to be numerically more stable. See figure 6-5.

The applied load, as shown in the above figure, was then applied vertically as a nodal load at point C. At section AC, 11 rollers are used to fully restrain the member’s horizontal degree of freedom. The support of the beam is represented in this calculation by a roller at nodal point B.

The presence of all these constraints rules out any possible rigid body motion.

Loads were divided into many small stages with a predefined stage-wise increment of about 10kN. This value was set small enough in order to capture the ultimate load at the peak point of load-displacement curve with a reasonable degree of accuracy.
For members with very different reinforcement ratio in the X and Y direction like that of SB2003/0/6, SB2012/0/6 and YB2000/0/6, the rotation and slipping of the cracks are particularly noticeable. The element slip distortion is hence considered, in this calculation, option 5 – Hybrid-II model is adopted. A complete list of the models available is attached in the Appendix.

After cracking, concrete can continue to carry tensile stresses as a result of two independent mechanisms: tension softening and tension stiffening. Tension softening refers to the fracture-associated mechanisms and is particularly significant in concrete structures containing little or no reinforcement. SB2003/0 and SB2012/0 which contain no shear reinforcement, hence adopted this model in their calculations. Post-cracking tensile stresses in the concrete also arise from interactions between the reinforcement and the concrete. In areas between cracks, load is transferred from the reinforcement to the concrete via bond stresses, producing significant levels of average tensile stress in the concrete. This model, central to the MCFT theory, is used in the calculation. It is the one based on Collins-Mitchell 1987 model.

After the basic calculation models were chosen, material properties were then entered following the format defined in the job file of Vector2 “S2R”.

Those values have been detailed systematically in Chapter 4 under “Steel properties” and “Concrete properties”.

Nodal applied loads, apart from the one entered previously at node C simulating central machine loading, are those representing self-weight of the member. Hence, nodally, a vertical downward load of $P_v=0.188$ kN/node is entered.

$$P_v=23.5 \times 0.3 \times 2 \times 6 / (41 \times 11) = 0.188 \text{ kN/node}$$

Where, 23.5 – Gravity density of the normal weight concrete.

0.3x2x6 – Member width x Member depth x Half span length

41x11 – Number of horizontal nodes x Number of vertical nodes
After the computation was done, constraining forces at support B (See figure 6-5) was subtracted from the program output files, mid-span deflections are the nodal vertical displacement at A. Baldwin central loads are then calculated in the following manner.

\[ P_{\text{Baldwin}} = (F_B - F_{\text{self-weight}}) \times 2 \]

Where, \( F_B \) – Support reactions
\( F_{\text{self-weight}} \) – Support reaction due to self weight
\( P_{\text{Baldwin}} \) – Central concentrated applied load (equivalent to Baldwin load)

The maximum shear capacity of the failure section is taken as:

\[ V_{\text{max}} = P_{\text{baldwin}} / 2 + (d \times 0.9) \times 0.3 \times 2 \times 23.5 \]

Note failure section is taken at a distance \( d \) from the loading point.

The results, once obtained, were then reorganized to generate Figures 6-1 through Figure 6-4. It can be seen that generally, the results from this non-linear finite element analysis program are reasonably accurate (also see table 6-2, 6-3).

6.1.2 Response2000

In order to trace the development of ultimate failure mechanism in SB2003/6, a separate, stage-wise calculation procedure was used.

The 3 loading levels corresponding to the steel bar yielding, steel bar strain hardening and the ultimate shear failure of the whole member, respectively, were obtained. As compared with the observations at the test, the behavior of member SB2003/6 is accurately predicted (See test descriptions for SB2003/6 in Chapter 5).

Maximum shear capacity of the failure section allowing self-weight is taken as:

\[ V_{\text{max}} = V_{\text{calculated}} - V_{\text{self-weight}} \]

Where \( V_{\text{self-weight}} \) is the self-weight of the member from central loading point to the section considered failing from Response2000’s shear strain vs. span length diagram, this may give a more accurate prediction for the positioning of the failure section.
6.1.3 Calculation Based on CSA 23.3-94 Shear Provisions

Calculations based on generalized shear design method were also conducted in order to obtain a comprehensive comparison between various code shear design provisions. The detailed calculation and a summary of the calculating parameters are listed in the appendix B.

6.1.4 Calculation Based on ACI 318-99

See appendix B also.
6.2 Comparison between Response2000, Vector2, CSA and ACI

6.2.1 Graphical Overview of the Results

The three data points in figure 6-6 ~ figure 6-10 represent member SB2003/0&6, YB2000/0&6 and SB2012/0&6 respectively from leftmost rightwards. The reinforcement ratios are listed in table 6-4.

![ACI vs. Experimental results graph](image)

*ACI vs. Experimental results*

- Zero shear reinforcement (Exp)
- Min. shear reinforcement (Exp)
- ACI prediction(Zero shear rein.)
- ACI prediction(Min. shear rein.)
- ACI Bottom rebar flexural yielding

Longitudinal reinforcement ratio (%)

Figure 6-6
CSA vs. Experimental results

Figure 6-7

Response2000 vs. Experimental results

Figure 6-8
Vector2 vs. Experimental results

Zero shear reinforcement (Exp)
- Min. shear reinforcement (Exp)
- V2 prediction (Zero shear rein.)
- V2 prediction (Min. shear rein.)

Longitudinal reinforcement ratio (%)

Figure 6-9

Simplified equ(1),(2) vs. Experiment (Zero shear reinforcement)

Longitudinal reinforcement ratio (%)

Figure 6-10
6.2.2 Observations and Analysis

1. It can be seen from the above figures that the behavior of the member is largely influenced by its size, i.e., the differences observed between ACI predicted failure shear stresses, which excludes the size effect, and the predictions based on the MCFT, such as Response2000 or procedures related to the current CSA, which consider the size effect in an explicit manner, are evidently due to the size effect.

2. The greatest discrepancies between the ACI predictions and the test results happen for the lightly reinforced members. For the worst cases, the ratio of the actual experimental observation over this prediction reaches the surprisingly low values of 0.42 for SB2003/0 and 0.47 for YB2000/0 (see table 6-2). For the more heavily reinforced members, however, this discrepancy tends to be mitigated by the beneficial effects of large amount of longitudinal reinforcement. These phenomena are also captured explicitly in the CSA shear design procedures by incorporating $\varepsilon_x$ as a parameter in deciding proper $\theta$ and $\beta$ values.

Table 6-4.

<table>
<thead>
<tr>
<th>Member</th>
<th>$\nu/\sqrt{f_c}$</th>
<th>$\rho$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB2003/0</td>
<td>0.07</td>
<td>0.36</td>
</tr>
<tr>
<td>YB2000/0</td>
<td>0.08</td>
<td>0.74</td>
</tr>
<tr>
<td>SB2012/0</td>
<td>0.15</td>
<td>1.52</td>
</tr>
</tbody>
</table>

For lower portion curve

<table>
<thead>
<tr>
<th>Member</th>
<th>$\nu/\sqrt{f_c}$</th>
<th>$\rho$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB2003/6</td>
<td>0.12</td>
<td>0.36</td>
</tr>
<tr>
<td>YB2000/6</td>
<td>0.18</td>
<td>0.74</td>
</tr>
<tr>
<td>SB2012/6</td>
<td>0.25</td>
<td>1.52</td>
</tr>
</tbody>
</table>

For upper portion curve
Hence, the conclusion, at this point, is that heavily reinforced members are not so sensitive to the size effect as their lightly reinforced counterparts.

This observation gives rise directly to the notion that the longitudinal reinforcement ratio alongside with the member size (Se) are the two important factors affecting a structural component's sensitivity to the size effect. Hence, a simplified equation that is able to account for both of these factors in the prediction of the shear capacity of a structural component without or with less than the CSA required minimum amount of shear reinforcement is proposed. Further details in this regard are given in Chapter 7.

3. The presence of even the minimum amount of shear reinforcement \( \frac{A_v f_y}{b_w s} \) = 0.33 MPa will improve both the shear capacity and the ductility by a large margin. This increased shear capacity, as measured by the ratio of shear capacities between the span with transverse reinforcement and the span without such reinforcement for SB2003 and SB2012 are 1.61 and 1.63, respectively. The ratio showing the increasing ductility, likewise, is measured by the increasing mid-span deflection at failure, for SB2003 and SB2012, they are respectively 4.7 and 2.5 times. (Figure 6-1 to 6-4).

4. As can be seen clearly from Figure 6-6 that even with the presence of minimum shear reinforcement as required by the ACI shear provisions, the nominal shear capacity from test for specimen SB2003/6 still remains very low, even lower than the ACI shear strength prediction for SB2003/0. Similarly, the experimental shear strength for YB2000/6 is almost equal to the ACI prediction for YB2000/0. Evidently, this fact shows that at this minimum amount of shear reinforcement, the size effect will not be mitigated enough to be comfortably ignored, hence the part of the ACI shear formulation for the concrete contribution is still not qualified to be adopted here. On the other hand, if the CSA 94 minimum shear reinforcement formulation is invoked, a sharp yet expected difference in the amount of minimum shear reinforcement needed shows up as compared in table 6-5.
5. The shear capacity increment due to the presence of shear reinforcement varies even when the same amount of shear reinforcement is provided. The more closely spaced stirrups result in a higher increase in the shear capacity due to its better crack control ability\textsuperscript{15}.

6. From the above arguments, it is quite worthwhile mentioning that the MCFT based programs and shear calculation procedures, such as CSA are capable of providing reliable predictions for these large beams either with or without shear reinforcement. The trends are generally well predicted comparing to the test results.

It was also observed during the calculation with Vector2, that when the grid size is increased from 20x10 to 40x10, the final result is reduced by a magnitude of almost 15%. It can be seen that the increased degrees of freedom introduced by the extra elements reduce the stiffness of the calculation model, see figure below (figure 6-11).

\textit{Load vs. displacement for SB2003/0 by Vector2 using two different grid size (20x10 and 40x10)}

![Load vs. displacement graph](image)

\textbf{Figure 6-11.}
Table 6-1  Experimental Result for SB and YB Series

<table>
<thead>
<tr>
<th>Specimen</th>
<th>h  (mm)</th>
<th>bw (mm)</th>
<th>D  (mm)</th>
<th>fc'  (MPa)</th>
<th>Ax (mm²)</th>
<th>ρx (%)</th>
<th>s  (mm)</th>
<th>ρv fy (MPa)</th>
<th>a/d</th>
<th>Pexp (kN)</th>
<th>Vexp (kN)</th>
<th>δexp(max) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB2012/0</td>
<td>2000</td>
<td>300</td>
<td>1845</td>
<td>28.7/26.21</td>
<td>8400</td>
<td>1.52</td>
<td>~</td>
<td>~</td>
<td>2.92</td>
<td>758</td>
<td>402</td>
<td>13</td>
</tr>
<tr>
<td>SB2012/6</td>
<td>2000</td>
<td>300</td>
<td>1845</td>
<td>28.7/26.21</td>
<td>8400</td>
<td>1.52</td>
<td>1350</td>
<td>0.33</td>
<td>2.92</td>
<td>1225</td>
<td>635</td>
<td>28</td>
</tr>
<tr>
<td>SB2003/0</td>
<td>2000</td>
<td>300</td>
<td>1925</td>
<td>33.3/28.31</td>
<td>2100</td>
<td>0.36</td>
<td>~</td>
<td>~</td>
<td>2.81</td>
<td>399</td>
<td>224</td>
<td>14</td>
</tr>
<tr>
<td>SB2003/6</td>
<td>2000</td>
<td>300</td>
<td>1925</td>
<td>33.3/28.31</td>
<td>2100</td>
<td>0.36</td>
<td>1350</td>
<td>0.33</td>
<td>2.81</td>
<td>652</td>
<td>350</td>
<td>66</td>
</tr>
<tr>
<td>YB2000/0</td>
<td>2000</td>
<td>300</td>
<td>1890</td>
<td>35.4/31.8</td>
<td>4200</td>
<td>0.74</td>
<td>~</td>
<td>~</td>
<td>2.86</td>
<td>461</td>
<td>254</td>
<td>8</td>
</tr>
<tr>
<td>YB2000/6</td>
<td>2000</td>
<td>300</td>
<td>1890</td>
<td>37.3/34.5</td>
<td>4200</td>
<td>0.74</td>
<td>1350</td>
<td>0.33</td>
<td>2.86</td>
<td>1053</td>
<td>550</td>
<td>34</td>
</tr>
</tbody>
</table>

Note:  
1. First batch concrete cylinder strength / second batch concrete cylinder strength  
2. S- Spacing of T heads.  
3. A/d- Half span depth ratio  
4. δ- Mid-span displacement.
Table 6-2 Comparison between Different Code Provisions and Test Results for Members Without Shear Reinforcement

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d</th>
<th>f'c</th>
<th>(\rho_v f_y)</th>
<th>P_{exp}</th>
<th>V_{exp}</th>
<th>V_{R2K}</th>
<th>V_{vector2}</th>
<th>V_{ACI}</th>
<th>V_{CSA}</th>
<th>(V_{exp}/V_{R2K})</th>
<th>(V_{exp}/V_{vector2})</th>
<th>(V_{exp}/V_{ACI})</th>
<th>(V_{exp}/V_{CSA})</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB2012/0</td>
<td>1845</td>
<td>27</td>
<td>~</td>
<td>758</td>
<td>402</td>
<td>284</td>
<td>293</td>
<td>480</td>
<td>337</td>
<td>1.42</td>
<td>1.37</td>
<td>0.84</td>
<td>1.19</td>
</tr>
<tr>
<td>SB2003/0</td>
<td>1925</td>
<td>31</td>
<td>~</td>
<td>399</td>
<td>224</td>
<td>199</td>
<td>218</td>
<td>537</td>
<td>230</td>
<td>1.13</td>
<td>1.03</td>
<td>0.42</td>
<td>0.97</td>
</tr>
<tr>
<td>YB2000/0</td>
<td>1890</td>
<td>34</td>
<td>~</td>
<td>461</td>
<td>254</td>
<td>250</td>
<td>244</td>
<td>548</td>
<td>294</td>
<td>1.02</td>
<td>1.04</td>
<td>0.47</td>
<td>0.87</td>
</tr>
</tbody>
</table>

Note:  
1. \(\delta\) - Middle span displacement.  
2. R2K- Response2000

Table 6-3 Comparison in Members Containing Minimum Amount Required Shear Reinforcement

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d</th>
<th>f'c</th>
<th>(\rho_v f_y)</th>
<th>P_{exp}</th>
<th>V_{exp}</th>
<th>V_{R2K}</th>
<th>V_{vector2}</th>
<th>V_{ACI}</th>
<th>V_{CSA}</th>
<th>(V_{exp}/V_{R2K})</th>
<th>(V_{exp}/V_{vector2})</th>
<th>(V_{exp}/V_{ACI})</th>
<th>(V_{exp}/V_{CSA})</th>
</tr>
</thead>
<tbody>
<tr>
<td>SB2012/6</td>
<td>1845</td>
<td>27</td>
<td>0.33</td>
<td>1225</td>
<td>635</td>
<td>673</td>
<td>674</td>
<td>667</td>
<td>650</td>
<td>0.94</td>
<td>0.94</td>
<td>0.95</td>
<td>0.98</td>
</tr>
<tr>
<td>SB2003/6</td>
<td>1925</td>
<td>31</td>
<td>0.33</td>
<td>651</td>
<td>350</td>
<td>322</td>
<td>362</td>
<td>733</td>
<td>366</td>
<td>1.09</td>
<td>0.97</td>
<td>0.48</td>
<td>0.96</td>
</tr>
<tr>
<td>YB2000/6</td>
<td>1890</td>
<td>36</td>
<td>0.33</td>
<td>1053</td>
<td>550</td>
<td>548</td>
<td>614</td>
<td>753</td>
<td>641</td>
<td>1.00</td>
<td>0.90</td>
<td>0.73</td>
<td>0.86</td>
</tr>
</tbody>
</table>

Note:  
1. \(V_{R2K}\) value for SB2012/6 is a revised response2000 prediction.
Table 6-5. Minimum Shear Reinforcement as Required by ACI and CSA

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f_v$ (MPa)</th>
<th>$f_{c'}$ (MPa)</th>
<th>$s$ (mm)</th>
<th>ACI min. shear reinforcement area needed per 1350mm spacing (As, mm$^2$)</th>
<th>CSA 94 min. shear reinforcement area per 1350mm spacing (As, mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>YB2000/6</td>
<td>465</td>
<td>37.3/34.5</td>
<td>1350</td>
<td>284</td>
<td>434</td>
</tr>
<tr>
<td>SB2003/6</td>
<td>483</td>
<td>31</td>
<td>1350</td>
<td>284</td>
<td>389</td>
</tr>
</tbody>
</table>

Table 6-6. Influence of the Shear Reinforcement Spacing

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$H$ (mm)</th>
<th>$D$ (mm)</th>
<th>$f_{c'}$ (MPa)</th>
<th>$A_x$ (mm$^2$)</th>
<th>$\rho_x$ (%)</th>
<th>$\rho_v f_y$ (MPa)</th>
<th>$s$ (mm)</th>
<th>$P_{exp}$ (kN)</th>
<th>$\delta_{exp(max)}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>YB2000/4</td>
<td>2000</td>
<td>1890</td>
<td>35.4/31.8</td>
<td>4200</td>
<td>0.74</td>
<td>0.33</td>
<td>590</td>
<td>1300</td>
<td>Ductile failure</td>
</tr>
<tr>
<td>YB2000/6</td>
<td>2000</td>
<td>1890</td>
<td>37.3/34.5</td>
<td>4200</td>
<td>0.74</td>
<td>0.33</td>
<td>1350</td>
<td>1053</td>
<td>34</td>
</tr>
</tbody>
</table>
7. Effect of Longitudinal Reinforcement Ratio on the Shear Capacity of Structural Members without Shear Reinforcement

7.1 Background and Layout of Work

As a necessary result from MCFT, the size effect will be influenced not only by the $s_e$ parameter, but also by the longitudinal reinforcement ratio. The presence of this reinforcement restrains opening of the cracks and so enhances the shear interlock mechanism, which relies heavily on the crack width. This is clearly shown in the following equation.

$$
\eta = \frac{M}{jd} + 0.5N_e + 0.5V_u \cot \theta - A_n f_u
\]

$$

The dowel effect will also help towards an enhanced shear capacity for a cracked member, however, this effect is often limited by the ability of the concrete to resist longitudinal splitting along the rebar which usually is very low. Hence, in examining the influence of the longitudinal reinforcement ratio on the overall shear capacity of a cracked member, this effect is omitted.

The layout of the work is as following:

1. Non-linear sectional analysis program (Response 2000) was used on a specifically selected group of members to generate referencing data series for the possible format of relations between the testing parameter and the member’s ultimate shear strength.
2. Data filtering and curve fitting were then performed to decide on an appropriate relationship.
3. The candidate relationship was then calibrated against some major test results.
7.2 Generation of the Reference Data

Response 2000, as summarized in the previous few sections, is reasonably accurate in giving consistent reliable predictions while being easy to use. Hence, the program is chosen to perform the calculations.

7.2.1 Data Generation/Member Selection Criteria

The two criteria in selecting the sample members for the calculation are:
1. Span to depth ratio must be properly chosen so that the longitudinal reinforcement in most of the members calculated won't yield before they fail in shear.
2. The data set must include enough big members to allow for the size effect.

Based on the above two requirements, the beam series were so chosen to have a span to depth ratio of between 2.86 and 4. It was found that although larger a/d value may reduces the shear capacity of the calculated member, the more reduced shear capacity may not be suitable to be included into the selected data set. The yielding of longitudinal reinforcement usually precedes the shear failure, resulting in a post flexure failure mode. Hence this flexure strength, usually is well predicted in the design, will be lower than the maximum shear capacity.

Since the members will not be allowed to yield in their response history, the $f_y$ value will not be so critical, this parameter is then entered as 430Mpa.

The data were systematically generated following the above-mentioned criteria.

It was found then, after a careful examination of the results obtained from the above calculations that the relations linking the shear capacity and $\rho$ is most likely to be in the order of a cube root.

Note all the data points examined were sifted to allow for only brittle shear failure with all other post flexure shear failure modes ruled out.
7.2.2 Curve Fitting

All the above-generated data were then reorganized and sorted into a series of ordered value sets. A chart with $\rho$ as X-axis, $\beta = \frac{V}{bwdv \sqrt{fc'}}$ as Y-axis was generated, 12 series of different Se group the different trend lines that simultaneously shows the influence of Se in a single $\rho$-\(\beta\) figure. A curve fitting was performed later on this figure, as already been expected, the $\rho$'s influence is not that straightforward as $s_e$ in affecting the shear capacity of the member. Hence, a relation with $\rho$ under the cube-root was proposed and proved to be sufficient in tracing most part of the curve. As under a $\rho$ of 0.004, it was found that even under a half span to depth ratio of 2.86, some small members in the data series were still prone to fail in post flexure manner. The value of 0.004, quite accidentally, coincides with the minimum required flexural reinforcement ratio for the flexural members like slabs as required in the current ACI 99 code. Hence, it was decided that when $\rho$ is equal to 0.004, the equation should yield a $V_c$ equal to equation

$$\beta = \frac{245}{1275 + s_e} \quad \text{(1) [mm and Mpa units]}$$

(Collins, ACI Structural Journal/July-August 1999) predicts. Equation 1 is shown in figure 7-1.

Finally, the equations, as assembled in the above mentioned manner, are:

$$\beta = \frac{1150\sqrt{\rho}}{1275 + s_e} \quad \text{[For } \rho <= 0.015, \text{ if } \rho > 0.015, \rho = 0.015] \quad \text{(2)}$$

$$\beta = \frac{1098\sqrt{\rho} \times s_e^{-0.04}}{1275 + s_e} \quad \text{[For } \rho <= 0.015, \text{ if } \rho > 0.015, \rho = 0.015] \quad \text{(3)}$$

For $\rho = 0.028$, $\beta$ would be 1.23 times greater if limit were not there.
In equation (2) and (3), \( \beta \) is defined as

\[
\beta = \frac{V}{b_w d \sqrt{f'_e}}
\]

Where, \( s_e = \frac{35 \times s_x}{(a + 16)} \) [mm units], and

\[
\beta = \frac{V_c}{b_w d \sqrt{f'_c}}
\]

However, as can be seen in the data correlation table that is attached at the end of this chapter (table 7-1), equation (3) generally gives better predictions than equation (2) statistically. Trend lines for equation 2,3 are illustrated in figure 7-4,7-3, respectively.

7.2.3 Calibrating Data to the Valid Test Results

The factor of \( \text{C} \times \text{Se}^{-0.04} \), actually, was introduced in this step. It was found, upon checking equation 2 with the experimental data, that the predictions tend to overestimate the shear capacity of the real member by giving too much percentage to the influence from the reinforcement in low Se region. The prediction usually goes well in complying with the test results in high Se regions, the demarcation region, lies somewhere between Se of 1000mm to 2000mm. Hence a formulation (equation 4), which gives a correction factor showing larger reduction effect in low Se region while tapers to give no reduction in high Se regions, was introduced into equation 2. The trend line of this corrective formulation is show in figure 7-1.

\[
\text{Correction factor} = 1.4 \, s_{\text{mxe}}^{-0.04} \quad \text{(4)}
\]

The predictions by equation (3), is show in figure 7-3 in large red hollow circles, it was found that this equation fits well to the experiments.
Equation (3) gives an average for $\beta_{test}/\beta_{equ(3)}$ of 1.04 and a standard deviation about 11%, the correlation factor between the experimental results and the generated data set is about 97%. See lists in table 7-1.

Since the data generated were from those with medium strength normal weight concrete, limitation on the strength of the concrete should be observed besides the other necessary conditions that had been laid out in the previous sections.

Note for convenience, table 7-1 listed part of the data used in this research.
Factor: 1.4*Se^(-0.04)

Figure 7-1 Correction Factor

Equation predictions and its correlation with tests

Figure 7-2 Comparison between Equ 1. and Experiments
Equation predictions and its correlation with tests

Figure 7-3 Comparison between Equ 3. and Experiments

Equation predictions and its correlation with tests

Figure 7-4. Comparison between Equ 2. with Experiments
<table>
<thead>
<tr>
<th>Name</th>
<th>$dv$</th>
<th>$\rho$ (%)</th>
<th>$f_r$</th>
<th>$s_o$ (mm)</th>
<th>$a$ (mm)</th>
<th>$a/d$</th>
<th>Test $\beta$</th>
<th>Equ.(1) $\beta$</th>
<th>Equ.(2) $\beta$</th>
<th>Equ.(3) $\beta$</th>
<th>Test/Equ(1)</th>
<th>Test/Equ(2)</th>
<th>Test/Equ(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BN100</td>
<td>925</td>
<td>0.76</td>
<td>37.2</td>
<td>1121</td>
<td>10</td>
<td>2.92</td>
<td>0.113</td>
<td>0.102</td>
<td>0.094</td>
<td>0.119</td>
<td>1.105</td>
<td>1.197</td>
<td>0.947</td>
</tr>
<tr>
<td>B100</td>
<td>925</td>
<td>1.01</td>
<td>36</td>
<td>1120</td>
<td>10</td>
<td>2.92</td>
<td>0.135</td>
<td>0.102</td>
<td>0.104</td>
<td>0.131</td>
<td>1.320</td>
<td>1.301</td>
<td>1.029</td>
</tr>
<tr>
<td>B100L-R</td>
<td>925</td>
<td>1.01</td>
<td>36</td>
<td>1120</td>
<td>10</td>
<td>2.92</td>
<td>0.136</td>
<td>0.102</td>
<td>0.104</td>
<td>0.131</td>
<td>1.329</td>
<td>1.310</td>
<td>1.036</td>
</tr>
<tr>
<td>BN25</td>
<td>225</td>
<td>0.89</td>
<td>37</td>
<td>273</td>
<td>10</td>
<td>3.00</td>
<td>0.177</td>
<td>0.158</td>
<td>0.154</td>
<td>0.184</td>
<td>1.118</td>
<td>1.150</td>
<td>0.962</td>
</tr>
<tr>
<td>BN12.5</td>
<td>110</td>
<td>0.91</td>
<td>37</td>
<td>133</td>
<td>10</td>
<td>3.07</td>
<td>0.199</td>
<td>0.174</td>
<td>0.171</td>
<td>0.198</td>
<td>1.144</td>
<td>1.167</td>
<td>1.005</td>
</tr>
<tr>
<td>Shioya</td>
<td>2000</td>
<td>0.40</td>
<td>28.5</td>
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**Remark:**

```
Average    1.284        1.296        1.046
STDEV      0.26         0.16         0.11
CORREL     0.88         0.97         0.97
```

A ----- Aggregate size

$s_{mx} = s_e \times 35/(a+16)$
8. Conclusions and Recommendations:

8.1 Conclusions

1. Shear capacities evaluated by ignoring the size effect can be very unconservative for large, lightly reinforced structural members. The shear strength estimated by the ACI expressions for some structures can be more than twice as large as the actual shear strength.

2. The addition of even a small amount of shear reinforcement greatly enhances the shear capacity as well as the ductility of such large structural members.

3. The ACI required minimum amount of shear reinforcement may not be enough to eliminate the size effect, in this situation, the use of its conventional equation in predicting shear strength provided by the concrete can be very unsafe.

3. The size effect will be strongly influenced not only by the member’s size – or more precisely, the crack spacing parameter of the reinforced concrete but also by the amount of the longitudinal flexural reinforcement.

4. The proposed simplified equations (Equ. 1, 2, 3) for the current ACI allows for the fact mentioned above and are able to capture these primary features in a reasonable manner hence are considerably more reliable if adopted in the design.

5. Methods based on the Modified Compression Field Theory (MCFT), such as Response2000, Trix98, 99 or Vector2 and CSA-A23.3-94 are reasonably accurate as evidenced in the experiments.
8.2 Recommendations

If a reasonably high minimum amount of shear reinforcement is specified, the size effect in shear will be negligible. However, as ACI required 0.33MPa doesn’t demonstrate this effect as observed in the tests, experiments conducted with the CSA required minimum amount of shear reinforcement in large structural members as a comparison with the current ACI are suggested. Other major influencing parameters such as the concrete strength, reinforcement bonding characters should also be investigated.
References

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-99) and Commentary ACI 318 R-99", American Concrete Institute, Detroit, 1999
6. Frank J. Vecchio and Michael P. Collins "Predicting the Response of Reinforced Concrete Beams Subjected to Shear Using Modified Compression Field Theory" ACI STRUCTURAL JOURNAL May-June 1988
15. Yoichi Yoshida "Size Effect of the Large, Lightly Reinforced Concrete Members", M.A.Sc thesis, Department of Civil Engineering, University of Toronto
Appendix A

Part 1: Available Models in Vector2
[As of Apr 25, 2000]

Structure Type:
1. Beam Section (2-D)
2. Plane Membrane (2-D)
3. Solid (3-D)
4. Shell
5. Plane Frame (2-D)
6. Space Frame (3-D)
7. Axisymmetric Solid
8. Axisymmetric Shell
9. Mixed Type

Concrete Compression Pre-Peak Response:
0. Linear
1. Nonlinear - Hognestad (Parabola)
2. Nonlinear - Popovics (High Strength)
3. Nonlinear - Hoshikuma Et Al

Concrete Compression Post-Peak Response:
0. Base Curve
1. Modified Park-Kent
2. Popovics
3. Hoshikuma Et Al

Concrete Compression Softening Model:
0. No compression softening
1. Vecchio 1992-A (e1/e2-Form)
2. Vecchio 1992-B (e1/e0-Form)
3. Vecchio-Collins 1982
4. Vecchio-Collins 1986

Concrete Tension Stiffening Model:
0. No tension stiffening
1. Modified Bentz
2. Vecchio 1982
4. Bentz 1999
5. Izumo, Maekawa Et Al
Concrete Tension Softening:
0. Not Considered
1. Linear - No Residual
2. Linear - w/ Residual
3. Residual Only (10%)
4. Yamamoto 1999

Concrete Tension Splitting:
1. Not Considered
2. DeRoo 1995

Concrete Confinement Strength:
0. Strength enhancement neglected
1. Kupfer / Richart Model
2. Selby Model

Concrete Lateral Expansion:
0. Constant Poisson's ratio
1. Variable Poisson's ratio

Concrete Cracking Criterion:
0. Uniaxial cracking stress
1. Mohr-Coulomb (Stress)
2. Mohr-Coulomb (Strain)
3. CEB-FIP Model
4. Gupta 1998 Model

Concrete Crack Slip Check:
0. Crack shear check omitted
1. Vecchio-Collins 1986
2. Gupta 1998 Model

Concrete Crack Width Check:
0. Stability check omitted
1. Check based on 5 mm max crack width
3. Check based on 2 mm max crack width

Concrete Hysteretic Response:
0. No plastic offsets
1. Plastic offsets; linear loading/unloading
2. Plastic offsets; nonlinear loading/unloading
3. Plastic offsets; nonlinear w/ cyclic decay
4. Mander Model - Version 1
5. Mander Model - Version 2
Reinforcement Hysteretic Response:
  0. Linear
  1. Elastic-Plastic
  2. Elastic-Plastic w/ Hardening
  3. Seckin Model w/ Bauschinger Effect

Element Strain Histories:
  0. Previous loading neglected
  1. Previous loading considered

Element Slip Distortion:
  0. Not considered
  1. Stress Model (Walraven)
  2. Stress Model (Maekawa)
  3. Stress Model (Vecchio/Lai)
  4. Hybrid-I Model
  5. Hybrid-II Model
  6. Hybrid-III Model
  7. Rotation lag of 5 degrees
  8. Rotation lag of 7.5 degrees
  9. Rotation lag of 10 degrees
  10. Rotation lag of 15 degrees

Convergence Criteria:
  1. Secant Moduli - Weighted Average
  2. Displacements - Weighted Average
  3. Displacements - Maximum Value
  4. Reactions - Weighted Average
  3. Reactions - Maximum Value

Results File Storage:
  1. ASCII and binary files
  2. ASCII files only
  3. Binary files only
  4. Last load stage only
Part 2: Vector2 Structure Input File for SB2003/0&6, SB2012/0&6
**SB2003/0 Vector2 Structure File**

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* D A T A            *                    
* * * * * * * * * * * * * * * * * * * * *

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No. of Rectangular elements : 400
No. of Triangular elements : 0
No. of Truss elements : 0
No. of Joints : 451
No. of Restraints : 12

MATERIAL SPECIFICATIONS
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<NOTE:> TO BE USED IN RECTANGULAR AND TRIANGULAR ELEMENTS ONLY

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(B) STEEL
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<NOTE:> TO BE USED FOR TRUSS ELEMENTS ONLY

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COORDINATES
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**SB2003/6 Vector2 Structure File**

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No. of Joints : 451
No. of Restraints : 12

MATERIAL SPECIFICATIONS
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<NOTE:> TO BE USED IN RECTANGULAR AND TRIANGULAR ELEMENTS ONLY

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<NOTE:> TO BE USED FOR TRUSS ELEMENTS ONLY

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(B) STEEL
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<NOTE:> TO BE USED FOR TRUSS ELEMENTS ONLY

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COORDINATES
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No. of Restraints : 12

**MATERIAL SPECIFICATIONS**

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(A) REINFORCED CONCRETE

<NOTE:> TO BE USED IN RECTANGULAR AND TRIANGULAR ELEMENTS ONLY

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/  REINFORCEMENT COMPONENTS  L o o k  ! ! ! ---->

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/  (B) STEEL

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(B) TRIANGULAR ELEMENTS
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COORDINATES
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SUPPORT RESTRAINTS
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128
**SB2012/6 Vector2 Structure File**

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* S T R U C T U R E *
* D A T A *
* * * * * * * * * * * *

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No. of Rectangular elements : 400
No. of Triangular elements : 0
No. of Truss elements : 0
No. of Joints : 451
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MATERIAL SPECIFICATIONS
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<NOTE:> TO BE USED IN RECTANGULAR AND TRIANGULAR ELEMENTS ONLY

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(B) STEEL
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<NOTE:> TO BE USED FOR TRUSS ELEMENTS ONLY

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ELEMENT INCIDENCES

(A) RECTANGULAR ELEMENTS

(B) TRIANGULAR ELEMENTS

(C) TRUSS ELEMENTS

MATERIAL TYPE ASSIGNMENT

COORDINATES

SUPPORT RESTRAINTS
Appendix B

Calculations of ACI 318-99 and CSA-A23.3-94 (General Method) Code Predictions
Shear Load Calculated from CSA-A23.3-94 (General Method)

Sample Calculation for Specimen SB2003/0

Concrete Strength: $f_c'=31 \text{ Mpa}$  
Maximum Aggregate Size: 10mm  
Yield Strength of Longitudinal Rebars: 436 Mpa  
Shear Span Length: $a=5.4 \text{mm}$  
Unit Weight of Concrete: 23.5 kN/m$^3$

1. Find the flexural lever arm, $d_v$

$$
\alpha_1 = 0.85-0.0015 \times f_c' = 0.85-0.0015 \times 31 = 0.8 \geq 0.67
$$

$$
d_v = d - \frac{a}{2} = 1925 - \frac{3 \times 700 \times 436 / (0.8 \times 31 \times 300)}{2} = 1863 \text{mm}
$$

2. Find dead load effects

$$
V_D = 23.5 \times 0.3 \times 2 \times 1.863 = 26.3 kN
$$

$$
M_D = 23.5 \times 0.3 \times 2 \times 6 \times (5.4 - 1.863) - \frac{23.5 \times 0.3 \times 2 \times (5.4 - 1.863)^2}{2} = 299.2 - 88.2 = 211 kN
$$

3. Calculate the equivalent crack spacing parameter, $s_{ze}$.

$$
s_z = d_v = 1863 \text{mm}
$$

$$
s_{ze} = s_z \frac{35}{a + 16} = 1.863 \times \frac{35}{10 + 16} = 2.51 \text{m}
$$

Hence, $s_{ze} > 2000 \text{ mm}$, $s_{ze} = 2000 \text{ mm}$

4. Assume $\epsilon_X = 0.0010$, from table 2-2, $\theta = 66^\circ$, $\beta = 0.084$

$$
V_u = \beta \sqrt{f_c' b_w d_v} = 0.084 \sqrt{31 \times 300 \times 1863} / 1000 = 261 kN
$$

Hence,

$$
V_p = V_u - V_D = 261 - 26.3 = 235 kN
$$

$$
M_u = M_D + M_p = M_D + V_p (a - d_v) = 211 + 235(5400 - 1863) = 831 kN.m
$$
5. The longitudinal strain at the rebar level is then estimated at:

\[
\varepsilon_x = \frac{M_u}{E_s A_s} + \frac{0.5V_u \cot \theta}{d_v} = \frac{831 \times 10^6 / 1863 + 0.5 \times 261 \times \cot 66}{200000 \times 2100} = 0.0011
\]

\( >0.0010 \) , a bigger \( \varepsilon_x \) value hence is imperative.

6. Re-assume \( \varepsilon_x = 0.0015 \), from table 2-2, \( \theta = 69^\circ \), \( \beta = 0.064 \)

Repeat steps 4 and 5, \( V_p = 173 \) kN, \( M_p = 823 \) kN.m

\( \varepsilon_x = 0.00105 < 0.0015 \), O.K.

the resultant shear capacity is then \( V_u = 200 \) kN.

However, In order to obtain a somewhat more accurate prediction about the ultimate shear capacity of SB2003/0 for the convenience of comparison, a linear interpolation was used to select a closer \( \theta \) and \( \beta \) value. Assume \( \theta = 67.5 \) then \( \beta = 0.074 \),

\[
V_u = 0.074 \times \sqrt{31 \times 300 \times 1863 / 1000} = 230 kN
\]

\( V_p = 203.7 \) kN, \( M_u = 931 \) kN.m. Finally, check the \( \varepsilon_x \) value, which is calculated at \( 0.00119 < 0.00125 \), O.K. It can then be seem that 0.00119 comes quite close to the presumed \( \varepsilon_x 0.00125 \). Hence the result obtained here is deemed to be accurate enough.

Finally, the ultimate shear capacity as calculated by CSA for SB2003/0 is:

\[
V_u = 230 kN
\]
Sample Calculation for Specimen SB2003/6

Concrete Strength: $f_{c} = 31$ Mpa
Maximum Aggregate Size: 10mm
Yield Strength of Longitudinal Rebars: 436 Mpa
Yield Strength of Transverse Rebars: 483 Mpa
Shear Span Length: $a = 5.4$ mm
Unit Weight of Concrete: 23.5 kN/m$^3$

1. Find the flexural lever arm, $d_v$

$$\alpha_1 = 0.85 - 0.0015 f_{c}' = 0.85 - 0.0015 \times 31 = 0.8 \geq 0.67$$
$$d_v = \frac{d - a}{2} = 1925 - \frac{3 \times 700 \times 436/(0.8 \times 31 \times 300)}{2} = 1863 \text{mm}$$

2. Find dead load effects

$$V_D = 23.5 \times 0.3 \times 2 \times 1.863 = 26.3kN$$
$$M_D = 23.5 \times 0.3 \times 2 \times 6 \times (5.4 - 1.863) - \frac{23.5 \times 0.3 \times 2 \times (5.4 - 1.863)^2}{2} = 299.2 - 88.2 = 211kN$$

3. Calculate the equivalent crack spacing parameter, $s_{zc}$.

$$s_z = d_v = 1863 \text{mm}$$
$$s_{zc} = s_{z} \frac{35}{a + 16} = 1.863 \times \frac{35}{10 + 16} = 2.51m$$

Hence, $s_{zc} > 2000$ mm, $s_{zc} = 2000$ mm

4. Estimate the shear capacity

$$V_u = \beta \sqrt{f_{c}^{'}} b_w d_v + \frac{A_v f_y d_v}{s} \cot \theta$$

$$= \beta \sqrt{31 \times 300 \times 1863/1000} + \frac{284 \times 483 \times 1863 \times \cot \theta}{1000}$$

$$= 3112 \beta + 189 \cot \theta$$
Then, Choose $\beta$ and $\theta$ from table 2.3

First, Assume $\frac{v}{f'_{c}} \leq 0.050, \quad \varepsilon_{x} \leq 0.0020$

Then from table 2-2, $\theta = 43^\circ, \beta = 0.143$

$V_{u} = 3112 \times 0.143 + 189 \cot 43 = 648kN$

$V_{p} = V_{u} - V_{D} = 648 - 26.3 = 622kN$

$M_{u} = M_{D} + V_{p}(a - d_{v}) = 211 + 622 \times (5.4 - 1.863) = 2410kN.m$

$\varepsilon_{x} = \frac{M_{u}/d_{v} + 0.5V_{u} \cot \theta}{E_{s}A_{s}} = \frac{2410 \times 10^{6}/1863 + 0.5 \times 648 \times \cot 43}{200000 \times 2100} = 0.0031 >> 0.002$

Not satisfactory, $\varepsilon_{x}$ has to be larger.

5. Try $\varepsilon_{x} = 0.005$, from table 2-2, $\theta = 56^\circ, \beta = 0.077$

Repeat step 4,

$V_{u} = 3112 \times 0.077 + 189 \cot 56 = 366kN$

$V_{p} = V_{u} - V_{D} = 366 - 26.3 = 340kN$

$M_{u} = M_{D} + V_{p}(a - d_{v}) = 211 + 340 \times (5.4 - 1.863) = 1413kN.m$

$\varepsilon_{x} = \frac{M_{u}/d_{v} + 0.5V_{u} \cot \theta}{E_{s}A_{s}} = \frac{1413 \times 10^{6}/1863 + 0.5 \times 366 \times \cot 56}{200000 \times 2100} = 0.0018 < 0.005$

Next, check if $\frac{v}{f'_{c}} \leq 0.050$

$\frac{v}{f'_{c}} = \frac{366 \times 1000}{300 \times 1863 \times 31} = 0.021 < 0.05$

Also satisfactory, Hence the assumed values for $\theta$ and $\beta$ are found to be O.K.

Finally, the shear capacity for beam SB2003/6 is calculated at: $V_{u} = 366kN$.  

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Summary of the calculations for specimen in SB series

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Shear Capacity Prediction Using ACI 318-99

Sample Calculation for Specimen SB2003/0 and SB2003/6

For SB2003/0:
Concrete strength: $f'_c = 31\text{Mpa}$

$$V = V_u = 0.167 \sqrt{f'_c b_w d} = 0.167 \times \sqrt{31} \times 300 \times 1925 = 537\text{kN}$$

Therefore, ACI predicts an ultimate shear capacity for SB2003/0 of about 537 kN.

For SB2003/6:
Concrete strength: 31 Mpa
Yield strength of transverse rebars: 483 Mpa
Cross section area of the transverse rebar: 284 mm$^2$
Transverse reinforcement spacing: 1350 mm

$$V = V_u = 0.167 \sqrt{f'_c b_w d} + \frac{A_v f_y d}{s}$$

$$= 0.167 \times \sqrt{31} \times 300 \times 1925 + \frac{284 \times 483 \times 1925}{1350}$$

$$= 537 + 196$$

$$= 733\text{kN}$$

Therefore, ACI prediction for the shear capacity of SB2003/6 is 733 kN.
Appendix C

Experiment Output

SB2003 / 0
SB2003 / 6
SB2012 / 0
SB2012 / 6
Specimen SB2003 / 0

SB2003/0 - Load vs. Deflection

SB2003/0 - Deflection Distribution

Distance From the Midspan (mm)
SB2003/0 - Load vs. Longitudinal Reinforcement Strain
[E series, SB2003/0 End]

SB2003/0 - Load vs. Longitudinal Reinforcement Strain
[E series, SB2003/6 End]
Specimen: SB2003/0   Load Stage: 1   Applied Load: 133 kN

All Values are in mm/m
Specimen: SB2003/0  Load Stage: 3  Applied Load: 281 kN

All Values are in mm/m
Specimen SB2003/6

SB2003/6 - Load vs. Deflection

SB2003/6 - Deflection Distribution [SB2003/6 end]
SB2003/6-Transverse Reinforcement Strain Distribution at bottom

[SB2003/6 End]

Strain Gauge Readings (microstrain)

Distance from midsapn (mm)

---

SB2003/6-Transverse Reinforcement Strain Distribution at Middle depth [SB2003/6 End]

Strain Gauge Readings (microstrain)

Distance from midsapn (mm)
Specimen SB2012/0

SB2012/0 - Load vs. Deflection

SB2012/0 - Deflection Distribution
SB2012/6 End ---------------- SB2012/0 End
SB2012/0 - Shear force vs. Shear strain

Shear force (kN)

Shear strain (mm/m)

SB2012/0 - Shear force vs. Shear Strain.

Shear force (kN)

Shear strain (mm/m)
SB2012/0 - Load vs. Longitudinal Reinforcement Strain

[SB2012/0 End, top layer]

Load (kN) vs. Strain Gauge Reading (microstrain)

SB2012/0 - Load vs. Longitudinal Reinforcement Strain

[SB2012/6 End, Bottom layer]

Load (kN) vs. Strain Gauge Reading (microstrain)
SB2012/0 - Load vs. Longitudinal Reinforcement Strain
[SB2012/6 End, middle layer]

SB2012/0 - Load vs. Longitudinal Reinforcement Strain
[SB2012/6 End, top layer]
SB2012/0 - Longitudinal Reinforcement Strain Distribution
[Top layer rebar]

- P = 100 KN
- P = 200 KN
- P = 300 KN
- P = 400 KN
- P = 500 KN
- P = 600 KN
- P = 700 KN
- P = 758 KN

Strain Gauge Reading (microstrain) vs. Distance from mid-span (mm)

SB2012/0 - Longitudinal Reinforcement Strain Distribution
[Middle layer rebar]

- P = 100 KN
- P = 200 KN
- P = 300 KN
- P = 400 KN
- P = 500 KN
- P = 600 KN
- P = 700 KN
- P = 758 KN

Strain Gauge Reading (microstrain) vs. Distance from mid-span (mm)
SB2012/0 - Transverse Reinforcement Strain Distribution at Bottom [SB2012/6 End]

- P=200 kN
- P=400 kN
- P=500 kN
- P=600 kN
- P=700 kN
- P=760 kN

Strain Gauge Reading (microstrain)

Distance from midspan (mm)
Specimen SB2012/6

SB2012/6 - Load vs. Deflection

SB2012/6 Deflection Distribution
SB2012 - Shear Force vs. Shear Strain
[SB2012/6 End]

SB2012/6 - Load vs. Longitudinal Reinforcement Strain
[SB2012/6 End, bottom layer]
Strain Gauge Reading (microstrain)

SB2012/6 - Load vs. Transverse Reinforcement Strain

[SB2012/6 End]

Load (kN)

Strain Gauge Reading (microstrain)

SB2012/6 - Transverse Reinforcement Strain Distribution

at mid-depth [SB2012/6 End]

Distance from midspan (mm)

Strain Gauge Reading (microstrain)
SB2012/6-Transverse Reinforcement Strain Distribution at Bottom [SB2012/6 End]

- P=300 kN
- P=600 kN
- P=800 kN
- P=1000 kN
- P=1152 kN
- P=1225 kN

Distance from midspan (mm)

Strain Gauge Reading (microstra)
Specimen: SB2012/6  Load Stage: 4  Applied Load: 1000 kN

All Values are in mm/m