UNDERSTANDING AND SAFELY PREDICTING THE SHEAR RESPONSE OF LARGE-SCALE REINFORCED CONCRETE STRUCTURES

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

Infrastructure commonly built today incorporates very thick reinforced concrete members that serve as critical elements in the load-carrying system of the structure. Certain design codes specify that members such as mat foundations and transfer slabs in high-rise buildings can be designed without shear reinforcement, allowing for more economical constructability. Omission of shear reinforcement results in structures being susceptible to the well-established size effect, and thus prone to brittle shear failures. The research comprised in this thesis focuses on investigating the applicability of international design codes in predicting the shear strength of very thick concrete members. To this end, a specimen representing a 4 m thick slab strip and a 300 mm thick companion strip were constructed and tested to failure. Results from the experimental program support the recommendation to use at least minimum shear reinforcement in large-scale structures to yield safe designs.
ACKNOWLEDGEMENTS

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A project of this magnitude and complexity would not have been possible without the assistance from many different people in the various stages of construction, casting, experimentation, and demolition of the test specimens. Headed Reinforcement Canada (HRC) donated both the headed flexural tension and headed shear reinforcement along with mechanical couplers used for the longitudinal bars. Dufferin Construction, an operating division of CRH Canada Group Inc. (formerly Holcim), generously donated the approximately 21 cubic meters of concrete used in the cast. Aluma Systems supplied, erected, and dismantled the formwork using their practical pre-engineered modular form panels. Amherst Group performed the concrete cast using one of their very highly maneuverable boom pump trucks, the first ever used in the University of Toronto Mark Huggins Structural Testing Facilities. Ontario Cutting and Coring executed the demolition of the specimen using a diamond blade concrete wall saw, even making an extra cut for me so that I may keep a small sliver as a memento.

In addition to external contractors, it was the constant support from our very own highly skilled laboratory technicians at University of Toronto that allowed the project to come to fruition in a successful manner. I would like to thank those individuals as follows: Renzo Basset, Giovanni Buzzeo, John MacDonald, Xiaoming Sun, Bryant Cook, Michel Fiss, and Alan McClanaghan for helping me on countless different project tasks.
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Notes: All dimensions of length in figures/schematics are in units of millimeters unless noted otherwise.

SI/US Unit Conversions

1 kN ≈ 0.225 kips
1 mm ≈ 0.04 in
1 m ≈ 3.28 ft

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## Latin Symbols

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<th>Definition</th>
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<td>$a$</td>
<td>shear span</td>
</tr>
<tr>
<td>$a_g$</td>
<td>specified nominal maximum size of coarse aggregate</td>
</tr>
<tr>
<td>$A_b$</td>
<td>area of an individual reinforcing bar</td>
</tr>
<tr>
<td>$A_p$</td>
<td>area of prestressing tendons on the flexural tension side of a member</td>
</tr>
<tr>
<td>$A_s$</td>
<td>area of main flexural tension reinforcement</td>
</tr>
<tr>
<td>$A_s'$</td>
<td>area of main flexural compression reinforcement</td>
</tr>
<tr>
<td>$A_v$</td>
<td>area of transverse shear reinforcement</td>
</tr>
<tr>
<td>$b_w$</td>
<td>member web width</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>coefficient of variation of specified parameter, equal to $\sigma/\mu$</td>
</tr>
<tr>
<td>$d$</td>
<td>distance from extreme compression fibre to centroid of tension reinforcement</td>
</tr>
<tr>
<td>$d_v$</td>
<td>effective shear depth (flexural lever arm), taken as the greater of $0.9d$ or $0.72h$</td>
</tr>
<tr>
<td>$E_c$</td>
<td>modulus of elasticity of concrete</td>
</tr>
<tr>
<td>$E_p$</td>
<td>modulus of elasticity of prestressing tendons</td>
</tr>
<tr>
<td>$E_s$</td>
<td>modulus of elasticity of non-prestressed reinforcement</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>compressive strength of concrete</td>
</tr>
<tr>
<td>$f_{c,\text{max}}$</td>
<td>peak compressive strength of concrete cylinder</td>
</tr>
<tr>
<td>$f_{c,\text{ult}}$</td>
<td>stress of concrete at peak strain of cylinder test</td>
</tr>
<tr>
<td>$f_{cu}$</td>
<td>limiting compressive stress in concrete strut due to coexisting principal tensile strains (i.e. compression softened cylinder response)</td>
</tr>
<tr>
<td>$f_{po}$</td>
<td>stress in the prestressing tendons when strain in the surrounding concrete is zero (may be taken as 0.7 times the specified tensile strength of the tendons if bonded)</td>
</tr>
<tr>
<td>$f_s$</td>
<td>stress in reinforcement (tension positive)</td>
</tr>
<tr>
<td>$f_{s,\text{ult}}$</td>
<td>stress in reinforcement at peak strain of coupon test</td>
</tr>
<tr>
<td>$f_y$</td>
<td>yield strength of steel reinforcement</td>
</tr>
<tr>
<td>$F_{lt}$</td>
<td>tensile force demand in longitudinal reinforcement on flexural tension side of member</td>
</tr>
<tr>
<td>$h$</td>
<td>overall thickness or height of member</td>
</tr>
<tr>
<td>$h_{\text{avg}}$</td>
<td>average concrete cylinder height</td>
</tr>
<tr>
<td>$I_{cr}$</td>
<td>cracked section moment of inertia</td>
</tr>
<tr>
<td>$I_g$</td>
<td>gross section moment of inertia</td>
</tr>
<tr>
<td>$j_d$</td>
<td>flexural lever arm</td>
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</tbody>
</table>
LIST OF SYMBOLS

\( k \) = stiffness of load-deformation response

\( L \) = support-to-support member length

\( LS \) = load stage

\( M_f \) = factored moment

\( N_f \) = factored axial load normal to cross-section (tension positive)

\( P \) = applied point load

\( P_{unload} \) = applied point load after unloading from target load stage for crack measurements

\( s \) = spacing of shear reinforcement measured parallel to longitudinal axis of member; also the measured slip of a crack

\( s/w \) = self-weight

\( s_z \) = crack spacing parameter or characteristic crack spacing, taken as \( d_v \) or the maximum spacing between layers of skin reinforcement

\( s_{ze} \) = effective crack spacing parameter that allows for influence of aggregate size and crack control provided by well distributed reinforcement

\( S_1 \) = concrete stress corresponding to a longitudinal strain, \( \varepsilon_1 \), of 0.00005 (ASTM C 469)

\( S_2 \) = concrete stress corresponding to 40\% of \( f'_c \) (ASTM C 469)

\( S_x \) = longitudinal crack spacing measured from experiments

\( S_{x,crit} \) = longitudinal crack spacing between main diagonal failure crack and adjacent crack

\( v \) = shear stress

\( v_c \) = shear stress resistance provided by the concrete

\( v_s \) = shear stress resistance provided by shear reinforcement

\( V \) = shear force

\( V_c \) = shear resistance attributed to concrete

\( V_p \) = shear resistance offered by the vertical component of any effective prestressing force (i.e. by means of clamping stresses)

\( V_s \) = shear resistance attributed to shear reinforcement

\( w \) = crack width
**Greek Symbols**

- $\beta$ = factor accounting for shear resistance of cracked concrete (i.e. aggregate interlock factor)
- $\gamma_{xy}$ = shear strain
- $\Delta$ = deflection (downwards positive if vertical)
- $\Delta_{unload}$ = vertical deflection after unloading from target load stage for crack measurements
- $\varepsilon'_c$ = concrete strain at peak compressive stress
- $\varepsilon_{c, ult}$ = concrete strain at peak measured displacement of cylinder test
- $\varepsilon_s$ = strain in steel reinforcement (tension positive)
- $\varepsilon_{s, ult}$ = steel strain at peak strain of coupon test
- $\varepsilon_{sh}$ = strain at which strain-hardening commences in reinforcing steel response
- $\varepsilon_x$ = longitudinal strain at mid-depth of member due to applied load effects (tension positive)
- $\varepsilon_y$ = yield strain of reinforcement
- $\varepsilon_1$ = principal tensile strain in cracked concrete due to applied loads; principal compressive strain used in ASTM C 469 calculation of $E_c$
- $\theta$ = angle of inclination of diagonal compressive stresses to the longitudinal axis of the member; measured crack angle from the horizontal
- $\theta_s$ = smallest angle between concrete compressive strut and adjoining reinforcement tensile ties in strut-and-tie models
- $\lambda$ = factor to account for low-density concrete, equal to unity in this thesis
- $\mu$ = statistical average of specified parameter
- $\rho$ = concrete density
- $\rho_x$ = longitudinal tension reinforcement ratio, equal to $A_s/bd$
- $\rho_x'$ = longitudinal compression reinforcement ratio, equal to $A_s'/bd$
- $\rho_z$ = transverse reinforcement ratio, equal to $A_v/b_wd_v$
- $\sigma$ = sample standard deviation of specified parameter
- $\phi_{avg}$ = average diameter of concrete cylinder
- $\phi_c$ = material resistance factor for concrete, equal to 0.65, but taken as unity in this thesis owing to material strengths being known from testing
- $\phi_s$ = material resistance factor for non-prestressed reinforcement, equal to 0.85, but taken as unity in this thesis owing to material strengths being known
- $\omega$ = uniformly distributed load
CHAPTER 1 INTRODUCTION AND JUSTIFICATION FOR RESEARCH

1.1 INTRODUCTION

The ongoing trend of “bigger and better” means that more massive concrete structures are now being built each day. Some examples of these large-scale structures are the mat foundations and transfer slabs of high-rise buildings, intake assemblies of run-of-river hydroelectric facilities, and underground box structures for subway networks (See Figure 1.1). These types of reinforced concrete infrastructure are essential to society as they serve various sectors ranging from commercial housing to power generation. As a result, it is imperative that these structures are safely designed and appropriately assessed during their service lives.

Figure 1.1 – Examples of very thick structural elements in large-scale infrastructure

The inherent size of these structures makes them prone to potentially brittle and dangerous failures if their shear behaviour is not well understood. Ductile flexural response has been, and continues to be well understood owing to the “plane-sections” hypothesis that is the foundation of engineering beam theory [2]. Flexural failures are comprised of large deflections,
severe cracking and significant redistribution of internal forces, and therefore can resist huge overloads before collapse. Conversely, shear failures are characterized by almost imperceptible deformations occurring prior to collapse, providing no warning for occupants to safely exit the vicinity of the infrastructure in jeopardy. The danger with large-scale reinforced concrete structures is that they are more at risk of failures because of the “size effect” in shear. This phenomenon postulates that the shear stress to cause failure of a member without shear reinforcement decreases as the depth increases [3]. In essence, shear resistance of reinforced concrete does not linearly scale up with the depth of the member.

Considerable advances in understanding the shear response of reinforced concrete have been made through extensive experimental programs around the world. It is of concern, however, that some global design codes still do not accurately and safely capture the shear capacity of a reinforced concrete structure. The purpose of this research is to investigate the applicability and accuracy of various international shear strength procedures. To that end, it was decided to construct and test to failure a 4 meter thick reinforced concrete slab strip specimen at the University of Toronto Structural Laboratories. The massive specimen was tested in two phases, the first of which involved shear response of members without transverse reinforcement, and the second investigated the beneficial effects of widely spaced stirrups. A 300 mm thick companion specimen was also cast from the same batch of concrete to study the detrimental consequences of the size effect in shear.
1.2 **EARLY APPROACHES TO UNDERSTANDING SHEAR TRANSFER AND FAILURE**

It is important to first discuss the development of understanding the complex shear phenomenon in reinforced concrete as historical research often serves as the basis of the modern theories established in the present.

1.2.1 **45-Degree Truss Model**

Emil Mörsch was a German civil engineer who derived two key fundamentally groundbreaking innovations regarding the behavior of reinforced concrete subject to shear. The first of which was the 45-degree truss model that he explained in his famous 1902 text [4]. Mörsch proposed that a cracked reinforced concrete beam, with its complex system of internal stresses, can be simplified as diagonal compression fields in the cracked concrete struts of the web and tension in the reinforcement [4]. As suggested by the name, the model negated the inherently high level of indeterminacy possessed by the internal stress system through the assumption that cracks formed at 45 degrees from the horizontal with the diagonal concrete struts and steel ties forming the individual members of an idealized truss (See Figure 1.2). Mörsch recognized from experimental work that the angle of inclination of the diagonal compressive stresses were not usually 45 degrees, but concluded that it was mathematically impossible to determine the actual slope of the struts [2].

![General truss model](Figure 1.2 – General truss model)

1.2.2 **Shear Stress Distribution**

Mörsch once again revolutionized the comprehension of shear behaviour in reinforced concrete through his second key discovery – the shear stress distribution in a beam derived from simple equilibrium. He determined that shear stress increased parabolically from zero, at the extreme compression fiber of a section, to its maximum value at the neutral axis where it
remains constant down until it reaches any longitudinal flexural reinforcement \([4]\). The stress then reduces in a step-wise manner for each layer of steel and reaches zero shear stress at the bottom-most layer (See Figure 1.3). This stress profile indicates that a section primarily resists the applied shear loads through the portion that lies between the extents of the flexural lever arm (i.e. the web). A key conclusion that can be drawn from his derivation is that the effective area of shear resistance lies within the web. Vertical shear in this tension zone is transferred across the crack interface, hinting at the primary mechanism of shear transfer in reinforced concrete beams – aggregate interlock (explained later in Section 1.5).

![Figure 1.3 – Shear stress distribution according to Mörsch](image)

1.2.3 Variable-Angle Truss Model

The 45-degree truss model is used as the foundation for the ACI 318, the American equivalent of the Canadian concrete design code CSA A23.3. As a basic model with many limitations, it gave rise to the variable-angle truss model proposed to achieve less conservative, more cost-efficient designs. This was achieved via allowance of flatter strut angles that result in higher utilization of the transverse reinforcement \([2]\).

Removal of the fixed direction of principal compression in the variable-angle truss improved accuracy as tests had shown the concrete struts were typically inclined at less than 45 degrees in the latter stages of loading, especially when ultimate conditions were attained (See Figure 1.4) \([2]\).
The added complexity created an indeterminate problem in itself, as equilibrium conditions were now insufficient for solving for the four unknowns of the system. In order to solve the shear quandary, the principal concrete compressive stress, tensile stresses in both the transverse and longitudinal reinforcement, and the angle of the principal compression were all required to be known simultaneously [2].

Figure 1.4 – Variable angle truss model
1.3 THE MODIFIED COMPRESSION FIELD THEORY (MCFT)

The Modified Compression Field Theory (MCFT) is a comprehensive analytical model developed at the University of Toronto capable of determining the complete response of reinforced concrete subject to various combinations of in-plane stress states. A result of over 100 pure shear and combined shear and biaxial tests of reinforced concrete panels, the MCFT (See Figure 1.5) is founded on the 3 basic tools of structural engineering: equilibrium of forces and stresses, compatibility of displacements, and the accurate constitutive relations of the component materials [5].

The MCFT is essentially a smeared rotating crack model that forms the basis of the CSA A23.3-14 general shear design method [6]. Evolved from its predecessor, the Compression Field Theory (CFT), it accounts for principal tensile stresses in the concrete, among other refinements [7]. The MCFT is analogous to the variable-angle truss model through the internal system of stresses it represents in a reinforced concrete member. It does however, solve the problem of...
indeterminacy though the use of compatibility equations in conjunction with specific assessment of the local conditions at a crack interface [7].

Of particular importance to the research at hand is Equation 15 in Figure 1.5 which defines the shear stress on the crack surface that can be resisted prior to a slipping failure of the crack interface [2]. This equation was determined from investigative tests performed by Walraven (1981) which found that significant shear stresses can be transmitted across a crack by the aggregate interlock transfer mechanism visualized in Figure 1.6 below [8].

Figure 1.6 – Free body diagram of the aggregate interlock mechanism in reinforced concrete
1.4 **ONE-WAY SHEAR BEHAVIOUR**

1.4.1 **One-Way Shear Action**

One-way action of reinforced concrete members is the focus of shear behaviour considered in this thesis as two-way shear is a different phenomenon characterized by a punching-type failure. One-way shear, also regarded as beam action shear, results in failure occurring over a uniform surface across the entire width of the member (See Figure 1.7).

![Figure 1.7 – Two-way vs. one-way shear failure modes (Adapted from [9])](image)

1.4.2 **Beam Action vs. Arch Action**

It is important to differentiate between beam action (sectional behaviour) and arch action (strut behaviour) in the one-way shear response of reinforced concrete. Shear can be decoupled into these two components through the fundamental relationship between shear and moment expressed as follows [10]:

\[
V = \frac{dM}{dx} = \frac{d(T \times jd)}{dx} \quad [1.1]
\]

\[
V = jd \frac{d(T)}{dx} + T \frac{d(jd)}{dx} \quad [1.2]
\]

The first term in Equation [1.2] represents flexural (beam) action, where axial tension and compression forces that form the moment couple vary along the member at a constant lever arm, jd. Tied-arch action is delineated by the second term, characterized by constant flexural forces spanning over a varying lever arm, analogous to a truss. Distinction between these two mechanisms can be visualized in Figure 1.8, an illustration of compressive stress trajectories in a transfer girder.
Beam action occurs in regions far from applied loads, reactions, or geometric discontinuities ("undisturbed regions") where the classical theory of “plane sections remain plane” holds true. This theory is violated in those aforementioned “disturbed regions” where stress trajectories are concentrated in the vicinity of the disruption. The disturbed area was found by St. Venant to extend roughly one member depth away from the discontinuity [2].

![NLFEA Results of an RC Transfer Girder]

*Figure 1.8 – Beam action vs. arch action (Adapted from [11])*

Arch action governs the behaviour of beams with shear-span-to-depth ratios (a/d) of less than 2.5 and generally cannot be engaged at values significantly greater than 2.5 [2]. Members with a/d ratios less than 2.5 exhibit the formation of an internal arch mechanism after the breakdown of beam action and this gives rise to higher resistance [9].

Size effect is, in general, constrained to undisturbed regions, and so particular focus will be on behaviour where failure is precipitated through beam action collapse typified by a major inclined flexural-shear crack. This diagonal crack is distinctive to the brittle failure mode common in these types of members and it is imperative in understanding the associated shear resistance mechanisms [2].
1.5 Shear Transfer Mechanisms

Shear is carried by three fundamental transfer mechanisms in a reinforced concrete member devoid of stirrups: shear in the compression zone, dowel action of longitudinal reinforcement, and aggregate interlock (See Figure 1.9) [6]. Several tests and research have revealed that members without transverse reinforcement resists about 80% of total shear by the vertical component of aggregate interlock stresses integrated over the crack surface ($V_{ci}$) [9], [12], [13]. The remaining resistance is attributed to shear transmitted in the compression zone ($V_{cz}$), and dowel action ($V_d$). Presence of stirrups adds a supplementary element of shear resistance through the tensile straining of the transverse bars that cross a crack ($V_s$).

$Figure \, 1.9 \, - \, Shear \, transfer \, mechanisms \, in \, reinforced \, concrete \, members$

Mörsch demonstrated that shear is primarily transmitted in the web, as it was explicit that the extreme compressive fiber of a cross-section carries zero shear stress. Consequently, the uncracked compression zone constitutes only a small portion of the total shear resistance relative to contribution from the web [6].

Dowel action of the longitudinal reinforcement creates a component of shear resistance due to the physical shearing of the bar across a crack. It can be an important mechanism in beams with short shear spans, specifically for capturing complete response and overall member ductility [14]. The Two-Parameter Kinematic Theory (Mihaylov, 2013) stresses the importance of dowel action in deep beams, as double-curvature deformations of the longitudinal reinforcement
engages effective resistance against shear. It has been conservatively ignored in the MCFT formulations as it generally does not affect total shear resistance in an appreciable manner, particularly at ultimate conditions [6]. For large, lightly reinforced specimens, dowel action is practically negligible due to the relative size of the bars to the overall specimen. There is also not enough slip to engage this particular mechanism in these deep shear-critical members prior to breakdown of aggregate interlock.

Mörsch’s derivation of shear stresses infer that high shear in the flexural tension region were carried across the cracks. This interfacial shear transfer became the topic of study and further development by other researchers after Mörsch. Appropriately deemed “aggregate interlock,” it comprises of the shearing mechanism of the aggregates and cement paste over a diagonal flexural-shear crack [7]. Subsequent tests and literature indicate that aggregate interlock is the primary mechanism of shear transfer in large elements that do not contain stirrups, and thus, is an essential factor in the size effect phenomenon exhibited by reinforced concrete [9].

1.5.1 Tooth bending

Kani (1964) postulated an idealized comb model for the load-carrying mechanisms of a reinforced concrete beam flexurally cracked and subject to shear [15]. The concept suggests that a beam resists shear through compression in the backbone of the comb and tension via bending action in the cantilevered teeth of the comb (See Figure 1.10).

![Tooth bending mechanism of shear resistance](image)

*Figure 1.10 – Tooth bending mechanism of shear resistance (Adapted from [15])*

Compression in the backbone represents a tied arch that is indicative of strut-and-tie action that would continue to resist additional load in members with low a/d ratios if the teeth fail.
Figure 1.11 below depicts the “tooth-bending” mechanism in more detail. Tension is transferred via bond stresses in the reinforcement, and the change of moment over a concrete tooth causes a net tensile force to pull the tooth towards the point of highest flexure. This action is what creates the cantilevered effect of tensile stresses developing at root of the tooth which governs the capacity since Kani assumed flexural cracks were not capable of transmitting shear stresses (Case 2 in the figure below).

![Diagram of tooth-bending mechanism]

**Figure 1.11 – Development of cracks when shear is carried by beam action**

When cracks are small enough, they can indeed transmit shear stress through aggregate interlock; case 1 of Figure 1.11 illustrates the equilibrium conditions of the concrete tooth in this scenario. What this tooth bending model posits, is that the cantilevering action of cracked concrete teeth is the primary means by which long shear spans resist applied loads after aggregate interlocking has deteriorated.
1.6 **The Size Effect in Shear**

1.6.1 **Collapse of the De La Concorde Overpass**

The De La Concorde in Quebec was designed with abutments utilizing thick cantilevered reinforced concrete slabs that acted as simple supports for the central drop in span of the overpass. These slabs were constructed without any shear reinforcement as per the Canadian Highway Bridge Design Code at that time (CSA S6-1966), and had an effective depth of just under 1300 mm [16].

Conforming to the code provisions at the time, it was deemed that stirrups were not required for the slab under the subjected design stresses as they were less than the prescribed safe working shear stress limit [9]. The current Canadian code, CSA A23.3-14, would necessitate the thick abutment slab to contain at least minimum shear reinforcement in order to be compliant.

On Saturday September 30, 2006 at around noon, the slab forming the cantilevered overhang in the east abutment of the overpass collapsed onto the intersection below (See Figure 1.12), killing five people and injuring six more [16]. The tragic catastrophe was caused by a brittle shear failure that occurred within the undisturbed region of the deep member.

An as-built specimen of the cantilever was constructed and tested (See Figure 1.13 on the next page) as a part of the collapse investigation. Although other factors such as cyclic freeze-thaw deterioration contributed to undermining the structure, results concluded that stirrups would have undeniably prevented the shear failure through crack width arrest [16].
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1.6.2 Size Effect Explained

The size effect essentially states that the shear stress to cause failure of a reinforced concrete member decreases as the effective depth increases [17]. One explanation behind this occurrence is that deeper beams tend to have larger crack spacings and corresponding wider cracks that reduce the effectiveness of aggregate interlock capacity, the principal shear transfer mechanism in these members [6]. Breakdown of this interlock resistance on one of the primary inclined cracks is understood as the cause of size effect failure in an unreinforced member in this thesis.

As cracking is controlled through well-distributed transverse reinforcement, the size effect is predominantly manifested in members without stirrups [6]. Yoshida (2000) showed that even a single stirrup at the midspan of a beam can almost double shear strength and greatly increase ductility through attenuation of the size effect when compared to an unreinforced equivalent [18]. In fact, the MCFT predicts that it is the distance between longitudinal rebar rather than total member depth that underlies the size effect [5]. This becomes relevant for members containing distributed skin reinforcement, introduced later.
1.6.3 Parameters Related to Size Effect

1.6.3.1 Depth

Extensive testing of reinforced concrete beams has revealed that crack patterns are geometrically consistent across specimens of varying cross-sectional dimensions [6]. These tests illustrate that as the depth of a member is increased, both crack spacing and crack widths will increase proportionally.

Walraven (1981) established that wider cracks translate to diminished aggregate interlocking capacity and hence lower shear strength [8]. Additionally, deep beams demonstrated an evidently more brittle nature than small specimens of similar geometric scale; Sherwood’s tests in 2008 revealed that shallower beams achieved more than twice the deformation capacity of their deep counterparts [9]. The lack of ductility in these massive members provide little or no warning prior to failure, as was the case for the De La Concorde Overpass.

1.6.3.2 Aggregate Size

Aggregate interlock relies on the interfacial shear transfer on a crack, and as a result, it is heavily influenced by crack roughness. Irregularities on a crack surface arise from both the coarse aggregate and inherent unevenness of the fractured cement paste [7]. As such, the coarse aggregate size, and implicitly the concrete strength, are responsible for the ability of a crack to transfer shear stresses [19]. Increasing coarse aggregate size up to 25 mm improves shear resistance through the formation of rougher crack surfaces [9]. Bentz and Collins (2006) state that the effect of larger aggregate is more predominant in deeper beams, due to the fact that aggregate interlock governs the shear capacity of such members and aggregate is usually not scaled up with member depth [6]. Beyond 25 mm, experimental results from Sherwood (2008) indicate unpredictable outcomes of shear strength, and thus aggregate larger than 25 mm should be taken as effectively 25 mm in size [20].

Higher strength concrete has stronger cementitious paste that fractures a majority of the aggregate at the crack interface, creating more even crack planes [6]. These smooth cracks have a severe reduction on shear stress transmissibility as the aggregate size will fundamentally be rendered useless. In such cases, a_{b} should be taken as zero.
1.6.3.3 Crack Control

Cracking is expected under service conditions for most, if not all, reinforced and partially prestressed concrete structures; it allows for the reinforcement to be passively engaged and enables more practical designs. Deep members without shear reinforcement tend to have wider and more largely spaced cracks within the web as there is no steel in this region to restrain crack growth through tension stiffening (See Figure 1.15). The lack of well distributed reinforcement consequentially reduces aggregate interlock capacity on account of wider crack widths over a larger portion of the crack [9].

Crack control can be achieved by incorporating transverse reinforcement meeting the spacing requirements prescribed by codes or alternatively through the use of distributed flexural skin reinforcement [6]. The latter specifically requires bars that can resist the increased demand at a crack caused by the lack of concrete tensile contribution at these interfaces [7]. Research and testing by Sherwood (2008) highlights the inadequacy of using small bars to effectively control cracking because of the larger stresses they develop across the crack. Neglecting strain hardening, yielding of the bars by definition means that no additional load is required to continue elongating the reinforcement. Subsequently, straining of the member at a crack is further amplified at the onset of steel yielding, resulting in inadequate reserve capacity to impede crack width expansion [9]. On that account, larger area bars will have better crack control characteristics as they are more apt to a lower degree of stress across a crack plane, provided they are well distributed over the cross-section.

Figure 1.15 – Influence of reinforcement on crack spacing
1.6.3.4 Longitudinal Reinforcement

Bearing resemblance to the influence of crack control on shear capacity, primary longitudinal reinforcement also provides restraint to the widening of cracks. Although this is heavily constrained to the region within the local tension stiffened vicinity of the bar, the effect of larger reinforcement ratios results in smaller overall longitudinal strains and crack widths [21]. Crack spacing at the mid-depth of beams are therefore less affected by the amount of longitudinal steel simply because of their distance away from the bars. As in the case of skin reinforcement, higher ratios of longitudinal steel serve to increase aggregate interlock capacity and enhance shear strength.
1.7 THE TORONTO SIZE EFFECT SERIES

There exists a great amount of experimental work documenting the size effect in deep, reinforced concrete members lacking shear reinforcement dating back to 1967 when Kani first investigated beams just over 1 m deep [2]. Some of the more recent research into this detrimental effect arose from University of Toronto, with specimens having an effective depth ranging from 1.5 m to 2.0 m being tested by Yoshida (2000), ShenCao (2001), and Sherwood (2008). Extensive experimental programs carried out in Toronto, including those by the aforementioned authors, have resulted in a large database of results termed the Toronto Size Effect Series (See Table 1.1).

<table>
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<th>Name</th>
<th>$f'_c$ [MPa]</th>
<th>$a_g$ [mm]</th>
<th>$\rho$ [%]</th>
<th>a/d</th>
<th>$b_w$ [mm]</th>
<th>$d$ [mm]</th>
<th>$V_{exp}$ [kN]</th>
<th>$v_{exp}$ [MPa]</th>
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Specimen data obtained from [17]

The large and small slab strips that form the basis of this thesis were designed to be compatible with the previous investigative tests on the size effect at the University of Toronto. More detail regarding their designs are discussed later in Chapter 2. Both slab strips are consistent with the other test specimens in terms of concrete cylinder strength, aggregate size, longitudinal reinforcement ratio, and shear-span-to-depth ratio (a/d), providing a rational comparison to the series.
A plot of the experimental results compared with American (ACI) and Canadian (CSA) code predictions excellently portrays the size effect (See Figure 1.16). It can be seen how the MCFT-based CSA predictions of shear strength accurately match the experimental results with nonlinear strength degradation. On the other hand, the empirically based ACI predicts a highly unconservative linear increase in shear resistance with depth, showing a complete disregard of the size effect. The two code equations used to generate the blue and red prediction lines of Figure 1.16 are described in the next section.

Figure 1.16 – The size effect in shear for members without shear reinforcement

The slab strip specimens in this project, predominantly the larger one, will add vital data points to the Toronto Size Effect series, significantly expanding the range of the dataset. Up until now,
the largest shear specimen ever tested in North America was 2 m in overall depth, and the largest in the world was 3 m deep tested in Japan. However, this specimen was designed for a specific industrial application, so test conditions involved a very particular scenario with aspects such as bar cut-offs and uniformly distributed loading applied against self-weight, making it more difficult to generalize the results [2]. The large, 4 m deep, slab strip in the current investigation is a more simple experiment but will double the range of the Toronto Size Effect Series bringing it closer to some of the very thick slabs now being constructed.

The Toronto Size Effect Series has provided convincing evidence that the size effect does in fact exist. Furthermore, these results offer valuable insight as to what causes it so that engineers may be more acquainted when designing large scale structures common in everyday infrastructure. An example of such is the Wilshire Grand Tower in Los Angeles targeted for completion in 2017 which has a massive 5.3 m thick reinforced concrete mat foundation (See Figure 1.17) [22]. To really appreciate the scale of this structure, the picture below shows an array of T-headed stirrups protruding just about half their full height from the congested cage of flexural steel upon which construction workers are standing on.

*Figure 1.17 – Mat foundation reinforcement of the Wilshire Grand Hotel (Taken from [17])*
1.8  **INTERNATIONAL SHEAR DESIGN PROCEDURES**

Many international codes of practice rely on purely empirical relations and/or plastic truss models that, by their nature, cannot result in a globally unified approach to shear design [23]. Physical-mechanical models like the MCFT are required to formulate accurate expressions that lend tangible meaning to each and every parameter used. A brief outline of various shear provisions from codes around the world are outlined in the ensuing sections.

1.8.1  **CSA A23.3-14 / AASHTO-LRFD**

The shear resistance model presented by the current Canadian Standards Association (CSA) “Design of Concrete Structures” A23.3-14 [24] is derived from simplified MCFT (SMCFT) equations. Two approaches for determining shear resistance are outlined by the code: the General Method and the Simplified Method. The latter, as its name states, uses various simplifications from the former to create more straightforward design equations. The general method is the more accurate approach most directly tied with the original MCFT formulations. The American Association of State Highway and Transportation (AASHTO) LRFD shear design procedures [25] are also based on the SMCFT and is analogous to the CSA, so the following will strictly discuss A23.3.

1.8.1.1  **Historical Background**

The size effect was first accounted for by CSA in 1994 after the development of the MCFT allowed estimation of shear strength to be made for members without stirrups, something that was not possible with the original CFT [6]. A significant downfall of the code at the time is that it relied on tabulated values of design parameters for different member depths, as well as other properties. Herein lies the issue: a table can inherently have only a finite number of entries, hence A23.3-94 essentially limited the influence of the size effect in reinforced concrete members to 2 m in depth. The code specified that for members with depths greater than 2 m, the $s_{2e}$ term could be taken as 2 m (See 1.8.1.6).

In an attempt to generalize the next code iteration to allow for simpler application of the design provisions by engineers, Bentz and Collins (2006) derived simple equations for shear parameters that would basically result in the tabulated values from the 1994 code [6].
The outcome was a much improved A23.3-04 code that: a) harmonized the simplified and general methods which previously produced incompatible results of safety factors for certain structural members, b) made the general method easier to use for practitioners in industry, and c) predicted shear strengths for members larger than 2 m.

The 2014 code cycle did not bring any major overhauls, but two particular changes are worth mentioning because of their relevance to the research at hand. The first of which is related to the size effect through setting a lower limit of concrete resistance (β limit), and the second relaxes the spacing restrictions ($s_{max}$) of stirrups. These will be mentioned in the subsequent detailed breakdown of the A23.3-14 code provisions.

1.8.1.2 Factored Shear Resistance

Total factored shear resistance outlined by CSA, $V_r$, is determined from a concrete contribution, a steel contribution, and the vertical component of any prestressing serving to resist applied shears (See Equation [1.3] and Figure 1.18).

$$V_r = V_c + V_s + V_p \ [kN]$$  \[1.3\]

![Figure 1.18 – Basic shear resisting mechanism of A23.3-14 (Taken from [6])](image)

1.8.1.3 Concrete Contribution

The concrete contribution, $V_c$ (Equation [1.4]), arises from the aggregate interlock component of the diagonally cracked web. $\phi_c$ is the material resistance factor for concrete, taken as unity in this thesis because material strengths are known from testing.
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$V_c$ is primarily a function of: the aggregate interlock factor $\beta$ comprising of strain effect and size effect components (Equation [1.5]), the specified 28-day compressive cylinder strength of the concrete, and the effective shear area designated by the product of the web width $b_w$, and estimated flexural lever arm $d_v$ at shear failure.

$$V_c = \phi_c \lambda \beta \sqrt{f'_c} b_w d_v \ [kN] \quad [1.4]$$

$$\beta = \left( \frac{0.4}{1 + 1500 \epsilon_x} \right) \times \left( \frac{1300}{1000 + s_{ze}} \right) \quad [1.5]$$

The first relevant change to the 2014 code is that $\beta$ need not be taken as less than 0.05. This imposed lower limit on $\beta$ represents the alternate shear resistance mechanism of “tooth bending,” described earlier in this thesis, that enables sustained concrete resistance after cracks have widened beyond the point where they can transmit shear stresses.

1.8.1.4 Steel Contribution

The steel contribution, $V_s$ (Equation [1.6]), is simply the resistance offered by the transverse reinforcement yielding across a crack ($\phi_s = 1.0$ in this thesis). This equation shows that demand imposed on the stirrups is affected by the angle of inclination of principal compressive stresses, $\theta$ (Equation [1.7]). Effective flattening of cracks occurs as response progresses towards failure, and this increases utilization of the transverse steel since the principal tensile strains have rotated to remain orthogonal to the compressive struts [2].

$$V_s = \frac{\phi_s A_v f_y d_v \cot \theta}{s} \ [kN] \quad [1.6]$$

$$\theta = 29^\circ + 7000 \epsilon_x \ [degrees] \quad [1.7]$$

Spacing of transverse reinforcement is limited by:

$$s_{max} = minimum(0.7 d_v, 600) \ [mm] \quad [1.8]$$

Maximum stirrup spacing of 600 mm can be waived only if the general method is used and $s_{ze}$ is calculated from Equation [1.13]. This was the second relevant revision adapted into the 2014 edition of A23.3 recognizing the beneficial effects of widely spaced shear reinforcement. Previous testing has proven that the addition of even a minor amount of shear reinforcement can greatly increase both the strength and ductility of a reinforced concrete beam [18].
1.8.1.5 Strain Effect

Directly derived from equilibrium equations of the MCFT, the strain component of the β factor (Equation [1.9]) is a relation quantifying the aggregate interlock capacity of cracked concrete based on Walraven’s expression for interfacial shear resistance [6].

\[
\beta_{\text{Strain Effect}} = \frac{0.4}{1 + 1500\varepsilon_x} \tag{1.9}
\]

\[
\varepsilon_x = \frac{M_f}{d_p} + V_f - V_p + 0.5N_f - A_p f_{p0} \quad \frac{2(E_s A_s + E_p A_p)}{2(E_s A_s + E_p A_p)} \tag{1.10}
\]

The aforementioned discussions on the mechanisms behind the size effect phenomenon emphasize the importance of arresting crack widths to achieve sufficient aggregate interlock. As such, any tensile straining of a member would directly reduce the shear strength, and this is accounted for by the longitudinal strain parameter at mid-depth, \(\varepsilon_x\) (Equation [1.10]). This parameter lumps together the various geometric and load effects influencing shear strength in a single equation that elegantly delineates the components causing tensile straining [26].

1.8.1.6 Size Effect

The second component of the β factor (Equation [1.11]) makes the shear resistance a function of different member depths and aggregate sizes; therefore, explicitly accounting for the size effect. This size effect correction was determined from fitting an equation to the results of multiple MCFT analyses of various crack spacings [6]. A key variable in this equation is the effective crack spacing parameter, \(s_{ze}\) (Equation [1.12]) that is grounds for both the influence of aggregate size and the crack control characteristics of well distributed reinforcement [5].

\[
\beta_{\text{Size Effect}} = \frac{1300}{1000 + s_{ze}} \tag{1.11}
\]

\[
s_{ze} = \frac{35s_z}{15 + a_g} \geq 0.85s_z \quad [mm] \tag{1.12}
\]

If the maximum stirrup spacing of 600 mm is exceeded, then β must be calculated using:

\[
s_{ze} = (s - 300) \geq 300 \quad [mm] \tag{1.13}
\]
CHAPTER 1

INTRODUCTION AND JUSTIFICATION FOR RESEARCH

1.8.1.7 Moment-Shear Interaction

Shear causes diagonal compression fans to develop within a member that tend to push the member apart vertically as well as longitudinally. The horizontal component of these stresses induce additional demand on the flexural reinforcement on top of which is produced by bending or externally applied axial load alone:

\[
F_{lt} = \frac{M_f}{d_v} + 0.5N_f + (V_f - 0.5V_s - V_p)\cot\theta \quad [kN]
\]  

[1.14]

1.8.1.8 Simplified Method

In lieu of the more accurate general method, equations for the simplified method of shear design can be used, provided these conditions are met:

1. \( f_y \leq 400 \) MPa (specified yield strength)
2. \( f'_c \leq 60 \) MPa (specified concrete strength)
3. The member being designed or assessed shall not be subject to significant tension

\[
\theta = 35^\circ
\]  

[1.15]

The fixed angle of principal compression (Equation [1.15]) used in the simplified method is obtained from Equation [1.7] assuming an \( \varepsilon_x \) of \( 0.85 \times 10^{-3} \) which corresponds to a conservative value for yielding of the commonly used grade 400 steel in Canada [6]. If the section contains at least the minimum shear reinforcement, then:

\[
\beta = 0.18
\]  

[1.16]

For members containing no shear reinforcement:

\[
\beta = \frac{230}{1000 + s_{ze}}
\]  

[1.17]

Where \( s_{ze} \) is determined from Equation [1.12]. Specimens containing at least minimal stirrups have shown to exhibit crack spacings on the order of 300 mm. The \( \beta \) value of 0.18 in Equation [1.16] stems from using \( s_{ze} \) equal to this representative 300 mm spacing in Equation [1.17].

These equations allow for closed-form design without iteration since \( \varepsilon_x \) is not explicitly used. The simplified method is thus much easier to use than its general counterpart, but it is obvious that there are strict limitations that must be met in order to take advantage of the simplicity.
1.8.1.9 Strut-and-Tie Modelling

For structural members whose shear-span-to-depth ratios are below about 2.0, shear strength can be determined through a strut-and-tie (S&T) idealization. The strut-and-tie model consists of reinforcing steel tensile ties and concrete compressive struts connected at nodal zones forming a truss system (See Figure 1.19).

![Strut-and-Tie Model Diagram](image)

*Figure 1.19 – CSA A23.3-14 Strut-and-Tie Model (Adapted from [24])*

Concrete strut stress capacity is given by:

\[
f_{cu} = \frac{f'_{c}}{0.8 + 170\epsilon_1} \leq 0.85f'_{c} \quad [MPa]
\]

\[
\epsilon_1 = \epsilon_s + (\epsilon_1 + 0.002) \cot^2 \theta_s
\]

This limit arises from coexisting principal tensile stresses softening the pure cylinder response of concrete, also known as the compression softening effect [7].

Steel tie capacity is simply determined as the factored tensile resistance of the reinforcement. Nodal zones are also restricted to specific compressive stress limits based on the multi-axial stress state of the node since A23.3 assumes tie anchorage passing through the back of the node, creating a strain incompatibility in the always biaxially compressed zone.

The strut-and-tie model must be capable of carrying applied loads though arch action to the supports without any components of the truss being overstressed. More information on these calculations can be found in A23.3-14.
1.8.2 ACI 318-14

In 1962, the original “basic expression” for \( V_c \) was developed by the Joint ACI-ASCE Committee 326 through empirical means (See Figure 1.20) [27]. At this time, the experimental data available was very limited in terms of the range of test specimen size as well as other important parameters resulting in a primitive understanding of shear behaviour. As such, the empirically fitted equation based on small scale tests can give highly unconservative estimates of shear strength for large reinforced concrete members such as those discussed previously.

Like the CSA code, the American Concrete Institute (ACI) also separates total shear resistance into a concrete and steel contribution:

\[
V_n = V_c + V_s + V_p \quad [kN]
\]  

Equation [1.21] given below is the SI unit version of the concrete contribution according to the most recent ACI 318-14 provisions [28] that essentially remain unchanged from the original expression developed in 1962.

\[
V_c = \left(0.158 \sqrt{f'_c} + 17.2 \rho_w \frac{Vd}{M} \right) b_w d \quad [kN]
\]  

It can be seen that the derived expression shows a decrease in failure shear stress associated with increasing tension in the flexural reinforcement (i.e. the strain effect). However, ACI fails to capture the realistic severity of this shear capacity reduction stemming from the presence of
moment. Furthermore, there is no factor accounting for the aforementioned size, and hence predicts no detriment in shear resistance for large-scale structures.

Another downfall of the ACI code is that it is still based on the 45-degree truss model and hence neglects the flattening of the compressive struts that occurs towards shear failure. This can be seen from the omission of a principal angle of compression $\theta$ in the equation for the steel contribution (Equation [1.22]). As a result, the steel contribution to shear resistance is fixed and the actual increase in web reinforcement straining, and intrinsically steel utilization, as crack inclinations flatten is not captured resulting in a conservatively result in this regard.

\[ V_s = \frac{A_v f_{ytl} d}{s} \quad [kN] \quad [1.22] \]

ACI underestimates the substantial resistance provided by minimum shear reinforcement.

### 1.8.3 fib Model Code (2010)

The International Federation for Structural Concrete, or Fédération Internationale du Béton (fib) Model Code for Concrete Structures 2010 (MC2010) [29] uses the SMCFT in deriving its shear provisions for members without stirrups. Similarities between the CSA and fib can easily be seen by simple observation of the different expressions for shear resistance below. The corresponding CSA parameters are noted after each equation in round brackets for clarity. $\theta$ is allowed to be chosen freely by the designer conforming to the limits in Equation [1.28].

\[ V_{Rd} = V_{Rd,S} + V_{Rd,C} \quad [kN] \quad [1.23] \]

Where:

\[ V_{Rd,C} = k_v \frac{f_{ck}}{Y_c} b_w z \quad (\equiv V_c) \quad [kN] \quad [1.24] \]

\[ k_v = \frac{0.4}{1 + 1500 \varepsilon_x} \times \frac{1300}{1000 + k_{dg} z} \quad (\equiv \beta) \quad [1.25] \]

\[ k_{dg} z = \frac{32 z}{16 + d_g} \geq 0.75z \quad (\equiv s_{ze}) \quad [mm] \quad [1.26] \]

\[ \theta_{min} = 20^o + 10000 \varepsilon_x \quad [deg] \quad [1.27] \]

\[ \theta_{min} \leq \theta \leq 45^o \quad [1.28] \]
\[ \varepsilon_x = \frac{\left( M_{Ed} + V_{Ed} + N_{Ed} \frac{Z_p - e_p}{Z} \right)}{2 \left( \frac{Z}{Z} E_s A_s + \frac{Z_p}{Z} E_p A_p \right)} \quad (\equiv \varepsilon_x) \quad [1.29] \]

1.8.4 Eurocode 2 (2004)

Eurocode 2 (EC2) [30] are the set of standards set out by the European Committee for Standardization regarding the design of concrete structures.

EC2, like the ACI, used empirical equations statistically fit through experimental data available when the provisions were created to formulate their expressions of shear resistance [13]. While the resistance without transverse reinforcement is empirically derived, a variable angle truss model neglecting concrete tensile stresses is used to determine the shear resistance of a member with stirrups [13].

\[ V_{Rd} = V_{Rd,S} + V_{ccd} + V_{Ed} \quad [kN] \quad [1.30] \]

Where:

\[ V_{Rd,c} = C_{Rd,c} k \left( 100 \rho_1 f_{ck} \right)^{1/2} + k_1 \sigma_{cp} \right] b_w d \geq V_{Rd,c,min} \quad [kN] \quad [1.31] \]

\[ V_{Rd,c,min} = \left( 0.035 k^{1.5} \times f_{ck}^{0.5} \right) + k_1 \sigma_{cp} \right] b_w d \quad [kN] \quad [1.32] \]

\[ \rho_1 \leq 2.0\% \quad [1.33] \]

Eurocode 2 actually does account for the size effect with their empirical k factor (Equation [1.34]), although it is not strong enough for larger members.

\[ k = 1 + \sqrt{\frac{200}{\sqrt{d}}} \leq 2.0 \quad [1.34] \]

It is also noted that the EC2 equation for concrete contribution erroneously implies that a member without longitudinal reinforcement (\( \rho_1 = 0 \) in Equation [1.31]) has zero shear resistance and is consequentially shear critical (assuming no axial load or prestressing, \( \sigma_{cp} = 0 \)). This is in fact fundamentally incorrect because it is obvious that the lack of flexural reinforcement would result in an immediate bending failure at the time of first cracking; shear failure cannot be triggered without the formation of flexural cracks. Accordingly, Eurocode 2 compensates for this by implementing the \( V_{Rd,c,min} \) limit in Equation [1.32].
1.8.5 Australian Standard (2009)

Standards Australia is responsible for the creation of the AS 3600-2009 Concrete Structures [31] design provisions. Similar to other countries around the world, they intuitively decouple total shear resistance into concrete and steel components:

\[ V_u = V_{uc} + V_{us} \ [kN] \tag{1.35} \]

Where:

\[ V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o \left( \frac{f'}{c} \right)^{\frac{1}{3}} \left( \frac{A_{st}}{b_v d_o} \right)^{\frac{1}{3}} \tag{1.36} \]

The \( \beta \) factors in Equation (1.36) account for: 1) the size effect, 2) the presence of externally applied axial loads, and 3) strut-and-tie strength simulation for short shear spans, respectively.

For members with at least minimum shear reinforcement:

\[ \beta_1 = 1.1(1.6 - d_o/1000) \geq 1.1 \tag{1.37} \]

Otherwise:

\[ \beta_1 = 1.1(1.6 - d_o/1000) \geq 0.8 \tag{1.38} \]

\[ \beta_2 = 1 + \left( \frac{N^*}{14A_y} \right) \geq 0 \tag{1.41} \]

\[ \beta_2 = 1 - \left( \frac{N^*}{3.5A_y} \right) \geq 0 \tag{1.40} \]

\[ \beta_3 = 1, \text{ or alternatively: } \beta_3 = 2d_o/a_v \leq 2 \tag{1.42} \]

Where \( a_v \) is the distance from the nearest support face to the section where the resistance is being considered (i.e. M/V). It can be deduced that the size effect limit is applicable up to a depth of only 872 mm, after which \( \beta_1 \) is capped off at the lower limit of 0.8. Results from University of Toronto tests discussed previously explicitly showed that there is a continued degradation of shear resistance with increasing depth well beyond the 1000 mm mark. This forecasts that AS-3600 will likely overestimate the shear strength of thick members without stirrups such as the specimens in this thesis.
CHAPTER 2 EXPERIMENTAL PROGRAM

A summary of the experimental program shall be outlined in this chapter with fairly simplified schematics and limited pictures of the test specimens and setup. Detailed engineering drawings for the complete design, construction, and experimental program of this thesis can be found in Appendix A and a time-lapse of the entire project is located in Appendix B.

2.1 TEST SPECIMENS

2.1.1 Design

The experimental program for this project consisted of the design, construction, and testing of the two reinforced concrete slab strip specimens shown in Figure 2.1. The first of which, named PLS4000, is a Point Loaded Strip (PLS) intended to represent a thin slice abstracted from a 4 m thick one-way slab. Previous tests at the University of Toronto revealed that member width has little to no influence on one-way shear behaviour of slabs [32], and as a result, a thin strip is advantageous in a full-scale experimental context as it results in less materials required for construction. PLS4000 is therefore characteristic of many of the structural members in the aforementioned large-scale infrastructure. A companion specimen only 300 mm in depth, called the PLS300, was also built to represent the more traditionally sized members that were used in the empirical formulation of some shear design provisions as noted in Chapter 1.

Figure 2.1 – PLS4000 and PLS300 specimens in the University of Toronto structural laboratories
2.1.1.1 PLS4000

At 4 meters thick (13’’) and having a simple span of 19 meters (62’’-4’’), the PLS4000 specimen is the deepest shear specimen ever constructed in the world to date. The large slab strip is comprised of 19.4 m$^3$ of concrete (46700 kg) and 1270 kg of reinforcing steel, resulting in a total specimen mass of 47.7 tonnes (See Figure 2.2 for specimen details).

PLS4000 was designed with the desire to obtain two tests out of the single specimen, and for that reason, the 19 meter simple span was divided at the location of the point load into a 12 meter long east shear span containing no shear reinforcement and a 7 meter long west shear span that had close to minimum shear reinforcement. This would result in an initial sectional shear failure in the long east span (Phase I), followed by failure in the stronger west span once
the east was properly repaired (Phase II). It is worth mentioning that direct strut action may increase the shear strength of the west span due to the low shear-span-to-depth ratio. The overall dimensions of the slab strip were chosen such that it could fit within the strong floor of the structural laboratory as well as to ensure the proper strength hierarchy for the two desired phases of testing.

Flexural tension reinforcement consisted of nine 30M rebars coupled with HRC tapered threaded mechanical splices able to achieve the full strength of the bars. These bars have 65 mm diameter steel heads (HRC 150 head) friction welded to each end to ensure proper development. The “fins” at each end of the specimen provide a region for the longitudinal bars to develop and also constrains local stress concentrations in the vicinity of the heads away from the supports.

![Figure 2.3 – PLS4000 Bottom 30M flexural tension reinforcement: headed ends, male/female splices, and spacer blocks](image)

Minor flexural compression reinforcement was provided on the top of the specimen in the form of three 20M bars butted end-to-end since tension splices were not required in this zone.

Transverse shear reinforcement in the west span consisted of single leg 20M bars with 45 mm diameter HRC 150 heads welded on each end in a similar manner to the 30M longitudinals. These stirrups were spaced at 1500 mm centered within the shear span resulting in a total of five bars, the two outermost of which are located 500 mm from the support and load lines respectively.

75 mm of clear cover to the tension reinforcement, quite large by code standards, was set to mimic conditions found in hydroelectric facility intake structures.

The slender nature of the PLS4000 necessitated a check for lateral torsional buckling and it was found that there would be no occurrence of this instability under the anticipated loads.
2.1.1.2 PLS300

To provide a direct illustration of the size effect, the companion PLS300 specimen (See Figure 2.4) was designed with specifications similar to that of the PLS4000 (See Table 2.1).

![Diagram of PLS300 specimen](image)

**Figure 2.4 – PLS300 specimen details**

2.1.1.3 Specimen Comparison Summary

Both PLS4000 and PLS300 were designed to be shear critical with the former having a flexural capacity well in excess of predicted shear failure. PLS300 had a much closer gap between shear and flexural modes as predicted by CSA A23.3-14, but it was considered to be sufficient for the test objective.

<table>
<thead>
<tr>
<th></th>
<th>PLS4000</th>
<th>PLS300</th>
<th>PLS4000 / PLS300</th>
</tr>
</thead>
<tbody>
<tr>
<td>h [mm]</td>
<td>4000</td>
<td>300</td>
<td>13.3</td>
</tr>
<tr>
<td>d [mm]</td>
<td>3840</td>
<td>264</td>
<td>14.5</td>
</tr>
<tr>
<td>L [mm]</td>
<td>19000</td>
<td>1650</td>
<td>11.5</td>
</tr>
<tr>
<td>a [mm]</td>
<td>12000</td>
<td>825</td>
<td>14.5</td>
</tr>
<tr>
<td>b_w [mm]</td>
<td>250</td>
<td>175</td>
<td>1.4</td>
</tr>
<tr>
<td>a/d</td>
<td>3.13 (East)</td>
<td>3.13</td>
<td>1.0 (East)</td>
</tr>
<tr>
<td></td>
<td>1.82 (West)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ρ_l [%]</td>
<td>0.66</td>
<td>0.65</td>
<td>1.0</td>
</tr>
<tr>
<td>m* [kg]</td>
<td>46456</td>
<td>227</td>
<td>205</td>
</tr>
<tr>
<td>V [m³]</td>
<td>19.36</td>
<td>0.10</td>
<td>194</td>
</tr>
</tbody>
</table>

* Concrete only, reinforcement mass neglected
2.1.2 Construction

2.1.2.1 PLS4000

The PLS4000 was constructed in the University of Toronto Mark Huggins Structural Testing Facilities as per the stages delineated below in Table 2.2.

Table 2.2 – PLS4000 Construction Schedule

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Formwork Erection Phase I</td>
<td></td>
</tr>
<tr>
<td>a.</td>
<td>North (back) face, base, and end forms</td>
<td></td>
</tr>
<tr>
<td>b.</td>
<td>Flexural tension and shear reinforcement placement</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Start: 2015/04/03</td>
<td>End: 2015/04/15</td>
</tr>
<tr>
<td>2</td>
<td>Formwork Erection Phase II</td>
<td></td>
</tr>
<tr>
<td>c.</td>
<td>South (front) face</td>
<td></td>
</tr>
<tr>
<td>d.</td>
<td>Flexural compression reinforcement placement</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Start: 2015/04/16</td>
<td>End: 2015/04/24</td>
</tr>
<tr>
<td>3</td>
<td>Concrete Cast</td>
<td>2015/04/27</td>
</tr>
<tr>
<td>4</td>
<td>Formwork removal Phase I</td>
<td></td>
</tr>
<tr>
<td></td>
<td>North &amp; South face, end forms</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Start: 2015/05/04</td>
<td>End: 2015/05/07</td>
</tr>
<tr>
<td>5</td>
<td>Formwork removal Phase II</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Jacking procedure, base form removal</td>
<td>2015/06/10</td>
</tr>
</tbody>
</table>

The inherently massive scale of the PLS4000 necessitated pre-engineered modular formwork for construction due to time constraints, available floor space, and the importance of preventing form failures due to the large hydrostatic pressures associated with the 4 m height.

As a result, the Megalite Wallform© industrial forming system by Aluma was used to build up the PLS4000 formwork in the structural laboratories with construction being performed by their very own experienced personnel (See Figure 2.5). Aluma form systems are widely used in massive infrastructure around the world, so their product was an ideal solution for this project because of the tight geometric tolerances provided by the formwork even on such a large scale.

Weighing 47.7 tons, the PLS4000 was impossible to lift off the base forms using the 10 ton capacity overhead crane in the experimental facility. For that reason, the specimen was built in-place of testing and two predetermined locations in the base formwork were constructed for easy removal in order to carry out the jacking procedure described later in this section.
Roughly 2.5 m by 0.6 m by 0.1 m in size, the individual formwork panels are made up of specially coated plywood reinforced by a steel support frame. These panels are designed to be able to “gang” up and join with other panels creating a versatile modular system that was adapted to the geometry of the PLS4000 (See Figure 2.14). Accessories in the form of connecting clamps, stiffener walers, form ties, and bracing posts were used to assemble the entire system together. Formwork erection was completed in two phases: 1) north face, base, and end forms, and 2) south face closure. The back face panels were self-supporting via tension/compression brace posts that were anchored into the bolt caps of the strong floor as shown in Figure 2.5. End bulkheads for the anchorage fins were made from traditional plywood and 4-by-4 wooden reinforcing posts attached directly to the north and south panels (See Figure 2.6). Using ¾” threaded bar form ties, the front forms were affixed to the back forms to complete the closure as the final stage.
Reinforcement placement occurred between the two phases and involved coupling and placing the full length bottom longitudinal bars on temporary rebar chairs as the actual cover would be dictated by the lowest layer of form tie rods once forms were closed (See Figures 2.7 and 2.8)

Figure 2.7 – Reinforcement splices & headed bars at east end fin

Top compression reinforcement was placed during the cast as the use of a pump truck necessitated an unobstructed opening for the concrete hose. As-built conditions of the top and bottom flexural reinforcement are depicted in Figure 2.8.

Figure 2.8 – “As-built” conditions of bottom & top flexural reinforcement

Concrete was cast using three 7 m³ concrete trucks feeding a mobile concrete boom pump truck situated on the laboratory loading dock (See Figure 2.9). Scheduling was imperative as the feed of concrete into the hopper of the pump was required to be effectively continuous to
prevent plugging of the line. Consequently, the ready-mix trucks were ordered at 1 hour and 15 minute time intervals apart to allow for the three lifts necessary to fill the formwork, pour the companion specimen, and cast cylinders for material testing in an efficient manner.

A total of fifteen concrete cylinders were taken from each truck once the consistency of the mix was deemed acceptable for pumping (slump ≈ 150 mm). Superplasticizers were added to all three concrete trucks to achieve the desired workability.

![Image](image.jpg)

Figure 2.9 – Ready-mix concrete truck feed into hopper for pumping

The mobile pump truck was chosen for its ability to efficiently navigate the articulated pump hose along a majority of the specimen span within the tight confines of the laboratory (See Figure 2.10). A flexible hose was attached to the end of the boom to prevent a 4 m drop of fresh concrete which could have led to a high degree of segregation and honeycombing of the concrete.

Concrete was consolidated using industrial concrete vibrators along the entire PLS4000 span a couple of times each lift. An unfortunate event occurred during the pour when one of the two vibrating heads separated from the shaft and fell into the specimen at about the location of the point load. This was assumed to be the cause of the damaged strain gauges in that vicinity, as discussed later. Finishing of the top surface upon completion of the concrete placement was achieved by standard trowelling. After early initial concrete setting, specimens were watered twice a day and remained covered with a poly-tarp vapor barrier to prevent moisture loss during the curing process.
Formwork removal began seven days after the concrete was cast, and involved the concurrent implementation of lateral supports to provide out-of-plane stability to the specimen in lieu of the restraint originally contributed by the form bracing (See Figure 2.11). Details of the lateral support will be discussed in section 2.3.
Pockets in the base formwork were removed at two locations slightly offset inwards from the desired pin and roller supports for jack placement. A set of two hydraulic jacks with a steel spreader plate were used at each lifting position and all four were activated simultaneously to elevate PLS4000 (See Figure 2.12).

![Figure 2.12 – Jacks in position and support assemblies ready for placement](image)

During jacking of the large slab strip as shown in Figure 2.13, the base formwork fell off by themselves allowing for easy removal followed by placement of the support pin (west) and roller (east) assemblies.

![Figure 2.13 – PLS4000 jacking procedure](image)

Figure 2.14 displays a panoramic image summary of the major construction milestones from Stage 1 to Stage 5.
Figure 2.14 – PLS4000 construction summary: stage 1, 4, & 5 complete
2.1.2.2 PLS300

PLS300 was cast from Truck 1 of the PLS4000 to create a direct contrast for size effect because of the identical concrete mix.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Formwork Construction</td>
<td>2015/04/22</td>
</tr>
<tr>
<td>2</td>
<td>Reinforcement Placement</td>
<td>2015/04/23</td>
</tr>
<tr>
<td>3</td>
<td>Concrete Cast</td>
<td>2015/04/27</td>
</tr>
<tr>
<td>4</td>
<td>Formwork removal</td>
<td>2015/05/15</td>
</tr>
</tbody>
</table>

Standard wooden formwork was constructed for the PLS300, and concrete placement was achieved by shoveling/scooping from a wheelbarrow directly filled by the ready-mix truck.

Consolidating and finishing were executed using a concrete vibrator and trowels respectively. Wet burlap and poly-tarp were applied to the top surface one day after casting, and the specimen was then kept damp for six days. Forms were removed 18 days after the pour.
2.1.3 Demolition

PLS4000 weighed about five times the capacity of the overhead crane in the structural testing facilities. Appropriately, measures were taken during design and construction to allow for safe and efficient demolition of the slab strip. Multiple lifting hooks in the form of bent rebar embedded into the concrete were placed 1 m apart along the entire span for crane hoisting. Details of the lift hooks can be found in Drawing S-005 in Appendix A. After conclusion of the testing program, PLS4000 was cut into manageable full-height strips of about 2 m long using an industrial concrete saw (See Figure 2.17).

![Demolition of PLS4000 using concrete cutting saw](image)

Figure 2.17 – Demolition of PLS4000 using concrete cutting saw

The severely cracked and damaged state of the slab strip in addition to the light degree of reinforcement, particularly in the east span, required utilization of the same external post-tensioning used for Phase II of testing (Section 2.3.1.1) to vertically clamp the segments together during removal. Remains of the PLS4000 were placed flat into disposal bins situated on the loading dock and then transported to a landfill.
2.2 **Material Properties**

Key aspects relating to material property testing are outlined in this subsection while the full set of test data can be found in Appendix C.

2.2.1 **Concrete**

Concrete used in the test specimens was a pump specific ready-mix from a local supplier with a specified 28-day compressive strength of 30 MPa and a maximum aggregate size of 14 mm. A total of forty four cylinders were used for the determination of concrete properties as well as the full material constitutive relations closer to test days (Figure 2.18). Full concrete cylinder response was assessed by uniaxial compression tests on a 4500 kN capacity MTS Stiff Frame Testing Machine equipped with instrumentation for measuring strains.

![MTS Stiff Frame Test Machine](image)

*Figure 2.18 – MTS stiff frame (left) & Forney (right) compressive test machines*

Only the peak compressive strength of the cylinders were sought on the intermediary days before and between specimen tests to track the aging process, so a much simpler compression test was carried out using a Forney test machine. As self-weight of the PLS4000 specimen would be significant in the experiments, thirty eight of the tested cylinders were weighed and the average concrete density was determined to be 2373 kg/m³.

Concrete Truck 1 constitutes the lowest 1.5 m height of PLS4000 and the entire PLS300 specimen. Truck 2 makes up the next 1.5 m of PLS4000 height, while Truck 3 completes the last 0.5 m of the large slab strip (See Figure 2.20). Subsequently, concrete properties of PLS4000 are calculated as a weighted average of the three trucks based on the height of respective each lift.
The first truck was evidently the weakest from the batch as revealed by the complete collection of cylinder test results. Due to the time constraints of the testing periods, cylinder tests were completed after-the-fact such that test-day results could be interpolated from existing data rather than extrapolated.

Table 2.4 – Summary of Concrete Strength Development with Time

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Date</th>
<th>Age [Days]</th>
<th>( f'_{c} ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLS4000 Phase I Start</td>
<td>2015/06/10</td>
<td>44</td>
<td>42.6*</td>
</tr>
<tr>
<td>PLS4000 Phase I End</td>
<td>2015/06/15</td>
<td>49</td>
<td>43.2</td>
</tr>
<tr>
<td>PLS4000 Phase II Start</td>
<td>2015/06/26</td>
<td>60</td>
<td>44.0*</td>
</tr>
<tr>
<td>PLS4000 Phase II End</td>
<td>2015/06/29</td>
<td>63</td>
<td>44.2*</td>
</tr>
<tr>
<td>PLS300</td>
<td>2015/08/04</td>
<td>99</td>
<td>44.8†</td>
</tr>
</tbody>
</table>

* Values linearly interpolated from tested cylinders bracketing specific test dates
† Value taken from cylinders tested on 2015/08/05 (Day 100)

Full stress-strain cylinder responses allowed key parameters such as the strain at peak stress and modulus of elasticity to be obtained. A condensed set of concrete properties is shown in Table 2.5, followed by samples of constitutive relations in Figure 2.19.

Table 2.5 – Concrete Properties

<table>
<thead>
<tr>
<th>Age [Days]</th>
<th>Date [YYYY/MM/DD]</th>
<th>( f'_{c} ) [MPa]</th>
<th>( \varepsilon'_{c} ) [mm/m]</th>
<th>( E_{c} ) [MPa]</th>
<th>Relevant Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>49*</td>
<td>2015/06/15</td>
<td>43.2</td>
<td>1.890</td>
<td>34,500</td>
<td>PLS4000 Phase I</td>
</tr>
<tr>
<td>80*</td>
<td>2015/07/16</td>
<td>45.3</td>
<td>1.910</td>
<td>36,300</td>
<td>PLS4000 Phase II</td>
</tr>
<tr>
<td>100†</td>
<td>2015/08/05</td>
<td>44.8</td>
<td>-</td>
<td>-</td>
<td>PLS300</td>
</tr>
</tbody>
</table>

* Determined from full stress-strain response
† Determined from basic peak compression test

Specified compressive strength was exceeded as early as at the 7-day point. This high early strength gain is characteristic of modern day concrete, owing to the ability to grind cement more finely. The apparent counterintuitive decrease in strength for the PLS300 on day 100 is attributed to the fact that the smaller slab strip was cast from the weaker Truck 1.
2.2.1.1 Spatial Variability of Concrete Strength

As a part of another project at the University of Toronto, Hunter (2016) carried out ultrasonic pulse velocity (UPV) testing on the PLS4000 to assess the spatial variability of concrete strength over the massive area of the slab strip [33]. It was desired to determine if there was significant differences in concrete strength over the east span of the specimen through a correlation between UPV measurements of the actual specimen and its corresponding test cylinders.

Results from his experimental work show a vertical stratification of concrete strength consistent with the multiple-lift placement of concrete during casting with three different ready-mix trucks (See Figure 2.20). The wavy pattern of the lift interface seen in the figure was also partially visible from hints of surface discolouration upon formwork removal. The nonlinearity is due to the way in which fresh concrete from the pump displaces and causes concrete below from the previous lift to roll over to either side of the impact zone.
It is noted that Figure 2.20 shows an extrapolation of concrete strengths for Day 46. For more information regarding the correlation procedure as well as details on how spatial concrete strength variability was related to the actual experimental response, the reader is referred to Hunter (2016) [33].

### 2.2.2 Reinforcing Steel

Reinforcement for the test specimens came from two different steel suppliers, a local one from Southern Ontario and a more specialized one from Newfoundland. The former provided the flexural compression steel for the PLS4000 and main tension steel for the PLS300. The latter, HRC Inc., supplied the remaining rebar for the PLS4000 due to the specific need for headed and coupled bars as previously described.

Tensile coupon tests were performed on the various types of steel used for the specimens in order to determine their full constitutive relations. A sample of the reinforcement was placed between vice grips of a tensile test frame, and a clip gauge was attached to the bar over a 50 mm distance to measure displacements under load (See Figure 2.21).

![Figure 2.21 – Reinforcement coupon test on MTS 1000 kN testing machine](image)

Flexural tension reinforcement for the PLS4000 arrived in two separate heats with three bars making up the first batch (Heat 1), and the remaining six making up the second (Heat 2). Heat 1 comprised of the central column of three bottom bars and was flanked by a column of Heat 2...
bars on either side as shown in Figure 2.22. Steel parameters for these bars are thus calculated as a weighted average of the two heats.

![Diagram showing reinforcement types]

*Figure 2.22 – Reinforcement types in PLS4000*

Key properties of the various steel batches obtained from the tensile coupon tests are reported in Table 2.6, and the corresponding constitutive relations can be seen in Figure 2.23. $p_x$ and $p_x'$ correspond to the flexural tension (bottom) and flexural compression (top) steel respectively while $p_z$ represents the vertical shear reinforcement.

**Table 2.6 – Reinforcement Properties**

<table>
<thead>
<tr>
<th>Application</th>
<th>Grade</th>
<th>Size</th>
<th>$A_b$ [mm$^2$]</th>
<th>$f_y$ [MPa]</th>
<th>$\varepsilon_y$ [mm/m]</th>
<th>$E_s$ [MPa]</th>
<th>$\varepsilon_{sh}$ [mm/m]</th>
<th>$f_{s,ult}$ [MPa]</th>
<th>$\varepsilon_{s,ult}$ [mm/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLS4000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$p_x^*$</td>
<td>500W</td>
<td>30M</td>
<td>700</td>
<td>572.7</td>
<td>3.007</td>
<td>190,461</td>
<td>13.4</td>
<td>688.0</td>
<td>96</td>
</tr>
<tr>
<td>$p_x'$</td>
<td>400W</td>
<td>20M</td>
<td>300</td>
<td>434.3</td>
<td>2.215</td>
<td>196,046</td>
<td>11.9</td>
<td>595.8</td>
<td>144</td>
</tr>
<tr>
<td>$p_z$</td>
<td>500W</td>
<td>20M</td>
<td>300</td>
<td>522.0</td>
<td>2.289</td>
<td>228,064</td>
<td>16.7</td>
<td>628.9</td>
<td>112</td>
</tr>
<tr>
<td>PLS300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$p_x^+$</td>
<td>400W</td>
<td>10M</td>
<td>100</td>
<td>457.7</td>
<td>2.469</td>
<td>185,414</td>
<td>28.0</td>
<td>563.6</td>
<td>148</td>
</tr>
</tbody>
</table>

* Weighted average between the two heats of steel used for these bars $= \frac{1}{3}(\text{Heat 1}) + \frac{2}{3}(\text{Heat 2})$

† E determined from Sample 2 only due to slip in the other two samples causing unrealistic initial stiffness
Figure 2.23 – Reinforcement stress-strain relation samples
2.3 **LOADING APPARATUS AND TESTING PROTOCOL**

The experimental apparatuses for the PLS4000 and PLS300 are very different because they are on completely opposite ends of the size spectrum in reinforced concrete shear tests. PLS300 was tested using quite standard beam test procedures, while the PLS4000 required much more complexity in the setup largely on account of safety precautions.

2.3.1 **PLS4000**

PLS4000 demanded a complex loading apparatus (Figure 2.24) due to the slender nature of the specimen and the associated dangers with potential lateral torsional buckling effects.

![Figure 2.24 – View of PLS4000 test setup, after phase II test completion](image)

Supports in the form of moment resisting frames (MRF) placed at three sections along the specimen provided out-of-plane stiffness to react against any possible buckling during testing. The frame located on the load application line was further stiffened by an outrigger brace anchored into the strong floor. Each MRF contained lateral restraint supports that were bearing directly on the concrete to prevent any out-of-plane movement during jacking, testing, and demolition. Inherent friction was easily overcome during testing and did not affect results.
Figure 2.25 – PLS4000 experimental setup diagram (1 of 2)
The columns of the MRFs were all post tensioned to the laboratory strong floor with two 75 mm diameter bolts preloaded to 42 tons (412 kN) each. Figures 2.25 and 2.26 show detailed schematics of the apparatus used for testing the PLS4000. The experimental setup (with the exception of the outrigger brace) was in place during the full construction and jacking stages even prior to testing for safety reasons.

Longitudinal restraint was added to the specimen using ratchet straps on the west and east ends attached to the MRF columns. These were installed in case a shear failure would trigger unrestricted sliding of the slab strip eastwards due to the roller support, but were loosened progressively during testing to prevent development of compressive restraint action.

Figure 2.26 – PLS4000 experimental setup diagram (2 of 2)  
“As-built” placement of lateral supports

Figure 2.27 – Longitudinal restraint with ratchet straps
Reactions were provided in the simply supported conditions by a steel pin on the west and a steel roller on the east. Both load and reaction plates were made of steel and were 400 mm in length, covering the entire 250 mm width of the specimen. Supplementary vertical support with large wooden blocks were used underneath the large slab strip preceding anticipated failure in both phases to prevent sudden, and potentially unsafe impact with the strong floor.

Loading was done in a three-point bending style with an off-centre point load, $P$, located at Gridline P shown in the various schematics (ex. Figure 2.28). The load was monotonically applied in displacement-control, $\Delta$, utilizing a hydraulic jack running on a servo-valve. During the experiments, loading was stopped and reduced by about 10% at various points to execute load stages. A typical load stage involved finding and marking cracks using black permanent marker to make them more discernible in photographs. Crack widths and crack slip were measured with crack comparators, and the values of the widths were labelled on the specimen.

Ease of crack identification and marking was facilitated by the pre-emptive painting of the entire front face with flat white latex paint. Photographs were then taken of the entire specimen along the span and detailed surface displacement data was obtained using a 3D metrology device (discussed later in 2.4). Lastly, panoramic shots of the PLS4000 were taken with a camera placed on a rolling cart to get full-span captures at various stages.

Phase II of testing utilized continuous sampling LED targets with the 3D measuring system as opposed to the discrete measurements taken in Phase I. Procedures accordingly changed to simply having to start and stop the scanning system before and after every load stage.

![Figure 2.28 – PLS4000 test schematic (simplified from Figure 2.2)](image-url)
As testing took place over the course of a few days in both phases, the point load was reduced to zero each night to minimize creep effects before reloading on the subsequent day.

2.3.1.1 Repair for Phase II of Testing

Phase II of testing involved repairing the failed east portion of the slab strip specimen by strapping the shear span with high-strength external reinforcement such that the west span would be capable of being loaded to failure (See Figure 2.29).

![Installation of external reinforcement for phase II of testing](image)

**Figure 2.29 – Installation of external reinforcement for phase II of testing**

Reinforcement was provided in the form of paired 36 mm diameter Dywidag threadbars that were post-tensioned with about 30 tons (270 kN) on each bar against two HSS stub sections that sandwiched the specimen from above and below as shown in Figure 2.30. This strapping system was placed at four locations along the damaged span intended to bridge the primary failure crack (See Figure 2.31). The post-tensioning operation completely closed up the primary diagonal failure crack with negligible residual deformations. As a result, premature failure of the east span would not be possible because of the substantial overstrength provided from the externally applied pre-compression.
To provide supplementary repair caused by the dowel action “unzipping” the slab strip apart along the bottom flexural reinforcement, tensioned ratchet straps were externally applied to the specimen near the east support (See Figure 2.32 on the next page).
The high magnitude of overstrength provided by the external reinforcement would lead to unusual behaviour on the east span. Consequently, Phase II testing involved standard load stage procedures focused only on the west span with no crack measurements being taken from the repaired shear span.
2.3.2 PLS300

Companion to the PLS4000, the much smaller PLS300 specimen was also tested in three-point bending except the load was applied at midspan using an MTS Test Machine (See Figure 2.33).

![Figure 2.33 – PLS300 in 2650 kN MTS mobile testing machine](image)

Loading was performed monotonically in displacement-control through a spherical head and bearing plate placed on top of a grouted surface (See Figure 2.34), preventing local stress concentrations between the plate and specimen arising from surface irregularities.

![Figure 2.34 – PLS300 experimental setup & spherical loading head](image)
Load and reaction plates were sized based on an almost direct proportioning from the PLS4000 to deliver scaled results that are self-consistent within the experimental program. In the test setup illustrated in Figure 2.35 below, the pin support was located on the south side while the roller was on the north.

*Figure 2.35 – PLS300 test schematic (simplified from Figure 2.4)*

Load stages were completed in a similar manner as explained for the PLS4000, with the exception that neither discrete surface displacement measurements nor panoramic shots (not required for the small-scale) were taken. The use of the 3D metrology device was implemented in an identical manner to Phase II of the large slab strip test protocol.
2.4 Instrumentation and Data Acquisition

The experimental program incorporated a large amount of instrumentation to properly observe the behaviour of the two slab strip specimens. Accompanying the measurements taken by numerous instruments was media capture in the form of high quality pictures and video being recorded during all stages of testing.

2.4.1 PLS4000

The large slab strip specimen used a wide variety of advanced instrumentation that measured loads, strains, and displacements from the onset of jacking to the very final stage of loading. There were two main streams of data for the test: continuous data acquisition (DAQ) channels routed from all the various instruments, and discrete 3D surface coordinate data captured by an advanced metrology system. In addition to the standard picture and video, PLS4000 called for a special panoramic photography technique that allowed the entire specimen to be captured in one shot as previously mentioned.

2.4.1.1 Load Cells

Load cell assemblies used at the support locations of the test were made up of two 3000 kN capacity load cells sandwiched between heavy duty steel plates. The support pin and roller were situated on top of the load cell plates within specially marked grooves that were machined finished to allow for full bearing to occur (See Figure 2.36).

Upon completion of the jacking procedure, it was found that the load cell readings were erroneous, evident through discrepancies between the two reaction values. Post-test calibration of the load cells was thus performed and led to the discovery that the south load cell
on the west support had failed to been properly zeroed. As a result, a positive 85 kN offset (i.e. load readings that were 85 kN greater than what they were supposed to be) was present in the readings and this has therefore been corrected in the experimental data accordingly.

Figure 2.37 shows the 4450 kN load cell connected to the hydraulic load actuator that was responsible for quantifying the load developed under the applied displacement.

![Figure 2.37 – Load cell & loading actuator assembly](image)

2.4.1.2 Reinforcement Strain Gauges

Strain gauges were attached to the various types of reinforcing bars to determine steel strains at discrete positions. A diagram showing the complete strain gauge and wire routing configuration is depicted in Figure 2.38 on the subsequent page.

Flexural tension reinforcement was gauged on two of the nine bars at thirteen locations along the complete span from west to east. The two gauged bars were the top and bottom bars of the central column of reinforcement. Strains at a specific point were taken as an average between the two gauges unless one of the gauges lost functionality due to casting damage. Quite a few strain gauges were damaged, most notably the ones underneath the point load perhaps owing to the severed vibrator head that occurred during casting, and they are listed in Figure 2.38. Strains in the top layer of reinforcement was not a major concern of the testing program, therefore strain gauges were only applied at one location just offset from the point load simply to validate the compression that would be occurring in that region.

Vertical shear reinforcement consisted of strain gauges at the mid-height of the specimen, with the three bars central to the shear span having two gauges at the same point for averaging. The two outermost bars were not anticipated to undergo tensile straining due to clamping stresses.
Figure 2.38 – PLS4000 strain gauge configuration diagram
Surfaces of selected bars at predetermined locations were ground down using a disc grinder, progressively sanded, and chemically cleaned prior to strain gauge application. The gauges all had a 5 mm gauge length and were directly affixed to the reinforcement with cyanoacrylate glue. Due to the sensitive nature of electrical resistance strain gauges, sealing was accomplished with layers of polyurethane and wax coatings followed by a final protective layer of aluminum foil tape to prevent damage during construction and casting (See Figure 2.39).

Figure 2.39 – Strain gauging process

Gauge wires were routed along the length of the various bars, attached at 200 mm increments with fibre reinforced tape, and channeled out the bottom of the specimen via notches in the base formwork at multiple points that were later sealed with caulk (See Figure 2.40). This was done in order to prevent splicing of the 5 m long lead wires during the construction procedure which would have led to delays for the contractors. Bundles of strain gauges were then sealed and placed underneath the formwork to protect them from the remaining construction and cast operations. Once formwork removal was finished, gauge wires were routed from the front of the specimen through the nearest form-tie holes and connected to the main DAQ system situated on the back side.

Figure 2.40 – Strain gauge wire routing
Strain gauges applied to the longitudinal tension reinforcement were mostly aligned on the side of the bar to prevent deviations in strain readings as a consequence of bar bending at crack locations from possible dowel action.

Measurements of shrinkage strains are not available due to the lack of time to take readings before casting and during the curing process. All strain gauges were zeroed preceding specimen jacking, and zeroed once again before testing.

2.4.1.3 Displacement Measurements

Displacements were measured using a combination of Linear Variable Differential Transformers (LVDTs) and Linear Potentiometers (Linear pots). These instruments were all affixed to the north (back) face of the specimen using various mounting brackets and plates (See Figure 2.41). LVDT and Linear pots were set up as outlined in Figure 2.42 on the next page.

![Figure 2.41 – Vertical and diagonal LVDTs (left), longitudinal LVDT on east fin (right)](image)

Vertical displacements at five locations along the span were measured with LVDTs mounted on the back of the specimen and bearing off of the strong floor to obtain a profile of the deflected shape. The LVDT arrangement was symmetrical about the specimen midspan with the experiment being controlled by the displacement beneath the location of the point load, delineated as \( \Delta \) in all schematics. An LVDT used on the actuator itself was also present to track the stroke of the hydraulic jack.

A single LVDT was used on the east fin to monitor the longitudinal movement as the specimen translated on the roller support under increasing load.
Figure 2.42 – PLS4000 displacement instrumentation configuration diagram
Shear strains were measured indirectly using two diagonal LVDTs in the standard 90 degree rosette encompassing a 3.8 m by 3.8 m \((L_x \times L_y)\) square region at three predicted critical sections along the specimen. The displacement values of each LVDT were divided by the original gauge length, \(L_g\), of 5374 mm to obtain axial strains and an average shear strain can be calculated from standard Mohr’s circle geometric transformations.

When the LVDTs are placed orthogonal to one another bisected by the longitudinal and vertical axes of the member (See Figure 2.43), shear strain can be calculated using the following simplified equation:

\[
y_{xy} = \left(\frac{\epsilon_1 - \epsilon_2}{\sin(2\theta)}\right) = \frac{\epsilon_1 - \epsilon_2}{1} = \frac{\Delta_1}{L_g} - \frac{\Delta_2}{L_g} = \frac{\Delta_1 - \Delta_2}{L_g}
\]

[2.1]

Where the gauge length is defined as:

\[
L_g = \sqrt{L_x^2 + L_y^2}
\]

[2.2]
The out-of-plane (O.O.P.) response of the PLS4000 was crucial in determining if a bifurcation from plane stress behaviour to a torsional buckling mode was occurring during testing because of the slab strip slenderness. Furthermore, this was also important during the jacking procedure as instability was a primary safety concern of the entire experimental program. To that end, LVDTs and Linear pots were placed on the back side of the specimen at three points over the height on both east and west spans to monitor lateral displacements and corresponding out-of-plane rotations (See Figures 2.44 and 2.45).

\[
\theta = \tan^{-1}\left|\frac{\Delta_{Top} - \Delta_{Bot}}{L}\right|
\]  \[2.3\]

Figure 2.44 – Diagonal and lateral (O.O.P.) LVDTs (outlined in red for clarity)

Figure 2.45 – Lateral (O.O.P.) displacement measurements
Instruments were held by magnetic mounts cantilevered off independent HSS columns on the strong floor. Connection with the specimen was achieved by inserting an extension rod, similar to those used for the diagonal LVDTs, into an anchor embedded in the concrete.

![Image](image_url)

*Figure 2.46 – Lateral (O.O.P.) LVDT*

Vertical movement from jacking and loading would certainly affect the lateral displacement readings. However, the error in using the raw translation values without an adjustment to account for change in angle from downwards displacement would be minimal relative to the magnitude of movement anticipated to cause concerns of geometric instability.

Phase II of testing the PLS4000 involved failure of the west span once the east span was repaired with external reinforcement. The method of rehabilitation using high strength post-tensioned bars proved to obstruct the diagonal LVDTs on the east span due to direct contact between the steel and the instrument rods, portrayed in Figure 2.47. Furthermore, the shorter shear span-to-depth ratio on the west may lead to direct strut action governing failure of this span containing shear reinforcement. Because of these two reasons, it was decided to remove the far east LVDTs (Grid D3 in Figure 2.42) and use them in a manner more appropriate for the predicted failure mechanism.
Mihaylov et al. (2013) previously defined the area of concrete near the loading plate as the Critical Loading Zone (CLZ) [34]. The CLZ was found to carry a large portion of the shear in members with short shear spans, and corresponds to the zone of crushed concrete often found in these deep beam tests. LVDTs removed from the east span were therefore mounted in a configuration that would capture CLZ behaviour in both vertical and horizontal directions in the west span (See Figure 2.48).
2.4.1.4 Surface 3D Coordinate Measurements

Surface displacements were measured using a highly advanced metrology system produced by Nikon (formerly Metris). The system is an optical 3D coordinate measuring machine (CMM) that consists of a triple-lens infrared camera (K-610), a space probe mounted with nine light-emitting diodes (LEDs) for discrete hand-held measurements, and individual wired LED targets for continuous sampling (See Figure 2.49). Traditional use of the K-610 at the University of Toronto involved using the wired LEDs in a fixed range of view because specimen size typically did not exceed the measurement volume of the system. At the maximum camera distance of 4.5 m afforded by the available lab space, a window of only 2.5 m long by 2.0 m high could be captured by the camera in segments along the full length. As a result of the uncertainty of the exact failure location, as well as the extents of cracking, it was decided to use the space probe to capture several overlapping frames that would later be stitched together to create a full displacement map of the specimen at a particular deformation state in time.

A special frame fabricated for the K-610 allows it to be placed in an upright direction with rotational freedom about a pivot point. Affixing the frame onto a skid moveable by a pump truck allowed the camera to be moved along the length of the specimen (See Figure 2.49). The rotational allowance coupled with the translatable skid setup permitted full length coordinate measurement scanning of PLS4000 over a 2.5 m height, encompassing most of the vital flexural tension side of the slab strip.

*Figure 2.49 – Nikon K-610 optical coordinate measuring machine system (camera, space probe, wired LED targets)*
Figure 2.50 shows the surface LED coordinate configuration diagram used in both Phase I and Phase II of testing. Phase I utilized the discrete space probe measurements mainly discussed in this section, with a typical square grid of targets covering the specimen spaced at 600 mm between rows and columns. The layout of targets was designed to encompass critical areas of cracking and deformations within the slab strip.

Due to technical difficulties that arose with the K-610 controller software and the resulting inability to take readings with the space probe, wired LEDs were alternatively used in Phase II. As mentioned before, the restricted field of camera view resulted in a very limited 2.5 m long by 2.0 m high capture of the west span. A 3 by 3 rectangular grid of LEDs were mounted at the mid-depth of PLS4000 shifted towards the point load rather than at mid-shear span.

Unfortunately, it was found out after testing that two of the targets (top west and east) did not record any data due to problems with the LEDs. Consequently, it was decided not to use this data stream at all owing to both a lack of valuable information and time constraints.
Chapter 2

Experimental Program

The shear response of large-scale reinforced concrete structures

Figure 2.50 – PLS4000 surface LED target coordinate configuration diagrams

ELEVATION – Phase I

ELEVATION – Phase II

R = ROW # & C = COL # AS DESIGNATED ON GRID [e.g. (1,5)]

600 x 600 GRID OF TARGETS (TYP. U.N.O.)

[Phase I Measurements]

[Phase II Measurements]
Design of the space probe required that a custom target be made compatible with the needle probe tip and also enable straightforward mounting onto the specimen. Several iterations of probe tips and target combinations resulted in a fabricated aluminium angle with a conical seat that would hold the needle tip in a stable and consistent manner while repeatedly providing accurate measurements (See Figure 2.51). The angle targets were affixed into the specimen using a drilled in anchor and screw with additional adhesion being provided by epoxy applied on the attached leg.

![Figure 2.51 – Space probe target](image)

A detailed calibration and error assessment of the designed angle yielded an average in-plane strain error of $0.049 \times 10^{-3}$ over a 600 mm gauge length between targets, deemed acceptable for the measurements desired in the test. Complete detailed calibration data, information about the capture procedure, and methodology behind the coordinate data post-processing can be found in Appendix F.

### 2.4.1.5 Media

Three types of media were used to capture the response of the PLS4000 during testing. First, high definition video was taken of both spans with an independent camcorder for the west and web camera for the east. Secondly, a DSLR camera was used to take periodic photo bursts of the respective failure spans for each of the test phases (i.e. Phase I: east, Phase II: west), while a compact advanced camera recorded general test occurrences. Lastly, panoramic photos of the entire slab strip from end-to-end was achieved using the camera of a smart phone mounted on a rolling jig.
2.4.2 PLS300

PLS300 had a much simpler instrumentation configuration because of the appreciably smaller scale of the setup, resulting in having only a minimal quantity of data acquisitions channels (See Figure 2.52).

2.4.2.1 Load Cell

The MTS Mobile Testing Machine, shown previously in Figure 2.33, has a built in load cell for the hydraulic system directly connected to the main DAQ center.

2.4.2.2 Displacement Measurements

Two displacements were measured for the scaled down slab strip specimen: one vertical LVDT directly in line with the point load to control the test, and one longitudinal LVDT just above the single layer of flexural reinforcement to assess tensile strains.

Figure 2.53 – PLS300 LVDT arrangement
2.4.2.3 Surface LED Coordinate Measurements

A grid of wired LED targets were glued to the west face of the PLS300 in a pattern analogous to the space probe targets used on the PLS4000 to measure surface displacements.

The traditional size of the small specimen conveniently allowed for all targets to sit in the allowable measurement volume of the K-610 camera, and therefore raw data did not have to be assembled together by post-process stitching procedures.

Time synchronization between the main DAQ and the 3D scanner was performed as part of the post-processing to properly link the MTS and Nikon systems. Real-time values were determined from the pseudo-time stamps produced by the two software packages. The actual times were then employed to harmonize load readings and thus allow displacement data representative of each particular load stage to be obtained (See Section 3.3.3).

The K-610 captured coordinate data from the west face, while other measurements were taken on the east side. Displacement data later shown in Chapter 3 have therefore been mirrored about midspan to provide a consistent comparison with the various media streams and crack diagrams.

2.4.2.4 Media

High-definition video recording and photographs using a compact advanced camera were taken continuously throughout the testing process.
2.5 Shear Strength Predictions

Shear-moment interaction diagrams were created to illustrate the shear strength predictions for both PLS4000 and PLS300 made by CSA and ACI design provisions (Figures 2.55 and 2.56). These diagrams tell a visual story of what loading combination of shear and flexure will cause failure, the location of the critical section, and the utilization of capacity by self-weight. Furthermore, the detrimental effect of increasing moment on shear resistance captured by each code is also depicted. At the time the predictions were made, the concrete strength was assumed to be 40 and 45 MPa for the large and small slab strips respectively. Therefore, the calculations that follow use the estimated strengths as opposed to the actual test day values.

![Figure 2.55 – PLS4000 shear strength predictions based on ACI and CSA/AASHTO standards](image)

\[
V_c = \frac{130.4 \text{ kips}}{1 + 1500 e_x} \\
\rho_w = 0.656\% \\
P = 150.6 \text{ kips} = 670 \text{ kN} \\
 bw = 9.84'' \\
d = 151.2'' \\
x = 12.26 \text{ ft (1)} \\
x = 19.69 \text{ ft (2)} \\
x = 26.11 \text{ ft (3)} \\

\text{Figure 2.55 – PLS4000 shear strength predictions based on ACI and CSA/AASHTO standards}

Shear Response of Large-Scale Reinforced Concrete Structures

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Chapter 2

Experimental Program

Equation [2.4] presented below is a rewritten form of [1.21] in U.S. customary units and represents the blue ACI 318-14 line in the figures. Equation [2.5] is the comparable CSA/AASHTO formula of shear resistance from [1.4] translated to U.S. units as well, and is shown by the red line in the interaction diagrams.

\[
V_c = \left(1.9 \sqrt{f'_{ce}} + 2500 \rho_w \frac{V_d}{M}\right) b_w d \geq 2 \sqrt{f'_{ce}} \quad [\text{kips}] \quad [2.4]
\]

\[
V_c = 2 \sqrt{f'_{ce}} \left(\frac{2.25}{1+1500 \varepsilon_x} \times \left(\frac{50}{38 + s_x}\right)\right) b_w d \quad [\text{kips}] \quad [2.5]
\]

A significant discrepancy between CSA and ACI codes is noticeable by the huge gap between the red and blue predicted failure envelopes. The latter simply does not attempt to capture the size effect phenomenon.

It is observed from both Figures 2.55 and 2.56 that the ACI is not adequately punitive regarding the adverse effects of increasing moment on shear resistance, suggesting only a minor strength decrease from a \(V_c\) of 1070 to 1010 kN (240 to 227 kip) for the larger slab strip for example [35]. The lower bound of that range is the ACI permitted simplified failure shear stress delineated on the right hand side of the inequality in Equation [2.4]. CSA, on the other hand, predicts a significant drop in \(V_c\) from 456 to 210 kN (103 To 47 kip) as the magnitude of moment increases in the PLS4000.

As a sectional failure was anticipated in both specimens, it was desired to check various sections along the span where combinations of shear and moment from the externally applied load and self-weight produce the most unfavourable condition. The presence of high net vertical compressive stresses, called clamping stresses, near the load and supports result in an inherent overstrength of these regions [5]. Consequently, a sectional shear failure will be triggered between the disturbed regions of the shear span. At any given section along the slab strips, the M/V ratio of moment-to-shear due to the applied point load is constant and equal to the distance from the section to the nearest support. As the point load is increased, the moments and shears at the section of interest increase from the self-weight values and head towards the failure envelopes along the dotted lines in the two interaction diagrams.
Owing to the massive self-weight of the PLS4000, it was decided to check three sections along the east shear span: Section 1 @ a distance $d$ away from the face of the support, Section 2 halfway along the span, and Section 3 @ $d$ away from the face of the load. Section 3 is typically critical for members with negligible self-weight as it is the applied load that drives the failure. In the case of PLS4000, Figure 2.55 shows that Section 1 is in fact critical with the smallest increment from the self-weight values to the failure envelope. This can be partly explained by the fact that the uniformly distributed self-weight causes the highest shear to occur at the supports rather than at the point load.

![Figure 2.56 - PLS300 shear strength predictions based on ACI and CSA/AASHTO standards](image-url)
Figure 2.56 paints a very different picture for the more traditionally sized PLS300 specimen. The self-weight utilization of shear capacity can be seen to be almost negligible in a member this small, and thus the critical section clearly becomes located at a distance \(d\) from the face of the load. A key observation to be made here is that the ACI and CSA predictions are much more closely spaced for the small specimen when compared to the PLS4000, showing the agreement in shear strength for conventionally sized members.

These shear strength predictions demonstrate that there is an obvious disparity between a fundamentally based equation and an empirically based one, potentially resulting in very unsafe estimates of structural strength for large concrete members not containing shear reinforcement. The question of which one of these two North American design procedures are in fact correct will be answered in the subsequent chapters by investigating what transpired from the tests.
CHAPTER 3 EXPERIMENTAL RESULTS

A summary of key results from the experimental program are presented in the table below followed by an in-depth account of the observations made for each test specimen. The set of experimental data in graphical and consolidated tabular format can be found in Appendix D and Appendix E respectively. All specimens failed in shear and exhibited no evident signs of a flexural mechanism.

Table 3.1 – Summary of Experimental Results

<table>
<thead>
<tr>
<th>Parameter</th>
<th>PLS4000 East Span</th>
<th>PLS300</th>
<th>PLS4000 West Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c$ [MPa]</td>
<td>43.2</td>
<td>44.8</td>
<td>44.2</td>
</tr>
<tr>
<td>d [mm]</td>
<td>3840</td>
<td>264</td>
<td>3840</td>
</tr>
<tr>
<td>a/d</td>
<td>3.13</td>
<td>3.13</td>
<td>1.82</td>
</tr>
<tr>
<td>$\rho_1$ [%]</td>
<td>0.66</td>
<td>0.65</td>
<td>0.66</td>
</tr>
<tr>
<td>$\rho_2$ [%]</td>
<td>None</td>
<td>None</td>
<td>0.08</td>
</tr>
<tr>
<td>$P_{crack}$ [kN]</td>
<td>198</td>
<td>35</td>
<td>755</td>
</tr>
<tr>
<td>$P_{ult}$ [kN]</td>
<td>685</td>
<td>95</td>
<td>2162</td>
</tr>
<tr>
<td>$(M/V)_{crit}$ [m]</td>
<td>3.66</td>
<td>0.57</td>
<td>3.50</td>
</tr>
<tr>
<td>$V_{ult}^*$ [kN]</td>
<td>393</td>
<td>47.7</td>
<td>1510</td>
</tr>
<tr>
<td>$v_{ult}^+$ [MPa]</td>
<td>0.45</td>
<td>1.16</td>
<td>1.75</td>
</tr>
<tr>
<td>$\beta^\dagger$</td>
<td>0.07</td>
<td>0.17</td>
<td>0.26</td>
</tr>
<tr>
<td>$\Delta_{exp}^§$ [mm]</td>
<td>12.06</td>
<td>4.26</td>
<td>39.27</td>
</tr>
<tr>
<td>$\Delta_{exp}/L$</td>
<td>1/1570</td>
<td>1/384</td>
<td>1/483</td>
</tr>
<tr>
<td>$S_x^{**}$ [mm]</td>
<td>0.58d, 0.68d</td>
<td>0.52d, 0.80d</td>
<td>-</td>
</tr>
<tr>
<td>$w_{max}^{††}$ [mm]</td>
<td>0.75</td>
<td>0.20</td>
<td>5.50</td>
</tr>
</tbody>
</table>

* Shear at the CSA determined critical section delineated by the $(M/V)_{crit}$ parameter

† $v = V/(b_w d_v)$

‡ $\beta = v/(\sqrt{f'_c})$, Aggregate interlock factor analogous to Equation [1.5]

§ Deflection measured at location of the point load

** Longitudinal crack spacing @ mid-depth on the failure span (See Chapter 4)

†† Maximum crack width at approximately mid-depth measured just prior to failure
3.1 PLS4000 Phase I – Failure of East Span

East span failure of PLS4000 exhibited a characteristic inclined flexural-shear crack that initiated near the support and traversed diagonally over the member depth towards the load plate (See Figure 3.1). The testing protocol took 3 days to accomplish from onset of loading to first failure, with an extra morning dedicated to reloading the damaged specimen to confirm no reserve capacity remained in the slab strip.

Figure 3.1 – Failure of PLS4000 east span after reloading (load stage 6)

The behaviour of the specimen is succinctly captured in the load-deformation response below and panoramic crack photographs shown in Figures 3.10 and 3.11.

Figure 3.2 – PLS4000 Phase I: Experimental load-deformation response
3.1.1 General Behaviour and Observations

A summary of the forces and displacements at the various load stages is tabulated in Table 3.2 followed by a descriptive account of what transpired during testing.

Table 3.2 – PLS4000 Phase I Load Stage Summary

<table>
<thead>
<tr>
<th>Load Stage</th>
<th>P [kN]</th>
<th>Δ [mm]</th>
<th>P\textsubscript{unload} \textsuperscript{‡} [kN]</th>
<th>Δ\textsubscript{unload} \textsuperscript{§} [mm]</th>
<th>Date [YYYY/MM/DD]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Start</td>
<td>0</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>2015/06/10</td>
</tr>
<tr>
<td>1\textsuperscript{st} Crack</td>
<td>198.2</td>
<td>1.80</td>
<td>-</td>
<td>-</td>
<td>2015/06/10</td>
</tr>
<tr>
<td>1</td>
<td>251.1</td>
<td>2.92</td>
<td>201</td>
<td>2.94</td>
<td>2015/06/10</td>
</tr>
<tr>
<td>2</td>
<td>377.2</td>
<td>5.35</td>
<td>304</td>
<td>5.13</td>
<td>2015/06/11</td>
</tr>
<tr>
<td>3</td>
<td>500.1</td>
<td>8.12</td>
<td>433</td>
<td>7.92</td>
<td>2015/06/11</td>
</tr>
<tr>
<td>4</td>
<td>625.2</td>
<td>11.14</td>
<td>560</td>
<td>10.85</td>
<td>2015/06/12</td>
</tr>
<tr>
<td>5\textsuperscript{*}</td>
<td>684.5</td>
<td>12.06</td>
<td>429</td>
<td>12.65</td>
<td>2015/06/12</td>
</tr>
<tr>
<td>6\textsuperscript{†}</td>
<td>432.8</td>
<td>13.68</td>
<td>13</td>
<td>15.89</td>
<td>2015/06/15</td>
</tr>
</tbody>
</table>

\* Ultimate load
\† Maximum load attained after post-peak reload
\‡ Average value of load readings within the entire timeframe from start to finish of the load stage
\§ Displacement corresponding to P\textsubscript{unload}

3.1.1.1 Load Stage 1 (LS 1) – P = 250 kN

Loading began at 2:56 pm on Wednesday June 10, 2015 with the targeted point load P of 250 kN. Flexural cracking first occurred underneath the point load when P reached 198 kN indicated only by a drop in the load-deformation response as there were no audible or visible signs of a crack. This load resulted in a peak bending moment of 1884 kN-m at the point load and a corresponding concrete tensile stress of 2.48 MPa at the extreme tension fiber of the slab strip. It can be seen that the response (effective stiffness, k\textsubscript{exp}, of 247 kN/mm) closely matches the linear elastic prediction (E\textsubscript{clg} line in Figure 3.2, k\textsubscript{Eclg} = 302 kN/mm) in the uncracked state as anticipated. Testing progressed until the applied load reached the target of 250 kN at which point a 50 kN unload was performed for safe marking and measuring of crack widths. There were a total of four flexural cracks, three of which extended somewhat above mid-height. The largest crack width measured was 0.25 mm.
3.1.1.2 Load Stage 2 (LS 2) – P = 375 kN

Load stage 2 commenced at 1:59 pm on Thursday June 11, 2015 after 3D coordinate measurements were captured under the reduced load from the day before. Four main flexural cracks propagated above mid-depth after the load had reached 375 kN and was reduced for crack assessment. Two of these, one on the west span and one on the east span, turned slightly towards the load at the top indicating the onset of flexure-shear cracking. Largest mid-depth crack width on the east was measured at 0.15 mm. It was likely the formation of the new flexure-shear crack that caused a noticeable drop in load capacity prior to reaching LS 2.

3.1.1.3 Load Stage 3 (LS 3) – P = 500 kN

Further loading up to an applied P of 500 kN resulted in a large propagation of the west span flexure-shear crack that nearly made its way to underneath the load plate. A relatively smooth progression to the target load indicated no new formation of major cracks. It was decided to measure the visible slip at mid-depth along the major diagonal cracks on both spans at this stage. Sizeable crack slip of 0.7 mm occurred in the west on account of the larger shear demand, while the east only sustained a slip of 0.1 mm. Crack width at mid-depth of the east span was found to be 0.4 mm. Some hysteresis was noticed in the full unload attributed to cracks not fully closing up due to slip.

3.1.1.4 Load Stage 4 (LS 4) – P = 625 kN

Load stage 4 began on Friday June 12, 2015 at 11:50 am with the applied jack force reaching the target of 625 kN after audible evidence of a crack was heard before at a point load of about 590 kN. No new visible major crack formed causing the large drop in the load-deformation response; instead, it was propagation of the flexure-shear crack on the east span towards the load that triggered it. This crack turned over at a much flatter angle as it tried to make its way into the flexural compression zone, developing it into a potential flexure-shear failure crack. Measured width of the prospective failure crack was 0.75 mm at mid-height.

3.1.1.5 Load Stage 5 (LS 5) – P = 685 kN (Ultimate)

PLS4000 did not exhibit any sign of impending failure while loading up to LS 5, with no new crack formation or propagation up until when P had reached 684.5 kN. As P was further increased from this point, a new flexural crack suddenly formed at 5 meters from the east
support and rapidly spread upwards, crossing mid-depth at about 45 degrees, then extended towards the loading plate. As the crack traversed this path, the applied hydraulic force decreased to 475 kN as the top actuator continued to displace slowly downwards. It was clear from the brittle and abrupt nature of the behaviour that a sectional shear failure had occurred.

Over most of the length of the diagonal failure crack the measured cracks ranged from 3.0 mm to 4.5 mm wide, substantially wider than those of the previously existing ones, and there was evidence of substantial slip between the two faces of the diagonal failure plane (See Figure 3.4).

*Figure 3.3 – PLS4000 east span after shear failure*

*Figure 3.4 – Slip along a portion of the main diagonal failure crack*
Figure 3.5 shows the crack widths and crack slips measured along the two major diagonal cracks. The crack slips are underlined and list their distance from the east support, while the crack widths are in a somewhat smaller font. Note that for the diagonal crack which did not fail, the ratio of slip-to-width is about 0.6 while for the diagonal failure crack these slip/width ratios were from 0.6 to 1.3 (See Figure 3.6).

Figure 3.5 – PLS4000 east span failure crack slip measurements

Figure 3.6 – PLS4000 east span failure crack slip and crack width comparison plot
CHAPTER 3

EXPERIMENTAL RESULTS

From the onset of cracking to the ultimate load, the response of the slab strip followed a fairly linear path with an effective secant stiffness of 47 kN/mm (grey dotted line in Figure 3.2), a substantial 81% decrease from the uncracked stiffness. Linear elastic formulations predicts post-cracking stiffness to be 101 kN/mm ($E_c I_{cr}$ line in Figure 3.2), resulting in an underestimated 66% reduction compared to the observed result due to the neglect of shear deformations.

3.1.1.6 Load Stage 6 (LS 6) – $P = 433$ kN (Reload)

Following a complete unloading over the weekend, PLS4000 was reloaded at 10:51 am on Monday June 15, 2015. At an applied point load $P$ of 432.8 kN, the major diagonal crack from load stage 5 suddenly burst wide open revealing sunlight from the windows on the other side, and the actuator force simultaneously dropped a substantial amount to only 13 kN. Crack width at mid-depth of the main failure crack increased from 3 mm to 35 mm in less than a second. The inability of the slab strip to resist a load only 63% of the ultimate concludes that a higher resistance mechanism could not engage, and consequently the point load to cause a textbook sectional shear failure of the PLS4000 is 685 kN.

Figure 3.7 – PLS4000 east span following post-ultimate reload
Figure 3.8 – Visible light through failure crack (crack width labels not yet updated)

Unzipping cracks along the flexural tension reinforcement near the E support are characteristic of dowel action, as the set of bars separate from the specimen due to the vertical shearing mechanism (See Figure 3.9).

Figure 3.9 – “Unzipping” cracks caused by dowel action after shear failure
Figure 3.10 – PLS4000 Phase I: crack photographs (1/2)
Figure 3.11 – PLS4000 Phase I: crack photographs (2/2)
3.1.2 Crack Diagrams

Crack diagrams for the various load stages were generated using the panoramic photographs in combination with CAD software to better visualize the cracking progression without obstruction from the experimental setup. A two dimensional scaling was performed on the image to best fit the actual slab strip dimensions, and cracks were meticulously traced out using a free hand sketch tool. The resulting crack schematics are shown in Figures 3.12, 3.13, and 3.14 where the crack widths in millimetres are also labelled. As mentioned previously, crack measurements were taken after a partial unload from the target force of the load stage for safety purposes. Therefore, the actual force when the cracks were measured is slightly lower, typically on the order of a 10% decrease from the target point load (Recall Table 3.2).

Inherent distortion arises from any photograph because of shot perspective, regardless of panoramic post-processing. As a result, geometric transformations converting the capture coordinates into real-world coordinates would yield proper and more accurately scaled crack diagrams. However, due to the overall size of the slab strip as well as the relatively consistent manner in which the panoramic shots were taken (with respect to subject distance and camera angle), it was decided that this increased level of accuracy was not essential to the focus of the research.
Figure 3.12 – PLS4000 Phase I: crack diagrams (1/3)
Figure 3.13 – PLS4000 Phase I: crack diagrams (2/3)
Figure 3.14 – PLS4000 Phase I: crack diagrams (3/3)
3.1.3 Shear Strain Response

Average shear strains at predicted critical sections along the large slab strip portrayed a response that closely matched the progression of cracking (Figure 3.15). Upon reaching the ultimate load of 685 kN, the shear strains in east span crossing the late-forming failure crack are seen to increase with a corresponding drop in load. This “negative stiffness” behaviour in a load-deformation type plot is indicative of failure as it shows that increasing displacements, in this case the shear deformations, results in an inability to carry additional load.

Superposition of the LVDT shear strain measurement grids and the final crack pattern in the Figure 3.15 inset helps to clarify the behaviour. The main diagonal crack initiated from a flexural crack at about 5 m from the east support and traversed towards the load crossing first grid D3, and then grid D2. Descending branches of the shear strain response for those particular grids occur instantaneously after attaining peak load. D1 on the other hand, located on the west span, experienced no such degradation in capacity as that portion of the specimen was not in any distress.

Figure 3.15 – PLS4000 Phase I: shear strain response
After reloading to load stage 6, shear deformations in the D2 and D3 grids grew significantly with the substantial widening of the main diagonal crack. Continuity in Figure 3.15 is not shown as the large strains reached would have undermined the clarity of the main portion of the response. However, D2 reached a shear strain of about $5.1 \times 10^{-3}$ rads while D3 displacements grew indefinitely due to exceeding the LVDT stroke.

Concerning shear strains, it is perhaps more intuitive to plot them against the shear stress (calculated by Equation [3.1]) at the center of the LVDT grid to provide an instructive visual of the local conditions. The result of this is illustrated in Figure 3.16.

![Figure 3.16 – PLS4000 Phase I: shear stress vs. shear strain response](image)

$$v = \frac{V}{b_w d_v} \quad [MPa] \quad [3.1]$$

D1 and D3 shear stress-strain responses do not drop back to zero stress, and that is because the self-weight induced shears present when the applied load is removed are rather significant. D2 on the other hand is located exactly at the centerline of the slab strip where, by definition,
shear force resulting from uniformly distributed loading is zero. Plotting shear stresses as opposed to simply applied load on the vertical axis also demonstrates the major effect that self-weight has for a specimen of this size, something that Figure 3.15 does not capture. Peak shear stresses from west to east of the three grids were 0.67, 0.29, and 0.43 MPa respectively.

### 3.1.4 Reinforcement Strain Response

As PLS4000 was designed to be shear critical, the longitudinal reinforcement was not expected to yield throughout the entire experimental program. With the exception of some abnormalities, strain in the rebar followed the expected profile of highest elongation at the location of largest moment underneath the point load and essentially zero strain near the supports (Figure 3.17). Strain gauges are displayed as small red ‘X’s in the figure.

![Strain Response Graph](image)

*Figure 3.17 – PL4000 Phase I: flexural tension reinforcement strain response*

The omission of strain gauge data from 4.5 m to 10 m in Figure 3.17 is a result of damaged strain gauges noted in the previous chapter. Furthermore, the remaining strain gauge at 8.2 m (B2.1) became anomalous after the first load stage perhaps due to a flexural crack opening up
in the vicinity of the instrument. Consequently, load stages 2 to 6 have discontinuities in their responses. The large spike at the 16 m mark is considered abnormal because a) strains would not have decreased with successive load stages, and b) both cracking load stage 1 would have not produced enough flexural tension to cause reinforcement strains on the order of $1 \times 10^{-3}$. Besides that, none of the other bars at the other locations reached tensile strains higher than $1 \times 10^{-3}$ during this phase of testing, well below yielding.

Rebar strains in the flexural compression steel located just west of the applied point load remained at low levels. A maximum compressive strain of about $-0.18 \times 10^{-3}$ was reached at the peak load stage, showing that the concrete was nowhere close to crushing as the mode of failure was unmistakably one of beam-action.

Figure 3.18 shows the strain response in the vertical T-headed shear reinforcement located in the west span. Denoted by the red ‘X’ s in the figure, strain gauges were affixed at the mid-height of each bar.

![Figure 3.18 – PLS4000 Phase I: shear reinforcement strain response](image)

**Figure 3.18 – PLS4000 Phase I: shear reinforcement strain response**
It is noted that stirrups were not highly stressed at all in Phase I; the strain profile in Figure 3.18 is only apparent due to the small vertical scale. Strains were all less than $0.05 \times 10^{-3}$ and thus are regarded as not being engaged due to minimal cracking in the west span near the gauges.

### 3.1.5 Displacement Response

Vertical displacements of the specimen measured from LVDTs all illustrate similar responses as one would expect. Peak downward movement was located underneath the point load with less displacement being measured towards the supports. Longitudinal elongation of the flexural tension side reached a maximum of 5.7 mm with about 2.5 mm of residual deformation considering cracks did not fully close up after final failure had occurred.

![Figure 3.19 – PLS4000 Phase I: vertical displacement profiles](image)

Deformed shapes at various load stages are shown in Figure 3.19 with the most rudimentary linear interpolation functions used between measurement nodes. The self-weight profile of deformations was not determined because of an error in zeroing some of the data acquisition channels after jacking.
For that reason, self-weight displacement underneath the point load (x = 7 m) was assumed to be 1.00 mm as determined by the elastic formula below:

\[
\Delta(x) = \frac{(w_{s/w}) \times x}{24E_cI_g} (L^3 - 2Lx^2 + x^3) \text{ [mm]}
\]

[3.2]

Where \(E_c\) is equal to 28000 MPa and \(I_g\) is \(1.333 \times 10^{12}\) mm\(^4\) calculated as follows:

\[
E_c \approx 4500\sqrt{f'_c} \text{ [MPa]}
\]

[3.3]

\[
I_g = \frac{b \times h^3}{12} \text{ [mm}^4\]
\]

[3.4]

Deformed shapes intuitively follow the bending moment diagram of the simply supported slab strip. Initial displacements peaked in a nonlinear fashion near the point load, showing the effect of concentrated cracking early in the response. Vertical deflections then grew in a proportional manner from load stages 2 to 6 after cracking became much more distributed along the span.

The out-of-plane displacements never reached magnitudes that warranted concerns of buckling instability with peak lateral movement of under 2 mm and corresponding rotations of 0.1 degrees. Lateral response is of no particular importance to the experimental program and is hence reserved for Appendix D and Appendix E.
3.1.6 Detailed Surface Displacement Plots

Deformed shapes of PLS4000 during phase I of testing were generated using the methods explained in Appendix F, and are presented below.

Figure 3.20 – PLS4000 Phase I: 3D surface displacement plots in XY elevation view (vertical displacements magnified x15)
Figure 3.20 displays the deformation state of the large slab strip at the various load stages, matching the profile obtained from the vertical LVDTs. The most expressive plot is perhaps load stage 6 (LS 06), whereby the significant distortions induced by the post-peak failure crack are unmistakeably portrayed in the exaggerated coordinate map.

![Figure 3.20 - Experimental Results](image)

*Figure 3.20 – PLS4000 Load stage 6 surface displacements superposed on crack photograph (vertical displacements exaggerated)*

Superimposing these displaced coordinates on the crack photograph leads to a fascinating image as shown above in Figure 3.6. Nodal translations of the space probe targets closely match the primary diagonal crack that precipitated the brittle drop in load carrying capacity at the post-peak ultimate state. Dowel action deformations from the longitudinal rebar “unzipping” apart the specimen is also noticeable from the deviation of the bottom two rows of targets near the origin of the diagonal crack.

Strain states could not be processed from this coordinate data because of the relative inconsistencies between load stages caused by the transformative procedures as elucidated in Appendix F.
3.2 PLS4000 Phase II – Failure of West Span

Phase II of testing the large slab strip involved an explosive concrete crushing failure adjacent to the load plate on the west span. This closely resembled the aforementioned highly stressed critical loading zone (CLZ) that Mihaylov et al. (2013) stated as carrying considerable shear in members with short shear spans [34].

*Figure 3.22 – Failure of PLS4000 west span (load stage 11)*

Phase II behaviour is presented in the load-deformation response below and crack photographs shown in Figures 3.28 and 3.29.

*Figure 3.23 – PLS4000 Phase II: Experimental load-deformation response*
3.2.1 General Behaviour and Observations

A summary of the forces and displacements at the various load stages are tabulated below followed by a descriptive account of testing. The discussion focuses purely on the west span response of the slab strip as the artificially enhanced east side was no longer of interest to the experimental program with regards to load stages.

Table 3.3 – PLS4000 Phase II Load Stage Summary

<table>
<thead>
<tr>
<th>Load Stage</th>
<th>P [kN]</th>
<th>Δ [mm]</th>
<th>P_{unload} † [kN]</th>
<th>Δ_{unload} ‡ [mm]</th>
<th>Date [YYYY/MM/DD]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Crack</td>
<td>754.9</td>
<td>8.66</td>
<td>-</td>
<td>-</td>
<td>2015/06/26</td>
</tr>
<tr>
<td>2nd Crack</td>
<td>973.6</td>
<td>12.81</td>
<td>-</td>
<td>-</td>
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<td>900</td>
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<td>2015/06/26</td>
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<td>1246</td>
<td>19.98</td>
<td>2015/06/29</td>
</tr>
<tr>
<td>9</td>
<td>1750.4</td>
<td>28.72</td>
<td>1592</td>
<td>27.27</td>
<td>2015/06/29</td>
</tr>
<tr>
<td>10</td>
<td>2019.1</td>
<td>35.36</td>
<td>1578</td>
<td>30.36</td>
<td>2015/06/29</td>
</tr>
<tr>
<td>11*</td>
<td>2161.9</td>
<td>39.27</td>
<td>200</td>
<td>44.08</td>
<td>2015/06/29</td>
</tr>
</tbody>
</table>

* Ultimate load
† Average value of load readings within the entire timeframe from start to finish of the load stage
‡ Displacement corresponding to P_{unload}

3.2.1.1 Load Stage 7 (LS 7) – P = 1000 kN

Load stage numbering was simply continued from Phase I of testing. Once repair and Phase II test preparation was complete, load stage 7 of the PLS4000 commenced at 3:46 pm on Friday June 26, 2015. After loading the slab strip up to the east span failure load of 685 kN, assessment of a potential load stage was carried out. Unloading the specimen and inspecting cracks led to the conclusion that not much had changed from the onset of testing, and hence it was decided to continue loading. The first noticeable occurrence was observed when P reached 750 kN and the existing diagonal cracks below mid-depth began extending towards the load. Further loading to P of 974 kN lead to increasing crack propagation and the joining of one extension to the main diagonal crack from Phase I. A total of three diagonal cracks crossed through the mid-depth and the largest width at this location was 1.8 mm. Up to first cracking,
CHAPTER 3 EXPERIMENTAL RESULTS

the repaired slab strip can be seen to follow \((k_{\text{exp}} = 87 \text{ kN/mm})\) the linear elastic cracked line in Figure 3.23 \((k_{\text{Eclcr}} = 101 \text{ kN/mm})\), demonstrating that the west span had indeed softened due to residual deformations occurring from Phase I (Recall \(k_{\text{Eclg}} = 302 \text{ kN/mm}\)).

3.2.1.2 Load Stage 8 (LS 8) – \(P = 1375 \text{ kN}\)

After a full unload over the weekend, testing commenced on the morning of Monday June 29, 2015. Prior to reaching the target load of 1375 kN, a pause at the previous jack force of 1000 kN was performed to loosen up the ratchet straps providing contingent longitudinal restraint on the east support. Attainment of load stage 8 occurred with no new sizeable cracking but had associated crack extensions and widening of existing ones. Mid-depth crack width was 2.0 mm at a diagonal crack initiated near the west pin support that began to traverse directly towards the load.

3.2.1.3 Load Stage 9 (LS 9) – \(P = 1750 \text{ kN}\)

Similar to the previous stage, load stage 9 was not accompanied by the formation of new major cracks. A concentration of deformations was noted to collect in the two main diagonal cracks as widths were unquestionably much larger than adjacent cracks, one of which was 4.0 mm wide above mid-height.

3.2.1.4 Load Stage 10 (LS 10) – \(P = 2020 \text{ kN}\)

Careful attention was paid to the overall stability of the PLS4000 at this juncture due to the massive amount of energy input associated with a load of over 2000 kN. The specimen was first unloaded at a \(P\) of 2018 kN due to concerns of imminent failure caused by apparent uncontrolled creep deformations of the CLZ. Upon stabilization of the deformations, reloading with noticeable hysteresis occurred until \(P\) reached 2020 kN, the maximum attainable force governed by the capacity of the hydraulic pump system in place. The load stage was held for longer than usual to allow creep deformations to stabilize such that the state of the slab strip was deemed safe enough for inspection. During unloading, a loud noise was heard which arose from slipping of the southeast lateral restraint. Once deformations were deemed to be at a steady state, the load stage was conducted.
Local support plates on both ends were discovered to be deformed as a piece of paper was able to slide between the steel and the specimen at particular locations. On the west pin support, about 37 mm of plate was found to be elastically bent on the east edge. There was also minor plate bending on the west end of the east roller support (See Figure 3.24).

![Evidence of plate bending at east support](image)

_A discovery of an almost imperceptible crack extension joining the two major diagonal cracks on both sides of the point load was executed when inspecting the area underneath the load plate (See Figure 3.25). This will be seen more clearly in the discussion of LS 11 accompanied by Figure 3.27. A full unload was done in order to switch out the hydraulic pump for a higher capacity replacement to facilitate pushing PLS4000 to complete failure. It is noted that there was no panoramic image taken during this load stage._

![PLS4000 horizontal crack under load plate](image)

_A full unload was done in order to switch out the hydraulic pump for a higher capacity replacement to facilitate pushing PLS4000 to complete failure._

![Evidence of plate bending at east support](image)
3.2.1.5 Load Stage 11 (LS 11) – P = 2162 kN (Ultimate)

Load-deformation response of the large slab strip was seen to slightly soften and “turn over” at about the 2000 kN load mark. An additional 0.5 mm of applied displacement and corresponding load increase of about 100 kN led to signs of imminent failure. The ensuing increase in displacement resulted in no escalation of the applied load reading. Abruptly after reaching P of 2162 kN, a loud concrete crushing failure occurred beside the load plate (See Figure 3.26).

![Figure 3.26 – PLS4000 west span at time of failure](image)

Immediately following the strut-and-tie mechanism of crushing concrete took place, the jack force reduced to about 10% of the ultimate value, signifying a complete loss of load carrying capacity after shear failure. Significant shear deformations were evident upon inspection of the damaged slab strip, as the concrete block below the critical crack impacted the auxiliary wooden supports underneath due to vertical translation of the CLZ. Two main diagonal cracks are visible: the first forming an almost direct path from the load to the support, and the second tracing a path from the load to the flexural cracks at the central portion of the span.
The highly stressed CLZ had completely spalled off, revealing the severely buckled compression reinforcing bars underneath (See Figure 3.27).

Due to the destructive nature of the failure, instrumentation measuring the CLZ displacements were torn off but no equipment was damaged.

In a similar fashion to the east span, the load-deformation response in this phase of testing exhibited a linear post-cracking stiffness up until failure with an effective $k$ of 47 kN/mm, a 46% reduction from the repaired “uncracked” state.
Figure 3.28 – PLS4000 Phase II: crack photographs (1/2)
Figure 3.29 – PLS4000 Phase II: crack photographs (2/2)
3.2.2 Crack Diagrams

Crack diagrams for Phase II were constructed utilizing the exact same procedure as mentioned in 3.1.2 from the panoramic photographs. There was no panoramic for load stage 10 due to an issue with the photographic device. Consequently, crack patterns for this state were deduced from the next failure load stage and cross-referenced with standard photographs to determine crack location and widths.
Figure 3.30 – PLS4000 Phase II: crack diagrams (1/3)
Figure 3.31 – PLS4000 Phase II: crack diagrams (2/3)

Shear Response of Large-Scale Reinforced Concrete Structures
Figure 3.32 – PLS4000 Phase II: crack diagrams (3/3)
3.2.3 Shear Strain Response

Shear strains in the central portion of the west span exhibited a response similar to the east end, although it may not be immediately clear from Figure 3.33. Just prior to the peak load being reached, there was no indication of failure from shear strain readings at grid D1 progressing with negative stiffness like in Phase I. Rather, the abrupt strut-action crushing failure occurred immediately after the applied load reached its maximum value, and shear strains are shown to drop to zero. This is misleading because it was observed that the explosive shear failure involved significant widening of the two main diagonal cracks.

The inset in Figure 3.33 shows the D1 LVDT grid, located at the central portion of the west span, and it can be seen that both diagonal instruments cross the same critical crack. Inspecting Equation [2.1] for the calculation of shear strain, the apparent decrease in shear deformations can be traced to the fact that both LVDTs underwent similar displacements and thus their algebraic difference diminished upon failure.
Normalizing the shear forces at each LVDT grid with respect to the effective web area, $b_wd_v$, gives rise to the shear stress-strain diagram in Figure 3.34 below.

Figure 3.34 – PLS4000 Phase II: shear stress vs. shear strain response

Peak shear stresses attained in the D1 and D2 grids were 1.75 and 0.92 MPa respectively. The presence of stirrups permits the slab strip to be able to resist much higher shears, therefore the utilization of capacity from self-weight has a reduced effect when compared to what Figure 3.16 revealed for the east span failure.
3.2.4 Reinforcement Strain Response

Flexural reinforcement strain profiles are displayed in Figure 3.35 below; strain gauges are denoted by small red ‘X’ s as before. Omission of strain data was because of faulty or damaged gauges that gave abnormal results.

![Figure 3.35](image)

*Figure 3.35 – PLS4000 Phase II: flexural tension reinforcement strain response*

The loss of many strain gauges perhaps results in Figure 3.35 not being as expressive as intended, however, general conclusions can still be drawn. By and large, reinforcement straining increases with applied force in a trend consistent with the loading profile. Occurring at the 14 m mark are large strain spikes that are not compatible with the statical system of the slab strip, likely a result of local bar bending at the location of the gauge. Strains in the west end develop in a constant manner along with the load; values close to $2 \times 10^{-3}$ illustrate that shear-moment interaction arose at the support. Despite there being little to no flexure-induced tension near the supports, the shear is highest at that location and results in tensile elongation of the reinforcement resisting the horizontal component of shear stresses on a crack.
Compression reinforcement was strained to \(-0.30 \times 10^{-3}\) on one of the bars, while the other strain gauge provided a highly noisy signal that was deemed anomalous. The CLZ crushing failure in the vicinity of the strain gauges would have likely resulted in compressive strains in the steel on the order of \(-3.0 \times 10^{-3}\). However, the inset in Figure 3.35 reveals that the gauges were outside of this highly compressed region, explaining the low readings.

Greater applied loading, and the corresponding higher shear forces developed in the west span during Phase II, resulted in evident engagement of the vertical shear reinforcement. Tensile strains at mid-height of the T-headed stirrups exhibited the strain profile in Figure 3.36.

The largest extension was registered at the third and fourth bars from the west support, while owing to net compressive stresses, the stirrups in the clamping regions did not become utilized at all. Peak tensile strain was found to be \(1.25 \times 10^{-3}\) at the fourth bar, which is about half of yield, although this is only representative of the average reinforcement strain at the location of the gauge. This is attributed to the gauge being located within the uncracked region of concrete. The large diagonal shear crack crossing the stirrups lends credence to the assumption that local steel stresses in the central three bars could have exceeded yield.
3.2.5 Displacement Response

The addition of minimal shear reinforcement significantly increased the deformation ductility of the PLS4000. Phase I of testing achieved an ultimate vertical displacement underneath the point load of 12 mm, corresponding to a deflection ratio of L/1570; Phase II of testing saw a full 39 mm (L/480) of downwards movement before failure occurred.

![Figure 3.37 – PLS4000 Phase II: vertical displacement profiles](image)

Formulated using linear interpolation functions between measured nodes, deformed shapes at the various stages of loading were created from the vertical LVDT data (See Figure 3.37). Widespread cracking along the specimen length resulted in the displaced profiles being much more rounded when compared to the failure in the east span that was characterized by a more bilinear shape.

Out-of-plane translations remained insignificant during testing and their response can be found in Appendix D.
3.2.5.1 Critical Loading Zone

The critical loading zone was tracked using two LVDTs measuring vertical and longitudinal movement over gauge lengths of about 1 m emanating from the edge of the load plate into the west span (Refer to Detail 1 in Figure 2.42). Load-deformation response of the CLZ illustrates the rapid progression of failure that occurred as a result of concrete crushing (See Figure 3.38).

Preceding the ultimate load, vertical translation of the CLZ began exhibiting signs of failure with a rounding off of the response. Further increase of displacements resulted in a lack of ability to carry additional load as depicted by the short plateau in Figure 3.38. During testing it was noted that vertical CLZ deformations were accelerating rapidly at this point while the longitudinal movement was essentially stagnant. Within a few seconds of reaching the ultimate load of 2162 kN, the load began decreasing and abrupt crushing of the CLZ ensued. The explosive failure resulted in the LVDTs exceeding their displacement capacity, but the data on the plot is a reliable record of pre-peak behaviour.
3.3 PLS300

The small companion slab strip specimen demonstrated a characteristic sectional shear failure with the major diagonal crack shaped much like the one for the PLS4000.

Figure 3.39 – Failure of PLS300

PLS300 behaviour is depicted in the load-deformation response below (Figure 3.40) and crack photographs shown in Figure 3.41.

![Load-deformation response graph](image)

Figure 3.40 – PLS300: Experimental load-deformation response
3.3.1 General Behaviour and Observations

The PLS300 load stage summary tabulated below followed by a brief account of the testing.

<table>
<thead>
<tr>
<th>Load Stage</th>
<th>P [kN]</th>
<th>Δ [mm]</th>
<th>P_{unload} † [kN]</th>
<th>Δ_{unload} ‡ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: 1st Crack</td>
<td>35.4</td>
<td>0.95</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>55.0</td>
<td>1.89</td>
<td>32.1</td>
<td>0.96</td>
</tr>
<tr>
<td>3</td>
<td>69.9</td>
<td>2.69</td>
<td>51.2</td>
<td>1.90</td>
</tr>
<tr>
<td>4</td>
<td>79.8</td>
<td>3.21</td>
<td>63.6</td>
<td>2.62</td>
</tr>
<tr>
<td>5</td>
<td>89.9</td>
<td>3.90</td>
<td>73.6</td>
<td>3.14</td>
</tr>
<tr>
<td>6*</td>
<td>94.8</td>
<td>4.26</td>
<td>83.2</td>
<td>3.80</td>
</tr>
</tbody>
</table>

* Ultimate load  † Average value of load readings within the entire timeframe from start to finish of the load stage  ‡ Displacement corresponding to P_{unload}

Elastic uncracked behaviour showed a nonlinearity early in the test at displacements of under P of 20 kN. This deviation from linear elastic response can be partly due to seating of the supports caused by the fact that the small specimen, unlike the PLS4000, was not heavy enough to engage full bearing prior to loading.

Flexural cracking in the small slab strip occurred at an applied load P of 35.4 kN, the first load stage of the test protocol. Before cracking, two pauses in loading were executed when the force in the actuator was at 20 kN and 30 kN in order to inspect the specimen; this is portrayed in Figure 3.40 as the subtle drops in response at those particular loads. Cracking occurred when the peak bending moment in the specimen was 15 kN-m, corresponding to a tensile stress in the concrete of 5.28 MPa, more than twice the stress required to crack the large slab strip.

Additional loading towards failure created a bilinear load-deformation response comparable to the PLS4000 tests, indicating a softened post-cracking stiffness of the specimen. Sectional shear failure occurred in a brittle manner when the applied load reached 94.8 kN. The shape of the primary diagonal shear crack bears resemblance to the failed east span of PLS4000 with an originating flexure-induced crack tracing an arc-like path towards the load plate.
Figure 3.41 – PLS300 crack photographs
Crack width at mid-depth of the small specimen measured 0.20 mm just preceding failure, compared to the much larger 0.75 mm cracks in the PLS4000 highlighting the basic premise of the size effect in shear.

It was discovered that potential torsional cracking that occurred could possibly explain the nonlinearity early in the load-deformation response. Upon inspection of the flexural tension face underneath the specimen, the failure crack was found to deviate in a diagonal path through the width of the PLS300 (See Figure 3.42).

![Figure 3.42 – Bottom face of PLS300 showing torsional deviation of failure crack](image)

Torsion in the member could have developed from the steel support plates plausibly being uneven or twisted, causing an imbalance of bearing stresses that develop over only a portion of the specimen. After first cracking, however, there was no evidence of significant twist or applied torsion.
3.3.2 Crack Diagrams

Figure 3.43 – PLS300 crack diagrams
3.3.3 Detailed Surface Displacement Plots

Utilizing the 3D CMM measurements, surface displacement plots for the PLS300 were created from the continuous LED coordinate data using a program written by the author on MATLAB©. Deformed shapes during the progression to failure are shown below in Figure 3.44 where displacements have been magnified for clarity.

*Figure 3.44 – PLS300: deformed shapes at 0, 25, 30, and 35 seconds after reaching peak load (displacements magnified x10)*
It can be seen that the shear failure involved significant distortion of the specimen in the vicinity of the main diagonal crack. The deformed shapes closely match the failure crack propagation observed during the testing.

Strain analysis was accomplished with Timeline®, an experimental data post-processing software that executes statistical error minimization with respect to an over-defined strain state resulting from the 3D coordinate data. This program was developed as part of the doctoral research of Ruggiero (2015); further information regarding the program and theory behind the Mohr’s circle fitting can be found in [36].

Figure 3.45 — PLS300 strain state and deformations from Timeline
Peak (load stage 6) & at 35 seconds after peak

Figure 3.45 illustrates the detailed strain states of the specimen at ultimate and at post-peak conditions. Mid-depth longitudinal strain at the critical section $d_c$ from the loading plate is $0.94 \times 10^{-3}$ at the ultimate load. It can be seen that the independent Timeline results validated the displacement profile obtained from the MATLAB program.
3.4 SUMMARY OF EXPERIMENTS

Test results from the PLS4000 and PLS300 slab strip specimens unmistakeably confirms the size effect in shear. The large slab strip failed at a shear stress which was only about 35% of that of the conventional sized shear specimen. This is accredited to the much wider crack spacings, almost four times the spacing in this particular case, exhibited by thick members without stirrups when compared to shallower members.

The presence of shear reinforcement in the west span allowed it to attain a significantly higher failure load than the east. This is also due to the shorter shear span bringing about an arching mechanism that engaged once sectional response began to break down. Although a direct comparison cannot be made between Phase I and II of the test program because of the different span-to-depth ratios, it can still be concluded that stirrups greatly enhanced the strength and ductility of the west end (See Figure 3.46).

Figure 3.46 – Comparison of PLS4000 load-deformation responses
Failure of the west span carried with it triple the deformation required to fail the unreinforced east. Shear reinforcement helps to control cracking, producing an overall wider dispersion of cracks over the length which also explains the more rounded vertical displaced shape as seen in Figure 3.47 below.

Figure 3.47 – Comparison of PLS4000 vertical displacements at failure

Further discussion on the experimental results will be reserved for the next two chapters which investigates the crack patterns and overall response in a more analytical light.
CHAPTER 4 SIZE EFFECT ANALYSIS

Analysis of the slab strip failures with respect to the size effect is made from the experimental results and compared against other relevant shear tests in this chapter. Crack patterns and spacings, as well as longitudinal strains are investigated to determine in detail how size effect affects the ultimate shear resistance of different sized members devoid of stirrups.

4.1 THE TORONTO SIZE EFFECT SERIES – REVISITED

Results from the PLS experimental program has contributed two key data points to the aforementioned Toronto Size Effect Series. Following suit with the format of the previous test data (See Table 1.1), the slab strip properties and results are delineated in Table 4.1.

Table 4.1 – Toronto Size Effect Series Data for PLS4000 and PLS300

<table>
<thead>
<tr>
<th>Name</th>
<th>$f'_c$ [MPa]</th>
<th>$a_g$ [mm]</th>
<th>$\rho$ [%]</th>
<th>$a/d$</th>
<th>$b_w$ [mm]</th>
<th>$d$ [mm]</th>
<th>$V_{exp}$ [kN]</th>
<th>$V_{MCFT}$ [kN/m]</th>
<th>$v_{exp}$ [MPa]</th>
<th>$v_{MCFT}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLS300</td>
<td>44.8</td>
<td>14</td>
<td>0.65</td>
<td>3.1</td>
<td>175</td>
<td>264</td>
<td>47</td>
<td>271</td>
<td>1.140</td>
<td>1.037</td>
</tr>
<tr>
<td>PLS4000</td>
<td>40.0</td>
<td>14</td>
<td>0.66</td>
<td>3.1</td>
<td>250</td>
<td>3840</td>
<td>350</td>
<td>1400</td>
<td>0.405</td>
<td>0.381</td>
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</tbody>
</table>

Reproduction of Figure 1.16 with the addition of the new data points is shown on the following page. It can immediately be seen that the MCFT-based CSA provisions continue to accurately predict the shear strength of members without stirrups even as depth increases to near the 4 m mark, twice that of the previously largest test of the series, YB2000/0. While CSA captures the realistic size effect strength degradation across the entire spectrum of member depths, ACI is shown to dangerously trend towards increasing overestimation with deeper specimens.

Figure 4.1 – Size effect in the PLS test series
Figure 4.1 highlights size effect in the PLS test series by juxtaposing the two very contrastingly sized slab strip specimens. Even at twice the actual dimensions, the PLS300 clearly displays much smaller crack spacings than its larger companion. As it was explained in Chapter 1, size effect manifests itself through cracks that are more spread out in larger members without shear reinforcement, precipitating wider cracks and reduced shear resistance.

![Graph](image)

*Figure 4.2 – The size effect in shear (Reproduced from Figure 1.16)*
4.2 Crack Investigation

A more detailed analysis into the crack patterns and their relation to the size effect will be presented in this section. Selected specimens previously tested from the Toronto Size Effect Series (Section 1.7) are brought into the evaluation to provide a comparison over the wide band of varying member depths.

Figure 4.4 on the next page shows that when uniformly scaled to the same height and length of shear span, crack patterns are fairly consistent between all specimens. It is deduced that most of the failure cracks initiate from the flexural crack nearest to the support regardless of the depth. This suggests that self-weight, although contributing to the varying utilization of shear capacity in different sized specimens, does not actually affect the crack patterns in shear critical members. Investigation into the crack spacing at various depths of the different specimens result in more interesting observations.

Average crack spacing at mid-depth (0.50h) is similar among all the experiments with a range from 0.4d to about 0.5d (See Table 4.2). The distance between the main failure crack and the next adjacent flexure-shear crack, deemed $S_{x,\text{crit}}$ in this thesis, was found to be larger than the average spacing at 0.82d but also remained essentially constant regardless of member depth. Crack spacings at 0.25h, 0.75h, and d were also determined from crack diagrams in order to visualize the longitudinal strain distribution over the height of the members. Failure crack angles measured 41 degrees on average within the mid-depth region, however, the two slab strip specimens had evidently higher inclinations than the other tests.

![Figure 4.3 – Longitudinal crack spacing measurements](image)
Figure 4.4 – Size effect crack pattern comparison
Table 4.2 – Mid-depth crack spacing data

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d [mm]</th>
<th>Sx/d*</th>
<th>Sx,avg [mm]</th>
<th>Sx,crit [mm]</th>
<th>θ† [°]</th>
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<td>0.58</td>
<td>0.38</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>YB2000/0†</td>
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<td>0.80</td>
<td>0.53</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>L-10N1§</td>
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<td>0.46</td>
<td>0.33</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.43</td>
<td>0.53</td>
<td>0.28</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>0.28</td>
<td>0.49</td>
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<td>L-10N2§</td>
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<td>0.28</td>
<td>0.58</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>0.56</td>
<td>0.42</td>
<td>-</td>
</tr>
<tr>
<td>S-10N1§</td>
<td>280</td>
<td>0.42</td>
<td>0.47</td>
<td>0.58</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.58</td>
<td>0.28</td>
<td>-</td>
</tr>
<tr>
<td>S-10N2§</td>
<td>280</td>
<td>0.23</td>
<td>0.58</td>
<td>0.60</td>
<td>0.46</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>0.50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PLS300</td>
<td>264</td>
<td>0.80</td>
<td>0.52</td>
<td>0.43</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.36</td>
<td>0.30</td>
<td>0.43</td>
</tr>
</tbody>
</table>

Average: 0.46d 0.82d 41.4

* Crack spacings measured from right (critical failure crack) to the left of specimen as per Figure 4.4; when the second crack joins the critical crack, Sx is taken as S1 + S2
† Angle of diagonal failure crack measured at mid-depth from the longitudinal axis [degrees], taken as a secant value between the crack crossing of the 0.25h and 0.75h horizontals (See Figure 4.3)
‡ Crack pattern created from [18] using methods explained in Chapter 3
§ Crack patterns created from [9] using methods explained in Chapter 3

Plotting the various spacings in a bar graph results in Figure 4.5 illustrating, first of all, the effect of tension stiffening on tightening up crack spacings at the level of the flexural reinforcing bars. As cracks traverse towards mid-depth, gaps between cracks grow due to increasing distance from the tension-stiffened region. The flexural compression zone, typically within the top quarter height of the specimens, reverses the trend and causes a reduction of crack spacing because of compressive stress trajectories tending towards the load.

Maximum crack spacing is observed to occur at the mid-depth of members since this is the location of highest unrestrained tensile straining furthest from the reinforcing steel and the compression region. It is acknowledged that the distance between layers of flexural rebar rather than the absolute depth of the member that is the underlying cause of the size effect. However, to remain consistent with the Size Effect Series that omit both shear and distributed longitudinal bars (skin reinforcement), crack spacing is appropriately defined as only a function of d. Previous experimental studies revealed that the presence of stirrups and crack control reinforcement negates the size effect through reduced crack spacings (Recall Figure 1.15) [37].
Neglecting the small concrete elastic strain between cracks, widths can be estimated by the simple expression relating longitudinal tensile strain due to stress and mean crack spacing:

\[ w_m = \varepsilon_x \times S_x \ [mm] \]  

[4.1]

Where:

\[ S_x \approx C \times d \ [mm] \]

[4.2]

A fairly consistent trend in Figure 4.5 lends credence to the fact that reinforced concrete members without shear or skin reinforcement have crack spacings directly proportional to member depth. As a result, when two different sized specimens are subject to the same stresses, and thus strains (i.e. \( \varepsilon_x \)), the larger member will exhibit wider cracks.

Although this trend is evident at all locations over the height of the member, it is better defined over the entire range of member sizes, particularly at mid-depth. The smaller tests did not exhibit a large change in crack spacing from 0.75h to d and that can be explained by the shorter distance between those two positions when compared to the large specimens simply because of the greater influence of cover requirements on shallower members.
CHAPTER 4

SIZE EFFECT ANALYSIS

Using this knowledge of crack spacing correlation with member depth in combination with the Toronto Size Effect Series data, crack width estimates can be compared against experimental results. Table 4.3 shows such a comparison and despite a few specimens demonstrating a deviation from the trend, the size effect phenomenon is quite evident.

Table 4.3 – Crack width comparison with experimental results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d [mm]</th>
<th>$S_x^*$ [mm]</th>
<th>$\varepsilon_x^\dagger$</th>
<th>$w_m^\ddagger$ [mm]</th>
<th>$w_{exp}^§$ [mm]</th>
<th>$w_{exp}^\parallel / w_m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLS4000</td>
<td>3840</td>
<td>1766</td>
<td>0.381</td>
<td>0.67</td>
<td>0.75</td>
<td>1.12</td>
</tr>
<tr>
<td>YB2000/0**</td>
<td>1890</td>
<td>869</td>
<td>0.557</td>
<td>0.48</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>L-10N1</td>
<td>1400</td>
<td>644</td>
<td>0.637</td>
<td>0.41</td>
<td>0.40</td>
<td>0.98</td>
</tr>
<tr>
<td>L-10N2</td>
<td>1400</td>
<td>644</td>
<td>0.637</td>
<td>0.41</td>
<td>0.35</td>
<td>0.85</td>
</tr>
<tr>
<td>S-10N1</td>
<td>280</td>
<td>129</td>
<td>0.993</td>
<td>0.13</td>
<td>0.05</td>
<td>0.39</td>
</tr>
<tr>
<td>S-10N2</td>
<td>280</td>
<td>129</td>
<td>0.993</td>
<td>0.13</td>
<td>0.10</td>
<td>0.78</td>
</tr>
<tr>
<td>PLS300</td>
<td>264</td>
<td>121</td>
<td>1.001</td>
<td>0.12</td>
<td>0.20</td>
<td>1.64</td>
</tr>
</tbody>
</table>

* Determined from Equation [4.2] using $C = 0.46$
† Determined from CSA calculations used in Figure 4.2; specimen properties normalized to those shown in the figure inset
‡ Determined from Equation [4.1]
§ Experimental value of maximum mid-depth crack width just prior to failure found in [18] and [9]
** Not measured before failure; previous load stage not indicative of pre-failure state

Recalling the 3D coordinate data from Chapter 3, the PLS300 had a longitudinal mid-depth strain of $0.94 \times 10^{-3}$ which is quite close to the CSA estimated value of $1.00 \times 10^{-3}$ provided in Table 4.3.
The research comprised in this thesis was conducted to assess the shear response of large-scale reinforced concrete structures by investigating, in particular, thick slabs. It was revealed earlier that procedures in various design codes treat shear resistance inconsistently, possibly resulting in huge differences in their capacity estimates. This chapter will therefore focus on the evaluation of code-based as well as analytical predictions to appreciate the manner in which they compare to the experimental results.

Table 5.1 presents the comparison summary of various predictions to the observed outcomes for both PSL4000 and PLS300 specimens. Predictions were calculated using the provisions formerly examined in Chapter 1.

Table 5.1 – Comparison of code and analytical load predictions with experimental results

<table>
<thead>
<tr>
<th>Prediction</th>
<th>PLS4000 East $P_{exp} = 685$ kN</th>
<th>PLS4000 West $P_{exp} = 2162$ kN</th>
<th>PLS300 $P_{exp} = 95$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSA A23.3-14 – Sectional: General Method</td>
<td>620 1.10</td>
<td>1485 1.46</td>
<td>81 1.17</td>
</tr>
<tr>
<td>CSA A23.3-14 – Sectional: Simplified Method</td>
<td>361 1.90</td>
<td>2198* 0.98</td>
<td>98 0.96</td>
</tr>
<tr>
<td>CSA A23.3-14 – Strut &amp; Tie</td>
<td>417 1.64</td>
<td>1660 1.30</td>
<td>61 1.55</td>
</tr>
<tr>
<td>AASHTO LRFD</td>
<td>640 1.07</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ACI 318-14</td>
<td>2505 0.27</td>
<td>2090 1.03</td>
<td>102 0.93</td>
</tr>
<tr>
<td>fib Model Code 2010</td>
<td>702 0.98</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Eurocode 2 (EC2) 2004</td>
<td>1351 0.51</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Australia AS 3600-2009</td>
<td>1002 0.68</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>VecTor2</td>
<td>721 0.95</td>
<td>1969 1.10</td>
<td>73 1.29</td>
</tr>
<tr>
<td>Response-2000</td>
<td>704 0.97</td>
<td>1412 1.53</td>
<td>81 1.17</td>
</tr>
<tr>
<td>Flexural Failure</td>
<td>2730 0.25</td>
<td>2730 0.79</td>
<td>92 1.03</td>
</tr>
</tbody>
</table>

* Detrimental effect of widely spaced stirrups on west span neglected ($s_{ee} = 300$ mm)
A re-evaluation of the moment-shear interaction curves presented at the end of Chapter 2 can be done with the test results now at hand (See Figure 5.1 below).

![Figure 5.1 – Shear strength prediction comparison for PLS4000 (left) & PLS300 (right) (Reproduced from Figures 2.55 & 2.56)](image)

Failure of the PLS4000 is well predicted by the Canadian code, as it directly accounts for the size effect and strain effect on shear resistance. On the contrary, ACI 318 overestimates the strength of the thick slab by a margin much larger than what typical safety factors allow for. Both North American design procedures are accurate in determining failure of conventionally sized members.

Analytical calculations were also performed to provide corroborating analyses to the standard code predictions. The various levels of approximation used for both the analytical models and the CSA code-based equations are illustrated in Figure 5.2 on the ensuing page.
The first type of calculation uses the program Response-2000© which is based on a fiber model approach to sectional analysis of reinforced concrete [38]. Response-2000 utilizes MCFT formulations for the biaxial stress-strain behaviour through member depth in a layer-by-layer methodology where elements are stacked over the height of the specimen. VecTor2©, a nonlinear finite element analysis program, was used as the second analytical method for predicting shear strength. The VecTor2 processor incorporates the MCFT material constitutive relations directly in the formulation of the elemental stiffness matrices as per standard finite element procedures [39]. Analytical results from both Response-2000 and VecTor2 are briefly discussed in this chapter while the detailed aspects can be found in Appendix G.
5.1 Prediction Competition

A prediction competition regarding the failure of the PLS4000 was held to assess the ability of the engineering profession in accurately estimating the shear response of thick slabs. The challenge, issued out to both practicing engineers in the industry as well as members from academia, was to determine the following:

1. The magnitude of the applied point load, $P$, required to cause failure of the specimen $[P_{\text{ult, East}}]$;
2. The location where the first failure would occur;
3. The magnitude of $P$ required to cause specimen failure if the west shear span had identical shear reinforcement as the east span $[P_{\text{ult, West}}]$; and
4. The load-deformation response of the actual specimen ($\Delta$ underneath the point load @ 0.25P, 0.50P, 0.75P, and 1.00P $[\Delta_{\text{ult, East}}]$)

A total of 66 predictions, with a coincident 33 of which coming from industry and the remaining 33 from universities, were made from around the world. 26 predictions originated from Europe, 23 from the United States, 14 from Canada, and 1 each from Australia, Brazil, and Mexico. A wide variety of analysis methods were used by the participants ranging from complicated nonlinear finite element methods to simpler sectional models, as well as numerous straightforward code forecasts. Entries were all collected prior to first loading of the slab strip, thus making them true predictions.

Only two entries predicted the failure loads of both shear spans within 10% of the experimental values. These predictions were submitted by Červenka Consulting and by SNC-Lavalin Hydro in collaboration with École Polytechnique de Montréal, Canada; both submissions utilized their own in-house finite element programs. Červenka Consulting was deemed the most accurate pertaining to all aspects of the challenge, and was fittingly chosen as the overall winners of the prediction competition.
CHAPTER 5  COMPARISON OF EXPERIMENTAL, CODE-BASED, AND ANALYTICAL PREDICTIONS

A statistical summary of the predictions is outlined in Table 5.2 below.

<table>
<thead>
<tr>
<th>Statistical Parameter</th>
<th>$P_{ult, East}$ [kN]</th>
<th>$\Delta_{ult, East}$ [mm]</th>
<th>$P_{ult, West}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>3773</td>
<td>140</td>
<td>3997</td>
</tr>
<tr>
<td>Minimum</td>
<td>250</td>
<td>5</td>
<td>820</td>
</tr>
<tr>
<td>Average</td>
<td>1155</td>
<td>20</td>
<td>2031</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>724</td>
<td>22</td>
<td>650</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>63%</td>
<td>108%</td>
<td>32%</td>
</tr>
</tbody>
</table>

The subsequent sections in this chapter shall discuss the prediction competition results pertaining to each respective test accompanied by the relevant code and analytical predictions.
5.2 **PLS4000 Phase I – East Span Failure**

Comparison of the submitted failure load predictions by engineers and the experimental result of the PLS4000 are illustrated in Figure 5.3. Furthermore, the plot shows the values of shear resistance as predicted by six different national code provisions.

![Comparison of point load predictions to cause PLS4000 east span shear failure](image)

*Figure 5.3 – Comparison of point load predictions to cause PLS4000 east span shear failure*
Significant scatter can be instantly noticed with a fairly uniform distribution of predictions across the entire range. This indicates that forecasting the shear strength of very thick slabs without shear reinforcement is indeed a challenge for the structural engineering profession. The bands of color in the figure correspond to prediction accuracy with respect to the experimental outcome. The upper red zone encompasses very unconservative estimates of shear strength, with ratios of predicted failure load to observed failure load ranging from 1.5 to 5.5. The overall factor of safety in structures is typically 1.5 to 2.0 when using LRFD procedures that take into account the statistical uncertainty of material strengths and applied loads. An overestimate in capacity beyond that range can be expected to cause collapse at service loads. Very precise predictions are demarcated by the yellow band, representing the “gold standard” prediction range of ±10% from the experimental failure load. Based on these metrics, 8 of the industry predictions – 5 from academia, and 3 code-based predictions – were considered to be excellent. It is quite disconcerting however, that only 20% of the submissions are deemed to be very accurate while 44% of entries and 2 very influential shear design procedures, ACI and EC2, lie in the red zone and therefore can be dangerously unconservative.

In addition to the magnitude of P required to fail the PLS4000, the prediction competition also challenged contestants to determine the load-deformation response of the specimen with points determined at specified ratios of the ultimate load. Out of the 66 total entries, 36 submitted monotonic pushover curves for the slab strip with 13 coming from industry and 23 from academia. Figure 5.4 compares load-deformation predictions with the response of the actual experiment as well as linear elastic response estimates mentioned previously in Chapter 3. The initial uncracked elastic prediction agrees with the observed behaviour as expected. However, the cracked elastic line, offset with greater self-weight deflection (Δ_s/w \text{cracked} = 3 \text{ mm} \text{ vs. } Δ_s/w \text{uncracked} = 1 \text{ mm}), does not capture the realistic post-cracking stiffness. This is because despite the cracked moment of inertia accounting for a completely flexurally cracked member, it neglects shear deformations which in the case of a shear-critical deep specimen such as the thick slab strip can significantly soften the response. After first cracking in the experiment, the load-deformation response continued with a linear stiffness that was only 19% of the observed uncracked slope whereas the linear elastic estimate dictated an inaccurately high 33.5% of the uncracked stiffness.
A yellow region surrounding the experimental response illuminates a “cone of excellence” where deflections at specified loads were within ±20% of the measured values, indicating an accurate assessment of post-cracking stiffness. 5 of the predictions lie within this area, while 2 stay well below and 18 remain above representing underestimates and overestimates of stiffness respectively. The remaining 11 responses intersect the cone owing largely to initial degrees of post-cracking stiffness being miscalculated as too high.

Only 3 submissions of load-deformation predictions satisfied the strict requirements of having the failure load within ±10% and the four deflections within ±20% of the experimental values.
These predictions were submitted by: Červenka and Sajdlova from Červenka Consulting in Prague (Czech Republic), Conforti and Facconi from the University of Brescia in Brescia (Italy), and Bentz from the University of Toronto (Canada). The first 2 submissions used nonlinear finite element method based analyses, and the third utilized a newly developed version of the sectional analysis program Response© [38].

As the challenge also requested the location of failure to be determined by participants, failure crack patterns were also assessed. Červenka and Sajdlova provided the most accurate depiction of the specimen at failure, as shown by the inset in Figure 5.4.

It is worth noting that 6 of the contest entries inaccurately predicted an initial failure occurring in the shorter west shear span on account of larger shear demand, while 2 anticipated loads even higher than a ductile flexural failure which would have been well in excess of the observed failure load if the member had not been shear critical. Consequently, these 8 submissions concluded that the addition of shear reinforcement in the east span would not increase the failure load for the specimen and resulted in great overestimates of predictions.

With regards to the CSA provisions, the general method resulted in the most accurate prediction of observed failure load. The simplified method proved to be more conservative as it takes $\varepsilon_x$ as a value associated with flexural failure. Strut-and-tie analysis using A23.3-14 also underestimated the failure load, but intuitively so, as arch action was anticipated to break down in the longer east span and give rise to sectional behaviour.

ACI 318-14 erroneously estimated that first failure would occur in the west because of the higher shear force demand in combination with a lack of proper account for the beneficial effects of shear reinforcement in thick slabs.
5.2.1 Analytical Predictions

Corroborating analyses were made in addition to the code-based predictions to assess the accuracy of these more theoretically based methods. Figure 5.5 illustrates the different load-deformation responses from VecTor2 and Response-2000 compared against the experimental behaviour.

![Figure 5.5 – Load-deformation comparison of analytical predictions for PLS4000 east span](image)

It can be seen that both analytical forecasts embody the overall response of the specimen quite well with both linear elastic and post-cracking stiffness being well represented. However, the finite element analysis overestimates the cracking load by more than a factor of two. The model actually predicts first cracking accurately, but it is rather the transition point to the softened post-cracking response that is incorrectly captured. This can be explained by the fact that cracking in concrete with little reinforcement, such as the case for the PLS4000, is governed by fracture mechanics. Plain concrete over the depth of the slab strip exhibits almost instantaneous crack propagation due to the low fracture energy. As a result, the first flexural
crack shot up to the compression zone within less than a second during testing, whereas VecTor2 predicts a much slower propagation of cracking.

Full member response based on sectional analyses executed by Response-2000 (Bentz) follows the observed behaviour very well as mentioned previously in the discussion of the prediction competition.

Figure 5.6 – Crack patterns for PLS4000 east span failure (from top to bottom: VecTor2, Response-2000, & observed)

VecTor2 provided a reasonably accurate depiction of the cracking in the slab strip, closely representing the observed crack patterns as seen in Figure 5.6. Response-2000 predicts the extents of cracking along the span quite well, although the diagonal failure crack was not captured. The reason for this is that full crack patterns are difficult to determine from the simpler methodology of a sectional analysis program when compared to more intricate finite element tactics. Response-2000 also defines failure as the load where a crack at any depth first slips, leading to ultimate conditions being reached without capturing any post-peak behaviour. However, experiments and VecTor2 often show a kinematic mechanism forming after this point as the crack slips and propagates up into the compressive zone of a member.
5.3 PLS4000 Phase II – West Span Failure

The comparison of predictions for the west shear span failure of the thick slab strip are shown in Figure 5.7. A total of 44 submissions were made by engineers and academics; the color bands in the plot below are equivalent to those in Figure 5.3 with yellow signifying excellent values and red designating highly unconservative ones.

*Figure 5.7 – Comparison of point load predictions to cause PLS4000 west span shear failure*
A comparison with the equivalent figure for the east span illustrates that a much smaller amount of predictions lie in the dangerous red zone for this phase of testing. The east shear span had nearly half of the entries (29 out of 66) within the red zone compared to only a single entry that was highly overestimated for the west. Additionally, only 24% of the east span estimates were conservative whereas a much larger 66% percent of entries were conservative for the west. 10 predictions lie within the “gold standard” zone for the second phase of testing with 5 coming from industry, 4 from academia, and 1 representing the ACI 318-14 value.

Due to a shorter shear-span-to-depth ratio for the west span (a/d < 2), a strut-and-tie (S&T) mechanism was predicted to potentially govern the shear failure. In view of that, the CSA A23.3-14 strut-and-tie model was used in addition to the sectional model of beam-action shear to determine two different resistances as presented in Figure 5.7. The small discrepancy of only 11% between the S&T and sectional values signifies that the span was not quite short enough to have strut action dominate the response. Nevertheless, it can still be appreciated that strut-and-tie models give higher and more accurate estimates of failure loads for thick members with short spans relative to their depths.

Despite not being shown on Figure 5.7, the CSA Simplified Method estimated an excellent experimental-to-observed load ratio of 0.98 when the detrimental effect of widely spaced stirrups was neglected.
5.3.1 Analytical Predictions

The west span of the PLS4000 was predicted to be governed by a strut-and-tie mechanism of failure. Sectional approaches would likely underestimate capacity, and thus the only analytical corroboration for the second phase of testing was accomplished with VecTor2.

Load-deformation response as predicted by the finite element analysis match the observed behaviour reasonably well, with the exception of post-cracking stiffness being somewhat softer than reality (See Figure 5.8). This can be explained by the fact that in modelling the specimen, the shear reinforcement present in the east span was assumed to also exist in the west. In reality however, the east was over strengthened using post-tensioning that would logically lead to a stiffer response than the simulated internal minimum reinforcement.

![Graph](image)

*Figure 5.8 – Load-deformation comparison of analytical predictions for PLS4000 west span*

Furthermore, the conservative VecTor2 prediction is justified by the assumed cylinder strength of 40 MPa to keep consistent with the information provided to contestants at the time of the prediction competition; recall that $f'_{c}$ was actually 44 MPa at testing.
One of the observed failure cracks traced an almost direct path from the edge of the reaction plate to the load plate where concrete crushing took place. This particular crack was not predicted at failure by either VecTor2 or Response-2000 (See Figure 5.9). The other failure crack that formed below the main diagonal one during the experiment is estimated by the finite element model in a fairly accurate manner.

*Figure 5.9 – Crack patterns for PLS4000 west span failure (from top to bottom: VecTor2, Response-2000, & observed)*
5.4 PLS300

The more traditionally sized slab strip specimen was not explicitly part of the prediction competition, nevertheless, it provides a direct comparison to the ACI code that is empirically determined from tests similar in scale. Contrasted against the large slab strip, the two North American code predictions are much more similar, indicating that size effect does not play a role in diminishing shear capacity for such a shallow member.

5.4.1 Analytical Predictions

Chapter 3 described the initial elastic response of the PLS300 specimen as having abnormal nonlinearity likely due to support seating. Figure 3.6 exemplifies this anomaly through a visual offset apparent between the two analytical predictions and the observed response.

Close inspection confirms that extrapolation of the experimental data back to the horizontal axis from the linear portion of the elastic response can negate the anomalous nonlinearity. A corresponding displacement shift to offset the plot back to the origin results in a more representative response of anticipated behaviour to a certain degree.
Both detailed finite element and the relatively simpler sectional analyses yield almost identical results in the case of the traditional sized specimen. Given that peak load was underestimated by the two software packages, predictions are still practically accurate but more importantly, conservative.

*Figure 5.11 – Crack patterns for PLS300 (from top to bottom: VecTor2, Response-2000, & observed)*

VecTor2 once again predicts crack patterns fairly well for the small slab strip as evident in Figure 5.11 above.
CHAPTER 6  CONCLUSIONS AND RECOMMENDATIONS

The research comprised in this thesis has revealed that is challenging to predict the shear strength of very thick reinforced concrete elements that do not contain stirrups. There is a large disparity between many design procedures and analytical methods around the world in being able to accurately capture the response of large members subject to shear.

Engineers are recommended to use at least minimum shear reinforcement to eliminate the size effect in thick members, as they often serve as critical components in the load-carrying system of a structure. Even if conventional spacing limits are exceeded, stirrups can triple the shear strength of thick slabs, increase ductility, and completely transform the overall behaviour of a member. Headed bars have become an effective means of providing shear reinforcement in thick slabs, enabling practical constructability, something that was previously unattainable with standard hooked rebar used in the past.

It is imperative that design procedures around the world account for the size effect in shear in order to yield safe designs of large-scale infrastructure.
CHAPTER 7  REFERENCES


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APPENDIX A  ENGINEERING DRAWINGS

Engineering drawings containing more detailed information than the illustrated schematics in the main body of the thesis are located in this appendix. The complete list of drawings is presented in Table A.1 below. These drawings are intended to be plotted on 11 x 17 (ANSI D) paper for true representation of scale.

A.1  DRAWING LIST

Table A.1 – Engineering Drawing List

<table>
<thead>
<tr>
<th>DWG #</th>
<th>Drawing Title</th>
<th>Rev #</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-000</td>
<td>PLS300 – Construction Drawing and Experimental Setup</td>
<td>R2</td>
</tr>
<tr>
<td>S-001</td>
<td>PLS4000 – Construction Drawing</td>
<td>R2</td>
</tr>
<tr>
<td>S-001-AB</td>
<td>PLS4000 – As-Built Conditions</td>
<td>R0</td>
</tr>
<tr>
<td>S-002</td>
<td>PLS4000 Experimental Setup – General Arrangement (1 of 3)</td>
<td>R2</td>
</tr>
<tr>
<td>S-003</td>
<td>PLS4000 Experimental Setup – General Arrangement (2 of 3)</td>
<td>R3</td>
</tr>
<tr>
<td>S-004</td>
<td>PLS4000 Experimental Setup – General Arrangement (3 of 3)</td>
<td>R1</td>
</tr>
<tr>
<td>S-005</td>
<td>PLS4000 Reinforcement – Details and MTO</td>
<td>R2</td>
</tr>
<tr>
<td>S-006</td>
<td>PLS4000 Instrumentation – Strain Gauge Details</td>
<td>R2</td>
</tr>
<tr>
<td>S-007</td>
<td>PLS4000 Instrumentation – LVDT, Linear Pot, Load Cell Details</td>
<td>R2</td>
</tr>
<tr>
<td>S-008</td>
<td>PLS4000 Instrumentation – Phase I Surface LED Target Details</td>
<td>R2</td>
</tr>
<tr>
<td>S-009</td>
<td>PLS4000 Instrumentation – Phase II Surface LED Target Details</td>
<td>R0</td>
</tr>
<tr>
<td>S-010</td>
<td>PLS4000 Experimental Setup – Phase II East Span Repair</td>
<td>R0</td>
</tr>
<tr>
<td>S-100</td>
<td>Misc. – Load Plate Details</td>
<td>R0</td>
</tr>
<tr>
<td>S-101</td>
<td>Misc. – Lateral Restraint HSS Cut Schematic</td>
<td>R0</td>
</tr>
<tr>
<td>S-102</td>
<td>Misc. – Bottom Reinforcement Spacer Block Details</td>
<td>R1</td>
</tr>
<tr>
<td>S-103</td>
<td>Misc. – Lateral Restraint HSS End Plates</td>
<td>R0</td>
</tr>
</tbody>
</table>
1. Use this block construction drawing for complete specimen details.
2. All notes in small caps.
3. Use of red targets was necessary due to technical limitations with the original targets used in phase I of testing.
4. Test hidden due to:
   4.1. Range of camera
   4.2. Length of API(3) targets used
5. (API) & (API(3)) targets did not record data.
APPENDIX B  TIME-LAPSE PHOTOGRAPHS

A wide angle lens GoPro® digital camera was used during the entire duration of the project to document overall progress. It was mounted on the shell element tester platform and time-lapse photographs were captured during construction, testing, and demolition as summarized in this appendix. A video showing the complete project time-lapse can be found at the following link: https://youtu.be/VQ2_tBF8mBs.

Figure B.1 – GoPro camera setup on elevated platform
B.1 Formwork Construction

![Figure B.2 – Laboratory strong floor prior to formwork delivery](image1)

![Figure B.3 – Assembly of first panel group starting on west end](image2)
Figure B.4 – Using overhead crane to move assembled formwork

Figure B.5 – Placement of first formwork assembly
Figure B.6 – Subsequent north face panel placement (1 of 3)

Figure B.7 – Subsequent north face panel placement (2 of 3)
Figure B.8 – Subsequent north face panel placement (3 of 3)

Figure B.9 – North (back) face and base formwork placement complete
Figure B.10 – Flexural tension reinforcement placement after form-release agent application on formwork

Figure B.11 – Shear reinforcement placement
Figure B.12 – Beginning of south (front) face formwork construction

Figure B.13 – Subsequent south face panel placement (1 of 2)
Figure B.14 – Subsequent south face panel placement (2 of 2)

Figure B.15 – South (front) face formwork placement complete and concrete casting preparation
B.2 CONCRETE CAST

Figure B.16 – Concrete mobile pump truck unfolding boom

Figure B.17 – Concrete pumping in east span
Figure B.18 – Concrete pumping in west span

Figure B.19 – Concrete pump line flushing
B.3 Formwork Removal

Figure B.20 – Beginning of south face formwork removal

Figure B.21 – South face formwork removal complete
Figure B.22 – North face formwork removal

Figure B.23 – North face formwork removal complete
B.4 Jacking Procedure

Figure B.24 – Jacking preparation

Figure B.25 – Jacking complete
B.5 Testing

Figure B.26 – Load stage crack identification

Figure B.27 – Load stage crack marking
Figure B.28 – Phase I east span failure (LS 5)

Figure B.29 – Phase I post-peak reloading failure (LS 6)
Figure B.30 – Repairing east span for Phase II of testing

Figure B.31 – East span repair complete
Figure B.32 – Hydraulic pump replacement for Phase II ultimate load

Figure B.33 – Loading up to ultimate load for Phase II
Figure B.34 – Phase II west span failure (LS 11)

Figure B.35 – Test completion instrumentation removal complete
B.6 DEMOLITION

Figure B.36 – Demolition of PLS4000 using concrete cutting saw

Figure B.37 – Removal of cut segment with overhead crane
Figure B.38 – Lifting demolished segment into disposal bin on loading dock

Figure B.39 – Demolition complete
APPENDIX C  DETAILED MATERIAL PROPERTIES

This appendix summarizes detailed material data along with information regarding the methodologies for both the concrete cylinder compressive tests and reinforcement coupon tensile tests.
C.1  CONCRETE CYLINDER TEST DATA – STRENGTH DEVELOPMENT SUMMARY

Concrete properties were obtained from cylinders nominally 150 mm in diameter and 300 mm high. Actual dimensions along with the mass of the cylinders were measured prior to testing and used to calculate stresses and strains. Average results using two cylinders (A and B) from each of the three concrete trucks were used for all tests.

Table C.1 – Concrete Cylinder Strength Development

<table>
<thead>
<tr>
<th>Day (Year 2015)</th>
<th>Truck 1 (T1)</th>
<th>Truck 2 (T2)</th>
<th>Truck 3 (T3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age [Days]</td>
<td>Cyl A</td>
<td>Cyl B</td>
<td>Avg</td>
</tr>
<tr>
<td>0</td>
<td>04/27</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>05/04</td>
<td>27.01</td>
<td>26.80</td>
</tr>
<tr>
<td>14</td>
<td>05/11</td>
<td>32.66</td>
<td>33.99</td>
</tr>
<tr>
<td>22</td>
<td>05/19</td>
<td>37.25</td>
<td>35.79</td>
</tr>
<tr>
<td>28</td>
<td>05/25</td>
<td>38.45</td>
<td>41.99</td>
</tr>
<tr>
<td>35</td>
<td>06/01</td>
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<td>39.09</td>
</tr>
<tr>
<td>49*</td>
<td>06/15</td>
<td>42.82</td>
<td>40.61</td>
</tr>
<tr>
<td>80*</td>
<td>07/16</td>
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<tr>
<td>100</td>
<td>08/05</td>
<td>45.56</td>
<td>44.06</td>
</tr>
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</table>

* Calculated as follows: $f_{c, max \text{ avg}} = \frac{3}{8}(f_{c, max \text{ T1}} + f_{c, max \text{ T2}}) + \frac{1}{4}(f_{c, max \text{ T3}})$

† Test results determined from complete stress-strain response of cylinder

Figure C.1 – Average concrete strength development over time
## Appendix C

### Detailed Material Properties

#### C.2 Complete Concrete Cylinder Test Data

Table C.2 – Concrete Cylinder Test Data

<table>
<thead>
<tr>
<th>Age [Days]</th>
<th>Date [MM/DD]</th>
<th>Truck</th>
<th>Cylinder</th>
<th>( \Phi_{avg} ) [mm]</th>
<th>( h_{avg} ) [mm]</th>
<th>Mass [kg]</th>
<th>( \rho ) [kg/m(^3)]</th>
<th>Peak Load [kN]</th>
<th>Peak Stress [MPa]</th>
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</thead>
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<td>$h_{AVG}$ [mm]</td>
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<td>$\rho$ [kg/m$^3$]</td>
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<td>Peak Stress [MPa]</td>
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</tbody>
</table>

* Test results determined from complete stress-strain response of cylinder
APPENDIX C

DETAILED MATERIAL PROPERTIES

Using actual as-cast dimensions, cylinders tested in the MTS Stiff Frame axial test machine provided the full concrete compressive stress-strain response up until at least peak, in addition to the corresponding material parameters presented below in Table C.3.

Table C.3 – Concrete Material Properties

<table>
<thead>
<tr>
<th>Age [Days]</th>
<th>Date [MM/DD]</th>
<th>Truck</th>
<th>Cylinder</th>
<th>$f'_c$ [MPa]</th>
<th>$\varepsilon'_c$ * [mm/m]</th>
<th>$E_c$ † [MPa]</th>
<th>$f_{c,ult}$ ‡ [MPa]</th>
<th>$\varepsilon_{c,ult}$ ‡ [mm/m]</th>
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<td>A</td>
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<td>34.27</td>
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* Strain at peak compressive stress, $f'_c$
† Determined using ASTM C 469 Static Method (See Equation [C.1])
‡ Simply the values occurring at the end of the response, not necessary corresponding to ultimate conditions, as test was terminated early in the case of the Day 80 cylinders
§ Calculated for each parameter, $X_c$, as follows: $X_{c,\text{max}\text{avg}} = \frac{3}{8}(X_{c,T1} + X_{c,T2}) + \frac{1}{4}(X_{c,T3})$

ASTM C 469 (2002) states that the modulus of elasticity is calculated by:

$$E_c = \frac{S_2 - S_1}{\varepsilon_2 - 0.000050} \text{ [MPa]}$$  [C.1]

More information regarding the ASTM methodology can be found in [40].
It is apparent from the concrete constitutive relations that Day 80 behaviour does not show much post-peak response compared to the Day 49 results. This can be attributed to scheduling constraints in the laboratories that necessitated early termination of the cylinder test shortly after peak attainment in order to save time.

Figure C.2 – Concrete stress-strain response, day 49 [2015/06/15]

Figure C.3 – Concrete stress-strain response, day 80 [2015/07/16]
C.3 Reinforcement Coupon Data

Three coupons were tested from each batch of steel to obtain material properties. A complete set of reinforcing bar constitutive relations and the corresponding properties acquired from them are reported in this section. Material properties extracted from the stress-strain responses were done so with the methodology discussed below.

Yield stress ($f_y$) was obtained from a mathematical average of the values within the yield plateau of the response. The corresponding yield strain ($\varepsilon_y$) was taken as the strain at which the initial elastic stiffness, $E_s$, intersects the yield plateau as follows:

$$\varepsilon_y = \frac{f_y}{E_s} \quad [C.2]$$

It is noted that this is analogous to the Autographic Diagram Method found in the ASTM A370 Standard for mechanical testing of steel [41].

Young’s modulus ($E_s$) was determined from the average tangent stiffness of all data points between 20% and 80% of the yield stress. Deviations from the theoretical value of 200 GPa are largely associated with calculations being made on the assumed nominal cross-sectional bar area which often differs from the actual area of each individual bar. An alternative method of calculating $E_s$ is to utilize linear regression through the data up until about 25% of the yield strength, constrained to pass through the origin. This gave rise to very similar results to the tangent stiffness technique for a particular batch of steel (PLS300 $p_x$) and was not used for any further corroboration.

The strain at which strain hardening occurs ($\varepsilon_{sh}$) is denoted as the point where the yield plateau ends and the tensile response begins to have a nonlinear, nonzero, positive stiffness.

Ultimate stress ($f_{ult}$) and corresponding strain ($\varepsilon_{ult}$) are simply the values at the peak stress of the entire response.
Table C.4 – Reinforcement Material Properties, PLS4000 ρx Heat 1 [500W, 30M]

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<th>$f_y$ [MPa]</th>
<th>$\varepsilon_y$ [mm/m]</th>
<th>$E_s$ [MPa]</th>
<th>$\varepsilon_{sh}$ [mm/m]</th>
<th>$f_{s,ult}$ [MPa]</th>
<th>$\varepsilon_{s,ult}$ [mm/m]</th>
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Figure C.4 – Reinforcement stress-strain response, PLS4000 ρx Heat 1

Table C.5 – Reinforcement Material Properties, PLS4000 ρx Heat 2 [500W, 30M]

<table>
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<th>$\varepsilon_y$ [mm/m]</th>
<th>$E_s$ [MPa]</th>
<th>$\varepsilon_{sh}$ [mm/m]</th>
<th>$f_{s,ult}$ [MPa]</th>
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Figure C.5 – Reinforcement stress-strain response, PLS4000 ρx Heat 2
### Table C.6 – Reinforcement Material Properties, PLS4000 $\rho_x'$ [400W, 20M]

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<th>$\varepsilon_y$ [mm/m]</th>
<th>$E_s$ [MPa]</th>
<th>$\varepsilon_{sh}$ [mm/m]</th>
<th>$f_{s,ult}$ [MPa]</th>
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*Figure C.6 – Reinforcement stress-strain response, PLS4000 $\rho_x'$*

### Table C.7 – Reinforcement Material Properties, PLS4000 $\rho_z$ [500W, 20M]

<table>
<thead>
<tr>
<th>Sample</th>
<th>$f_y$ [MPa]</th>
<th>$\varepsilon_y$ [mm/m]</th>
<th>$E_s$ [MPa]</th>
<th>$\varepsilon_{sh}$ [mm/m]</th>
<th>$f_{s,ult}$ [MPa]</th>
<th>$\varepsilon_{s,ult}$ [mm/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>222,801</td>
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<td>628.3</td>
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<td>2</td>
<td>521.9</td>
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<td>110</td>
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<td>3</td>
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<td>220,973</td>
<td>17.5</td>
<td>629.2</td>
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*Figure C.7 – Reinforcement stress-strain response, PLS4000 $\rho_z$*
Table C.8 – Reinforcement Material Properties, PLS300 ρₜ [400W, 10M]

<table>
<thead>
<tr>
<th>Sample</th>
<th>f_y [MPa]</th>
<th>ε_y [mm/m]</th>
<th>E_s [MPa]</th>
<th>ε_sh [mm/m]</th>
<th>f_s,ult [MPa]</th>
<th>ε_s,ult [mm/m]</th>
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</thead>
<tbody>
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<td>1</td>
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<td>168,718</td>
<td>26.6</td>
<td>564.5</td>
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<tr>
<td>2*</td>
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<td>2.482</td>
<td>185,414</td>
<td>29.8</td>
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<td>144</td>
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<tr>
<td>3</td>
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<td>562.4</td>
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* Only sample 2 was used for the value of Eₛ owing to the unrealistic initial elastic moduli produced by Samples 1 and 2 (likely because of bar slippage during testing).

Figure C.8 – Reinforcement stress-strain response, PLS300 ρₜ
APPENDIX D  ANCILLARY RESPONSE PLOTS

Graphical responses of the other various data acquisition channels not presented in the thesis body are reserved for this appendix; the corresponding condensed data sets can subsequently be found in Appendix E.
D.1 Load Cells

Figure D.1 – PLS4000 Phase I: Load cell response

Figure D.2 – PLS4000 Phase II: Load cell response
D.2 Reinforcement Strain Gauges

Figure D.3 – PLS4000 Phase I: West span flexural tension reinforcement strain response

Figure D.4 – PLS4000 Phase I: East span flexural tension reinforcement strain response
Figure D.5 – PLS4000 Phase I: Flexural compression reinforcement strain response

Figure D.6 – PLS4000 Phase I: Shear reinforcement strain response
**Figure D.7 – PLS4000 Phase II: West span flexural tension reinforcement strain response**

**Figure D.8 – PLS4000 Phase II: East span flexural tension reinforcement strain response**
Figure D.9 – PLS4000 Phase II: Flexural compression reinforcement strain response

Figure D.10 – PLS4000 Phase II: Shear reinforcement strain response
D.3 DISPLACEMENTS

**Figure D.11** – PLS4000 Phase I: Vertical displacement response

**Figure D.12** – PLS4000 Phase I: Longitudinal displacement response
Figure D.13 – PLS4000 Phase I: Lateral displacement response

Figure D.14 – PLS4000 Phase I: Out-of-plane rotation response
Figure D.15 – PLS4000 Phase II: Vertical displacement response

Figure D.16 – PLS4000 Phase II: Longitudinal displacement response
Figure D.17 – PLS4000 Phase II: Lateral displacement response

Figure D.18 – PLS4000 Phase II: Out-of-plane rotation response
D.4 PLS300

D.4.1 Longitudinal Displacement

Figure D.19 – PLS300 Longitudinal displacement response
D.4.2 Detailed Surface Displacements Plots

Figure D.20 – PLS300 deformed shapes (1 of 2)
(Displacements magnified x10)
Figure D.21 – PLS300 deformed shapes (2 of 2); final image shows deformations at test end (Displacements magnified x10)
D.4.3 Detailed Strain States

Figure D.22 – PLS300 Strain data and deformations from Timeline
Load Stage 1 [02:11:22]

Figure D.23 – PLS300 Strain data and deformations from Timeline
Load Stage 2 [02:30:16]

Figure D.24 – PLS300 Strain data and deformations from Timeline
Load Stage 3 [02:46:07]

Figure D.25 – PLS300 Strain data and deformations from Timeline
Load Stage 4 [03:00:42]

Figure D.26 – PLS300 Strain data and deformations from Timeline
Load Stage 5 [03:14:45]
Figure D.27 – PLS300 Strain data and deformations from Timeline
Load Stage 6 (Ultimate) [03:18:20]

Figure D.28 – PLS300 Strain data and deformations from Timeline
25 seconds after ultimate [03:18:45]

Figure D.29 – PLS300 Strain data and deformations from Timeline
30 seconds after ultimate [03:18:50]

Figure D.30 – PLS300 Strain data and deformations from Timeline
35 seconds after ultimate [03:18:55]
APPENDIX E  EXPERIMENTAL DATA SUMMARY TABLES

Data consolidation to reduce the experimental data into more manageable sets was performed after publication of this thesis. Sampling was executed using a quantitative metric for specified parameters that minimizes redundancies. Should the reader be interested in obtaining this consolidated dataset for their own use, they are directed to email Evan C. Bentz to request the information [bentz@ecf.utoronto.ca].

Tables provided in this appendix will present the full data stream (i.e. all parameters) for only key points during the testing such as load stages and test commencement/termination. Single dashes in the place of numerical values signify anomalous data typically due to instrumentation being damaged or their capacities being exceeded.
## E.1 Load Cells

### Table E.1 – PLS4000 Load Cell Data

<table>
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<tr>
<th>Load Stage</th>
<th>Data #</th>
<th>$P_{\text{applied}}$ [kN]</th>
<th>$R_{\text{West, North}}$ [kN]</th>
<th>$R_{\text{West, South}}$ [kN]</th>
<th>$R_{\text{East, North}}$ [kN]</th>
<th>$R_{\text{East, South}}$ [kN]</th>
<th>$\sum R_{\text{West}}$ [kN]</th>
<th>$\sum R_{\text{East}}$ [kN]</th>
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### E.2 Reinforcement Strain Gauges

Table E.2 – PLS4000 Strain Gauge Data: West span flexural tension reinforcement

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<th>$\varepsilon_{B1.2 \text{ TOP}}$</th>
<th>$\varepsilon_{B1.2 \text{ BOT}}$</th>
<th>$\varepsilon_{B1.3 \text{ BOT}}$</th>
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<td>[mm/m]</td>
<td>[mm/m]</td>
<td>[mm/m]</td>
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Table E.3 – PLS4000 Strain Gauge Data: East span flexural tension reinforcement (1 of 2)

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<th>$\varepsilon_{B2.2}$ TOP [mm/m]</th>
<th>$\varepsilon_{B2.3}$ BOT [mm/m]</th>
<th>$\varepsilon_{B2.3}$ TOP [mm/m]</th>
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### Table E.4 – PLS4000 Strain Gauge Data: East span flexural tension reinforcement (2 of 2)

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### Table E.5 – PLS4000 Strain Gauge Data: Shear reinforcement

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## Table E.6 – PLS4000 Strain Gauge Data: Flexural compression reinforcement

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<tr>
<td>Start</td>
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</tr>
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### E.3 Displacements

Table E.7 – PLS4000 Displacement Data: Vertical & Longitudinal

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<th>Load Stage</th>
<th>$\Delta V_1$ [mm]</th>
<th>$\Delta V_2$ [mm]</th>
<th>$\Delta V_3$ [mm]</th>
<th>$\Delta V_4$ [mm]</th>
<th>$\Delta V_5$ [mm]</th>
<th>$\Delta_{\text{Long}}$ [mm]</th>
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* Manually set based on linear elastic calculation (See Section 3.1.5)
Table E.8 – PLS4000 Displacement Data: Diagonals, CLZ, & calculated shear strains

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<th>Load Stage</th>
<th>$\Delta_{D1}\text{ West}$ [mm]</th>
<th>$\Delta_{D1}\text{ East}$ [mm]</th>
<th>$\Delta_{D2}\text{ West}$ [mm]</th>
<th>$\Delta_{D2}\text{ East}$ [mm]</th>
<th>$\Delta_{D3}\text{ West}$ [mm]</th>
<th>$\Delta_{D3}\text{ East}$ [mm]</th>
<th>$Y_{xy, D1} \times 10^3$ [rad]</th>
<th>$Y_{xy, D2} \times 10^3$ [rad]</th>
<th>$Y_{xy, D3}^* \times 10^3$ [rad]</th>
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<td></td>
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<td>0.090</td>
<td>0.040</td>
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<td>0.047</td>
<td>0.004</td>
<td>0.024</td>
<td>0.005</td>
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<td>0.052</td>
<td>0.011</td>
<td>0.012</td>
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<td>0.105</td>
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<td>0.023</td>
<td>0.012</td>
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<td>0.052</td>
<td>0.168</td>
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<td>0.159</td>
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<tr>
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<td>-0.802</td>
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<td>-0.045</td>
<td>0.243</td>
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<td>5.144</td>
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| **Phase II**| $\Delta_{CLZ}\text{ Horiz}$ | $\Delta_{CLZ}\text{ Long}$ |                               |                               |                               |                               |                                 |                                 |                                 |
|------------|-------------------------------|-------------------------------|                               |                               |                               |                               |                                 |                                 |                                 |
| Start      | 0.002                         | 0.050                         | 0.000                         | -0.001                        | 0.000                         | -0.002                        | 0.009                           | 0.000                           | -                               |
| 1st Crack  | -1.853                        | 0.171                         | -0.014                        | -0.740                        | 0.169                         | -0.028                        | 0.377                           | 0.140                           | -                               |
| 2nd Crack  | -3.457                        | 0.010                         | -0.058                        | -1.216                        | 0.234                         | -0.062                        | 0.645                           | 0.237                           | -                               |
| LS 7       | -4.362                        | -0.131                        | -0.063                        | -1.281                        | 0.245                         | -0.100                        | 0.787                           | 0.250                           | -                               |
| LS 7 Unload| -4.176                        | -0.131                        | -0.050                        | -1.224                        | 0.244                         | -0.104                        | 0.753                           | 0.237                           | -                               |
| LS 8       | -6.731                        | -0.346                        | -0.151                        | -2.600                        | 0.331                         | -0.196                        | 1.188                           | 0.512                           | -                               |
| LS 8 Unload| -6.437                        | -0.363                        | -0.124                        | -2.497                        | 0.331                         | -0.200                        | 1.130                           | 0.488                           | -                               |
| LS 9       | -9.344                        | -0.668                        | -0.066                        | -3.993                        | 0.440                         | -0.364                        | 1.615                           | 0.755                           | -                               |
| LS 9 Unload| -8.998                        | -0.710                        | -0.036                        | -3.844                        | 0.433                         | -0.394                        | 1.542                           | 0.722                           | -                               |
| LS 10      | -11.784                       | -1.153                        | 0.073                         | -5.302                        | 0.479                         | -0.728                        | 1.978                           | 0.973                           | -                               |
| LS 10 Unload| -10.487                       | -1.241                        | 0.113                         | -4.644                        | 0.397                         | -0.760                        | 1.720                           | 0.843                           | -                               |
| LS 11      | -13.614                       | -0.546                        | 0.207                         | -6.141                        | 0.503                         | -1.179                        | 2.432                           | 1.104                           | -                               |
| LS 11 Unload| -23.980                       | -15.045                       | 0.098                         | -0.665                        | -                            | -                            | 1.663                           | 0.105                           | -                               |
| End        | -23.544                       | -15.070                       | 0.102                         | -0.514                        | -                            | -                            | 1.577                           | 0.077                           | -                               |

* Not obtained in Phase II due to instrumentation interference with external post-tensioning (Recall Figure 2.47); Therefore $\Delta_{D3}\text{ West}$ and $\Delta_{D3}\text{ East}$ become $\Delta_{CLZ}\text{ Horiz}$ & $\Delta_{CLZ}\text{ Long}$ respectively for Phase II.
### Table E.9 – PLS4000 Displacement Data: Out-of-plane & calculated rotations

<table>
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<th>Load Stage</th>
<th>$\Delta_{LW \cdot BOT}$ [mm]</th>
<th>$\Delta_{LW \cdot MID}$ [mm]</th>
<th>$\Delta_{LW \cdot TOP}$ [mm]</th>
<th>$\Delta_{LE \cdot BOT}$ [mm]</th>
<th>$\Delta_{LE \cdot MID}$ [mm]</th>
<th>$\Delta_{LE \cdot TOP}$ [mm]</th>
<th>$\theta_{\text{OOP WEST}} \times 10^3$ [deg]</th>
<th>$\theta_{\text{OOP EAST}} \times 10^3$ [deg]</th>
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<td>0.033</td>
<td>-0.011</td>
<td>0.072</td>
<td>0.021</td>
</tr>
<tr>
<td>LS 1 Unload</td>
<td>-0.081</td>
<td>0.124</td>
<td>0.203</td>
<td>-0.024</td>
<td>0.073</td>
<td>0.003</td>
<td>0.095</td>
<td>0.009</td>
</tr>
<tr>
<td>LS 2</td>
<td>-1.700</td>
<td>0.052</td>
<td>0.106</td>
<td>-0.232</td>
<td>-0.046</td>
<td>-0.133</td>
<td>0.602</td>
<td>0.033</td>
</tr>
<tr>
<td>LS 2 Unload</td>
<td>-1.699</td>
<td>0.064</td>
<td>0.126</td>
<td>-0.197</td>
<td>-0.022</td>
<td>-0.114</td>
<td>0.608</td>
<td>0.028</td>
</tr>
<tr>
<td>LS 3</td>
<td>-1.427</td>
<td>0.260</td>
<td>0.268</td>
<td>0.687</td>
<td>0.569</td>
<td>0.582</td>
<td>0.565</td>
<td>0.035</td>
</tr>
<tr>
<td>LS 3 Unload</td>
<td>-1.412</td>
<td>0.269</td>
<td>0.272</td>
<td>0.734</td>
<td>0.623</td>
<td>0.649</td>
<td>0.561</td>
<td>0.028</td>
</tr>
<tr>
<td>LS 4</td>
<td>-1.513</td>
<td>0.104</td>
<td>0.069</td>
<td>0.495</td>
<td>0.496</td>
<td>0.494</td>
<td>0.528</td>
<td>0.000</td>
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<tr>
<td>LS 4 Unload</td>
<td>-1.495</td>
<td>0.117</td>
<td>0.071</td>
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<td>0.497</td>
<td>0.482</td>
<td>0.522</td>
<td>0.015</td>
</tr>
<tr>
<td>LS 5</td>
<td>-1.566</td>
<td>0.079</td>
<td>0.031</td>
<td>0.408</td>
<td>0.437</td>
<td>0.436</td>
<td>0.532</td>
<td>0.009</td>
</tr>
<tr>
<td>LS 5 Unload</td>
<td>-1.496</td>
<td>0.205</td>
<td>0.171</td>
<td>0.292</td>
<td>0.184</td>
<td>0.556</td>
<td>0.006</td>
<td></td>
</tr>
<tr>
<td>LS 6</td>
<td>-1.523</td>
<td>0.284</td>
<td>0.315</td>
<td>0.193</td>
<td>0.306</td>
<td>0.251</td>
<td>0.613</td>
<td>0.020</td>
</tr>
<tr>
<td>LS 6 Unload</td>
<td>-1.430</td>
<td>0.494</td>
<td>0.650</td>
<td>0.079</td>
<td>0.984</td>
<td>0.418</td>
<td>0.693</td>
<td>0.113</td>
</tr>
<tr>
<td>End</td>
<td>-1.430</td>
<td>0.493</td>
<td>0.650</td>
<td>0.081</td>
<td>0.973</td>
<td>0.410</td>
<td>0.693</td>
<td>0.110</td>
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<tr>
<td><strong>Phase II</strong></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>Start</td>
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<td>0.000</td>
<td>0.004</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.001</td>
<td>0.000</td>
</tr>
<tr>
<td>1st Crack</td>
<td>-0.578</td>
<td>-0.443</td>
<td>-0.419</td>
<td>-0.733</td>
<td>-0.271</td>
<td>-0.402</td>
<td>0.053</td>
<td>0.110</td>
</tr>
<tr>
<td>2nd Crack</td>
<td>-0.676</td>
<td>-0.434</td>
<td>-0.212</td>
<td>-0.881</td>
<td>-0.125</td>
<td>-0.251</td>
<td>0.155</td>
<td>0.210</td>
</tr>
<tr>
<td>LS 7</td>
<td>-0.772</td>
<td>-0.471</td>
<td>-0.223</td>
<td>-0.931</td>
<td>-0.092</td>
<td>-0.194</td>
<td>0.183</td>
<td>0.246</td>
</tr>
<tr>
<td>LS 7 Unload</td>
<td>-0.727</td>
<td>-0.480</td>
<td>-0.234</td>
<td>-0.866</td>
<td>-0.087</td>
<td>-0.194</td>
<td>0.165</td>
<td>0.224</td>
</tr>
<tr>
<td>LS 8</td>
<td>-1.612</td>
<td>-1.522</td>
<td>-1.350</td>
<td>-1.704</td>
<td>-0.848</td>
<td>-1.072</td>
<td>0.087</td>
<td>0.210</td>
</tr>
<tr>
<td>LS 8 Unload</td>
<td>-1.543</td>
<td>-1.524</td>
<td>-1.397</td>
<td>-1.624</td>
<td>-0.866</td>
<td>-1.104</td>
<td>0.049</td>
<td>0.174</td>
</tr>
<tr>
<td>LS 9</td>
<td>-2.395</td>
<td>-2.334</td>
<td>-2.084</td>
<td>-2.317</td>
<td>-1.483</td>
<td>-1.768</td>
<td>0.104</td>
<td>0.183</td>
</tr>
<tr>
<td>LS 9 Unload</td>
<td>-2.303</td>
<td>-2.336</td>
<td>-2.097</td>
<td>-2.179</td>
<td>-1.453</td>
<td>-1.794</td>
<td>0.069</td>
<td>0.128</td>
</tr>
<tr>
<td>LS 10</td>
<td>-3.027</td>
<td>-2.935</td>
<td>-2.542</td>
<td>-2.733</td>
<td>-1.897</td>
<td>-2.393</td>
<td>0.162</td>
<td>0.113</td>
</tr>
<tr>
<td>LS 10 Unload</td>
<td>-2.557</td>
<td>-2.754</td>
<td>-2.402</td>
<td>-2.867</td>
<td>-2.333</td>
<td>-3.056</td>
<td>0.052</td>
<td>0.063</td>
</tr>
<tr>
<td>LS 11</td>
<td>-3.343</td>
<td>-3.243</td>
<td>-2.587</td>
<td>-3.716</td>
<td>-2.931</td>
<td>-3.708</td>
<td>0.252</td>
<td>0.003</td>
</tr>
<tr>
<td>LS 11 Unload</td>
<td>-4.432</td>
<td>-3.320</td>
<td>-2.232</td>
<td>-2.223</td>
<td>-0.216</td>
<td>-0.477</td>
<td>0.733</td>
<td>0.582</td>
</tr>
<tr>
<td>End</td>
<td>-3.969</td>
<td>-3.115</td>
<td>-2.243</td>
<td>-2.087</td>
<td>-0.313</td>
<td>-0.597</td>
<td>0.576</td>
<td>0.497</td>
</tr>
</tbody>
</table>
### Table E.10 – PLS300 Load-Deformation Data

<table>
<thead>
<tr>
<th>Load Stage</th>
<th>( P ) [kN]</th>
<th>( \Delta_{vert} ) [mm]</th>
<th>( \Delta_{long} ) [mm]</th>
<th>( \varepsilon_{long}^{*} ) [mm/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Start</td>
<td>0.2</td>
<td>0.00</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>Pause 1</td>
<td>20.0</td>
<td>0.53</td>
<td>0.02</td>
<td>0.046</td>
</tr>
<tr>
<td>Pause 2</td>
<td>29.9</td>
<td>0.72</td>
<td>0.05</td>
<td>0.106</td>
</tr>
<tr>
<td>1</td>
<td>35.4</td>
<td>0.95</td>
<td>0.14</td>
<td>0.271</td>
</tr>
<tr>
<td>1 Unload</td>
<td>32.1</td>
<td>0.96</td>
<td>0.14</td>
<td>0.283</td>
</tr>
<tr>
<td>2</td>
<td>55.0</td>
<td>1.89</td>
<td>0.50</td>
<td>1.001</td>
</tr>
<tr>
<td>2 Unload</td>
<td>51.2</td>
<td>1.90</td>
<td>0.51</td>
<td>1.014</td>
</tr>
<tr>
<td>3</td>
<td>69.9</td>
<td>2.69</td>
<td>0.77</td>
<td>1.545</td>
</tr>
<tr>
<td>3 Unload</td>
<td>63.6</td>
<td>2.62</td>
<td>0.75</td>
<td>1.493</td>
</tr>
<tr>
<td>4</td>
<td>79.8</td>
<td>3.21</td>
<td>0.93</td>
<td>1.865</td>
</tr>
<tr>
<td>4 Unload</td>
<td>73.6</td>
<td>3.14</td>
<td>0.90</td>
<td>1.806</td>
</tr>
<tr>
<td>5</td>
<td>89.9</td>
<td>3.90</td>
<td>1.10</td>
<td>2.195</td>
</tr>
<tr>
<td>5 Unload</td>
<td>83.2</td>
<td>3.80</td>
<td>1.06</td>
<td>2.126</td>
</tr>
<tr>
<td>6</td>
<td>94.7</td>
<td>4.26</td>
<td>1.19</td>
<td>2.385</td>
</tr>
<tr>
<td>End</td>
<td>0.4</td>
<td>3.44</td>
<td>0.29</td>
<td>0.589</td>
</tr>
</tbody>
</table>

* Calculated as: \( \varepsilon_{long} = \Delta_{long}/L_g \), where \( L_g = 500 \text{ mm} \)
APPENDIX F  SURFACE 3D COORDINATE MEASUREMENTS

To the author’s knowledge, surface 3D coordinate measurements using the Nikon K-610 system in combination with the LED-equipped space probe has not yet been accomplished or properly documented at the University of Toronto. Furthermore, capturing multiple frames of target coordinates individually and stitching them together requires an intricate methodology. Data sets gathered inconsistently amongst different load stages, due to optimization during the experimental program, further add to the complexity.

This appendix summarizes the procedures used to collect and process the 3D coordinate data that form the basis of the surface displacement plots for Phase I of the PLS4000 established in Chapter 3; notation from Figure 2.50 depicting the LED instrumentation schematic is used and referenced throughout.

A MATLAB© program was written by the author to completely automate the highly redundant and elaborate process of 1) reading/processing the raw data as exported by the Nikon CMM software, and 2) implementing all of the transformative stitching procedures outlined in this appendix for any given load stage.
F.1 **COORDINATE MEASUREMENT PROCEDURE**

Coordinate captures were performed from east, starting at the roller support, to west with vertical frames added as required for the higher level targets. Overlap of two columns and two rows of targets for longitudinal and vertical stitches respectively, were achieved for each successive frame. This allowed the data to be properly stitched together after testing completion using the methodology discussed later in Section F.3. A visual representation of the coordinate scanning procedure is shown below in Figure F.1.

![Figure F.1 – 3D coordinate capture procedure](image)

Consecutive longitudinal frame captures were carried out by moving the coordinate measuring machine (CMM) on a translatable skid along the specimen span, while vertical overlap was done via rotation of the camera about a central pivot point (See Figure F.2 on next page). With regards to the former, a line was marked on the laboratory floor for consistent capture distance that maximized the measurement volume of the camera. This line was also marked with ideal skid (i.e. camera) placement for the proper scanning of each region, as the limited field of view required quite precise centering of the CMM within the area under consideration.
Each load stage resulted in a different capture procedure from the original intended process because of optimization required to reduce measurement time. Prior to jacking the PLS4000, a full target scan ($\theta_1$, $\theta_2$, and $\theta_3$) was carried out to create the reference state of deformations preceding successive load stages. Differences in coordinate captures are presented in Table F.1.

Table F.1 – 3D Coordinate Capture Testing Matrix

<table>
<thead>
<tr>
<th>Stage</th>
<th>$\theta_1$</th>
<th>$\theta_2$</th>
<th>$\theta_3$</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>LS0a</td>
<td>✔</td>
<td>✔</td>
<td>✔</td>
<td>Full scan, complete redundant overlap (all points in overlapping region captured)</td>
</tr>
<tr>
<td>Pre-jacking</td>
<td></td>
<td></td>
<td></td>
<td>First 3 rows of targets, complete redundant overlap</td>
</tr>
<tr>
<td>LS0b</td>
<td>✔</td>
<td>×</td>
<td>×</td>
<td>First 4 rows of targets, started streamlined vertical overlap midway through capture (only obtaining 4 points from previous stitch below)</td>
</tr>
<tr>
<td>Self-weight</td>
<td></td>
<td></td>
<td></td>
<td>First 4 rows of targets, complete streamlined overlap in both vertical and longitudinal directions (4 point overlap)</td>
</tr>
<tr>
<td>LS 01</td>
<td>✔</td>
<td>✔</td>
<td>×</td>
<td>All 5 target rows, streamlined overlap, skipped regions 12 &amp; 13 due to time constraints</td>
</tr>
<tr>
<td>LS 02</td>
<td>✔</td>
<td>✔</td>
<td>×</td>
<td>All 5 target rows, streamlined overlap, no Region 12 &amp; 13</td>
</tr>
<tr>
<td>LS 03</td>
<td>✔</td>
<td>✔</td>
<td>✔</td>
<td>All 5 target rows, streamlined overlap, no Region 12 &amp; 13</td>
</tr>
<tr>
<td>LS 04</td>
<td>✔</td>
<td>✔</td>
<td>✔</td>
<td>All 5 target rows, streamlined overlap, no Region 12 &amp; 13</td>
</tr>
<tr>
<td>LS 05</td>
<td>✔</td>
<td>✔</td>
<td>✔</td>
<td>All 5 target rows, streamlined overlap, no Region 12 &amp; 13</td>
</tr>
<tr>
<td>LS 06</td>
<td>✔</td>
<td>✔</td>
<td>✔</td>
<td>All 5 target rows, streamlined overlap, no Region 12 &amp; 13</td>
</tr>
</tbody>
</table>

Figure F.2 – CMM camera setup and alignment (left), rotation schematic (right)
F.2 SPACE PROBE CALIBRATION DATA

Custom targets required for the surface 3D coordinate measurements using the space probe necessitated an assessment of the error to determine if it was within acceptable limits for the desired displacement and strain data to be obtained. Multiple measurements at a single target were triggered repeatedly with the space probe being taken off each time to mimic the testing conditions. Due to the vertical rotation of the camera necessary to scan the full specimen depth, accuracy of the same targets with overlapping captures at different angles was also validated. Furthermore, planar yaw rotation of the space probe equal to about ±10° was sometimes required to capture the LED targets of the device in specific frames (See Figure F.3).

![Figure F.3 – CMM space probe planar rotation](image)

It was found that over the standard nominal 600 mm gauge length ($L_{gauge}$) between targets in both directions, average strain errors ($\varepsilon_{error}$) of $0.061 \times 10^{-3}$, $0.036 \times 10^{-3}$, and $0.037 \times 10^{-3}$ were found for the X, Y, and Z coordinate measurements respectively.

Where strain error is defined as follows:

$$ \varepsilon_{error} = \frac{\sigma}{L_{gauge}} \ [mm/m] $$  \hspace{1cm} [F.1]

Complete calibration data can be found in Table F.2 on the subsequent page as well as in the error analysis figures following it.
Table F.2 – LED Space Probe Target Calibration Data

<table>
<thead>
<tr>
<th>Row</th>
<th>X</th>
<th>μ [mm]</th>
<th>σ* [mm]</th>
<th>C.O.V. † [%]</th>
<th>εerror [mm/m]</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>X</td>
<td>312.9582</td>
<td>0.0283</td>
<td>0.009%</td>
<td>0.047</td>
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<td>0.0099</td>
<td>-0.001%</td>
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<tr>
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<td>X</td>
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<td>0.033</td>
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<td></td>
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<td></td>
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<td>0.000%</td>
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<td></td>
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<td>-0.002%</td>
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<tr>
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<td>0.0207</td>
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<tr>
<td>4</td>
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<tr>
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<tr>
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<td></td>
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<tr>
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</tr>
<tr>
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</tr>
<tr>
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</tr>
<tr>
<td></td>
<td>Z</td>
<td>-4374.7253</td>
<td>0.0198</td>
<td>0.000%</td>
<td>0.033</td>
</tr>
</tbody>
</table>

* Sample standard deviation
† C.O.V. = σ / μ
Figure F.4 – Error analysis of LED target, row 1
Figure F.5 – Error analysis of LED target, row 2
APPENDIX F

SURFACE 3D COORDINATE MEASUREMENTS

Figure F.6 – Error analysis of LED target, row 3
Figure F.7 – Error analysis of LED target, row 4
Figure F.8 – Error analysis of LED target, row 5
F.3 3D Change of Basis Coordinate Transformations

Change of basis transformations for two and three dimensional coordinate systems shall be explained in this section. This process of “changing bases” effectively stitches together two different frames of reference to unify their coordinate systems [42]. Change of basis occurs in the Euclidean space (i.e. the Cartesian system) and is considered a linear transformation because it relates coordinates in one frame to those in another frame through a system of linear algebraic equations.

F.3.1 General Concept

![Diagram](https://via.placeholder.com/150)

*Figure F.9 – 3D change of basis visualized (Adapted from [43])*

To understand how this is achieved, consider the general case of 3D coordinates in $\mathbb{R}^3$. Figure F.9 shows a vector $\{v\}$ in two different coordinate systems. It is evident that the vector can be represented by a linear combination of the 3 basis vectors of each respective frame. In essence:

<table>
<thead>
<tr>
<th>Frame 1 (F1)</th>
<th>Frame 2 (F2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>${v}_1 = {x, y, z}_1$</td>
<td>${v}_2 = {x, y, z}_2$</td>
</tr>
</tbody>
</table>

The coefficients of the basis vectors $\{X\}$, $\{Y\}$, and $\{Z\}$ are simply the coordinates of the vector $\{v\}$ and the actual basis vectors themselves are any set of 3 linearly independent (orthogonal) vectors in the space (e.g. axes). Writing the linear transformation in vector form gives:

$$[T]_{2 \rightarrow 1} \{v\}_2 = \{v\}_1$$

$$\begin{bmatrix} \{X\}_1 \\ \{Y\}_1 \\ \{Z\}_1 \end{bmatrix} \times \{v\}_2 = \{v\}_1$$

$[T]$ represents a square matrix consisting of the 3 basis vectors, and is called a *change of basis matrix*. Equation [F.5] thus allows the representation of a vector (i.e. coordinates of a single point) in frame 2 with respect to frame 1 and vice versa via transformation with $[T]$. 

**Shear Response of Large-Scale Reinforced Concrete Structures**

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F.3.2 Application to 3D Test Measurements

In the context of the PLS4000 surface coordinate measurements taken with the 3D CMM, two overlapping capture frames resulted in a shared set of points with different origins that need to have identically harmonised coordinates in a unified reference coordinate system (i.e. they should coincide, See Figure F.10):

\[
[T]\{v\}_\text{new} + \{S\} = \{v\}_\text{old}
\]

\[
\begin{bmatrix}
a & b & c \\
d & e & f \\
g & h & i \\
\end{bmatrix}
\begin{bmatrix}
x \\
y \\
z \\
\end{bmatrix}_\text{new} + 
\begin{bmatrix}
x_S \\
y_S \\
z_S \\
\end{bmatrix} = 
\begin{bmatrix}
x \\
y \\
z \\
\end{bmatrix}_\text{old}
\]

Where: \([T]\) is the change of basis transformation matrix now represented simply by coefficients, \(\{v\}_\text{new}\) represents a column vector of the 3D coordinates of a point in the new reference frame, \(\{v\}_\text{old}\) are the coordinates of the same point (overlapped between the 2 frames of capture) in the original reference frame, and \(\{S\}\) represents an origin shift vector. A shift in origin is also required since the two frames do not share the same origin.

Figure F.10 – LED coordinate change of basis capture schematic
The change of basis matrix $[T]$ must be determined such that the coordinates of the point in the new frame are equivalent to the coordinates in the original reference frame (i.e. Equation [F.7] must be satisfied). To solve for $[T]$, we expand Equation [F.7] out into linear form:

$$a \times x_{new} + b \times y_{new} + c \times z_{new} + S_x = x_{old}$$  \hspace{1cm} [F.8]

$$d \times x_{new} + e \times y_{new} + f \times z_{new} + S_y = y_{old}$$  \hspace{1cm} [F.9]

$$g \times x_{new} + h \times y_{new} + i \times z_{new} + S_z = z_{old}$$  \hspace{1cm} [F.10]

It can be seen that there are 9 unknowns as the coefficients of the $[T]$ matrix and therefore 9 equations are required to solve the system. Each row vector of $[T]$ can be solved using Equations [F.8] to [F.10] by utilizing the coordinates of different points matched within the overlap. Consequently, it is deduced that at least 3 points are required for an overlap change of basis because in order to solve Equation [F.8] for the vector $[a \ b \ c]$ for example, 3 equations are needed to determine the 3 unknown coefficients. As a result, Equation [F.8] can be rewritten using 3 different points to be coincided between the 2 overlapping reference frames:

$$a \times x_{new_{pt1}} + b \times y_{new_{pt1}} + c \times z_{new_{pt1}} + S_x = x_{old_{pt1}}$$  \hspace{1cm} [F.11]

$$a \times x_{new_{pt2}} + b \times y_{new_{pt2}} + c \times z_{new_{pt2}} + S_x = x_{old_{pt2}}$$  \hspace{1cm} [F.12]

$$a \times x_{new_{pt3}} + b \times y_{new_{pt3}} + c \times z_{new_{pt3}} + S_x = x_{old_{pt3}}$$  \hspace{1cm} [F.13]

Rewriting the above 3 equations in matrix form:

$$
\begin{bmatrix}
  x_{pt1} & y_{pt1} & z_{pt1} \\
  x_{pt2} & y_{pt2} & z_{pt2} \\
  x_{pt3} & y_{pt3} & z_{pt3}
\end{bmatrix}_{new}
\begin{bmatrix}
a \\
b \\
c
\end{bmatrix}
+
\begin{bmatrix}
S_x \\
S_y \\
S_z
\end{bmatrix}
=
\begin{bmatrix}
x_{pt1} \\
x_{pt2} \\
x_{pt3}
\end{bmatrix}_{old}
$$  \hspace{1cm} [F.14]

This allows $\{a \ b \ c\}$ to then be solved:

$$
\begin{bmatrix}
a \\
b \\
c
\end{bmatrix} =
\begin{bmatrix}
x_{pt1} & y_{pt1} & z_{pt1} \\
x_{pt2} & y_{pt2} & z_{pt2} \\
x_{pt3} & y_{pt3} & z_{pt3}
\end{bmatrix}_{new}^{-1}
\begin{bmatrix}
x_{pt1} \\
x_{pt2} \\
x_{pt3}
\end{bmatrix}_{old}
-
\begin{bmatrix}
S_x \\
S_y \\
S_z
\end{bmatrix}
$$  \hspace{1cm} [F.15]

Repeating the process for the remaining 2 row vectors of $[T]$ yields:

$$
\begin{bmatrix}
d \\
e \\
f
\end{bmatrix} =
\begin{bmatrix}
x_{pt1} & y_{pt1} & z_{pt1} \\
x_{pt2} & y_{pt2} & z_{pt2} \\
x_{pt3} & y_{pt3} & z_{pt3}
\end{bmatrix}_{new}^{-1}
\begin{bmatrix}
y_{pt1} \\
y_{pt2} \\
y_{pt3}
\end{bmatrix}_{old}
-
\begin{bmatrix}
S_x \\
S_y \\
S_z
\end{bmatrix}
$$  \hspace{1cm} [F.16]
The change of basis matrix is then constructed in the form previously presented:

\[
\{g\} = \begin{bmatrix} x_{Pt 1} & y_{Pt 1} & z_{Pt 1} \\ x_{Pt 2} & y_{Pt 2} & z_{Pt 2} \\ x_{Pt 3} & y_{Pt 3} & z_{Pt 3} \end{bmatrix}_{\text{new}}^{-1} \times \left\{ \begin{bmatrix} z_{Pt 1} \\ z_{Pt 2} \\ z_{Pt 3} \end{bmatrix}_{\text{old}} - \begin{bmatrix} S_x \\ S_y \\ S_z \end{bmatrix} \right\} \quad \text{[F.17]}
\]

Accordingly, \([T]\) is called the transformation or translation matrix because it performs the conversion of coordinates from one basis (i.e. coordinate system) to another.

It should be emphasized that the procedure only works properly when the 3 overlapping points form a well-defined plane. When overlap was achieved with points essentially in a line, the transformation resulted in highly skewed and erroneous results. Triangular planes spanning as many elements as possible were thus used throughout the procedure. An analogy can be made to a finite element, whereby numerical accuracy of the method is undermined when aspect ratios deviate from ideal shapes such as squares and equilateral triangles stemming from the formulation of the interpolation functions.

Illustrations of the basis change transformations shown henceforth include both the “idealized” coordinate grid that assumes nominally spaced intended target locations (shown by the blue dots), and the processed coordinate grid (depicted as red ‘X’s). The discrepancy between the two is largely because of tolerances associated with the fabrication methods and mounting of the aluminum angle targets. The process of using a hammer drill in concrete is quite a vigorous one; inherent vibrations often cause the desired hole to be offset from the intentional location. Hence, it is very likely that targets were installed about ±10 mm out of place. An apparent artificial longitudinal elongation is revealed when comparing the idealized coordinates with the processed data in the subsequent figures and this can be attributed to the aforementioned errors arising from target installation in addition to numerical error aggregation in the stitching process. The final column of targets are found to be 238 mm west of the anticipated point for the self-weight case, which at first seems like a massive error. However, when averaged out over the 32 elements on the bottom row along the span, this corresponds to about 7.4 mm deviation from the nominally 600 mm (typical) grid, which is quite reasonable.
Figure F.11 below shows a short visual sequence of the change of basis procedure:

Figure F.11 – Change of basis coordinate transformations
F.3.2.1 Note on Homogenous Coordinates

Using homogeneous coordinates in projective space is an alternative form of accomplishing the change of basis transformations and is worth mentioning. Projective space is a technique used in 3D computer graphics that introduces an extra variable in the linear system of equations [44]. Representation of an \( n \)-dimensional space is accomplished using \( n+1 \) dimension matrices, conducive to allowing different combinations of affine transformations to be performed if one requires. The details of affine spaces are out of the scope of this thesis, but is well defined in linear algebraic literature. Rewriting Equation [F.7] in this method results in:

\[
\begin{bmatrix}
    a & b & c & S_x \\
    d & e & f & S_y \\
    g & h & i & S_z \\
    0 & 0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
    x \\
    y \\
    z \\
    1
\end{bmatrix}_{\text{new}} =
\begin{bmatrix}
    x \\
    y \\
    z \\
    1
\end{bmatrix}_{\text{old}}
\] [F.19]

The origin shift matrix is now built into the transformation matrix \([T]\), but the system of equations to be solved remains identical to those derived from the Euclidean space if one expands out Equation [F.19] using dot product multiplication.

Homogenous coordinates are mentioned because alternative transforms may be capable of improving the accuracy of the procedure used by the author; this is left for future investigation.

F.3.3 Rotational Transformation Corrections

Successive execution of 3D change of basis transformations from the east roller of PLS4000 all the way to the west pin results in a rotated map of coordinates (See Figure F.12). The reason for this is because error is introduced with every change of basis inherently through variable target movement (angle bending) and computational numerical round off. Each additional frame being stitched onto the previous frame thus contains both the error accumulated from the preceding transformations as well as new error introduced by the current transformation.

![Figure F.12 – 3D rotational skew in XY elevation view due to error accumulation](image)
Consequently, a transformative correction should be applied that effectively rotates the coordinate data back into the proper orientation. There are 3 orthogonal rotation matrices corresponding to twist about each respective axes in Cartesian space:

\[
R_x(\theta_x) = \begin{bmatrix}
1 & 0 & 0 \\
0 & \cos\theta_x & -\sin\theta_x \\
0 & \sin\theta_x & \cos\theta_x
\end{bmatrix}
\]  \hspace{1cm} [F.20]

\[
R_y(\theta_y) = \begin{bmatrix}
\cos\theta_y & 0 & \sin\theta_y \\
0 & 1 & 0 \\
-\sin\theta_y & 0 & \cos\theta_y
\end{bmatrix}
\]  \hspace{1cm} [F.21]

\[
R_z(\theta_z) = \begin{bmatrix}
\cos\theta_z & -\sin\theta_z & 0 \\
\sin\theta_z & \cos\theta_z & 0 \\
0 & 0 & 1
\end{bmatrix}
\]  \hspace{1cm} [F.22]

Where the angles \( \theta_i \) are determined from the slope of a vector orthogonal to the axis being rotated about. Generally speaking, for rotation of the plane \( j-k \) about axis \( i \):

\[
\theta_i = \tan^{-1}(\text{slope}_{jk}) = \tan^{-1}\left( \frac{\text{rise}_j}{\text{run}_k} \right)
\]  \hspace{1cm} [F.23]

Coordinate rotation is then achieved by:

\[
\{v\}_{\text{new}} = \left[ R_x \right] \left[ R_y \right] \left[ R_z \right] \times \{v\}_{\text{old}}
\]  \hspace{1cm} [F.24]

Where: \( \{v\}_{\text{old}} \) are the coordinates of a specific point obtained from the change of basis procedure (pre-rotation correction), and \( \{v\}_{\text{new}} \) are the coordinates of that same point after rotational transformation. The sequence of rotations starting from the change of basis transformed coordinates are illustrated in Figure F.14 where each distinct revolution can be observed. Vectors used to define the slope of the rotations were methodically chosen to minimize error; averaging of two different vectors were typically used for each rotation. Visual depiction of vector trajectory and corresponding rotation is shown below in Figure F.13.
APPENDIX F

SURFACE 3D COORDINATE MEASUREMENTS

Figure F.14 – 3D coordinate rotational transformation process
APPENDIX F  SURFACE 3D COORDINATE MEASUREMENTS

F.3.4 Alternate Stitching Procedure

Regions 12 and 13 of the coordinate map were positioned just outside the line of the point load. As a consequence, these captures were very difficult to obtain due to interference from the moment resisting frame (MRF) column that supported the loading assembly. Though they were crucial in obtaining a complete east to west stitch of the coordinates, time constraints required that each load stage occur more rapidly. It was hence decided to bypass regions 12 and 13 during the scanning process, as supplemental displacement data could be cross-referenced from the vertical LVDTs to complete the stitch after-the-fact.

The lack of region 12 and 13 captures for load stages 3 to 6 to facilitate faster scanning required an alternative stitching procedure to be implemented. It was decided to first create the east span coordinate map up until column 21 of the targets, just adjacent to the MRF pillar on the east side. This was followed by construction of the west span grid starting from target column 22 and moving all the way to the pin support. The alternate stitch prior to rotational correction is illustrated in Figure F.15 below.

![Figure F.15 – Alternate 3D coordinate stitch procedure, before rotation](image)

Vertical displacements from LVDT V2 (Refer to Figure 2.42) directly underneath the point load were then utilized as a reference to rotate each individual span into place. The reasonable assumption made to accomplish this is that vertical translation of the bottom 2 nodes flanking the MRF column undergoes identical displacements. Rotation of each respective span was attained using the same methodology discussed in Section F.3.3. The final result of coordinates after LVDT referencing and ensuing planar rotations is shown in Figure F.16 on the next page.

It was discovered that an insufficient amount of points were collected for certain regions in the load stage 3 data set, leading to no complete stitch being generated for LS 3.
Figure F.16 – Alternate 3D coordinate stitch procedure, after rotation (displacements exaggerated for illustrative purposes)

Load stages 0a (zero load), 0b (self-weight), 1, and 2 used standard stitch procedures while stages 4, 5, and 6 were processed with the alternate stitching just described. The difference in methodology between the standard and alternative coordinate mapping leads to unavoidable inconsistencies in the data.

Exaggerated displaced shapes are typically relativized from a state of reference (i.e. magnifying only the differences between loaded and unloaded conditions), therefore the comparative error between states is crucial in the accuracy of the final result. The ensuing section presents the completed set of stitched coordinates in raw, unexaggerated form. It is observed that the incompatible longitudinal displacements undermines the results. However, Table F.3 shows that vertical translations ($\Delta_{3D}$) are within practical limits when compared to LVDT readings ($\Delta_{LVDT}$). Consequently, surface displacement plots in Section 3.1.6 were fabricated by means of magnifying only the vertical Y displacements relativized to LS 0a in order to produce qualitative renderings of the deformed profiles (Figure 3.20).

Table F.3 – Vertical displacement data comparison

<table>
<thead>
<tr>
<th>Load Stage</th>
<th>$\Delta_{LVDT}$ [mm]</th>
<th>$\Delta_{3D}$ [mm]</th>
<th>$\Delta_{LVDT} / \Delta_{3D}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0a</td>
<td>-0.010</td>
<td>0.685</td>
<td>0.01</td>
</tr>
<tr>
<td>0b</td>
<td>1.051</td>
<td>0.733</td>
<td>1.43</td>
</tr>
<tr>
<td>01</td>
<td>2.917</td>
<td>2.924</td>
<td>1.00</td>
</tr>
<tr>
<td>02</td>
<td>5.352</td>
<td>4.751</td>
<td>1.13</td>
</tr>
<tr>
<td>04</td>
<td>11.139</td>
<td>11.255</td>
<td>0.99</td>
</tr>
<tr>
<td>05</td>
<td>12.055</td>
<td>12.172</td>
<td>0.99</td>
</tr>
<tr>
<td>06</td>
<td>13.685</td>
<td>13.777</td>
<td>0.99</td>
</tr>
</tbody>
</table>
F.4 COMPLETE 3D SURFACE DISPLACEMENT PLOTS

Figure F.17 – PLS4000 surface displacement plots: XY elevation view
Figure F.18 – PLS4000 surface displacement plots: XZ plan view
Figure F.19 – PLS4000 surface displacement plots: YZ section view

Missing individual coordinates are due to accidental omission of different targets during the 3D scanning process at the time of testing.
APPENDIX G  ANALYTICAL MODELLING AND RESULTS

This appendix is used to describe in detail the aspects of analytical modelling demonstrated in Chapter 5. Specifics regarding each of the software packages are also explained to elucidate exactly how the author produced the analytical results such that they may be recreated. Results are not discussed as it was primarily the load-deformation responses that were of importance to the research and time constraints did not allow for their full documented assessment in this thesis. Nevertheless, they are presented for potential in-depth investigation by others.
G.1 VecTor2 Finite Element Analysis

VecTor2® (VT2) is a two-dimensional nonlinear finite element analysis (NLFEA) program used in combination with the preprocessor FormWorks® and the postprocessor Augustus®. The former allows simple model creation to be executed using an intuitive graphical user interface (GUI), while the latter is used to visually interpret analysis results (See Figure G.1) [45].

![Figure G.1 – FormWorks and Augustus processors for VecTor2](image-url)
APPENDIX G

ANALYTICAL MODELLING AND RESULTS

G.1.1 PLS4000

G.1.1.1 Finite Element Model

Constitutive material models used were Popovics for concrete in uniaxial compression due to the near “high-strength” classification of the concrete, and standard tri-linear reinforcement response for the steel. Load and support plates were modelled as steel in purely elastic state.

<table>
<thead>
<tr>
<th>Table G.1 – PLS4000 Finite Element Model Concrete Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>$f'_c$ [MPa]</td>
</tr>
<tr>
<td>$f'_t$ [MPa]</td>
</tr>
<tr>
<td>$E_c$ [MPa]</td>
</tr>
<tr>
<td>$\varepsilon'_c$ [mm/m]</td>
</tr>
<tr>
<td>$S_x$ * [mm]</td>
</tr>
<tr>
<td>$S_y$ † [mm]</td>
</tr>
</tbody>
</table>

* Spacing of cracks parallel to the Y (vertical) direction
† Spacing of cracks parallel to the X (longitudinal) direction

Default concrete parameters used in VecTor2 are determined as follows [45]:

$$f'_t = 0.33 \sqrt{f'_c} \ [MPa] \quad [G.1]$$

$$E_c = 5500 \sqrt{f'_c} \ [MPa] \quad [G.2]$$

$$\varepsilon'_c = 1.8 + 0.0075 \times f'_c \ [mm/m] \quad [G.3]$$

The numerical model was assumed to be represented by two dimensional plane-stress behaviour. Isoparametric quadrilateral finite elements were used over the entire slab strip to accommodate the changes of geometry that occur at the anchorage fins. A total of 53 elements were used over the specimen depth, corresponding to a very fine mesh size of 75 mm and a total of about 14,000 quadrilaterals in the model. Longitudinal reinforcement was modelled as discrete truss bars, while the transverse shear reinforcement was smeared over the respective spans for the different testing phases.
Bond slip mechanisms were modelled, however, after a preliminary analysis it was determined that bar slip would not occur even at the tensile forces expected to fail the stronger west span.

Self-weight was applied in a single load step through uniformly applied density assignments ($\rho = 2400$ kg/m$^3$) to all concrete elements. The point load was imposed as a nodal displacement at the loading point in monotonic fashion with increments of 0.1 mm until failure.

Key results from the VecTor2 analyses shall be presented in following subsections.

**G.1.1.2 Phase I Results**

<table>
<thead>
<tr>
<th>Stage</th>
<th>$P_{\text{tot}}$ [kN]</th>
<th>$P_{\text{tot}} - P_{\text{s/w}}$ [kN]</th>
<th>$\Delta$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Crack</td>
<td>629</td>
<td>174</td>
<td>1.70</td>
</tr>
<tr>
<td>Major Crack*</td>
<td>944</td>
<td>489</td>
<td>3.40</td>
</tr>
<tr>
<td>Peak (Ultimate)</td>
<td>1176</td>
<td>721</td>
<td>12.30</td>
</tr>
<tr>
<td>Post-Peak †</td>
<td>1036</td>
<td>580</td>
<td>13.50</td>
</tr>
</tbody>
</table>

* Indicated by visual drop in load-deformation response
† Lowest practical point in response after drop from ultimate

![Figure G.2 – Finite element model of PLS4000](image)

![Figure G.3 – VT2 crack pattern and deformations at post-peak for PLS4000 Phase I](image)
Figure G.4 – VT2 total principal tensile strain localization at post-peak for PLS4000 Phase I

Figure G.5 – VT2 principal concrete compressive stresses at peak load for PLS4000 Phase I

Figure G.6 – VT2 flexural reinforcement average stresses at peak load for PLS4000 Phase I
Average conditions (top) & conditions at crack (bottom)
Figure G.7 – VT2 shear reinforcement stresses at peak load for PLS4000 Phase I
Average conditions (left) & conditions at crack (right)

G.1.1.3 Phase II Results

Table G.3 – PLS4000 Phase II VecTor2 Load Stage Summary

<table>
<thead>
<tr>
<th>Stage</th>
<th>P_{tot}</th>
<th>P_{tot} - P_{s/w}</th>
<th>Δ</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kN]</td>
<td>[kN]</td>
<td>[mm]</td>
</tr>
<tr>
<td>First Crack</td>
<td>629</td>
<td>174</td>
<td>1.70</td>
</tr>
<tr>
<td>Major Crack*</td>
<td>936</td>
<td>480</td>
<td>3.30</td>
</tr>
<tr>
<td>Peak (Ultimate)</td>
<td>2425</td>
<td>1969</td>
<td>42.40</td>
</tr>
<tr>
<td>Post-Peak †</td>
<td>2195</td>
<td>1739</td>
<td>43.10</td>
</tr>
</tbody>
</table>

* Indicated by visual drop in load-deformation response
† Lowest practical point in response after drop from ultimate

Figure G.8 – VT2 crack pattern and deformations at post-peak for PLS4000 Phase II
Figure G.9 – VT2 total principal tensile strain localization at post-peak for PLS4000 Phase II

Figure G.10 – VT2 principal concrete compressive stresses at peak load for PLS4000 Phase II

Figure G.11 – VT2 flexural reinforcement average stresses at peak load for PLS4000 Phase II

Average conditions (top) & conditions at crack (bottom)
G.1.2 PLS300

G.1.2.1 Finite Element Model

The finite element analysis models and assumptions used for the PLS4000 slab strip were also used for the scaled down specimen. Table G.4 lists the concrete properties used in the model and Figure G.13 shows the finite element model.

<table>
<thead>
<tr>
<th>f'c [MPa]</th>
<th>f't [MPa]</th>
<th>E [MPa]</th>
<th>ε'c [mm/m]</th>
<th>Sx [mm]</th>
<th>Sy [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td>2.21</td>
<td>36,900</td>
<td>2.14</td>
<td>1000</td>
<td>1000</td>
</tr>
</tbody>
</table>

The PLS300 model is comprised of 5496 plane stress rectangular finite elements in a uniform 10 mm fine mesh. Imposed nodal displacement at the midspan was monotonically incremented by 0.05 mm until failure.
G.1.2.2 Results

Table G.5 – PLS300 VecTor2 Load Stage Summary

<table>
<thead>
<tr>
<th>Stage</th>
<th>P [kN]</th>
<th>Δ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Crack</td>
<td>18.4</td>
<td>0.15</td>
</tr>
<tr>
<td>Major Crack*</td>
<td>42.0</td>
<td>0.60</td>
</tr>
<tr>
<td>Peak (Ultimate)</td>
<td>75.6</td>
<td>2.60</td>
</tr>
<tr>
<td>Post-Peak †</td>
<td>58.2</td>
<td>2.95</td>
</tr>
</tbody>
</table>

* Indicated by stiffness change in response
† Lowest practical point in response after peak

Figure G.14 – VT2 crack pattern and deformations at post-peak for PLS300

Figure G.15 – VT2 total principal tensile strain localization at post-peak for PLS300
Figure G.16 – VT2 principal concrete compressive stress fields at peak load for PLS300

Figure G.17 – VT2 flexural reinforcement average stresses at peak load for PLS300
Average conditions (top) & conditions at crack (bottom)
G.2 **Response-2000 Sectional Analysis**

Response-2000© (R2k) is a sectional analysis program created by Evan C. Bentz used to analyze reinforced concrete members subjected to various loading states. Response-2000 has a full-fledged GUI designed for simple analysis of essentially any cross-section as well as a newly featured member analysis which is a powerful abstraction from the standard single section methodology (See Figure G.18).

![Figure G.18 – Response-2000 program graphical user interface](image-url)
G.2.1 PLS4000

G.2.1.1 Model

Default constitutive models were used for the concrete and reinforcement to provide a direct comparison with the VecTor2 results.

Abstraction to full-member response requires the definition of the various cross-sections representing the slab strip. PLS4000 requires three sections: a) east span without shear reinforcement, b) west span containing stirrups, and c) the repaired east span with external reinforcement simply modelled as closed stirrups in the program (See Figure G.19).

![Figure G.19 – PLS4000 cross-section definitions in Response-2000](image)

![Figure G.20 – PLS4000 full member properties control dialogue in Response-2000](image)
G.2.1.2 Phase I Results

**Figure G.21 – R2k displaced shape at failure for PLS4000 Phase I**

**Figure G.22 – R2k shear force diagram for PLS4000 Phase I [kN]**

*Effective force demand in red & resistance envelope in blue*

**Figure G.23 – R2k strain diagrams for PLS4000 Phase I**

**Figure G.24 – R2k shear stress profiles at critical section for PLS4000 Phase I [MPa]**
G.2.1.3 Phase II Results

**Figure G.25 – R2k displaced shape at failure for PLS4000 Phase I**

**Figure G.26 – R2k shear force diagram for PLS4000 Phase II [kN]**

*Effective force demand in red & resistance envelope in blue*

**Figure G.27 – R2k strain diagrams for PLS4000 Phase II**

**Figure G.28 – R2k shear stress profiles at critical section for PLS4000 Phase II [MPa]**
G.2.2 PLS300

G.2.2.1 Model

The PLS300 was modelled using a single cross-section definition with default material relations.

![Figure G.29 – PLS300 cross-section definition in Response-2000](image)

Self-weight is almost negligible for the much smaller specimen, and results are barely affected by excluding its effects.

![Figure G.30 – PLS300 full member properties control dialogue in Response-2000](image)
G.2.2.2 Results

**Figure G.31 – R2k displaced shape at failure for PLS300**

**Figure G.32 – R2k shear force diagram for PLS300 [kN]**

Effective force demand in red & resistance envelope in blue

**Figure G.33 – R2k strain diagrams for PLS300**

**Figure G.34 – R2k shear stress profiles at critical section for PLS300 [MPa]**
APPENDIX H  COPYRIGHT ACKNOWLEDGEMENTS

An article based on this thesis has been published by the American Concrete Institute (ACI) Concrete International Magazine in the November 2015 issue. Additionally, a technical paper similarly constructed from the research in this document is being submitted to the ACI Structural Journal for publication in the near future. It is acknowledged that the tables, figures, and text in both the ACI pieces are based on this thesis.