2012

Interfacial Strength Between Prestressed Hollow Core Slabs and Cast-in-Place Concrete Toppings

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INTERFACIAL STRENGTH BETWEEN PRESTRESSED HOLLOW CORE SLABS AND CAST-IN-PLACE CONCRETE TOPPINGS

A Thesis Presented

by

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ACKNOWLEDGEMENTS

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ABSTRACT

INTERFACIAL STRENGTH BETWEEN PRESTRESSED HOLLOW CORE SLABS AND CAST-IN-PLACE CONCRETE TOPPINGS

FEBRUARY 2012

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The horizontal shear strength of the interface between prestressed concrete hollow core slabs and cast-in-place concrete topping slabs was evaluated through a set of 24 push-off experiments. The push-off test specimens featured segments of dry-mix and wet-mix hollow core slabs with a variety of surface treatments including machine finished, sandblasted, broom roughened, rake roughened and grouted. A cast-in-place slab was poured on top of the hollow core specimens to form a 15 inch by 15 inch interface between the two materials. Results indicate the average horizontal shear strength of the push-off specimens was 227 psi. Higher shear strength and slip capacity was observed in specimens that were broom roughened in the direction transverse to the applied shear force and in grouted dry-mix specimens. Specimens with machine finished surfaces had lower average horizontal shear strength than those with intentionally roughened surfaces, but still exceeded the shear strength of 80 psi specified in the ACI 318-08 code. A method to comparatively quantify the surface roughness of the hollow core slabs with different surface treatments was adapted from an existing ASTM standard for pavements. This standard specifies the procedure to determine mean texture depth that can be correlated to horizontal shear strength of the push-off specimens. Analytical studies were also performed to estimate the maximum horizontal shear stresses that can
be expected in composite hollow core slabs under normal construction conditions. A finite element model was developed to observe the behavior of the horizontal shear failure mode for composite hollow core slabs.
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CHAPTER 1
INTRODUCTION

1.1 Introduction

Prestressed hollow core slabs have been used in a variety of structural applications since their origination in the 1950s, including residential and commercial buildings, parking structures and short span bridges. These slabs contain voids that run continuously along their length, which help reduce dead weight and material cost. In some applications, the voids within the slabs have been used to carry utility runs or heated air throughout a building. A photograph of typical hollow core slabs can be seen in Figure 1.1.

![Figure 1.1. Photograph of a hollow core slab.](image)

Hollow core slabs are often used because they are economical, have good fire resistance and sound insulation properties, and are capable of spanning long distances with relatively small depths. Hollow core slabs make use of prestressing strands, which allow slabs with depths between 6 and 10 in. (150 and 250 mm) to span over 30 ft. (9 m). Slabs with depths as high as 16 in. (410 mm) are available, and are capable of spanning over 50 ft. (15 m).

When used in buildings, several hollow core slabs are placed next to each other to form a continuous floor system. The small gap that is left between each slab is usually
filled with a non-shrink grout. To give the floor a smooth finished surface, a topping slab overlay is poured on the top surface of the hollow core slabs. This topping slab is typically 2 in. (5.08 cm) deep. The increase in slab depth may be used advantageously to develop higher flexural and shear strength. However, in order for the topping slab to contribute to the flexural strength of the slab, horizontal shear stresses must be transferred along the interface between the top of the precast slab and the bottom of the overlay. If the interface is capable of transferring these horizontal shear stresses, the enlarged section will behave as if it were composite. If it is determined that the section is not capable of acting composite, the additional strength of the topping slab must be neglected and its weight must be considered as superimposed dead load.

There are currently seven different companies that provide fabricators with the machinery needed to produce hollow core slabs. All hollow core slabs are formed by machines that traverse long casting beds ranging from 300 to 600 ft. (90 to 180 m). Some hollow core slab fabricators use a dry, low slump concrete that is extruded through a machine into the desired element shape. Other slabs are made using normal slump concrete that is poured by machine and allowed to cure within stationary formwork or moving slip forms. Beyond this, there are many other differences between fabrication approaches, such as the technique used to form the inner cores or outer edges of the slabs.

Due to the automated fabrication procedures for hollow core slabs, transverse reinforcement cannot be placed. The machines used to create the hollow core slabs are capable of working around the prestressing strands that run longitudinally within the slabs.
1.2 Motivation

An increase in the flexural and shear strength of hollow core slabs can be obtained through the addition of a 2 in. (5.1 cm) cast-in-place concrete topping slab, as long as the two elements act compositely. The addition of the topping slab increases the flexural strength of the element by increasing the effective depth between the compression block and the prestressing strands. Similarly, the shear strength is increased from the added depth. The additional capacities can lead to higher applied loading or longer spans.

In order to develop composite action, horizontal shear stresses must be adequately transferred along the horizontal interface between the hollow core slab and the topping slab. In situations where the horizontal shear stress at the composite interface exceeds available strength, steel ties may be placed crossing the interface for additional capacity. However, placement of these ties is not feasible in hollow core slab fabrication procedures. This leaves designers few options in situations where the horizontal shear strength at the interface must be increased.

Currently, section 17.5 of the *Building Code Requirements for Structural Concrete* (ACI 318-11) specifies a single horizontal shear strength value for all composite interfaces without steel, regardless of the roughness of the substrate surface. Several research projects on concrete shear friction and concrete-to-concrete composite connections have been conducted to date. It is generally acknowledged that surface roughness of the substrate concrete (in this case, the hollow core slab) is a parameter that affects the shear strength (Hanson, 1960, Seible and Latham, 1990, Djazmati and Pincheira, 2004, Raths and Hoigard, 2004, Kovach and Naito, 2008).

Various surface preparation techniques can be used to roughen the top surface of a hollow core slab prior to placement of the cast-in-place topping slab. Surface
preparation techniques are intended to increase strength of the connection between the precast and topping slab without the use of steel interface reinforcement. These techniques include brooming and raking the uncured concrete surface in the longitudinal or transverse direction, sandblasting the post-cured surface or applying a non-shrink grout to the surface.

Each surface preparation technique has different roughness properties, which can be seen in the shape, size, depth and orientation of the undulations. The techniques used when applying the surface treatment can also influence the roughness properties. For example, the amount of pressure applied with a rake or broom can lead to different roughness depths. When attempting to predict the strength of a composite interface, accurately quantifying these surface properties is a challenge. Surface profilometer equipment is capable of measuring surface roughness with accuracy, but is expensive, requires training to operate and has low portability. A method to quickly and easily quantify surface roughness may be useful to accurately predict the strength of the connection between precast and cast-in-place concrete.

The relevant research studies found in the literature have concentrated on concrete interfaces cast using traditional placement methods. None of the studies have specifically investigated the interface strength of machine-produced concrete elements such as hollow core systems. Machine production is economical but limits the variety of techniques (either surface roughening or placement of ties crossing the interface) that can be employed efficiently to increase interface shear strength. The motivation of this research is to provide interface shear strength data of machine produced precast elements, and to investigate the benefits of surface roughening techniques that can be employed without significantly affecting production efficiency.
1.3 Scope and Organization

Background information on the horizontal shear evaluation methods specified in current design codes is provided in Chapter 2. The mechanisms used to resist horizontal shear are explained and a summary of existing research literature on this topic is also included in this chapter.

To determine the most effective surface roughening procedures for precast members that receive a composite topping slab, an experimental program has been developed (Chapter 3). A series of push-off tests were performed on hollow core slabs segments with cast-in-place toppings. A variety of different surface roughening methods were evaluated during the testing program, as well as both dry-mix (low slump) and wet-mix (normal slump) hollow core slab concrete types. A surface roughness quantification method for precast concrete surfaces was adapted from an existing measurement standard. The results of the surface roughness quantification measurement are used to evaluate the relationship between interface shear strength and surface roughness.

A numerical study to determine the conditions producing the most critical horizontal shear stresses was conducted as described in Chapter 4. In addition, a finite element model was developed to develop a better understanding of the horizontal shear failure mode and to observe the effects of interface strength and stiffness on horizontal shear failure (Chapter 5). Finally, a summary, concluding remarks and suggestions for future work are given in Chapter 6.
CHAPTER 2
BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

This chapter provides background on the mechanisms that resist horizontal shear and the methods that are used to evaluate horizontal shear strength. An understanding of the mechanisms used by concrete-to-concrete composite interfaces to resist horizontal shear is valuable when interpreting experimental testing results. The horizontal shear strength specifications in current building codes and design guides help put the experimental results into context with the design practice for composite prestressed members.

In addition, relevant existing literature is summarized in this chapter. The findings of past research were valuable when designing aspects of the experimental research program such as the testing matrix, the test specimen and instrumentation.

2.2 Building Codes and Design Guides

The design of hollow core slabs in the United States is governed by the Building Code Requirements for Structural Concrete (ACI 318-11). The 7th edition of the PCI Design Handbook (PCI 2010) includes additional design guidance and insight into standard design practices. This latest edition of the PCI Design Handbook (PCI 2010) conforms to ACI 318-05, but the appendix of the handbook discusses the impacts of changes in the ACI 318 code published in 2008. Design procedures pertaining to hollow core slabs in ACI 318-11 remain largely unchanged from ACI 318-08, so the latter will be used for comparison with the latest PCI Design Handbook in this thesis.
Design guidance specific to hollow core slabs can be found in the *PCI Manual for the Design of Hollow Core Slabs* (Buettner and Becker, 1998). This document was last updated in 1998 and is in conformance with a much older version of the ACI Building Code (ACI 318-95).

### 2.3 Horizontal Shear

Horizontal design shear strength calculated in ACI 318-08 in section 17.5.3 must exceed the factored shear force, as shown in Equation 2.1.

\[
\phi V_{nh} \geq V_u
\]

2.1

where:
- \( \phi \) = Strength reduction factor, equal to 0.75
- \( V_{nh} \) = Nominal horizontal shear strength
- \( V_u \) = Factored horizontal shear force

The horizontal shear strength according to ACI 318-08 code may be calculated using one of two methods in a composite member. The first method, identified as method A in this thesis, corresponds to the method in section 17.5.3 of ACI 318-08. In this method, the design horizontal shear strength must exceed the vertical shear force at any section (Equation 2.2). This method is also illustrated in Figure 2.1.

\[
\begin{align*}
\phi V_{nh} & \geq V_u \\
\phi V_{nh} &= \phi(80)b_v d \quad \text{(psi)} \\
\phi V_{nh} &= \phi(0.55)b_v d \quad \text{(MPa)}
\end{align*}
\]

2.2

where:
- \( V_u \) = Factored horizontal shear force, found from vertical shear diagram
- \( b_v \) = Composite interface contact width
- \( d \) = Depth of composite section, measured from the top of the composite section to the centroid of all tension steel
If horizontal shear evaluation method A is used for the simply supported beam with distributed loading shown in (a), the vertical shear diagram shown in (b) is taken as $V_u$.

At all sections, the horizontal shear strength, $V_{nh}$, must exceed $V_u$ as shown in (c).

**Figure 2.1.** Horizontal shear evaluation method A.

The second method to compute horizontal shear demand is given in ACI 318-08 section 17.5.4. In this method, the horizontal shear force is calculated as the change in compressive force in the topping slab between two beam sections; this force must not exceed the horizontal shear strength given by Equation 2.2. This method is also presented in Section 5.3.5 of the *PCI Design Handbook* as summarized in Equation 2.3.

The *PCI Design Handbook* description of this evaluation method, identified as method B in this thesis, limits the average horizontal shear stress along the length of a simply supported beam to a maximum horizontal shear strength of 80 psi (0.55 MPa). The distance over which average horizontal shear stresses are calculated is $l_{sh}$, which is defined as the distance between the section of maximum moment and a section of zero moment. For a simply supported beam with uniformly applied load, the actual magnitude of the shear force at the supports is twice as large as the average shear force between support and section of maximum moment (Figure 2.2). However, the approach adopted in the *PCI Design Handbook* neglects the variation of shear stresses within the shear span and uses an average value for simplicity.
In ACI 318R-08 section R17.5.4.1, it is stated that the distribution of shear stresses along the composite beam length should resemble the distribution of vertical shear along the member. The assumptions in the shear stress calculations when applying method B differ if one adopts the ACI 318 or the *PCI Design Handbook* interpretations. The assumption of average horizontal stress made by the *PCI Design Handbook* could only be relied on if redistribution of horizontal shear occurs. It could be argued that if horizontal shear stress redistribution does occur after failure initiates at the section of maximum shear, then the horizontal shear strength could be estimated by using an average value over the shear span. It is not clear, however, that this redistribution can occur specifically with interfaces that do not contain reinforcement crossing the horizontal planes.

\[
\phi V_{nh} \geq V_u \\
V_u = \left( \min \begin{array}{c} 0.85f'_c A_{top} \\ 0.85f'_c a b_v \end{array} \right)
\]

\[
\phi V_{nh} = \phi (80)b_v l_{vh} \quad \text{(psi)}
\]

\[
\phi V_{nh} = \phi (0.55)b_v l_{vh} \quad \text{(MPa)}
\]

where:
- \(f'_c\) = Topping slab concrete compressive strength
- \(A_{top}\) = Cross sectional area of the topping slab
- \(a\) = Depth of compression block at section of maximum positive flexure
- \(b_v\) = Width of composite interface
- \(l_{vh}\) = Length of horizontal shear span, equal to half the total span length for simply supported beams.
If horizontal shear evaluation method B is used for the simply supported beam with distributed loading shown in (a), $V_u$ is taken as the difference in compression force in the topping slab between points f and g in diagram (b).

This method results in an average $V_u$ value along the entire span length as shown in (c). The magnitude of shear found near the supports is underpredicted by this method as seen in (d).

At all sections, the horizontal shear strength, $V_{nh}$, must exceed $V_u$ as shown in (e).

**Figure 2.2.** Horizontal shear evaluation method B.

When using the *PCI Design Handbook* approach to method B, higher horizontal shear strength results would be obtained compared to method A due to the averaging of horizontal shear stress. Another difference between these two methods is the use of the depth parameter $d$ in method A, which is a simplification of the lever arm between the tensile and compressive cross section forces, which can be stated more formally as $(d - \frac{a}{2})$, where $a$ is the depth of compressive stress block.

Authors have also made use of Equation 2.4 to calculate horizontal shear stress. This equation was used in studies by Hanson (1960), Saemann and Washa (1964) and Kovach and Naito (2008) as a common basis for comparison of shear stress between experimental test specimens. Since this equation is based on a homogeneous elastic beam, accuracy is lost when cracking or interface slip occurs.
$v_u = \frac{VQ}{lb_y}$

where:

$v_u$ = Horizontal shear stress  
$V$ = Vertical shear force at location of interest  
$Q$ = First moment of the topping slab area about the neutral axis  
$l$ = Moment of inertia of cross section  
$b_y$ = Width of composite interface

ACI 318-08 does not currently specify the use of Equation 2.4 for evaluation of horizontal shear. However, this equation was included in the ACI code prior to 1970 (Loov and Patnaik, 1994). According to Loov and Patnaik (1994), when working with cracked beams, this equation can be made more accurate if $I$ and $Q$ are found using cracked section properties.

### 2.4 Horizontal Shear Mechanisms

Over the past 50 years, many models have been proposed to evaluate the shear strength of a concrete plane. These models have been used to evaluate the shear transfer capacity of a concrete crack, a composite interface, or a plane of monolithic concrete. These models account for shear resistance through actions known as cohesion, aggregate interlock and shear friction. The models that have been used more widely over the years are described in the following sections.

#### 2.4.1 Shear Friction

As a concrete plane is subjected to in-plane shear forces (whether it be monolithic concrete, a composite interface or a crack), interlocking of aggregate particles protruding from either side of the shear plane occurs. Particle roughness generates normal forces
which have the effect of separating the shear plane. If reinforcing steel passes through the plane, normal (clamping) tensile forces are developed in response to the tendency of the shear plane to separate. The tensile forces in the reinforcing steel generate a normal stress on the shear plane causing friction forces to develop and resist the applied shear force (Figure 2.3).

1. A composite interface between two concrete elements is subjected to shear.

2. As the two surfaces slip relative to each other, the protruding aggregate particles or surface roughness must travel over each other. This provokes a tendency for the interface to separate apart.

3. If reinforcing steel crosses the interface, a normal tensile stress is generated in response of the tendency to separation.

4. The tension in the steel ties applies a compressive stress normal to the interface. This compressive stress generates friction that resists the applied shear.

**Figure 2.3.** Illustration of shear friction mechanism.

In addition, surface roughness is fundamental to develop composite action between surfaces with and without steel ties. When ties are not present, surface roughness causes aggregate interlock to occur and facilitates additional cohesion by providing more contact surface area. However, when ties are present, the surface roughness also provides undulations that must travel over each other in order to generate tension in the steel (and therefore friction at the interface).
2.4.2 Cohesion and Aggregate Interlock

Cohesion and aggregate interlock are the primary strength mechanisms that develop between old and new concrete composite interfaces that lack steel reinforcement across them. Walraven et al. (1987) explain that a concrete plane has both general and local roughness, as illustrated in Figure 2.4. Cohesion is a term used to describe the cementitious bond that resists shear in uncracked concrete planes. According to Walraven et al., cohesion occurs between local roughness asperities.

Aggregate interlock is a term used to describe the mechanical interlocking of aggregate particles on either side of the shear plane. According to Walraven et al., this interlocking occurs between general roughness asperities.

![Figure 2.4. Illustration of general and local roughness as defined by Walraven et al. (1987).](image)

When steel cannot be used across a composite concrete interface, cohesion and aggregate interlock must be relied on to resist shear. The lack of existing data on these mechanisms, particularly between hollow core products and cast in place concrete has inspired this research project.

2.5 Past Research on Horizontal Shear and Composite Concrete Toppings

Many studies have been performed to quantify the shear strength of a concrete plane. A majority the studies focusing on composite concrete interfaces also included the
effects of interface reinforcement. The studies shown below were relevant to this thesis because they introduced noteworthy shear friction models, involved the testing of unreinforced concrete to concrete interfaces or otherwise provided information on unreinforced composite connections.

2.5.1 Hanson, 1960

Hanson (1960) performed a series of push-off and girder tests to explore the behavior of horizontal shear transfer. The experiments explored a variety of interfaces with and without steel reinforcement.

Of the push-off tests without steel reinforcement, three different surface treatments were explored: smooth, rough, and rough aggregate bare. The roughened surfaces were made by scraping the concrete surface with the edge of a sheet of steel. The rough aggregate bare surfaces were made by applying a retarder to delay the set of the top inch of the roughened concrete surface, then washing this uncured layer away 24 hours later using a water jet. Hanson found that the maximum shear stress for composite action of roughened and smooth surfaces was 500 psi (3.44 MPa) and 300 psi (2.07 MPa), respectively. It was determined that the rough aggregate bare specimens performed similarly to the roughened specimens.

Hanson also performed a series of residual slip experiments on rough bonded push-off specimens without reinforcement. The specimens were loaded to a non-peak slip value, then unloaded to zero load, at which point the residual slip was measured. It was found that loading to slip magnitudes less than approximately 0.001 in. (0.25 mm) resulted in no residual slip when unloaded. If the push-off specimens were loaded to a
slip level beyond 0.001 inches, a residual slip of approximately 1/3 the applied slip would remain.

Only two of the ten girders tested by Hanson were without steel reinforcement. One of these two specimens had a rough bonded surface while the other was a monolithic control specimen. Hanson noted that as load increased flexural cracks formed and progressed upwards until reaching the composite interface. At this point, the cracks propagated along the interface for a short distance. Failure of the girder was caused by a loss of composite action which led to a compression-shear failure. Hanson noted that “there was no marked difference between the monolithic girder and that with a rough, bonded connection.”

Hanson concluded that push-off tests showed similar shear and slip characteristics as the girder tests. Hanson also concluded that composite action is often lost at a slip of 0.005 inches; however, it is unclear if this value applies to all composite interfaces that were studied or just those with steel reinforcement.

### 2.5.2 Birkeland and Birkeland, 1966

Birkeland and Birkeland (1966) introduced the concept of shear friction in 1966 as a helpful tool for evaluating heavily loaded concrete connections. The model was developed to be applicable to any plane of concrete, including monolithic concrete, concrete cracks or composite interfaces. The model, expressed mathematically in Equation 2.5, idealized the concrete shear plane as a series of interlocked frictionless sawtooth ramps that would need to slide over each other when loaded in shear. According to the model, horizontal shear strength increased linearly with the quantity of steel crossing the shear plane and a plane without any steel would have no strength.
where:

\[ \nu_n = p f_y \tan \phi \]

- \( \nu_n \) = Horizontal shear strength (limited to 800 psi)
- \( p \) = Steel ratio, \( A_s / A_g \)
- \( A_s \) = Area of steel crossing the shear plane
- \( A_g \) = Gross area of the shear plane
- \( \phi \) = Angle of sawtooth ramps in idealized model
- \( \tan \phi = 1.7 \) for monolithic concrete
- \( \tan \phi = 1.4 \) for artificially roughened construction joints
- \( \tan \phi = 0.8 \) to \( 1.0 \) for ordinary construction joints

### 2.5.3 Loov, 1978

Loov (1978) was the first author to add the concrete compressive strength to the shear friction formulation, as shown in Equation 2.6. As with the Birkeland and Birkeland (1966) model, this model could not be used for shear planes without steel reinforcement. The shear strength contribution of cohesion and aggregate interlock were not explicitly included in this model. Experimental tests by other authors (Kovach and Naito, 2008) were unable to prove a relationship between horizontal shear strength and concrete compressive strength for interfaces without steel reinforcement.
\[ v_n = k \sqrt{\rho \sigma_y f_c'} \]

where:

- \( v_n \) = Horizontal shear strength
- \( \rho \sigma \) = Steel ratio, \( A_s/A_g \)
- \( A_s \) = Cross sectional area of steel crossing the shear plane
- \( A_g \) = Gross area of the shear plane
- \( f_y \) = Yield strength of steel crossing the shear plane
- \( f_c' \) = Compressive strength of concrete
- \( k \) = Constant, based on shear plane type
- \( k = 0.5 \) suggested for monolithic concrete

### 2.5.4 Seible and Latham, 1990

Seible and Latham (1990) performed two series of experimental tests on concrete-to-concrete composites. The study consisted of 14 small scale shear tests and 12 large scale composite slab panel load tests.

The small scale test phase evaluated the strength of monolithic, lubricated, lightly sandblasted, and post-cure scarified surface specimens. The surfaces of the lubricated specimens were lightly sandblasted and sprayed with bond breaking formwork oil prior to placement of the overlay concrete. The authors noted that the post-cure scarification process removed the entire laitance layer of the substrate concrete block; the same note was not made for the lightly sandblasted specimens. Specimens with and without reinforcing steel crossing the interface were tested for each of the small scale specimen types listed. A drawing of the small scale specimen used by Seible and Latham can be seen in Figure 2.5. The small scale specimens were tested in a vertical arrangement so the shear strength of the interface with minimal normal stress could be found.
The lubricated specimens without steel reinforcement had an average interface shear strength of 17 psi (.12 MPa). The pre-cure trowel roughened and post-cure scarified specimens without steel reinforcement had average interface shear strengths of 56 and 105 psi (.39 and .724 MPa), respectively.

These results indicate that different methods of surface preparation could lead to different interface shear strengths. The contrast between the low strength of the lubricated specimens and the significantly higher strengths of the bonded specimens suggests that cohesion is capable of providing resistance to shear. If it is assumed that no chemical bond existed between the surfaces of the lubricated specimens and the testing apparatus applied no normal stress to the interfaces, then the interface shear strength contributions of cohesion and shear friction would have been zero for these specimens. It would then follow that the aggregate interlock of the pre-cure trowel roughened surface provided all 17 psi (0.12 MPa) of interface shear strength.

The authors noted that the post-cure scarification process removed the entire laitance layer of the substrate concrete block. It was not clear if the difference in strengths between the pre-cure trowel roughened and post-cure scarified surfaces is attributed to different contributions of cohesion or aggregate interlock.
Gage readings revealed that the horizontal slip along the composite interface was approximately equal in magnitude to the vertical separation of the interface during all stages of loading. This relationship was valid for all small scale specimens, including those intentionally roughened.

The small scale specimens with steel reinforcement passing through the interface had significantly higher ultimate strengths than those without steel. For these specimens, $\rho_v$, the ratio between the area of steel provided and the concrete contact surface area, was equal to 0.0028 (0.28%). This $\rho_v$ value was much larger than the minimum reinforcement ratio, $\rho_{v,min}$, required by ACI 318-08 section 11.4.6.3 of 0.083% for concrete strength of 4000 psi (28 MPa) and steel yield strength of 60 ksi (400 MPa).

The large scale phase involved testing 8 simply supported composite reinforced concrete slab panels and 4 continuous reinforced concrete slab panels. The same surface preparation conditions were used as in the small scale test phase. The specimens with interface steel reinforcement had a $\rho_v$ value of 0.07%, which was much lower than that of the small scale tests (0.28%). The reinforced concrete panel had a height of 6.0 in. (15 cm) and a width of 24 in. (61 cm). The concrete topping slab added to this panel also had a height of 6.0 in. (15 cm) and width of 24 in. (61 cm).

During the testing of the slab panels with the lubricated surface treatment, tension cracks formed independently on the bottom of the reinforced concrete panels and the bottom of the concrete topping slabs. Similar cracking patterns were not noted for any of the bonded slab tests, indicating that cohesion is a significant factor for resisting horizontal shear associated with flexural loading.
The authors concluded from the panel tests that interface shear strength could only be influenced by adding very high quantities of reinforcement (well above the minimum required, specified in the ACI code).

2.5.5 Scott, 1973

A full scale load test of a hollow core slab with a composite cast-in-place topping was performed by Scott (1973) as a means of quality assurance for a prestressed concrete manufacturer. The hollow core slab had a total depth of 8 in. (20 cm), a width of 24 in. (61 cm) and a span of 31.5 ft. (9.60 m). The composite topping had a depth of 2 in. (5 cm). Prior to placement of the topping slab, the hollow core slab had a smooth machine cast finish (not intentionally roughened) and did not have any steel reinforcement passing through the composite interface.

The slab failed in flexure at a load higher than that corresponding to the nominal flexural strength of the composite section calculated using the 1971 ACI Code (ACI 318-71). Scott noted that composite action was evident up to ultimate load, which coincided with a midspan deflection of over 15 in. (38 cm).

2.5.6 Djazmati and Pincheira, 2004

Djazmati & Pincheira (2004) performed a series of push-off experiments to investigate the stiffness of concrete construction joints made with varying surface preparation techniques. In this study, small scale push-off tests were performed on composite specimens without reinforcing steel passing across the interface. A schematic of the test specimen used by Djazmati and Pincheira (2004) can be seen in Figure 2.6.
Figure 2.6. Push-off test specimen used by Djazmati and Pincheira (2004)

This study examined the influence of roughening technique, dampness of surface prior to concrete placement, and consolidation technique of topping concrete on the strength and stiffness of the resulting interface. Monolithic push-off specimens and specimens with shear keys were also tested.

The researchers found that a connection formed by casting concrete onto a smooth existing concrete surface has a lower horizontal shear strength than that of a roughened surface. Few distinctions in strength were observed between specific strategies of roughening, such as raking or brooming. Vibration consolidated specimens with trowel finished surfaces had an average interface shear strength of 361 psi (2.49 MPa). Vibration consolidated specimens that were roughened with a broom in a direction perpendicular to loading had an average interface shear strength of 582 psi (4.01 MPa).

Djazmati and Pincheira found that composite interfaces can have the same initial shear stiffness as monolithic concrete up to a shear stress of approximately 400 psi (2.8 MPa) for roughened surfaces and 250 psi (1.7 MPa) for trowel finished surfaces. It was also found that wet joints (saturated with water prior to placement of topping concrete) had lower strength than dry joints. The authors recommended moist curing the bottom
surface, but to not allow freestanding water prior to the placement of the topping concrete.

2.5.7 Raths and Hoigard, 2004

Raths and Hoigard (2004) stressed the importance of quality control for composite concrete toppings without interface reinforcement. The authors provide recommendations for achieving a high quality bond between precast concrete components and topping overlays.

Intentionally roughening the substrate surface to promote mechanical interlock and to increase the contact surface area with the topping slab is strong recommended. Although Section 5.3.5 of the PCI Design Handbook states that normal finishing used for precast concrete components may qualify as intentionally roughened, Raths and Hoigard state that this provision “conflicts with [their] experience and knowledge of concrete topping disbanding.” It also is stressed that proper cleaning of debris on the substrate surface is needed prior to placement of the topping overlay in order to achieve bond.

Raths and Hoigard also explain that improper composite bond can lead to debonding through processes other than overloading. Improperly detailed building systems can develop cracking within the topping (or near the edges due to curling), which could bring about further debonding if exposed to an outdoor environment.

2.5.8 Kovach and Naito, 2008

Kovach and Naito (2008) performed two phases of load tests on prestressed concrete girders with composite topping slabs without interface ties. The testing program examined four different surface preparation techniques: as-placed, broomed, raked and sheepfoot roughened. Monolithic control specimens were also tested.
The 7 of the 19 girders tested during the first experimental phase were simply supported and subjected to five-point (simulated distributed) loading. The remaining 12 girders were simply supported and subjected to two-point loading.

The girders tested under five-point loading failed due to either flexural or flexural-shear cracking. It was noted that the interface remained composite throughout the five-point loading tests, regardless of surface preparation.

The girders tested under two-point loading exhibited interface failure followed immediately by a flexural-shear failure once composite action was lost. From the two-point girder tests the authors found that the smooth surface had the lowest horizontal shear strength, 787 psi (5.4 MPa). The rake roughened surface had the highest shear strength of 1084 psi (7.48 MPa).

The second phase of testing was performed with the goal of increasing the number of companion specimens so more reliable design recommendations could be made. All test specimens during this phase were simply supported and subjected to two-point loading. An average horizontal shear stress of 498 psi (3.43 MPa) was found for broom finished specimens. An average horizontal shear stress of 821 psi (5.66 MPa) was found for the rake finished specimens.

After experiencing problems during the second testing phase involving smooth trowelled surface specimens debonding prior to loading, the authors recommended against the use of smooth trowelled specimens due to a lack of cohesion.
3.1 Introduction

The experimental testing program was developed with the goal of finding the horizontal shear strength of composite interfaces formed between precast hollow core slab surfaces with a variety of surface roughnesses and cast-in-place concrete overlays. Through this testing program, the interfacial strength generated by different surface preparation techniques was found.

3.2 Testing Methods

A test specimen was designed to subject the interface between precast and cast-in-place concrete to horizontal shear (push-off specimen). A typical specimen consisted of a block of cast-in-place concrete cast directly on top of the surface of a hollow core slab segment. The use of actual hollow core slabs in the push-off specimens ensured that realistic composite interfaces were examined and that the actual surface conditions generated by precast fabricators were tested. During testing, a monotonically increasing shear force was applied to the composite interface using a hydraulic actuator.

In the discussion of this testing program, the cast-in-place block will be referred to as the top block. The hollow core slab segment will be referred to as the bottom block.

3.2.1 Test Specimen Description

Push-off testing was performed on a total of 24 specimens with six different surface preparation techniques used in the hollow core segments. The hollow core slabs segments were provided by the two precast concrete fabricators shown in Table 3.1. All
hollow core slab specimens were created using typical fabrication techniques. These hollow core slab segments were used as the bottom blocks.

Twelve slab samples for small scale testing were provided by Oldcastle Precast Inc. of South Bethlehem, New York. Oldcastle produces Elematic hollow core slabs, which are fabricated using a low slump, dry concrete mix. The concrete had a design compressive strength of 5000 psi at 28 days. A photograph of an Oldcastle hollow core slab cross section can be seen in Figure 3.1.

![Figure 3.1. Photograph of Oldcastle hollow core slab cross sections.](image)

J.P. Carrara & Sons, Inc. of Middlebury, Vermont provided twelve slab samples to be tested. J.P. Carrara produces Dynaspan hollow core slabs, which are fabricated using a normal slump, wet concrete mix. A photograph of a J.P. Carrara & Sons hollow core slab cross section can be seen in Figure 3.2.

![Figure 3.2. Photograph of J.P. Carrara & Sons hollow core slab cross sections.](image)

The geometry of the push-off test specimen was designed such that the load application, reaction and interface were all located along the same plane in order to only generate horizontal shear stresses on the interface. This configuration avoided the creation of normal stresses on the composite interfaces, which may affect the
experimental results. A conceptual drawing of a push-off test specimen is shown in Figure 3.3.

To transfer loading into the hollow core slabs, a steel assembly was cast into each bottom block as shown in Figure 3.4, Figure 3.5 and Figure 3.6. The steel assembly was embedded into the bottom block using eight shear studs and additional cast-in-place concrete that filled into the cylindrical voids of the slab. A portion of the steel assembly beared directly against the edge of the bottom block. The top block was cast in an L-shape to allow the line of load application to be along the top surface of the bottom block.

![Figure 3.3. Schematic of push-off test specimen.](image)
**Figure 3.4.** Reinforcing details of push-off specimen (elevation view)

**Figure 3.5.** Reinforcing details of push-off specimen (side view)
The top blocks contained 4 #5 mild reinforcing bars. Since the bottom blocks were cut from actual hollow core slabs, they contained prestressing strands. It was expected that the full prestressing force did not develop in the bottom blocks due to their short length (36 in., 914 mm). No steel reinforcement passed through the composite interface between top and bottom blocks for any of the test specimens.

The interface was created by promoting contact on a 15 in. by 15 in. (38.1 cm by 38.1 cm) surface between the top and bottom blocks. The remaining surface of the top block was separated from the bottom block using smooth tape. Shin & Lange (2004) showed that curling at the edge of a topping layer of concrete may occur due to shrinkage. A 3 in. (76.2 mm) border of debonding tape was applied around the perimeter.
of the interface to isolate the test region from any edge effects such as edge curling due to shrinkage.

The test matrix listed in Table 3.1 was followed during the small scale testing phase. Two machine finished specimens, two longitudinally roughened specimens, two transversely roughened specimens, two sandblasted specimens, two grouted machine finished specimens, and two grouted longitudinally roughened specimens were tested for both the dry and wet-mix hollow core slabs. Photographs of typical hollow core slab surfaces can be seen in Figure 3.7 for the dry-mix specimens and Figure 3.8 for the wet-mix specimens.

Table 3.1. Number of specimens of each surface preparation type provided by the hollow core slab fabricators participating in this study.

<table>
<thead>
<tr>
<th>Fabricator:</th>
<th>Dry-mix bottom block</th>
<th>Wet-mix bottom block</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oldcastle Precast, Inc.</td>
<td></td>
<td>J.P. Carrara &amp; Sons, Inc.</td>
</tr>
<tr>
<td>Casting Machine:</td>
<td>Elematic</td>
<td>Dynaspan</td>
</tr>
<tr>
<td>Concrete mix type:</td>
<td>Dry-mix</td>
<td>Wet-mix</td>
</tr>
<tr>
<td>Concrete slump:</td>
<td>Low slump</td>
<td>Normal slump</td>
</tr>
<tr>
<td>Number of specimens provided*:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Machine finished</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Post-cure roughened (Sandblasted)</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Longitudinally rake roughened</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>Longitudinally broom roughened</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>Transversely broom roughened</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Machine finished and grouted</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Longitudinally rake roughened and grouted</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>Longitudinally broom roughened and grouted</td>
<td>-</td>
<td>2</td>
</tr>
</tbody>
</table>

*Note: “Longitudinally” describes the direction that is parallel to the span length. “Transversely” describes the direction that is perpendicular to the span length.

Machine finished specimens are those where the surface of the hollow core samples were left as produced by the fabrication machine without subsequently using any methods intended to roughen the surface. This type of surface is free of undulations caused by rakes, brooms or any other means of intentional roughening. It should be
noted, however, that machine finished surfaces still contained roughness typical of concrete elements but dependent on the fabrication method used.

The dry-mix longitudinally roughened specimens were raked by the fabrication machine used to extrude the hollow core slab concrete. This was done by a rake apparatus that was attached to the machine and automatically dragged over the surface. The dry-mix specimens with transverse roughness were manually broomed. The wet-mix specimens with both transverse and longitudinal roughness were also manually broomed.

<table>
<thead>
<tr>
<th>Machine finished</th>
<th>Post-cure roughened (Sandblasted)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Broom roughened in the transverse direction</td>
<td>Rake roughened in the longitudinal direction</td>
</tr>
<tr>
<td>Machine finished and grouted</td>
<td>Rake roughened in the longitudinal direction and grouted</td>
</tr>
</tbody>
</table>

**Figure 3.7.** Photographs of dry-mix hollow core slab surfaces.

Hollow core slabs are typically erected side by side to form a building slab. Any gap left between hollow core elements is typically filled with a flowable grout. This grout occasionally flows onto regions of the hollow core slab surface prior to placement of the topping concrete layer. To simulate this construction condition, grout was applied to the interface surface in some specimens. Grout was applied in a thin (1/16 in. [1.59 mm]) layer and was allowed to dry prior to pouring the top block concrete. The top block
would normally be cast between 24 and 48 hrs after applying the grout on these specimens. With the exception of the grout application, all surface preparations of the bottom blocks (hollow core slab) were performed by the fabricators to ensure consistency.

![Figure 3.8. Photographs of wet-mix hollow core slab surfaces.](image)

### 3.2.2 Test Specimen Construction

A typical concrete mix representative of topping slabs constructed in the field was used to fabricate the cast-in-place top block for the push-off specimens. This mix had a water/cement ratio of 0.46 and a design compressive strength of 4000 psi. Small 3/8 in (10 mm) coarse aggregate was used in this mix. Additional details of the concrete mix can be found in Table 3.2. The top surface of the bottom block was cleaned using compressed air and lightly dampened without leaving standing water prior to placement of the top block concrete.

**Table 3.2.** Top block concrete mix design.
<table>
<thead>
<tr>
<th>Component</th>
<th>Notes</th>
<th>Percent (by mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Portland Type I/II</td>
<td>20%</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>3/8 in. diameter or smaller</td>
<td>42%</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>Fineness Modulus = 3.0</td>
<td>28%</td>
</tr>
<tr>
<td>Water</td>
<td></td>
<td>10%</td>
</tr>
</tbody>
</table>

To ensure that the composite interface was not damaged prior to testing, the top block was cast after the bottom block was positioned on the testing frame. Once the top block was cast, the test specimen was not moved until after testing was completed. A photograph of a top block during construction is shown in Figure 3.9. A photograph of a test specimen ready for testing can be seen in Figure 3.10.

![Figure 3.9. Photograph of top block being cast onto bottom block.](image)

![Figure 3.10. Photograph of a test specimen prior to testing.](image)

### 3.2.3 Test Specimen Instrumentation

Push-off specimens were instrumented with as many as eight displacement gages to record relative movement between top and bottom blocks during testing. Six
transducers were arranged along both sides (North and South) of the specimen to measure relative movement between the top and bottom blocks. Two gages were installed near the far end of the top block to monitor vertical movement of the top block relative to the hollow core segment (Figure 3.11).

The displacement transducers were anchored to the bottom block with the active end measuring the displacement of a target adhered to the top block. A 110 kip (489 kN) load cell on the hydraulic actuator recorded the force being applied to the specimen. The location of each gage is shown in Figure 3.11.

For the first 12 specimens, string potentiometers were used to measure the horizontal slip. For the remaining 12 specimens, the string potentiometer gages at locations “North 1” and “South 1” were replaced with linear potentiometers. The string potentiometers and linear potentiometers were attached to each specimen as shown in Figure 3.12.
Vertical displacements were recorded for 19 of the 24 test specimens. These displacements were measured by linear potentiometers, attached to the specimens as shown in Figure 3.13. Displacement and force data were recorded at one second intervals throughout each experiment for all push-off specimens.
3.2.4 Determination of Concrete and Grout Strength

The concrete strength of all push-off specimens was determined using concrete cylinder tests. Three 8 by 4 in. (200 by 100 mm) compression cylinders and two 12 by 6 in (300 by 150 mm) split (tension) cylinders were tested for each top block in accordance with ASTM C39 (2005) and ASTM C496 (2004), respectively. These cylinders were
fabricated in accordance with ASTM C192 (2007) and exposed to the same curing
conditions as their companion top block. Curing of the top block was accomplished by
covering the concrete with wet burlap for 48 hours after casting. To avoid excessive
evaporation, a plastic sheet was placed on top of the burlap.

Both participating hollow core slab manufacturers provided twelve 8 by 4 in. (200
by 100 mm) concrete cylinders which were tested under compression in accordance with
ASTM C39 (2005). Compression testing of these cylinders was conducted at different
times throughout the duration of the entire set of tests from each manufacturer to track
cement strength variation with time and estimate the strength of each hollow core slab at
the time of push-off testing. The cylinders were tested in pairs after every other push-off
specimen was tested. The first pair of these cylinders was tested along with the first
push-off specimen from each manufacturer. The hollow core slab specimens provided by
the dry-mix fabricator all originated from a single cast at the fabrication facility. All
specimens provided by the wet-mix fabricator came from a single cast with the exception
of one longitudinally broomed specimen, which was poured on a different day. Care was
taken to test cylinders that were cast on the same date as the corresponding push-off test
specimen.

Specimens that were roughened using a layer of grout were accompanied with 2
by 2 in. (50 by 50 mm) grout cubes, which were tested under compression in accordance

3.2.5 Surface Roughness Quantification

A surface roughness quantification method was established to eliminate the
ambiguity associated with classifying surface roughness only by the tool used to cause
the texture. The roughness of two surfaces prepared using the same type of tool may vary considerably depending on the condition of the tool, the pressure applied by the operator and the properties of the concrete mix. Even machine finished surfaces may vary in roughness based on the casting machine used and the concrete mix. Due to these ambiguities, concrete surfaces classified as “broom roughened” may encompass a wide range of textures. To cope with possible roughness variations, a method was developed to quantify the surface roughness of each hollow core slab specimen.

The surface roughness quantification procedure, referred to as the sand patch test, was adapted from ASTM E965, “Standard Test for Measuring Pavement Macrotexture Depth Using a Volumetric Technique” (ASTM E965, 2006). The ASTM standard is intended to be used on pavement surfaces, but was adjusted to be more suitable for use on precast concrete surfaces.

The test involves spreading a known volume of well graded sand onto the surface in a circular patch using a rubber spreading disc. Using equation 3.1, the diameter of the sand patch can be related to the macrotexture depth (MTD) of the surface, which is “the average depth between the bottom of the surface voids and the top of the surface aggregate particles” (ASTM E965, 2006). The test was performed four times for each specimen surface. The mean macrotexture depth (MMTD) was found by averaging the results from the four tests.

\[
 MTD = \frac{4V}{\pi D^2}
\]

where:
- \( MTD \) = Macrotexture depth
- \( V \) = Volume of sand used in the measurement
- \( D \) = Average measured diameter of the sand patch

\[ 3.1 \]
According to the ASTM measurement procedure, sand passing a No. 50 sieve and retained on a No. 100 sieve should be used for this test. Since this measurement standard has been made for use with pavement surfaces, which generally have larger surface voids than concrete surfaces, it was necessary to alter the standard before it could be used on precast concrete surfaces. To make this procedure more suitable, sand passing a No. 100 sieve and retained on a No. 200 sieve was used. Additionally, it was found that a volume of 1 in$^3$ (16 cm$^3$) of sand was more suitable for use on precast concrete surfaces, rather than 2 in$^3$ (33 cm$^3$) recommended in the ASTM standard for pavement. A hockey puck was used as the rubber spreading disc; the puck had a diameter of 3.0 in. (76 mm) and weighed 0.36 lb. (160 g).

When performing the test, an area on the precast concrete surface approximately 2 ft. (0.6 m) in diameter must be cleaned using a brush or compressed air to remove any loose particles and excessive laitance dust. The measured quantity of graded sand is then poured near the center of the cleaned area. This sand pile is then spread by slowly moving the rubber disc in a circular pattern on top of the sand. The spreading disc is only moved using a circular motion and additional downward pressure is not applied while spreading the sand. Spreading the sand is continued until the peaks of the concrete surface roughness become visible through the sand patch, giving the patch a blotchy appearance and the sand patch diameter no longer increases. The sand patch must approximately have an even thickness. To find the average diameter of the sand patch, four diameter measurements are taken across the patch at different locations, as illustrated in Figure 3.14. After recording these diameter measurements, the sand is removed from the concrete surface by using a brush and compressed air.
The selected surface roughness quantification procedure does not require expensive equipment to perform, and can be completed in less than 15 minutes. It is a procedure that can be easily implemented in practice either in the precasting plant or at the construction site for quality control purposes. However, there are drawbacks to this simple procedure. The test is not capable of differentiating between types of roughness or the trending direction of the roughness, so it is necessary for the operator to observe and record these details separately. Also, since the test is susceptible to operator influence, it is necessary to perform the measurements carefully and with consistency. It is not possible to use the sand patch test on concrete with very irregular surfaces where the rubber spreading tool is unable to sit flat.

In this testing program, the surface roughness of the rake roughened specimens could not be measured using the sand patch test due to large surface irregularities. For these specimens, the mean texture depth was estimated by taking measurements of surface features using Vernier calipers. The depth probe of the Vernier calipers was used to measure the depth of the rake grooves and the height of the pieces of concrete protruding from the surface. The width of the rake grooves was measured using the inside jaws of the Vernier calipers. After taking these measurements, an average depth of the rake roughened surface was calculated. The calculation assumed that the surface area

\[
D = \frac{d_1 + d_2 + d_3 + d_4}{4}
\]
between rake undulations had machine finished MTD properties. The calculation also assumed that the area of concrete material protruding from the concrete surface beside each rake groove was approximately equal to two thirds of the rake groove area. This approximation was made because the width of the raised ridges of protruding concrete was very inconsistent.

3.3 Testing Results

Each push-off specimen was identified using the label presented in Table 3.3. Also shown in this table are the dates of bottom and top block construction and push-off testing.

3.3.1 Results of Concrete Strength Quantification

A total of twelve bottom block compression cylinders from both hollow core slab manufacturers were tested throughout the experimental program. The 24 compression cylinders were tested in twelve sets of two. Using the results of these cylinder tests, the average strength-time curve was obtained to estimate the bottom block compressive strength at the time of testing for both hollow core slab manufacturers. Separate curves were determined for the dry-mix and wet-mix concretes. The compressive strength-time data along with equations for the best-fit curves are shown in Figure 3.15 and Figure 3.16, for the dry cast and wet cast hollow core samples, respectively.
Table 3.3. Specimen labels, construction dates and testing dates.

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Surface*</th>
<th>Bottom block cast date</th>
<th>Top block cast date</th>
<th>Push-off test date</th>
<th>Age of concrete at time of testing bottom / top block</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRY-MFX-1</td>
<td>MF</td>
<td>3/16/10</td>
<td>6/14/10</td>
<td>6/23/10</td>
<td>99 days / 9 days</td>
</tr>
<tr>
<td>DRY-MFX-2</td>
<td>MF</td>
<td>3/16/10</td>
<td>6/29/10</td>
<td>7/6/10</td>
<td>112 days / 7 days</td>
</tr>
<tr>
<td>DRY-SBX-1</td>
<td>SB</td>
<td>3/16/10</td>
<td>7/7/10</td>
<td>7/13/10</td>
<td>119 days / 6 days</td>
</tr>
<tr>
<td>DRY-SBX-2</td>
<td>SB</td>
<td>3/16/10</td>
<td>7/29/10</td>
<td>8/4/10</td>
<td>141 days / 6 days</td>
</tr>
<tr>
<td>DRY-LRX-1</td>
<td>LR</td>
<td>3/16/10</td>
<td>7/22/10</td>
<td>7/28/10</td>
<td>134 days / 6 days</td>
</tr>
<tr>
<td>DRY-LRX-2</td>
<td>LR</td>
<td>3/16/10</td>
<td>8/12/10</td>
<td>8/18/10</td>
<td>155 days / 6 days</td>
</tr>
<tr>
<td>DRY-TBX-1</td>
<td>TB</td>
<td>3/16/10</td>
<td>7/14/10</td>
<td>7/20/10</td>
<td>126 days / 6 days</td>
</tr>
<tr>
<td>DRY-TBX-2</td>
<td>TB</td>
<td>3/16/10</td>
<td>8/5/10</td>
<td>8/11/10</td>
<td>148 days / 6 days</td>
</tr>
<tr>
<td>DRY-MFG-1</td>
<td>MFG</td>
<td>3/16/10</td>
<td>8/20/10</td>
<td>8/25/10</td>
<td>162 days / 5 days</td>
</tr>
<tr>
<td>DRY-MFG-2</td>
<td>MFG</td>
<td>3/16/10</td>
<td>9/3/10</td>
<td>9/8/10</td>
<td>176 days / 5 days</td>
</tr>
<tr>
<td>DRY-LRG-1</td>
<td>LRG</td>
<td>3/16/10</td>
<td>8/27/10</td>
<td>9/1/10</td>
<td>169 days / 5 days</td>
</tr>
<tr>
<td>DRY-LRG-2</td>
<td>LRG</td>
<td>3/16/10</td>
<td>9/10/10</td>
<td>9/16/10</td>
<td>184 days / 6 days</td>
</tr>
<tr>
<td>WET-MFX-1</td>
<td>MF</td>
<td>4/13/10</td>
<td>9/20/10</td>
<td>9/25/10</td>
<td>165 days / 5 days</td>
</tr>
<tr>
<td>WET-MFX-2</td>
<td>MF</td>
<td>4/13/10</td>
<td>10/7/10</td>
<td>10/13/10</td>
<td>183 days / 6 days</td>
</tr>
<tr>
<td>WET-SBX-1</td>
<td>SB</td>
<td>4/13/10</td>
<td>10/15/10</td>
<td>10/20/10</td>
<td>190 days / 5 days</td>
</tr>
<tr>
<td>WET-SBX-2</td>
<td>SB</td>
<td>4/13/10</td>
<td>10/23/10</td>
<td>10/28/10</td>
<td>198 days / 5 days</td>
</tr>
<tr>
<td>WET-LBX-1</td>
<td>LB</td>
<td>4/13/10</td>
<td>9/30/10</td>
<td>10/5/10</td>
<td>175 days / 5 days</td>
</tr>
<tr>
<td>WET-LBX-2</td>
<td>LB</td>
<td>4/22/10</td>
<td>11/9/10</td>
<td>11/15/10</td>
<td>207 days / 6 days</td>
</tr>
<tr>
<td>WET-TBX-1</td>
<td>TB</td>
<td>4/13/10</td>
<td>11/17/10</td>
<td>11/23/10</td>
<td>224 days / 6 days</td>
</tr>
<tr>
<td>WET-TBX-2</td>
<td>TB</td>
<td>4/13/10</td>
<td>11/24/10</td>
<td>12/1/10</td>
<td>232 days / 7 days</td>
</tr>
<tr>
<td>WET-MFG-1</td>
<td>MFG</td>
<td>4/13/10</td>
<td>12/10/10</td>
<td>12/15/10</td>
<td>246 days / 5 days</td>
</tr>
<tr>
<td>WET-MFG-2</td>
<td>MFG</td>
<td>4/13/10</td>
<td>12/19/10</td>
<td>12/26/10</td>
<td>257 days / 7 days</td>
</tr>
<tr>
<td>WET-LBG-1</td>
<td>LBG</td>
<td>4/13/10</td>
<td>1/1/11</td>
<td>1/7/11</td>
<td>269 days / 6 days</td>
</tr>
<tr>
<td>WET-LBG-2</td>
<td>LBG</td>
<td>4/13/10</td>
<td>1/14/11</td>
<td>1/20/11</td>
<td>282 days / 6 days</td>
</tr>
</tbody>
</table>

Figure 3.15. Model used to estimate the compressive strength of dry-mix hollow core slab bottom blocks at the time of push-off experimentation.

Figure 3.16. Model used to estimate the compressive strength of wet-mix hollow core slab bottom blocks at the time of push-off experimentation.
The strength of each top block was determined by testing three companion 8 by 4 in. (200 by 100 mm) cylinders in compression and two companion tension (split) 12 by 6 in. (300 by 150 mm) cylinders on the same day of the push-off experiment. The average results of these cylinder tests can be seen in Table 3.4 for all specimens. Specimen DRY-MFX-1 did not have accompanying tension (split) cylinder tests. The presented tensile strength of this specimen has been inferred by normalizing the tensile strength of all other top block mixes by the square root of their compressive strength, then multiplying the average of these normalized strengths by the compressive strength of specimen DRY-MFX-1. On average, the top block tensile strength was equal to 6.2 multiplied by the square root of the top block compressive strength.

In four specimens from each manufacturer, a thin layer of grout was applied prior to placement of the cast-in-place top block as described in Section 3.2.1. Grout cubes were tested in compression to determine the strength of the grout layer. The results of the grout cube compression tests are shown in Table 3.5.
Table 3.4. Compressive and tensile strength of concrete used during push-off experiments.

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Top Block Compressive Strength, psi (MPa)</th>
<th>Top Block Tensile Strength, psi (MPa)</th>
<th>Bottom block Compressive Strength, psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRY-MFX-1</td>
<td>4670 (32.2)</td>
<td>420 (2.9)</td>
<td>6920 (47.7)</td>
</tr>
<tr>
<td>DRY-MFX-2</td>
<td>3780 (26.1)</td>
<td>360 (2.5)</td>
<td>7170 (49.4)</td>
</tr>
<tr>
<td>DRY-SBX-1</td>
<td>4510 (31.1)</td>
<td>410 (2.8)</td>
<td>7310 (50.4)</td>
</tr>
<tr>
<td>DRY-SBX-2</td>
<td>5000 (34.5)</td>
<td>440 (3.0)</td>
<td>7740 (53.4)</td>
</tr>
<tr>
<td>DRY-LRX-1</td>
<td>4630 (31.9)</td>
<td>420 (2.9)</td>
<td>7600 (52.4)</td>
</tr>
<tr>
<td>DRY-LRX-2</td>
<td>5020 (34.6)</td>
<td>450 (3.1)</td>
<td>8010 (55.3)</td>
</tr>
<tr>
<td>DRY-TBX-1</td>
<td>4750 (32.7)</td>
<td>450 (3.1)</td>
<td>7450 (51.3)</td>
</tr>
<tr>
<td>DRY-TBX-2</td>
<td>5140 (35.4)</td>
<td>480 (3.3)</td>
<td>7880 (54.3)</td>
</tr>
<tr>
<td>DRY-MFG-1</td>
<td>4670 (32.2)</td>
<td>410 (2.8)</td>
<td>8150 (56.2)</td>
</tr>
<tr>
<td>DRY-MFG-2</td>
<td>5050 (34.8)</td>
<td>440 (3.0)</td>
<td>8430 (58.1)</td>
</tr>
<tr>
<td>DRY-LRG-1</td>
<td>4820 (33.2)</td>
<td>450 (3.1)</td>
<td>8290 (57.2)</td>
</tr>
<tr>
<td>DRY-LRG-2</td>
<td>5110 (35.3)</td>
<td>440 (3.1)</td>
<td>8580 (59.2)</td>
</tr>
<tr>
<td>WET-MFX-1</td>
<td>5180 (35.7)</td>
<td>440 (3.0)</td>
<td>9700 (66.9)</td>
</tr>
<tr>
<td>WET-MFX-2</td>
<td>5180 (35.7)</td>
<td>450 (3.1)</td>
<td>9730 (67.1)</td>
</tr>
<tr>
<td>WET-SBX-1</td>
<td>4960 (34.2)</td>
<td>430 (3.0)</td>
<td>9750 (67.2)</td>
</tr>
<tr>
<td>WET-SBX-2</td>
<td>4530 (31.2)</td>
<td>440 (3.0)</td>
<td>9760 (67.3)</td>
</tr>
<tr>
<td>WET-LBX-1</td>
<td>5190 (35.8)</td>
<td>450 (3.1)</td>
<td>9720 (67.0)</td>
</tr>
<tr>
<td>WET-LBX-2</td>
<td>4810 (33.2)</td>
<td>410 (2.8)</td>
<td>9780 (67.4)</td>
</tr>
<tr>
<td>WET-TBX-1</td>
<td>4930 (34.0)</td>
<td>410 (2.8)</td>
<td>9810 (67.7)</td>
</tr>
<tr>
<td>WET-TBX-2</td>
<td>4140 (28.5)</td>
<td>470 (3.2)</td>
<td>9830 (67.8)</td>
</tr>
<tr>
<td>WET-MFG-1</td>
<td>4310 (29.7)</td>
<td>420 (2.9)</td>
<td>9850 (67.9)</td>
</tr>
<tr>
<td>WET-MFG-2</td>
<td>5420 (37.3)</td>
<td>420 (2.9)</td>
<td>9880 (68.1)</td>
</tr>
<tr>
<td>WET-LBG-1</td>
<td>4510 (31.1)</td>
<td>380 (2.6)</td>
<td>9900 (68.2)</td>
</tr>
<tr>
<td>WET-LBG-2</td>
<td>4100 (28.3)</td>
<td>370 (2.5)</td>
<td>9920 (68.4)</td>
</tr>
</tbody>
</table>

*Strength values obtained from cylinder tests
‡Strength values obtained using strength-time curves introduced in Figure 3.15 and Figure 3.16
†Strength value inferred, as explained in Section 3.3.1
Table 3.5. Compressive strength of grout used during push-off tests (ASTM C109).

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Grout Compressive Strength, psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRY-MFG-1</td>
<td>7790 (53.7)</td>
</tr>
<tr>
<td>DRY-MFG-2</td>
<td>6320 (43.5)</td>
</tr>
<tr>
<td>DRY-LRG-1</td>
<td>5660 (39.0)</td>
</tr>
<tr>
<td>DRY-LRG-2</td>
<td>6700 (46.2)</td>
</tr>
<tr>
<td>WET-MFG-1</td>
<td>5990 (41.3)</td>
</tr>
<tr>
<td>WET-MFG-2</td>
<td>5250 (36.2)</td>
</tr>
<tr>
<td>WET-LBG-1</td>
<td>5540 (38.2)</td>
</tr>
<tr>
<td>WET-LBG-2</td>
<td>4610 (31.8)</td>
</tr>
</tbody>
</table>

3.3.2 Results of Surface Roughness Quantification

The roughness of each surface was quantified using a measurement procedure adapted from an existing ASTM standard for pavements, as explained in Section 3.2.5. The measured mean macrotexture depth (MMTD) for each specimen is presented in Table 3.6. For grouted specimens, MMTD measurements were taken prior to grout application. Additional MMTD measurements were taken on representative grouted surfaces after grout application and drying (Table 3.7). The MMTD of dry and wet-mix specimens are also shown in Figure 3.17 and Figure 3.18, respectively.

Table 3.6. Mean macrotexture depth (MMTD) of all push-off specimens

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>MMTD, in. (mm)</th>
<th>Specimen Label</th>
<th>MMTD, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRY-MFX-1</td>
<td>0.0099 (0.2515)</td>
<td>WET-MFX-1</td>
<td>0.0146 (0.3708)</td>
</tr>
<tr>
<td>DRY-MFX-2</td>
<td>0.0094 (0.2388)</td>
<td>WET-MFX-2</td>
<td>0.0149 (0.3785)</td>
</tr>
<tr>
<td>DRY-SBX-1</td>
<td>0.0113 (0.2870)</td>
<td>WET-SBX-1</td>
<td>0.0173 (0.4394)</td>
</tr>
<tr>
<td>DRY-SBX-2</td>
<td>0.0122 (0.3099)</td>
<td>WET-SBX-2</td>
<td>0.0173 (0.4394)</td>
</tr>
<tr>
<td>DRY-LRX-1</td>
<td>0.0221 (0.5613)</td>
<td>WET-LBX-1</td>
<td>0.0423 (1.0744)</td>
</tr>
<tr>
<td>DRY-LRX-2</td>
<td>0.0233 (0.5918)</td>
<td>WET-LBX-2</td>
<td>0.0366 (0.9296)</td>
</tr>
<tr>
<td>DRY-TBX-1</td>
<td>0.0294 (0.7468)</td>
<td>WET-TBX-1</td>
<td>0.0474 (1.2040)</td>
</tr>
<tr>
<td>DRY-TBX-2</td>
<td>0.0326 (0.8280)</td>
<td>WET-TBX-2</td>
<td>0.0400 (1.0160)</td>
</tr>
<tr>
<td>DRY-MFG-1</td>
<td>0.0208 (0.5283)</td>
<td>WET-MFG-1</td>
<td>0.0208 (0.5283)</td>
</tr>
<tr>
<td>DRY-MFG-2</td>
<td>0.0208 (0.5283)</td>
<td>WET-MFG-2</td>
<td>0.0208 (0.5283)</td>
</tr>
<tr>
<td>DRY-LRG-1</td>
<td>0.0270 (0.6858)</td>
<td>WET-LBG-1</td>
<td>0.0210 (0.5334)</td>
</tr>
<tr>
<td>DRY-LRG-2</td>
<td>0.0270 (0.6858)</td>
<td>WET-LBG-2</td>
<td>0.0210 (0.5334)</td>
</tr>
</tbody>
</table>

Note: MMTD of grouted specimens shown in this table represent the texture characteristics of surfaces after grout application and drying.
Table 3.7. Mean macrotexture depth (MMTD) before and after application of grout.

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>MMTD before grout, in. (mm)</th>
<th>MMTD after grout, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRY-MFG-1</td>
<td>0.0086 (0.2184)</td>
<td>0.0208 (0.5283)</td>
</tr>
<tr>
<td>DRY-MFG-2</td>
<td>0.0087 (0.2210)</td>
<td>0.0208 (0.5283)</td>
</tr>
<tr>
<td>DRY-LRG-1</td>
<td>0.0215 (0.5461)</td>
<td>0.0270 (0.6858)</td>
</tr>
<tr>
<td>DRY-LRG-2</td>
<td>0.0193 (0.4902)</td>
<td>0.0270 (0.6858)</td>
</tr>
<tr>
<td>WET-MFG-1</td>
<td>0.0130 (0.3302)</td>
<td>0.0208 (0.5283)</td>
</tr>
<tr>
<td>WET-MFG-2</td>
<td>0.0130 (0.3302)</td>
<td>0.0208 (0.5283)</td>
</tr>
<tr>
<td>WET-LBG-1</td>
<td>0.0380 (0.9652)</td>
<td>0.0210 (0.5334)</td>
</tr>
<tr>
<td>WET-LBG-2</td>
<td>0.0359 (0.9119)</td>
<td>0.0210 (0.5334)</td>
</tr>
</tbody>
</table>

Figure 3.17. Mean macrotexture depth (MMTD) of dry-mix hollow core slab surfaces.

Figure 3.18. Mean macrotexture depth (MMTD) of wet-mix hollow core slab surfaces.
From these results it can be seen that the grouting process increased the MMTD of machine finished surfaces. The grout layer only increased the roughness of the rake roughened surfaces by a small amount. This was because the grout partially filled the rake grooves resulting in a wavy surface. For broom roughened surfaces, the grout completely filled the roughness striations caused by brooming. This resulted in a greatly decreased roughness for grouted broom roughened surfaces.

3.3.3 Results of Push-Off Testing

The force-displacement plots in this section present the average displacement between the two transducers positioned in each row as illustrated in Figure 3.11. In experiments where a transducer malfunctioned, the corresponding force-displacement plot shows only the data of the properly functioning transducer in the row. Locations of these transducers are discussed in Section 3.2.3.

3.3.3.1 Overview of Push-Off Testing Results

The test results are summarized in Table 3.8, which lists the following measured values for each specimen: maximum shear force, average interfacial shear stress at maximum force, horizontal slip at failure, and vertical displacement of top block at failure. The interface strength was found by dividing the maximum force by the total interface area of the specimen (15 by 15 in. [381 mm by 381 mm]). In this table, horizontal slip at failure has been calculated by averaging the maximum slip readings of the S1 and N1 transducers. Vertical displacement of the top block was determined by averaging the maximum readings of the S4 and N4 transducers.

All push-off specimens failed in shear along the composite interface between the hollow core slab and the cast-in-place topping block. In most specimens, the first row of
transducers recorded the largest displacements prior to failure. Typically, each row of transducers recorded similar magnitude displacements. The force-displacement plots shown in the Sections 3.3.3.2 through 3.3.3.13 represent the average displacement measured by each row of gages unless otherwise noted. Each dotted line grid unit on the horizontal axis of the force-displacement plots represents 0.001 in. (0.254 mm) of horizontal slip.

Table 3.8. Push-off testing results summary.

<table>
<thead>
<tr>
<th>Specimen Label</th>
<th>Max shear force, kip (kN)</th>
<th>Max shear stress, psi (Mpa)</th>
<th>Horizontal slip at failure, in. (mm)</th>
<th>Vertical disp. at failure, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DRY-MFX-1</td>
<td>46.5 (207)</td>
<td>207 (1.42)</td>
<td>0.0006 (0.016)</td>
<td>not recorded</td>
</tr>
<tr>
<td>DRY-MFX-2</td>
<td>34.2 (152)</td>
<td>152 (1.05)</td>
<td>0.0001 (0.002)</td>
<td>not recorded</td>
</tr>
<tr>
<td>DRY-SBX-1</td>
<td>36.4 (162)</td>
<td>162 (1.12)</td>
<td>0.0010 (0.025)</td>
<td>0.0003 (0.009)</td>
</tr>
<tr>
<td>DRY-SBX-2</td>
<td>48.4 (215)</td>
<td>215 (1.48)</td>
<td>0.0017 (0.043)</td>
<td></td>
</tr>
<tr>
<td>DRY-LRX-1</td>
<td>50.2 (223)</td>
<td>223 (1.54)</td>
<td>0.0021 (0.053)</td>
<td>not recorded</td>
</tr>
<tr>
<td>DRY-LRX-2</td>
<td>46.1 (205)</td>
<td>205 (1.41)</td>
<td>0.0013 (0.033)</td>
<td>0.0012 (0.031)</td>
</tr>
<tr>
<td>DRY-TBX-1</td>
<td>64.7 (288)</td>
<td>288 (1.98)</td>
<td>0.0018 (0.046)</td>
<td>not recorded</td>
</tr>
<tr>
<td>DRY-TBX-2</td>
<td>71.8 (319)</td>
<td>319 (2.20)</td>
<td>0.0034 (0.086)</td>
<td>0.0018 (0.046)</td>
</tr>
<tr>
<td>DRY-MFG-1</td>
<td>62.0 (276)</td>
<td>276 (1.90)</td>
<td>0.0016 (0.041)</td>
<td>0.0011 (0.028)</td>
</tr>
<tr>
<td>DRY-MFG-2</td>
<td>84.8 (377)</td>
<td>377 (2.60)</td>
<td>0.0029 (0.074)</td>
<td>0.0017 (0.043)</td>
</tr>
<tr>
<td>DRY-LRG-1</td>
<td>62.2 (277)</td>
<td>276 (1.91)</td>
<td>0.0029 (0.074)</td>
<td>0.0013 (0.033)</td>
</tr>
<tr>
<td>DRY-LRG-2</td>
<td>59.8 (266)</td>
<td>266 (1.83)</td>
<td>0.0026 (0.066)</td>
<td>0.0012 (0.030)</td>
</tr>
<tr>
<td>WET-MFX-1</td>
<td>44.6 (198)</td>
<td>198 (1.37)</td>
<td>0.0014 (0.036)</td>
<td>0.0002 (0.005)</td>
</tr>
<tr>
<td>WET-MFX-2</td>
<td>28.7 (128)</td>
<td>128 (0.88)</td>
<td>0.0009 (0.023)</td>
<td>0.0002 (0.006)</td>
</tr>
<tr>
<td>WET-SBX-1</td>
<td>60.2 (268)</td>
<td>268 (1.85)</td>
<td>0.0019 (0.048)</td>
<td>0.0006 (0.016)</td>
</tr>
<tr>
<td>WET-SBX-2</td>
<td>50.6 (225)</td>
<td>225 (1.55)</td>
<td>0.0024 (0.061)</td>
<td>0.0007 (0.017)</td>
</tr>
<tr>
<td>WET-LBX-1</td>
<td>49.9 (222)</td>
<td>222 (1.53)</td>
<td>0.0019 (0.048)</td>
<td>0.0006 (0.015)</td>
</tr>
<tr>
<td>WET-LBX-2</td>
<td>32.4 (144)</td>
<td>144 (0.99)</td>
<td>0.0011 (0.028)</td>
<td>0.0002 (0.005)</td>
</tr>
<tr>
<td>WET-TBX-1</td>
<td>57.9 (257)</td>
<td>257 (1.77)</td>
<td>0.0025 (0.064)</td>
<td>0.0010 (0.025)</td>
</tr>
<tr>
<td>WET-TBX-2</td>
<td>55.7 (248)</td>
<td>248 (1.71)</td>
<td>0.0022 (0.056)</td>
<td>0.0005 (0.012)</td>
</tr>
<tr>
<td>WET-MFG-1</td>
<td>35.4 (157)</td>
<td>157 (1.08)</td>
<td>0.0010 (0.025)</td>
<td>0.0001 (0.001)</td>
</tr>
<tr>
<td>WET-MFG-2</td>
<td>37.2 (165)</td>
<td>165 (1.14)</td>
<td>0.0015 (0.038)</td>
<td>0.0004 (0.011)</td>
</tr>
<tr>
<td>WET-LBG-1</td>
<td>55.6 (247)</td>
<td>247 (1.70)</td>
<td>0.0021 (0.053)</td>
<td>0.0009 (0.024)</td>
</tr>
<tr>
<td>WET-LBG-2</td>
<td>49.1 (218)</td>
<td>218 (1.50)</td>
<td>0.0021 (0.053)</td>
<td>0.0006 (0.016)</td>
</tr>
</tbody>
</table>

3.3.3.2 Dry-Mix Specimens with Machine Finished Surface

The dry-mix specimens with machine finished surfaces exhibited lower average interface strength than the specimens with intentional roughening. This lower strength is
likely attributed to smoother surfaces on the machine finished specimens. Specimens DRY-MFX-1 and DRY-MFX-2 had MMTD measurements of 0.0099 and 0.0094 in. (0.25 and 0.24 mm), respectively.

It was also found that the machine finished specimens had a lower horizontal slip capacity compared with intentionally roughened specimens. The force-displacement plots are presented in Figure 3.19 and Figure 3.20 for specimens DRY-MFX-1 and DRY-MFX-2, respectively. Vertical displacement of the interface was not recorded for these two specimens.

![Figure 3.19. Force-displacement plots for specimen DRY-MFX-1.](image)

Specimen DRY-MFX-1 exhibited very little slip at forces below 30 kips (130 kN). At this force, the shear stress applied to the interface was 130 psi (0.919 MPa). After failure, pieces of exposed aggregate were observed on the top block in a single localized area, as shown in Figure 3.21.
Specimen DRY-MFX-2 had the lowest interface strength and displacement of all dry-mix specimens tested. At a maximum force of 34.2 kips (152 kN), a small yet measurable horizontal slip of $9 \times 10^{-5}$ in. (0.002 mm) was observed. No pieces of exposed aggregate were visible on the interface of this specimen after failure occurred.
3.3.3.3 Dry-Mix Specimens with Sandblasted Surface

The roughness added through sandblasting increased the MMTD by 22% compared with the dry-mix machined finished specimens. A photograph of a dry-mix sandblasted surface can be seen in Figure 3.22. Despite this noticeable increase in roughness, the average interface strength of the dry-mix sandblasted specimens was only 5% higher than that of the dry-mix machine finished specimens.

![Figure 3.22. Photograph of dry-mix sandblasted surface.](image)

The force-displacement plots for specimens DRY-SBX-1 and DRY-SBX-2 can be seen in Figure 3.23 and Figure 3.24, respectively. Of these two specimens, vertical displacements were recorded for DRY-SBX-2 only.
The force-displacement plots for specimen DRY-SBX-1 suggest that the region of the composite interface between the two first row displacement transducers was most
active in resisting the applied shear force. This region of the interface is closest to the shear force application point on the top block. The middle region of the composite interface (between gages N2 and S2) did not exhibit any slip prior to failure.

As shown in Figure 3.25, transducers N1 and S1 in specimen DRY-SBX-2 had very different behavior during testing. At the location of these transducers, the interface displaced significantly more on the south side than on the north side. By observing the failure surface, it became apparent that the composite interface had a stronger bond beside the N1 transducer, where a region of top block concrete remained bonded to the bottom block after failure. This observation shows that regions with higher bond will exhibit smaller horizontal relative displacements (slip). A photograph of the top block interface after failure is shown in Figure 3.26, where the darker region near transducer N1 corresponds to the concrete that remained attached to the bottom block.

![Figure 3.25. Force-displacement plots of first row gages of specimen DRY-SBX-2.](image)
3.3.3.4 Dry-Mix Specimens with Longitudinally Rake Roughened Surface

The dry-mix rake roughened specimens had deep, trench like grooves that were spaced at 5 in. (125 mm) on center. The depth of the rake grooves was approximately 0.25 in. (64 mm). On these specimens, the raking procedure was performed automatically by the hollow core slab fabrication machine as it passed over the recently cast concrete. Excess concrete material that was removed during raking formed raised ridges of hardened concrete on both sides of the rake grooves. The area between the rake grooves had a machine finished surface. Prior to the placement of the topping concrete, the rake roughened specimens required considerably more surface cleaning than other specimens. The rake grooves were filled with loose concrete debris, as shown in Figure 3.27. This material was removed with a broom and compressed air, but additional attention was not given to pieces of debris that were loose from the surface yet wedged within the rake grooves. Many pieces of concrete within the raised ridges were weakly attached to the concrete surface and could be removed by hand if pushed; additional effort was not made to remove these weakly attached pieces if they were not removed
with the broom and compressed air. A photograph of the rake grooves after cleaning is shown in Figure 3.28.

![Figure 3.27. Photograph of rake grooves filled with concrete debris prior to cleaning.](image)

![Figure 3.28. Photograph of a rake groove after cleaning.](image)

After failure, pieces of the raised ridges of raked concrete that were previously attached to the bottom block had been sheared off (Figure 3.29). Additionally, some pieces of the top block concrete that protruded into the rake grooves on the bottom block were found to be crushed after failure due to the applied shear force.
Figure 3.29. Photograph a rake groove on the bottom block after failure. Arrows indicate the raised ridges of concrete that were formally attached to the bottom block.

Force-displacement plots for specimens DRY-LRX-1 and DRY-LRX-2 can be found in Figure 3.30 and Figure 3.31, respectively. Of these two specimens, vertical displacements were recorded for DRY-LRX-2 only.

Figure 3.30. Force-displacement plots for specimen DRY-LRX-1.
Specimen DRY-LRX-1 failed at a force of 50.2 kips (223 kN). At this force, the average interface shear strength was determined to be 223 psi (1.54 MPa).

Specimen DRY-LRX-2 failed at a force of 46.1 kips (205 kN). The average interface shear strength was 205 psi (1.41 MPa). The displacement data for this specimen are unusual because the second row of horizontal slip transducers recorded higher slip than the first row of gages throughout nearly the entire experiment. It is possible that the bond near the center of the interface (between gages N2 and S2) broke at a lower shear stress due to a surface irregularity caused by raking. This behavior was not observed in any of the other push-off tests.

3.3.3.5 **Dry-Mix Specimens with Transversely Broomed Surface**

The surface of the dry-mix broom roughened specimens is characterized by many thin and shallow parallel striations. Unlike the dry-mix rake roughened specimens, the
broom roughening process was conducted by hand. This resulted in roughness striations of varying width, height and depth. In some regions, the surface appeared to be bumpy and without a clear pattern (Figure 3.32).

![Roughness striations caused by broom]

**Figure 3.32.** Broom roughened surfaces had many thin shallow striations, but also had regions where striations were not evident.

The dry-mix specimens that were broom roughened in the direction perpendicular to the span had the highest average interfacial shear strength and horizontal slip capacity of all non-grouted test specimens (including wet-mix specimens). The average interface strength of these specimens was 304 psi (2.09 MPa).

The high strength and displacement capacities of the dry-mix broom roughened specimens may be attributed to the higher roughness of the surface, as determined from the MMTD method. Additionally, the direction of the ridges formed by the broom was oriented perpendicularly to the applied shear force. This allowed for a more uniform interlocking to occur between the top and bottom blocks of concrete through the roughness undulations caused by brooming. Similar uniform interlocking behavior was not notorious in the raked specimens because the direction of the rake grooves was parallel to the applied force and interlocking was higher in localized regions.

Problems during the testing of specimen DRY-TBX-1 resulted in the specimen being loaded to a force of 50 kips (220 kN), unloading to a force of 0 kips and re-loading.
to failure. Although this loading pattern was not intentional, the data collected during the repeated loading provides some insight on the behavior of such interfaces. Force-displacement plots of the first and second loading stages are compared in Figure 3.33. It can be seen in this figure that the force-displacement behavior is nearly identical between the two loading attempts. This is consistent with findings by Hanson (1960), who performed measurements of residual slip in loaded and unloaded push-off specimens (see Section 2.5.1). The force-displacement plots of the second loading attempt, which continued until failure, are shown in Figure 3.34.
Figure 3.34. Force-displacement plots of second loading attempt for specimen DRY-TBX-1.

Of the two dry-mix broom roughened specimens, vertical displacements were recorded for DRY-TBX-2 only.

A sudden increase of horizontal slip was recorded by the second row of gages at a force of approximately 30 kips (130 kN). This was recorded on gage N2 only, and was not apparent on gage S2. It is possible that this sudden increase was caused by a region of interfacial bond breaking in a brittle manner.

Force-displacement plots for specimen DRY-TBX-2 can be found in Figure 3.35.
The top block failure surfaces of specimens DRY-TBX-1 and DRY-TBX-2 had regions of darker concrete where broom striations had been crushed and some aggregate particles were exposed. The photograph of specimen DRY-TBX-1 (Figure 3.36) shows these areas with crushed broom striations. A noticeable region of exposed aggregate near gage S1 is visible in this photograph. Despite the asymmetrical appearance of this failure surface, the horizontal slip measurements taken along the north and south sides of both dry-mix transversely broomed specimens were close in magnitude.
3.3.3.6 Dry-Mix Specimens with Grouted Machine Finished Surface

The dry-mix grouted machine finished specimens had the highest average interfacial shear strength of all push-off specimens. As mentioned in Section 3.2.1, the grout was applied in a thin (1/16 in., 1.59 mm) layer using a broom. The broom was used in a motion perpendicular to the span; however the texture of the broom was only faintly apparent after the grout layer dried (Figure 3.37). It was typical for several small clumps of grout to remain scattered on the interface surface after grout application. As shown in Section 3.3.2, the grouting process increased the MMTD of the machine finished surface by 140%.
The force-displacement plots for specimen DRY-MFG-1 can be found in Figure 3.38. The horizontal slip measurements varied significantly between the two sides of this specimen; this is best shown in the force-displacement plots of select transducers presented in Figure 3.39. It appears that displacements were larger on the south side of the specimen.
Figure 3.38. Force-displacement plots for specimen DRY-MFG-1.

Figure 3.39. Force-displacement plots for gages N3, S3, N1 and S1 for specimen DRY-MFG-1.
A photograph of the failure surface of specimen DRY-MFG-1 is shown in Figure 3.40. For this specimen, the layer of grout that was added to the bottom block surface stayed bonded to the bottom block after failure except for an irregularly shaped region between transducers in row 3, which instead remained bonded to the top block after failure. A thin, bumpy layer of top block concrete remained bonded to the layer of grout attached to the bottom block around the edges of the interface, but not in the center of the interface area. By analyzing the photographs taken of the bottom block failure surface, it was determined that approximately 81% of the grout layer remained attached to the bottom block after failure. The remaining 19% of the grout either remained attached to the top block or broke off of both parts of the specimen during experimentation. The failure surface showed an irregularly shaped pattern of grout debonding, which may explain the discrepancy in horizontal slip measurements between symmetric gages.

**Figure 3.40.** Analysis of bottom block failure surface of specimen DRY-MFG-1.  
(a) Photograph of specimen DRY-MFG-1 bottom block failure surface.  
(b) Region of failure surface where grout remained bonded to the bottom block is colored light gray.
Specimen DRY-MFG-2 had the highest failure force of all specimens, at 84.8 kips (377 kN). At this force, the interface shear stress was 377 psi (2.60 MPa). The force-displacement plots for specimen DRY-MFG-2 are shown in Figure 3.41.

![Figure 3.41. Force-displacement plots for specimen DRY-MFG-2.](image)

A photograph of the failure surface of specimen DRY-MFG-2 is shown in Figure 3.42. Through analysis of the photo of the bottom block failure surface, it was determined that approximately 71% of the grout layer remained attached to the bottom block after failure. The remaining 29% of the grout either remained attached to the top block or broke off of both parts of the specimen during experimentation. The horizontal slip measurements were approximately symmetric between the north and south sides. Despite having an irregular shape, the grout debonding was very widespread throughout the interface and did not lead to unsymmetrical horizontal shear resistance.
The thin layer of grout that was applied to the machine finished surfaces showed strong bonding capabilities. The bond created between the grout and the bottom block was strong enough to remain intact after failure within over 50% of the interface area for both specimens. The limiting strength of these grouted specimens seemed to be the strength of the top block and the bond between the bottom block and the grout. It appears that the top block concrete formed a stronger bond with the grout layer for specimen DRY-MFG-2 than specimen DRY-MFG-1. This is indicated by the higher percentage of grout remaining on the top block surface after failure for specimen DRY-MFG-2.

3.3.3.7 Dry-Mix Specimens with Grouted Longitudinally Rake Roughened Surface

The MMTD of the dry-mix grouted longitudinally raked roughened specimens was approximately 30% greater than the grouted machine finished surfaces. The grout
layer partially filled the rake grooves and covered the raised ridges of hardened concrete, causing the grouted surface to form waves, as shown in Figure 3.43.

The sand patch test was performed on the bottom block surfaces prior to grouting, and again on a representative grouted surface. It was found that the grout increased the MMTD of the dry-mix raked surfaces an average of 33%.

The dry-mix grouted longitudinally raked roughened specimens had the highest horizontal slip capacity of all push-off specimens. The average peak horizontal slip measured by gages N1 and S1 was 0.0028 in. (0.071 mm).

![Figure 3.43](image-url) Photograph of grouted rake groove and raised ridges.

Similarly to the non-grouted raked specimens, the raised ridges fractured from the bottom block during failure. The grout and top block concrete that filled into the rake grooves remained entirely bonded to the top block of specimen DRY-LRG-2. However, in specimen DRY-LRG-1, the grout material within the rake grooves remained bonded to the bottom block in the region of the interface between gages N1 and S1, as shown in Figure 3.44.

The force-displacement plots for specimens DRY-LRG-1 and DRY-LRG-2 is shown in Figure 3.45 and Figure 3.46. Horizontal slip and vertical displacement were recorded for both of these specimens. The dry-mix grouted longitudinally raked
roughened specimens resisted an average force of 61.0 kips (271 kN), which corresponds to an average interfacial stress of 271 psi (1.87 MPa).

**Figure 3.44.** The grout that filled the rake grooves of specimen DRY-LRG-1 remained attached to the top block after failure, except in the region of the interface between gages N1 and S1.

![Figure 3.44](image)

**Figure 3.45.** Force-displacement plots for specimen DRY-LRG-1.

![Figure 3.45](image)
Figure 3.46. Force-displacement plots for specimen DRY-LRG-2.

The measurements taken by gages N2 and S2 for specimens DRY-LRG-1 and DRY-LRG-2 show a sudden increase in horizontal slip at a load of approximately 50 kips (220 kN). This slip is likely due to a local failure of a portion of the interfacial bond between gages N2 and S2.

3.3.3.8 Wet-Mix Specimens with Machine Finished Surface

The surfaces of the wet-mix machine finished specimens had higher MMTD than the dry-mix machine finished specimen surfaces. The higher MMTD of the wet-mix machine finished specimens was caused by the presence of rougher texture (Figure 3.47) and the slightly undulating characteristic of the surface (Figure 3.48). The wave-like undulations measured approximately 1/8 in. (3.2 mm) from crest to valley.
Figure 3.47. Comparison of dry-mix and wet-mix machine finished surfaces.
(a) Photograph of a wet-mix machine finished surface.
(b) Photograph of a dry-mix machine finished surface.

Figure 3.48. The small wave-like undulations on the wet-mix machine finished surfaces could be seen by placing a straight edge on the concrete surface.

The force-displacement plots for specimens WET-MFX-1 and WET-MFX-2 can be seen in Figure 3.49 and Figure 3.50, respectively. Specimen WET-MFX-1 failed at a force of 44.6 kips (198 kN), while specimen WET-MFX-2 failed at a significantly lower force of 28.7 kips (128 kN). Even the weaker of these companion specimens, WET-MFX-2, had a maximum interfacial stress of 128 psi (0.88 MPa), which exceeded the horizontal shear strength limit specified by ACI 318-08 of 80 psi (0.55 MPa).
The failure surfaces of the wet-mix machine finished specimens were smooth and very similar to the machine finished surfaces prior to placement of topping concrete. No
pieces of aggregate were visible on either the top block or bottom block surfaces after failure.

Despite the difference in peak interfacial strength, specimens WET-MFX-1 and WET-MFX-2 had similar displacement behavior. Gages N1 and S1 (row 1) measured significantly higher slip than rows 2 and 3. Vertical displacements were approximately equal in magnitude to the horizontal slips recorded by gages N3 and S3. The low displacement capacity of these specimens may indicate that the machine finished surfaces lack capability to redistribute shear stress on the onset of failure. Failure of the entire interface may therefore progresses from a local horizontal shear failure.

3.3.3.9 Wet-Mix Specimens with Sandblasted Surface

The roughness achieved through sandblasting increased the MMTD of the wet-mix sandblasted specimens by 17% compared with the wet-mix machined finished specimens. A photograph of a wet-mix sandblasted surface can be seen in Figure 3.51. The sandblasting process effectively removed a thin layer of material from the top of the bottom block surface. This is the layer that would contain laitance and other debris that detract from the quality of interfacial bond. The wet-mix sandblasted specimens had, on average, 51% higher interfacial shear strength and 86% higher horizontal slip capacity compared to wet-mix machine finished specimens. The dry-mix sandblasted specimens, which had nearly the same interfacial shear strength as the dry-mix machine finished specimens, did not exhibit these benefits from sandblasting.

The surface of specimen WET-SBX-2 had a raised ridge that passed across the entire interface in the transverse direction (perpendicular to span) formed during fabrication. The ridge was approximately 1.5 in. (38 mm) wide and 1/16 in. (1.6 mm)
thick. Photographs of the ridge can be seen in Figure 3.52. This defect was removed from the specimen surface on either side of the interface using a grinder so the top block formwork could be positioned correctly. The ridge was not removed from the surface within the interface area to include inherent variability during fabrication of the hollow core units.

![Photograph of wet-mix sandblasted surface.](image)

**Figure 3.51.** Photograph of wet-mix sandblasted surface.

![Photographs of raised ridge on specimen WET-SBX-2 prior to testing.](image)

**Figure 3.52.** Photographs of raised ridge on specimen WET-SBX-2 prior to testing.

Force-displacement plots for specimens WET-SBX-1 and WET-SBX-2 can be seen in Figure 3.53 and Figure 3.54, respectively. Specimen WET-SBX-1 failed at a force of 60.2 kips (268 kN) and specimen WET-SBX-2 failed at a force of 50.6 kips (225 kN).
The force-displacement behavior of the two wet-mix sandblasted specimens appears to be different. The force-displacement plots for specimen WET-SBX-2 show...
that a sudden increase in horizontal slip and vertical displacement occurred immediately prior to failure. As shown in Figure 3.55, this sudden increase in displacement was recorded by all south side gages (S1, S2, S3 and S4), but was not recorded by any north side gages. The sudden increase in displacement is representative of slips occurring at incipient failure, perhaps triggered by presence of the ridge on the surface of the hollow core unit shown in Figure 3.52.

![Force-displacement plots of gages N3, S3, N1 and S1 for specimen WET-SBX-2.](image)

Figure 3.55. Force-displacement plots of gages N3, S3, N1 and S1 for specimen WET-SBX-2.

After failure, a different texture was observed on the surface of the wet-mix sandblasted bottom blocks. A very thin layer of top block material remained bonded to the bottom block throughout the interface area, causing the bottom block failure surface to have the texture seen in Figure 3.56. Also, eight small, approximately 0.50 in. (13 mm) diameter, pieces of top block material remained bonded to the bottom block of the
wet-mix sandblasted specimens after failure. These pieces were mainly distributed in the interface region between the row 1 gages (N1 and S1) and the row 2 gages (N2 and S2).

![Image](image.png)

**Figure 3.56.** Photograph of bottom block failure surface of specimen DRY-SBX-1.

A portion of the material making up the surface anomaly on the bottom block of specimen DRY-SBX-2 was sheared off during testing. The remaining portion of the anomaly was still bonded to the bottom block after failure, as shown in Figure 3.57.

![Image](image.png)

**Figure 3.57.** Photograph of raised ridge on the surface of specimen WET-SBX-2 after testing.
3.3.3.10  Wet-Mix Specimens with Longitudinally Broom Roughened Surface

The surface of the wet-mix broom roughened specimens is characterized by many thin and shallow parallel striations. Since the broom roughening process was conducted by hand, the roughness striations were skewed relative to the edge of the interface, as shown in Figure 3.58. The skew angle of the roughness striations was approximately 2.8 degrees in specimen WET-LBX-1 and 3.8 degrees in specimen WET-LBX-2. Very small, approximately 1/16 in. (1.5 mm) diameter, raised pieces of hardened concrete were scattered throughout the surface, as shown in Figure 3.59. These pieces hardened next to the grooves as concrete was displaced during the brooming process.

Laitance was observed in the small grooves between each broom striation. This laitance persisted after sweeping the hardened surface with a broom and blowing with compressed air. ACI 318-08 Section 17.5.3.1 and the PCI Design Handbook Section 14.1 require that the surface of a concrete substrate be free of dust and laitance for development of horizontal shear strength when interfaces lack reinforcement crossing the shear plane. In this case, however, a laitance-free surface could not be practically achieved. Additional effort to remove the laitance from the striations (beyond sweeping and blasting with compressed air) was not made because it would be impractical.
Figure 3.58. Measurements taken to measure the skew angle of the broom striations for specimen WET-LBX-2.

Figure 3.59. Photograph of the raised bumps of hardened concrete on a wet-mix broom roughened surface.

Force-displacement plots for specimens WET-LBX-1 and WET-LBX-2 can be seen in Figure 3.60 and Figure 3.61, respectively. The failure force of the two companion specimens differed by 17.5 kips (77.8 kN), which is equal to a difference in interfacial shear stress of 77.8 psi (0.54 MPa).
Figure 3.60. Force-displacement plots for specimen WET-LBX-1.

Figure 3.61. Force-displacement plots for specimen WET-LBX-2.

After the failure of specimen WET-LBX-1, no pieces of top block material remained bonded to the bottom block. After removal of the top block, a patch of
pulverized bottom block concrete approximately 4 in. (100 mm) in diameter was noticed in the middle of the bottom block interface between gages N3 and S3 (Figure 3.62).

![Figure 3.62. Photograph of bottom block failure surface of specimen WET-LBX-1.](image)

Specimen WET-LBX-2 failed at an interfacial stress of 144 psi (0.99 MPa), which was the second lowest of all push-off specimens. It must be noted that the bottom block of specimen WET-LBX-2 was cast on a different day from all other wet-mix bottom blocks used during this experimental program. Cylinder tests indicated that the compressive strength of this bottom block was approximately equal to all other wet-mix specimens. The bottom block surface of specimen WET-LBX-2 had significantly more laittance than the other wet-mix broom roughened specimens, as shown in Figure 3.63.
The failure surface of specimen WET-LBX-2 also featured a small region of crushed bottom block concrete between gages N3 and S3 (Figure 3.64). This region was smaller than the region of crushed concrete on specimen WET-LBX-1. These regions of damaged material may be indicative of the location where failure initiated.
3.3.3.11 Wet-Mix Specimens with Transversely Broom Roughened Surface

The roughness properties of the wet-mix transversely broom roughened specimens were the same as the wet-mix longitudinally broom roughened specimens except the broom striations ran in the direction perpendicular to the span, rather than parallel to span.

Laitance was observed within the broom striations in the two wet-mix transversely broomed specimens. For these specimens, however, the laitance dust was not as prevalent as in specimen WET-LBX-2.

Force-displacement plots for specimens WET-TBX-1 and WET-TBX-2 can be seen in Figure 3.65 and Figure 3.66, respectively. The two specimens exhibited very similar interfacial shear strength. Specimen WET-TBX-1 failed at a force of 57.9 kips (257 kN) and specimen WET-TBX-2 failed at a force of 55.7 kips (248 kN).

![Force-displacement plots for specimen WET-TBX-1.](image)

Figure 3.65. Force-displacement plots for specimen WET-TBX-1.
Figure 3.66. Force-displacement plots for specimen WET-TBX-2.

The higher interfacial strength of the wet-mix transversely broomed specimens in comparison to the wet-mix longitudinally broomed specimens indicate that the transverse orientation of broom striations increases the strength of composite bond through mechanical interlock. This interlocking is most effective when the roughness striations are oriented in the direction perpendicular to the applied shear force.

Pieces of crushed bottom block concrete were observed on the failure surfaces of the wet-mix transversely broomed specimens (Figure 3.67). The crushed concrete particles were concentrated mostly in the interface area between gages N1 and S1. A portion of the broom undulations on the bottom block crushed during loading, but the top block roughness undulations appeared to be intact after failure (Figure 3.68).

The force-displacement plots show that significantly less horizontal slip was recorded by gages N2 and S2 for specimen WET-TBX-2 than WET-TBX-1. No notable
differences in the failure surfaces of these two specimens that could explain the
difference in slips were observed.

Figure 3.67. Photograph showing bottom block failure surface of a
wet-mix transversely broom roughened specimen.

Figure 3.68. Photograph showing top block failure surface of a
wet-mix transversely broom roughened specimen.

3.3.3.12 Wet-Mix Specimens with Grouted Machine Finished Surface

The wet-mix grouted machine finished specimens were prepared by adding a thin
(1/16 in. [1.58 mm]) layer of flowable grout to the surface of wet-mix machine finished
hollow core slab specimens prior to casting the top block concrete. The sand patch test
was performed on the bottom block surfaces prior to grouting, and again on a
representative grouted machine finished surface. It was found that the grout increased the MMTD of the wet-mix machine finished surfaces by an average of 60%. Unlike the dry-mix grouted machine finished specimens, which had greatly improved strength over their non-grouted counterparts, the wet-mix grouted machine finished specimens had approximately the same average interfacial shear strength as the non-grouted wet-mix machine finished specimens.

Force-displacement plots for specimens WET-TBX-1 and WET-TBX-2 can be seen in Figure 3.69 and Figure 3.70, respectively. The force-displacement data for the wet-mix grouted machine finished specimens are very similar to the wet-mix non-grouted machine finished specimens. For both sets of companion specimens, very small horizontal slips were measured by gages N2 and S2 and the vertical gages (N4 and S4). The average interfacial shear stress of the wet-mix grouted machine finished specimens was 161 psi (1.11 MPa), and the average interfacial shear stress of the wet-mix non-grouted machine finished specimens was 163 psi (1.23 MPa).
After failure of specimen WET-MFG-1, the entire grout layer was debonded from the bottom block and remained attached the top block. The bottom block failure surface resembled a wet-mix machine finished failure surface. No aggregate pieces were visible and no crushed concrete was observed.

The bottom block failure surface of specimen WET-MFG-2 had several small patches of grout still attached after failure (Figure 3.71). A larger patch of grout remained bonded to the bottom block near gage N1. The force-displacement data shown in Figure 3.72 show that the north side gages registered a smaller displacement than the south side gages. This indicates that the north side of the interface had higher stiffness and therefore contributed in larger proportion to the shear strength than the south side.
Figure 3.71. Analysis of bottom block failure surface of specimen WET-MFG-2.
(a) Photograph of specimen WET-MFG-2 bottom block failure surface.
(b) Region of failure surface where grout remained bonded to the bottom block is colored light gray.

Figure 3.72. Force-displacement plots of gages N3, S3, N1 and S1 for specimen WET-MFG-2.
3.3.3.13 Wet-Mix Specimens with Grouted Longitudinally Broom Roughened Surface

The MMTD of the wet-mix grouted longitudinally broom roughened specimens was approximately 43% lower than the non-grouted broom roughened surfaces. The grout layer covered the shallow broom striations, which resulted in a surface with less roughness. As with all other grouted specimens, a broom was used in the transverse direction to spread the grout layer onto the bottom block surface (Figure 3.73). Despite the loss in roughness, the grouted wet-mix longitudinally broomed specimens had, on average, 27% higher interfacial shear strength than their non-grouted counterparts.

![Figure 3.73](image)

**Figure 3.73.** Direction of broom striations of bottom block layer and grout layer for specimens WET-LBG-1 and WET-LBG-2.

Force-displacement plots for specimens WET-LBG-1 and WET-LBG-2 can be seen in Figure 3.74 and Figure 3.75, respectively. Specimen WET-LBG-1 failed at a force of 55.6 kips (247 kN), while specimen WET-LBG-2 failed at a lower force of 49.1 kips (218 kN).
No pieces of grout remained bonded to the bottom block surface of specimen WET-LBG-1 after failure. For this specimen, the entire grout layer remained bonded to
the top block after failure. A photograph of the bottom block failure surface can be seen in Figure 3.76.

![Figure 3.76](image.png)

**Figure 3.76.** Photograph of bottom block failure surface of specimen WET-LBG-1.

Several small patches of grout remained bonded to the bottom block of specimen WET-LBG-2 (Figure 3.77). Although a large patch of grout remained bonded to the bottom block near gage N1, the north gages (N1, N2, N3) measured larger horizontal slips than the south gages (S1, S2, S3), as shown in Figure 3.78. Approximately 10% of the grout layer remained bonded to the bottom block after failure; the remaining 90% of the grout remained bonded to the top block.
Figure 3.77. Analysis of bottom block failure surface of specimen WET-LBG-2.
(a) Photograph of specimen WET-LBG-2 bottom block failure surface.
(b) Regions of failure surface where grout remained bonded to the bottom block are colored light gray.

Figure 3.78. Force-displacement plots of gages N1 and S1 for specimen WET-LBG-2.
3.4 Discussion of Results

Observations on the relationship between interface shear strength and other measured parameters have been made, and are shown in this discussion section.

3.4.1 Relationship Between Interface Strength and Surface Roughness

A strong relationship has been observed between mean macrotexture depth (MMTD) and interface shear strength of non-grouted dry-mix specimens. The correlation coefficient between these two properties was found to be 0.88 and the r-squared value was found to be 0.77. If the dry-mix grouted specimens are included, the correlation coefficient decreases to 0.69 and the r-squared value decreases to 0.472. The interface strength has been plotted against the MMTD for dry-mix specimens in Figure 3.79. In this plot, MMTD measurements of grouted specimens were taken on representative grouted surfaces. It should be recalled that the MMTD measurement does not identify the trending direction of roughness, which was found to contribute significantly to the high strength of the transversely broom roughened dry-mix specimens (DRY-TBX-1 and DRY-TBX-2).
A weak positive relationship between interface strength and MMTD has been observed for the wet-mix specimens, with a correlation coefficient of 0.28 and an r-squared value of 0.079. The correlation coefficient and r-squared values are unchanged if the grouted specimens are omitted. The interface strength has been plotted against the MMTD for wet-mix specimens in Figure 3.80. In this plot, MMTD measurements of grouted specimens were taken on representative grouted surfaces.
Figure 3.80. Relationship between mean macrotexture depth and interface shear strength for wet-mix specimens.

A summary of all correlation coefficients and r-squared values between interfacial strength and MMTD can be seen in Table 3.9.

Table 3.9. Summary of correlation coefficients and r-squared values between interfacial strength and surface MMTD.

<table>
<thead>
<tr>
<th>Data Set</th>
<th>Correlation coefficient</th>
<th>R-squared</th>
</tr>
</thead>
<tbody>
<tr>
<td>All dry-mix specimens</td>
<td>0.687</td>
<td>0.472</td>
</tr>
<tr>
<td>All non-grouted dry-mix specimens</td>
<td>0.878</td>
<td>0.771</td>
</tr>
<tr>
<td>All wet-mix specimens</td>
<td>0.281</td>
<td>0.079</td>
</tr>
<tr>
<td>All non-grouted wet-mix specimens</td>
<td>0.281</td>
<td>0.079</td>
</tr>
<tr>
<td>All specimens</td>
<td>0.306</td>
<td>0.093</td>
</tr>
<tr>
<td>All non-grouted specimens</td>
<td>0.428</td>
<td>0.183</td>
</tr>
</tbody>
</table>

The sandblasting process increased the MMTD of the dry-mix and wet-mix specimens 22% and 17% beyond that of the machine finished specimens, respectively. However, the average interface shear strength of the wet-mix sandblasted specimens raised 50% beyond that of the wet-mix machine finished specimens. The dry-mix
sandblasted specimens only gained 5% additional strength beyond the dry-mix machine finished specimens. For the wet-mix specimens, the sandblasting process not only increased roughness to the precast hollow core surface, but also removed a layer of laitance that formed on the surface due to the fabrication process. This laitance layer is detrimental to interface strength as it weakens the cohesive bond between the precast and cast-in-place concretes (Raths and Hoigard, 2004). The laitance layer forms as a result of drying of water-cement paste on the top concrete surface. Therefore, the dry-mix specimens had less laitance on the machine finished surfaces so the benefits from sandblasting on interface strength were less pronounced.

The broom roughened wet-mix specimens all had MMTD measurements between 0.04 and 0.05 in. (1.0 and 1.3 mm) with the exception of specimen WET-LBX-2, which had a MMTD of 0.0366 in. (0.929 mm). This particular specimen was cast on a different day than all other wet-mix specimens and, as a result, had a slightly different surface roughness. It was also noted that this specimen had more laitance on its surface than the other broom roughened specimens, which likely contributed to its low interface strength (Figure 3.81).

![Photograph of laitance on surface of specimen WET-LBX-2.](image)
The results of push-off testing suggest that mean macrotexture depth may be effective at predicting interface shear strength for dry-mix precast hollow core slabs. For wet-mix hollow core slabs, interfacial strength is influenced by both surface roughness and the presence of laitance on the composite surface. Higher interface shear strength was observed in the wet-mix specimens both when roughness was increased and when the laitance layer was removed.

### 3.4.2 Relationship Between Interface Strength and Horizontal Slip

The interface shear strength is plotted against the maximum horizontal slip in Figure 3.82 for all the specimens tested. The maximum horizontal slip values shown in this plot were calculated as the average peak horizontal slip measured by gages N1 and S1. These gages typically measured the highest slips in all specimens.

![Figure 3.82](image)

**Figure 3.82.** Push-off specimen interface strength plotted against maximum horizontal slip.

A strong positive relationship has been observed between interface shear strength and horizontal slip. The correlation coefficient between these two parameters was 0.80 and the r-squared value of the linear regression was 0.64.
3.4.3 Relationship Between Interface Strength and Vertical Displacement

The interface shear strength is plotted against the maximum vertical displacement in Figure 3.83. The maximum vertical displacement values shown in this plot were calculated as the average peak vertical displacement measured by gages N4 and S4. It should be noted that vertical displacement was not measured for five of the twelve dry-mix push-off specimens.

A strong positive relationship between interface shear strength and maximum vertical displacement is observed, with a correlation coefficient of 0.85 and an r-squared value of 0.72. Vertical displacement is only mobilized as the two blocks constituting the interface displace horizontally relative to each other. The surface roughness between the blocks generates unrestrained vertical movement because of lack of reinforcing steel crossing the interface.

![Graph showing relationship between interface shear strength and vertical displacement](image)

**Figure 3.83.** Push-off specimen interface strength plotted against maximum vertical displacement.

3.4.4 Relationship Between Interface Strength and Concrete Strength

No strong relationships were observed between material strength and interface shear strength during push-off testing. The influence of material strength on interface
shear strength was not directly investigated during the push-off testing phase, and therefore no effort was made to intentionally vary material strength during specimen construction. Only two specimens of each surface preparation type were tested, making it difficult to make any definitive observations on the influence of material strength on interface strength.

The relationship between top block compressive strength and interface shear strength is shown in Figure 3.86. The correlation coefficient between top block compressive strength and interface shear strength for dry-mix and wet-mix specimens was found to be 0.64 and -0.27, respectively. This indicates that there was a weak correlation between these two properties for dry-mix specimens. For wet-mix specimens, there appeared to be no correlation between top block compressive strength and interface strength. Similar correlations were found between top block tensile strength and interface strength, as shown in Figure 3.85.

![Figure 3.84. Push-off specimen interface strength plotted against top block compressive strength.](image)
Figure 3.85. Push-off specimen interface strength plotted against top block tensile strength.

The relationship between bottom block compressive strength and interface shear strength is shown in Figure 3.86. The apparent relationship between bottom block compressive strength and interface strength for dry-mix specimens was likely caused by the sequence of testing. Specimens with higher MMTD values or grout were tested at a later date when concrete strength had also increased due to aging. As shown in Section 3.3, the tests were conducted in such an order that the low strength (machine finished and sandblasted) specimens were generally tested first and the higher strength (broom roughened and grouted) specimens were tested last. The bottom block compressive strength of the wet-mix specimens varied very little throughout the testing program, and therefore its relationship to interface strength could not be observed.
Figure 3.86. Push-off specimen interface strength plotted against bottom block compressive strength.

It is clear that the varying surface roughness conditions had a greater influence on the interfacial strength of the specimens than material strengths. In order to make more conclusive observations on this relationship, variables that are highly influential on interface strength (such as surface roughness) must be held constant.
CHAPTER 4
NUMERICAL STUDY

4.1 Introduction

A numerical study was conducted to determine the most critical conditions for horizontal shear in composite hollow core slabs. This was accomplished by calculating the maximum superimposed live load required to reach flexural, vertical shear and horizontal shear capacity according to design equations for generic hollow core slab cross sections with a range of cross sectional properties and span lengths. The study was limited to simply supported single span slabs with distributed loading.

4.2 Numerical Study Methods

The numerical study was conducted by first selecting a generic composite hollow core slab cross section to serve as a baseline for further comparisons. Additional generic cross sections, referred to as comparison sections, were generated by altering material and individual cross sectional properties of the baseline section at a time while maintaining all other features constant. The area and moment of inertia of each generic hollow core slab cross section were estimated using the models presented in Section 4.2.2 of this thesis. Cross sectional properties of the composite section were then calculated using the dimensions of the topping slab and the hollow core slab properties.

The flexural and shear force capacity of the selected cross sections were calculated using the design equations in ACI 318-08 and the PCI Design Handbook (2010). Flexural capacity was calculated at the strength and serviceability limit states. Vertical and horizontal shear capacity was only calculated at the strength limit state. The
calculations performed to find the flexural and shear capacity of the generic cross sections can be seen in Section 4.2.3 and Section 4.2.4 of this thesis, respectively.

The maximum distributed unfactored live load, defined as the safe service load, required to reach the flexural and shear capacity was calculated for span lengths ranging from 5.0 ft. (1.5 m) to 55 ft. (17 m) at intervals of 1.0 ft. (0.30 m). The maximum service distributed live load was used as a means to compare whether flexure or shear force controlled capacity of elements using a common parameter.

4.2.1 Numerical Study Assumptions

Several assumptions were made during the numerical study to simplify the calculations. The prestressing strand initial pull was assumed to be 75 percent of the ultimate tensile strength of the strand. The total prestressing loss was assumed to be 18 percent of the initial pull force.

All safe service loads shown in the results section of this numerical study include load factors and strength reduction factors where applicable. That is to say, the safe service load capacities shown at the strength limit states represent the amount of unfactored live load needed to reach the strength-reduced design capacity for the given slab cross section and span length after load factors are applied. In calculations related to the strength limit state, it was assumed that no dead loads other than self-weight were present. In flexural serviceability calculations, no load factors or strength reduction factors were used.

In service limit state calculations, it was assumed that the hollow core slabs were unshored during the placement of the composite topping. As a result, the precast section was required to resist the entire dead load while the composite section resisted live load.
In accordance with ACI 318-08 Section 17.2.4, it was assumed that the composite section resisted all loads (dead and live) for strength limit state calculations.

It should be noted that the web width cross section parameter is the total width of all the webs in the hollow core slab. Changes to the web width were assumed to not affect the hollow core slab moment of inertia. Based on this assumption, alterations to the web width would make the theoretical hollow cores an oblong shape.

### 4.2.2 Hollow Core Slab Cross Section Property Models

The numerical study was conducted using generic hollow core slabs with cross sectional properties similar to those currently produced. Two models were created to determine the cross sectional area and moment of inertia of a generic hollow core slab using only the overall width and height as input parameters. These models allowed many generic hollow core slab cross sections to be generated without the need to design the precise quantity, size, shape and placement of the voids and other cross section features (such as keyways) that typically vary from producer to producer.

The models were calibrated using the cross sectional properties of existing hollow core slabs tabulated in the *PCI Manual for the Design of Hollow Core Slabs* (1998). This source included hollow core slab data from the manufacturing machines listed in Table 4.1.
Table 4.1. List of hollow core slab fabrication machines used in cross sectional area and moment of inertia models.

<table>
<thead>
<tr>
<th>Fabrication Machine</th>
<th>Quantity of hollow core slab cross sections included in model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dy-Core</td>
<td>5</td>
</tr>
<tr>
<td>Dynaspan</td>
<td>8</td>
</tr>
<tr>
<td>Elematic</td>
<td>5</td>
</tr>
<tr>
<td>Flexicore</td>
<td>7</td>
</tr>
<tr>
<td>Spancrete</td>
<td>6</td>
</tr>
<tr>
<td>Ultra-Span</td>
<td>5</td>
</tr>
</tbody>
</table>

The resulting models to compute cross sectional area and moment of inertia of generic hollow core slabs are illustrated in Figure 4.1 and Figure 4.2. The cross sectional area model was obtained by first plotting the cross sectional area of the existing hollow core slabs against the product of their respective width and height. A relationship between this product and the cross sectional area was found by performing a linear regression on this plotted data. The moment of inertia model was created using a similar approach, but involved conducting a linear regression between the moment of inertia of each hollow core unit and the product of cross sectional width and height cubed.

![Figure 4.1. Model used to find the cross sectional area of generic hollow core slabs.](image-url)
4.2.3 Flexural Capacity Calculation Approach

Flexural capacity was calculated at the strength and serviceability limit states. The design equations in Section 18.7 of ACI 318-08 and Section 5.2.1 of the PCI Design Handbook were used to perform the strength limit state calculations. The stress in the bonded prestressing steel, \( f_{ps} \), was found using the approximate equation given in Section 18.7.2 of the ACI 318-08 code. The equations used to assess flexural strength are provided in the appendix of this thesis.

The specifications in Section 18.4 of ACI 318-08 and Section 5.2.2 of the PCI Design Handbook were used to perform the serviceability limit state calculations. The stress requirements for class U (uncracked) members were used to limit flexural stresses at the serviceability limit state.

4.2.4 Shear Capacity Calculation Approach

The vertical shear capacity was considered to be the smaller of web-shear and flexural-shear capacities, which were found using the equations in Section 11.3 of ACI
318-08 as shown in the appendix of this thesis. The vertical shear capacity was calculated at two sections: (1) at a distance h/2 from the support, and (2) at a distance 0.25L from the support, where h is the height of the composite cross section and L is the span length.

The horizontal shear capacity was calculated using the evaluation method referred to as method A in Section 2.3 of this thesis. The horizontal shear calculations followed the specifications in Section 17.5.3 of ACI 318-08. Unless stated otherwise, the horizontal shear strength of 80 psi (0.55 MPa) specified by the ACI 318-08 code was used at any given section.

4.2.5 Baseline Composite Hollow Core Slab Material and Cross Sectional Properties

The use of a baseline section allowed the effect of altering individual material and cross sectional properties to be evaluated. The baseline attributes were chosen to be near the median of existing hollow core slab properties. The baseline material and cross sectional properties are listed in Table 4.2.
Table 4.2. Baseline composite hollow core slab material and cross sectional properties.

<table>
<thead>
<tr>
<th>Hollow core slab</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>10 in. (250 mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Width</td>
<td>4.0 ft. (1.2 m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web width *</td>
<td>10 in. (250 mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>f'c</td>
<td>5000 psi (34.5 MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth to bottom strands</td>
<td>8.75 in. (222 mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area</td>
<td>256 in.² (165 * 10³ mm²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moment of inertia</td>
<td>2880 in.⁴ (1.20 * 10⁹ mm⁴)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weight</td>
<td>67 lb/ft.² (3.2 kN/m²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Prestressing steel (in hollow core slab)

| Number of strands         | 7      |
| Strand diameter           | 7/16 in. (11.1 mm) |
| f'pu                      | 270 ksi (1860 MPa) |
| Initial pull              | 75%    |
| Total loss                | 18%    |
| Strand type               | 7 wire, low relaxation |

Topping slab

| Thickness                 | 2 in. (50 mm) |
| f'c                       | 4000 psi (27.6 MPa) |
| Effective width           | 43 in. (1100 mm) |
| Weight                    | 25 lb/ft.² (1.2 kN/m²) |

Composite section

| Height                    | 12 in. (300 mm) |
| Width                     | 4 ft. (1.2 m) |
| Depth to bottom strands   | 10.75 in. (270 mm) |
| Effective area            | 340 in.² (220 * 10³ mm²) |
| Effective moment of inertia | 5222 in.⁴ (2.17 *10⁹ mm⁴) |
| Weight                    | 92 lb/ft.² (4.4 kN/m²) |

Note: The web width is the sum of the widths of all webs within the hollow core slab.

4.3 Numerical Study Results

The results of the numerical study show that short to medium span hollow core slabs, typically ranging from 5.0 to 25 ft. (1.5 to 7.6 m), are controlled by shear. Due to the lack of transverse reinforcement in hollow core slabs, span lengths as long as 30 ft.
(9.1 m) were governed by shear. At longer spans, the slab units were controlled by flexural stresses at the serviceability limit state.

Horizontal shear was not often found to be the limiting criterion for design. Horizontal shear strength was found to be critical only when the vertical shear strength was increased, such as by increasing the web width or compressive strength of the hollow core slab. More detailed discussions of the results of the numerical study are presented in the following sections.

4.3.1 Baseline Composite Hollow Core Slab Results

The safe service load chart for the baseline composite hollow core slab can be seen in Figure 4.3. In Section 4.3.2 of this thesis, comparisons are made between this baseline section and comparison sections, which were generated by altering a single parameter of the baseline model while holding all other parameters constant.

4.3.2 Comparison Composite Hollow Core Slab Results

The comparison sections were generated to explore the effects of altering individual material or cross sectional properties of a composite hollow core slab. The parameters observed in this numerical study were hollow core slab compressive strength, depth, width, web width, prestressed reinforcement area, topping slab thickness and topping concrete compressive strength (Table 4.3).
Table 4.3. Parameters observed in the numerical study using comparison sections.

<table>
<thead>
<tr>
<th>Parameter Evaluated</th>
<th>Figure and Page Number</th>
<th>Parameter minimum value</th>
<th>Parameter maximum value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollow core slab depth</td>
<td>Figure 4.4, Page 114</td>
<td>6 in. (152 mm)</td>
<td>12.6 in. (320 mm)</td>
</tr>
<tr>
<td>Hollow core slab web width</td>
<td>Figure 4.5, Page 115</td>
<td>8 in. (203 mm)</td>
<td>14 in. (356 mm)</td>
</tr>
<tr>
<td>Hollow core slab width</td>
<td>Figure 4.6, Page 116</td>
<td>2.0 ft. (0.61 m)</td>
<td>8.0 ft. (2.4 m)</td>
</tr>
<tr>
<td>Hollow core slab concrete compressive strength</td>
<td>Figure 4.7, Page 117</td>
<td>3000 psi (20.7 MPa)</td>
<td>7000 psi (48.3 MPa)</td>
</tr>
<tr>
<td>Prestressing steel area</td>
<td>Figure 4.8, Page 118</td>
<td>0.575 in² (370 mm²)</td>
<td>1.06 in² (667 mm²)</td>
</tr>
<tr>
<td>Topping slab thickness</td>
<td>Figure 4.9, Page 119</td>
<td>1.0 in. (25 mm)</td>
<td>3.0 in. (76 mm)</td>
</tr>
<tr>
<td>Topping slab concrete compressive strength</td>
<td>Figure 4.10, Page 120</td>
<td>3000 psi (20.7 MPa)</td>
<td>6000 psi (41.4 MPa)</td>
</tr>
</tbody>
</table>

Safe service load charts for the comparison sections can be found in Figure 4.4 through Figure 4.10. Each of these figures contains three plots orientated vertically. The
plot situated in the middle represents the baseline section, while the plots on the top and bottom represent strengths of comparison sections computed for the lower and upper value of each parameter being examined.

As expected, it was found that increasing hollow core slab depth (Figure 4.4) increased the flexural strength of the composite unit. The vertical shear strength for short spans also increased as the hollow core slab depth increased, due to the larger web area. Vertical shear strength did not increase significantly for long spans because the vertical shear strength of long span units was controlled by diagonal tension cracking rather than web shear. Section 11.4.6.1 of the ACI 318-08 code states that the factored shear force, $V_u$, in hollow core slabs with depth greater than 12.5 in. (318 mm) that do not contain transverse reinforcement must be limited to $0.5V_c$. Due to this specification, it was found that hollow core slabs deeper than 12.5 inches have significantly lower vertical shear design strength and are therefore are not likely controlled by horizontal shear.

Widening the hollow core slab web caused the web shear strength of the composite unit to increase. As shown in Figure 4.5, this can lead to the section being controlled by horizontal shear at short spans. As expected, altering the web width had no effect on flexural or horizontal shear strength.

When studying the influence of hollow core slab width (Figure 4.6), the ratio of the width to web width and prestressing strand area was held constant. Altering the hollow core slab width had no observed effect on safe service load capacity.
It was found that altering the hollow core slab concrete compressive strength (Figure 4.7) had only a small effect on the flexural capacity at the ultimate limit state, because the compression block was contained entirely in the topping slab and the effects of shoring were neglected per ACI 318-08 Section 17.2.4 and 17.2.5. The flexural capacity at the service limit state was affected by a small amount at large span lengths where the section was limited by tensile stresses at the bottom of the cross section. As expected, the vertical shear strength of the composite section was found to be highly dependent on the hollow core slab concrete compressive strength. However, the horizontal shear strength design equations do not account for higher strength associated with an increase in concrete compressive strength parameters and are therefore unaffected by changes in concrete compressive strength. Similarly to hollow core slab web width, horizontal shear may control the design of short span units as hollow core slab concrete compressive strength is increased due to the increase in vertical shear strength.

It was found that altering the total area of prestressing steel (Figure 4.8) had a large effect on the flexural strength of the units at long span lengths. The vertical shear strength for short spans, controlled by the web shear strength, increased slightly because of the influence of higher effective prestressing force.
Topping slab thickness variations (Figure 4.9) had little effect on the safe service load of the section. Vertical shear strength increased for thicker toppings but this strength increase was mostly offset by the increased weight of the composite unit. Increasing the topping thickness also increased flexural strength, but at long spans the increase in dead weight seemed to overcome the increase in strength. This was especially noticeable at the serviceability limit state, where dead load was resisted entirely by the hollow core slab section.

Increasing the topping slab compressive strength (Figure 4.10) had no effect on vertical or horizontal shear strength. Bottom fiber tensile stresses generated by bending under service loads governed long-span hollow core units so these slabs were also unaffected by this parameter.
**Hollow core slab (HCS):**

- **Width = 4 ft. (1.22 m)**
- **Depth varies**
  - Web width = 10 in. (254 mm)
  - $f'_c = 5000 \text{ psi (34.5 MPa)}$
  - **Strands:** 270 ksi (1862 MPa)
    - Prestressing steel area = 0.805 in$^2$ (519 mm$^2$)
- **Topping slab:**
  - Thickness = 2.0 in. (51 mm)
  - $f'_c = 4000 \text{ psi (27.6 MPa)}$

---

**Figure 4.4.** Effect of hollow core slab depth on safe service load.

---

- **Horizontal Shear (80 psi)**
- **Flexure (Strength)**
- **Flexure (Service)**
- **Vertical Shear**

HCS Depth = 6 in. (152 mm)

HCS Depth = 10 in. (254 mm)

HCS Depth = 12.6 in. (320 mm)
Hollow core slab (HCS):
- Width = 4 ft. (1.22 m)
- Depth = 10 in. (254 mm)

**Web width varies**
- $f'_{c} = 5000$ psi (34.5 MPa)
- Strands: 270 ksi (1862 MPa)
  - Prestressing steel area = 0.805 in$^2$ (519 mm$^2$)

**Topping slab:**
- Thickness = 2.0 in (51 mm)
- $f'_{c}' = 4000$ psi (27.6 MPa)

---

**Figure 4.5.** Effect of hollow core slab web width on safe service load.
Hollow core slab (HCS):

**Width varies**
- Depth = 10 in. (254 mm)
- Web width equal to 10/48 of HCS width
- $f'_{c} = 5000$ psi (34.5 MPa)
- Strands: 270 ksi (1862 MPa)
- 0.201 in$^2$ (130 mm$^2$) of prestressing steel per 1.0 ft. (0.305 m) of hollow core slab width

**Topping slab:**
- Thickness = 2.0 in (51 mm), $f'_{c} = 4000$ psi (27.6 MPa)

---

**Figure 4.6.** Effect of hollow core slab width on safe service load.
Hollow core slab (HCS):
Width = 4 ft. (1.22 m)
Depth = 10 in. (254 mm)
Web width = 10 in. (254 mm)

**HCS $f_{c'}$ varies**
Strands: 270 ksi (1862 MPa)
Prestressing steel area = 0.805 in$^2$ (519 mm$^2$)

**Topping slab:**
Thickness = 2.0 in. (51 mm)
$f_{c'}$ = 4000 psi (27.6 MPa)

**Figure 4.7.** Effect of hollow core slab compressive strength on safe service load.
Figure 4.8. Effect of prestressing steel area on safe service load.
Hollow core slab (HCS):
- Width = 4 ft. (1.22 m)
- Depth = 10 in. (254 mm)
- Web width = 10 in. (254 mm)
- $f'_{c} = 5000$ psi (34.5 MPa)
- Strands: 270 ksi (1862 MPa)
- Prestressing steel area = 0.805 in$^2$ (519 mm$^2$)

Topping slab:
**Thicknes varies**
- $f'_{c} = 4000$ psi (27.6 MPa)

---

**Figure 4.9.** Effect of topping slab thickness on safe service load.
Hollow core slab (HCS):
- Width = 4 ft. (1.22 m)
- Depth = 10 in. (254 mm)
- Web width = 10 in. (254 mm)
- $f'_c = 5000$ psi (34.5 MPa)
- Strands: 270 ksi (1862 MPa)
- Prestressing steel area = 0.805 in$^2$ (519 mm$^2$)

Topping slab:
- Thickness = 2.0 in. (51 mm)

**Topping $f'_c$ varies**

---

**Figure 4.10.** Effect of topping slab compressive strength on safe service load.
CHAPTER 5

FINITE ELEMENT ANALYSIS

5.1 Introduction

A finite element model was used to calculate the behavior of a simply supported hollow core slab with a composite topping. The finite element model was created using the SAP 2000-V.14.2 computer program with the goal of understanding the mechanism of horizontal shear failure in full scale components. The model was effective at showing the distribution and progression of shear stresses leading to horizontal shear failure. Additionally, the model provided insight into the role of interface strength and stiffness on the behavior of the horizontal shear failure mode. Results from the push-off testing phase were used to generate the interface strength-slip relationships used in the finite element model.

5.2 Finite Element Modeling Approach

A simply supported hollow core slab with a composite topping slab was modeled as shown in Figure 5.1. The hollow core slab was modeled to resemble the baseline hollow core slab introduced in Section 4.2.5 (see Table 4.2 for material and cross section properties). A span length of 12 ft. (3.66 m) was selected to ensure development of high interface shear stress demands as found through the numerical study. To be consistent with the assumptions made in the numerical study, an additional bearing length of 3.0 in. (76 mm) was added to either side of the hollow core slab. To evaluate the influence of interface shear strength and stiffness on the behavior of the horizontal shear failure mode,
three different sets of interface strength-slip curves were examined as described in detail in the following section.

5.3 Description of Finite Element Model

The hollow core slab and topping were modeled using four node thin shell elements, resulting in a 2D plane stress model of the system. A 2D plane stress model was chosen because it was judged that it gave a sufficiently accurate representation of the problem by including the primary effects contributing to interface shear capacity (in plane forces and moments). The transverse (out-of-plane) distribution of interface shear stresses was, therefore, neglected because its effect on the global element behavior was felt to be negligible. Displacements in the out-of-plane directions were disabled in the model. Each shell element was assigned a thickness as shown in Figure 5.2. Element thicknesses varied along the depth of the modeled member to approximate the cross sectional geometry of a composite hollow core slab.

![Figure 5.1. Composite hollow core slab modeling approach.](image)

*Note:* The spacing between the topping slab and the hollow core slab is exaggerated in this figure to better show the placement of the interface elements.
The effect of the prestressing force was modeled as point loads applied at the prestressed reinforcement level. A point load was placed on both ends of the hollow core slab at a location 1.25 in. (31.8 mm) above the extreme bottom fiber (Figure 5.3). Each point load was assigned a magnitude of 133.7 kip (595 kN), which is equal to the total prestressing force used for the baseline hollow core slab in the numerical study.

Interaction between topping slab elements and hollow core slab elements was modeled using a series of multi-linear elastic link elements (Figure 5.4). The shape of the force-displacement relationship assigned to the interface links in the direction resisting horizontal shear is shown in Figure 5.5. The links were given infinite stiffness in the axial (vertical) direction and zero rotational stiffness. Each link was spaced at 2 in. (51 mm), and therefore had a tributary area of 96 in² (619 cm²).
5.3.1 Loading

The non-linear staged construction load case in SAP 2000 was used to accurately model the construction of the prestressed member. Using this staged construction load case, elements and loads can be added to the model sequentially. This allowed the prestressing and self-weight stresses to only affect the hollow core slab elements.

In the first stage, the hollow core slab elements were introduced to the model and the eccentric prestressing force was applied, generating an upward camber (Figure 5.6a). Also during this stage, the self-weight of both the hollow core slab and the topping slab was applied to the hollow core slab as a series of point loads (approximating a distributed load). The point loads were applied along the centerline of the hollow core slab.

The topping slab and interface link elements were added to the model in the second stage. These elements were attached to the hollow core slab elements after deformations
from prestressing and self-weight had already occurred. As shown in Figure 5.6b, this resulted in the topping slab having a level surface after being added, as expected.

![Figure 5.6. Deformed shaped of the model after the application of prestressing force and self-weight loads on the hollow core slab (a) and after the addition of the topping slab and interface elements (b).](image)

Note: The spacing between the topping slab and the hollow core slab is exaggerated in this figure to more clearly show the interface elements.

In the third and final stage, imposed displacements were applied to the model. Due to the presence of the interface link elements, both the hollow core and topping slab elements participated in resisting the externally generated loads in this stage from the imposed displacements.

To observe the behavior of the composite section as the interface elements reached strength, a displacement controlled loading scheme was applied. Displacement control could have been achieved through applying a downward displacement to a single node at midspan. A drawback to this approach was that a nearly constant magnitude of shear would be applied to the entire slab length, which is not typical for floor slab elements subjected to distributed loading. To simulate distributed loading, a feature in SAP 2000 called conjugate displacement control was used. In conjugate displacement control, the magnitude of a distributed load is adjusted in small increments until a predetermined amount of midspan deflection is achieved. This technique was used repeatedly to gradually increase midspan deflection so the loss of composite action could be observed.
5.3.2 Interface Element Properties

Three different strength-slip curves for the interface elements were evaluated using the finite element model (Figure 5.7). These strength-slip curves were used to define properties of the multi-linear link elements that represent the interface in the models. The horizontal shear stress values shown in Figure 5.7 were multiplied by the width of the hollow core slab and the spacing of interface link elements to obtain a force-deformation curve for each link element.

![Figure 5.7. Strength-slip properties of multi-linear elastic link elements. Note: Failure branch of the strength slip curves not shown for clarity.](image)

One model used interface link properties determined using the push-off testing results of the transversely broomed push-off specimen, DRY-TBX-2 (Figure 5.8). Another model used interface link properties calculated using the push-off testing results from the machine finished push-off specimen, WET-MFX-2 (Figure 5.9). Since the interface area used during the push-off testing was 225 in$^2$ (1451 cm$^2$) and the tributary area of each interface link element in the model was 96 in$^2$ (619 cm$^2$), the force values on each of these plots were multiplied by an adjustment factor of 0.427.
Figure 5.8. The use of the results from push-off test specimen DRY-TBX-2 to generate interface link element properties.

Figure 5.9. The use of the results from push-off test specimen WET-MFX-2 to generate interface link element properties.
In the third model, interface link properties were defined using the 80 psi (55 MPa) horizontal shear limit used in the ACI 318-08 code and the *PCI Design Handbook*. The slip capacity for these interface links were obtained from the linear relationship between interface shear strength and horizontal slip found using the aggregated results of the push-off testing phase (see Section 3.4.2). This maximum slip value was obtained using a line drawn two standard deviations below the mean linear trend of the data plotted in Figure 5.10. This approach was considered to approximate the 95 percentile confidence level commonly used in design codes. The initial stiffness of this interface model was adjusted to be equivalent to that of typical push-off test specimens.

![Figure 5.10](image)

**Figure 5.10.** Method used to find slip capacity for interface model based on ACI 318-08 horizontal shear stress limit.

### 5.4 Validation of Finite Element Model

Output from the finite element model has been validated using hand calculations. First, output from the hollow core slab without the topping slab was validated. The stresses at the extreme top and bottom fibers of the hollow core slab at midspan due to prestressing, self-weight and a superimposed live load were verified independently.
A comparison between stresses found from the finite element model and hand calculations for the hollow core slab can be seen in Table 3.1. Stress distribution plots of the hollow core slabs under prestressing, self-weight and superimposed live load can be seen in Figure 5.11, Figure 5.12 and Figure 5.13, respectively. The superimposed live load used to validate this model had a magnitude of 2 kip/ft. (29 kN/m). The stress distribution plot of the hollow core slab under all loading (prestressing, self-weight and superimposed live load) can be seen in Figure 5.14. The difference in prestressing stress at the bottom of the hollow core slab between the finite element model and hand calculations is likely due to the approximated cross sectional shape and area obtained by using shell elements. This validation was considered to be sufficient for modeling the horizontal shear behavior of the composite slab.

**Table 5.1.** Comparison of top and bottom stress values found from the finite element model and hand calculations (hollow core slab only).

<table>
<thead>
<tr>
<th></th>
<th>Stress at top of HCS, ksi (MPa)</th>
<th>Stress at bottom of HCS, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Finite Element Model</td>
<td>Hand Calculation</td>
</tr>
<tr>
<td>Prestressing</td>
<td>0.350 (2.41)</td>
<td>0.347 (2.39)</td>
</tr>
<tr>
<td>Self-weight</td>
<td>-0.140 (-0.965)</td>
<td>-0.138 (-0.951)</td>
</tr>
<tr>
<td>Live load</td>
<td>-0.759 (-5.23)</td>
<td>-0.750 (-5.17)</td>
</tr>
<tr>
<td>All</td>
<td>-0.549 (-3.79)</td>
<td>-0.541 (-3.73)</td>
</tr>
</tbody>
</table>

*Note: Compression is negative.*

**Figure 5.11.** Stress distribution of the simply supported hollow core slab under prestressing load.

*Note: Compression is negative, contours are in units of ksi (1.00 ksi = 6.90 MPa).*

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Figure 5.12. Stress distribution of the simply supported hollow core slab under self-weight load.
Note: Compression is negative, contours are in units of ksi (1.00 ksi = 6.90 MPa).

Figure 5.13. Stress distribution of the simply supported hollow core slab under superimposed live load.
Note: Compression is negative, contours are in units of ksi (1.00 ksi = 6.90 MPa).

Figure 5.14. Stress distribution of the simply supported hollow core slab under prestressing, self-weight and superimposed live load.
Note: Compression is negative, contours are in units of ksi (1.00 ksi = 6.90 MPa).

Output from the finite element model of the composite member, which included the topping slab and the hollow core slab, was also validated. A comparison between stresses found from the finite element model and hand calculations for the composite member can be seen in Table 5.2. The composite member was subjected to prestressing, self-weight and live load stresses during the validation. The prestressing and self-weight stresses were resisted by the hollow core slab only. Live load stress was resisted by the
composite member. A live load of 2 kip/ft. (29 kN/m) was used. A stress distribution plot of the composite member under these loads can be seen in Figure 5.15.

The interface link elements based on the 80 psi code limit were used during the validation. Since these link elements are not perfectly rigid, less horizontal shear stress was transferred into the top slab than predicted by hand calculations.

**Table 5.2.** Comparison of stress values found from the finite element model and hand calculations (composite member).

<table>
<thead>
<tr>
<th>Cross Section Location</th>
<th>Finite element model, ksi (MPa)</th>
<th>Hand calculation, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of topping slab</td>
<td>-0.405 (-2.79)</td>
<td>-0.495 (-3.41)</td>
</tr>
<tr>
<td>Bottom of topping slab</td>
<td>-0.250 (-1.72)</td>
<td>-0.330 (-2.28)</td>
</tr>
<tr>
<td>Top of hollow core slab</td>
<td>-0.121 (-0.834)</td>
<td>-0.121 (-0.834)</td>
</tr>
<tr>
<td>Bottom of hollow core slab</td>
<td>-0.777 (-5.36)</td>
<td>-0.760 (-5.24)</td>
</tr>
</tbody>
</table>

Note: Compression is negative.

**Figure 5.15.** Stress distribution of the simply supported composite member under prestressing, self-weight and superimposed live load. Note: Compression is negative, contours are in units of ksi (1.00 ksi = 6.90 MPa).

5.5 **Finite Element Modeling Results**

The interface links were the only elements in the FE model given a finite amount of strength. Other elements of the model, such as the hollow core slab and topping slab shell elements were assumed to have infinite strength. Cracking in the hollow core slab, whether caused by flexural or shear stresses, was not simulated in this model. The model is therefore unable to simulate the crack formation and growth into the interface region.
The displacement controlled distributed loading began after the topping slab was added to the model. At this stage, the hollow core slab had an initial upward camber of approximately 0.1 inches (2.54 mm) due to the eccentric prestressing force and self-weight. As more displacement was applied to the model, the interface link elements resisted an increasing amount of horizontal shear stress. Interface failure would occur as a critical load was reached, which was in equilibrium with a prescribed displacement. At the onset of interface failure, the magnitude of compressive stress that the topping slab was capable of transferring to the hollow core slab through horizontal shear decreased due to the failure branch of the multi-linear interface link elements. This led to a reduction in the compression force within the topping slab, a decrease in the lever arm between tension and compression, and a resulting lower flexural strength.

From the results of the finite element models it was observed that the topping slab would suddenly lengthen axially when complete interface failure occurred (Figure 5.16). This sudden lengthening was caused by the loss of deformation compatibility between hollow core and topping slab elements.

**Figure 5.16.** The topping slab recovered from compressive deformations after interface failure.
Prior to interface failure, the entire composite section was effective in resisting externally applied load. This can be appreciated in the axial stress continuity across the interface, particularly in regions away from the support (Figure 5.17). After interface failure, the hollow core slab and topping slab resisted the applied loading independently through individual moment couples formed within each depth, as seen in Figure 5.18.

Shear stress diagrams before and after interface failure can be seen in Figure 5.19 and Figure 5.20, respectively. The wave-shaped stress contours seen in the shear diagram before interface failure are caused by the interface links, which provided horizontal shear resistance at discrete points evenly spaced along the beam rather than in a continuous pattern.

**Figure 5.17.** Axial stress diagram immediately prior to interface failure.  
*Note:* Compression is negative, contours are in units of ksi (1.00 ksi = 6.90 MPa).

**Figure 5.18.** Axial stress diagram immediately following interface failure.  
*Note:* Compression is negative, contours are in units of ksi (1.00 ksi = 6.90 MPa).
5.5.1 Influence of Interface Element Properties

Output from the finite element models featuring different interface link properties is summarized in Table 5.3. The model with interface properties based on the transversely broomed test specimen gave the highest failure load and displacement, which was expected because the interface properties had the highest force and displacement capacity.
Table 5.3. Summary of finite element model output.

<table>
<thead>
<tr>
<th>Link properties based on:</th>
<th>ACI 318-08 Code (80 psi limit)</th>
<th>Specimen DRY-TBX-2</th>
<th>Specimen WET-MFX-2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Prior to interface failure:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Midspan deflection</td>
<td>-0.173 in. (-4.39 mm)</td>
<td>-0.902 in. (-22.9 mm)</td>
<td>-0.303 in. (7.70 mm)</td>
</tr>
<tr>
<td>Vertical reaction at each support</td>
<td>75.3 kip (335 kN)</td>
<td>273 kip (1210 kN)</td>
<td>111 kip (494 kN)</td>
</tr>
<tr>
<td><strong>After interface failure:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Midspan deflection</td>
<td>-0.176 in. (-4.47 mm)</td>
<td>-0.914 in. (-23.2 mm)</td>
<td>-0.307 in. (7.80 mm)</td>
</tr>
<tr>
<td>Vertical reaction</td>
<td>44.52 kip (198 kN)</td>
<td>156.5 kip (696 kN)</td>
<td>64.4 kip (286 kN)</td>
</tr>
</tbody>
</table>

The force at which horizontal shear failure occurred in the model with interface properties based on the code horizontal shear stress limit of 80 psi was compared with hand calculations (Table 5.4). Comparisons were made between this finite element model output and calculations made using horizontal shear evaluation methods A and B (see Section 2.3).

The *PCI Design Handbook* horizontal shear evaluation method (method B) more closely predicted the horizontal shear strength of this composite slab. The horizontal shear strength found using the ACI 318-08 approach (method A) was conservative by a large margin, especially considering that the results shown in Table 5.4 do not include strength reduction factors nor load factors. The superimposed distributed live load needed to cause horizontal shear failure predicted by evaluation method A was 41% less than the value found through finite element analysis.

Although methods A and B both use 80 psi (0.55 MPa) as the horizontal shear strength, method B assumes horizontal shear stresses can redistribute from points of high shear, such as near the supports, to points of low shear, such as near midspan, prior to
failure. The results in Table 5.4 show that for the modeled hollow core slab dimensions and interface properties, method B is a more accurate evaluation approach.

Table 5.4. Comparison between finite element model and horizontal shear evaluation methods.

<table>
<thead>
<tr>
<th>Method</th>
<th>Superimposed distributed live load needed to cause horizontal shear failure:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finite Element Model†</td>
<td>12.2 kip/ft (178 kN/m)</td>
</tr>
<tr>
<td>Method A (ACI 318-08)*</td>
<td>7.18 kip/ft (105 kN/m)</td>
</tr>
<tr>
<td>Method B (PCI Design Handbook)*</td>
<td>12.9 kip/ft (188 kN/m)</td>
</tr>
</tbody>
</table>

† Note: Finite element model result shown is from the model using interface link properties based on the 80 psi horizontal shear stress limit. * Note: Strength reduction factor and load factors not used in calculating these values.

To verify the redistribution phenomenon, the magnitude of horizontal shear resisted by the 80 psi code limit interface link elements along the length of the slab is plotted at several stages of loading in Figure 5.21. It can be seen that at near interface failure plastification of the interface links at a value of 80 psi (0.55 MPa) has occurred in a short region near the end of the beam (at a midspan deflection of -0.173 in. [43.9 mm]). This observed plastification is directly related to the nearly perfectly plastic second branch of the assumed interface link properties shown in Figure 5.7. Although the finite element model was not able to converge at a larger midspan deflection beyond this condition, it can be expected that plastification of shear links would progress toward midspan prior to full separation of the topping slab from hollow core elements.
Of the three interface models evaluated, the model with interface properties based on the transversely broomed test specimens had the highest horizontal shear strength and slip capacity. However, these link elements have an abrupt loss of strength at their ultimate load, and are therefore unable to exhibit the same plastification behavior as the 80 psi code limit interface link elements. As seen in Figure 5.22, these links were able to sustain more horizontal shear; however redistribution of horizontal shear stress along the span length was less evident. As seen in the load – displacement plot (Figure 5.23), the interface shear failure of the three models all occurred abruptly.
Figure 5.22. Progression of interface shear stress for model using interface link elements based on the transversely broomed test specimens.

Figure 5.23. Load – deflection plots for the three interface models examined.
CHAPTER 6
SUMMARY AND CONCLUSIONS

6.1 Summary of Experimental Testing

The horizontal shear strength of the interface between prestressed concrete hollow core slabs and cast-in-place concrete topping slabs was evaluated through a set of 24 push-off experiments. The push-off specimens consisted of a cast-in-place block of concrete cast directly on top of a segment of dry-mix or wet-mix precast hollow core slab. A variety of surface roughening techniques were performed on the top surface of the hollow core slab prior to placement of the cast-in-place block. The 15 in. by 15 in. (381 mm by 381 mm) interface created between the cast-in-place and precast concrete surface was subjected to a monotonically increasing shear force. Eight displacement transducers monitored relative horizontal displacement (slip) and vertical displacements between the top and bottom blocks.

An existing ASTM standard roughness measurement procedure for pavements (ASTM E965, 2006) was adapted to quantify hollow core slab surface roughness. The measurement procedure involved spreading a known volume of well graded sand onto the concrete surface using a handheld rubber spreading device. The diameter of the resulting sand patch is used to determine the mean macrotexture depth (MMTD), an average measure of roughness depth. This procedure provides an easily quantifiable parameter (MMTD) that can be linked to surface roughness.

The average interface shear strength of all push-off specimens was 227 psi (1.57 MPa). No specimens exhibited lower interface shear strength than the horizontal shear strength specified by the ACI 318-08 code, 80 psi (0.55 MPa). Surface roughness
seemed to be most effective when the grooves were oriented in the direction perpendicular to the applied shear force. A strong positive correlation was found between the interface shear strength and both horizontal slip and vertical displacement capacity.

A strong positive correlation was found between the interface shear strength of the eight non-grouted dry-mix specimens and their corresponding MMTD values. Each of the four dry-mix grouted specimens had higher interfacial shear strength than their un-grouted counterparts.

A weak positive correlation was found between the interface shear strength of the twelve wet-mix specimens and their corresponding MMTD values. It was evident that the interfacial shear strength of the wet-mix specimens was based not only on surface roughness but also presence of surface laitance. For some wet-mix specimens, the amount of surface laitance was too high to reasonably remove using compressed air or a broom. It was found that sandblasted wet-mix specimens had 51% higher interface shear strength and 86% higher horizontal slip capacity than the machine finished. The large increase in strength from sandblasting the wet-mix specimens was caused by a combination of increasing the surface roughness and removing the entire laitance layer.

6.2 Summary of Numerical Study

A numerical study was conducted by calculating the maximum superimposed live load required to reach flexural, vertical shear and horizontal shear capacity for a simply supported hollow core slab with a composite topping under distributed loading. The study was performed to find the cases where horizontal shear is most critical for composite hollow core slabs.
It was found that horizontal shear was not often the limiting criterion for design. Horizontal shear strength was found to be critical only for span lengths shorter than 20 ft. (6.1 m) where the vertical shear strength was increased, such as by increasing the web width or compressive strength of the hollow core slab. Although the numerical study found that horizontal shear, taken as 80 psi (0.55 MPa), is not often the limiting design criterion for hollow core slabs, the surface roughening methods shown to resist more than 80 psi of horizontal shear stress during experimental testing might be useful if any innovations in hollow core slab manufacturing procedures lead to a higher vertical shear strength.

6.3 Summary of Finite Element Analysis

A finite element model was developed to observe the mechanism of horizontal shear failure in full scale hollow core slab components. The interface between hollow core slab and topping slab was modeled using a series of multi-linear elastic link elements. Three different sets of strength-slip properties were applied to multi-linear elastic link elements: one to simulate the horizontal shear strength limits used in the current design codes and two others idealizing the behavior of push-off test specimens.

The results showed that models implementing interface link elements with higher peak strength were capable of sustaining higher amounts of superimposed live load. The sudden horizontal shear failures observed in all models is believed to be a consequence of the unloading branch chosen to represent link properties. Further studies are needed to verify that these models accurately represent behavior in large-scale components.

Comparisons were made between the peak superimposed live load capacity according to the finite element model and evaluation methods specified in ACI 318-08
and the *PCI Design Handbook* (2010). During these comparisons strength reduction factors and load factors were not used. The strength found using the ACI 318-08 code was conservative by a factor of 0.59. The evaluation method in the *PCI Design Handbook* provided a more accurate prediction of the finite element model strength, however over-predicted the strength by a factor of 1.06.

### 6.4 Areas of Future Work

Further experimental research focusing on the quality of surface preparation and the removal of strength detracting materials such as laitance would provide valuable insight into the importance of quality control. The quantity of laitance on concrete surfaces can vary greatly, and it is difficult to assess the amount of laitance on a concrete surface, even qualitatively. It may be difficult to recognize situations where the amount of laitance might significantly detract from horizontal shear strength. This research project has shown that sandblasting the precast surface can remove the laitance layer and therefore increase horizontal shear strength; however, sandblasting is a costly and time consuming process for fabricators. Research and development on other methods that could ease the removal of laitance and other strength detracting materials from the top surface of precast members at either a construction site or fabrication facility could result in more reliable composite connections.

Other experimental research involving roughened concrete surfaces should make use of the surface roughness quantification method (sand patch test) that has been developed in this thesis. Additional use of this test may help validate it as a reliable and reproducible measurement procedure of surface roughness of concrete elements. Unlike
many other surface roughness measurement procedures, the sand patch test is inexpensive, simple to perform and doesn’t require any electronic equipment.

Research into the effect of interface size on the behavior of push-off testing specimens would help validate the use of push-off testing as an indicator of horizontal shear strength. Currently, push-off testing is most effective when making comparisons between a set of specimens, such as the comparisons made between surface roughness in this thesis. A better understanding of the size effect of interfaces in push-off testing may allow results from these tests to be applied more easily into large scale models.

This research project has shown that a thin layer of grout can potentially increase the strength of the bond between precast and cast-in-place concretes. Further research into different grout mixes and grout placement techniques could lead to more detailed design recommendations regarding the use of grouts for composite bond.

Large scale testing of composite hollow core slabs could provide insight into the horizontal shear failure mode. Results from such testing could be used to more accurately calibrate the interface properties used in finite element models of composite slabs. Furthermore, large scale tests could establish a correlation between push-off test results and the behavior of real hollow core slabs.
APPENDIX

NUMERICAL STUDY EQUATIONS

This appendix provides a selection of the equations used to perform the numerical study. The equations in this appendix are included to provide a better understanding of the parameters affecting the design strength values reported in the numerical study.

Equations used to calculate flexural strength:

To calculate the stress in prestressing steel at nominal flexural strength:

(Based on Equation 18-3 in Section 18.7.2 of ACI 318-08.)

\[
f_{ps} = f_{pu} \left\{ 1 - \frac{\gamma_p}{\beta_1} \left[ \rho_p \frac{f_{pu}}{f'_c} \right] \right\}
\]

where:

- \( f_{ps} \) = Stress in prestressing steel at nominal flexural strength
- \( f_{pu} \) = Specified tensile strength of prestressing steel
- \( \gamma_p \) = Factor based on type of prestressing steel used, equal to 0.28 for low relaxation strands
- \( \beta_1 \) = Factor based on the depth of the equivalent rectangular compressive stress block, equal to 0.85 for 4000 psi (27.6 MPa) concrete
- \( \rho_p \) = Ratio between \( A_{ps} \) and \( b_d \)
- \( A_{ps} \) = Area of prestressing steel in the flexural tension zone
- \( f'_c \) = Specified compressive strength of concrete, strength of top block concrete used

Note:

The equation shown above is simplified from the ACI 318-08 formulation because mild reinforcement and compression reinforcement were not considered in the numerical study.

To calculate the nominal flexural strength:

\[
\phi M_n = \phi \left( f_{ps} A_{ps} \right) \left( d - \frac{a}{2} \right)
\]

where:

- \( \phi \) = Strength reduction factor, equal to 0.90 for flexure
- \( M_n \) = Nominal flexural strength
- \( a \) = Depth of the equivalent rectangular compressive stress block

Equations used to calculate vertical shear strength:
To calculate the shear force to cause flexure-shear cracking:
(Equation 11-10 in Section 11.3.3.1 of ACI 318-08)

\[
\phi V_{ci} = \phi \left\{ 0.6 \sqrt{f'_c} b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}} \right\}
\]

\[
V_{ci} \geq 1.7 \sqrt{f'_c} b_w d
\]

\[
M_{cre} = \left( \frac{I_{comp}}{y_t} \right) \left( 6 \sqrt{f'_c} + f_{pe} - f_d \right)
\]

where:

- \( \phi \) = Strength reduction factor, equal to 0.75 for shear
- \( V_{ci} \) = Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment
- \( b_w \) = Web width, equal to the sum of the widths of the multiple web regions for hollow core slabs
- \( d_p \) = distance from extreme compression fiber of composite section to the centroid of prestressing steel, need not be taken less than 0.80h
- \( h \) = Overall height of the composite section
- \( V_d \) = Shear force due to unfactored dead load
- \( V_i \) = Shear force due to all applied loads except dead load
- \( M_{cre} \) = Moment causing flexural cracking due to externally applied loads
- \( M_{max} \) = Maximum factored moment due to externally applied loads
- \( I_{comp} \) = Moment of inertia of composite section
- \( y_t \) = Distance from centroid of composite section to extreme fiber of section where tensile stress is caused by externally applied loads
- \( f_{pe} \) = Compressive stress in concrete due to prestressing only at extreme fiber of composite section where tensile stress is caused by externally applied loads; calculated at the bottom of the section for simply supported hollow core slabs with distributed loading
- \( f_d \) = Stress due to unfactored dead load at extreme fiber of composite section where tensile stress is caused by externally applied loads

To calculate the shear force to cause web-shear cracking:
(Equation 11-11 in Section 11.3.3.2 of ACI 318-08)

\[
\phi V_{cw} = \phi \left\{ 3.5 \sqrt{f'_c} + 0.3 f_{pc} \right\} b_w d_p + V_p \}
\]

where:

- \( V_{cw} \) = Nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in the web
- \( f_{pc} \) = Resultant compressive stress at the neutral axis of the composite section due to self-weight and prestressing only
- \( V_p \) = Vertical component of prestressing force at section, equal to zero when harped/draped strands are not used
BIBLIOGRAPHY


