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INELASTIC WEB CRUSHING CAPACITY OF HIGH-STRENGTH-CONCRETE STRUCTURAL WALLS

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ABSTRACT

Recent research findings indicate that high-strength concrete (HSC) can lead to new seismic design possibilities by allowing the incorporation of ductile shear failures as a new genre of ductile failure mechanisms. This paper reports research aimed at proving the noted hypothesis for HSC structural walls. Eight 1/5-scale thin-webbed walls with well-confined boundary elements and an aspect ratio of 2.5 were tested with design concrete compressive strengths of 34, 69, 103, and 138 MPa (5, 10, 15 and 20 ksi) under in-plane monotonic and cyclic loading. Results have proven that the use of HSC can enhance system ductility of structural walls by increasing web crushing shear capacity and thus allowing the development of stable inelastic flexural response. The enhancement, however, was found to be dependent on damage accumulation and thus limited to varying degrees by the damage tolerance of HSC. Preliminary findings on establishing the limits to inelastic web crushing are discussed.

Introduction

Increased reliability in the production of high-strength-concrete (HSC) is opening new possibilities for the seismic design of structures. However, maximizing the benefits from HSC requires reconsidering design approaches and the establishment of new performance limits. Long span bridges typically incorporate unique pier designs that are made hollow to reduce weight. Under non-seismic conditions, such designs are elegant and slender. Design of these elements for seismic demands creates challenges to ensure adequate inelastic behavior. For rectangular hollow piers this leads to relatively thick walls with heavily confined corner elements. Recent research on the shear strength show that diagonal compression shear strength is linearly related to concrete compressive strength, indicative of new possibilities for increasing shear capacities of lighter structural members with increased concrete strength. (Hines 2004). Such behavior for thin-walled elements with confined boundary regions can apply not only to pier walls, but also to flanged building walls, integral wall-panels for frame systems, thin-webbed girders, and hollow box pier and girder sections. However, this potential is currently impaired by outdated design provisions and lack of experience.

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Web crushing, or diagonal compression, shear failures consist of compression failure of concrete struts formed by diagonal tension cracking in the wall web before yielding of the transverse reinforcement. The failure is brittle and can result in rapid strength degradation and curtailed ductility. Therefore it is mandated to be avoided in design. Yet, in structural walls with well-confined boundary elements, web crushing shear failures can be forestalled until adequate ductile flexural response is attained. Experimental studies since the mid-seventies (Oesterle 1979; Vallenas 1979, Hines 2004) have demonstrated that structural walls with well-confined boundary elements or flanged sections could exhibit adequate ductile behavior before web crushing failure in the plastic hinge region at displacement ductility levels of 4 or greater. By establishing reasonable performance limits, ductile shear failures may control the strength and ductility of structural components for design purposes. The response of an element limited by shear failure but only after displaying moderate stable inelastic response can give rise to a new genre of ductile failure mechanism that may be termed "ductile shear failure." Like ductile flexural failures, a ductile shear failure should conform to appropriate seismic design principles. First, a stable inelastic deformation capacity must be possible. Second, stiffness degradation of the inelastic flexure-shear region under load reversals must be understood to define dependable performance limits. Unfortunately, knowledge and confidence on both of these issues is limited.

This paper presents results from a project evaluating the unconventional potential of using HSC to obtain ductile shear failures as an acceptable ductile failure mode in seismic design. The approach is non intuitive as the failure mode in question is a brittle failure and it is well known that concrete becomes more brittle as its compressive strength increases. Establishing dependable limits to inelastic web crushing behavior thus requires the evaluation of its dependence on concrete compressive strength, on inelastic deformation and on damage, or degradation, under cyclic loading. Establishing performance limits for HSC structural walls behaving in alternative ductile modes of failure is expected to contribute to the groundwork of the next stage in earthquake engineering design of thin-webbed elements and systems.

Web Crushing of Structural Walls

Evidence supporting the hypothesis that HSC increases web crushing capacity of structural walls has been available for some time. First, intuitively, web crushing strength should be proportional to the concrete compressive strength in view of the fact struts fracture under compression. Research at PCA in the mid seventies on walls with boundary elements (Oesterle 1979) noted that that specimen B6 with a concrete compressive strength of 22 MPa [3,165 psi] failed in web crushing at significantly lower deformation capacity than specimen B7 with a concrete compressive strength of 49 MPa [7,155 psi]. However, no further HSC structural walls were tested. These studies led to web crushing capacity models (Oesterle 1984) that recognized dependence on the concrete compressive strength, the inelastic deformation over the inelastic region of the wall (δ), and the applied axial forces (*N*) (Eq. 1). Studies in the mid eighties by Paulay (Paulay 1992) led to similar conclusions as shown by the recommended limits for shear stress in the plastic hinge regions of walls given by Eq. 2, which reflects that web crushing is directly proportional to concrete strength but that it degrades with increasing ductility (μ_Δ).

$$\upsilon_{\omega c} = \frac{1.8 f_c^{'}}{1+420\delta} \le 0.18 f_c^{'} \qquad \text{when } \frac{N}{A_g f_c^{'}} > 0.09 \tag{1}$$

$$\upsilon_{\omega c} = \left(\frac{0.22}{\mu_{\Delta}} + 0.03 \right) f_c^{'} \le 0.16 f_c^{'} \tag{2}$$

A recent study by Hines (Hines and Seible 2004) recognized the mechanisms noted above and proposed a flexure-shear model based on a truss analogy from the observed fanning crack pattern inside the plastic hinge region compared to the elastic (and parallel) shear cracking pattern outside this region (Fig. 1). The demand and capacity of the critical flexure-shear truss, N_{Ds} and N_{Cfs}, were defined as in Eq. 3 below.

$$\begin{aligned} N_{Dfs} &= \frac{\Delta T}{\cos\theta_{fs}} - f_1 s t_w \sin\theta_{fs} \le \frac{A_v f_y}{\sin\theta_{fs}} + f_1 s t_w \cos\theta_{fs} \cot\theta_{fs} \\ N_{Cfs} &= k f_c^{'} t_w R d\theta \end{aligned} \tag{3}$$

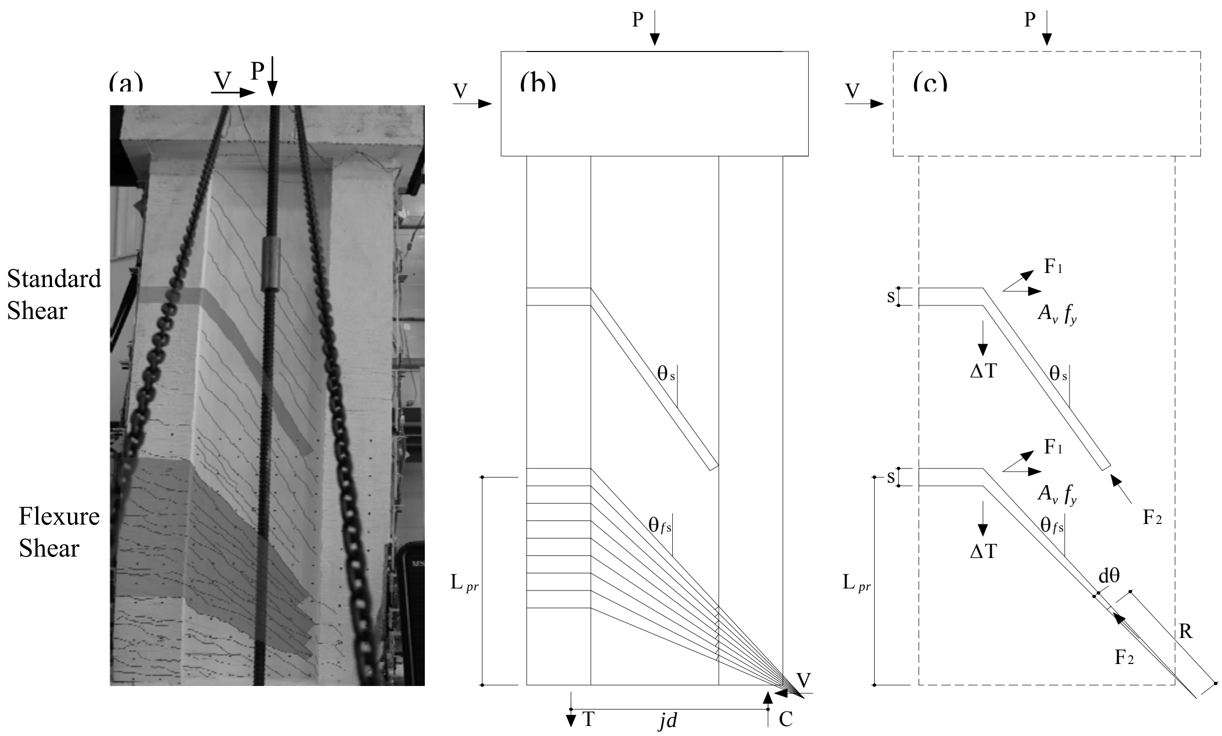


Figure 1. Flexure-shear and elastic shear mechanisms: (a) Flexure-shear and elastic shear regions in test wall, (b) Idealized force transfer regions, (c) Truss models. After (Hines and Seible 2004).

The direct dependence of web crushing capacity on concrete compressive strength is thus well recognized, as evidenced by the mentioned research. Yet, the lack of data has been dealt with by means of upper limits as noted in Eq. 1 and Eq. 2. While there is no explicit limit in the model by Hines and Seible, the prediction quality for HSC deteriorates in view of the fact that the available test data for calibration of the model is from tests of normal-strength-concrete walls. It should be noted that current design guidelines (ACI Committee 318 2008) do not consider direct dependence on the noted parameters controlling web crushing but rather limit it based on the square root of f'_c, a parameter commonly related to concrete tensile strength.

$$\upsilon_{\omega c} \le 10\sqrt{f_c^{'}} \quad \text{in psi} \tag{4}$$

Experimental Program

To verify the above-noted hypothesis and establish rational performance levels on the inelastic web crushing limits for HSC structural walls, eight 1/5-scale cantilever structural walls with design concrete compressive strengths of 34, 69, 103, and 138 MPa [5, 10, 15 and 20 ksi] were tested under cyclic and monotonic loading (Liu et al. 2009). The walls (Fig. 2) consisted of thin webs with heavily confined boundary elements and were designed to induce the desired failure mode and not to represent a component from a prototype structure. The identification name for the test units starts with 'M' followed by two digits denoting the design concrete compressive strength in kips/in^2 and then by a letter describing loading protocol: 'C' for cyclic and 'M' for monotonic loading, as is shown in Table 1. All walls had the same dimensions with an aspect ratio of 2.5 (Fig. 2). The steel reinforcement was essentially the same (Fig. 2) with a small variation for test unit M15C and for the wall transverse reinforcement spacing for the M20M and M20C walls which was 76 mm (3 in.).

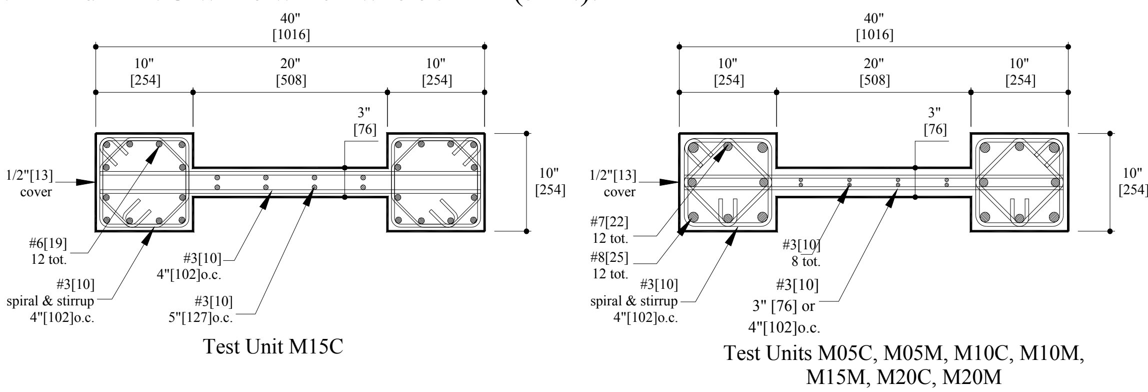


Figure 2. Test unit cross sections with reinforcement details.

Table 1. Test matrix and unit identification

	M05M	M05C	M10M	M10C	M15M	M15C	M20M	M20C
$f'_{c,nominal}$ MPa (ksi)	34 (5)	34 (5)	69 (10)	69 (10)	103 (15)	103 (15)	138 (20)	138 (20)
$f'_{c,test}$ MPa (ksi)	39 (5.6)	44 (6.4)	86 (12.5)	57 (8.2)	111 (16.1)	101 (14.7)	116 (16.8)	131 (19.0)
Loading	M	C	M	C	M	C	M	C

M = Monotonic; C = Cyclic.

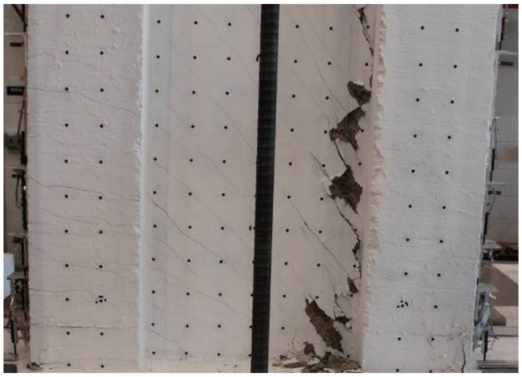
The test units were loaded monotonically and cyclically according to a standard, incrementally increasing, fully-reversed cyclic pattern, with constant axial load. The axial load for all test units was 580 kN (103 kips), corresponding to $0.10f'_cA_g$ for a compressive strength of 34 MPa (5 ksi). The axial load was applied by means of hydraulic jacks and high-strength rods reacting against the wall top load stub through a spandrel beam. The horizontal load was applied with a servo-controlled actuator connected to a load stub at the top of the wall. Lateral stability was provided by means of a pair of inclined tensioned chains on either side of the wall. An overview of the test setup is shown in Fig. 3.

Figure 3. Test setup overview.

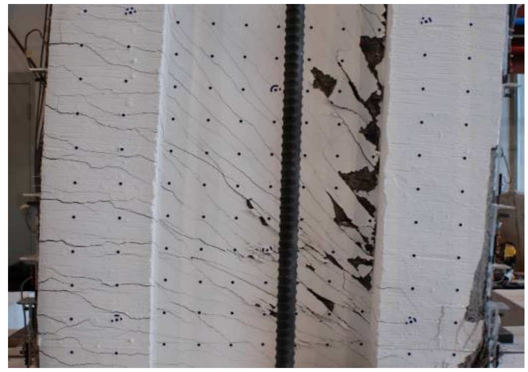
Experimental Results

All eight test units failed in web crushing according to the design requirements. Test units with lower concrete compressive strength (M05C, M05M and M10C) failed after only minor levels of inelastic response ($\mu_{\Delta} \sim 1.5$). Tensile cracking was minimal and cracks fully closed upon load reversal. No crack realignment was observed and thus these walls were limited by standard, or elastic, web crushing (Hines and Seible 2004). The failures were sudden with crushing of the concrete struts along the interface of the wall web and the compression boundary element almost simultaneously. This follows from the fact that the wall is under a constant shear demand and at low levels of inelastic deformation the elastic struts have essentially the same geometry (Fig. 1) and demand. The failure mode of M10C is shown in Fig. 4(a).

The rest of the test units exhibited moderate to high ductile behavior before web crushing. Cracking was much more extensive and crack spacing was much smaller. Fig. 4(b) and (c) show the failure modes of walls M10M and M15C. The fanning flexure-shear cracking patterns, as is shown in Fig. 4(d), was formed within the plastic hinge region with fairly flat cracks close to the bottom and much steeper cracks at the top w. For walls M15C and M20C, the crisscross cracking pattern under cyclic loading broke the wall web into small concrete blocks. The excessive stress introduced by crack misalignment, shear friction and distortion caused the web cover concrete to loose its bond to the reinforcement and spall off. The test units gradually lost their load-carrying capacity as a result diminished load transfer efficiency of the concrete struts. Web crushing was observed to expand a large area within the plastic hinge region of the wall and crushing of the flexure-shear struts initiated in the center of the web and then rapidly extended to the edge of the compression boundary element. Under monotonic loading, the compression struts remain integral though severely cracked and therefore the test units sustained larger inelastic deformations.

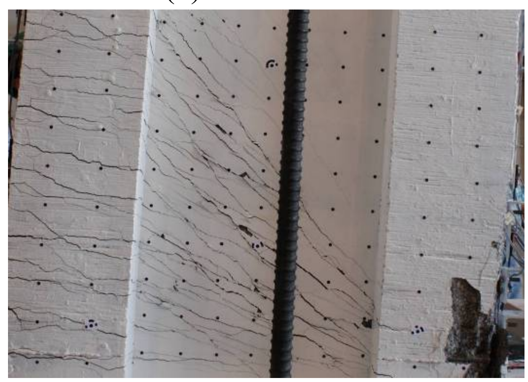


(a) Wall M10C



(b) Wall M10M

(c) Wall M15C



(d) Wall M20M

Figure 4. Representative crack patterns and failure modes.

The hysteretic force-displacement response of the four walls under cyclic loading is shown in Fig. 5. A comparison of the force-displacement envelopes of the monotonic and cyclic tests is shown in Fig. 6. Walls M05C, M05M and M10C failed at displacement ductility of about 1.5. The other walls exhibited moderate to high ductility before web crushing. Test units M15C and M20C achieved a displacement ductility of 4 while M10M, M15M and M20M failed at a displacement ductility of 6~9. The results clearly show that the higher compressive strength allowed the walls to obtain higher inelastic deformation and a stable hysteretic ductile response.

It can be seen in Fig. 6(a) that cyclic loading made no significant effect on the capacity of the 34 MPa (5 ksi) test units. However, cyclic loading, or degradation, had an increasing effect on the limit to web crushing with increasing concrete compressive strength. Thus, the gains in forestalling web crushing with higher concrete compressive strength were curtailed by the greater susceptibility of higher strength concrete to damage under cyclic loading, as the higher concrete strength results in more brittleness and less energy-dissipation capacity from the materials. This explains the reasons for the comparable ductility levels achieved by walls M15C and M20C. Thus, the shear stiffness degradation and damage of HSC structural walls under load reversals has to be evaluated appropriately. Nonetheless, comparison of the response of walls M15M and M20M shows that M20M had a larger deformation capacity, which supports the tendency that higher the concrete strength can increase the inelastic capacity of structural walls with confined boundary elements.

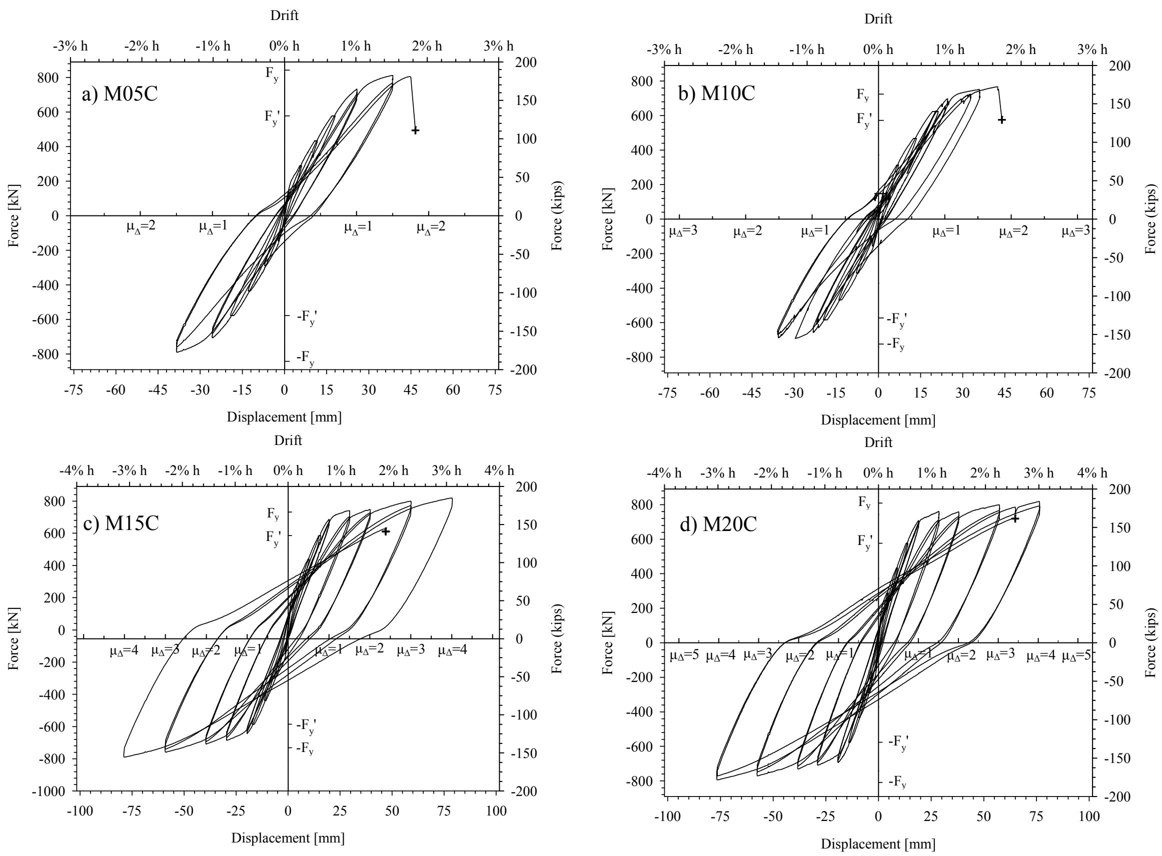


Figure 5. Hysteretic loops of the four test unit under cyclic loading.

Discussion

Fig. 7 shows analytical force-displacement responses (Hines et al. 2004) and the web crushing predictions of the test units with nominal concrete strengths. The web-crushing capacity model by Hines and Seible (2004) along with the ACI code limit is shown. It can be seen that the web crushing capacity is predicted to increase dramatically with an increase of concrete strength. Conversely, the force-displacement responses are essentially unchanged as the flexural response is dictated by the longitudinal reinforcement in walls' the boundary elements. Fig. 8 compares the experimental hysteretic and monotonic force-displacement responses with the predictions. It should be noted that ACI shear provisions (Eq. 4) considerably under-estimate the web crushing strength. At the same time, the prediction quality of the model by Hines and Seible deteriorates with the increase of concrete strength since the model was calibrated on wall tests with normal strength concrete and the strength and stiffness degradation of HSC was not taken into account. Thus, the experimental program revealed that rational web crushing models like the one by Hines and Seible need further considerations to be applicable to HSC structural walls.

The experimental program has clearly demonstrated the hypothesis that compressive strength can forestall web crushing failures and allow the system to attain larger levels of inelastic deformation, thus obtaining an inelastic flexure-shear response or a ductile shear failure. It is clear that the shear stress demands on the walls are well in excess of currently prescribed limits. However, the experiments revealed that while HSC enables the shear-carrying compressive struts to be stronger, and thus sustain higher effective wall shear stresses, the effect damage accumulation due to cyclic loading on HSC needs to be carefully evaluated.

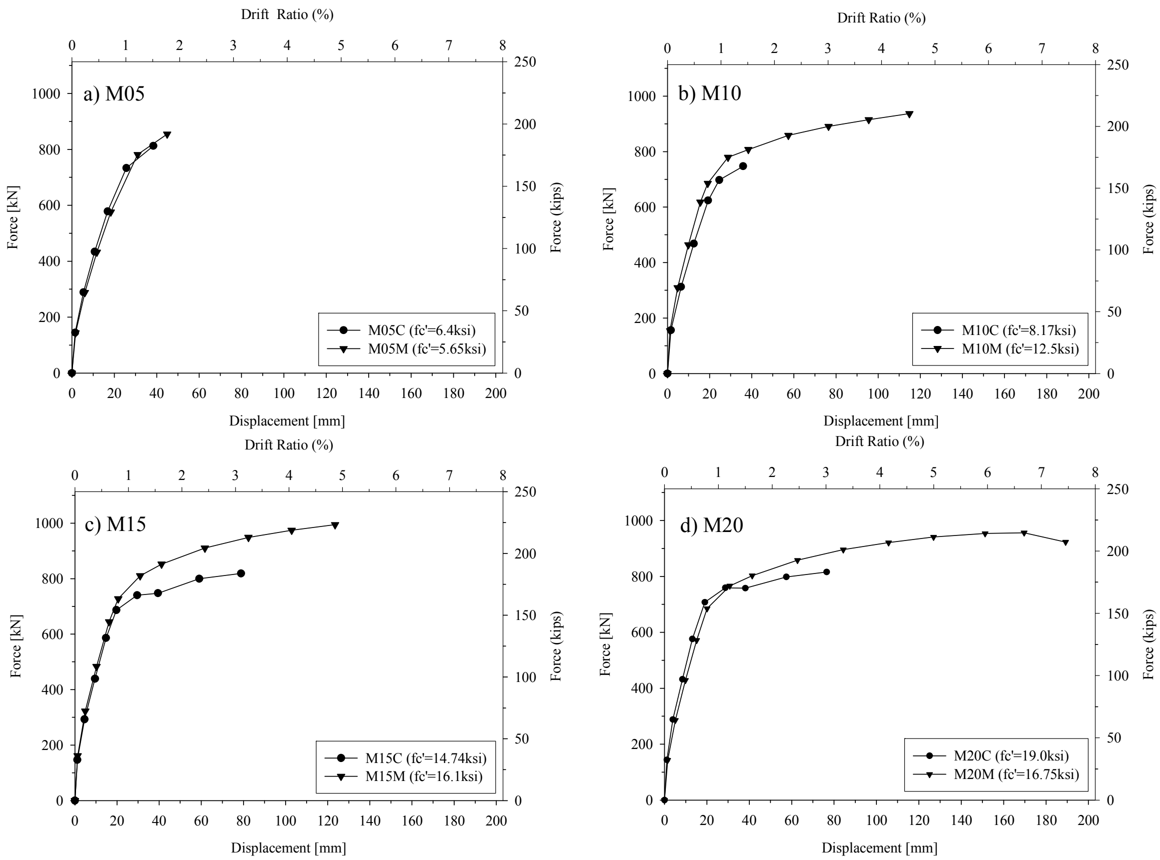


Figure 6. Comparison of the force-displacement envelopes under monotonic and cyclic loading.

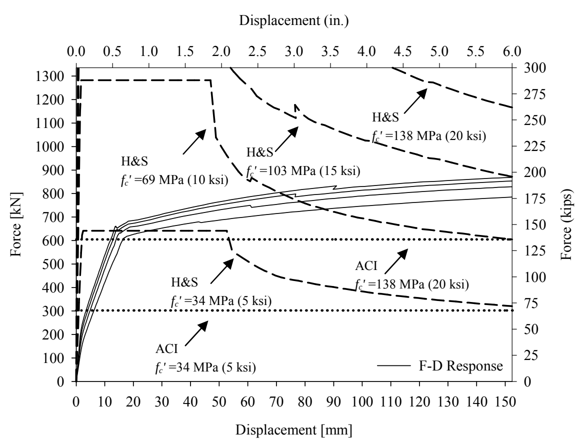


Figure 7. Analytical force-displacement (F-D) response with predictions.

An initial evaluation was done by considering model by Park and Ang (1985) to evaluate the damage on the test units due to ultimate deformation and hysteretic energy dissipation. The damage indices at web crushing are shown in Table 2 for the four cyclic test units. It can be seen that with the increase of concrete strength, the damage due to ultimate deformation decreases while the energy dissipation damage increases. Thus, using HSC increases the energy dissipation capacity and ductility of the structural walls as it has been verified by the tests. However, it can be noted that the damage indices remain relatively unchanged for test units M15C and M20C, indicating that no additional inelastic deformation capacity was gained by increasing the concrete strength between these two test units. This effect is attributed to the decrease in fracture toughness of the concrete with compressive strength, which curtails the capacity of the shear-carrying struts. Further studies on this aspect are ongoing and are considered fundamental in establishing limits to the inelastic web crushing behavior being characterized in this study.

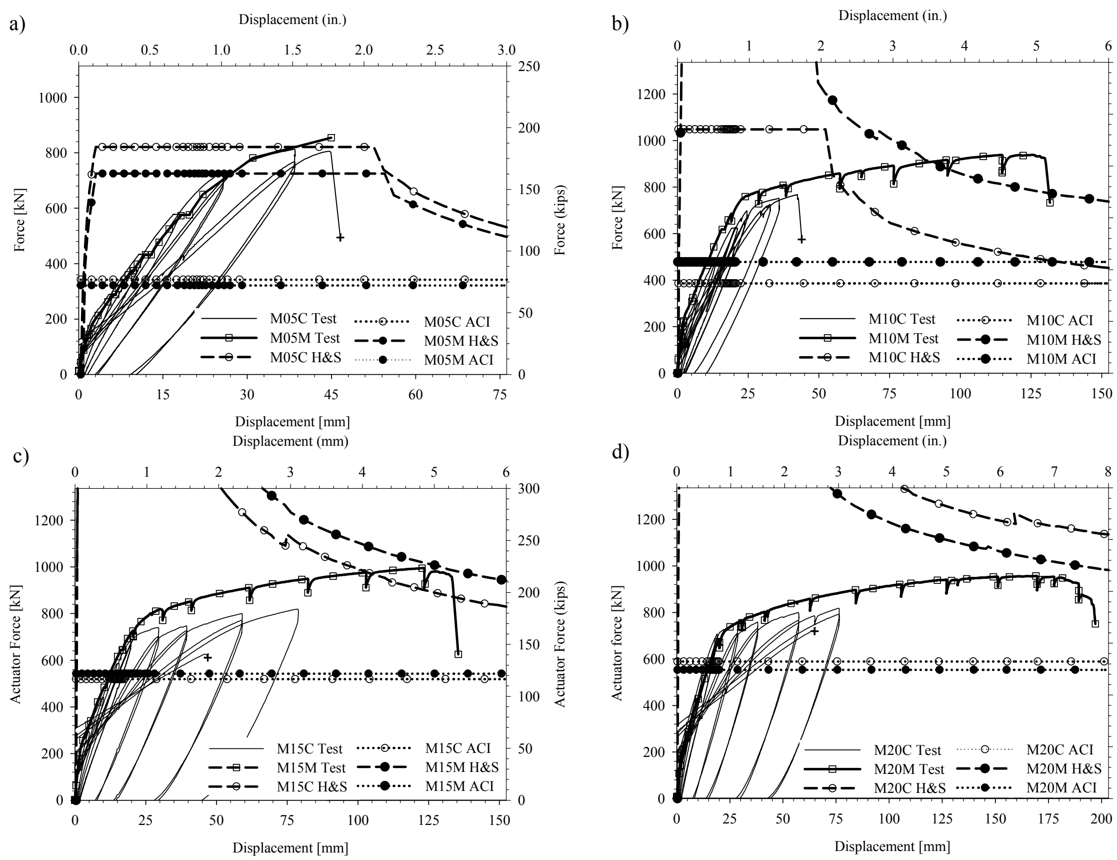


Figure 8. Comparison of the experimental hysteretic and monotonic force-displacement behavior with web-crushing capacity curves.

Table 2. Damage indices of test units under cyclic loading.

Damage	**M05C**	**M10C**	**M15C**	**M20C**
Deformation	0.85	0.53	0.41	0.40
Energy dissipation	0.15	0.47	0.59	0.60

Conclusions

Eight cantilever walls were tested with design concrete compressive strengths of 34, 69, 103, and 138 MPa [5, 10, 15 and 20 ksi] under cyclic and monotonic loading to study the effect of high-strength-concrete (HSC) and load reversal on the inelastic web-crushing capacity of structural walls. Two conclusions are offered based on the presented work and findings to date. First, high-strength-concrete (HSC) can effectively delay web-crushing shear failures in structural walls thus allowing the system to attain stable inelastic force-displacement response before failure. This response is possible by the strength gained by the shear-carrying concrete struts and their anchorage into well-confined boundary elements, which in turn govern the inelastic flexural response of the system. The result is a stable and dependable ductile response, which supports the research hypothesis and the possibility of accepting what the authors name 'ductile shear failures' as acceptable inelastic failure mechanisms for seismic design. Second, comparison of the monotonic and cyclic test results reveal that cyclic loading significantly curtails the compression capacity of the inclined shear-resisting struts in HSC walls. Such effect is attributed to the lower fracture toughness of HSC, which leads to rapid shear strength and stiffness degradation. In the reported study, such behavior was most noticeable for concretes with compressive strength over 103 MPa (15 ksi). This aspect is essential in establishing dependable limits to the inelastic web crushing capacity of structural walls.

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