AN INVESTIGATION OF THE DESIGN AND REPAIR OF LOW-RISE SHEAR WALLS

by

F. Barda, J. M. Hanson and W. G. Corley[I]

SYNOPSIS

Background information on design provisions for low-rise shear walls is presented. Results of eight tests on approximately one-third scale walls with boundary elements are briefly summarized. The findings indicate that (1) severe reversals of loading reduce ultimate shear by about 10%, (2) vertical wall reinforcement is more effective than horizontal reinforcement in resisting shear, (3) boundary elements enhance the post-ultimate load carrying characteristics, and (4) simple repairs provide walls with adequate strength and with improved capability for energy absorption.

BACKGROUND

The development of design provisions for shear walls in the United States is discussed in companion papers.(1,2) These design provisions consider that ultimate shear stress is made up of two parts. A portion of the shear is carried by the concrete while the remainder is carried by reinforcement.

For low-rise shear walls, it is of interest to note that less shear was attributed to the concrete in design by the 1971 ACI Building Code (3) than by provisions in effect at that time in the Uniform Building Code.(4) The UBC provisions were recently revised and will be available for use in 1973.

For walls with minimum reinforcement and with a height-to-horizontal length ratio, h_w/ℓ_w, of one or less, the UBC provisions assumed that the ultimate shear stress carried by the concrete was $5.7\sqrt{f'_c}$.[II] They also did not allow any additional contribution to shear strength from web reinforcement. The ACI provisions limit the ultimate shear stress carried by the concrete to $3.3\sqrt{f'_c}$, if it is considered that the effect of axial stress in low-rise walls is negligible. However, they allow the ultimate shear stress to be increased to $10\sqrt{f'_c}$ by the addition of sufficient equal amounts of horizontal and vertical web reinforcement.

There were several reasons for the changes in approach used in the ACI provisions. Among these was concern that the contribution of the concrete to shear resistance may be affected both by the manner in which the shear forces are applied and by the effect of severe reversals of loading. The ACI provisions also reflected evidence that web reinforcement was effective in deep members, although the respective contribution of either the horizontal or vertical bars was uncertain.

[I] Former Associate Research Engineer, former Assistant Manager, and Manager, respectively, Structural Development Section Portland Cement Association, Skokie, Illinois.

[II] Where f'_c is in psi. $\sqrt{f'_c}$ English equals $0.265\sqrt{f'_c}$ Metric.

OBJECTIVE AND SCOPE

To obtain experimental information on the strength and serviceability of low-rise shear walls, the Portland Cement Association recently conducted a series of eight tests on approximately one-third scale specimens.(5) These specimens, with height-to-horizontal length ratios between 1/4 and 1, were constructed with boundary elements representing walls or columns at each end, a floor or roof slab on top and a foundation at the base. The amount of reinforcement in the end walls varied from 1.8 to 6.4% of its areas. Both the horizontal and vertical reinforcement in the shear walls were varied from 0 to 0.5% of the wall area. Six of the eight specimens were subject to a continually increasing pattern of load and deflection reversals up to and far beyond their ultimate load. After this severe loading, one of the specimens was repaired and retested. This paper briefly describes the experimental investigation, summarizes the findings, and presents initial recommendations for the design of low-rise walls.

EXPERIMENTAL INVESTIGATION

Test Specimen - Details of the test specimen are shown in Fig. 1. It consists of a wall integral with vertical end boundary elements that represent either a cross wall or column. The concrete slab at the top represents a floor or roof slab framing into the wall. The large, heavily reinforced foundation base was prestressed to the laboratory test floor.

Design of each specimen was based on concrete having a compressive strength of 3000 psi at 28 days and reinforcement with a yield stress of 60,000 psi. Dimensions and reinforcement of the eight specimens are listed in Table 1. Measured properties of the concrete and reinforcement are summarized in Table 2.

Method of Loading - All but two specimens were subjected to reversals of loading. Specimens B1-1 and B2-1 were subjected to an unidirectional loading. Loading on all specimens continued until a lateral deflection of about 3 in. had been reached. The application of load reversals was intended to represent a severe seismic loading. To make comparisons of strength, a systematic pattern of increasing stress or deflection was followed, as illustrated in Fig. 2. At each level, the load was cycled twice.

After reaching ultimate, the specimen was displaced until a desired deflection was obtained. At this deflection, and at each increasing increment of deflection, the specimen was cycled twice in a manner that displaced it to approximately equal displacements in each direction.

Principal Test Results - The principal test results are presented in Fig. 3. Shear stress, v, was calculated as V/hd, where V = the applied shear force, h = the thickness of the web, and d = the effective depth, defined as the distance from extreme compression fiber to the centroid of the tension reinforcement. Table 1 lists the effective depth for each specimen.

In most specimens the first observed cracking occurred in the lower portion of the web, near the flange closest to the applied load. Generally one or two very short cracks inclined at about 40 degrees were found. This cracking was probably influenced by residual tensile stresses in the web. The cracking occurred at nominal shear stresses between 110 and 230 psi. Its development did not noticeably change the deflection and strain measurements.

At a higher stress, one or more long inclined cracks occurred suddenly in previously uncracked concrete. Development of inclined cracks significantly affected the measurements. This occurrence is referred to as first shear cracking.

Figure 3 shows the nominal value of $v_{cr}/\sqrt{f'_c}$ for each specimen. Except for Specimen B3-2R, a retest of B3-2 after it was repaired, the range of $v_{cr}/\sqrt{f'_c}$ for the specimens with an h_w/ℓ_w of 1/2 was from 4.9 to 6.5. Values of $\Delta\ell_w/h_w$ ranged from 0.00032 for B5-4 and B7-5 to 0.00072 for B1-1.

At load levels near ultimate, the two specimens subjected to load in one-direction exhibited a uniform pattern of parallel inclined cracks. Except for B5-4, specimens subjected to load reversals, exhibited a nearly orthogonal pattern of cracks. The cracks caused by load in one-direction had about the same spacing, width, and inclination as the cracks caused by reversed load. The average crack inclinations for all of these specimens ranged between 37 and 43 degrees.

A series of web struts were formed by the development of the parallel cracks. Load reversals did not seem to adversely affect the behavior of the struts or the transfer of shear from the top slab through the struts to the base.

Specimen B5-4 contained no vertical web reinforcement. The crack pattern for this specimen was irregular. At comparable loads, inclined crack widths were larger for B5-4 than for any other specimen.

Specimen B4-3 contained no horizontal web reinforcement. The crack pattern for this wall was not as regular as in the specimens containing horizontal web reinforcement. Furthermore, at comparable loads, the inclined crack widths in B4-3 were about twice as large as in a similar specimen containing horizontal web reinforcement.

Figure 3 shows the nominal shear at ultimate, $v_u/\sqrt{f'_c}$. The value of $v_u/\sqrt{f'_c}$ ranged from 8.3 to 15.8. The value of $\Delta\ell_w/h_w$ ranged rom 0.0053 to 0.0130. Except for B3-2R, the range of $\Delta\ell_w/h_w$ for the specimens with h_w/ℓ_w of 1/2 ranged from 0.0053 to 0.0069.

Figure 3 also shows the effect of principal variables on v_u and v_{cr}. In Fig. 3(a) the effects of the amount of flange reinforcement and the method of loading are shown. These three specimens contained 0.5% horizontal and vertical web reinforcement. Their height-to-horizontal length ratio was 1/2. In comparing the two specimens subjected to loading in one-direction, it can be seen that the amount of flange reinforcement had little effect on the shear strength. The specimen subjected to load reversals, simulating seismic loading, exhibited a shear strength about 10% lower than that of comparable specimens subjected to loading in one-direction.

The effect of the amount of horizontal web reinforcement is shown in Fig. 3(b). The specimens compared contained 4.1% reinforcement in the flanges, and 0.5% vertical web reinforcement. Their height-to-horizontal length ratio was 1/2. As can be seen, the amount of horizontal web reinforcement had little effect on the shear strength.

Figure 3(c) shows the effect of the vertical web reinforcement. The specimens compared contained 4.1% reinforcement in the flanges, and 0.5% horizontal reinforcement in the web. Their height-to-horizontal length ratio was 1/2. It may be observed that the amount of vertical web reinforcement had a significant effect on the shear strength.

The effect of the height-to-horizontal length ratio is shown in Fig. 3(d). The specimens compared contained 4.1% reinforcement in the flanges. In the web, 0.5% vertical and horizontal reinforcement was used. As shown for the specimen with the largest h_w/ℓ_w ratio exhibited the lowest v_u and v_{cr}.

In two specimens, B1-1 and B8-5, the flexural reinforcement in the flanges was just sufficient to develop the shear capacity before yielding would have occurred. For these specimens, the measured tensile force in the flange reinforcement at the base of the wall was approximately equal to that calculated by assuming that strains in the reinforcement and concrete are proportional to the distance from the neutral axis.

In the remaining specimens, the stress in the flange reinforcement remained considerably below yield even at the ultimate loads. For these specimens, the tensile force was always greater than the computed force. In the tests on B3-2 and B7-5, the measured force was 15% and 68% greater, respectively, than the computed force at the ultimate load. Apparently the high forces are caused by a shift that places the resultant compressive force in the web. This shift is caused by the development of high compressive forces in the web struts.

The tests were concluded after pushing the specimens to a maximum deflection of about 3 in. Corresponding values of nominal shear stress at the end of the test, $v_m/\sqrt{f'_c}$ ranged from 2.6 to 5.7. As anticipated, the tallest specimen, B8-5, had the smallest $v_m/\sqrt{f'_c}$. For the specimens with h_w/ℓ_w of 1/2, $v_m/\sqrt{f'_c}$ ranged from 3.0 to 5.5.

The analysis of the data verified that prior to ultimate, the shear force was transmitted from the top slab to the base through compressive struts in the web. At the top of the web, the compressive force was concentrated at the applied load side. At the base of the web, it was concentrated at the far side. Therefore, it appeared that the shear was transmitted from the top to the base by truss or lattice type behavior.

At the ultimate load, four specimens exhibited some form of distress at the top construction joint. These specimens were B1-1, B3-2R, B5-4 and B7-5. The distress was generally characterized by the development of very short inclined cracks across the joint. This was followed by development of interconnecting horizontal cracks. In some cases, horizontal movement was observed during loading.

With B1-1, the first test specimen, the distress may have been due to inadequate roughening of the joint during construction. However, this specimen contained the smallest amount of flange reinforcement, a factor that also may have contributed.

In the process of repairing B3-2R, concrete was packed by hand to the underside of the top slab. Consequently, contact between the web and the top slab was not as good as in other specimens. This caused a plane of weakness at the joint.

Specimen B5-4 did not contain any web reinforcement across the construction joint. Therefore the joint was not expected to be as effective as in the other specimens. Specimen B7-5 was the shortest specimen.

For the specimens in which distress was observed at the top construction joint, the load-carrying capacity beyond ultimate decreased more slowly than for the other specimens. Beyond the ultimate load, web crushing and spalling occurred in the vicinity of the construction joint near the side where the load was applied. In general, the joint distress appeared to provide additional energy absorption, a desirable situation for earthquake resistant walls.

With B2-1, B3-2, B4-3, B6-4 and B8-5, crushing and spalling started in the web, generally at the location of a vertical bar. This crushing was apparently due to the eccentric compressive forces that were transmitted through the struts. Crushing and spalling extended in a horizontal plane located within the middle one-third of the height of the specimen. Little distress was observed at the top construction joint. For these specimens, the load carrying capacity decreased more rapidly beyond the ultimate load.

In all specimens, the top slab was observed to deflect upward during application of loadings prior to and at ultimate. This upward movement was associated with web cracking and subsequent yielding of the vertical web bars. It was evident that the top boundary condition had a significant effect on the behavior. Under loadings after ultimate, the top slab regained its original undeflected shape.

FINDINGS

The shear strength of the specimens was not affected by differences in the amount of flexural reinforcement.

A nearly orthogonal pattern of cracking developed in the specimens subjected to load reversals. This cross-cracking did not detrimentally affect the behavior of the specimens. A specimen subjected to load reversals had a shear strength about 10% less than a similar specimen subjected to loading in one-direction.

For the specimens with a height-to-horizontal length ratio of 1/2 and less, the horizontal wall reinforcement did not contribute to the shear strength. It was effective in producing a distributed crack pattern and in controlling crack widths. For design, minimum horizontal reinforcement should be provided in all walls.

The vertical wall reinforcement was more effective as shear reinforcement in the specimens with a height-to-horizontal length ratio of 1/2 and less than in the specimen with a height-to-horizontal length ratio of 1. However, in both cases it appeard to contribute to the shear strength. It was also effective in producing a distributed crack pattern and in controlling crack widths. For design, minimum vertical reinforcement should be provided in all walls.

The restraint of the upper boundary appeared to have a significant effect on the shear strength of the specimens with a height-to-horizontal length of 1/2 or less. This suggests that the behavior of piers and spandrels would differ from low-rise walls.

The shear strength of a specimen with a height-to-horizontal length ratio of 1/4 was not significantly higher than the shear strength of a comparable specimen with a height-to-horizontal length ratio of 1/2.

The shear strength of a specimen with a height-to-horizontal length ratio of 1 was about 20% lower than the shear strength of comparable specimens with height-to-horizontal length ratios of 1/2 and 1/4.

Slip or distress at construction joints may have somewhat reduced the strength of a few walls. This same behavior was observed for the repaired test specimen. However, slip appeared to absorb energy and produced better behavioral characteristics after ultimate.

The behavior of the specimens was found to be similar to that of deep beams and corbels. Walls without shear reinforcement had a shear strength above the stress associated with first shear cracking. Application of the load through the top boundary element did not appear to influence the results.

Load-carrying capacity beyond ultimate depends mainly on the ability of the boundary elements to act as a frame. Undamaged wall concrete offers restraint to the boundary elements. In these tests, the mode of failure was gradual rather than sudden and catastrophic.

RECOMMENDATIONS FOR DESIGN AND REPAIR

Design - For walls with a height-to-horizontal length ratio, h_w/ℓ_w, of one or less, it is recommended that the shear stress carried by the concrete, v_c, be not greater than

$$v_c = 8\sqrt{f'_c} - 2.5\sqrt{f'_c}\,\frac{h_w}{\ell_w} \qquad .....(2)$$

The shear stress carried by the wall reinforcement v_s, shall be computed

$$v_s = \rho\, f_y \qquad .....(3)$$

but not greater than $v_s = 6\sqrt{f'_c}$, where ρ is the lesser of the amount of wall reinforcement in either the vertical or horizontal direction, and ρ shall not be less than 0.0025.

Repair - A shear wall damaged due to extreme deformation was effectively repaired by removing and recasting loose and spalled concrete. In this investigation, the shear strength of the repaired specimen was 20% less than that of the new wall. However, its strength was still higher than that implied for undamaged walls by current design procedures (3). Similarly, the energy absorption of the repaired wall was higher.

REFERENCES

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3. ACI Committee 318, "Building Code Requirements for Reinforced Concrete" (ACI 318-71), American Concrete Institute, Detroit, 1971, 78 pp.

4. Uniform Building Code, International Conference of Building Officials, Pasadena, California, 1967 and 1970 Editions.

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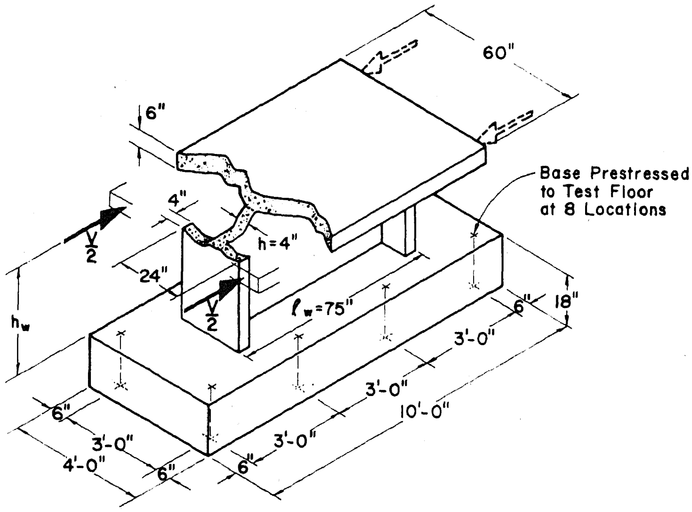
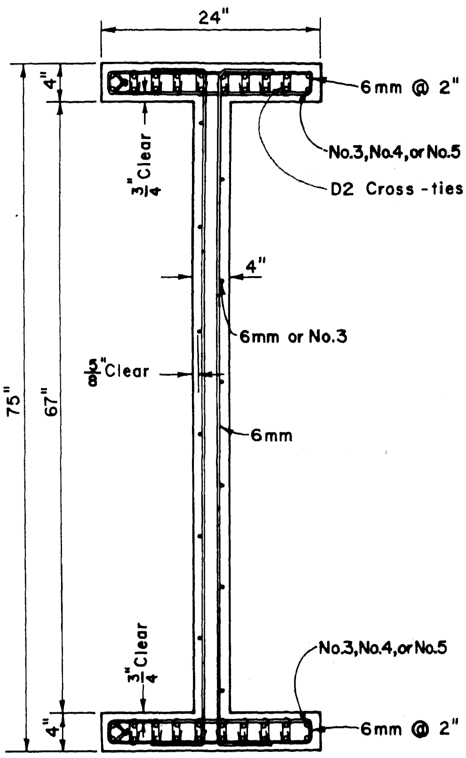
METRIC EQUIVALENTS

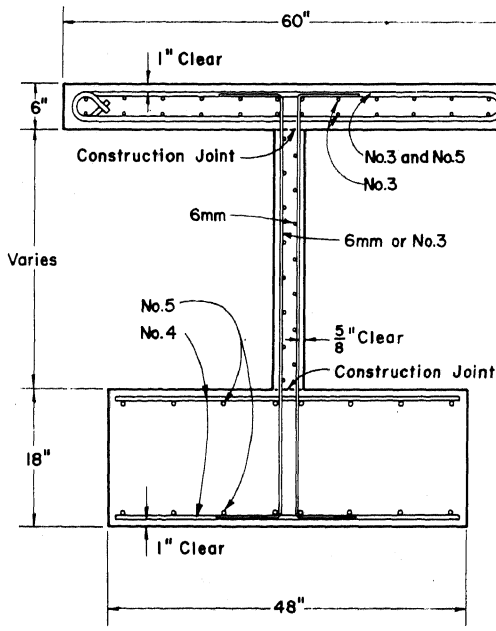
Pound-Foot-Second	Meter-Kilogram-Second
1 in.	2.54 cm
1000 psi	70.3 kg/cm^2
$\sqrt{f'_c}$	$0.265\sqrt{f'_c}$



Isometric View



Vertical Section



Horizontal Section

Fig. 1 Details of Test Specimens

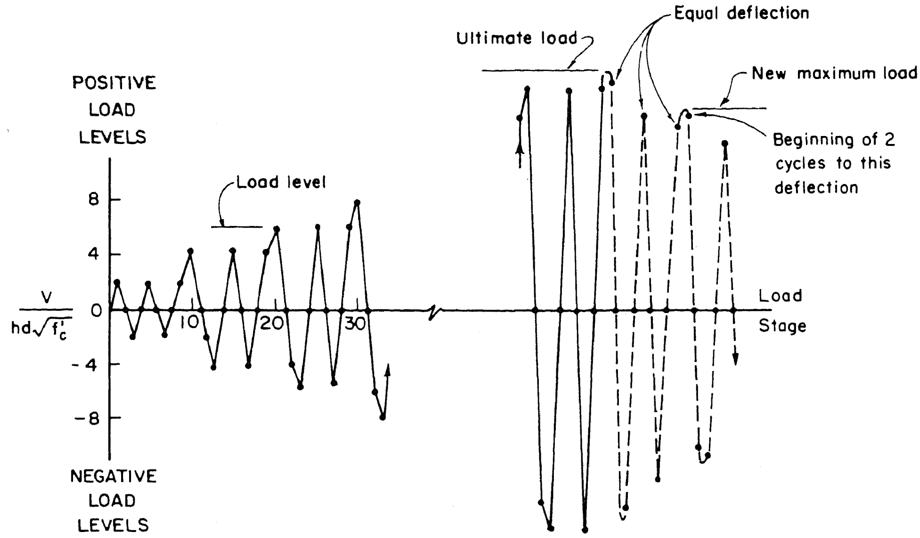


Fig. 2 Method of Loading

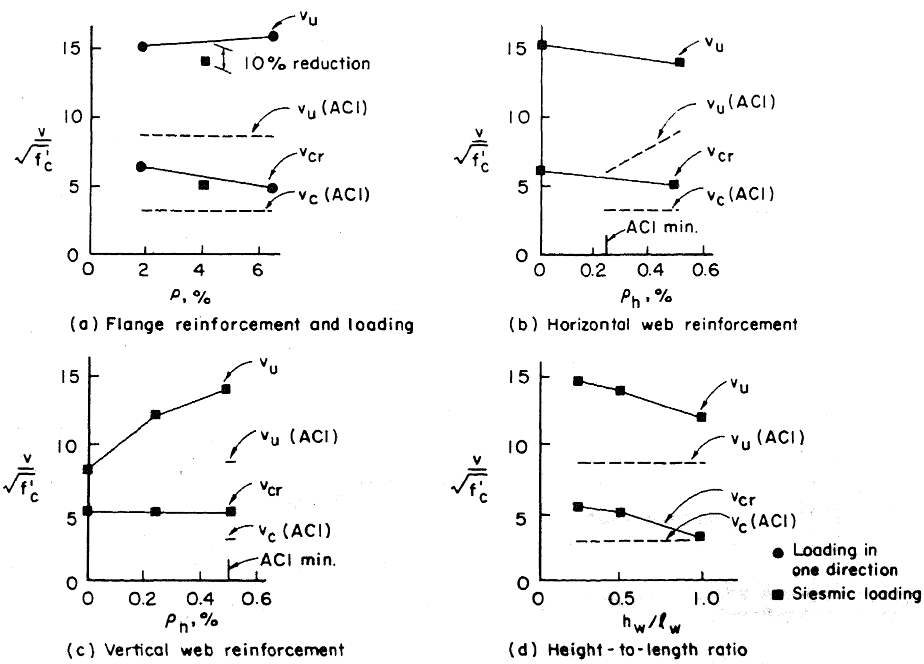


Fig. 3 Effect of Principal Variables

TABLE 1 - DIMENSIONS AND REINFORCEMENT OF SPECIMENS

Specimen	Height h_w in.	$\frac{h_w}{\ell_w}$	Effective Depth d in.	Reinforcement: Flange No.-Size	Flange Reinf. ratio ρ	Web Vertical No.-Size	Web Vertical Reinf. ratio ρ_n	Web Horizontal No.-Size	Web Horizontal Reinf. ratio ρ_h
B1-1	37.5	1/2	67.8	16-No.3	0.018	12-No.3	0.005	15 - 6mm	0.005
B2-1	37.5	1/2	71.7	20-No.5	0.064	12-No.3	0.005	15 - 6mm	0.005
B3-2	37.5	1/2	70.7	20-No.4	0.041	12-No.3	0.005	15 - 6mm	0.005
B4-3	37.5	1/2	70.6	20-No.4	0.041	12-No.3	0.005	None	0
B5-4	37.5	1/2	73.0	20-No.4	0.041	None	0	15 - 6mm	0.005
B6-4	37.5	1/2	71.8	20-No.4	0.041	16 - 6mm	0.0025	15 - 6mm	0.005
B7-5	18.75	1/4	70.7	20-No.4	0.041	12-No.3	0.005	7 - 6mm	0.005
B8-5	75.0	1	70.7	20-No.4	0.041	12-No.3	0.005	33 - 6mm	0.005

TABLE 2 - PROPERTIES OF CONCRETE AND REINFORCEMENT

Specimen	Wall: Age at test days	f'_c psi	f_{ct} psi	$\frac{f'_{ct}}{\sqrt{f'_c}}$	E_c million psi	Reinforcement: Flanges[1] Yield Stress f_y ksi	Flanges[1] Ult. Stress f_u ksi	Web Vertical[4] Yield Stress f_y ksi	Web Vertical[4] Ult. Stress f_u ksi	Web Horizontal[5] Yield Stress f_y ksi	Web Horizontal[5] Ult. Stress f_u ksi
B1-1	35	4200	510	7.9	3.4	76.2[2]	118.7[2]	78.8	123.1	71.9	98.6
B2-1	29	2370	320	6.6	2.5	70.6[3]	119.0[3]	80.0	126.5	72.4	97.0
B3-2	27	3920	470	7.5	3.4	60.0	96.7	79.0	123.3	74.4	97.5
B3-2R*	7	3410	370	6.3	2.6	-	-	-	-	-	-
B4-3	29	2760	370	7.0	2.7	76.5	117.2	77.6	119.6	-	-
B5-4	28	4190	530	8.2	3.4	76.4	117.0	-	-	71.8	98.3
B6-4	27	3080	410	7.4	3.1	76.7	116.3	72.0[5]	94.8[5]	72.0	94.8
B7-5	26	3730	470	7.7	3.3	78.2	115.1	77.0	120.4	72.7	96.0
B8-5	26	3400	430	7.4	3.3	70.9	112.8	76.5	109.5	71.9	97.9
Average		3450	430	7.3	3.1	73.2	114.1	77.3	116.7	72.4	97.2

Notes:

[1]No. 4 bars unless otherwise noted

[2]No. 3 bars

[3]No. 5 bars

[4]No. 3 bars unless otherwise noted

[5]6mm bars

*Repaired Specimen B3-2