CHAPTER II

EXPERIMENT

In order to study the influence of diagonal web reinforcement and axial load, six RC structural wall specimens were tested under cyclic loading. The parameters included the amount and orientation of web reinforcement and axial load level. To study shear behavior, an aspect ratio of 1.5 was selected for all specimens. Furthermore, the specimens were designed to resist a high shear stress level around $0.85\sqrt{f_c'}$ (MPa), which is higher than the upper limit provided by ACI318-05 provision (2005) by 28%. The aim was to explore the advantages of diagonal web reinforcement in suppressing web crushing.

2.1 Test Specimens

The dimensions of the test specimens and the reinforcement details are shown in Figs. 2.1 and 2.2, respectively. The specimens had a barbell-shaped cross section with a web thickness of 130 mm and 250 mm by 250 mm boundary columns. The overall length of the cross section was 1500 mm. On the top of the specimen, a 250-mm wide by 250-mm deep transfer beam was cast monolithically with the wall. A construction joint was located between foundation and wall. This joint was roughened, using a chisel, before placing the concrete for the walls.

The reinforcement details and material properties are summarized in Table 2.1. The specimen nomenclatures are such that the letters C and D denote conventional and diagonal reinforcement configurations, respectively, and the numbers designate the spacing of the web reinforcement. The longitudinal and transverse reinforcements in the boundary elements were the same in all specimens. Eight 16-mm deformed bars were used as the longitudinal reinforcement in each boundary element (longitudinal reinforcement ratio = 2.57%). The transverse reinforcement satisfied the ACI318-05 seismic design provisions for boundary elements in walls (ACI318-05 2005). The hoops were fabricated using 10-mm deformed bars and were spaced at 60 mm on center over the lower 1500 mm of the specimen and at 150 mm on center over the remaining higher section of the specimen.

The web reinforcement in specimen WC150 was arranged in the horizontal and vertical directions. The amount of reinforcement provided was such that the nominal shear strength, computed in accordance with the ACI318-05 provision against diagonal tension failure, slightly exceeded the shear associated with the development of the nominal moment strength of the wall. The web reinforcement was distributed in two curtains of 10 mm deformed bars spaced at 150 mm on center in both the horizontal and vertical directions (ratio of web reinforcement area to gross concrete area, $\rho_n = \rho_v = 0.8\%$). To study the influence of the orientation and the amount of the web reinforcement on the shear response, specimens WD150, WD170 and WD200 were constructed with 0.8%, 0.7% and 0.6% web reinforcement, respectively. All web reinforcement in these specimens was arranged in diagonal directions at 45 degrees.

The primary difficulty of constructing walls with diagonal web reinforcement is that many different lengths of bars are required. A considerable amount of time was spent calculating the length and cutting each bar. Fabricating a wall with conventional web reinforcement is much easier from this perspective. Because it is not convenient to arrange web reinforcement in diagonal directions, a compromise was proposed for specimen WCD170, where diagonal web reinforcement was used over the lower third of the wall height and conventional web reinforcement was used over the remaining higher section of the wall. The amount of diagonal and conventional web reinforcement in this specimen was 0.7%. The combination of diagonal and conventional web reinforcement helped considerably in reducing the time needed to lay out the web reinforcement. The vertical web reinforcement was spliced to diagonal web reinforcement as shown in Fig. 2.3.

The specimens mentioned above were loaded with an axial load of 7% of the axial load capacity based on concrete ultimate strength f_c' and gross cross sectional area A_g. In order to study the effect of axial load, specimen WD170A that had the same amount of diagonal web reinforcement as specimen WD170 was tested with twice the axial load.

2.2 Test Setup and Procedure

The overview of the test setup is shown in Fig. 2.4. The vertical uniform load was simulated by means of a three-point force system with the forces distributed to the cross-section through the top transfer beam. Three hydraulic jacks, all connected to the same pump to maintain a constant axial force, were used to apply the vertical loads at the loading points. The axial load did not vary by more than 5% during each test. The lateral force was applied to the specimen by a calibrated actuator attached to the reaction wall. The specimen was fixed to a rigid testing floor at its base and braced laterally at the level of the transfer beam in order to prevent out-of-plane distortion.

All specimens were subjected to a cyclic lateral displacement history as shown in Fig. 2.5. The displacement history consisted of several stages and each stage consisted of three loading cycles. During the first stage, the specimen was pushed (pulled) under load control until the first cracking of the concrete was observed. In subsequent stages, the specimen was loaded under displacement control to integer multiples of the measured first yield displacement until failure. The first yield displacement, Δ_{y1}, was determined by monitoring the strain gages attached in the outermost tensile reinforcement at the base of the wall. During the first loading cycle to the first yield displacement state in the positive direction, the displacement at which the maximum strain reached the yield strain of the reinforcement was taken as the first yield displacement. This displacement was used as the basis for all subsequent loading cycles in both the positive and negative directions. During each stage of loading, the wall was subjected to three cycles to the same displacement level.

It should be noted that the first yield displacement adopted in this study provides convenience in defining the loading history in the testing. The effective yield displacement, however, should be based on an elastic- plastic idealization of the entire force-displacement diagram, the yield point being defined by the intersection of the elastic segment and the yield plateau representing the calculated flexural load (Paulay et al. 1982). Two main testing protocols have been adopted in cyclic tests of walls. Testing with multiples of the effective yield displacement as introduced by Paulay et al. (1982) has been widely adopted. However, another commonly used procedure is to cycle displacements at multiples of some specific drift ratio, typically in the range of

0.1% to 0.5% (Oesterle et al. 1976, 1979, Iliya and Bertero 1980, Paulay et al. 1982, Salonikios et al. 1999, 2000, Sittipunt et al. 2001). The procedure used in this study is equivalent to applying cyclic loading at multiples of 0.4% drift ratio.

During testing, lateral displacements, shear deformations, lateral loads and strains in reinforcement were recorded using LVDT's, load cells and strain gages. The instrument signals were recorded by a computerized data acquisition system. Figure 2.6 shows the position of the LVDT's. The total displacement at the top of the wall and the sliding displacement at the base of the wall were measured by LVDT's L1 and L2, respectively. Average shear distortion was obtained from displacements measured along the diagonals of a square located in the lower (LVDT's L3 and L4) and upper (LVDT's L5 and L6) portions of the web. The procedure used to calculate the shear component of the deformation is also shown in Fig. 2.6 (Oesterle et al. 1979). Although the flexural component of the displacement was not measured directly in the experiments, it was calculated by subtracting the displacement components corresponding to diagonal and sliding shear from the total lateral displacement. The consequences of this approximation are that the reported flexural component includes the other effects such as slippage/pull-out of vertical bars from the foundation as well as errors arising from measurements. As will be pointed out in Section 2.3.1, no sign of slippage of vertical bars was observed in the tests.

2.3 Test Results

The test results for each specimen, including cracking load, lateral load and displacement at first yield, peak load, maximum displacement ductility (based on the effective yield displacement), maximum drift and mode of failure, are summarized in Table 2.2. It may be noted from this table that there is a discrepancy of about 10% in the lateral peak loads of specimens WD170 and WCD170 whose material properties were almost the same. The reason for this will be given in Section 2.3.3 "Lateral load capacity".

Important observations obtained from the tests are summarized in the following sections.

2.3.1 Crack patterns and failure modes

The first cracks of all specimens were of flexural type which occurred in the boundary elements. Most flexural cracks and diagonal shear cracks developed during loading to the first cycle of the first yield displacement state and then gradually propagated during subsequent displacement cycles. Figure 2.7 shows the observed crack patterns at drift ratios of 1.2%-1.4% (depending on specimen). More shear cracks were observed in the wall with conventional web reinforcement than in the walls with diagonal web reinforcement. The spacing of cracks observed in the wall with conventional web reinforcement was about 70-80 mm whereas the observed crack spacing in walls with diagonal web reinforcement was approximately equal to the spacing of the diagonal web reinforcement (150-200 mm). The maximum width of the shear cracks, however, was about the same (2.0 mm) for all specimens. This indicates that the diagonal web reinforcement helped transfer shear stress more effectively than conventional web reinforcement.

In the wall specimen with both diagonal and conventional web reinforcement (WCD170), the transition between the two types of reinforcement was located 750 mm above the base. Lap splices were used to transmit stresses between the bars, and many more shear cracks developed in this region compared with the specimens with only diagonal web reinforcement. However, the specimen WCD170 did sustain lateral load until failure without crushing of the concrete in the web.

Figure 2.8 shows photographs of all specimens at failure. The crack patterns and failure characteristics of each specimen are described below.

WC150: Normal small flexural and shear cracks developed up to the drift ratio of 0.9%. Vertical cracks then formed in the boundary element when the lateral load was 815.6 kN at the first loading cycle of the drift ratio of 1.3%. The maximum width of shear crack about 1.5 mm was observed at this loading cycle. A large shear crack then penetrated to the boundary elements at the second loading cycle to this drift level. The maximum width of shear crack was about 3.0 mm at the third loading cycle of the drift ratio of 1.8%. The specimen failed due to web crushing during the first loading cycle to the drift ratio of 2.2%. The wall could sustain a lateral load of only 46% of the peak load during the next loading cycle. Buckling of the longitudinal bars was not observed in this specimen.

WD150: A significant shear crack developed in the boundary element at drift ratio of 0.9%, with a crack width of about 0.5 mm in the third loading cycle of this drift level. The vertical crack at the boundary element was extended in the next loading cycle and then the concrete cover of the boundary elements started to spall off, with complete spalling in the plastic hinge zone observed at the first loading cycle of the drift ratio of 1.5%. Buckling of the longitudinal reinforcement in boundary elements was clearly evident at the completion of the third loading cycle to this drift level. During the first loading cycle to the drift ratio of 1.8%, the wall could only sustain 70% of the peak load. Failure was due to buckling of the longitudinal reinforcement in the boundary elements.

WD200: Vertical cracks were observed in the boundary element at the first loading of the drift ratio of 0.8% and the maximum shear crack width was about 1.0 mm. Shear cracks in the boundary element were observed at the first cycle of the drift ratio of 1.2%. The maximum crack width of shear crack was about 3.0 mm at the conclusion of this drift level. Since the spacing of web reinforcement was the largest in this specimen, the concrete cover in the web spalled near the base and web reinforcement buckled during loading cycles to the drift ratio of 1.6%. Longitudinal bars also buckled at this ductility level but the concrete in the cores of the boundary elements was still confined and was capable of carrying load. Therefore, this specimen failed by buckling of web reinforcement.

WD170: The maximum shear crack width at the drift ratio of 0.8% was about 0.3 mm. The vertical crack in boundary element was observed at the first loading cycle of the drift ratio of 1.2%. A maximum shear crack width at this drift level was 2.0 mm. The maximum shear crack width increased to 4.0 mm at the drift ratio of 1.6%. The concrete cover in the web of this specimen spalled near the base during the second loading cycle to the drift ratio of 1.6%. Consequently, buckling of web reinforcement was observed during the third cycle to this drift level. However, the wall could still sustain 92% of the peak load at this ductility level. Severe buckling of longitudinal reinforcement was observed during the second loading cycle to the drift ratio of 2.0%. The wall could sustain only 60% of the maximum load at this cycle. Therefore, this specimen failed by buckling of the longitudinal reinforcement.

WCD170: More shear cracks developed in the middle third of the specimen in the intersection region of the diagonal and conventional web reinforcement, compared to the specimen with complete diagonal reinforcement. The maximum width of shear crack was about 0.5 mm at the drift ratio of 0.9%. Subsequently, a shear crack formed in the boundary element. The maximum width of the shear crack increased to 5 mm at the drift ratio of 1.4%. Concrete covering in the boundary element spalled off at the second loading cycle of the drift ratio of 1.9%. Buckling of the longitudinal reinforcement was observed at the conclusion of this drift level. The web reinforcement also buckled during the first loading cycle to the drift ratio of 2.4%. During subsequent cycles to this drift level, several longitudinal bars fractured, which led to failure of the specimen.

WD170A: Most of shear cracks and flexural cracks were observed at the drift level of 0.4%. The maximum width of shear cracks was about 0.45 mm at the drift ratio of 0.8% and it increased to 1.25 mm at the drift level of 1.2%. A few vertical cracks were observed in the boundary element at the second loading cycle to the drift level of 1.6% and, subsequently, the concrete cover in the boundary element spalled off at the conclusion of this drift level. The boundary elements were severely damaged at the first cycle of the 2.0% drift level. In addition, buckling of web reinforcement was observed in this cycle. In the second cycle at this level, one of the longitudinal bars fractured. The wall could sustain only 60% of maximum load in the next cycle.

As described above, the wall with conventional web reinforcement failed due to web crushing and experienced an abrupt drop in load capacity, whereas the walls reinforced with diagonal web reinforcement failed in slightly more ductile modes. Buckling of diagonal bars in the web was observed in several of the test specimens due to the high compressive stresses. The subsequent loss of cover concrete in the web could be responsible for the reduced lateral capacity of specimen WD200. Cross ties may be effective in the delaying the onset of buckling of the diagonal reinforcement but would add to the complexity of construction of the walls.

It should be pointed out that in all the tests no sign of slippage of vertical bars was observed. Furthermore, even though the walls were subjected to a high shear stress level, there was no sign of distress due to sliding since the sliding displacements that occurred were small, in the order of 10% to 20% of the total displacement for the walls with diagonal and conventional reinforcement, respectively. It is interesting to note that the experimental peak lateral load of WC150 (890 kN) was larger than the sliding shear capacity computed in accordance with Eurocode 8 (CEN 2004) by as much as 38%. The discrepancy reduced to about 13% for diagonally reinforced walls. It appears that Eurocode 8 gives quite conservative estimates of the sliding shear capacities for walls with boundary elements.

2.3.2 Influence of diagonal web reinforcement

Figure 2.9 shows the cyclic load-displacement relationships. The total displacement at the top of each wall is decomposed into the flexural, shear, and sliding components and depicted in Fig. 2.10. Figure 2.11 plots the displacement components as percentage of the total displacement.

Clearly, the hysteresis loops for wall specimens with diagonal web reinforcement exhibited less pinching than the conventionally reinforced wall specimen. The pinching of the hysteresis loops was most pronounced in specimen WC150 due to the large shear and sliding displacements in the hinging region (see Fig. 2.10(a)). Test results indicate that the diagonal web reinforcement reduces the shear and sliding displacement components. Table 2.3 summarizes the experimental value of the shear displacement as percentage of total displacement at the maximum ductility level attained by each specimen. The shear displacement of specimen WD150 was reduced by 32% compared with that of WC150. However, it should be noted that the concrete strength of specimen WD150 was about 48% higher than that of specimen WC150, contributing to 21% decrease in shear deformation based on the assumption that the shear modulus varies with the square root of the concrete strength. Thus, with the same concrete strength as specimen WC150 the diagonal reinforcement (in specimen WD150) caused a reduction of 18% in shear deformation. This finding is also confirmed by specimen WD170 which experienced 22% decrease in shear deformation in comparison with WC150 (after adjustment for difference in concrete strength) even though the former specimen was reinforced with a smaller amount of web reinforcement (by 12.5 %).

The influence of diagonal web reinforcement on the compressive strain in the concrete strut may be approximately investigated by considering a web element at the base of the wall adjacent to the boundary element (Fig. 2.12). The average shear strain, γ_{xy}, was obtained from the test measurement, and the vertical strain, ε_y, in this element was estimated using sectional analysis. Values of γ_{xy} and ε_y are reported in Table 2.3 for each specimen at peak load (The reported values of γ_{xy} were corrected for the measured concrete compressive strength using the procedures already discussed). As suggested by Mo and Rothert (1997), the lateral strain, ε_x, was assumed to be zero due to the restraint provided by the boundary elements.

The principal compressive strain, ε_d, was then obtained from transformation of strains. The value of ε_d (Table 2.3) can be approximated to be the compressive strain in the concrete strut. It should be observed from Table 2.3 that ε_y was significantly smaller than γ_{xy}. Therefore, ε_d would not be so sensitive to variation in ε_y. The calculated compressive strain in the concrete strut for specimen WD150 was 23% less than the calculated compressive strain for specimen WC150. This result suggests that diagonal web reinforcement contributes to reducing the compressive strain in the concrete strut and hence walls with diagonal reinforcement are less susceptible to web crushing.

It is to be emphasized that the simple analysis presented here is for the purpose of giving some comparative values of the compressive strains in the concrete struts. For a detailed analysis, it is necessary to consider the nonlinear mechanics of the structure through the cyclic loading history. Further research is obviously needed.

It is to be noted that the foregoing treatment does not include specimen WD200 since the transducers failed after buckling of the web reinforcement took place.

2.3.3 Lateral load capacity

The lateral load capacity of a wall may be limited by flexure or shear modes of failure, assuming the base of the wall is adequately anchored to prevent excessive sliding. Generally, the nominal moment strength (M_n) of a wall is calculated by using sectional analysis with the classical Euler – Bernoulli assumption (ACI318-05 2005), although this assumption is incorrect for walls. In this study, the concrete stress-strain relationships proposed by Hognestad (1951) and Saatcioglu and Razvi (1992) were adopted for the unconfined and confined concrete, respectively, and the stress-strain curve for reinforcing steel was modeled with four linear segments representing the initial elastic portion, the yield plateau, strain hardening, and tensile capacity plateau (Dhakal and Maekawa 2002). The contribution of the diagonal web reinforcement to the moment capacity can be obtained based on compatibility and constitutive considerations, with the longitudinal strains transformed to the directions of the diagonal web reinforcement from which the axial forces in the web reinforcement can be determined. These inclined bar forces can then be decomposed into the longitudinal direction for moment computation.

The nominal shear strength of a wall associated with diagonal tensile crack failure (unit in MPa) is specified by the ACI318-05 code (2005) as follows:

$$V_n \leq A_{cv}\left(\frac{\alpha_c}{12}\sqrt{f_c'} + \rho_n f_y\right) \tag{2.1}$$

where α_c is a coefficient varying linearly between 3.0 and 2.0 for aspect ratio ranging from 1.5 to 2.0, and ρ_n is the horizontal web reinforcement ratio. It should be noted that conventional reinforcement is implied in Eq. (2.1). The shear resisted by the inclined web reinforcement can be computed from the general formula (ACI318-05 2005):

$$\begin{aligned} v_s = \frac{V_s}{bd} &= \frac{A_v f_y(\sin\alpha + \cos\alpha)}{sb} \\ &= \frac{A_v f_y(\sin\alpha + \cos\alpha)}{s_d b / \cos\alpha} \end{aligned} \tag{2.2}$$

where A_v and α are the area and inclination of the web reinforcement, respectively. The spacings of web reinforcement in the horizontal and diagonal directions are denoted by s and s_d, respectively. However, the code limits the nominal shear strength to $\frac{2}{3}\sqrt{f_c'}A_{cv}$ to prevent web crushing failure. For α equal to $45°$, Eq. (2.2) becomes

$$v_s = \rho_d f_y \tag{2.3}$$

The computed nominal moment strength, M_n, and the associated lateral shear, $V_{n,f}$, are tabulated in Table 2.4 for the walls tested, together with the shear capacities stipulated by ACI318-05 code.

For specimens WD150, WD170, and WCD170 whose material properties differed very little, the theoretical peak lateral loads based on sectional analyses and mean values of material properties were 957-975 kN. The tested peak load of WD170 was 980 kN, about 90 kN or 10% higher than the other two. An examination of the compressive strength test results of the cylinder test specimens (8 taken for specimen WD170) reveals that the concrete strengths varied by about 18% from the mean value, whereas the variation was around 6% for the yield strength of steel bars. Based on the upper limits of the test results of material strengths, the theoretical peak load would be 1015 kN, or about 55 kN higher than the mean value. The experimental peak load discrepancy of 90 kN is thus acceptable, allowing for an error in measurement of 30-40 kN (in the order of 3-4%).

It may be observed that the peak lateral load capacities obtained from the tests exceeded the upper limit in ACI318-05 for all specimens, suggesting conservatism of the ACI318-05 code provisions. The discrepancy ranges from 21% for WC150 to 7-29% for the rest. It should also be noted that ACI318-05 shear model would predict web crushing mode of failure for all the walls tested, whereas in reality only the wall with conventional web reinforcement failed by web crushing. Clearly, a refined model is needed to determine the average shear stress associated with web crushing for walls with diagonal web reinforcement.

2.3.4 Energy dissipation and viscous equivalent damping ratio

One of the basic requirements of performance-based design is to control damage in the structure during an earthquake. To achieve this, the structural walls must be able to dissipate energy reliably when subjected to cyclic loads. Therefore, a primary aim of this investigation is to determine the arrangement of web reinforcement to minimize the inevitable degradation in both stiffness and strength of the wall and to ensure the desired energy dissipation characteristics.

A convenient way to quantify the energy dissipation characteristics is to determine the area under the load-displacement hysteresis loops. The wall with conventional web reinforcement exhibited more pinching of the hysteresis, and, hence, less energy dissipation, than the walls with diagonal reinforcement. The pinched shape was attributed to shear deformations within the web and sliding of the wall at the base. Figure 2.13 shows the cumulative energy dissipation as a function of the drift ratio. Specimens with diagonal web reinforcement achieved higher energy dissipation capacities than the specimen with conventional web reinforcement.

Alternatively, energy dissipation capacity can be quantified in terms of damping. For a structural component subjected to cyclic loading, the equivalent viscous damping, ξ_{eq}, can be obtained from Eq. (2.5) (Prestley 1996):

$$\xi_{eq} = \frac{E_i}{4\pi E_e} \tag{2.5}$$

where E_e is the elastic strain energy stored in an equivalent linear elastic system when the maximum displacement is reached at cycle i, and E_i is the energy dissipated at cycle i.

Figure 2.14 shows the equivalent viscous damping as a function of drift ratio. At a drift ratio less than 0.5%, the damping ratio was about 0.04 for all specimens. The damping ratio of the wall specimens with diagonal web reinforcement reached 0.2 while that of specimen with conventional web reinforcement was only 0.13, indicating superior damping characteristics of diagonally reinforced walls.

2.3.5 Strain in web reinforcement

Figure 2.15 illustrates the strain responses of diagonal web reinforcement in specimen WD150 and horizontal web reinforcement in specimen WC150. It should be observed that the diagonal web reinforcement experienced alternating cycles of compressive and tensile strains, while the horizontal web reinforcement experienced only tensile strains. It appears that the diagonal web reinforcement resists sliding displacements at the base and shear deformations within the web. The horizontal web reinforcement is much less efficient in resisting these deformations. These observations are consistent with Oesterle et al. (1979) and Sittipunt et al. (2001) who concluded that the amount of horizontal shear reinforcement did not have a significant effect on the shear behavior.

2.3.6 Performance of the wall with mixed web reinforcement

As already described, the performance of the walls with diagonal web reinforcement is clearly superior to the conventionally reinforced wall. However, it is more difficult to lay diagonal bars. The combination of diagonal and conventional web reinforcement is thus proposed to partially alleviate the difficulty in laying the diagonal reinforcing steel.

The shear and sliding components of specimen WCD170 were different from those of specimen WD170 by not more than 2.0% of the total displacement. The energy dissipation capacity of the specimen with mixed types of web reinforcement (WCD170) was also only slightly less than that for specimens with solely diagonal reinforcement (by around 6%). Thus, the combination of types of web reinforcement appears to be quite promising for practice.

2.3.7 Effect of axial load on inelastic behavior of walls

The effect of axial load on inelastic behavior of walls has been found to be significant. Axial load has a favorable effect of reducing the shear deformation and sliding displacement (Oesterle et al. 1979, Salonikios et al. 2000). Consequently, the energy dissipation and hence the damping increases with an increase in axial load. However, axial load reduces the ductility capacity especially at a high levei of axial load. (Oesterle et al. 1979, Zhang and Wang 2000).

In this study, specimens WD170 and WD170A with applied axial load of $0.07f'_cA_g$ and $0.14f'_cA_g$, respectively, were employed to investigate the axial load effect. Contrary to expectation, the test results indicate that the axial load had an insignificant effect on the ductility capacity. Both specimens failed by flexural mode at about the same drift ratio of 2.0%. During the test, the concrete cover in boundary elements of both specimens spalled at the same drift ratio of 1.6% although it was expected that concrete cover of specimen WD170A should have spalled earlier than that of specimen WD170. There was probably no consistency in the concrete properties. Obviously more investigations with more specimens are needed.

2.4 Summary

The conventionally reinforced wall failed due to web crushing with an abrupt drop in load capacity, whereas the walls reinforced with diagonal web reinforcement failed in a more ductile mode. Test results clearly indicate that the diagonal web reinforcement caused a reduction of about 20% in shear deformation (after adjustment for difference in concrete strength) and the estimated peak compressive strain in the concrete strut in specimen WD150 was 23% less than that in specimen WC150. This result suggests that diagonal web reinforcement contributes to reducing the compressive strain in the concrete strut and hence walls with diagonal reinforcement are less susceptible to web crushing. Furthermore, the diagonal web reinforcement reduces the sliding displacement components. The reduction of sliding and shear displacements result in less pinching in hysteresis loops in the specimens with diagonal web reinforcement in comparison with the conventional one. Consequently, the energy dissipation capacity of the former is significantly larger than the latter.

An alternative web reinforcement configuration which combines the superior performance of the diagonal reinforcement and the simplicity of placement of the conventional type is also proposed. Test results reveal that the wall with mixed web reinforcement exhibits performance comparable with the wall with diagonal reinforcement.

The effect of axial load level cannot be concluded from this study and further study is needed.