The allowable shear stress between crack surfaces is defined by the tensile strength of concrete and the ratio of the calculated crack width, Δw, to a critical crack width associated with total loss of aggregate interlock, Δw_u as

$$\tau_{fu} = const \cdot f_{ct}\left(1-\frac{\Delta w}{\Delta w_u}\right) \tag{18}$$

An expression for V_c is obtained by adding the contributions from the uncracked compression zone and the friction force between crack surfaces

$$\begin{aligned} V_c &= 0.4 \cdot f_{ct} \cdot b \cdot kd + 0.4 \cdot f_{ct} \cdot b \cdot (d-kd)\left(1-\frac{\Delta w}{\Delta w_u}\right) \\ \Leftrightarrow V_c &= 0.4 \cdot \sqrt[3]{f'_c}\, bd\left[k+(1-k)\left(1-\frac{\Delta w}{\Delta w_u}\right)\right] \end{aligned} \tag{19}$$

where

$$\Delta w_u = 1.0 \text{ mm} \tag{20}$$

The coefficient of 0.4 in Eq. (19) and the critical crack width value in Eq. (20) were obtained from the calibration of the model carried out by von Ramin and Matamoros (2004 and 2005). The magnitude of the crack width Δw is calculated based on the strain in the longitudinal reinforcement and the average crack spacing s_{cr} as

$$\begin{aligned} \Delta w &= \frac{0.5 \cdot \varepsilon_s \cdot s_{cr}}{\sin 30^\circ\left(1-0.336 \cot 30^\circ\right)} + \frac{0.01 \cdot \cot 30^\circ}{1-0.336 \cdot \cot 30^\circ} \le 1.0 \text{ mm} \\ &\cong 2.4 \cdot \varepsilon_s \cdot s_{cr} + 0.04 \le 1.0 \text{ mm} \end{aligned} \tag{21}$$

For members subjected to point loads, the strain in the reinforcement at a critical distance d from the support is given as

$$\varepsilon_s = \frac{V \cdot d}{\rho_s \cdot bd \cdot jd \cdot E_s} \tag{22}$$

and the critical crack spacing is given by

$$s_{cr} = (d-kd) \tag{23}$$

In reinforced concrete members in which the longitudinal reinforcement yields before failure, the relatively large crack widths in the hinge region make the contribution of the friction component negligible. Consistent with this behavior, when strain in the longitudinal reinforcement is large, Eq. (21) results in crack widths greater than the critical value of 1 mm, and Eq. (19) simplifies to:

$$V_c = 0.4 \cdot \sqrt[3]{f'_c}\, k\, b\, d \tag{24}$$

Because distinct compression and tension zones do not form in "D-regions" the contributions from friction and the compression zone are negligible in deep members. The effect of these components becomes increasingly more significant as the shear span-to-depth ratio of the member increases. In order to account for the difference in behavior as the shear span-to-depth ratio increases, and members transition from deep to slender, the greater of the terms V_c and V_a is adopted as the contribution of the concrete to the shear strength (von Ramin and Matamoros, 2006). Summarizing, the monotonic shear capacity is found from Eq. (1) as the superposition of equations (2), (8), (10), and (19).

A detailed evaluation of the method for calculating shear strength under monotonic loads can be found elsewhere (von Ramin and Matamoros, 2004, von Ramin and Matamoros, 2006). The model was found to be in good agreement with test results for slender and deep members with and without transverse reinforcement. In members with transverse reinforcement, the mean values of measured to calculated strength within a 95 percent confidence interval were 1.20 ± 0.24 % (COV 13 %) and 1.14 ± 0.20 % (COV 15 %) for deep and slender members, respectively. In members without transverse reinforcement, mean values of 1.11 ± 0.80 % (COV 23 %) and 1.36 ± 0.33 % (COV 29 %) were obtained for deep and slender members, respectively.

FAILURE MODES RELATED TO SEISMIC LOADS

The proposed model is based on the observation that both the flexural and shear strengths of reinforced concrete members decrease with load reversals (Wight and Sozen, 1973). Shear failure is defined as yielding of the transverse reinforcement. The broken line in Fig. 2 represents the change in shear strength under load reversals with increasing displacement. In flexure-controlled members, the maximum shear demand is limited by yielding of the longitudinal reinforcement. In these cases, the member is assumed to have reached its limit state at a displacement corresponding to a reduction of 20% of the maximum shear force. An envelope curve for the load–deflection response can be defined as shown by the solid line in Fig. 2.

In members with highly confined concrete the reduction in the flexural strength is likely to be the limiting factor, while shear failure is likely to occur in members with lower amounts of confining reinforcement. The reduced strength for each of these two limit states must be evaluated separately to determine the controlling mode of failure. In order to ensure ductile behavior under load reversals, the shear-failure line must not drop below the flexural failure envelope for the anticipated maximum displacement demand. Other modes of failure such as sliding shear or rupture of the transverse reinforcement were not explicitly evaluated in the proposed model and must be investigated independently.

According to the proposed method, if the expected displacement demand on a member is less than the displacement that causes yielding of the longitudinal reinforcement, there is no reduction of the monotonic shear strength due to repeated load reversals because the member does not reach the non-linear range of response.

STRENGTH REDUCTION IN FLEXURE-CONTROLLED MEMBERS

The behavior of flexure-controlled members subjected to reversed loading was studied based on a set of 116 columns from a database developed at the University of Washington (2003), and by Brachmann (2002). Detailed calculations are presented in a previous study (von Ramin and Matamoros, 2004).

The shear demand and displacement associated with yielding of the flexural reinforcement provide the starting point to define the failure envelope depicted in Fig. 2. The reduction in shear demand with increasing displacement was established through a function describing the slope of the curve between the yield point and the point corresponding to 80 percent of the shear at yielding of the flexural reinforcement.

Shear demand associated with flexural yielding of members with axial loads below the balanced load

The shear demand at yielding of flexure-controlled members $V_{y,flex}$ is calculated based on the flexural capacity. For example, for a cantilever column fixed at one end and subjected to a point load at the other free end, the maximum shear demand is calculated based on the bending moment at yield M_y

$$V_{y,flex} = \frac{M_y}{a} \tag{25}$$

in which a is the shear span of the member.

The displacement at yielding of the longitudinal reinforcement is calculated as the sum of deflections due to flexure, bar slip, and shear deformation. A deformation component due to sliding shear was not considered, because this failure mode was not in the scope of this study. The total drift ratio is given by

$$\delta_y = \delta_{y,flex} + \delta_{y,slip} + \delta_{y,shear} \tag{26}$$

The flexural component of the drift ratio at yielding of the longitudinal reinforcement is calculated as

$$\delta_{y,flex} = \frac{1}{L}\varphi_y \frac{L^2}{3} = \varphi_y \frac{L}{3} \tag{27}$$

where L = member length, and φ_y = curvature at yielding, given by

$$\varphi_y = \frac{\varepsilon_{sy}}{d(1-k)} \tag{28}$$

The drift ratio related to slip of the longitudinal reinforcement is determined by (Lopez, 1988)

$$\delta_{y,slip} = \frac{1}{L}\frac{L}{\left(d-kd\right)}\frac{f_y}{0.50\sqrt{f'_c}}\frac{1}{8}\varepsilon_{sy}d_b \tag{29}$$

where d_b = bar diameter.

The component of the drift ratio related to shear deformation was calculated as (von Ramin et al., 2002)

$$\delta_{y,shear} = \frac{1}{L}\frac{V_{y,flex}}{A_{eff}\cdot k\,G_c} \tag{30}$$

with A_{eff} = total cross-sectional area of the member, G_c = shear modulus of the member, and k = reduction factor relating the average shear stress to shear strain. For the data set of flexure-controlled beams and columns used in the study the calculated shear deformations at yield were significantly small compared with the components related to flexure and slip of the reinforcement.

A dimensionless ratio m is defined to characterize the degradation of strength due to repeated load reversals based on the shear demand $V_{y,flex}$ and the drift ratio at yield δ_y. The flexural degradation ratio m is taken as the ratio of shear strength reduction to the corresponding change in the drift ratio

$$m = \frac{\left(V_{y,flex} - 0.8 \cdot V_{y,flex}\right) / V_{y,flex}}{\delta_s} = \frac{0.2}{\delta_s} \tag{31}$$

with

$$\delta_s = \delta_{\lim,flex} - \delta_{y,flex} \tag{32}$$

where $\delta_{lim,flex}$ = drift ratio when the shear force is reduced to 80 % of the shear demand at yielding of the flexural reinforcement.

<u>Effect of load reversals on flexural strength</u>
The flexural strength reduction ratio m was established by investigating the effect of several parameters on the experimental response of flexure-controlled beams and columns. The effect of five parameters on the reduction of flexural strength was evaluated for the data set, which comprised 116 members. The dimensionless parameters examined were the shear span-to-depth ratio, the ratio of gross cross-sectional area to core area of the member, the confinement ratio (taken as the effective yield strength of the web reinforcement divided by the compressive strength of concrete, $\rho_w f_{wy} / f'_c$), the ratio of flexural to monotonic shear strength, and the axial stress demand $P/(A_g f'_c)$. Table 1 shows the range of parameters in the data set that was examined. A preliminary analysis of the data was carried out using a feedforward neural network. Result from the trained network were used to identify the most significant parameters and to define functions describing the effects of each parameter (von Ramin and Matamoros, 2004). Following this analysis, two parameters were found to be the most significant for the data set that was considered. These were the confinement ratio and the axial load demand. Functions were derived and calibrated to yield a safe lower bound of the parameter m as the product of these two parameters.

$$m = m_{confinement} \cdot m_{f_c} \tag{33}$$

with

$$m_{confinement} = 3 - 10 \frac{\rho_w f_{wy}}{f'_c} \tag{34}$$

$$m_{f_c} = 1.25 + 5.4 \frac{P}{A_g f'_c} \tag{35}$$

A plot showing values of m inferred from experimental data as defined by Eq. (31) versus values calculated from Eq. (33) is presented in Fig. 3. It is evident from the plot that the amount of scatter was very significant. This can be attributed to several factors, such as the simplified nature of Eq. (33) to (35), experimental errors, the effect of parameters not included in the analysis, and differences in material properties. Although increasing the number of parameters in the formulation did reduce the amount of scatter, this form was selected because it is difficult to justify a more detailed expression given the number of data points in the set, and because this formulation achieved the main objective of providing a safe estimate of strength (Fig. 3). The mean value of measured to calculated strength reduction ratio m was 1.67 ± 1.71 % within a 95 percent confidence region. The standard deviation was 0.93, resulting in a coefficient of variation of 56 percent.

STRENGTH REDUCTION IN SHEAR-CONTROLLED MEMBERS

According to the proposed methodology, the reduced shear strength of shear-controlled members subjected to load reversals is determined as a function of the monotonic shear strength of the member and the displacement demand. This behavior is described by the broken line in Fig 2.

In order to insure the ductile behavior of slender reinforced concrete members, a brittle shear failure must be avoided by maintaining the shear strength above the shear demand caused by yielding of the flexural reinforcement. Detailing a reinforced concrete member so that the monotonic shear strength exceeds the shear demand at first yield guarantees that the member will behave in a ductile manner at displacement demands lower than and slightly greater than that causing yielding of the flexural reinforcement. However, if the shear strength degrades due to load reversals in the inelastic range of response at a faster rate than the flexural strength, the two failure envelopes in Fig. 2 can cross. In this case, a change from a flexure-controlled response to shear-controlled behavior occurs, and a shear failure is likely to occur if the displacement demand exceeds the displacement at which the two lines cross.

The initial shear strength (under monotonic loading) is determined as previously described by the superposition of arch action, truss action, and the contributions from components related to the strength of the compression zone and friction. In members subjected to load reversals, only the first three components are considered effective in the calculation of the reduced shear capacity. Walraven (1986 and 1987) examined the friction mechanism under cyclic loading in detail. The contribution of the friction component is neglected because damage along the surface of the cracks increases with load reversals, and the degradation of this shear transfer mechanism between crack surfaces is very difficult to characterize. Furthermore, the tensile strains in the longitudinal reinforcement are significantly large in members in which yielding of the flexural reinforcement occurs, leading to crack widths that are relatively large compared with the critical crack width Δw_u. Furthermore, it is well documented that the strains in the longitudinal reinforcement, and consequently crack widths, increase with increasing drift demand (Wight and Sozen, 1973, Matamoros and Sozen, 2002). Equations (18) and (21) show that the contribution of the friction component decreases with increasing strain in the longitudinal reinforcement and that it is negligible compared with the other components when the longitudinal reinforcement yields.

As the number of load reversals beyond yield increases, the contributions from the shear strength of the compression zone and arch action decrease. This reduction in strength occurs because concrete that was initially subjected to compression, and therefore uncracked, is damaged by cracks originated by loading in the opposite direction. These cracks weaken the concrete in the struts, reducing the strength of the two aforementioned components. As the strength provided by the arch and concrete components degrades, the demand on the truss mechanism increases. At the same time, the truss mechanism, which relies on the strength of the concrete in the compression field and the transverse reinforcement, is weakened with load reversals also. A variable angle truss model suggests that a reduction in the strength of the concrete in the compression field may be offset by an increase in the angle of the compression field, which increases the demand on the reinforcement. This model indicates that the strain demand on the transverse reinforcement must increase in order to maintain the strength of the truss mechanism, as the concrete in the compression field weakens. This is consistent with observed strain measurements in stirrups of beams and columns subjected to repeated load reversals (Matamoros and Sozen, 2003, Wight and Sozen, 1973).

After the total loss of arch and compression zone contributions, the degraded truss component remains the only shear-carrying mechanism, until the increasing demand causes yielding of the transverse reinforcement. Yielding of the transverse reinforcement is adopted as a limit state for several reasons. At this point the passive confining pressure acting on the concrete no longer increases with lateral expansion, which increases the rate of damage to the concrete with load reversals, and also reduces the effectiveness of the transverse reinforcement in preventing buckling of the longitudinal reinforcement.

Functions describing the degradation of strength were derived separately for the arch, compression zone, and truss components. Detailed data from columns that failed in shear under cyclic loads was very limited. Information about the strain in the transverse reinforcement was obtained for 20 members in the database (Ichinose et al., 2001, Matamoros, 1999, Wight and Sozen, 1973).

The reduced shear strength is obtained by specifying strength reduction coefficients for each of the different shear transfer mechanisms

$$V_n = (1-\eta)(V_{cz} \; or \; V_a) + \chi V_t \tag{36}$$

The first term in Eq. (36) reflects the degradation of strength in the compression zone and arch components, while the second term describes the reduced strength of the truss mechanism.

Effect of load reversals on compression zone and arch components

Results available from column tests conducted by Ichinose (2001) were used to establish a function describing the reduction in the strength of the arch and compression zone components. The data from these tests was selected for the calibration because it allowed evaluating the drift ratio at which the contribution of the arch and compression zone components, indicated by the first term in Eq. (36), became negligible. The reduction of the compression zone and arch components was defined by means of a dimensionless parameter which is a function of the confinement ratio and the drift ratio. Using the test data provided by Ichinose, a non-linear fit to the data yielded the function η as

$$\eta = \frac{8\delta}{\left(\rho_w f_{wy} / f'_c\right) + 0.01} \tag{37}$$

Equation (37) provided a safe estimate of the observed reduction in the strength of the components V_{cz} and V_a. The mean ratio of measured to calculated strength reduction rate η was 1.07 ± 4.3 % within a 95 percent confidence interval. The standard deviation for the data set used in the study was 0.3, resulting in a coefficient of variation of 28 %. Figure 4 shows the strength reduction parameter η plotted against the confinement ratio for different drift ratios. From the graph it follows that, according to the proposed model, a relatively high amount of confinement is necessary to maintain the contributions of the arch action and compression zone components at large drift ratios. For instance, in order to maintain the same rate of strength reduction η, a 100% increase in the drift demand δ would require more than twice the amount of confining reinforcement. This confirms the observation that proper confinement of the concrete is essential to limit the reduction in strength of the components related to the compression zone and arch action.

<u>Strength reduction of the truss component</u>

A simple analysis of experimental data showed that the reduction in shear strength observed in members subjected to reversed cyclic loading occurs not only because there was a reduction in the arch and compression zone components, but also because the strength of the truss mechanism degraded. This is confirmed by the plots shown in Fig. 5 and 6. The *y*-axis of both these graphs corresponds to the ratio of the demand on the truss at the point corresponding to yielding of the web reinforcement to the calculated strength provided by the truss under monotonic loading conditions, V_t

$$\frac{V_{yt} - V_{cz+a}}{V_t} \tag{38}$$

This ratio is plotted against the drift ratio at yielding of the transverse reinforcement, and the axial stress demand, in Fig. 5 and 6, respectively. It is apparent from these two graphs that, at failure, the strength of the truss mechanism was considerably smaller than the expected strength under monotonic loading. Moreover, it is evident that the truss strength decreased non-linearly with increasing drift ratio and axial stress demand. The following strength reduction function for the truss mechanism was found to represent the behavior of experimental results well (von Ramin and Matamoros, 2004)

$$\chi = \frac{1}{1 + 1.5 \cdot \delta_{yt} \cdot 6^{\lambda}} \tag{39}$$

$$\text{with } \lambda = 1 + 2 \cdot (P / Af'_c)^{0.35}$$

Equation (39) was calibrated on the basis of a set of 20 columns tested by Ichinose et al. (2001), Matamoros (1999), and Wight and Sozen (1973) for which the data sets included strain demands on the transverse reinforcement. According to Eq. (39), the strength of the truss component decreases with increasing drift ratio and increasing axial load, which is consistent with trends observed in Fig. 5 and 6. Because the data set used for calibration was very limited, the main objectives of the calibration were to obtain a conservative estimate of strength, and to capture the influence of the axial load and drift ratio. The reduction factor in Eq. (39) approaches unity as the drift ratio approaches zero. This implies that at very small drift demands, the truss component is not reduced, independently of the axial load demand.

Figure 7 shows a plot of the measured versus calculated truss strength, accounting for the effects of load reversals. For the data set considered, the mean value of the measured to calculated strength at yielding of the transverse reinforcement was 1.51 ± 2.8 % within a 95 percent confidence interval. The standard variation was 0.63, resulting in a coefficient of variation of 42%.

Evaluation of the proposed model using experimental results from shear walls

The physical model used in the study of members subjected to monotonic loading (von Ramin and Matamoros, 2004, von Ramin and Matamoros 2006) is applicable to deep beams and structural walls. Furthermore, the design equations developed in that study were calibrated partially on the basis of experimental results from deep beams and walls. For this reason it follows that the proposed method should be applicable to shear walls subjected to load reversals. The contributions of truss action and arch action to the shear strength of walls are calculated similarly to those of deep beams. The main difference between shear walls and deep beams is the presence in some cases of boundary elements. Because the boundary elements represent a well-defined compression zone, their contribution to the shear strength is not negligible, regardless of the shear span-to-depth ratio. Consequently, the shear strength component associated with the compression zone is calculated using Eq. (14), which is the same as using Eq. (24) with *b* equal to the width of the boundary element. The proposed methodology was evaluated using a database comprised of 146 test results from structural walls that failed in shear prior to yielding of the longitudinal reinforcement and had concrete compressive strengths ranging from 15 to 100 MPa. The average ratio of measured to calculated shear strength was 1.28 ± 0.76 % within a 95 percent confidence interval. The standard deviation was 0.47, resulting in a coefficient of variation of 37%.

A broader evaluation of the proposed methodology was conducted using walls that had different modes of failure, such as shear failure in the non-linear range of response under reversed loading conditions, and flexural failure. The initial flexural strength and the flexural strength reduction factor *m* were calculated, and the monotonic shear capacity and the reduced shear strength were determined. The failure envelopes were found to represent the measured response of the walls that were evaluated very well. A detailed evaluation can be found elsewhere (von Ramin and Matamoros, 2004).

The flexural strength of the walls was determined by an approximate equation, which is based on flexural theory (Kabeyasawa and Hiraishi, 1998). According to this method, the moment at yielding of the longitudinal reinforcement is given by

$$M_y = A_{s,be} f_{y,be} l_w + 0.5 A_v f_{vy} l_w + 0.5 P \cdot l_w \tag{40}$$

where $A_{s,be}$ = area of reinforcing steel in the boundary element, A_v = area of vertical reinforcing steel in the web, $f_{y,be}$ = yield strength of longitudinal reinforcement in the boundary element, f_{vy} = yield strength of vertical reinforcing steel in the web, P = axial load, and l_w = wall length.

The shear force at yield $V_{y,flex}$ was obtained by dividing the flexural demand at yielding of the longitudinal reinforcement M_y by the shear span, in this case the height of the wall. The load-displacement response of walls (for example the two envelopes of nominal strength presented in Fig. 8 and 9) was calculated using a procedure proposed by Sozen and Moehle (1993). Deflection components at yield were calculated according to Eq. (26) through (30). A bilinear relationship was adopted for the shear force vs. displacement relationship (Sozen and Moehle, 1993). This procedure was found to yield adequate estimates of shear deformations in reinforced concrete walls in a previous study by the authors (von Ramin et al., 2002).

In the following, the response of two wall specimens is discussed. These two cases are representative of a wall failing in shear prior to flexural yielding and a wall failing in shear after yielding of the longitudinal reinforcement.

The first specimen corresponds to a squat wall (specimen B7-5) tested by Barda et al. (1977). The shear demand caused by the nominal flexural strength of specimen B7-5 greatly exceeded both the calculated and the measured shear strengths. Because this type of wall would be expected to fail in shear prior to yielding of the longitudinal reinforcement, the wall would have to be proportioned based on the monotonic shear strength obtained with the proposed model. The horizontal and vertical web reinforcement ratios were 0.5 %. The longitudinal reinforcement ratio in the boundary elements was very high, with a value of 4.2 %. The axial load applied to specimen B7-5 was relatively low, with an axial stress demand of $P/(Af'_c)$ = 0.2 %. The wall panel B7-5 had a very low aspect ratio of 0.25 and failed in shear before reaching its flexural yield strength. The measured and calculated responses for this wall specimen are shown in Fig. 8. The calculated shear strength, indicated by the solid grey line in Fig. 8, was slightly conservative. The measured failure envelope is displayed as a straight line from the origin to the limiting drift at the maximum load.

The behavior of slender walls is illustrated using specimen B5, tested by Oesterle et al. (1976, 1980). Specimen B5 was part of a series of walls with barbell cross-sections, and an aspect ratio of 2.4. No axial load was applied to any of the specimens in the test series. Specimen B5 had a relatively light amount of web reinforcement in the vertical direction, of approximately 0.3 %, and a horizontal web reinforcement ratio ρ_h = 0.63 %. The longitudinal reinforcement ratio in the boundary elements was ρ_{be} = 3.7 %. According to Oesterle et al., specimen B5 failed due to web crushing after yielding of the longitudinal reinforcement. The measured and calculated responses of specimen B5 are shown in Fig. 9. The limiting shear strength of this specimen, indicated by point B in Fig. 9, was underestimated, whereas the calculated flexural strength (point A) exceeded the measured capacity. Point B represents the point at which according to the proposed model the member behavior changes from flexural to shear-controlled. It is apparent from the graphs in Fig. 8 and 9 that the proposed method was able to provide reasonable estimates of the strength at the limiting drift ratio.

SUMMARY AND CONCLUSIONS

A model was proposed to calculate the shear strength of reinforced concrete members subjected to shear reversals as a function of the shear strength under monotonic loading, the axial load, the amount of confining reinforcement, and the maximum drift demand. According to the proposed model, the shear strength of members subjected to monotonic loads is calculated as a weighted superposition of components related to arch-action, truss-action, friction (aggregate interlock), and the shear strength of the compression zone. Because the proposed model includes components associated with arch action and the strength of the compression zone, calculated shear strengths correlated well with experimental results from specimens with a wide range of shear span-to-depth ratios and levels of axial load.

Equations were developed to evaluate whether members subjected to shear reversals are likely to fail due to a reduction in the flexural or shear strength. The reduction in flexural strength was defined by means of a dimensionless factor that represents the slope of the envelope of the shear demand vs. the drift ratio, drawn from the point corresponding to yielding of the flexural reinforcement to the point corresponding to a 20% reduction of the shear demand. The confinement ratio, defined as the ratio of effective yield strength of the transverse reinforcement, $\rho_w f_{wy}$, to the compressive strength of concrete, f'_c, and the axial stress demand, were found to be the most significant parameters influencing the reduction in flexural strength.

Shear failure of members subjected to repeated load reversals was assumed to occur at yielding of the transverse reinforcement. According to the proposed model, the reduction in shear strength with repeated load reversals stems from the progressive reduction in strength of the components related to arch-action, shear strength of the compression zone, and the truss mechanism. The reduction of shear strength was quantified by strength reduction factors applied to the components of the monotonic shear strength. The strength reduction of the arch and compression zone components was found to depend primarily on the drift demand and the confinement ratio, $\rho_w f_{wy} / f'_c$. It was found also that a relatively high amount of confinement is necessary to maintain the contributions of the arch action and compression zone components at large drift demands, and that proper confinement of the concrete is essential to limit the rate of degradation of the components related to the compression zone and arch action.

Test data showed that in some cases a total loss of the strength components related to the concrete was not sufficient to account for the observed reduction in shear strength at yielding of the transverse reinforcement, which shows that the strength of the truss mechanism deteriorates with load reversals also. A strength reduction factor for the truss mechanism was developed based on experimental data. The strength reduction mechanism for the truss component was found to be most sensitive to the axial stress and the drift demands.

The proposed methodology was evaluated using test data from shear walls. It was found that for walls in which flexural yielding occurred the proposed method provided reasonable estimates of the shear strength and drift demand corresponding to failure.