CHAPTER 5

EXPERIMENTAL RESULTS AND DISCUSSIONS

5.1 GENERAL

Shake table test, can realistically simulate the effects of seismic forces on the structure, but requires large facility. Pushover load test and cyclic load test to some extent can convincingly simulate the effects of seismic forces on the structures and these can be performed with less effort. Pushover load test and cyclic load test are essentially a type of quasi-static testing methods; in this the structure is subjected to slowly changing prescribed forces or deformations by means of hydraulic actuators or jacks. Inertial forces within the structure are not considered in these methods. In pushover load test, the structure pushed geometrically similar to its predominant mode. The performance during the seismic condition is evaluated based on the inelastic demand spectrum and performance point. Computationally called, pushover analysis, is convenient to perform for the buildings and recently most widely used. The purpose of cyclic load test method is to observe the material behaviour of the structural elements, components, or joints when they are subjected to cycles of loading and unloading. In quasi-static testing methods, forces are applied to the test structure which is anchored rigidly to an immobile ground; these forces are gradually increased and are proportional to the product of ground acceleration and the local masses. The deflections measured in this test correspond to the motions of the structural points relative to the ground which could be observed had the specimen been subjected at its base to the actual earthquake.
A comparison is made for the nonlinear behaviour obtained from above three types of tests performed on five similar shear walls. Pushover, cyclic and shake table tests are performed on five identical shear wall specimens; one specimen tested with pushover loads and two specimens each in cyclic loads and shake table tests. The experimental observations from the above tests are given in this chapter. Nonlinear behaviour obtained from pushover and cyclic load tests are compared with shake table tests on shear wall. Limitations and inability in predicting the realistic seismic behaviour in the test methods is also discussed.

5.2 EXPERIMENTAL OBSERVATIONS FROM PUSHOVER LOAD TEST

Pushover loads are applied at the top slab of the shear wall (SW-1) at stages. At each stage of applied load, the strain in rebars and the top displacement is measured. The load is increased up to 206.7 kN until no further load rise is seen. In the initial stages of the loading, a single horizontal crack formed at the bottom of the shear wall. After this, due to subsequent loading, additional cracks are formed and at the ultimate stage these cracks coalesce and merge into a larger crack. Maximum crack width observed is 13 mm. Figure 5.1 shows the typical crack pattern obtained experimentally. The shear wall has failed in flexure by rupture of rebar. Not much of bond slip is observed in the experiment. At the failure stage due to cracking of surrounded concrete, a slight bond slip is observed.

Figure 5.2 shows the experimentally obtained nonlinear load deflection behaviour of shear wall under pushover loads, which can be approximated as trilinear. The tested shear wall has shown a cracking load of 80 kN. At approximately 123 kN, the stiffness of the shear wall has started changing due to initiation of yield in the extreme tension rebar. Marginally
nonlinear load increase is seen up to 192 kN, before the monotonically loaded shear wall gets in to the plastic stage. This is due to sequential yielding of the bars at the extreme tension fiber. Overstrength upon first bar yield is an important parameter used in design. The ratio of the ultimate load to the load at first yield can be considered as over strength ratio. Because of low percentage of tension steel of around 0.2%, the stage of cracking and the stage of yielding of outer most layer of steel are close. The theoretical value of cracking load including the effects of axial loading can be computed as 95 kN while the first yielding load can be computed as 120 kN. The experiments showed a cracking load of 80 kN and first yield load of 123 kN. A marginal reduction in cracking load is possible due to moment shear interaction as the aspect ratio is 1.92, in the present case. No loss of stiffness is observed as there is only one crack near the base at that stage. Hence for all practical purposes the load deformation behaviour can be idealized as trilinear.

The curve is basically a trilinear curve, idealized as bilinear for evaluation of ductility factor, based on equal energy concept. The equivalent yield point is identified by assuming the idealized bilinear behaviour by making equal area between the tangential stiffness at the ultimate stage and the secant stiffness at the yield stage to the load deflection curve. The equivalent yield displacement and ultimate displacement are 10.3 mm and 65 mm respectively with a displacement ductility of 6.3. The drift at ultimate stage is 2.17%.

Figure 5.3 shows the load versus strain variation in the compression side rebars, the strain variation shows a shifting trend for bars 5 and 6 from compressive to tensile strains due to crossing of the neutral axis under pushover loads. The vertical rebar on the extreme compressive side (bar 4) is experiencing large compression strains at ultimate stages, but concrete
spalling is observed only at the ultimate stage. A similar observation is seen in
the strain variation at the compression side rebars, when strain variation is
plotted with reference to displacement (Figure 5.4). Similarly Figure 5.5
shows the tensile strain variation in tension-side end rebars under pushover
loads on shear wall (zoomed portion at the initial stage is given). Multi-linear
variation is clearly seen with yielding, and ultimate zones of the strain
variations. The neutral axis depth is calculated from extreme compression end
using the compression strain and tensile strain in the outer bars and same is
plotted against the load in Figure 5.6. It shows that the neutral axis remains at
half-depth till the cracking load of 80 kN and tends to decrease afterwards.

Figure 5.1 Typical crack pattern obtained in pushover load test (SW-1)
Figure 5.2  Load vs. deflection behaviour of shear wall tested under pushover loading

Figure 5.3  Load vs. strain variation in compression side rebars
Figure 5.4 Displacement vs. strain variation in compression side rebars

Figure 5.5 Tensile strain variation in rebar showing sequential yielding
5.3 EXPERIMENTAL OBSERVATIONS FROM CYCLIC LOAD TESTS

Cyclic load tests are conducted on two similar shear wall specimens (SW-2 and SW-3) of same dimension used in pushover load test. The cyclic hysteresis behaviour and deterioration modes are obtained. The cracks obtained in the cyclic load tests are initially horizontal and turned inclined subsequently. Figures 5.7 and 5.8 show the crack pattern obtained in cyclic load test-1 (SW-2) and cyclic load test-2 (SW-3), respectively. During the second cycle of 50 mm peak displacement, the first specimen failed due to buckling of bars and spalling of concrete (Figure 5.9). Similar observation in the second specimen is observed at first cycle of 50 mm peak displacement.

The experimentally obtained hysteresis curves proved that the hysteresis loop is generally stable without large energy loss between consecutive cycles of displacements. They also exhibited mild pinching-type
behaviour accompanied with large energy dissipation characteristics in both
the cyclically tested shear walls, and a good amount of repeatability is
observed (Figures 5.10 and 5.11). After 10 mm of the peak displacement
cycle, the shear wall has started showing a permanent set. The ultimate load
in the cyclic load test is 170 kN as compared with 206.7 kN in pushover test.
The corresponding ultimate displacement value in cyclic load test is 53 mm as
compared with 65 mm in pushover test. The post yield region in the hysteresis
behaviour obtained in the cyclic loading, however, has not shown significant
strength degradation due to the P-Δ effect.

The progressive damage under reversed cycles of loading resulted
in decreased peak loads and decreased peak displacements for cyclic loading
as compared to monotonic loading. A 20% reduction in both load and
displacement is observed between the monotonic and cyclic loadings. In
general the load capacity also reduces when the specimen is subjected to
cyclic loading. Figure 5.12 shows the secant stiffness variation of shear wall
under both cyclic load tests. The stiffness degradation is quantified using
pivot rule and guidelines given by Reinhorn et al (2009) are followed in
computing this. Both the shear wall tests have shown a stiffness degrading
parameter ($\alpha_3$) of 12.5, which indicates a mild to moderate degrading.
Similarly the pinching parameter ($\alpha_4$) is 0.4, indicating a mild pinching.
Pinched hysteretic loops usually are a result of crack closure. Figure 5.13
shows cumulative hysteretic energy of shear walls under both cyclic load
tests. The quantum of energy dissipated per cycle remained fairly constant
over all three cycles of constant displacement, at all displacement levels. The
cumulative energy varies quadratically with the cyclic deformation. The ratio
of change of energy at each deformation is proportional to the displacement at
that level. Assuming that damage is proportional to cumulative energy at any
displacement level, the percentage of damage suffered at life safety (where
the displacement is half the ultimate displacement) shall be one fourth of the
total energy capacity of the structure. The steel ties have experienced relatively less tensile strain, and consequently snapping of stirrups is not observed. As the diameters of vertical bar (10 mm) and stirrup (8 mm) are comparatively the same, the stirrup is relatively strong and the extreme vertical bar has witnessed buckling between two stirrups. The test results emphasize on providing closer stirrups at the bottom location of the shear wall, where the compressive strains are greater. This will enable avoidance of buckling of the vertical bars. Similar to the monotonic test, bond slip is not observed in the cyclic load tests. The last cyclic load step has, however resulted in a mild bond slip as shown in Figure 5.10. Bond slip in a reinforced concrete member reduces hysteretic energy dissipation and hence damping. The vertical rebar on the extreme compressive side experiencing large compression strains at ultimate stages, but concrete spalling is observed only at the ultimate stage. The concrete cover for columns, as suggested by codes may not be dictated by spalling considerations, but from durability requirements. The ratio of peak loads and peak displacements as well as ductility between cyclic and pushover loading has significant bearing on the corrections to be adopted for the well-known pushover analysis procedure.

Figure 5.7 Typical crack pattern obtained in cyclic load test-1 (SW-2)
Figure 5.8 Typical crack pattern obtained in cyclic load test-2 (SW-3)

(a)                      (b)

Figure 5.9 Compression crushing of concrete and steel buckling observed in (a) cyclic load test-1 (SW-2); (b) cyclic load test-2 (SW-3)
Figure 5.10  Hysteresis behaviour of shear wall obtained in cyclic load test-1 (SW-2)

Figure 5.11  Hysteresis behaviour of shear wall obtained in cyclic load test-2 (SW-3)
Figure 5.12 Comparison of secant stiffness variation of shear wall under two cyclic load tests

Figure 5.13 Comparison of cumulative hysteretic energy (for one cycle at each stage) of shear walls under two cyclic load tests (SW-2 and SW-3)
5.4 EXPERIMENTAL OBSERVATIONS FROM SHAKE TABLE TESTS

Shake table tests are conducted on two shear wall (SW-4 and SW-5) specimens of similar dimension used in pushover and cyclic load tests. Axial loading on shear wall is same in all cases. Spectrum compatible time history is applied in progressively increasing PGA levels and the response of the shear wall is studied. The seismic behaviour and failure patterns are obtained. The first shear wall (SW-4) failed at the 1st cycle of excitation for a PGA level of 0.9 g and the second shear wall (SW-5) has failed at the 3rd cycle of 0.8 g. The measurements are recorded and plotted up to 3rd cycle of 0.8g excitation for SW-4 and 2nd cycle of 0.8g excitation for SW-5. The initial cracks observed in the two tests are predominately horizontal (Figure 5.14 and Figure 5.15). The final failure happened due to crushing of concrete at base in both ends and fracture of reinforcement bars. At final failure stage, due to continuous horizontal cracks a sliding kind of failure happened (Figure 5.16) and vertical rebars are snapped (Figure 5.17). Figure 5.18 show the hysteresis response of shear wall (SW-4) for 0.1g to 0.8g base excitation (3rd cycle response for 0.1g to 0.8g cases). The hysteresis behaviour obtained shows good energy dissipation capacity for shear wall with stable hysteresis loops. However at final failure stage due to sliding and rocking, small oscillating kind of response is observed. This may be clear in Figure 5.19, the hysteresis response of shear wall (SW-4) of 1st, 2nd and 3rd cycle for 0.8g base excitation, shows a distinct change in response just before the final failure. The observed horizontal crack at the base (just above the foundation) prior to ultimate state, may be causing considerable reduction in the strength, stiffness and energy dissipation of the specimens, leading to shear or sliding kind of final failure.

Repeatability of this experiment is verified by testing identical shear wall (SW-5) specimen under seismic loading conditions using shake
The hysteresis behaviour has not varied much (Figure 5.20). Failure pattern and occurred behaviour obtained is same with the SW-4 test. The final failure has happened during the 3rd cycle of 0.8g excitation. The same distinct behaviour in hysteresis loops is also observed in the SW-5 also just before the final failure stage (Figure 5.21). The displacement amplification is around 1.25. The deflection profile observed is triangular rather than parabolic and clear rocking about the base in the in-plane direction is observed at failure stage (Figure 5.22). The time history records of displacement on top of shake table and top of specimen generally shows that peaks in displacements occur at the same time. However at one or two places the relative amplitude is much larger as indicated in hysteretic plots. Figure 5.22 gives the peak relative displacements at top and at middle of shear wall based on hysteretic plots.

The average value of peak displacements taken on five peak cycles at top of specimen, middle of the specimen and table top is obtained and plotted. Figure 5.23 shows displacement amplification of shear wall (SW-4) for 0.8g base excitation (3rd cycle). Figure 5.24 and Figure 5.25 shows a typical displacement response of shear wall (SW-4) for 0.8g base excitation and 0.7g base excitation (3rd cycle) respectively. Figure 5.26 and Figure 5.27 shows a typical acceleration response of shear wall (SW-4) for 0.8g base excitation (3rd cycle) at base and top respectively. Figure 5.28 show displacement amplification observed in shake table test of shear walls (SW-4 and SW-5) respectively, clearly indicating the fairly constant amplification in the displacement. It also shows the good repeatability between the two shake table tests. The total displacement at top of specimen is taken as an average of peak displacements at top of the shear wall. The ratio of total displacement at top of the shear wall to the table displacement is defined as displacement amplification ratio, and is plotted at different acceleration levels in Figure 5.28.
The natural frequency of shear wall is around 5-Hz in the in-plane direction and in out-of-plane direction it is 1.25-Hz. Theoretical calculation gives the natural frequency of 3-m high shear wall with mass of 16-t is higher than the measured value, if the mass 16-t lumped at a point. Because the mass of 16-t is composed of top slab of 0.5-m high and additional mass, is block of 1.65-m high, totally 2.15-m additionally has significant height compared with the shear wall height. This contributes in lowering the natural frequency of the test shear wall. Uncertainties in analytical or finite element modeling like, construction joints, interaction of non-structural components, flexibility of floor diaphragm, torsional effects, base fixity (type of foundation and soil-structure interaction) and uncertainties in experimental set-up like significant space requirement for simulation of mass, connectivity, test set-up fixture interactions makes the natural frequency differ. The free vibration test is conducted at the initial stage and after 0.3g, 0.6g and 0.8g earthquake excitation (Table 5.1). A reduction in the natural frequency of the shear wall is noted after accomplishment of the series of seismic tests.

### Table 5.1 Natural frequency of shear wall

<table>
<thead>
<tr>
<th>Natural frequency, Hz</th>
<th>SW-4</th>
<th>SW-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial frequency</td>
<td>4.8</td>
<td>5.0</td>
</tr>
<tr>
<td>Frequency after 0.3g</td>
<td>4.1</td>
<td>4.2</td>
</tr>
<tr>
<td>Frequency after 0.6g</td>
<td>3.7</td>
<td>3.9</td>
</tr>
<tr>
<td>Frequency after 0.8g</td>
<td>2.4</td>
<td>2.7</td>
</tr>
</tbody>
</table>
Figure 5.14  Typical crack pattern obtained in shake table test on shear wall (SW-4)

Figure 5.15  Typical crack pattern obtained in shake table test on shear wall (SW-5)
Figure 5.16 Failure stage crack pattern obtained in shake table test on shear wall, shows the sliding type failure

Figure 5.17 Snapping of vertical rebars in shake table test on shear wall
Figure 5.18  Hysteresis response of shear wall (SW-4) for 0.1g to 0.8g base excitation (3rd cycle)
Figure 5.19 Hysteresis response of shear wall (SW-4) for 0.8g base excitation, 1\textsuperscript{st}, 2\textsuperscript{nd} and 3\textsuperscript{rd} cycle (1C, 2C and 3C)
Figure 5.20  Hysteresis response of shear wall (SW-5) for 0.1g to 0.8g base excitation (3rd cycle)
Figure 5.21 Hysteresis response of shear wall (SW-5) for 0.8g base excitation, 1st and 2nd cycle

Figure 5.22 Deflection profile observed over the progressive base excitation for shear wall (SW-4)
Figure 5.23 Displacement response at base and top of the shear wall (SW-4) for 0.8g base excitation (3rd cycle)

Figure 5.24 Displacement response at the top of shear wall (SW-4) for 0.8g base excitation (3rd cycle)
Figure 5.25  Displacement response at the top of shear wall (SW-4) for 0.7g base excitation (3\textsuperscript{rd} cycle)

Figure 5.26  Base acceleration response of shear wall (SW-4) for 0.8g base excitation (3\textsuperscript{rd} cycle)
Figure 5.27  Top acceleration response of shear wall (SW-4) for 0.8g base excitation (3\textsuperscript{rd} cycle)

Figure 5.28 Displacement amplification observed in shake table tests
5.5 COMPARISON OF LOAD DEFORMATION BEHAVIOUR

The nonlinear load deflection behaviour of shear wall, under monotonic and cyclic load tests (envelop curve) are compared to the seismic response obtained in the shake table test conducted on a similar dimension shear wall (Figure 5.29). The cyclic load test is conducted in the displacement control mode by applying three cycles for each peak displacements of 2, 4, 6, 8, 10, 15, 20, 25, 30, 40 and 50 mm. The lateral load deformation behaviour is obtained by enveloping the peak points in hysteresis loops. Similarly envelope the peak points in hysteresis loops in the case of shake table test is used to develop the lateral load deformation behaviour.

The progressive damage under reversed cycles of loading has resulted in decreased peak loads and decreased peak displacements for cyclic loading and shake table loading as compared to pushover loading. A 20% reduction in both ultimate load and ultimate displacement is observed between the pushover and cyclic or shake table loadings. The nonlinear response obtained from the shake table has shown reduced stiffness in the initial, this is mainly due to each excitation applied on three times causing concrete cracking. The shear wall has shown an approximate trilinear behaviour in pushover load test, due to sequential yielding of the tension bars. This is exhibited in the cyclic and shake table test results, but not in a distinct way. There are some limitations like inability to predict pinching behaviour from the pushover load test, but this can be possible from the cyclic load test.
A procedure is described to extrapolate the pushover load test results to the shake table test results. The procedure described is based on the equivalent ductility and equivalent damping concept. The improved procedures for equivalent linearization given in FEMA-440 (FEMA-440 2005) are similar to suggested methodology except that the multi-linear effect is not considered. However, the procedure described accounts for piece-wise multi-linear approximation for a nonlinear pushover capacity curve. The pushover curve is converted into Acceleration Displacement Response Spectra (ADRS) format representing the capacity curve. The capacity spectrum can be developed from the pushover curve by a point by point conversion to the first mode spectral co-ordinates using modal mass

Figure 5.29 Comparison of nonlinear behaviour of shear wall

5.6 COMPARISON OF PUSHOVER AND SHAKE TABLE RESPONSE USING EQUIVALENT DUCTILITY AND EQUIVALENT DAMPING
coefficient and modal participation factors as per the procedure described in
ATC-40 (ATC-40, 1996). Alternatively an interesting method proposed by
Midorikawa et al (2003) is used to convert the lateral force deformation
characteristics of the structure into ADRS format using the following set of
expressions:

\[
S_a = \frac{\{\delta\}^T \{M\} \{\delta\}}{\{\delta\}^T \{M\} \{\delta\}} V_b
\]  
(5.1)

\[
\omega_1^2 = \frac{\{\delta\}^T \{f\}}{\{\delta\}^T \{M\} \{\delta\}}
\]  
(5.2)

\[
S_d = \frac{S_a}{\omega_1^2}
\]  
(5.3)

Where, \(\{M\}\) is mass matrix of the structure; \(V_b\) is the base shear; \(\{\delta\}\) is the
obtained displacement profile; \(\{f\}\) is the applied force profile, \(\omega_1\) is natural
circular frequency of the equivalent linear system, \(S_a\) is the spectral
acceleration and \(S_d\) is the spectral displacement. The input motion given in
shake table test is used as the demand in the pushover analysis in order to
compare the shake table test results. The input spectrum for corresponding
base excitation of 0.8g given in the shake table is used as the elastic demand
spectrum (Spectral accelerations vs. Time period) converted in to the ADRS
format using the Equation 5.4.

\[
S_d = \frac{T^2}{4\pi^2} \cdot S_a \cdot g
\]  
(5.4)

Where, \(T\) is the time period and \(g\) is the acceleration due to gravity.
The inelastic demand spectrum is obtained using the equivalent time period,
equivalent ductility and equivalent damping of the structure by idealising the
multi-linear load deformation behaviour of the structure to reflect the effects
of energy dissipation. For that, obtained capacity spectrum from pushover
analysis is idealised into a multi-linear force-displacement curve of positive or negative post-yield slopes. For each linear portion, the procedure needs to be applied so as to modify the elastic demand spectrum to inelastic demand spectrum. $\alpha$ is the ratio of positive post-yield stiffness to the previous stiffness and $\mu$ is the ductility ratio. The time period of the equivalent system ($T_{eq}$) as compared to that of the original system time period ($T_o$) is given by

$$T_{eq} = T_o \left( \frac{\mu}{(1-\alpha)^{0.5} + \alpha \mu} \right)$$

(5.5)

$$\alpha_i = \frac{(s_{d_i} - s_{d_{i-1}})}{(s_{d_{i}} - s_{d_{i-1}})}$$

(5.6)

$$\mu_i = \frac{s_{d_i}}{s_{dy}}$$

(5.7)

Where, ‘$i$’ stands for the each linear portion in the capacity spectrum. $s_{dy}$ is spectral displacement at yield. The concept of equivalent ductility ratio in the place of actual ductility depending on the equivalent time period of the structure is used in the calculation of hysteretic damping ($\zeta_h$). Ductility demand for elasto plastic systems for various time period are derived (Chopra AK, 1995). Equivalent ductility ($\mu_{eq}$) can be stated as follows, depending on the equivalent time period of the system:

$$\mu_{eq} = (1 - 10T_{eq}) + 10T_{eq}\sqrt{2\mu - 1}$$

for $0 < T_{eq} < 0.1$ Sec

(5.8)

$$\mu_{eq} = \sqrt{2\mu - 1} + (\mu - \sqrt{2\mu - 1})(T_{eq} - 0.1)$$

for $0.1 < T_{eq} < 1.1$ Sec

(5.9)

$$\mu_{eq} = \mu$$

for $T_{eq} > 1.1$ Sec

(5.10)
Equivalent hysteretic damping to be used for generating the inelastic demand spectrum of a pushover curve is obtained using the following relationship.

\[
edeq = \frac{(1-\alpha)(\mu_q - 1)}{\left(\mu_q - a\mu_q + a\mu_q^*\right)} \left[ \frac{2}{\pi} \right]
\]  

(5.11)

Hence, the equivalent hysteretic damping (\(\xi_{eq}\)) is derived using the equivalent values of ductility and not with the actual values. In ATC-40 (ATC-40, 1996), \(\xi_{eq}\) given in Equation 5.11 is used with the proviso that

\[
\xi_{eq} = \xi_o + \kappa \xi_h
\]

(5.12)

Where, \(\kappa\) is damping modification factor and \(\xi_h\) is limited to 45\%.

\[
\kappa = \begin{cases} 
1.0 & \text{for } \xi_h = 16.25\% \\
0.77 & \text{for } \xi_h = 45\%
\end{cases}
\]

with linear interpolation for other damping values for structures having stable and full hysteretic behaviour. Considering the hysteresis behaviour a factor is need to apply on \(\kappa\), is 0.67 for reasonably well behaved systems and 0.33 for systems with poor hysteretic behaviour. Viscous damping (\(\xi_o\)) is 5\%. The spectral acceleration levels are reduced using the modification factor of equivalent hysteretic damping using Table 5.2 (IS 1893 (Part-1), 2002). The obtained inelastic demand spectrum meets the capacity spectrum at performance point. The displacement demand at performance point of a particular level of acceleration (0.8 g) input is merely matched with the displacement demand obtained during the shake table test of same acceleration level (0.8 g) of base excitation. The displacement demand at performance point in the spectral co-ordinates, need to be convert back to the displacement co-ordinates, to match with shake table response. Table 5.3
gives the comparison of displacement demand at performance point to maximum response obtained in corresponding shake table input. However, in the present case study, the shear wall tested on the shake table by applying base excitation in various levels of acceleration starting with 0.1g, 0.2g and so on until failure (0.8g), each acceleration level excitation is applied on three times. This kind progressive weakening the shear wall made the fast stiffness degradation and early failure. So considering the accumulated damage, a correction factor is applied to match the displacement response obtained in the shake table test with the displacement demand at performance point. The procedure can be effectively used in to extrapolate the seismic response from the pushover results to real seismic behaviour obtained in the shake table test. Figure 5.30 show the pushover curve and capacity spectrum of shear wall tested with pushover loads, the load drop after final failure stage and residue portion in the pushover is removed for simplification. Figure 5.31 shows the performance point obtained for 0.8g seismic input from the interaction of inelastic demand spectrum to capacity spectrum respectively. For both the shear wall cases the displacement at performance point is merely matched with peak response obtained in the shake table test for corresponding seismic input acceleration.
Figure 5.31  Performance point obtained for 0.8g seismic input (DS-Demand Spectrum)

Table 5.2  Multiplying factors for obtaining values ($S_a$) for other values of damping (IS 1893 (Part-1), 2002)

<table>
<thead>
<tr>
<th>Damping, Percent</th>
<th>0</th>
<th>2</th>
<th>5</th>
<th>7</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor</td>
<td>3.2</td>
<td>1.4</td>
<td>1.0</td>
<td>0.9</td>
<td>0.8</td>
<td>0.7</td>
<td>0.6</td>
<td>0.55</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 5.3  Comparison of displacement demand at performance point to maximum response obtained in corresponding shake table input

<table>
<thead>
<tr>
<th>Seismic input level</th>
<th>Displacement at performance point (mm)</th>
<th>Max. response obtained in shake table (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8g</td>
<td>40</td>
<td>SW-4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1-C</td>
</tr>
<tr>
<td></td>
<td></td>
<td>31.46</td>
</tr>
</tbody>
</table>

(1-C is 1st Cycle; 2-C is 2nd Cycle; and 3-C is 3rd Cycle)
5.7 HYSTERETIC MODEL FOR NONLINEAR CYCLIC BEHAVIOUR OF SHEAR WALLS

The prominent characteristics of hysteretic behaviour for RC shear walls under repeated cyclic deformation, is invariably deterioration. Hence, such deterioration must be taken into account while modelling and designing RC shear walls under repeated cyclic deformation. The basic requirement to perform seismic analyses is the availability of accurate constitutive models capable of representing deteriorating structural behaviour. Hysteretic model proposed by Ibarra et al (2005) is used to fit the hysteretic behaviour obtained in the cyclic load test. This model includes energy-controlled strength and stiffness degradation under cyclic loading and can simulate successfully the response of steel, plywood and reinforced-concrete components under cyclic loading. This model incorporates an energy-based deterioration parameter that controls the cyclic deterioration modes.

Ibarra et al (2005) has defined the backbone curve from the monotonically increasing deformation response, and the three hysteretic models are defined using the backbone force displacement curve. Three traditional hysteretic models are modified, namely, bilinear, peak-oriented, and pinching models by incorporating four cyclic deterioration modes: basic strength, post-capping strength, unloading stiffness, and accelerated reloading stiffness. An energy-based approach is used to implement these deterioration modes. The backbone curve (Figure 5.32) is piece-wise linear and includes three control points, which correspond to changes in wall stiffness with increasing displacement. The first control point is associated with yielding, the second corresponds to peak strength, and the third point is associated with residual strength. The backbone curve is defined by three parameters: the elastic (initial) stiffness $K_e$, the yield strength $F_y$, and the strain-hardening stiffness $K_s = \alpha_s K_e$. Where $\alpha_s$ is the strain hardening coefficient. If
deterioration of the backbone curve is included, a softening branch begins at the cap deformation ($\delta_c$), which corresponds to the peak strength ($F_c$) of the load–deformation curve. The softening branch is defined by the post-capping stiffness, $K_c = \alpha_c K_e$, which is usually a negative value. In addition, a residual strength ($F_r$) would be also assigned to the model, $F_r = \lambda F_y$, which represents the fraction of the yield strength of the component that is preserved once a given deterioration threshold is achieved. Where, $\alpha_c$ is the post-capping coefficient and $\lambda$ residual strength factor.

![Figure 5.32 Backbone curve for the Ibarra et al (2005) hysteretic models](image)

The bilinear model is based on bilinear hysteretic rules with kinematic hardening. The peak-oriented model uses the hysteretic rules proposed by Clough & Johnston (1966) as later modified by Mahin & Bertero (1976), but the backbone curve is modified to include strength capping and residual strength. The pinching model is similar to the peak-oriented model except that the reloading curve includes two segments to simulate pinched hysteretic behaviour which is a characteristic due to crack opening and closing in the wall. Figure 5.33 presents a load-displacement hysteretic
behaviour constructed by using the pinching model of Ibarra et al (2005) hysteretic model. The dashed lines represent the backbone curve.

Figure 5.33 Basic rules for the pinching hysteretic model (Ibarra et al 2005)

The load-displacement relationship follows the backbone curve up to the displacement at which the component is unloaded (point 2). In the absence of deterioration, the unloading stiffness is equal to the elastic stiffness ($K_e$). The reloading part of the load displacement relationship consists of two segments with stiffness’s $K_{rel,a}$ and $K_{rel,b}$. Initially the reloading path is directed towards a ‘break point’, which is a function of the maximum permanent deformation and the maximum load experienced in the direction of loading. The force at break point is defined by the parameter $\zeta_f$, which modifies the maximum ‘pinched’ strength (points 4 and 8 of Figure 5.33), and the parameter $\zeta_d$, which defines the displacement of the break point (points $4'$ and $8'$). The coefficients $\zeta_d$ and $\zeta_f$ are the empirical parameters which must be determined using experimental data. Reloading is directed initially towards a break point that is a function of the maximum force ($F_{max}$) and the maximum residual displacement ($\delta_{per1}$) in the same quadrant of the previous cycle. As seen in Figure 5.33, the break point associated with the pinching region (point
8’) is established using the (1- \( \kappa_d \))\( \delta_{per1} \), \( \kappa_f F_{max} \) displacement force coordinates of the second displacement cycle. The second part of the reloading (represented by stiffness, \( K_{rel,b} \)) targets the maximum displacement achieved in the same quadrant in the previous cycle.

The Ibarra et al (2005) pinching model incorporates an energy-based deterioration parameter that controls four cyclic deterioration modes: basic strength, post-capping strength, unloading stiffness, and accelerated reloading stiffness deterioration. Four cyclic deterioration modes may be activated once the yield point is has exceeded in at least one direction. All four deterioration modes are implemented using a parameter that is a function of energy dissipated under cyclic loading using the rules proposed by Rahnama & Krawinkler (1993). The cyclic deterioration in excursion ‘i’ is defined by the parameter \( \beta_i \), which is given by the following expression:

\[
\beta_i = \left( \frac{E_i}{E_t - \sum_{j=1}^{i} E_j} \right)^c
\]  

(5.13)

\( E_i \) is the hysteretic energy dissipated in excursion i, \( \sum_{j=1}^{i} E_j \) is the sum of the hysteretic energy dissipated in all previous excursions, \( E_t \) the reference hysteretic energy dissipation capacity that is equal to \( \gamma F_y \delta_y \). The parameter ‘\( \gamma \)’ expresses the hysteretic energy dissipation capacity as a function of twice the elastic strain energy at yielding \( (F_y \delta_y) \), which is calibrated from experimental results and can be different for each deterioration mode. ‘\( c \)’ determines the rate of deterioration and range between 1.0 and 2.0. If the displacement history consists of constant amplitude cycles, a unit value for \( c \) implies an almost constant rate of deterioration. For the same displacement history, a value \( c = 2 \) slows down the rate of deterioration in early cycles and accelerates the rate of deterioration in later cycles. Throughout the loading history, \( \beta_i \) must be within the limits \( 0 < \beta_i \leq 1 \). If \( \beta_i \) is outside these limits, the hysteretic energy capacity is exhausted and collapse is assumed to take place.
The four deterioration modes: basic strength, post-capping strength, unloading stiffness, and accelerated reloading stiffness deterioration are incorporated using the energy-based deterioration parameters corresponding to the each deterioration mode. Basic strength deterioration corresponds to reduction in yield strength (\(F_y\) in the backbone curve) and post-yielding stiffness (\(K_s\) in the backbone curve) with cyclic loading. Basic strength deterioration is defined by translating the strain hardening branch towards the origin by an amount equivalent to reducing the yield strength to

\[
F_i^+ = (1 - \beta_{s,i})F_{i-1}^+
\]

and \(F_i^- = (1 - \beta_{s,i})F_{i-1}^-\) (5.14)

where \(F_i^{+/−}\) and \(F_{i−1}^{+/−}\) are the deteriorated yield strength after and before excursion \(i\) respectively. The parameter \(\beta_{s,i}\) is calculated with Equation 5.13 each time the elastic path crosses the horizontal axis and is associated with the appropriate \(\gamma\) value to model basic strength deterioration (\(\gamma_s\)). The deterioration parameter \(\beta_s\) is calculated as the force changes its arithmetic sign. The yield strength in the direction of the loading are updated using the deterioration parameter calculated using Equation 5.14 and Equation 5.15. Simultaneously along with the yield strength, slope of the strain hardening branch is also modified according to the yield strength, slope of the strain hardening branch is also modified according to the Equation 5.16 and Equation 5.17.

\[
K_{s,l}^+ = (1 - \beta_{s,i})K_{s,l−1}^+
\]

and \(K_{s,l}^- = (1 - \beta_{s,i})K_{s,l−1}^−\) (5.17)

where \(K_{s,l}^{+/−}\) and \(K_{s,l−1}^{+/−}\) are the deteriorated strain hardening slopes independently in both directions.
Post-capping strength deterioration corresponds to the translation of the post-capping branch towards the origin and, unlike basic strength deterioration; the slope of the post-capping branch is kept constant. The reference strength in post-capping branch is reduced by

\[ F_{ref,i}^{+/−} = (1 − \beta_{c,i}) F_{ref,i−1}^{+/−} \]  

(5.18)

where \( F_{ref,i}^{+/−} \) and \( F_{ref,i−1}^{+/−} \) are the intersection of the vertical axis with the projection of the post-capping branch after and before excursion \( i \) respectively. The parameter \( \beta_{c,i} \) is calculated with Equation 5.13 each time the elastic path crosses the horizontal axis and is associated with the appropriate \( \gamma \) value to model post-capping strength deterioration (\( \gamma_c \)).

The third deterioration mode is unloading stiffness deterioration mode which modifies the unloading stiffness (\( K_u \)) in accordance with the following equation:

\[ K_{u,i} = (1 − \beta_{k,i}) K_{u,i−1} \]  

(5.19)

where \( K_{u,i} \) and \( K_{u,i−1} \) are the deteriorated unloading stiffnesses. The parameter \( \beta_{k,i} \) is calculated with Equation 5.13 each time when a load reversal takes place in the inelastic range and is associated with the appropriate \( \gamma \) value to cyclic deterioration parameter (\( \gamma_k \)). The unloading stiffness deteriorates up to twice as much as the other deteriorating quantities, and if the same rate of deterioration is expected in the four deterioration modes, \( \gamma_k \) should be about twice as large as the other \( \gamma \) values.

The last deterioration mode is accelerated reloading stiffness. This deterioration mode increases the absolute value of the target displacement, defined as the maximum positive or negative displacement of past cycles,
according to the direction of loading. The target displacement increased by the following equation:

\[ \delta_{t,i}^+/^- = (1 + \beta_{a,i})\delta_{t,i-1}^{+/^-} \]  

(5.20)

where \( \delta_{t,i}^+/^- \) and \( \delta_{t,i-1}^{+/^-} \) corresponds to the target displacement after and before excursion \( i \) respectively. The parameter \( \beta_{a,i} \) is calculated with Equation 5.13 each time the elastic path crosses the horizontal axis and is associated with the appropriate \( \gamma \) value to model accelerated reloading stiffness deterioration (\( \gamma_{ad} \)).

In the present investigation, monotonic load deformation curve obtained in the pushover test of shear wall is idealized to the backbone curve to identify model parameters \( F_y, F_c, F_r, \delta_y, \delta_c \) and \( \delta_r \). The stiffness parameters \( K_e, K_s, K_c \), strain hardening coefficient (\( \alpha_s \)) and post-capping coefficient (\( \alpha_c \)) are determined. Pinching model is used and various deterioration modes are considered. This model incorporates an energy based deterioration parameter (\( \gamma \)) that controls the cyclic deterioration modes. ‘c’ is the exponent defining the rate of deterioration, \( c = 1.3 \) is used. The parameters \( \kappa_i \), which modifies the maximum pinched strength and \( \kappa_d \), defines the displacement of break point, and 0.6 is used for both. The analytical prediction of the hysteresis loop is obtained by trial and error method by varying the value of \( \gamma \) (\( \gamma = 150 \)). Post-capping strength deterioration mode is dropped since the shear wall has not shown significant strength degradation in its cyclic behaviour. The basic strength deterioration parameter (\( \gamma_s \)) and accelerated reloading stiffness deterioration parameter (\( \gamma_{ad} \)) are taken equal to \( \gamma \), but the unloading stiffness parameter (\( \gamma_k \)) is twice the \( \gamma \). The analytical prediction from the model is matched with the experimental results obtained from the both cyclic load tests SW-2 and SW-3 (Figure 5.34 and Figure 5.35).
Figure 5.34  **Hysteretic model for nonlinear cyclic behaviour of shear wall (SW-2)**

Figure 5.35  **Hysteretic model for nonlinear cyclic behaviour of shear wall (SW-3)**
5.8 SUMMARY

Experimental observations are presented for five similar shear walls. There are three types of tests, namely pushover, cyclic and shake table tests. A comparison is made for the nonlinear behaviour obtained from above three types of tests performed on five similar shear walls. Limitations and inability in predicting the realistic seismic behaviour in the test methods also discussed. The shear wall has shown a load deformation behaviour which can be approximated as trilinear in pushover load test. A procedure is suggested to extrapolate the pushover load test results to the shake table test results. The procedure used is based on the equivalent ductility and equivalent damping concept. Stiffness and strength degradation and pinching parameters are evaluated from cyclic tests. Suitable hysteretic model is fitted for the hysteretic behaviour obtained in the cyclic load test and various model parameters are suggested for the prediction of nonlinear cyclic response of shear walls.