Motivation

The behaviour of shear walls under cyclic loads has not sufficiently been tested in the past. Test results for various shear wall cases are desired.

Main Results

Several large testing campaigns have been performed in India. Results for a specific type of shear wall applied in the nuclear industry became available.
29-1 Introduction

Performance based design approach is gaining importance for assessing structures subjected to earthquake loading. Evaluation of seismic performance of nuclear power plants requires assessment of shear walls which are its main structural elements. In performance based seismic design approach, the failure event occurs when the structure fails to satisfy the requirements of a prescribed performance level e.g. immediate occupancy, life safety or collapse prevention [FEMA-273]. Evaluation can be achieved by performing detailed experiments. It is known that accurate estimates of moment/shear capacity of shear walls under cyclic loading with axial load are extremely difficult to make. This necessitates experimental testing for accurate assessment of shear response and capacity of the shear walls. Experimental studies provide physical insight in the force resisting mechanisms of these walls and help us to validate and improve the numerical models and corresponding constitutive relations.

Damage to structures subjected to earthquakes are observed due to exceeding of design loads, design inadequacy or loss of capacity due to ageing effects. This makes it very important to assess the vulnerability of structures against different levels of earthquake loads. The primary reason of failure of structures due to earthquake is ground shaking. At a very low level of ground shaking, it can be said with confidence that the structure will not fail. However, at a high magnitude of ground shaking the structure may fail. The capacity of the structure is dependent on many parameters. There is always a scatter observed in the values of these parameters. The demand on the structure can also exhibit large scatter especially if the demand is of the postulated accident loads like earthquake. The likelihood that a structure would fail due to different levels of earthquake is estimated using the principles of structural reliability and curve so plotted is termed as the Fragility Curve [EPRI, 1994]. The design of the structure based on the Fragility Curves generated for structures and components provides a quantified safety level against the effect of earthquake loading. This work is aimed at providing a demonstration for estimating the design loading for a typical shear wall used in a nuclear power plant based on the probabilistic theory. The modelling of the shear wall is validated using the experiments.

Scaled midrise shear walls of a typical Nuclear Power Plant have been tested under monotonic and cyclic loading. Finite Element Analysis is carried out to evaluate the ultimate load and drift of the shear wall and load displacement relation under cyclic loading is evaluated. The same wall is analyzed based on structural reliability methods and a fragility curve is generated. The IRIS Risk Paradigm model is thus applied to the shear walls used in the nuclear industry. The uncertainty in the design parameters is modelled using the probability distribution function. Thus the design load is arrived using the sound logic of probabilistic analysis rather than a deterministic factor of safety.

Finite Element analysis of short stiff shear wall tested at JRC, Italy in WP1 is also carried out and the comparison between the test and analysis results are presented.
29-2 Performance of Shear Walls

Shear walls used in RCC buildings, nuclear power plants and other structures under sufficient lateral excitation may fail under various mechanisms like flexure, shear or sliding shear thus resulting in significant lateral displacement and strength degradation. The ultimate load on the shear wall as well as the ultimate drift of the shear wall are two important parameters which need to be assessed experimentally. In many investigations, loss of equipment function housed within a nuclear power plant (NPP) structures has been considered to occur when the ultimate drift limits are reached, thus proving that ultimate drift is a failure parameter. Earlier researchers have carried out cyclic tests on squat walls [Lopes, 2001a, 2001b] in which the response was shear dominated. Also predictions of the behaviour of the shear walls for monotonic static and cyclic loading using finite element analysis and simplified approaches have been carried out [Salonikios, 2002; Farrar and Baker, 1990]. The models used were able to predict the maximum load more accurately than the displacements at the peak load. However the ductility of the walls could not be accurately predicted and also accurate prediction of crack pattern from FE analysis was difficult. Tests were also carried out on walls with aspect ratio 1 to 1.5 [Kazaz et al., 2006] wherein sliding shear failure was emphasized. Similarly tests carried out on mid-rise walls [Tasnimi, 2000] and high-rise walls [Ellingwood and Hwang, 1985] showed flexure dominated failure. From the literature available it is found that improvement in the ductility model and hysteretic response is needed. Hence in the present work behaviour of
mid-rise shear walls is investigated through FE analysis and experiments and behaviour of low rise shear walls tested in WP1 tests of IRIS project is simulated by performing nonlinear FE analysis.

Monotonic and cyclic tests provide the ultimate load and the ultimate drift of the shear walls. Comparison of force displacement data obtained from monotonic tests and cyclic tests with experiments is made for both shear walls. The exact mode of failure of the wall is obtained and parameters like ultimate load, ultimate drift limit and ductility ratio are obtained.

### 29-3 Details of Mid-Rise Shear Wall

The shear wall considered for the tests is a 1:5 scaled model of the internal structure of a nuclear power plant building. The schematic of the reactor building showing the shear wall is shown in F.29-1. The shear wall (7.8 m wide, 15 m high and 1 m thick) supports the steam generator and floor of Indian type nuclear power plant and its first fundamental frequency is 3.1 Hz. This shear wall shows a cantilever type mode as the mass of steam generators and floor is lumped at the top of the shear wall.

Scaling laws are used and lumped mass on the top of the shear wall is obtained so that the frequency of the shear wall model along with lumped mass is 15.5 Hz. The shear wall is designed as per Indian standard codes [IS13920, 2002] for dead load and 0.4 g earthquake acceleration at the mass level of the wall. The model has 3 m height (h), 1.56 m width and 0.2 m thickness and percentage reinforcement of 0.4% in vertical direction and 0.3% in horizontal direction. The foundation is 0.4 m deep and 2 m × 2 m in area and the top slab
A squat short shear wall with a width of 3 m, a height of 1.2 m and a thickness of 0.4 m was tested at JRC, Italy; the test programme was called TESSH (TEsts on Stiff SHear wall). Vertical reinforcement consisted of 22 bars of 16 mm diameter arranged in two layers and horizontal reinforcement consisted of 6 bars of 16 mm diameter arranged in two layers. Top and bottom beams were 4 m long, 0.8 m deep and 1.25 m thick.

Vertical 60 tons of load were applied by vertical actuators. There was control of rotation of the shear wall during testing, however, vertical displacements were allowed. A rigid loading device was designed so that it should transfer the load and be easily mountable. The shear wall is loaded in pure shear with no rotations allowed and the structure was connected to the reaction wall. There were four horizontal actuators, each actuator having a capacity of 300 tons with a total capacity of 1200 tons. Instrumentation was carried out using optical measurements to measure the crack width and crack pattern. Cyclic loading was applied initially starting with three cycles of 50 tons. Then two cycles each of 100, 200, 300, 400, 500, 600, 650, 700 and 750 tons were applied. Finally a monotonic loading test was conducted till failure of the wall.
29-5 Material Constitutive Law

29-5-1 Concrete Material Constitutive Model

The constitutive model is formulated on the basis of non-linearity of concrete. Concrete model is a damaged-based model in which a smeared approach is used to model both cracks. This model comprises non-linear compressive behaviour that is capable of modelling hardening and softening. The pre-peak relation is based on the [CEB-FIP Model Code 1990]. The post peak compressive behaviour is linear descending (F.29-5), and the strain at zero stress is 4.7 $\varepsilon_b$. Also, compressive strength in the direction parallel to the cracks is reduced based on work done by [Vecchio and Collins, 1986] and formulated in the Modified Compression Field Theory. Presence of large transverse strains in cracked biaxially stressed concrete serves to decrease the strength and stiffness of the concrete in the direction of principal compression. A reduction of the compressive strength after cracking in the direction parallel to the cracks is done in a similar way as found from experiments of [Vecchio et al., 1994] and formulated in the Compression Field Theory.

In the analysis, a different function in the form of Gauss's function is used for the reduction of concrete strength shown in F.29-6. The parameters for this function were derived from experimental data of [Vecchio et al., 1994].

\[
\begin{align*}
\sigma_c' & = \sigma_c'' \\
\varepsilon_c' & = \varepsilon_c'' + (1 - \varepsilon_c''/\varepsilon_b) \varepsilon_c
\end{align*}
\]

E.29-1 E.29-2

\[
\varepsilon_c'' = c + (1 - c)e^{-(128c)^r}
\]

Thus for zero principal tensile strain there is no strength reduction and for large strain the strength asymptotically approaches value $c_f'$. The evaluation of the parameter $c$ is an important factor. F.29-5 shows the uniaxial stress-strain law used for the concrete.

High strength concrete is more brittle with cracks forming through aggregates rather than around them. Thus cracking related damage is more pronounced in high strength concrete. Thus the concrete softening effect is more pronounced in high strength con-
crete. These aspects are taken care in the constitutive model by considering the modification factor to reduce the peak stress of the stress-strain curve of concrete by factor $\beta$. The factor $\beta$ suggested by them is given by following relation

$$\beta = \frac{1}{1 + K_c \cdot K_f}$$  \hspace{1cm} E.29-3

Where $K_c$ represents the effect of transverse cracking and $K_f$ represents the dependence on the strength of concrete.

$$K_c = 0.35(\varepsilon_1 / \varepsilon_2 - 0.28)^{0.8}$$ \hspace{1cm} E.29-4

$$K_f = 0.1825 \sqrt{f_c}$$ \hspace{1cm} E.29-5

Where $\varepsilon_1$ and $\varepsilon_2$ are the principle tensile and compressive strains in the concrete respectively. The maximum value of $\beta$ will be obtained at peak compressive stress (F.29-5). Thus at peak compressive strain of 0.002 the principal tensile strain will be about 0.0075 as obtained by Vecchio and Collins by testing various shear wall panels having 0.8 % vertical reinforcement [Vecchio et al., 1994]. Thus the value of factor $\beta$ at the strain value of 0.002 (i.e. strain at peak cylindrical stress) is 0.46. Thus the minimum value of parameter $c$ considered for analysis mentioned in E.29-1 and E.29-2 is 0.46 and its maximum value is 1.

In the analysis, the model also incorporates a biaxial failure criterion as given by [Kupper et al., 1969]. Under cyclic loading, unloading hysteretic rule is according to origin oriented model. It can be observed from the uniaxial stress-strain diagram (F.29-5) that unloading is a linear function that returns to the origin. Upon reloading, the stress-strain relation follows the unloading path until the last loading point is reached. Also, the tensile response before cracking is considered linear elastic and tension stiffening effects are considered after cracking.

After cracking, the constitutive relation is used in combination with the crack band to model crack propagation based on a crack-opening law and fracture energy. The concrete
constitutive model considering the FE analysis is based on smeared crack approach. In this analysis smeared fixed crack approach based on non-linear fracture mechanics is used. When equal amounts of reinforcement are provided in the longitudinal and transverse directions, cracks experience minimal rotation, and a fixed crack procedure will provide an accurate simulation. Cracks are formed when the principal tensile stress exceeds the tensile strength of the concrete. In the fixed crack model [Cervanka, 1985; Darwin and Pecknold, 1974], once the crack forms, the crack direction is defined by the direction of principal stress. The direction remains the same upon continued loading. Also, the shear modulus is reduced to represent the reduction in shear stiffness due to the crack opening and the crack shear stresses are considered.

29-5-2 Reinforcement Constitutive Relations

Reinforcement is modelled as discrete using truss elements in wall and smeared in top and bottom beams. For smeared reinforcement it is considered as a component of composite material. In either case, the reinforcement stress-strain relationship is defined by the bilinear law in which elastic-plastic behaviour is assumed. The discrete reinforcement elements of the wall are fully bonded to the surrounding concrete with limited prescribed bond strength (cohesion stress). Slippage occurs if the cohesion stress rises above the bond strength. Bond-slip relation is considered using CEB-FIP model code 90 bond model.

29-6 Analysis and Experiments on Mid-Rise Shear Wall

29-6-1 Moment Curvature Relationship

Mid-rise shear wall failure of the shear wall is based on flexural yielding and the evaluation of flexural strength of the shear wall. The shear capacity of the shear wall is quite higher than the flexural strength. For concrete in uniaxial compression the stress-strain relation covering compression softening as shown in F.29-5 is used. The steel is assumed as elastic perfectly plastic. The normal force, \( N \) and moment, \( M \) equilibrium conditions for the shear wall using stress-strain distribution across the section shown in F.29-7, are written as follows

\[
N = k_f' b kd + \sum_{j=1}^{8} \sigma_{ij} A_{ij} - k_s f' b (h - kd)
\]

\[
M = k_f' b kd \left( \frac{h}{2} - k_s kd \right) + \sum_{j=1}^{8} \sigma_{ij} A_{ij} \left( \frac{h}{2} - d_i \right) + k_s f' b (h - kd) \left( \frac{h}{2} - k_s (h - kd) \right)
\]
Stress-strain relation across the cross section of the wall

\[ f'_c \] is the compressive strength of concrete, \( f'_t \) is the tensile strength of concrete, \( \sigma_{ij} \) is the stress in steel in the \( j \)-th layer and \( d_i \) is the distance of the steel layer from top fibre. Where parameter \( k_1 \) defines the average compressive stress and the resultant force acts at \( k_2 kd \) below the compression face. Similarly \( k_3 \) denotes the resultant of tensile stress in concrete and the force acts at distance of \( k_4 (h - kd) \) from tension face.

The parameters \( k_1, k_2 \) and \( k_3, k_4 \) are described by expressions given below

\[
k_1 = \frac{\int_0^m \sigma_c d\varepsilon_c}{f'_c \varepsilon_{cm}}, \quad k_2 = 1 - \frac{\int_0^m \varepsilon_c \sigma_c d\varepsilon_c}{\varepsilon_{cm} \int_0^m \sigma_c d\varepsilon_c}
\]

\[
k_3 = \frac{\int_0^m \sigma_t d\varepsilon_t}{f'_t \varepsilon_{tm}}, \quad k_4 = 1 - \frac{\int_0^m \varepsilon_t \sigma_t d\varepsilon_t}{\varepsilon_{tm} \int_0^m \sigma_t d\varepsilon_t}
\]

The curvature of the shear wall at any moment capacity is given by

\[
\kappa = \frac{\varepsilon_{cm}}{kd}
\]

In order to calculate the moment curvature relation, for a given axial load \( N \) a succession of values of \( \varepsilon_{cm} \) increasing in small increments is considered and for each value, \( kd \) is obtained using E.29-6. The curvature is then obtained from E.29-10 and finally the moment for the particular value of curvature is obtained from E.29-7. For the shear wall, \( b = 200 \text{ mm}, h = 1560 \text{ mm}, A_{ij} = 157 \text{ mm}^2 \) for \( j = 1 \) to \( 8 \), \( d = 1520 \text{ mm} \), \( f'_c = 39.1 \text{ MPa} \), \( f'_t = 3.08 \text{ MPa} \), \( E_c = 3.586 \times 10^4 \text{ MPa} \), \( f_y = 500 \text{ MPa} \), \( E_s = 2 \times 10^5 \text{ MPa} \). The moment curvature relation is obtained and the maximum displacement of the shear wall is calculated for particular load from moment curvature relationship using the principle of virtual work thus load displacement relation is obtained and the peak load is obtained as 195 kN.
29-6-2 FE Analysis

2D non-linear, FE analyses [ATENA, 2006] are carried out on the RC wall. The testing boundary conditions are simulated in the analyses as accurately as possible. The horizontal and vertical displacement at the base of the wall is assumed to be zero in the FE model. The vertical translational degree of freedom at top slab right end is free and incremental horizontal displacements are applied at that node of top slab from the right towards left direction. The vertical load of 8.5 tons is applied uniformly on the top slab. The concrete is modelled with the non-linear model with a cubic strength of 46 MPa. The steel is modelled as reinforcement bars with a bilinear elastic-plastic model with a yielding strength of 500 MPa. The analysis is carried out for monotonic loading and pseudo-static cyclic loading. Three cycles of each value of maximum lateral displacements will be applied starting with 0.4 mm displacement as cracking occurred at 0.4 mm from analysis for monotonic loading. The loading is applied in incremental displacement with increment of peak displacement of 2 mm displacement after every three cycles till failure. The loading history for pseudo-static cyclic loading is shown in F.29-9. The load deflection relationship obtained for monotonically loading cyclic load is shown in F.29-11. It is observed that the monotonic load deflection characteristics envelopes the cyclic characteristics. It is observed that the ultimate load taken by the wall is 185 KN and the displacement at ultimate load is 16 mm. The drift failure, i.e. at 85% of ultimate load, is 39 mm (1.3%) from the analysis for monotonic loading. It is observed from the analysis that the deflections corresponding to the performance states of first cracking, yielding of reinforcement and ultimate state are 0.4 mm, 4 mm and 16 mm respectively. F.29-9 shows the crack pattern in the wall at ultimate load. It is observed from the figure that failure of the shear wall is due to bending loads and flexural cracks are seen in the FE model.

29-6-3 Experimental Programme

Three identical shear wall specimens, considered to represent the critical structural elements with a rectangular cross-section, and the average 28 days strength of concrete
obtained was 46 MPa. Tension tests were carried out on six HYSD reinforcement bars with a yield strength of 415 Mpa and average ultimate tensile strength of 555 MPa.

Methodology of Tests

One specimen was subjected to slow monotonic load to obtain the backbone load displacement curve. In order to simulate loading sequence that might be expected to occur during an earthquake, horizontal cyclic loading history was adopted for other two wall specimens. The torsional mode of the wall is prevented by supporting arrangement made for the top slab using rollers as shown in F.29-2. The horizontal load was applied at a quasi-static rate in displacement controlled cycles with a hydraulic actuator having 500 KN capacity through a load cell. Displacements were applied in incremental fashion. Initially one cycle each of 2 mm and 4 mm peak displacement were given till cracking occurred. Then three cycles each of 6 mm, 8 mm, 10 mm, 15 mm, 20 mm, 25 mm, 30 mm, 40 mm peak displacements were given in the loading programmes used for the tests in terms of horizontal load v/s cycles. Strain gauges were used for measuring strains at 16 locations on vertical reinforcement. Extensometers were used at seven locations on the surface of concrete for monitoring crack openings. Laser sensors were used in monitoring the in-plane horizontal displacements at top slab level (height 1560 mm above the foundation) of the wall. In order to ensure that the foundation is fixed to the laboratory’s strong floor, vertical displacement of the top of the foundation was measured. The measured values of load, horizontal displacement and strains were recorded by a computer data logger.

Results of the Tests

For all three wall specimens, flexural cracks initially appeared near the bottom part of the tensile zone of the wall at a load of 80 kN and horizontal displacement of 3 mm. Yielding of the main reinforcement occurred at about 5 mm lateral displacement and strain in longitudinal reinforcement in the walls near bottom reached 2000 micro strain. For specimens subjected to cyclic load at the cyclic lateral displacement of 6 mm flexural cracks progressed and peak load of 155 kN was reached. As the lateral displacement approached 15 mm on ultimate load of 185 kN was reached and significant inclined cracks
Ultimate load = 185 kN
Failure load = 85% of ultimate load
= 155 kN

were formed (F.29-10). These cracks continued to penetrate deeply into the centre of the wall towards the compressive zone. These cracks diagonally progressed down and the crack pattern for the two specimens subjected to cyclic load is shown in F.29-10. At a lateral displacement of 40 mm, the width of the major flexural cracks already developed increased. At this displacement which was just prior to failure, few vertical cracks formed in the compressive zone and the concrete cover at the lower compressive edge of the wall spalled off, which finally led to failure of the compressive zone of the wall at 50 mm lateral displacement. The final failure of the wall occurs due to crushing of concrete in the compression zone of the wall and buckling of main reinforcement at the bottom between the two stirrups. This type of failure occurs at both sides at the bottom corners. F.29-10 illustrates the representative mode of failure of the walls which shows the buckling of the main reinforcement in between two stirrups.

F.29-11 illustrates the lateral load versus top displacement curve established from the tests for one of the specimen subjected to cyclic load. It is observed that the average secant stiffness at yield was 30 kN/mm and was reduced at ultimate load. It is clear that the stiffness is gradually decreased for all specimens due to formation of cracks and yield of steel. The ultimate load of the wall is 18.5 tons and the ultimate drift obtained is 50 mm from tests. Displacement at failure load of 85% of ultimate is 50 mm. The displacement ductility of the wall which is the capacity of the wall to deform beyond its elastic limit is obtained as 10 from the experiment. The force displacement curve obtained from FE analysis is compared with that obtained from experiments in F.29-11. It shows that the experimental and analytical load displacement curve results are in good agreement. However the instabilities, such as rebar buckling, are typically not considered in models. Hence the exact failure displacement of the wall obtained in experiments is not captured in analysis.
29-6-4 Discussions and Inference

The crack patterns and failure modes of the walls indicate that the wall capacity is affected by the flexural as well as the shear strength of the specimen. This caused considerable reduction in the strength, stiffness and energy dissipation of the specimens. The analysis force displacement relation predicts the test accurately up to a horizontal deflection of 25 mm. From the experiments, the ultimate load is obtained as 185 kN and ultimate drift is obtained as 1.6% which is more than the drift for collapse prevention performance level of 0.75% stated in [FEMA-356, 2000]. An analysis of short shear wall tested in WP1 tests is carried out and evaluation force displacement characteristics are presented henceforth.

29-7 Analysis of Stiff Shear Wall (IRIS – WP1 Tests)

29-7-1 Finite Element Model

In complex 3D-RC shear wall slab structures, concrete with material non-linearity is modelled using four noded iso-parametric 2D plane quadrilateral elements for wall, top and bottom beams. Steel is modelled in discrete form of truss element for wall and smeared form in the top and bottom beams. The FE model of a shear wall with a width of 3 m, a height of 1.2 m and a thickness of 0.4 m is made. The RC wall has been investigated through 2D non-linear FE analyses. Concrete strength $f_{ck}$ is considered as 54 Mpa. Steel strength is 500 Mpa. During the experiment, the wall was fixed at the base and the test was conducted for pure shear case, however, vertical deformations were allowed to take place. Also the axial load applied on the wall was 60 tons.
29-7-2 FE Analysis

The exact experimental boundary conditions are simulated in the analysis and the realistic constitutive model described before is used for the concrete. The model is fixed in the horizontal direction and vertical direction at all the nodes of bottom beam. The nodes of the top beam are free. The horizontal displacements are applied at the four nodes of the top beam in incremental fashion (displacement controlled analysis). The vertical load is applied on the wall on junction of the top beam and wall along line so that it creates the same compressive stresses in the wall when the uniformly distributed load \((V = 60 \text{ tons})\) is applied on the top beam. Horizontal displacements are applied at the top slab incrementally in cyclic manner. Steel plates with less (fictitious) density are attached to three sides of the top slab so that there is no local failure in the top slab due to point load application. The extension of the steel plates as present in experiments is modelled. Non-linear analysis is carried out for cyclic load on the wall with constant axial load of 60 tons. Analysis is carried out according to the loading cycles given in the experiment till failure of the wall, i.e. the load drops to 85 % of ultimate peak load. The shear displacement v/s steps and shear force taken by wall v/s steps is shown in F.29-12.
29-7-3 Discussion of Results

The comparison of experimental and analytical load displacement diagram for monotonic load and cyclic load is shown in F.29-13. The comparison of experimental and analytical vertical displacement at the top ends of the top beam v/s the shear displacements of the wall are plotted in F.29-14. The vertical displacement of the wall at failure is obtained from analysis as 14 mm at failure (i.e. at shear displacement of 17.5 mm). The experimentally obtained vertical displacement is 16 mm at failure. Thus there is twelve percent difference in the analysis and actual experimental vertical displacement. However, the overall nature of the graph showing the relation of horizontal and vertical displacement from analysis is in good agreement with the tests.

From the analysis it is seen that the first cracks with a crack width of 0.16 mm are observed at 0.8 mm shear displacement and 300 tons horizontal load of the wall. The same is observed in the tests. Analysis showed that at 3.0 mm shear displacement of the wall the longitudinal steel reaches yield strain of 0.002 and maximum principle tensile strain of 0.0065 is reached in the wall. It is observed that at 8 mm shear displacement the wall yields with constant peak load of 7000 kN. Most of the longitudinal reinforcement yields at this load and the maximum principal tensile strain obtained in wall at ultimate load is 0.012 as obtained from analysis. The final failure of the wall is due to concrete crushing after the reinforcement in the wall yields. The maximum strain is observed in the longitudinal steel near the bottom side location. The crack pattern obtained from analysis at the failure of the wall is shown in F.29-15. The maximum crack width at failure is 3.7 mm. The crack pattern observed at the end of the WP1 test is shown in F.29-15. The peak load and displacements from analysis and tests are in good agreement at each step.

However the crack pattern in 2D analysis is not in complete agreement with the test. 3D analysis of the shear wall is then performed for monotonic load using 3D solid wedge elements. The crack pattern obtained in 3D analysis and the principal tensile strain distribution is shown in F.29-16. It is observed that the crack pattern in the wall at failure obtained from 3D analysis is in good agreement with the test results. The force displacement diagram for monotonic loading from 2D analysis, 3D analysis and experiments is compared and is found to be in good agreement. It is observed that 3D analysis monotonic loading gives a fair estimate of crack pattern in the wall and the force displacement diagram.

---

Crack pattern in the wall at failure: 2D analysis (left) and experiments (right)  F.29-15
29-8 Fragility Analysis of Mid-Rise Shear Wall

A schematic of the shear wall is shown in F.29-17. The estimation of the maximum load for failure in the shear wall under discussion can be done as follows. Considering the state of stress (horizontal load $V_u$, vertical load $P$) in which the concrete at the bottom is under crushing strain ($e_u$). The strains in the reinforcement bars is shown in F.29-17. The location of the neutral axis ($x_u$) is found iteratively by solving E.29-11.

\[
P = 0.54f_{ck}Bx_u + \sum A_{st,i}f_{sc,i} - \sum A_{st,j}f_{st,j}
\]

\[
P = \text{vertical load}
\]
\[
f_{ck} = \text{crushing strength of concrete}
\]
\[
B = \text{width of the section}
\]
\[
x_u = \text{location of neutral axis}
\]
\[
A_{st} = \text{area of the reinforcement cross section}
\]
\[
f_{sc,i} = \text{stress in the } i\text{-th reinforcement bar under compression}
\]
\[
f_{st,j} = \text{stress in the } j\text{-th reinforcement bar under tension}
\]

Once the location of neutral axis is obtained, the maximum horizontal load ($V_u$) the wall can withstand is estimated by moment balance. Thus the limit state for structural reliability calculation is given by E.29-12.

\[
g = V_u(x_u) - V
\]

\[
V = \text{applied shear force}
\]
\[
V_u(x_u) = \text{shear capacity of the shear wall, function of variables } x_u
\]
\( x_1 = f_{ck} = \) crushing strength of concrete
\( x_2 = e_{uc} = \) maximum strain in concrete under compression
\( x_3 = a_s = \) area of the reinforcement bar
\( x_4 = P = \) vertical load on the shear wall
\( x_5 = \) width of the wall
\( x_6 = \) length of the wall
\( x_7 = \) height of the wall
\( x_8 = \) stress-strain curve of the reinforcement bar

The variables treated as the random variables are \( f_{ck}, e_{uc} \) and \( a_s \) for estimating the fragility of the shear wall. The definition of these variables is given below:

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
<th>Mean</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{ck} ) [N/mm²]</td>
<td>Lognormal</td>
<td>47.49</td>
<td>4.96</td>
</tr>
<tr>
<td>( e_{uc} )</td>
<td>Lognormal</td>
<td>0.0035</td>
<td>0.00063</td>
</tr>
<tr>
<td>( a_s ) [mm²]</td>
<td>Normal</td>
<td>157.1</td>
<td>7.855</td>
</tr>
</tbody>
</table>

The distribution for the crushing strength is estimated from the actual tests conducted on the concrete specimens. Total data is available for 43 samples. The goodness of fit testing showed that the lognormal distribution is acceptable for these data. A distribution plot of the data is shown in F.29-18.

The distribution and the coefficient of variation for \( e_{uc} \) is the same as that considered by [Klein and Henkel, 2011]. The distribution of \( a_s \) is assumed. The scatter in the values of the tensile properties of the steel reinforcement bar is not considered. Full stress-strain curve of the steel is used for estimating the limiting value of the shear load (F.29-19).

The scatter in the stress-strain curve can be modelled if it can be fitted in form of a power law, e.g. [Rahman, 1997]. In this case, a good fit in these terms could not be obtained, hence it is considered as a deterministic design variable.
The confidence levels on the fragility curves are obtained by considering the applied shear force to be a lognormal random variable with a coefficient of variation of 20%. Two methods are used to estimate the fragility curves.

Method 1

This method is similar to the method described in [EPRI, 1994]. This method is known as scaling method and assumes lognormal distribution for all random variables. The probability of failure for the given value of shear force $V$ is given by

$$P(V) = \Phi \left( \frac{\ln \left( \frac{V}{V_{u,\text{med}}} \right)}{\beta_{Vu}} \right)$$

where:
- $\Phi(\cdot)$ = standard normal distribution function
- $V_{u,\text{med}}$ = median value of the capacity of the wall
- $\beta_{Vu}$ = logarithmic standard deviation of $Vu$
The median value of $Vu$ is estimated by keeping all variables at their median values. The $\beta_{Vu}$ is estimated by the following procedure.

For each $x_i$, if the $x_i$ is a strength variable, $\beta_i$ is estimated using $E.29-14$

$$
\beta_i = \ln \left( \frac{Vu_{med}}{Vu(x_i - \sigma_i)} \right)
$$

$\hat{x}_i = \text{median value of } x_i$

$\sigma_i = \text{standard deviation of } x_i$

If $x_i$ is a demand variable then $\beta_i$ is estimated using $E.29-15$.

$$
\beta_i = \ln \left( \frac{Vu(x_i + \sigma_i)}{Vu_{med}} \right)
$$

$\hat{x}_i = \text{median value of } x_i$

$\sigma_i = \text{standard deviation of } x_i$

The $\beta_{Vu}$ is obtained from $E.29-16$.

$$
\beta_{Vu} = \left( \sum \beta_i^2 \right)^{1/2}
$$

The confidence bounds are obtained using the 20% coefficient of variation on the $V$ which is assumed to be lognormal. $F.29-20$ shows the fragility curve obtained using the scaling method.

The failure load obtained from tests is 18 tonnes. This corresponds to a probability of failure of 0.1 on the median curve. If the estimation model was perfect i.e. without any modelling uncertainty, then this should correspond to a probability of failure of 0.5 on the median curve. These curves help in arriving at the design value for $V$ for this shear wall. The design value used is High Confidence Low Probability of Failure (HCLPF). This value corresponds to the 95% confidence of a 5% probability of exceedance. Thus, the design value is arrived for a value of $V$ on curve 1, which corresponds to 5% probability of failure. This is shown in $F.29-21$. The HCLPF value estimated is 12.74 tonnes.

Method 2

The probability of failure is obtained using the Monte Carlo simulation method. The random variables $f_{ck}$, $e_o$ and $\alpha_{st}$ are simulated and then entered in $E.29-7$ for the given value of $V$. A count is maintained of the failure. This simulation is performed a number of times to estimate the probability of failure. The error in probability estimation is kept below 10%. Structural reliability software COMREL [STRUREL, 2003] was used to estimate the probability of failure. The fragility curve for the median value, 95% confidence level and 5% confidence level was estimated. The fragility curve is shown in $F.29-22$. 
Assessing Safety of Shear Walls: an Experimental, Analytical and Probabilistic Study

Fragility curves using scalability method

![Fragility curves using scalability method](image)

Estimation of design load using HCLPF value

![Estimation of design load using HCLPF value](image)

A comparison of the two methods for the median value of $V$ is shown in F.29-23. It is seen that there is no significant difference in the curves obtained from the two methods. The HCLPF value estimated is 12.83 tonnes.

29-9 Conclusions

The shear wall is designed for an earthquake load of 0.2 g peak ground acceleration using the IS code. The top mass of 16.5 tons lumped on the shear wall is subjected to 0.4 g horizontal acceleration and the horizontal load at the top of shear wall will be about 7 tons. Thus deterministically the shear wall is designed for a horizontal load of 7 tons. However it is observed analytically and experimentally that the shear wall carries a load up to 18.5 tons. Thus the over-strength factor available for the wall is 2.65.

This reliability based model is applied to the shear walls used in the nuclear industry. The scaled shear wall is modelled analytically as well using the Finite Elements. The uncer-
tainty in the design parameters is modelled using the probability distribution function. The model is validated by conducting scaled tests on this wall. The uncertainty propagation across the modelled behaviour is performed using the principles of structural reliability. The outcome of this estimation is the fragility curve. This fragility curve is used to estimate the safe design load for the shear wall. This load for the shear wall studied is 12.8 tons. This the design load is arrived using the sound logic of probabilistic analysis rather than a deterministic factor of safety given in the design code. This study suggests a basis for designing shear walls based on the probabilistic principles.

Non-linear finite element analysis of shear wall tested in WP1, IRIS project is described in detail. The shear wall exhibited ductile load deformation response. Analysis showed that the failure mode involved crushing of concrete occurring after yielding of longitudinal reinforcement. The load deformation characteristics of tests and analysis are also in good agreement with each other. The crack pattern obtained from 3D analysis is in good agreement with test results. In general the analysis considering accurate modelling of compression softening phenomenon of the concrete gives precise representation of the experimental results.
References

ATENA, 2006. FE Software for Nonlinear Analysis of Concrete Structures.


