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Dynamic nonlinear 2D and 3D analysis of RC shear wall under seismic loading

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ABSTRACT

In many nuclear power plant buildings, reinforced concrete (RC) shear walls are used to act as primary lateral force resisting systems. Under the auspices of OECD, the Nuclear Power Engineering Corporation of Japan (NUPEC) experimental test program (1991) provided valuable experimental data [1] on the dynamic behaviour of shear walls up to their ultimate state. The objective of this study is to evaluate through comparison with experimental results, the performance of various FEM analytical techniques and modelling assumptions.

1. EXPERIMENTAL TEST PROGRAMME

The NUPEC shear wall specimen [1] consist of an « H » shaped wall system (Figure 1). The web wall was 2020 mm clear height, 3000 mm long from flange wall centre to centre, 75 mm thick. The flange walls (2980 mm long and 100 mm thick) provide stability for the web from out-of-plane action and increase the flexural capacity of the shear wall. Normal concrete with coarse aggregate having a maximum grain size of 10 mm was used for the web and flange walls. The web wall and the flange walls were densely reinforced vertically and horizontally with two layers of reinforcing bars. Additional mass was added to the top slab of the specimen in order to simulate the appropriate gravity load in the wall. The total weight placed at the top of the specimen (including top slab) was 122.0 tonf. The dynamic testing involved subjecting the test specimen to 6 sequentially increased horizontal accelerations along the axis of the shear wall, on a shaking table up to collapse of the specimen. A single acceleration record was scaled to the desired seismic input level (Table 1) to obtain the response characteristics of the shear wall, ranging from the elastic to the ultimate state.

TABLE 1

EXCITATION	MAX HORIZONTAL ACCELERATION
RUN1	0.054g
RUN2	0.114g
RUN2D	0.310g
RUN3	0.359g
RUN4	0.588g
RUN5	1.255g

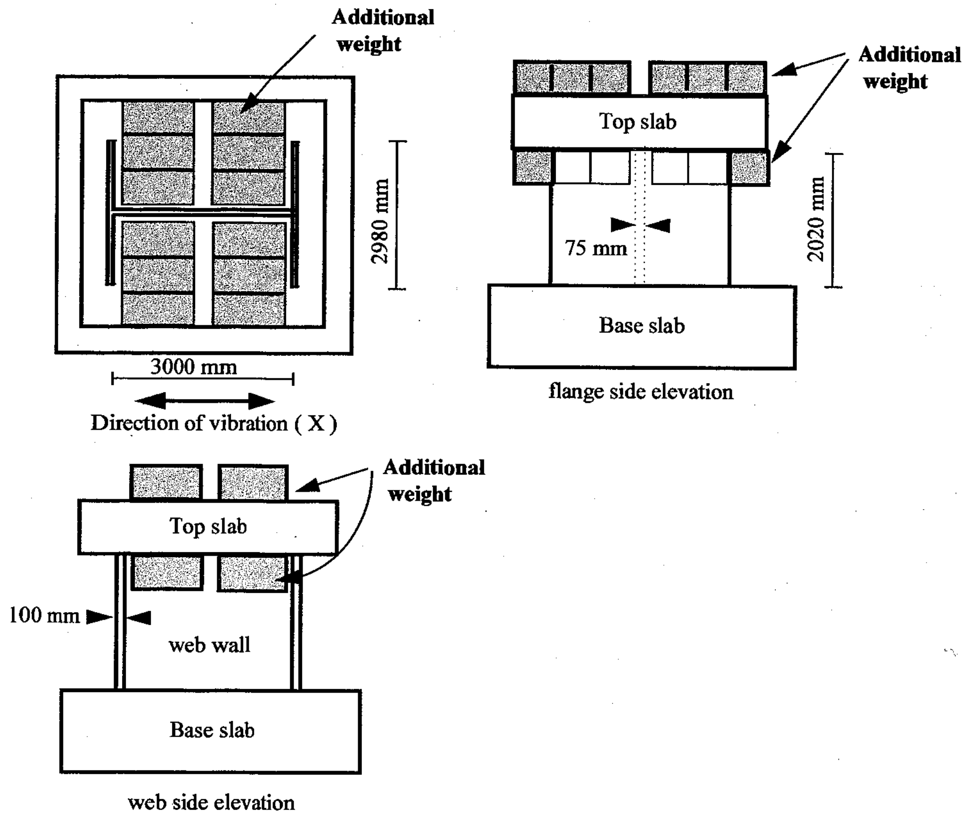
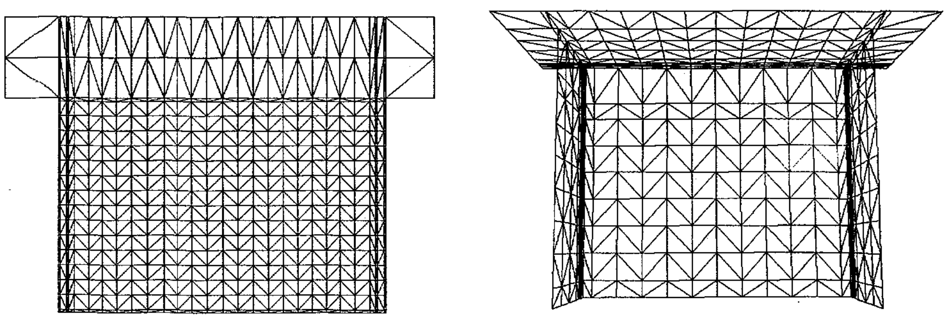


Figure 1 : NUPEC Dynamic test specimen geometry and additional mass [1]

2. ELASTIC STUDY

An initial parametric seismic analysis was conducted at a low level of excitation (RUN1) to better evaluate the selected structural models, the sources of approximation, the problem of initial material state, and the modelling of the vertical seismic excitation. Two different FEM analysis approaches were undertaken as part of this study: (1) a two-dimensional (2-D) representation of the NUPEC specimen and (2) a three-dimensional (3-D) representation of the same structure. The 2-D model represents the different parts of the specimen (top slab, web wall and flanges) as an equivalent plane mesh, while the 3-D model consists of a spatial assembly of the different parts of the structure. Taking into account the symmetry of the structure in relation to the direction of the seismic input, only half of the specimen is considered in the 3-D representation. Both finite element meshes are presented in Figure 2. The actual geometric dimensions of the different parts of the experimental specimen were used in the 2-D and 3-D approaches. The weight of the additional mass, top slab and wall is considered as concentrated masses in the nodes of the structure. In accordance with the construction details of the specimen, boundary conditions for all FEM models included the assumption that all the degrees-of-freedom (DOF) along the base of the shear wall were completely fixed. The 2-D model is made up by 1016 three-node plane stress elements representing the concrete and by 790 bar elements to represent the horizontal and vertical reinforcing bars. The 3-D model incorporates 732 thin shell DKT elements to represent the concrete and 722 bar elements to represent the reinforcing bars. The damping matrix for both models has been built to yield a 1% damping on the first two elastic modes, in accordance with the experimental measure of the critical damping determined on the « virgin » structure.



2-D Model 3-D Model

Figure 2 : 2-D and 3-D FEM meshes

The input artificial motion applied to the specimen consisted of three components: a principal horizontal acceleration and two vertical accelerations at right and left sides of the base slab. Due to the performance of the table driving system and interaction between the table and the specimen, vertical and rocking vibrations occured during the test. These vertical components, represent 15.2% to 38% of the maximum horizontal acceleration, for all the six input motions applied to the specimen and could have an influence on the dynamic response of the specimen. Another aspect to be noted is that test results of concrete material indicated some scattering of the concrete initial Young's modulus. In order to quantify the influence of the vertical excitation and to better estimate the initial concrete elastic modulus, four different cases were considered through a parametric study. These are summarised in the following table

TABLE 2

CASE	CASE 1	CASE 2	CASE 3	CASE 4
E_c (MPa)	23400	23400	22800	22800
Excitation	Horiz. Acc	Horiz.Acc Vert.Acc	Horiz.Acc	Horiz.Acc Vert.Acc

From the time - history analyses (four cases and two models, giving in all 8 analysis cases) the following points stand out:

- As expected at this stage of low level of seismic intensity, no damage was detected by the calculations and the predicted elastic behaviour is in agreement with that experimentally observed during RUN1.

- The predicted response is in closer approximation with that experimentally measured when considering horizontal, vertical and rocking motions in the analysis, as compared to the cases when only horizontal acceleration is considered.
- The decrease in concrete Young's modulus produces more favourable results. The max. horizontal displacement calculated for CASE 2 and corresponding to 3-D model represents 78% of the max. experimental value. The same case but corresponding to the 2-D model gives a percentage of 62% for the same item. Passing to CASE 4 the predicted response approaches the experimental one for the 3-D analysis, the maximum displacement representing 90% of that measured experimentally, while the 2-D model gives the same results as in CASE 2.
- As the 3-D model can automatically consider the effect of spatial bending of the flanges it gives a more flexible response as compared to that predicted by the 2-D model. This is illustrated in TABLE 3 where the natural frequencies obtained for the 2-D and 3-D models using the lowest value of the concrete Young's modulus (22800 Mpa), are compared to the test value.

TABLE 3

NATURAL FREQUENCY (Hz)		
EXPERIENCE	3-D MODEL	2-D MODEL
13.20	14.00	14.50

The results of this parametric elastic study confirme the choice of the lowest value of the Young's concrete modulus indicated by test results and the need of considering the vertical and rocking input motion in the seimic analysis. It is also necessary to evaluate the quality of the results obtained using a 2-D model, espacially in the non-linear range, by comparing them with the 3-D results.

3. 2-D SEISMIC NON-LINEAR ANALYSIS

3.1 *Input Motion*

The time-history analysis of the NUPEC shear wall was conducted using the 2-D FEM already developed for the previous elastic analysis. In combination with the ability of the CASTEM2000 code to save the damage state after each analysis, the following five artificial motions were considered in chronological order: RUN2, RUN2D, RUN3, RUN4, RUN5. For each input motion the horizontal as well as the vertical and rocking acceleration components were considered in modelling the input seismic motion.

3.2 *Material Constitutive Models*

The concrete model used here was developed at INSA de Lyon [2] and is based on the plasticity theory. The concrete is assumed as a softening material both in tension and in compression with a correct description of the unilateral opening and closing of cracks throughout the loading cycles. The model considers the permanent strains, the stiffness degradation and the stiffness restitution when the crack closes. The elasto-plastic strain hardening model developed for uncracked concrete is based on the four-parameters Ottosen's failure criterion with isotropic hardening and associated flow rule. The yield criterion is assumed on the basis of the known failure criterion by selecting it as a proportionally reduced shape of the failure surface and an associated flow rule is employed. For the concrete in tension a smeared fixed crack approach is considered. With this formulation the cracked concrete is treated as an orthotropic material with principal axes normal and parallel to the crack direction. Uniaxial behaviour is considered in these two directions according to Figure 3. Rough crack behaviour is incorporated in the model transmitting shear forces across the crack. According to the constitutive cyclic law for concrete presented in Figure 3, as soon as a crack starts to close, the concrete develops some compression, due to the imperfect overlapping of the crack surfaces. Furthermore the model considers damage of the elastic modulus and of the tensile resistance as the inelastic compressive strains increase.

1 - Elastic tension
2 - Crack opening
3, 8 Crack closing
4 - Nonlinear compression
5, 11 - Damaged unloading, $E_2 \neq E_0$
6 - Damaged unloading. Modulus = E_1
7 - Reopening of crack
9 - Reloading: Linear compression
10 - Softening behaviour in compression
12 - Elastic tension with resistance $f'_c < f_t$

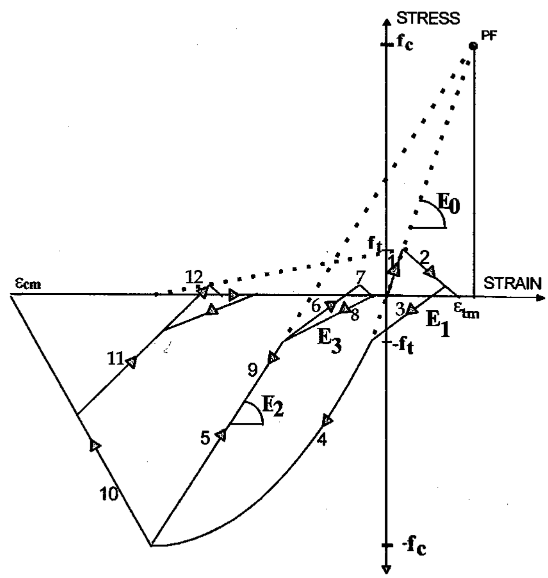


Figure 3 Concrete cyclic law

The stress-strain relationship of reinforcement was considered as for an isotropic hardening material. The same law was assumed for all the reinforcing bars in the web wall and in the flanges. Perfect bond is assumed between concrete and steel in the calculations.

3.3 *Damping*

The critical damping ratio on the first mode was determined experimentally, for each « RUN » by performing small amplitude vibration test immediately before each vibration step. The measure of the critical damping increased from slightly more than 1% to approximately 4%. The damping values measured for low level of seismic intensity (RUN1 and RUN2) were adopted without modification in the analysis. However, for the motions of greater intensity, as non-linear behaviour of the specimen was observed, these values have been reduced. This is due to the fact that measured critical damping contains implicitly some « extra-viscous » damping, corresponding to the damage accumulation within the specimen

during the previous RUNS. As rough crack behaviour is incorporated into the concrete model used herein [1], this model takes into account in a certain way, by hysteric dissipation the friction between the lips of the crack. A Rayleigh damping proportional to the mass and stiffness matrices was adopted in the calculations, using the following damping ratios:
RUN2: $\xi = 1\%$, RUN2D: $\xi = 2\%$, RUN3: $\xi = 2.5\%$, RUN4: $\xi = 2.5\%$, RUN5: $\xi = 2.5\%$

3.4 *Results*

Figure 4 shows a comparison between the predicted response generated by the time - history 2-D FEM analysis and that experimentally measured. The comparison is presented for 2.5 sec for the time interval where the horizontal displacements reach the most significant values.

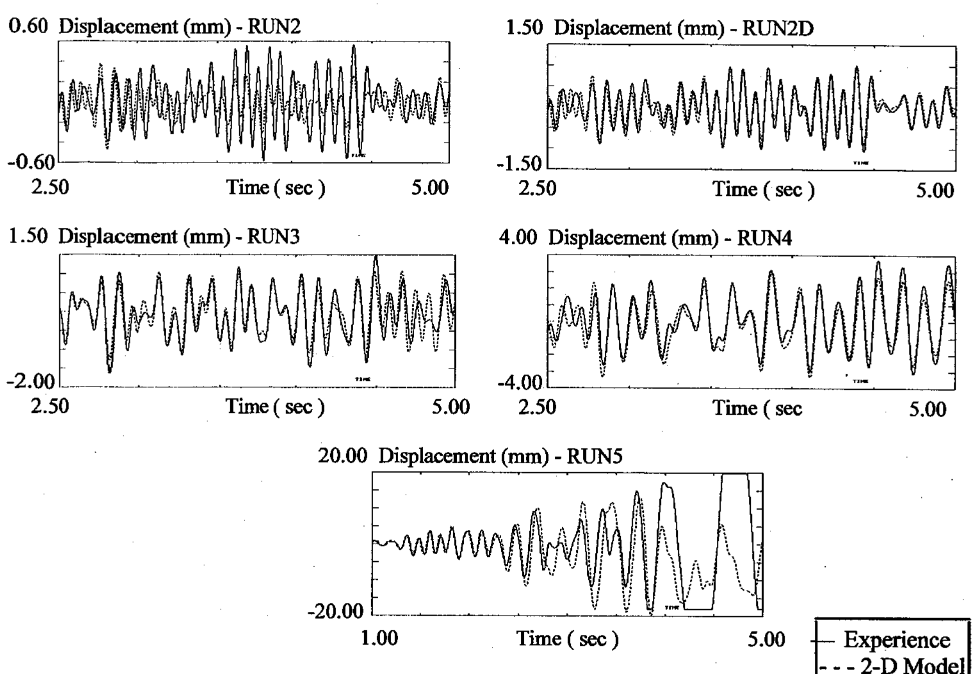
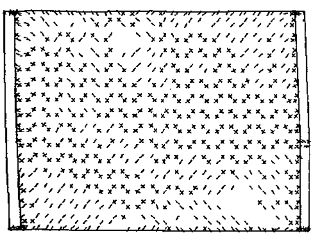


Figure 4 : Time-history of horizontal top displacement

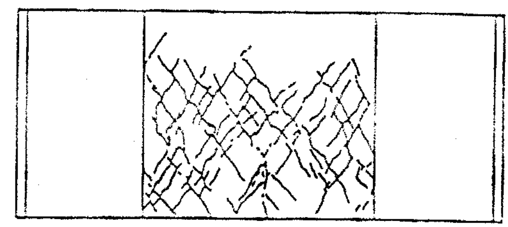
From this time - history analyses the following points stand out:
- For a low level seismic motion (RUN2) the difference in amplitude between calculated and experimental response is greater than that corresponding to higher intensity excitation level. As no damage was detected for the 2-D FEM subjected to input motion RUN2, while some cracks at the base of the web wall were observed experimentally for this RUN, the difference in behaviour may come from misreading the precise initial material state.
- As the degree of degradation increases (RUN2D, RUN3 and RUN4) a very good agreement in amplitude and frequency is obtained between calculated and test results.
- For the highest excitation level (RUN5) the calculated shape of the displacement curve agrees well with that determined experimentally up to approximately 2.5 sec. After 2.5 sec as

collapse of specimen approaches, the 2-D model is not able to reproduce the experimental results.

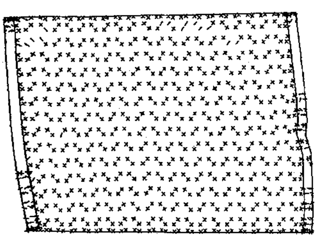
Figure 5 shows the predicted crack pattern (all the cracks appeared during RUN4 and RUN5) as compared to the visual observation of cracks after RUN4 and RUN5. It can be seen that in both cases a good prediction of the distribution and orientation of the cracks in the web wall was obtained. However, cracks in the flanges are not so well predicted due to the inherent two-dimensional behaviour of the flanges assumed by the 2-D model.



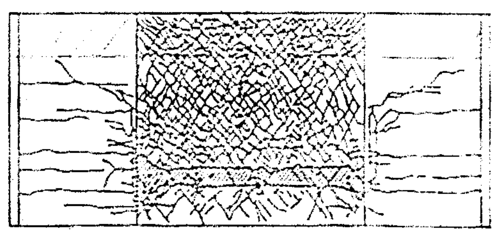
RUN4 - 2-D Analysis



RUN4 - Experience



RUN5 - 2-D Analysis



RUN5 - Experience

Figure 5 : Crack patterns for 2-D FEM analysis

3. COMPARATIVE 3-D AND 2-D ANALYSES

A supplementary 3-D FEM time-history analysis of the NUPEC shear wall was conducted for comparison purposes. The concrete model and material characteristics were the same as used in the two-dimensional calculations. Due to the limits on the use of computer time only RUN2D, RUN4 and RUN5 were considered in the analysis. Global results indicate that there is not much difference between 2-D and 3-D top displacements. For RUN5 after 2.5 sec, as collapse approaches, both models give approximate displacements. Figure 6 shows a comparison between some local results, the predicted strain time history of a reinforcing bar at the base of the flange generated by the FEM analyses and that experimentally measured. It can be seen that the 2-D model gives an unrealistic prediction of the steel strain, while the 3D model generates closer results to that experimentally measured. The crack pattern presented in Figure 7 indicates also that the 3-D model is able to produce good crack prediction in the web wall, but a less accurate crack prediction in the flanges.

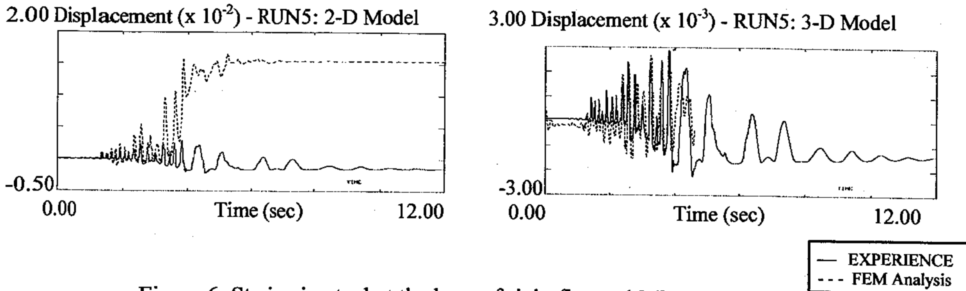


Figure 6 :Strains in steel at the base of right flange (2-D 3-D Comparison)

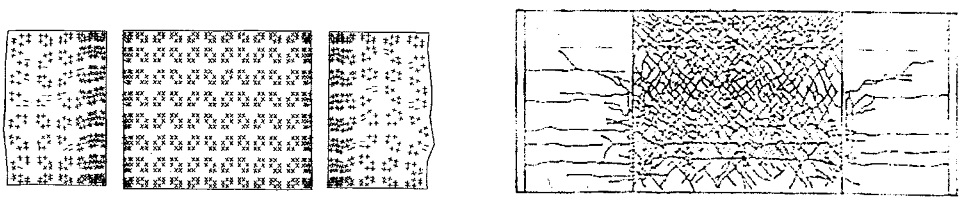


Figure 7 : Crack Pattern- RUN5

4. CONCLUSIONS

The numerical results indicate that the concrete model used herein (with a 2-D and 3-D approach) is able to simulate rather realistically the behaviour of shear walls under dynamic loading actions, before collapse of the specimen. The concept of fixed cracked model seems to be effective in the case of NUPEC shear wall, as it leads to crack directions that are consistent with the experimental observations. Global results as top displacements, are not very much influenced by the dimension of the modelling: 2-D and 3-D models show similar behaviour. However, near failure, the 3D effects can be very dominate (flange action) and the 2-D models will generally be inadequate. The failure mode - shear sliding failure which occurred at the bottom of the web wall was not demonstrated by the 2D and 3-D FEM models. It is to be mentioned that phenomena possibly appearing at failure stage as dowel action, bond slip and buckling of rebar, etc. were not considered. Further improvements are necessary in concrete material modelling, so that finite element model could specify realistically the failure location.

5. REFERENCES

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