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DYNAMIC RESPONSE OF LIGHTLY REINFORCED CONCRETE WALLS

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SUMMARY

There are many lightly reinforced concrete walls that were constructed in buildings and even in nuclear power plant installations in various countries. It is expected that these walls will behave in a nonductile manner during severe earthquakes. In order to rehabilitate this type of wall, it is necessary to evaluate the behaviour and determine its load carry capacity during moderate and major seismic events.

The objective of the investigation is to determine the response of typical lightly reinforced walls when subjected to scaled ground motion records up to failure and to establish the load carrying capacity and ductility of the walls. The wall was modeled using six node two dimensional panel elements. The panel elements have lumped Flexural/axial plasticity at their top and bottom fibre sections. Nonlinear static and dynamic time history analyses were conducted. The response of the wall was evaluated in terms of pushover, spectral, displacement-based, and time-history analyses.

The analytical results indicated that the wall behaved in a nonductile manner with brittle shear failure. The model and the response data were verified against available measurements from a test program conducted using a shake table. The comparison indicated that the model closely represented the behaviour observed in the test.

INTRODUCTION

Reinforced Concrete (RC) structural walls are used extensively in medium-to-high rise buildings and in nuclear power plant structures. It has been observed that lightly reinforced walls behave in a nonductile manner during severe earthquakes. The behaviour of structural walls near collapse is complex and has been difficult to predict. Due to the limited available research on the seismic response of structural walls, several modeling problems such as nonlinear shear model, including bending moment-shear-force-axial force interactions are yet to be solved.

Reinforced concrete bearing walls with low vertical reinforcement ratios of less than 0.2% are referred to as lightly reinforced walls. Recently, Eurocode 8 and the French code PS 92 adopted a design concept for lightly reinforced concrete walls based on the multifuse principal favouring rupture occurrence at several storeys. This design leads to lower reinforcement ratios with their optimized distribution allowing wide

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cracks to take place with large energy dissipation potential. In addition, the vertical displacement of the mass results in energy transformation from kinematic to potential.

Nonlinear models that are capable of predicting the behaviour of structural walls to failure are needed tools for the effective design of walls and vulnerability evaluation. An effective approach to verify the accuracy and efficiency of the computational tools in predicting the seismic response of wall structures is by comparing the analytical predictions to results from shake table tests. Several interesting aspects of the seismic behaviour of lightly reinforced walls have been experimentally studied within the scope of the CAMUS research program. CAMUS experiments refers to two –1/3rd scale– reinforced concrete bearing wall specimens (Figure 1) that were tested using the Azale shake table at the Commissariat à l'Energie Atomique (CEA) facilities in Saclay, France since 1996.

The wall in the test CAMUS I [1] was subjected to two types of input motion using a shake table. The Nice input motion is an artificial ground motion representative of a far field motion. The San Francisco input motion is an actual record representative of a near field record. Figures 2 and 3 show the two records and their acceleration response spectra. The records were modified because of the geometrical scale (time scale divided by √3). Four tests were conducted: two tests using the Nice record scaled to peak ground acceleration (PGA) of 0.24g (RUN1) and 0.41g (RUN4) and two tests using the San Francisco record 0.13g (RUN2) and the record scaled to 1.11g (RUN3). RUN1 can be considered as a typical design input motion. RUN1 and RUN2 can be regarded as low-level inputs while RUN3 and RUN4 are damaging level inputs.

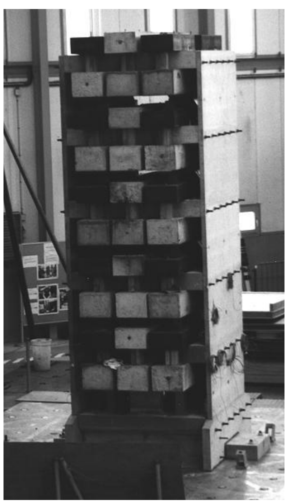


Figure 1 CAMUS test walls [1]

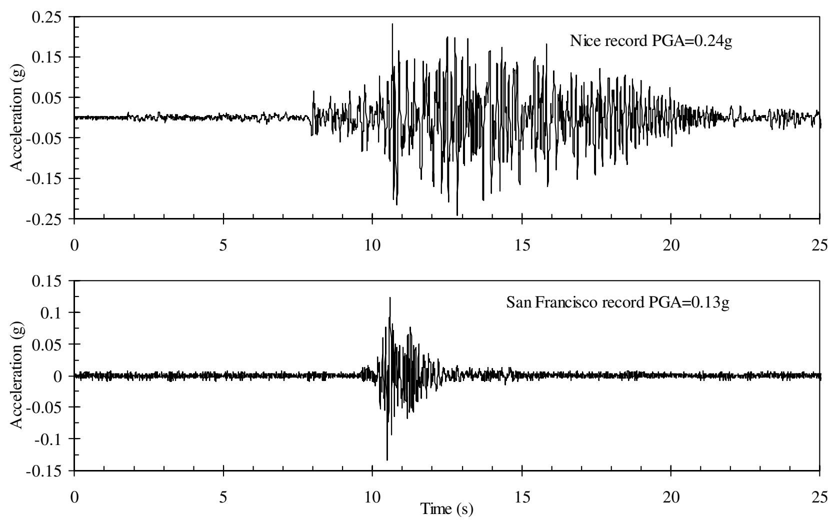


Figure 2 Measured accelerations on top of the shake table for the analysed runs

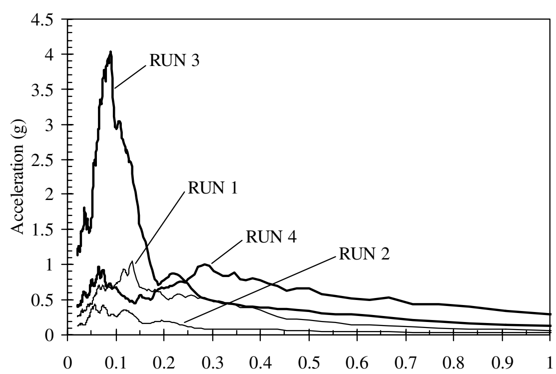


Figure 3 Response spectra for the four experimental runs.

In this study, an analytical model was proposed to investigate the behaviour of lightly RC bearing walls subjected ground motions up to collapse. The objective of this investigation is to examine the validity of the model assumptions and parameters used. The analytical predictions are compared to the measured dynamic response of CAMUS I test [1].

APPROACHES TO ANALYSIS

The tested walls were analyzed using nonlinear static pushover, linear spectral, displacement-based design, and nonlinear dynamic time history procedures. The CANNY99 program [2] was used for the pushover analysis. Nonlinear fibre type panel elements were used to model the wall. The section was discretized into steel and concrete fibres with uniaxial nonlinear properties that are determined on the basis of the wall models tested.

A conventional linear modal spectral analysis was conducted using the SAP2000 program [3] to represent a simple design procedure. This linear analysis is valuable in determining the appropriate structural reduction factor when compared to a nonlinear procedure such as the pushover or the dynamic time history analysis. The concept of capacity spectrum is applied to estimate the peak ground acceleration levels of the given ground motion that produce a specified target displacement. The procedure outlined in the ATC-40 [4], FEMA 273 [5] and FEMA 274 [6] was used. The estimated peak ground acceleration can be compared with the PGA used in the tests.

The nonlinear dynamic time-history analysis was conducted using the computer code CANNY99 [2]. The wall was modeled using macromodel panel elements where plasticity is confined at the element ends. Steel and concrete fibre elements having nonlinear uniaxial properties were used. Macromodels are widely used in the nonlinear dynamic analysis of large structures due to their computational efficiency and reasonable accuracy. A comparison between the top floor displacements of the wall calculated using a performance-based design and dynamic analysis was made. This analysis will provide verification of the applicability of the performance-based design capacity spectrum approach to lightly reinforced walls.

STATIC ANALYSIS

The pushover analysis has several advantages. The procedure is simple, does not rely on an estimate of the site-specific ground motion and can identify the critical regions where large deformations are expected. The analysis can verify the adequacy of load path considering all elements of the structural system and all connections [7,8]. Although the pushover analysis does not capture the time-deformation response, it captures only the essential features of the structure that significantly affect the total performance. The CANNY99 [2] computer program was used to conduct the pushover analysis for the tested wall. An inverted triangular load pattern was used. This pattern represents the first mode shape suitable for stiff structures. Adaptive load distribution was not included as the tested wall is stiff and the participation of higher modes is minimal.

Wall idealization

The analytical model of the tested wall is shown in Figure 4. The base of the wall is assumed fixed. The wall was idealized using five panel elements representing the wall's five floors. Each panel element is bounded by four nodes at its corners in addition to a node at the mid point of its top and bottom boundaries. The adjacent panels have compatible deformations at their common three nodes that are connecting them. The deformations of adjacent panels between their common three nodes are maintained equal through rigid links at their top and bottom boundaries. Each node has three displacement degrees of freedom: two translational displacements along the global axes X and Z, and one rotational in the vertical plane X-Z.

Member model

The panel element has six nodes; three on each of the top and bottom boundaries. The rigid links represent the assumption of plane section deformation of the panel element, keeping the adjacent panel elements deformation compatible along their common length. Fibre model is used to represent the flexural and axial tension/compression of the panel elements. It takes into account the coupling between the axial force and bending moment. The fibre element consists of a number of uniaxial fibres. Each single steel bar may be replaced by a steel fibre at the bar centre point. The concrete section may be discretized into a number of small areas with a concrete fibre placed at the centre point of each divided area.

The panel elements have lumped flexural/axial plasticity at their top and bottom fibre sections. Linear strain distribution between the top and bottom critical sections is assumed along the panel height. Each panel element was assigned fibre section properties at its top or bottom boundaries according to the reinforcement content and distribution at that section.

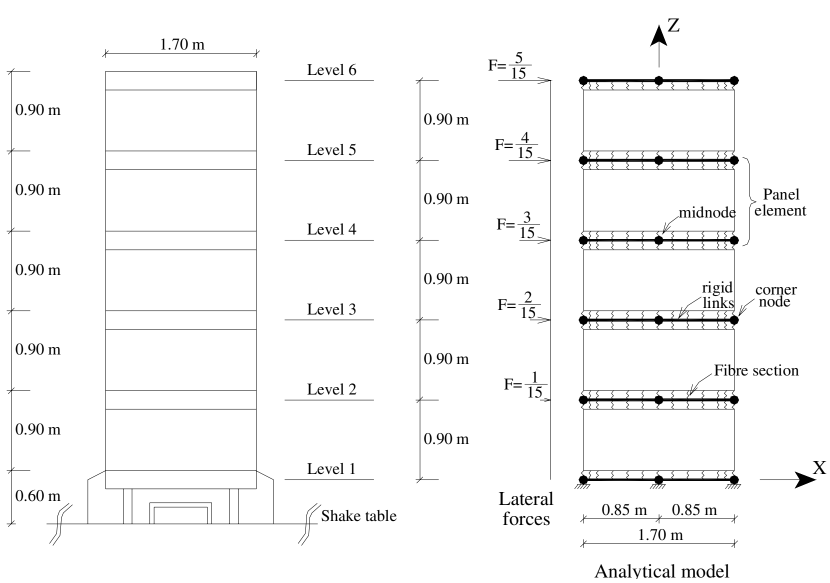


Figure 4 Idealization of the tested wall for the pushover analysis

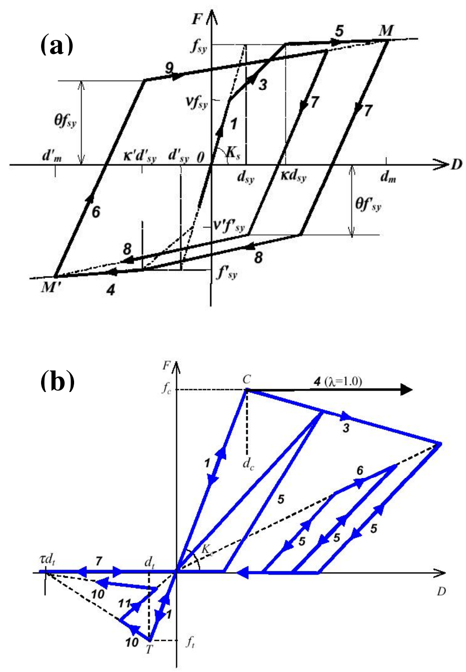


Figure 5 (a) Hysteresis model for steel, SS3; and (b) Concrete material bilinear model, CS2. [2]

Steel model

The trilinear/bilinear model SS3 shown in Figure 5a was used to represent the steel bars [2]. A bilinear skeleton curve with specified hysteretic parameters was used for the current analysis with the stress-strain properties provided for each steel bar. The parameters for the steel skeleton curve are listed in Table 1a.

Concrete model

The concrete behaviour was represented using the bilinear model CS2 [2] shown in Figure 5b. The model is simple with post peak degrading envelope and tension stiffening capabilities. The properties provided and the phases of casting concrete during the walls construction were accounted for. Each concrete section was divided into one hundred concrete fibre elements. The parameters for the concrete skeleton curve are listed in Table 1b.

Table 1a Parameters used for element SS3

Parameter	Value
Skeleton curve parameters υ and κ	1.0
Post-yielding parameter β	0.01
Parameter ϕ to direct unloading	0
Unloading stiffness degrading parameter γ	0.2
Unloading control parameter θ	0.75

Table 1b Parameters used for element CS2

Parameter	Value
Strain at maximum compressive strength	0.002
Compression post-peak residual/max capacity ratio λ	0.2
Ultimate strain / strain at maximum compressive strength ratio μ	1.75
Post-peak unloading stiffness parameter γ	0.2

Analysis results

Figure 6 shows the inelastic static pushover analysis force-displacement relationship for the wall. The maximum base shear was 126 kN at 45 mm top floor displacement, which represent 1% drift. From the pushover plot shown in Figure 6, the wall behaviour is well into the inelastic range with very small stiffness. The yield base shear was 75 kN corresponding to top floor displacement of 3.3 mm.

Figure 7 shows a comparison between the wall overturning moment-top level displacement relationships of the analytical (pushover) predictions and the experimental RUN3 measurements. From the figure it can be seen that the analytical moment capacity of the wall is higher than that of RUN3 by approximately 20%. This can be attributed to the difference between the boundary conditions of the wall base in the analysis model (fixed) and the test (partial flexibility exists due to the shake table supporting system).

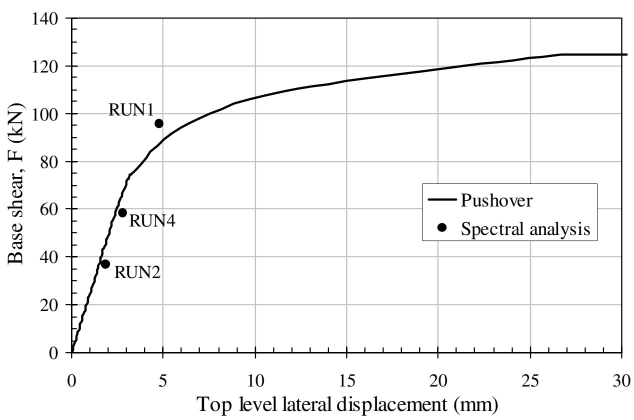


Figure 6 Static pushover analysis force-displacement relationship

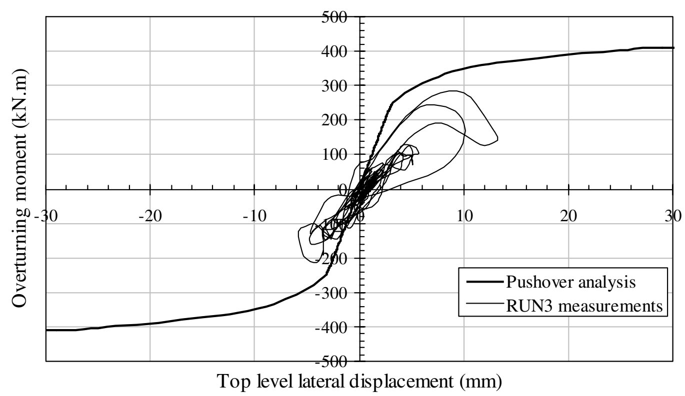


Figure 7 Comparison between the pushover and RUN3 (the measured) moment-displacement relationships

MODAL AND SPECTRAL ANALYSES

In the current application, spectral analyses were conducted on a finite element model for the tested wall using 5% damped acceleration response spectra for tests RUN1 to RUN4. The program SAP2000 [3] was used for the spectral analysis.

Figure 7 shows the finite element wall model. The wall was discretized using 4 node plain-stress shell elements representing concrete and 2 node bar elements representing steel. Each floor which is 900 mm high was divided into 9 layers, each having height of 100 mm. The wall width was divided such that the aspect ratio of shell elements were either 1 at the coarse mesh zones between the steel reinforcement locations, or 2 at the fine mesh zones at the vertical reinforcement locations. Full bond between steel and concrete elements was assumed. Appropriate area for steel bars was used according to the actual cut-off of the vertical bars. Equivalent area of the stirrups spaced at 100 mm in the model to that of the actual stirrup content spaced at 60 mm in the test, was used. The floor mass was distributed at each floor node. Young's modulus for steel was taken as 200 GPa. The damping coefficient was taken as 5% of critical.

Young's modulus for concrete was taken as 17500 MPa which is the secant young's modulus of concrete with $f_c' = 35$ MPa at a corresponding strain of 0.002. Choosing the secant modulus for concrete rather than the initial tangent, which is approximately equal to $\approx 4500\sqrt{f_c'} = 26{,}622$ MPa accounts for the stiffness reduction from $(E_cI)_{gross}$ to be $(E_cI)_{eff} \approx 0.65\ (E_cI)_{gross}$.

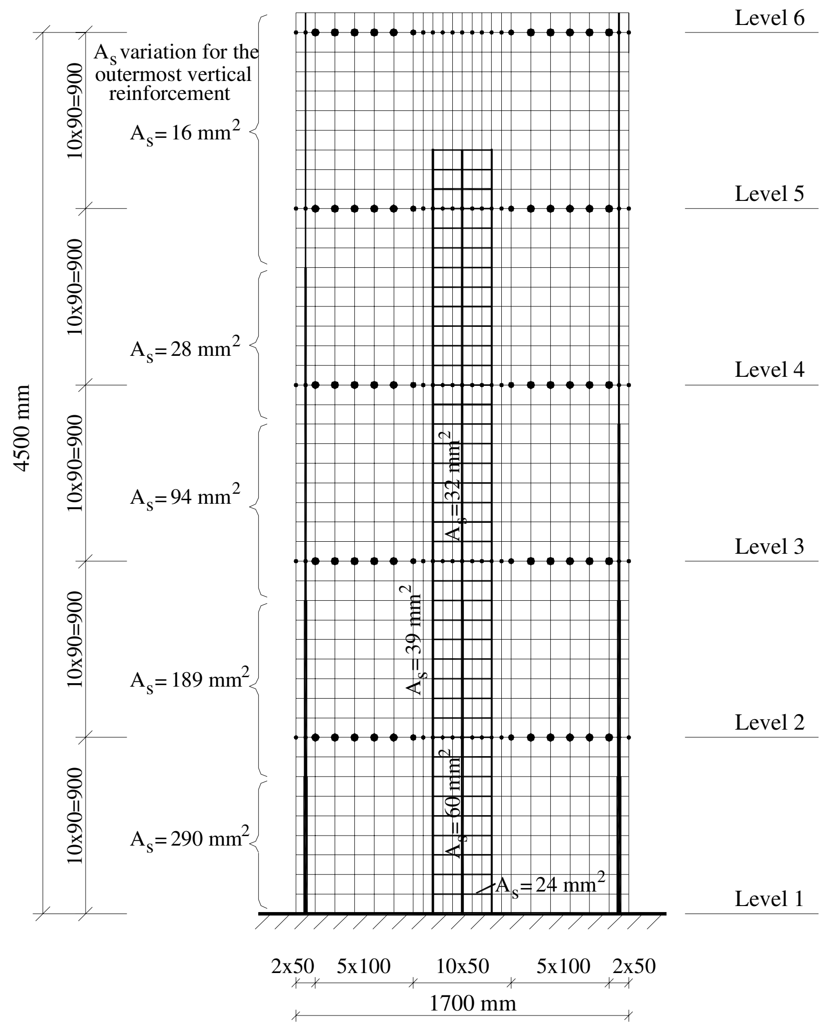


Figure 8 Finite element model for spectral analysis

The frequencies of free vibration of the first three modes were 7.92, 34.95, and 36.85 Hz, respectively. The first two modes are flexure modes in the lateral direction, while the third is an axial mode in the vertical direction. Table 2 contains the output for the response spectrum analyses of the wall model for tests RUN1 to RUN4.

Table 2 Output of the response spectrum analyses

Item	RUN1	RUN2	RUN3	RUN4
Top relative displacement (mm)	4.80	1.84	13.90	2.80
Top absolute horizontal acceleration (g)	1.22	0.47	3.58	0.75
Level 1 Bending moment (kN.m)	344.00	132.18	998.50	200.70
Level 1 Shear force (kN)	95.80	36.98	281.20	58.40
Strain in the external R-bar, level 4, ‰	0.14	0.03	0.40	0.05
Strain in the external R-bar, level 3, ‰	0.24	0.06	0.80	0.11
Strain in the external R-bar, level 2, ‰	0.40	0.09	1.13	0.17
Strain in the external R-bar, level 1, ‰	0.50	0.13	1.50	0.20

The modal analysis results shown in Table 2 are based on the response spectra for RUN1 to RUN4 (Figure 3). The calculated shear force at level 1 for RUN1 to RUN4, as listed in Table 2, are compared to the pushover analysis base shear-top floor displacement relationship as shown in Figure 6. It can be seen that RUN2 and RUN4 are in the elastic range. In this case, the response spectrum results give displacements similar to these obtained from the pushover analysis. RUN1 has small nonlinearity, while RUN3 exceeded the yielding capacity of the wall. In this case, the top level displacements from the pushover analysis are past the linear part and are larger than those obtained from the response spectrum analysis.

In the experimental program, the wall specimen was subjected to the table motion Run1 to Run4 consecutively. The response of the wall due to each test is affected by the behaviour of all the previous tests. For example, the recorded response of the wall to RUN4 is affected by the state of the wall (strength, stiffness, plastic deformations, etc.) and the accumulated damage from the previous three tests (RUN1 to RUN3). For this reason comparison of results between the two methods past initial yield is inappropriate.

DISPLACEMENT-BASED APPROACH

In the displacement-based approach, structures are designed to meet the selected performance objectives. Analysis procedures were developed to predict the demand in terms of forces and deformations imposed by the ground motion on structures. Simplified nonlinear analysis procedures developed by the Applied Technology Council [4] have been incorporated in the FEMA-273 and 274 documents [5,6] to determine the displacement demand imposed on a building expected to deform inelastically. The nonlinear static procedure in these documents is based on the capacity spectrum method [9].

The displacement-based procedure is applied to the tested walls to determine the peak ground acceleration PGA level of the scaled Nice and San Francisco records that will cause the test wall to deform 10, 15, and 20 mm at the top level. These displacements will be compared to the test results and analytical displacements from nonlinear dynamic analysis due to same PGA input.

The capacity spectrum method (Procedure A) was performed using the Nice and San Francisco record spectra to determine the PGA level that will cause a top floor displacement of 10, 15, and 20 mm. Table 3 contains the data used to calculate the reduced response spectra for Nice and San Francisco records. Figures 9 and 10 show the graphical representation of the procedure used to evaluate a target top floor displacement of 15 mm when subjected to Nice and San Francisco records, respectively. The PGA levels for Nice and San Francisco records corresponding to various given top floor target displacements, are plotted in Figure 11.

Table 3 Data used to construct the reduced demand spectrum for Nice and San Francisco records

Record	Roof displacement Mm	d_y mm	a_y g	d_p mm	a_p g	β_o %	κ	β_{eff} %	SRa	SRv
Nice	10	2.26	0.72	7.14	0.92	29.69	0.33	14.8	0.65	0.73
	15	2.4	0.77	10.71	1.00	35.02	0.33	16.56	0.61	0.70
	20	2.53	0.81	14.29	1.04	38.36	0.33	17.66	0.59	0.69
San Francisco	10	2.26	0.72	7.14	0.92	29.69	0.64	23.92	0.50	0.61
	15	2.4	0.77	10.71	1.00	35.02	0.60	26.0	0.47	0.59
	20	2.53	0.81	14.29	1.04	38.36	0.58	27.11	0.46	0.58

Where:

d_y and a_y = spectral displacement and acceleration at the yield point of the capacity spectrum diagram, respectively,

d_p and a_p = spectral displacement and acceleration at the target performance point of the capacity spectrum diagram, respectively,

$$\beta_0 = \frac{63.7(a_y d_p - d_y a_p)}{a_p d_p}$$

κ = damping modification factor specified in ATC-40 [4]

$$\beta_{eff} = \kappa\beta_0 + 5$$

$$SR_a = \frac{3.21 - 0.68\ln(\beta_{eff})}{2.12}$$ = Spectral acceleration reduction factor in the constant acceleration range

$$SR_v = \frac{2.31 - 0.41\ln(\beta_{eff})}{1.65}$$ = Spectral acceleration reduction factor in the constant velocity range

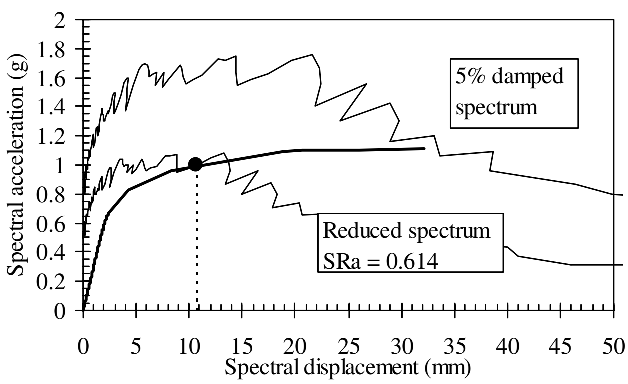


Figure 9 Response spectra to 15 mm performance for Nice record (PGA 0.61g)

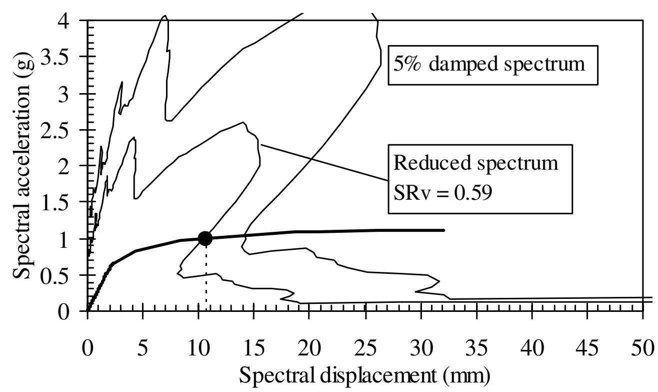


Figure 10 Response spectra to 15 mm performance for San Francisco record (PGA 1.30g)

Figure 11 shows the estimated PGA at various target top floor displacements using the displacement-based approach versus results of the four tests. From the graph it could be seen that for a target top floor displacement of 10 mm, the displacement-based approach over estimated the PGA level for Nice record, while it compares more favorably with the San Francisco record. The difference between the displacement-based method and the test results could be attributed to the single degree of freedom system used in the capacity spectrum approach and the use of initial uncracked stiffness in the pushover analysis, which is not the case in the second and subsequent tests.

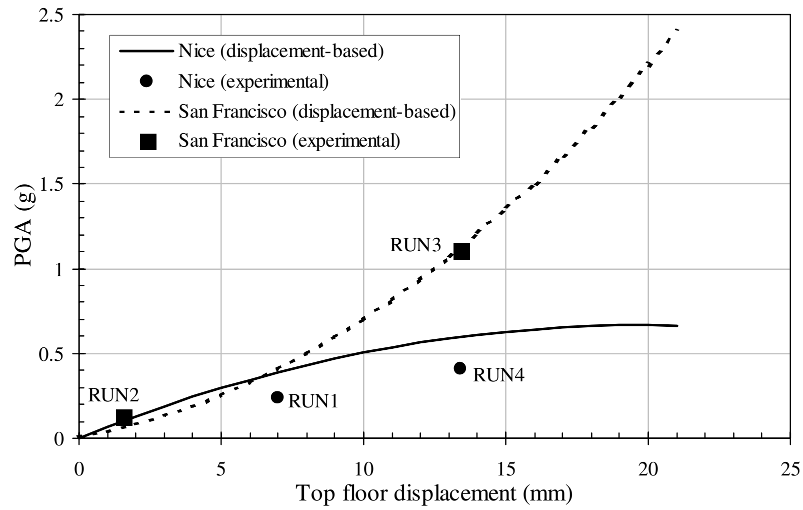


Figure 11 Estimated PGA at target top floor displacements from the displacement-based approach versus results of the four tests

TIME HISTORY ANALYSIS

Two-dimensional nonlinear time history analysis was conducted on the tested wall. The dynamic response of the wall to the four input motions identified as RUN1 to RUN4 was evaluated.

Approach

The dynamic response analysis is carried out using CANNY99 program [2]. The equation of motion is solved using step-by-step numerical integration method over a relatively small time step of 0.005 second. This allows for two integration points in each acceleration data interval. The integration time step is approximately 1/30 of the elastic fundamental period of the wall model. In each time step, the stiffness of the structure and elements are assumed linear. Iterations for the element force equilibrium are applied given a small tolerance. The overshooting due to this small tolerance is corrected during the next step. Rayleigh damping is used assuming the damping to be proportional to the instantaneous stiffness matrix. The damping coefficient is taken as 5% of critical.

The response calculations using the CANNY99 program accounts for P-Δ effects. However, the maximum lateral deformation of the wall did not exceed 12.6 mm or 0.3% drift. The reasonably small displacement is due to the high stiffness of the wall. Therefore, a first order analysis without P-Δ effects would have resulted in equally accurate results.

Wall model

Wall idealization

Figure 12 shows the analytical model of the tested wall. The wall was idealized as six panel elements representing the six floor levels. The wall panel elements are supported on three vertical springs representing the shake table. The panels are six node elements with three nodes at the top and the bottom boundaries. In other words, there are four nodes at the corners in addition to a node at the mid points of the top and bottom boundaries. The adjacent panels have compatible deformations at their common three nodes that are connecting them. The deformation of adjacent panels between the three common nodes is maintained equal by using rigid links between the nodes at the top and bottom boundaries of the elements. The floor masses were distributed among the three nodes at each floor level such that one half of the floor mass is lumped at the mid boundary node at the centre of the floor and one quarter of the floor mass is lumped at the two outer floor nodes at the corners of the panel. Each mass has two displacements and one rotational degree of freedom.

Member model
The panel element used in the pushover analysis and described earlier was also used for the time history analysis. Linear shear deformation was assumed. Figure 12 shows the properties of the analytical model at different floor levels.

Material models
The trilinear/bilinear model SS3 shown in Figure 5a was used to represent the steel bars. The parameters for the steel skeleton curve were taken as those used for the pushover analysis. The bilinear model CS2 shown in Figure 5b was used to represent the concrete material. The parameters for the concrete skeleton curve were taken as those used for the pushover analysis.

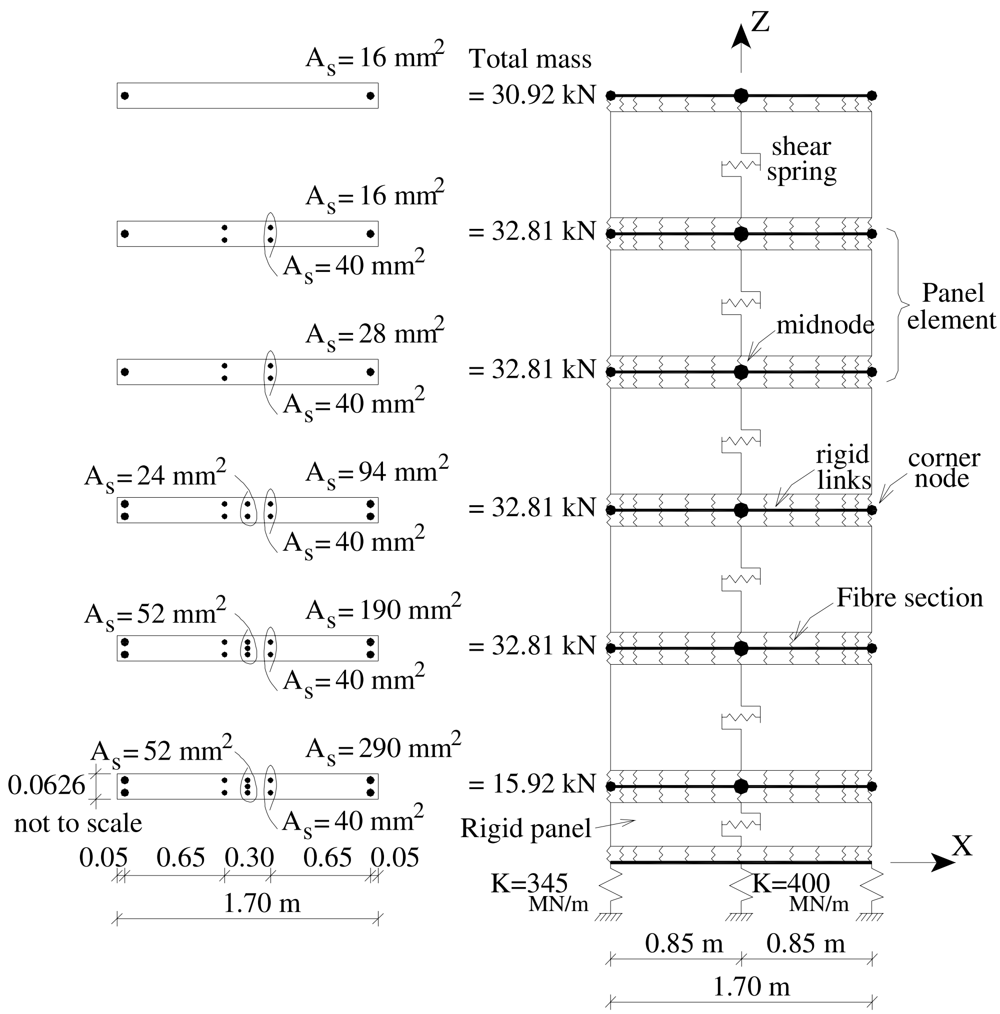


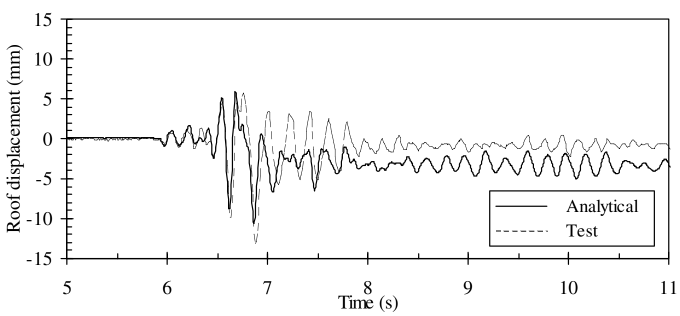
Figure 12 Properties of the modeled tested wall

Acceleration input data
Two sets of acceleration records were used as simultaneous input to the dynamic time history analysis of the wall. The two acceleration inputs to the wall were recorded on the shake table in the horizontal and vertical directions. Although there was no vertical acceleration input made to the shake table during the test, the measured vertical acceleration on the table was included in the analysis.

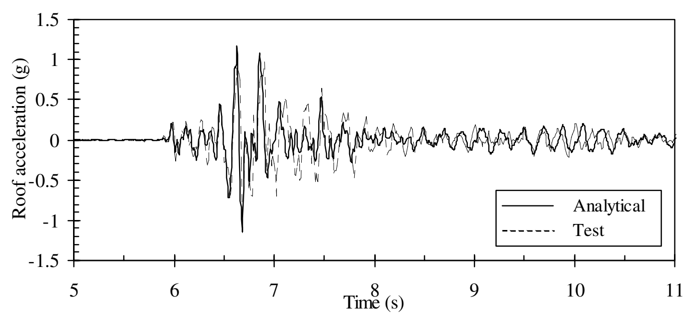
Results

Results of the time history analysis in the form of wall top displacement, top horizontal and vertical accelerations, shear force and bending moment at level 1 were obtained for the tests RUN1 to RUN4. As an example of the results, Figure 13 shows a comparison between the predicted and measured responses for the test RUN3 San Francisco record with PGA 1.11g.

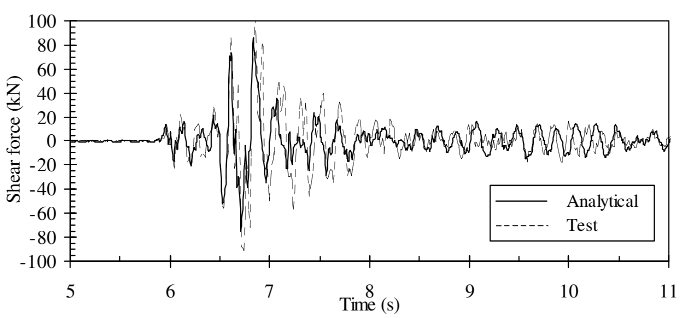
The maximum values reached during the analysis and the measurements of tests RUN1 to RUN4 for the top displacement, top acceleration, level 1 bending moment and shear force and axial force tension/compression, and the external re-bar strain at the first four levels are summarized in Table 4. The maximum tensile strains in the external vertical bars at floor levels 1 to 4 are shown for RUN1 to RUN4. Comparing the experimental and analytical results, it was found that analytical strain levels at the first floor are higher than the experimental ones, while the opposite occurs at the third floor level. This is because the analytical strains obtained using macro-model fibre panel elements are average strains at the element critical section. However, the experimental strains are measurements made at the specific location of the gauge.



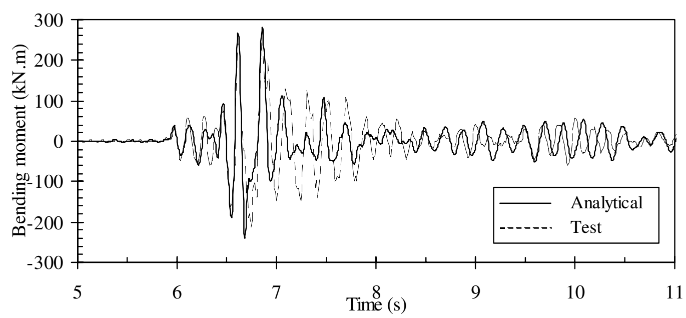
(a) Top floor lateral displacement response



(b) Top floor lateral acceleration



(c) Shear force at level 1



(d) Bending moment at level 1

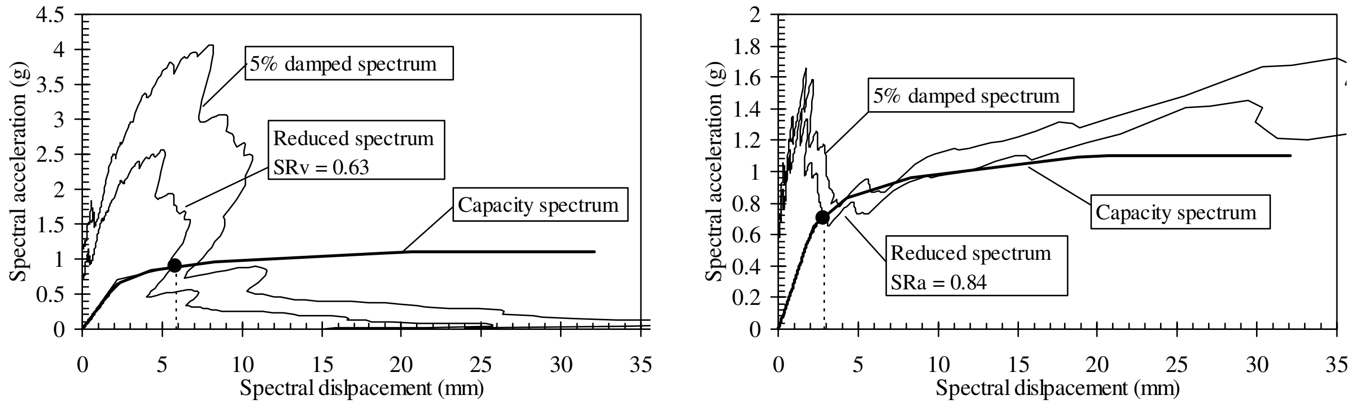
Figure 13 Comparison between the predicted and measured responses for the test RUN3

Comparison with performance-based design

The top floor displacement of the wall is evaluated using the performance-based design approach and the dynamic analysis. The static pushover curve of the wall in its initial condition was used in calculating its capacity spectrum. The two records RUN3 (San Francisco) and RUN4 (Nice) were used to represent near and far-field ground motions, respectively. Figure 14 shows the performance point representation by Capacity Spectrum Method – Procedure A [4] for the two records. Figure 15 shows the top floor lateral displacement time history for the two records.

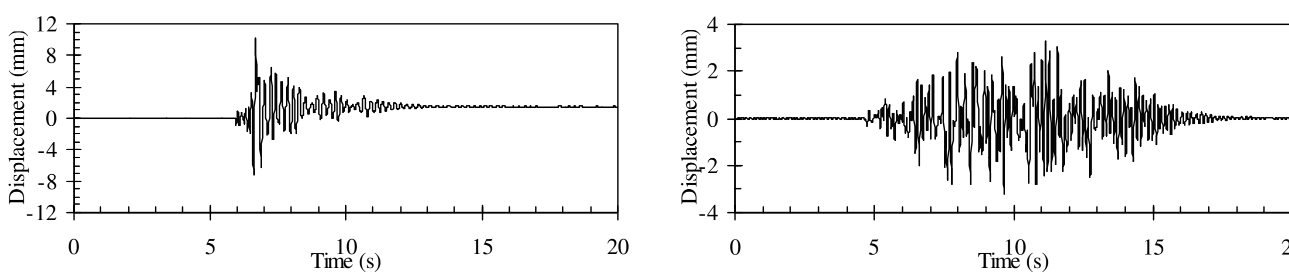
Table 4 Maximum response values from the dynamic analysis and test measurements

Item	RUN1		RUN2		RUN3		RUN4	
	Analysis	Test	Analysis	Test	Analysis	Test	Analysis	Test
Top relative horizontal displacement (mm)	6.1	7.0	1.9	1.54	10.6	13.2	12.6	13.4
Top absolute horizontal acceleration (g)	0.87	0.68	0.27	0.28	1.17	1.16	0.91	0.93
Level 1 Bending moment (kN.m)	242.3	211.0	73.6	75.5	282.9	280.0	281.3	276.0
Level 1 Shear force (kN)	74.7	65.9	22.0	23.5	86.0	106.0	85.7	86.6
Level 1 Axial tension (kN)	24.0	44.3	5.8	--	57.0	102.0	23.0	50.0
Level 1 axial compression (kN)	-35.0	-36.5	-9.7	--	-43.0	-105.0	-36.0	-51.9
External R-bar strain, level 4 (‰)	1.32	--	0.44	--	1.84	--	1.33	--
External R-bar strain, level 3 (‰)	1.94	2.23	0.62	0.14	2.43	3.29	2.14	25.4
External R-bar strain, level 2 (‰)	0.23	1.31	0.13	0.13	0.42	1.43	1.78	1.98
External R-bar strain, level 1 (‰)	1.87	1.43	0.61	0.25	3.67	1.55	3.46	2.26



(a) San Francisco record (b) Nice record

Figure 14 Performance point representation



(a) San Francisco record (b) Nice record

Figure 15 Dynamic analysis for San Francisco and Nice records

Table 5 Top floor displacement (in mm) for San Francisco and Nice records

Approach	San Francisco	Nice
Performance-based procedure	8.4	3.9
Dynamic analysis	10.2	3.2

Table 5 shows the top floor displacements of the wall when subjected to San Francisco and Nice records calculated using the performance-based approach and dynamic analysis. From the table, it can be seen that the difference between the two methods is approximately 20%.

CONCLUSIONS

Different analytical approaches for the seismic analysis of lightly reinforced concrete walls were evaluated. The analysis includes nonlinear static (pushover), linear spectral, displacement-based, and nonlinear dynamic time history analyses. The analytical model is verified against shake table dynamic test results. Based on the analysis results, the following conclusions are reached:

1- The nonlinear static pushover analysis using the fibre section macro model panel elements predicted the base shear-top floor displacement of lightly reinforced wall with reasonable accuracy compared to the shake table dynamic test results.

2- Conventional spectral analysis gives good estimation of the wall response in the elastic range. This linear analysis is valuable in determining the appropriate structural reduction factor when compared to a nonlinear procedure such as the pushover or the dynamic time-history analysis.

3- The analytical time history responses of the modeled wall with the associated parameters have good correlation with the measured results of the four shake table tests.

4- There is a difference of approximately ±20% between the top floor displacement values predicted by the performance-based approach and the dynamic analysis.

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