

Computer modeling and effective stiffness of concrete wall buildings

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ABSTRACT: Shake-table tests of a full-scale seven-story wall structure at the University of California at San Diego (UCSD) provide a crucial benchmark in evaluating methods that are currently being used to design mid-rise and high-rise concrete buildings in seismically active areas. The authors compare properties and characteristics of the UCSD test structure with twelve tall concrete core-wall buildings that have recently been designed for the western United States, and find that the test results are applicable to this type of structure. Using assumptions, methods, and software that are typical in design practice, the authors constructed linear and non-linear computer analysis models of the UCSD test structure. Iterations of assumptions for the linear models lead to recommended concrete stiffness properties, which are then compared to published recommendations that are often used in design. Recommended stiffness properties are lower than those commonly used in practice. Comparison of the non-linear models to test results shows a difficulty in matching building deformations while also matching overturning moments and shear forces. Both types of models show a significant influence of slabs engaging columns, and acting as outriggers, increasing overturning resistance and shear demand on the wall.

1 INTRODUCTION

In recent years a number of tall concrete wall buildings have been designed for high seismic areas in California, Washington and Utah using “non-prescriptive” seismic design methods (Maffei & Yuen 2007). The seismic design uses a capacity design process that includes both linear and non-linear dynamic analyses. A linear response-spectrum analysis is used at the code level of earthquake ground motion, and a non-linear response-history analysis is used at the Maximum Considered Earthquake ground motion (SEAONC 2007).

This paper compares the results from shake-table experiments on a seven-story test structure to analysis models that are typically used in the structural engineering design practice for tall concrete wall buildings. The objective of the comparison is to find which modeling assumptions best predict response.

Buildings in the United States are typically designed for earthquakes using a force-based procedure where the strength of elements is determined by linear analysis according to building code requirements. The analysis takes account of expected inelastic behavior through application of the force reduction factor R , which is applied to the spectrum in the analysis. For tall buildings, where wind forces or a minimum base shear governs the lateral strength, an effective reduction factor R_{eff} can be calculated as the base shear from an analysis at the unreduced code response spectrum, divided by the governing design base shear.

The appropriate stiffness for the linear analysis is an effective initial stiffness consistent with the bilinear force-deformation assumptions that were used to establish this traditional design approach. See for example Blume et al. (1961).

Thus the appropriate stiffness for linear seismic analysis is an effective initial stiffness of the structure as it reaches the limit of essentially linear behavior. Such a stiffness is illustrated by the $EI_{effective}$ line in Figure 1.

The effective initial stiffness should account for cracking of the concrete and other phenomena that reduce stiffness, such as strain penetration, bond slip,

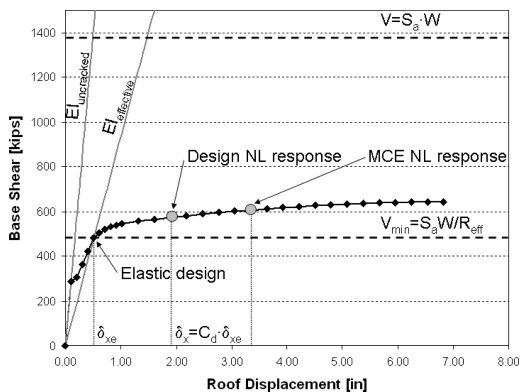


Figure 1. Modeling of structural response in design.

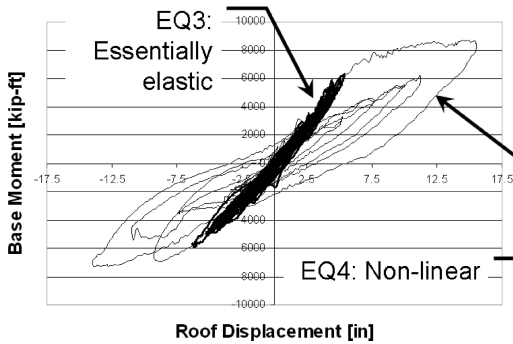


Figure 2. Base moment vs. roof displacement experimental behavior for EQ3 and EQ4 loading.

shear deformation, and tension shift associated with diagonal cracking.

Though non-linear response is not explicitly evaluated in conventional design, safe non-linear seismic performance under the design ground-motion is expected based on historic calibration of the force reduction factors in the building code. When validated R -factors are not available, as is the case for very tall concrete wall buildings, the need arises to verify non-linear response as part of the design process.

The UCSD test structure was subjected to several ground motions (Panagiotou et al. 2006). Figure 2 shows that the earthquake ground motion of medium intensity, EQ3, demonstrates essentially linear force versus deformation response, and EQ4 shows non-linear response. Comparing Figures 1 and 2, the behavior for EQ3 response, which approaches global yielding, can be described as representative of elastic design. By contrast, EQ4 shows significant excursions into the non-linear domain.

To evaluate the applicability of the test structure to actual tall concrete buildings, a number of characteristic parameters are compared in Table 1. For each parameter, both a range and a typical value have been determined from the review of twelve recently designed tall buildings, which are compared to those for the test structure.

The quantity V_{yield}/W in Table 1 provides a comparison of the design strength of the test structure compared to the typical high rise. Typical tall buildings on the West Coast are designed for a minimum base shear of between 4% and 6% of the building weight. Considering over-strength, typical high-rise buildings have a yield base shear capacity corresponding to about 7% to 12% of the building weight. The over-strength results from: (1) expected strength of reinforcing steel which is about 15% higher than nominal design strength, (2) the use of a strength-reduction factor of about 0.9 for flexural design, and (3) contribution of elements not designated as part of the

Table 1. Comparison of key properties for UCSD test structure and typical tall concrete core-wall buildings.

Parameter	UCSD Wall	High-Rise (range)	High-Rise (typical)
h/l_w	5.2	9–13 (weak way)	10 (weak way)
ρ_{vert} hinge zone	0.7%	0.7%–2.0%	1%
ρ_{vert} above hinge	0.8%	0.8%–2.2%	1.1%
ρ_{horiz} hinge zone	0.3%	0.3%–2.6%	1%
ρ_{horiz} above hinge	0.4%	0.3%–1.2%	0.7%
$V_u/(\sqrt{f'_c}A_g)$ at hinge	3.0	3–8	6
Axial load ratio ($P/A_g f'_c$)	0.05	0.06–0.13	0.09
Floor span-to-depth ratio	17	30–45	40
Effective elastic period	1 sec	4–9 sec (weak way)	6 sec (weak way)
V_{yield}/W	26%	6%–12%	8%

Seismic-Force-Resisting System. The upper range of yield capacity (9% to 12% of building weight) is reached for buildings with IBC S_1 factors greater than 0.6 (ICC 2006), for which a larger minimum base shear is specified, and for the stronger direction of building layouts that have a rectangular concrete core.

The V_{yield}/W value for the test structure was computed by performing a response spectrum analysis on the building, using the code spectrum for soil class C (very dense soil and soft rock). The response-spectrum results are then scaled to compute V_{yield} equal to the base shear that corresponds to a wall base moment equaling the wall's expected moment capacity: $V_{yield} = V_u (M_{n,exp}/M_u)$.

The values in Table 1 show that typical high-rise core-walls have, for most parameters, similar properties to the test structure. Dissimilar properties include the floor span-to-depth ratio and V_{yield}/W . The effects of these differences are considered when extrapolating the results for the test structure to actual high-rise buildings.

In practice, different engineers use different assumptions for the effective initial stiffness of a wall, based on any of the conflicting recommendations in various published guidelines. Table 2 lists reference documents and approaches proposed or used in practice. Three classes of recommendations can be distinguished, the first with fixed stiffness modifiers, the second with modifiers depending on axial load level, and the third that includes the reinforcement ratio in addition to the axial load level.

Table 2 shows the results of applying each published recommendation to the test structure, both for the first and the sixth story. Table 2 also shows the results of applying each published recommendation to the typical tall concrete core-wall building. Wall dimensions for each case are shown in Figure 3. The test specimen

Table 2. Effective elastic flexural stiffness properties for strong way bending of structural walls: Comparison for UCSD test specimen and typical high-rise.

Reference	Effective Stiffness			Considers:	
	Base	6th floor	Typ. High-Rise	P	ρ_{vert}
ASCE 41, <i>uncracked</i>	$0.80I_g$	$0.80I_g$	$0.80I_g$	NO	NO
ASCE 41, <i>cracked</i>	$0.50I_g$	$0.50I_g$	$0.50I_g$	NO	NO
Microys <i>et al.</i> (Eq. on p. 302)	$0.64I_g$	$0.61I_g$	$0.68I_g$	YES	NO
NZS:3101 ($\mu = 6$)	$0.29I_g$	$0.26I_g$	$0.33I_g$	YES	NO
NZS:3101 ($\mu = 3$)	$0.54I_g$	$0.51I_g$	$0.58I_g$	YES	NO
Paulay & Priestley (Eq. 5.7 on p. 376)	$0.25I_g$	$0.23I_g$	$0.29I_g$	YES	NO
Restrepo (Eq. 7)	$0.22I_g$	$0.26I_g$	$0.42I_g$	NO	YES
ACI 318 (Eq. 9–8)	$0.32I_g$	$1.0I_g$	–	YES	YES
FIB 27 (Eq. on p. 83)	$0.21I_g$	$0.20I_g$	$0.28I_g$	YES	YES
Moment-Curvature	$0.20I_g$	$0.21I_g$	$0.27I_g$	YES	YES

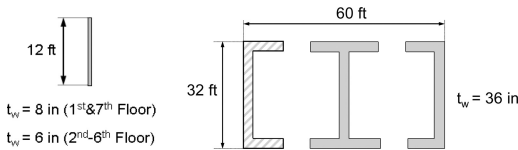


Figure 3. UCSD test versus typical layout of high-rise concrete core-wall.

could be considered to represent a scaled down version of the hatched part of the core wall.

2 ELASTIC MODELING FOR WALL BUILDINGS

A linear-elastic model of the test structure is built in the ETABS (2006) analysis software. For the wall and floor slabs wall/slab elements are used, which are shell-type elements with a membrane and a bending component. The analysis model is shown in Figure 4.

Mass and forces are calculated by ETABS from the concrete self-weight. The mass contribution from the post-tensioned wall that provides out-of-plane restraint to the test is added to the central node of the web wall, at each story.

Six models with different sets of assumptions on the effective wall and slab stiffness are created, and subjected to linear time-history analysis using an integration time step of 1/240s. With each elastic model the medium level ground motions EQ3 from the tests (Panagiotou *et al.* 2006) is used, as discussed in the introduction. All cases analyzed are summarized in Table 3. For simplicity the effective stiffness factor reported is applied to the material stiffness E_c , thus affecting the stiffness in both shear and flexure for

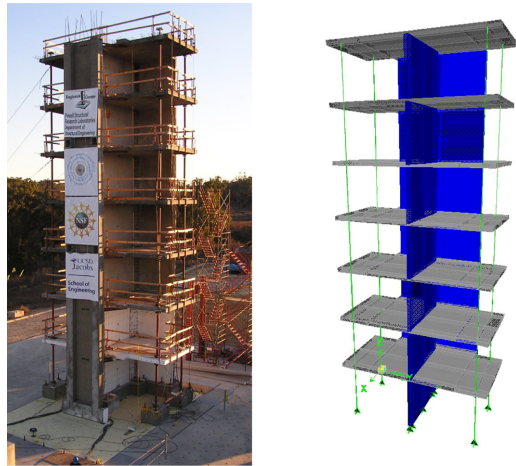


Figure 4. UCSD Wall (left) and ETABS model (right).

Table 3. Analysis models and runs used to determine appropriate stiffness modeling assumptions.

Wall 1st Floor	Wall 2nd–7th Floor	Floor slabs	Record	Results/comments
Factor on I_g				
0.8	0.8	0.5	EQ3	Typical practice assumption; too stiff
0.2	0.2	0.2	EQ3	OK match of displacement
0.2	0.2	0.1	EQ3	Better match of displacement history
0.13	0.3	0.3	EQ3	Accounts for bond slip; limited improvement in match
0.13	0.3	0.1	EQ3	Best match of displacement history and profile
0.1	0.3	0.15	EQ3	Improves match of outrigger factor
fiber	fiber	0	EQ3&EQ4	Slightly over predicts peak displacement, under predicts moment and shear
fiber	fiber	0.1	EQ3&EQ4	Matches peak displacement and profile, under predicts moment and shear
fiber	fiber	1	EQ3&EQ4	Under predicts displacement, matches system moment and shear

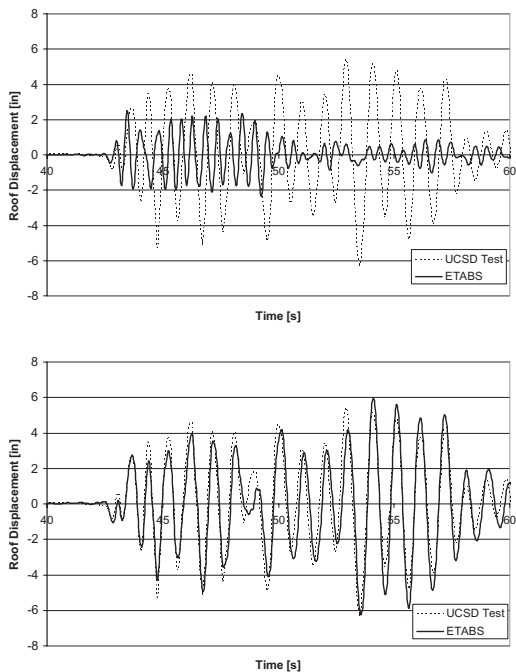


Figure 5. Comparison of roof displacement response for EQ3, using 80% of gross wall stiffness and 50% of gross slab stiffness (top), and 20% of gross wall stiffness and 10% of gross slab stiffness (bottom).

all degrees of freedom. Comparisons are made to the UCSD test wall response for the same earthquake record.

The first three models apply the same effective stiffness multiplier over the entire wall height. They represent the extremes of the published effective stiffness values from Table 2. The effective stiffness values used for the slab bound a fully cracked and an un-cracked response. Key response quantities are shown in Figures 5 through 7. Figure 5 includes response time-histories for roof displacement, Figure 6 shows the peak displacement envelope over the building height, and Figure 7 shows peak moment and shear values at each story. All values are shown together with the experimental results. The moment in Figure 7 is the system moment, including overturning resistance from the slab and column system. Included in the figure showing the system moments are the peak web-wall moment at the base (from analysis) and the expected wall yield moment under gravity loads. The difference between the system moment and these latter moments are indicators of the slab-outrigger effect on the structural response.

The results show that use of the lower stiffness values predicts the response much better. In particular, the results for the 20% wall stiffness model using 10%

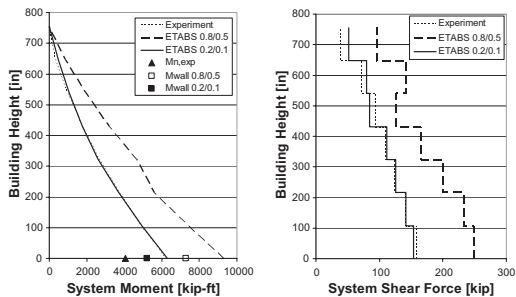


Figure 6. Comparison of peak displacement envelopes for EQ3, using 80% of gross wall stiffness and 50% of gross slab stiffness (left), and 20% of gross wall stiffness and 10% of gross slab stiffness (right).

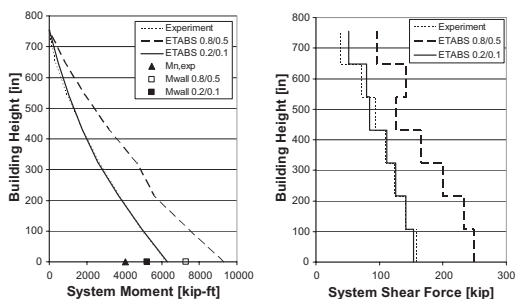


Figure 7. Comparison of peak moment (left) and peak story shear (right) for EQ3, using 80% of gross wall stiffness and 50% of gross slab stiffness, and 20% of gross wall stiffness and 10% of gross slab stiffness.

of gross slab stiffness show very good agreement in the displacement response, see Figure 5.

Looking at the displaced shape in Figure 6 (right-hand side), it can be seen that the displacement at the lower floors is slightly under-predicted, while the gradient at higher floors is slightly over-predicted. The additional displacement in the first floor could result from both bond-slip at the lap splice and strain penetration into the foundation. To account for such behavior, the stiffness of the ground floor is reduced to add this additional flexibility, based on an estimate of this contribution from strain penetration calculations. To keep the match of roof displacement, the effective stiffness for the upper floors is somewhat increased. Three additional models include this effect, as summarized in Table 3. Results are shown in Figure 8.

3 NON-LINEAR MODELING FOR WALL BUILDINGS

In addition to the linear modeling, a non-linear model of the structure is built, using the Perform 3D analysis software (2006). The model uses vertical fiber

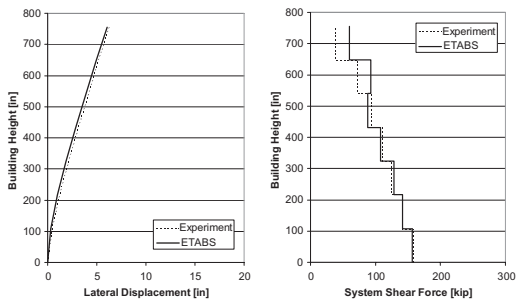


Figure 8. Comparison of key response quantities for EQ3, using 13% and 30% of gross wall stiffness and 10% of gross slab stiffness.

elements to explicitly model the non-linear properties of the wall cross section. The base of the model reflects the essentially fixed support at the shake table that was observed in the tests. The model restrains out-of-plane translational degrees of freedom to replicate the one-dimensional input and two-dimensional response.

As shown in Figure 9, two non-linear models are evaluated – one with, and one without, the slab and columns. The wall-only model corresponds to what is often modeled in practice, where only those elements designated as part of the seismic-force-resisting system are assumed to resist earthquake forces.

P-delta effects are explicitly included in the analyses.

3.1 Wall modeling

The wall is modeled with the Perform 3D shear wall compound element and includes inelastic sections over the height of the building. The compound element consists of three components: (1) an inelastic axial/bending fiber section containing vertical fibers modeling the vertical reinforcing steel and the gross concrete area, (2) an elastic shear component, and (3) an elastic out-of-plane flexural component. Similar elements have been shown to model wall flexural behavior well (Orakcal and Wallace, 2006).

In modeling the wall, one element is used over the story height, except at the first floor, where two elements are used. This is to better model the plastic hinge, which is expected to extend to about half the height of the first floor. Two planar elements are used over the wall length.

The inelastic fiber section is described by the wall thickness and reinforcement ratio, and a material law for both the steel and concrete. Since reinforcement is not distributed uniformly, the program's option to locate fibers manually is used to better replicate the reinforcement distribution.

Expected material properties are used for the parameters in the material models. Detailed

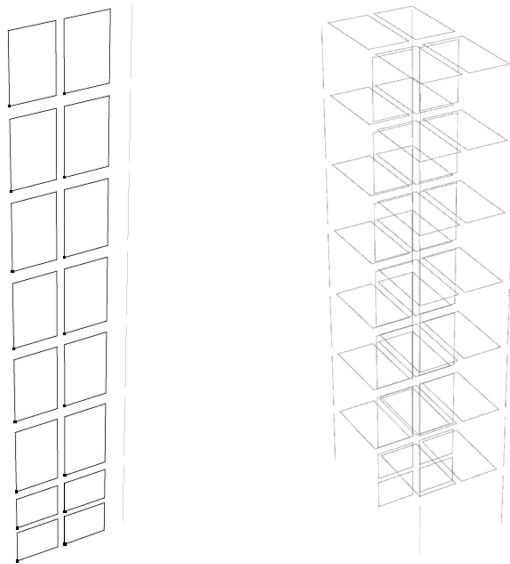


Figure 9. PERFORM 3D models of wall web, with and without slab.

information for concrete batches and steel reinforcement heats is available for the test structure (Restrepo 2006), and is used to determine an average value for the entire building.

Effective stiffness values are used for the elastic components of the wall and slab elements. For the wall in flexure, the fiber element properties determine effective initial stiffness.

3.2 Analysis parameters

After an initial static load step applying gravity loads, the earthquake records from the test structure are applied to the model. A Rayleigh damping model is used, with damping coefficients chosen to result in 2% equivalent viscous damping at the initial fundamental period of the structure T_1 , and at $0.2T_1$. The integration time-step used is 1/240 second, equal to the time step at which the ground motion records were sampled.

Table 3 shows the three analysis cases considered.

3.3 Results

From modal analysis, the initial fundamental period T_1 of each model is as follows: 0.490 seconds for the wall-only model, 0.485 seconds for the model with the slab at 10% gross stiffness, and 0.468 seconds for the model with the slab at 100% gross stiffness. The period found at the beginning of the testing cycle was reported in Panagiotou et al. (2006) to be 0.502 seconds, while the period of the structure prior to running EQ4 was approximately 1 second.

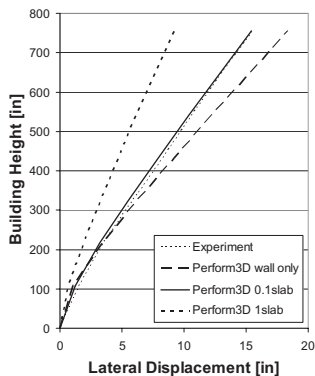


Figure 10. Comparison of peak displacement envelopes for EQ4.

Key response quantities for the three models for EQ4 are shown in Figures 10 and 11.

Figure 10 shows that the wall-only model over-predicts the displacement response. Figure 11 shows that this model under-predicts system forces. The wall shear values for the wall-only model for EQ4 underestimate the experimental values by up to 40%. The results for the wall-only model indicate that the interaction of the gravity system in the seismic response should not be neglected.

Results for the models including slab behavior improve the prediction of system forces. However, none of the models are able to capture both the displacement and force demands accurately.

A summary of the performance of all three models analyzed is included in Table 3.

4 CONCLUSIONS

The properties and characteristics of the UCSD seven-story test structure are applicable to tall concrete buildings that use core walls as their seismic-force-resisting system. The shake-table tests of the structure provide a valuable benchmark against which engineers can study the assumptions typically used in the seismic analysis and design of concrete wall structures.

Linear and non-linear analyses show that it is important to model the gravity framing system in order to include the influence it has on the structure's seismic response. Floor slabs engaging columns, and acting as outriggers, increase overturning resistance of the system, axial forces in columns, and shear demands on the wall. This effect is somewhat more pronounced in the test structure than it would be in structures where floor span-to-depth ratios are higher.

The results for the linear-elastic analyses confirm that effective stiffness formulations that include the effect of axial load and longitudinal reinforcement

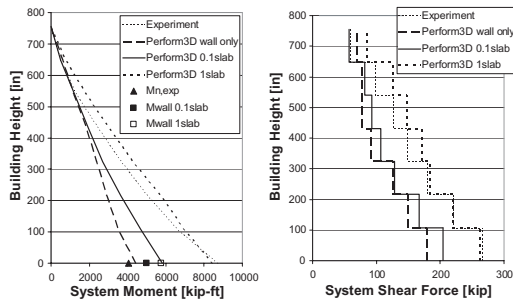


Figure 11. Comparison of peak moment (left) and peak story shear (right) for EQ4, using 100% gross slab stiffness.

ratio better predict effective initial stiffness. Actual stiffness values can be significantly lower than those used in some current published guidelines.

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APPENDIX I. NOTATION

The following symbols are used in this paper:

A_g = area of gross concrete section;
 C_d = deflection amplification factor (ICC 2006);
 δ_x = design deflection (ICC 2006);

δ_{xe} = deflection determined by linear analysis;
 E_c = concrete modulus of elasticity;
 f'_c = nominal concrete peak stress;
 h = wall height;
 I_g = moment of inertia of gross concrete section;
 l_w = wall length;
 μ = structure ductility capacity;
 $M_{n,exp}$ = expected wall moment strength;
 M_u = peak wall moment demand;
 P = wall axial load;
 ρ_{vert} = longitudinal wall reinforcement ratio;
 ρ_{horiz} = transverse wall reinforcement ratio;
 R = response modification coefficient;
 S_1 = mapped MCE spectral response acceleration parameter at a period of 1s (ICC 2006);
 S_a = design spectral response acceleration;
 t_w = wall thickness;
 T_1 = first period of vibration;
 V_u = peak wall shear force demand;
 V_{yield} = shear force demand at flexural yielding;
 W = total building weight.

