Proportioning of Earthquake-Resistant RC Building Structures

By JoAnn P. Browning,[1] Associate Member, ASCE

Abstract: A simple and efficient method is presented for proportioning of regular, moderate-rise reinforced concrete building structures. The method differs from conventional procedures in that member sizes are selected based on the demand defined by the displacement spectrum and criteria specified in relation to drift response. The maximum mean drift, or average distortion over the total height, is limited to reduce the expected damage to the structure. A series of analytical reinforced concrete frames are proportioned and tested using a suite of ground motions. Results of the analyses indicated that maximum displacement responses of the proportional frames were within the specified drift limit when a maximum-allowable period criterion was satisfied.

INTRODUCTION

Previous research has shown (Newmark et al. 1973) that the response of simple systems to ground motion can be represented by idealizing the linear response spectrum in three regions: nearly constant acceleration, nearly constant velocity, and nearly constant displacement response for a given damping factor. Using this idealization, reasonable estimates of maximum displacement response for structural systems can be determined for a given effective building period and damping. Shimazaki and Sozen (1984) provided a specific tool for estimating the nonlinear response of reinforced concrete structural systems by showing that, for systems with period (T_i) exceeding the nearly constant acceleration range of response, a reasonable upper bound for displacement response could be determined using a modified period of $\sqrt{2}T_i$ and an idealized linear displacement response spectrum with a coefficient of damping of 0.02. For structures having periods within the nearly constant acceleration range, strength became an influential parameter and the maximum displacement response could not be represented with the same spectrum. Lepage (1997) later extended the Shimazaki procedure to include the full spectrum of building periods for structures with threshold strength using a simplified displacement response curve

$$D = c \cdot T \tag{1}$$

where D = displacement demand; T = period of the system; and c = constant that defines the slope of the simplified demand curve.

In light of the ideas presented by Shimazaki and Lepage, a procedure was developed for proportioning of regular, moderate-rise earthquake-resistant reinforced concrete structures using a simplified displacement response curve and a maximum allowable period criterion to limit drift. The maximum drift, as a measure of the average distortion over the building height, is limited to reduce the economic damage expected for the structure. The proposed method provides a simple solution for separating the concepts of strength-related demands for gravity loads from stiffness-related demands for earthquake loads.

DESCRIPTION OF PROPORTIONING METHOD

The combination of the work done by Shimazaki and Sozen (1984) and Lepage (1997) present the opportunity to develop a simple and efficient proportioning method for reinforced concrete building structures. Using a linear displacement response spectrum that is defined for a particular site, the earthquake resistance of a structure can be assured by satisfying a simple target-period criterion. The underlying concept of the procedure is to control the drift expected in a structure subjected to strong ground motion. Drift is controlled by defining a maximum calculated period depending on regional seismicity and tolerable damage.

The flow of the proportioning algorithm is shown in Fig. 1 and comprises six steps:

1. Selection of the desirable maximum target period using an assumed or specified relationship between tolerable drift and building period
2. Selection of preliminary member sizes based on gravity-load requirements or experience

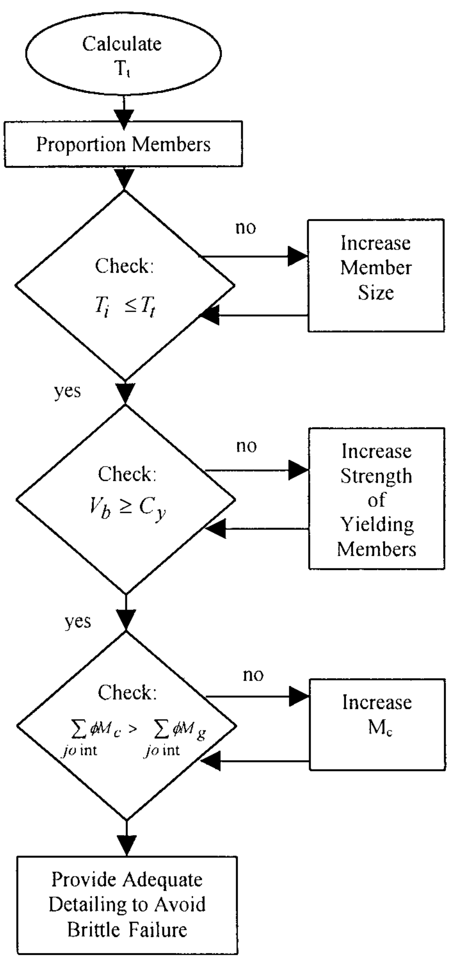

FIG. 1. Proposed Proportioning Procedure

[1]Asst. Prof., Dept. of Civ. and Envir. Engrg., Univ. of Kansas, Lawrence, KS 66045-2225.

Note. Associate Editor: Brad Cross. Discussion open until July 1, 2001. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on April 22, 1999. This paper is part of the ***Journal of Structural Engineering***, Vol. 127, No. 2, February, 2001. ©ASCE, ISSN 0733-9445/01/0002-0145–0151/ $8.00 + $.50 per page. Paper No. 20752.

3. Adjustment, if required, of member sizes depending on the comparison of calculated and required maximum periods
4. Comparison of base shear strength with respect to an acceptable minimum
5. Comparison of relative strengths of the columns and girders
6. Selection of structural details compatible with the maximum tolerable drift

The ‘‘drift’’ considered is the lateral deflection of a building structure in relation to a line perpendicular to its foundation and is a measure of the distortion of the structure and its nonstructural components.

Representing Displacement Response

The limiting quantity for determining the earthquake-resistance of a structure using the proposed method is drift. It follows that the drift response of a structure calculated for a particular design ground motion defines the earthquake demand. The described method provides a convenient and simple solution for representing this demand by using a displacement response curve such as the linear relationship defined by (1).

The simplified displacement response curve can be developed by considering typical acceleration amplifications in the range of nearly constant velocity response. A reasonable upper bound for acceleration response, A_s, in the region of nearly constant velocity response and for stiff ground (Newmark et al. 1973) is

$$A_s = \frac{2 \cdot \alpha \cdot g}{T} \tag{2}$$

where α = the coefficient of peak ground acceleration; and g = gravitational acceleration. Stiff ground is defined in the Uniform Building Code (ICBO 1997) as ‘‘rock’’ with an average shear wave velocity exceeding 760 m/s (2,500 ft/s) and ranging up to and including 1,500 m/s (5,000 ft/s). Considering the approximate relationship between acceleration and displacement response

$$A_s = \left(\frac{2\pi}{T}\right)^2 \cdot D_s \tag{3}$$

in combination with (2) provides

$$D_s = \frac{1}{(2\pi)^2} \cdot \alpha \cdot g \cdot T \tag{4}$$

For a region with seismic hazard characterized by α = 0.5, (4) can be used to represent the displacement demand

$$D_s = 250T \tag{5}$$

where T is in seconds and D_s = response displacement in mm ($D_s = 10T$ in.).

The maximum displacement response for a structure, D, can be estimated using the technique developed by Shimazaki and Lepage and the simplified response spectrum defined by (5). The expression

$$D = F_p \cdot c \cdot \sqrt{2} \cdot T_i \tag{6}$$

defines an estimate for the maximum displacement, D, where F_p is a participation factor for the mode considered; c = slope of the smoothed displacement response spectrum; and T_i = initial period of the building calculated using uncracked sections. For the spectrum defined by (5), the parameter c is 250 mm/s (10 in./s). A reasonable participation factor for a regular frame with a deformed shape defined by the first mode is 1.25.

Rearranging the terms in (6) provides an expression for determining the maximum allowable initial period of a structure for a specified amount of tolerable drift. A limiting period, or target period (T_t), is defined by replacing the estimated maximum displacement, D, in (6) by the total drift that is considered tolerable for the structure (D_t)

$$T_t \le \frac{D_t}{F_p \cdot c \cdot \sqrt{2}} \tag{7}$$

The engineer specified an amount of deformation considered tolerable for the intended structure and solves for a target period.

Satisfying Target-Period Criterion

For a given set of ground-motion parameters, the expected drift of a building structure is a function of its mass and stiffness and their distributions. The premise of the target-period method is that controlling the mass-stiffness relationship in the building (period) will control the drift associated with lateral excitation. Because the selection of the framing system and material will generally dictate the total mass of a structure, altering member sizes to provide adequate stiffness satisfies the target-period criterion.

The primary criterion for the proposed method is to ensure that the initial building period is within the bounds of the calculated target-period. Preliminary member sizes for a structure may be estimated either by selecting proportions based on gravity-load demands or by selecting proportions based on engineering judgement. These options may require iterations of increasing column sizes, calculating the initial period of the structure, and determining whether the target-period criterion has been satisfied.

EXAMPLE OF PROPORTIONING PROCESS

An example frame is proportioned to illustrate the procedure for controlling drift. The frame has five 3.0 m (10 ft) stories and three 9.1 m (30 ft) bays. The material used is reinforced concrete with an average modulus of elasticity of 27.6×10^3 MPa (4,000 ksi) and a compressive strength of 27.6 MPa (4,000 psi). The dead load carried by each floor is equal to 7.7 kN/m^2 (160 psf) applied over a tributary area with the frame width equal to the bay length. The frame is located in a region of high seismicity with the displacement demand defined by (5).

If the tolerable drift is to be limited to 1.5% of the total building height, the target period is calculated as

$$T_t = \frac{0.015 \cdot (15{,}000 \text{ mm or } 600 \text{ in.})}{1.25 \cdot (250 \text{ mm/s or } 10 \text{ in./s}) \cdot \sqrt{2}} = 0.5 \text{ s}$$

Appropriate column proportions are determined so that the maximum axial load does not exceed 45% of the capacity of the section. The resulting square column dimension is 508 mm (20 in.). The girders are assumed to have a depth of 762 mm (30 in.) and width of 406 mm (16 in.). The stiffness contributions of the girders are calculated using gross sectional properties and a factor to account for the contribution of the slab stiffness. The slab stiffness contribution depends on the definition of the effective slab width, which can range from a length defined by a 45° angle measured from the lower corner of the girder to the lower slab face, to the full slab width. For this range of effective slab widths and having a slab thickness of 152 mm (6 in.), the stiffness of the girders would increase by a factor ranging from 1.8 to 2.7. The stiffness of the girders in the study were increased by a factor of 2.

The period of the frame calculated using the initial column proportions and gross section properties is 0.6 s and does not satisfy the criterion defined by (7). Using the iterative procedure, it is determined that increasing the column dimensions to 660 mm (26 in.) results in a frame period of 0.5 s. The calculated period equals the target period and the proportioning process is complete.

DETAILING AND STRENGTH

The components of an earthquake-resistant building structure must have appropriate detailing and strength properties for the anticipated drift response. For the purposes of this study, it is stated that adequate detailing for shear strength and bond requirements must be provided for the structure to avoid brittle failure of any elements at the specified tolerable drift demand.

Strength requirements for an earthquake-resistant structure are addressed at the component level and the structural level. Although base shear strength has been shown to have only a small influence on drift control (Shimazaki and Sozen 1984; Qi and Moehle 1991; Lepage 1997), a threshold level of strength must be present. This requirement becomes particularly important in the region of nearly constant acceleration response (short-period structures). Lepage (1997) investigated the influence of base shear strength on the displacement response of a series of 3, 6, and 9 story notational frames using a minimum base-shear strength coefficient (C_y) defined as

$$C_y = \begin{cases} \dfrac{F_a \cdot \alpha}{(R_t - 1)\cdot TR + 1} \le \alpha & TR < 1 \\ \dfrac{F_a \cdot \alpha}{R_t \cdot TR} & TR \ge 1 \end{cases} \tag{8}$$

In (8), TR = period ratio and is defined as the ratio of the effective period ($\sqrt{2}T_i$) of the frame to the corner period (T_g) of the demand spectrum. The term F_a is the acceleration amplification factor and may be assumed as (15/4) to represent a wide range of ground motions for systems with a 2% damping factor (Shibata and Sozen 1976). The frames with base shear strengths defined using a reduction factor, R_t, equal to 4, 8, and 16 were shown to have similar displacement responses. A threshold base shear capacity such as the one defined by Lepage should be provided for the completed structural design. The proportioning of midrise reinforced concrete frame buildings rarely will be controlled by the minimum strength criterion.

Relative strength requirements for individual components of a frame are specified to prevent a story mechanism from defining the yielded shape of the structure. If story mechanisms are prevented, a low- to midrise system will deform approximately in the shape of its first mode. Current provisions for reinforced concrete buildings require that the total flexural strength contribution of the columns at a joint exceed that of the girders framing into that joint (a factor of 6/5 is specified in ACI 318-99, equation 21-1). This rule was used for the proportioned frames to encourage yielding in the horizontal members rather than the vertical members.

EVALUATION OF PROPORTIONING PROCESS FOR DRIFT CONTROL

A series of frames were proportioned according to the target-period criterion with a displacement demand curve defined for a region of high seismicity in order to test the proportioning process. The frames had three bays and ranged from five to 17 stories. Bay widths of 6.1 m (20 ft) and 9.1 m (30 ft) were used in combination with: (1) 3.0 m (10 ft) regular story heights; (2) 3.7 m (12 ft) regular story heights; and (3) 4.9 m (16 ft) first-story heights with 3.0 m (10 ft) upper-story heights. The concrete had a compressive strength of 27.6 MPa (4,000 psi) and a modulus of elasticity of 27.6 $\times$ 10^3 MPa (4,000 ksi). The yield stress of the steel reinforcement was 413 MPa (60 ksi). Reinforcement ratios in the columns were assumed to be 2.0% and average reinforcement ratios for the girders were 0.75%.

The member dimensions were selected considering strength requirements for gravity-load demands and stiffness requirements to limit drift. Girders were proportioned using depths of one-twelfth and one-tenth the span length. The columns of the frames with 9.1 m (30 ft) bays were not evaluated using the larger girder proportions, for two reasons. First, the selected column dimensions were found to be representative of member proportions in existing structures (Browning 1998). Second, many of the frames had column dimensions limited by gravity-load requirements and would not benefit from the increased stiffness provided by deeper girders.

Initial column sizes were selected so that the probable axial load [based on 7.7 kN/m^2 (160 psf) total distributed floor load] at the time of the earthquake would not exceed 45% of the axial-load capacity of the column. The resulting size would be considered too small for earthquake resistance, but it was on the ‘‘safe’’ side for testing the results of the proportioning method. Final square-column dimensions were selected to satisfy the criteria of maximum allowable axial load and maximum allowable initial period [(7)] for a mean drift ratio of 1.5% and using a displacement demand curve with a slope equal to 250 mm/s (10 in./s).

The second criterion for proportioning the columns, the target-period criterion, was generally the limiting factor for determining column dimensions. Column dimensions for frames with more than seven stories were reduced beginning at the midheight story of the frame to obtain a closer approximation to the calculated target period. The final selected girder and column sizes are listed in Table 1. Bold entries in the table indicate dimensions limited by the gravity-load criterion. The frames with 9.1 m (30 ft) bays and with more than 11 stories as well as two of the 17-story frames with larger girder proportions had column dimensions limited by this criterion (columns 7, 8, 11, and 12 of Table 1). Examples of the nine story frames proportioned with 9.1 m bays are shown in Fig. 2.

Consider the frames with dimensions that were not limited by gravity-load demands in Table 1. The column proportions increased by only 20% from the 5–17 story structures. Because the calculated target period increased linearly with building height, the small variance in column dimensions indicated that the stiffness of the additional stories was nearly proportional to the associated increase in building mass.

As expected, the increase in girder proportions for the frames with 6.1 m bays decreased the required column dimensions. When the girder depths were increased by 20%, the required column dimensions were reduced by approximately 30% (Table 1, columns 3 and 9). The ratios of the average total column height to base-column width ranged from 3.5 to 5.5 for the frames with girder depths equal to one-twelfth the span length and ranged from 5 to 7 for the frames with girder depths equal to one-tenth the span length.

The initial periods (gross-section) of the proportioned frames (T_i) also are listed in Table 1. The calculated period ratios (T_i/T_t, where T_t = target period), of all frames exceeded 0.9, with all but two frames having period ratios of at least 0.95. The periods of the proportioned frames were compared with the estimated periods of existing reinforced concrete frame structures provided by Goel and Chopra (1997) and were found to be representative of their sample (Browning 1998).

NONLINEAR ANALYSES

The nonlinear response of the proportioned frames subjected to statically incremented loading or earthquake motion was

TABLE 1. Member Dimensions and Initial Periods of Proportioned Frames

		Girder depths = $L/12$						Girder depths = $L/10$		
		L = 6.1 m			L = 9.1 m			L = 6.1 m		
		Column		Frame	Column		Frame	Column		Frame
Stories (1)	Story height (m) (2)	Base (mm) (3)	Top (mm) (4)	T_i (s) (5)	Base (mm) (6)	Top (mm) (7)	T_i (s) (8)	Base (mm) (9)	Top (mm) (10)	T_i (s) (11)
5	3.0	711	711	0.5	660	660	0.5	508	508	0.5
	3.7	711	711	0.6	711	711	0.6	559	559	0.6
	Tall 1st	762	762	0.6	762	762	0.6	610	610	0.5
7	3.0	711	711	0.7	660	660	0.7	508	508	0.7
	3.7	762	762	0.8	711	711	0.9	559	559	0.8
	Tall 1st	762	762	0.8	762	762	0.8	610	610	0.7
9	3.0	762	711	0.9	762	559	0.9	508	508	0.9
	3.7	813	711	1.1	762	610	1.1	559	508	1.1
	Tall 1st	813	711	1.0	813	610	1.0	610	457	1.0
11	3.0	813	711	1.1	762	559	1.1	508	508	1.1
	3.7	813	762	1.4	813	610	1.3	559	508	1.4
	Tall 1st	813	762	1.2	813	610	1.2	610	457	1.2
13	3.0	813	762	1.3	**813**	**610**	1.3	559	457	1.3
	3.7	864	762	1.6	**813**	**610**	1.6	559	559	1.6
	Tall 1st	864	762	1.4	**813**	**610**	1.4	660	457	1.4
15	3.0	864	762	1.5	**864**	**610**	1.5	559	457	1.5
	3.7	864	813	1.8	**864**	**610**	1.8	660	457	1.8
	Tall 1st	864	813	1.6	**864**	**610**	1.6	610	457	1.6
17	3.0	864	813	1.7	**914**	**660**	1.6	**610**	**457**	1.7
	3.7	864	864	2.1	**914**	**660**	2.0	660	457	2.1
	Tall 1st	864	864	1.8	**914**	**660**	1.7	**610**	**457**	1.8
Girders (all frames) height × width		508 × 254			762 × 406			6.10 × 305		

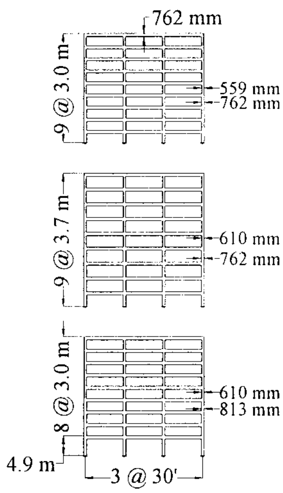

FIG. 2. Proportioned Frames with 9 Stories and 9.1 m Bays

calculated using a program developed by Otani (1974) and later modified by Saiidi and Sozen (1979a, 1979b) and Lopez (1988) called LARZ. The program has been used to successfully represent the displacement response of experimental and existing reinforced concrete structures subjected to strong ground motion (Saiidi 1979b; Lopez 1988; Eberhard 1989; Lepage 1997; Browning et al. 2000). The static version of the program was used to perform incremental load analysis and determine relative base shear strengths of the frames. Maximum drifts of the frames subjected to an array of ten ground motions were calculated using the dynamic version of LARZ.

Member Properties

Flexural properties were defined for all members as input for LARZ by the idealized moment-curvature relationship shown in Fig. 3. Gross sectional properties were used to define the initial stiffness of the systems. The girder stiffnesses were increased by a factor of 2 to account for the contribution of the slab stiffness. The yield strengths of the members were defined as the nominal ultimate moment capacities. Column flexural capacities were calculated considering axial loads equivalent to 7.7 kN/m^2 (160 psf) applied over the tributary area with bay widths equal to bay lengths. Calculations of column moment and curvature capacities were representative of capacities calculated using a concrete stress distribution defined by Hognestad (1951) with a limiting compressive strain of 0.004. A minimal value of postyield stiffness (0.01% of secant stiffness) was defined for all members. Hysteresis in the elements followed the rules defined by Takeda et al. (1970) with an unloading-slope coefficient of 0.4. Viscous damping in the system was defined with a coefficient of damping equal to 0.02. The calculated member properties are listed in the original report (Browning 1998).

Rotation caused by the relative slip between the longitudinal reinforcement and concrete at the face of the joints was also included in the displacement calculations. The amount of slip rotation was calculated for developing the full yield stress in

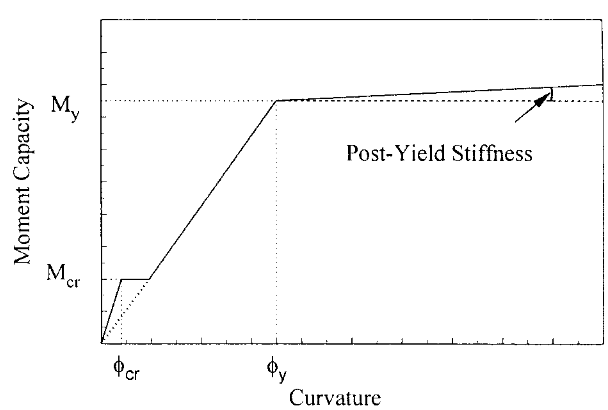

FIG. 3. Idealized Moment-Curvature Relationship for Member Input in LARZ

TABLE 2. Base Shear Capacities for Selected Frames

Number of stories (1)	Bay width (m) (2)	Regular or tall 1st story (3)	Story height (m) (4)	Girder Depth = $L/12$		Girder Depth = $L/10$	
				V_b (5)	V_b/C_y (6)	V_b (7)	V_b/C_y (8)
5	6.1	Regular	3.0	0.22	1.2	0.23	1.3
		Regular	3.7	0.19	1.3	0.21	1.3
		Tall	3.0	0.22	1.3	0.23	1.4
	9.1	Regular	3.0	0.22	1.2		
		Regular	3.7	0.19	1.3		
		Tall	3.0	0.22	1.4		
9	6.1	Regular	3.0	0.10	1.0	0.10	1.0
		Regular	3.7	0.09	1.0	0.09	1.1
		Tall	3.0	0.10	1.1	0.10	1.1
	9.1	Regular	3.0	0.11	1.1		
		Regular	3.7	0.09	1.1		
		Tall	3.0	0.11	1.1		
13	6.1	Regular	3.0	0.07	1.0	0.06	1.9
		Regular	3.7	0.05	0.9	0.05	0.9
		Tall	3.0	0.06	1.0	0.06	1.0
	9.1	Regular	3.0	0.07	1.0		
		Regular	3.7	0.05	0.9		
		Tall	3.0	0.06	0.9		
17	6.1	Regular	3.0	0.04	0.8	0.04	0.8
		Regular	3.7	0.03	0.7	0.04	0.8
		Tall	3.0	0.04	0.8	0.04	0.9
	9.1	Regular	3.0	0.05	0.8		
		Regular	3.7	0.04	0.8		
		Tall	3.0	0.05	0.9		

the steel with a uniform stress of $6\sqrt{f'c}$. Second-order effects (P-Δ) also were considered in the analyses.

Base Shear Strength and Yield Mechanisms

Base shear capacities for the proportioned frames were calculated using the static version of LARZ and a linearly increasing force distribution (Browning 1998). The lateral forces were applied to the frame model and increased incrementally. The base shear strength was determined as the total lateral load that is the lesser of:

1. The applied load associated with general yielding of the structure and for which additional lateral loads would cause the structure to collapse
2. The applied load associated with a roof drift approximately equal to 1.5% of the total height of the structure

The resulting base shear strengths normalized to the total building weights of selected frames are listed in columns 6 and 8 of Table 2. The threshold strength, C_y, was calculated with R_t = 8, α = 0.5, and T_g = 0.55 s. The selected parameters correspond to the idealized displacement response spectrum for the El Centro 1940 NS record scaled to a peak ground acceleration of 0.5 g. Because the period ratio (TR = $\sqrt{2}T_i/T_g$) exceeded one for all frames, the base shear strengths of the frames with V_b/C_y < 1.0 were considered adequate for the purposes of the study.

At all joints in the proportioned frames (excluding the roof), the total moment capacities in the columns exceeded 6/5 the total flexural capacities of the girders. This criterion was satisfied to discourage story mechanisms from forming in response to strong ground motion. The controlling yield mechanisms that were calculated for the proportioned frames using the nonlinear static analysis did not reveal any story mechanisms.

Calculated Displacement Response

The success of the proportioning method for providing earthquake-resistant reinforced concrete frames was tested using nonlinear dynamic analysis. The proportioned frames were subjected to a suite of ten scaled ground motions and the maximum displacement response was calculated for each frame. The performance of a frame was evaluated based on the calculated mean-drift ratio and the relative story drift per unit story height.

A series of ten earthquake motions were selected and scaled as described by Lepage (1997) to be used in the nonlinear dynamic analyses of the frames. Properties of the records are listed in Table 3. The selected motions refer to a variety of geographical locations and site conditions. They were chosen so that the combined displacement response spectra of the scaled motions would represent a maximum response envelope defined by (5) for response periods as high as three seconds. In this way, the frames were subjected to the prescribed level of displacement demand for the entire range of the proportioned building periods.

The displacement response spectra for the scaled earthquake motions are shown in Fig. 4 with the proposed idealized spectrum shown as a solid line. Although the displacement-response demand exceeds the defined spectrum in the period range around one second, the shapes of the spectra adequately represent the level of displacement demand expected for the full range of periods up to three seconds.

The maximum mean-drift ratios (ratio of maximum drift to total building height) calculated in the dynamic analyses for all the proportioned frames are shown in Fig. 5. The ratios for frames with girder depths equal to one-twelfth the span length ranged from 0.3 to 1.8%, with average ratios of 0.7% for both the 6.1 m (20 ft) bay frames and the 9.1 m (30 ft) bay frames. Only four out of the 420 analyses resulted in maximum mean-drift ratios that exceeded 1.5%. These occurred in five- and seven-story frames, three of which had 6.1 m (20 ft) bays.

The frames proportioned with 610 mm (24 in.) deep girders had maximum mean drift ratios ranging from 0.3 to 1.5%, with an average mean-drift ratio of 0.7%. The maximum mean-drift ratio occurred in the analysis of a seven-story frame with 3.0 m (10 ft) regular story heights and an initial period of 0.7 s. Because the initial periods of the frames with 610 mm (24 in.) deep girders were nearly identical to the initial frame periods

TABLE 3. Properties of Selected Ground Motions [from Lepage (1997)]

Event (1)	Station (2)	Component (3)	Source (4)	Duration (s) (5)	PGA (*g*) (6)	Scaled PGA (*g*) (7)
San Fernando: February 9, 1971	Castaic Old Ridge Route, California	N21E	CALTECH (1973b)	30	0.32	0.78
Northridge: January 17, 1994	Tarzana Ceder Hill Nursery, California	NS	CSMIP (1994)	30	0.99	0.62
Chile: March 3, 1985	Llolleo D.E.C., Chile	N10E	Saragoni et al. (1985)	75	0.71	0.55
Imperial Valley: May 18, 1940	El Centro Irrigation District, California	NS	CALTECH (1971)	45	0.35	0.50
Hyogo-Ken-Nanbu: January 17, 1995	Kobe KMMO, Japan	NS	"Strong" (1995)	30[a]	0.83	0.39
Kern County: July 21, 1952	Taft Lincoln School Tunnel, California	N21E	CALTECH (1971)	45	0.16	0.38
Western Washington: April 13, 1949	Seattle Army Base, Washington	S02W	CALTECH (1973a)	65	0.07	0.31
Miyagi-Ken-Oki: June 12, 1978	Sendai Tohoku University, Japan	NS	Mori and Crouse (1981)	40	0.26	0.29
Kern County: July 21, 1952	Santa Barbara Courthouse, California	S48E	CALTECH (1971)	60	0.13	0.27
Tokachi-Oki: May 16, 1968	Hachinohe Harbor, Japan	EW	Mori and Crouse (1981)	35	0.19	0.24

[a]Cut from original record at 25 s.

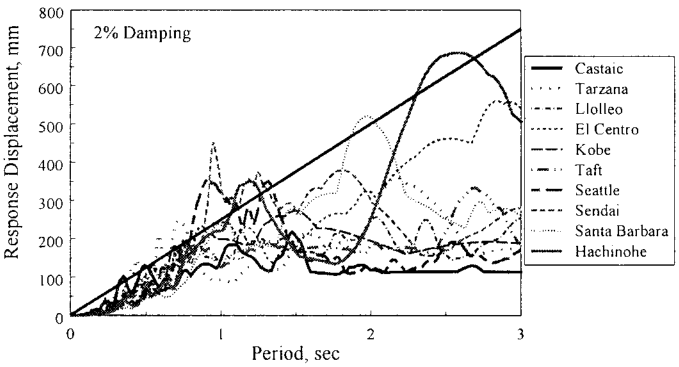

FIG. 4. Displacement Response Spectra for Scaled Motions

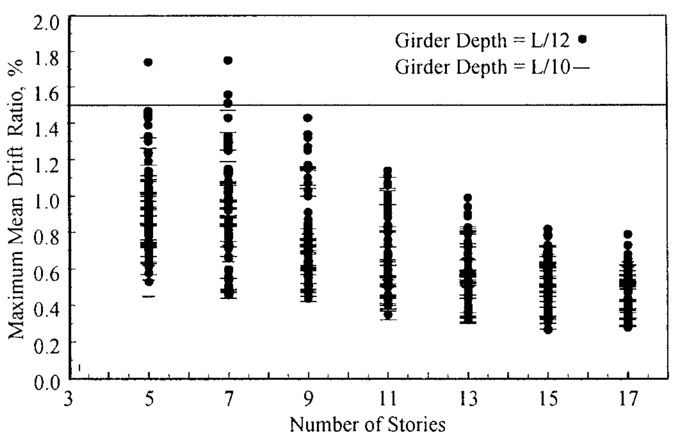

FIG. 5. Maximum Mean-Drift Ratios Calculated for All Frames

with 510 mm (20 in.) deep girders, it was evident that using deeper girder proportions had a greater effect on limiting the maximum drift (maximum drift-ratio of 1.5%) than increasing the column dimensions (maximum drift-ratio of 1.7%).

The maximum mean-drift ratios that were calculated for all frames with more than eleven stories were less than 1.0%. The taller frames experienced less drift because of a combination of two influences: the member sizes were controlled by gravity-load demands and the effective periods of the frames exceeded three seconds in some cases.

It is of interest to consider the inelastic response and local deformations calculated for the proportioned frames. For all frames, story mechanisms did not form in the response calculations for all earthquake records. Although some columns in the frames with 9.1 m (30 ft.) bays reached yield capacity, the distributions of yielding did not result in the formation of general or story yield mechanisms. In addition, all of the columns that yielded were located in the top half of the frames,

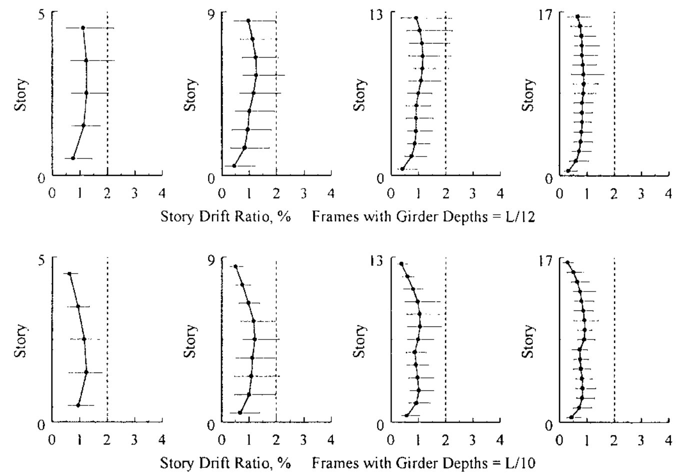

FIG. 6. Average Maximum Story-Drift Ratios with Maximum and Minimum Bounds for Selected Frames

so that second-order effects were less severe than the demands would have been if yielding had occurred in the lower stories.

Story-drift ratios calculated from the dynamic analyses indicate the relative local deformations that occurred in the frames. Graphical representations of the calculated maximum, average, and minimum story-drift ratios for selected frames are shown in Fig. 6. Twenty out of the 420 analyses for the frames with girder depths equal to one-twelfth the span length had maximum story-drift ratios that exceeded 2.0%. Only five of the analyses with frames having large girder proportions experienced story-drift ratios as large, indicating that using deeper girder proportions was more effective for limiting drift than increasing column sizes. It is shown in Fig. 6 that all of the average story-drift ratios were less than 1.5%.

CONCLUSIONS

The target-period method is a simple and efficient tool for proportioning of regular, moderate-rise earthquake-resistant reinforced concrete frames. A series of analytical reinforced concrete frames were proportioned using the target-period criterion and tested for the maximum displacement response using a suite of ten ground motions. Three primary conclusions were derived from the study:

- For the frames considered, lateral drift was controlled by satisfying the target-period criterion.
- The frames proportioned using a ratio of girder depth to span length (h_g/L) of 1:12 required a ratio of square column width to average column height (h_c/H_c) of approximately 1:4 to satisfy the target-period criterion for a max-

imum mean-drift ratio of 1.5% and the defined drift demand. When h_g/L was increased to 1:10, h_c/H_c decreased to approximately 1:6.
- For the range of parameters considered in this study, the calculated maximum drift was insensitive to strength. The critical decision for proportioning of earthquake-resistant structures then becomes the allocation of stiffness for a specified earthquake demand. Accordingly, the proposed target-period method makes the decision concerning structural stiffness for a specified earthquake demand the central issue in design.

The proposed method is advantageous because it implements a consistent philosophy between the response of structures to strong ground motion and the proportioning of these structures. This is achieved by separating the issues of design for gravity loads, which is conceptually dependent on force demands, from design for earthquake loads, which is conceptually dependent on drift demands. Another primary advantage of the proposed method is its simplicity. The earthquake demand is represented by a linear displacement demand spectrum that can be modified to include specific site condition effects. The proportions of an earthquake-resistant frame with threshold strength and adequate detailing are then determined simply by satisfying the target-period criterion.

ACKNOWLEDGMENTS

This investigation was completed with the support of The National Science Foundation Graduate Fellowship and under grant #9496280-CMS, Dr. S. C. Liu, Program Director. The writer would like to express her gratitude to Professor Mete A. Sozen for his guidance during the study. Special recognition also is given to Professors Julio Ramirez, Elisa Sotelino, and Terry West for serving on the advisory committee.

APPENDIX I. REFERENCES

American Concrete Institute Committee 318. (1999). ''Building code requirements for structural concrete and commentary.'' *ACI 318-99*, Detroit.

Browning, J. (1998). ''Proportioning of earthquake-resistant reinforced concrete building structures.'' PhD thesis, Purdue University, West Lafayette, Ind.

Browning, J., Li, Y. R., Lynn, A., and Moehle, J. P. (2000). ''Performance assessment for a reinforced concrete frame building.'' *Earthquake Spectra*, EERI, 16(3), 541–555.

California Institute of Technology (CALTECH). (1971). ''Strong motion earthquake accelerograms, digitized and plotted data, vol. II-A.'' *Rep. EERL No. 71-50*, Earthquake Engineering Research Laboratory, Pasadena, Calif.

California Institute of Technology (CALTECH). (1973a). ''Strong motion earthquake accelerograms, digitized and plotted data, vol. II-B.'' *Rep. EERL No. 72-50*, Earthquake Engineering Research Laboratory, Pasadena, Calif.

California Institute of Technology (CALTECH). (1973b). ''Strong motion earthquake accelerograms, digitized and plotted data, vol. II-D.'' *Rep. EERL No. 72-52*, Earthquake Engineering Research Laboratory, Pasadena, Calif.

California Strong Motion Instrumentation Program (CSMIP). (1994). ''Processed CSMIP strong-motion records from the Northridge, California, Earthquake of 17 January 1994.'' *Rep. No. OSMS 94-06–94-12*, California Dept. of Conservation, Div. of Mines and Geology, Office of Strong-Motion Studies, Sacramento, Calif.

Eberhard, M. O., and Sozen, M. A. (1989). ''Experiments and analyses to study the seismic response of reinforced concrete frame-wall structures with yielding columns.'' *Struct. Res. Series No. 548*, Civ. Engrg. Studies, University of Illinois, Urbana, Ill.

Goel, R. K., and Chopra, A. K. (1997). ''Period formulas for moment-resisting frame buildings.'' *J. Struct. Engrg.*, ASCE, 123(11), 1454–1461.

Hognestad, E. (1951). ''A study of combined bending and axial load in reinforced concrete members.'' *Bulletin Series No. 399*, University of Illinois Engineering Experiment Station, Urbana, Ill.

International Conference of Building Officials (ICBO). (1997). *Uniform building code*, Whittier, Calif.

Lepage, A. (1997). ''A method for drift-control in earthquake-resistant design of reinforced concrete building structures.'' PhD thesis, University of Illinois, Urbana, Ill.

Lopez, R. R. (1988). ''Numerical model for nonlinear response of R/C frame-wall structures.'' PhD thesis, University of Illinois, Urbana, Ill.

Mohraz, B., and Elghadamsi, F. E. (1989). ''Chapter 2.'' *The seismic design handbook*, F. Naeim, ed., Van Nostrand Reinhold, New York, 32–80.

Mori, A. W., and Crouse, C. B. (1981). ''Strong motion data from Japanese earthquakes.'' *Rep. SE-29*, Nat. Geophysical Data Ctr., National Oceanic and Atmospheric Administration, Boulder, Colo.

Newmark, N. M., Hon, M., Blume, J. A., and Kapur, K. K. (1973). ''Seismic design spectra for nuclear power plants.'' *J. Power Div.*, ASCE, 99(2), 287–303.

Otani, S. (1974). ''SAKE: a computer program for inelastic response of R/C frames to earthquakes.'' *Struct. Res. Series No. 413*, Civ. Engrg. Studies, University of Illinois, Urbana, Ill.

Qi, X., and Moehle, J. P. (1991). ''Displacement design approach for reinforced concrete structures subjected to earthquakes.'' *Rep. No. UCB/EERC-91/02*, Earthquake Engrg. Res. Ctr., University of California, Berkeley, Calif.

Saiidi, M., and Sozen, M. A. (1979a). ''Simple and complex models for nonlinear seismic response of reinforced concrete structures.'' *Struct. Res. Series No. 465*, Civ. Engrg. Studies, University of Illinois, Urbana, Ill.

Saiidi, M., and Sozen, M. A. (1979b). ''User's manual for the LARZ family: computer programs for nonlinear seismic analysis of reinforced concrete planar structures.'' *Struct. Res. Series No. 466*, Civ. Engrg. Studies, University of Illinois, Urbana, Ill.

Saragoni, R., Gonzalez, P., and Fresard, M. (1985). ''Analisis de los acelerogramas del terremoto del 3 de Marzo de 1985.'' *Publicacion SES-I-4/1985(199)*, Universidad de Chile.

Shibata, A., and Sozen, M. A. (1976). ''Substitute-structure method for seismic design in reinforced concrete.'' *J. Struct. Div.*, ASCE, 102(3), 1–18.

Shimazaki, K., and Sozen, M. A. (1984). ''Seismic drift of reinforced concrete structures.'' *Tech. Res. Rep. of Hazama-Gumi*, Tokyo, 145–166.

Strong motion accelerograms. (1995). Japan Meteorological Agency, Tokyo.

Takeda, T. M., Sozen, M. A., and Nielsen, N. N. (1970). ''Reinforced concrete response to simulated earthquakes.'' *J. Struct. Div.*, ASCE, 96(12), 2557–2573.

APPENDIX II. NOTATION

The following symbols are used in this paper:

A_s = spectral acceleration;
C_y = required base shear strength coefficient;
c = slope of idealized displacement demand spectrum;
D_s = spectral displacement;
D_t = tolerable displacement specified for system;
E = modulus of elasticity;
EPA = effective peak ground acceleration;
F_a = acceleration amplification factor;
F_p = participation factor;
f'_c = compressive strength of concrete;
f_y = yield stress of steel;
g = acceleration of gravity;
H_c = average column height for frame;
h = story height;
h_c = dimension of column;
L = total bay length;
M_c = flexural capacity of column;
M_{cr} = moment at cracking;
M_g = flexural capacity of girder;
M_y = flexural capacity of member at yield;
PGA = peak ground acceleration;
R_t = strength reduction factor;
T = period of system;
T_g = characteristic period of design ground motion;
T_i = initial calculated period of system;
TR = period ratio;
T_t = target period;
V_b = base shear strength coefficient;
α = coefficient of peak ground acceleration;
ϕ_{cr} = curvature at cracking; and
ϕ_y = curvature at yield.