ANALYSIS OF RECORDED BUILDING DATA TO VERIFY OR IMPROVE 1991 UNIFORM BUILDING CODE (UBC) PERIOD OF VIBRATION FORMULAS

Eugene E. Cole
Cole, Yee, Schubert & Associates

Christos V. Tokas
Cole, Yee, Schubert & Associates

John F. Meehan
Consulting Structural Engineer

ABSTRACT

A data base of 64 buildings was developed to investigate their fundamental period of vibration recorded during recent California earthquakes. These periods are compared with those calculated from the 1991 **Uniform Building Code** (1991 UBC) and the 1990 Structural Engineers Association of California **Recommended Lateral Force Requirements** (Blue Book). Steel moment frames, braced frames, concrete moment frames, and shear wall buildings are included in the data base. Also included in the study are buildings from the Gates & Foth report [12].

INTRODUCTION

The 1991 UBC establishes two methods of determining the fundamental period of vibration of buildings when the static force procedure is used. This report analyzes strong motion records from the California Strong Motion Instrumentation Program (CSMIP) and others to compare the fundamental period of vibration of buildings determined from these records and the calculated periods from the 1991 UBC. Graphs are presented showing the instrumented building periods and those obtained from the 1991 UBC period equations. These graphs indicate close correlation for steel moment frame and concrete moment frames. Shear wall periods are also shown. Insufficient data is available for braced frames and eccentrically braced frames. Further study is necessary to determine the accuracy of period formulas for shear wall buildings.

OBJECTIVE

The primary objective of this study is to review strong motion data to verify or recommend improvements in current seismic code formulas for the fundamental period of vibration of buildings.

METHODOLOGY

A data base of instrumented buildings that have experienced recorded ground motion of more than 0.05g was developed. This data base was obtained from information provided by CSMIP reports.

The buildings were tabulated and classified within one of the following lateral resisting system categories: Steel moment frames, concrete moment frames, concentric and eccentric braced frame, shear walls, and mixed lateral systems. Construction drawings of 64 buildings contained in the CSMIP archives were reviewed to verify the building classification.

Only regular buildings, instrumented by CSMIP or others, of a type contemplated by the 1991 UBC are included in this study. These buildings are shown in Tables 1, 2, and 3. The following unusual building types were omitted: Buildings containing a flexible first story, buildings constructed with base isolation devices, a building constructed in a pyramid shape, buildings with offset shear walls, buildings with mixed lateral systems, and buildings using a flat slab or a waffle slab as the horizontal moment frame member.

The fundamental period was obtained by scaling the processed data, adopting published values determined by other researchers, or by performing a non-parametric system identification procedure in the frequency domain.

The scaling process involves averaging the time interval between the peaks of the roof acceleration or displacement time-history curves processed by CSMIP in that portion of the record after the initial strong input motion. These values, however, include the influence of the site on the period. For flexible buildings, the values are competent. For stiff buildings, the scaling process is inaccurate since the displacement of the roof at the top of the wall is essentially the same as the ground. The following non-parametric system identification procedure in the frequency domain was used to identify the fundamental period of vibration for several of the buildings:

1) The Fourier amplitude spectrum graphs were computed from the corrected acceleration records by Fourier Transformations. The Fourier amplitude spectrum graph for the roof and base of a building were developed for each of the orthogonal building directions.

2) The transfer functions were computed by dividing the Fourier amplitude spectrum of acceleration recorded at the roof of the building by that recorded at the base. Thus, the building response was isolated from the soil-structure system, and therefore, the transfer function exhibited the dynamic characteristics of the building without the influence of the site.

3) The natural periods of the building can be obtained from the frequencies at which the peak values of the transfer function occur. The fundamental period is calculated from the frequency value at the first peak of the transfer function.

RESULTS

General: The building periods inferred from the strong motion records of the recent California Earthquakes are plotted against story height in Figures 1, 2, and 3 for moment resisting steel frame, moment resisting reinforced concrete frame, and shear wall buildings. Two values of period are shown for each building, and are joined with a connecting line. These values represent periods in each orthogonal direction. The graphs in Figures 1 and 2 were expanded by including buildings from the Gates & Foth Report [12].

A curve representing the period ,T_A, obtained from the code formula using Method A which is given in Section 2334(b) 2.A 1991 UBC, is shown on Figures 1, 2, and 3. It should be noted that this curve indicates shorter period values than the actual building periods obtained from the strong motion records for the moment resisting frame structures.

The UBC also allows the use of Method B for determining the period. When this method, based on the Rayleigh Formula, is used a limitation is placed on C of 80% of the value of C obtained by using T from Method A for regular buildings. This places an upper limit on the value of T obtained from Method B. This period is shown in Figures 1 and 2 as T_{MOD} ($T_{MODIFIED}$). T_{MOD} is calculated using the following procedure:

$$T_A = C_t\ (h_n)^{3/4} \qquad (1)$$

$$C_A = 1.25\ S \div T_A^{\ 2/3} \qquad (2)$$

$$C_{MOD} = .80\ C_A = 1.25\ S \div T_{MOD}^{\ 2/3} \qquad (3)$$

Solving equation (3) for T_{MOD}

$$T_{MOD} = [(1.25\ S \div (0.8\ C_A)]^{3/2} \qquad (4)$$

Combining equations (2), (3), and (4):

$$T_{MOD} = \{1.25\ S \div [(0.8)\ (1.25\ S \div T_A^{\ 2/3})]\}^{3/2} \qquad (5)$$

Simplifying further equation (5) becomes:

$$T_{MOD} = 1.4\ T_A \qquad (6)$$

Steel Moment Frames: In Figure 1, for building heights above 75 feet, the curve of the Method A period, T_A, is below the data points giving shorter periods than the records. The difference between the period value given by this curve and the actual data is large for taller buildings. The T_{MOD} curve provides a conservative, yet reasonable, approximation of the measured period for all buildings above 75 feet in height with the exception of two buildings from the Gates & Foth study [12]. The lateral load systems of both of these buildings were investigated and it was found that a steel tubular framing system was used for one, and composite concrete and steel column with steel truss girders was used for the other. These buildings are not included in Figure 1.

Concrete Moment Frames: In Figure 2, the curve for Method A period, T_A indicates shorter periods than those determined from the records. Six buildings from the Gates & Foth study are not included. The lateral load resisting system of these six buildings did not have a 100% moment resisting frame in one of the directions. For the remaining buildings, the T_{MOD} curve provides a reasonable approximation of the measured period.

Shear Walls: Figure 3 summarizes the data obtained from the shear wall buildings. The curve , T_A, obtained from the code formula using Method A is shown in Figure 3. The periods from the records are academically interesting; however, they have little significance for short period buildings since the maximum value of the spectral amplification factor C equal to 2.75 (regardless of soil type) establishes the lower limit for periods of shear wall buildings.

$$C = 1.25\ S \div T^{2/3}$$

$$\text{For soil type } S_1,\ 2.75 = 1.25\ (1.0) \div T^{2/3}$$

$$\text{Therefore, } T^{2/3} = 1.25\ (1.0) \div 2.75 = 0.455$$

$$\text{and , } T = 0.31$$

Therefore, only shear wall building periods longer than the lower limit set by the code at 0.31, 0.40, 0.56, and 0.81 seconds for soil types S_1, S_2, S_3, and S_4, respectively, affect the base shear coefficient. These values are plotted on Figure 3.

A recent CSMIP study of a low-rise stiff shear wall building with a flexible diaphragm shows that the response of this structure was dominated by the dynamic properties of the flexible roof diaphragm [26]. Conventional code design procedures for buildings with this type of lateral load resisting system assume dynamic amplification in the shear walls and a uniform acceleration of the diaphragm, which does not correlate with the observations of the above study.

Braced Frames: Insufficient data is available to develop a comparison between code period formulas for braced frames and those determined from instrumented buildings.

SUMMARY AND CONCLUSIONS

This study compares the results given from the code empirical formulas for estimating the fundamental period of building structures to the fundamental periods obtained from the strong motion records. The primary results of this study may be summarized as follows:

1) Steel and Concrete Moment Frames

   A) The fundamental periods of vibration of the buildings obtained from the strong motion records are longer than the period values computed by the Method A equations given in Section 2334 (b) 2.A 1991 UBC.

   B) The 80% limitation on C from Method A, when Method B (Section 2334 (b) 2.B 1991 UBC) is used, results in a maximum period from Method B of 1.4 times the period determined from Method A. This modified period correlates well with results obtained from the data.

2) Shear Walls

   A) Although measured periods of shear wall buildings indicate much lower values than the code formulas estimate, these lower measured periods have no effect on the base shear calculations since the upper limit of 2.75 is placed on the value of C. It is appropriate to re-evaluate this limit since it has the effect of negating soil amplification in short period buildings.

FUTURE RESEARCH

1) More data is necessary to evaluate periods for concentrically and eccentrically braced frames.

2) Further study is necessary to determine the accuracy of period formulas for shear wall buildings.

3) Investigate the influence of softer soils and flexible diaphragms on short period buildings to determine if these conditions cause larger forces than currently anticipated by the code.

4) The 1991 UBC requires only a single value for building period for both principal axes when both axes are framed in the same framing system. The stiffnesses along each orthogonal axes of a building may be different. This will result in different values for the fundamental period of vibration along each axis. Recent studies have shown that in buildings where a stiffer lateral load resisting system is employed in one direction than the other, the predominant motion occurs in the softer direction [17, 18]. Therefore, the building response along one principal axis is not only dependent on the period along that axis, but also relates to the building period in the other principal axis. This condition should be further studied and the code revised accordingly.

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TABLE 1

STEEL MOMENT RESISTING FRAME STRUCTURES AND RELATED PERIODS

BLDG No.	BUILDING NAME	CSMIP STATION No.	EARTHQUAKE	HEIGHT H (ft)	PERIOD T, (sec)			
					UBC MTHD-A	UBC T-MOD.	TRANS.	LONG.
1	Burbank 6-story	24370	Whittier	82.50	0.96	1.34	1.30	1.32
2	Long Beach 7-story	14323	Whittier	91.00	1.03	1.44	1.50	1.19
3	Palm Springs 4-story	12299	Palm Springs	51.50	0.67	0.94	0.63	0.71
4	Richmond 3-story office	58506	Loma Prieta	44.00	0.60	0.84	0.76	0.60
5	San Bernardino 3-story	23516	Whittier	42.00	0.58	0.81	0.46	0.50
6	San Francisco 18-story	58480	Loma Prieta	230.00	2.07	2.89	3.33	2.26
7	San Francisco 47-story	58532	Loma Prieta	564.00	4.05	5.66	5.00	6.50
8	San Jose 13-story	57357	Loma Prieta	186.60	1.77	2.47	2.23	2.23
9	San Jose 3-story	57562	Loma Prieta	49.50	0.65	0.91	0.69	0.69
10	South San Francisco - 4-story	58261	Loma Prieta	52.50	0.68	0.95	0.71	0.71

TABLE 2

CONCRETE MOMENT RESISTING FRAME STRUCTURES AND RELATED PERIODS

BLDG No.	BUILDING NAME	CSMIP STATION No.	EARTHQUAKE	HEIGHT H (ft)	PERIOD T, (sec)			
					UBC MTHD-A	UBC T-MOD.	TRANS.	LONG.
1	Los Angeles 5-story	24463	Whittier	119.00	1.08	1.51	1.30	1.40
2	N.Hollywood 20-story	24464	Whittier	169.00	1.41	1.97	2.21	2.15
3	Pomona 2-story	23511	Whittier	30.00	0.38	0.54	0.80	0.70
4	San Bruno 6-story	58490	Loma Prieta	78.00	0.79	1.10	1.10	0.85
5	Sherman Oaks 13-story	24322	Whittier	187.50	1.52	2.12	2.30	1.90
6	Van Nuys 7-story	24386	Whittier	65.71	0.69	0.97	1.20	1.40
7	Emeryvile 30-story	USGS	Loma Prieta	300.00	2.16	3.02	2.80	2.80

TABLE 3

SHEAR WALL BUILDINGS AND RELATED PERIODS

BLDG No.	BUILDING NAME	CSMIP STATION No.	EARTHQUAKE	HEIGHT H (ft)	PERIOD T, (sec)		
					UBC MTHD-A	TRANS.	LONG.
	R/C SHEAR WALL BUILDINGS						
1	Belmont 2-story	58262	Loma Prieta	28.00	0.24	0.20	0.13
2	Burbank 10-story	24385	Whittier	119.00	0.72	0.51	0.57
3	Goleta 3-story	25213	Santa Barbara	33.00	0.28	0.35	0.30
4	Hayward 4-story	58488	Loma Prieta	50.00	0.38	0.22	0.15
5	Long Beach 5-story	14311	Whittier	71.00	0.49	0.34	0.17
6	Los Angeles 17-story	24601	Sierra Madre	149.72	0.86	1.00	1.00
7	Oakland 24-story	58483	Loma Prieta	219.00	1.14	3.23	2.32
8	Palm Desert 4-story	12284	Palm Springs	50.20	0.38	0.60	0.50
9	Piedmont 3-story	58334	Loma Prieta	36.00	0.29	0.18	0.18
10	Pleasant Hill 3-story	58348	Loma Prieta	40.58	0.32	0.46	0.38
11	San Bruno 9-story	58394	Loma Prieta	104.00	0.65	1.30	1.20
12	San Jose 10-story resid.	57356	Loma Prieta	96.00	0.61	0.42	0.70
13	Saratoga 1-story	58235	Loma Prieta	33.00	0.28	0.18	0.31
14	Watsonville 4-story	47459	Loma Prieta	66.33	0.46	0.35	0.24
	R. MASONRY SHEAR WALL BUILDINGS						
15	Concord 8-story	58492	Loma Prieta	74.92	0.51	0.38	0.74
16	Lancaster 3-story	24517	Whittier	41.50	0.33	0.20	0.21
17	Palo Alto 2-story	58264	Loma Prieta	23.83	0.22	0.34	0.27

FIGURE 1. STEEL MOMENT RESISTING FRAMES

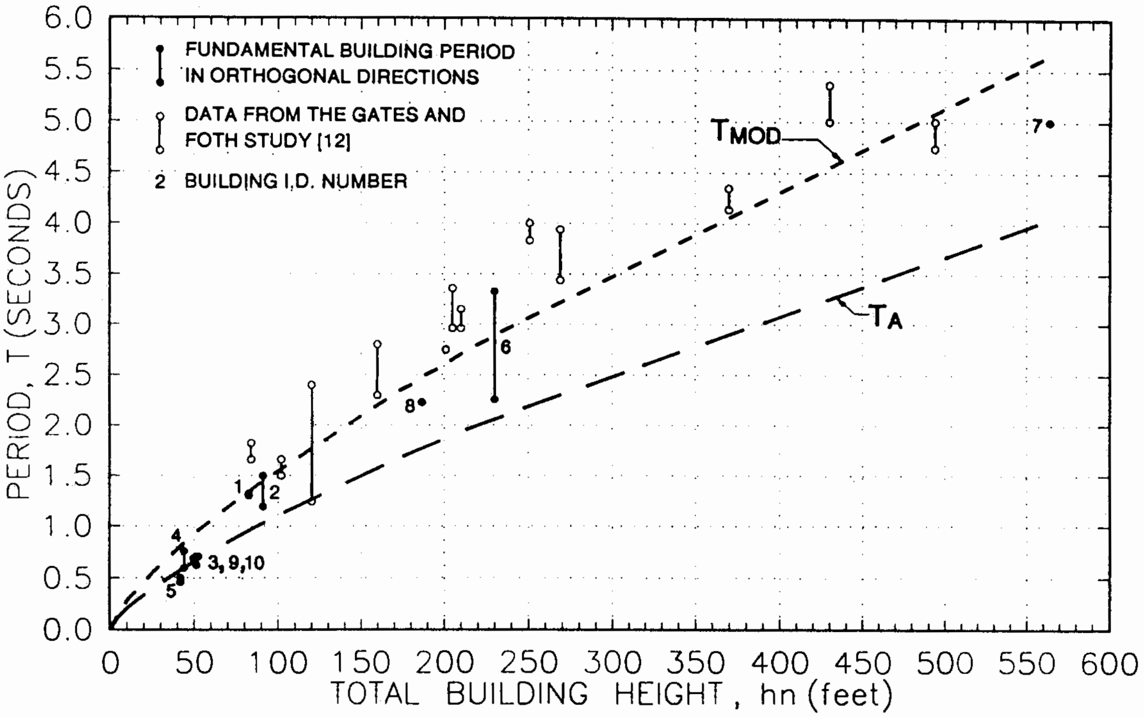

FIGURE 2. R/C FRAMES

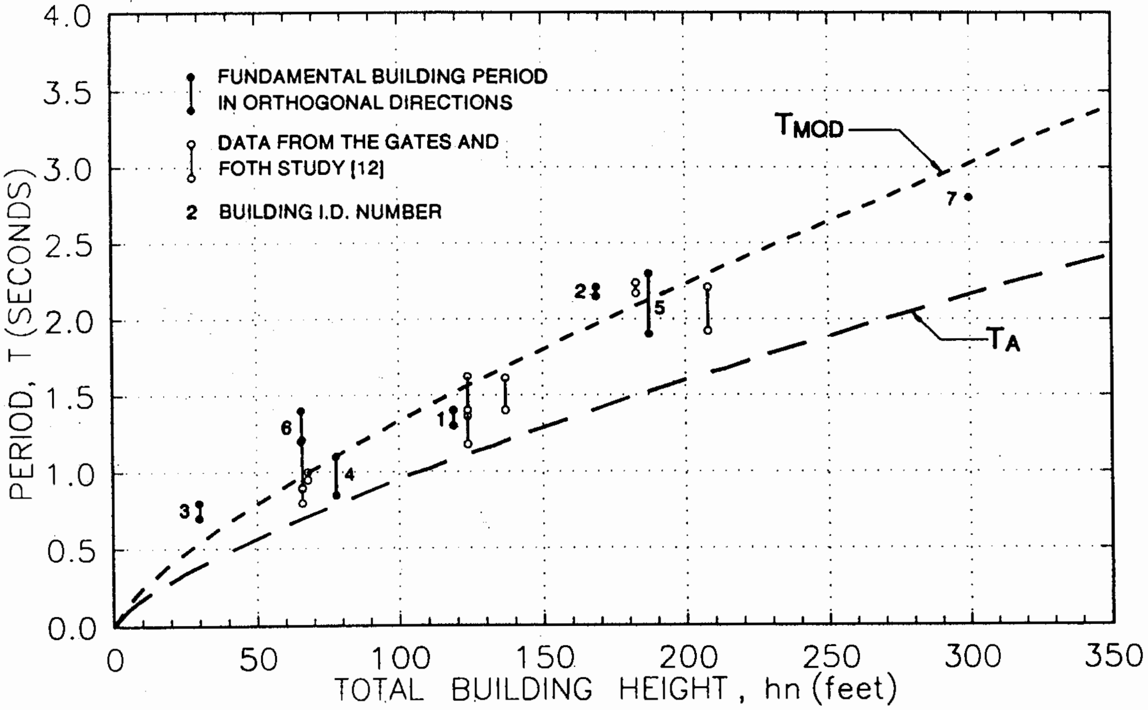

FIGURE 3. SHEAR WALL BUILDINGS

REINFORCED CONCRETE AND MASONRY