VERIFICATION OF A METHOD FOR ESTIMATING OVERTURNING MOMENT OF TALL BUILDINGS

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ABSTRACT

The objective of this research is to verify a newly developed method ,time delay (TD) method, for earthquake analysis using data from strong motion instrumentation program (SMIP), and then applying the TD method to calculate overturning moment of structures. The TD method incorporates the attribute of finite wave speed propagating upward into the structure.

One detailed 3-dimensional model and nine simplified 2-dimensional models were used for study. The effects on the overturning moment due to the use of TD method varies depending upon the predominant period of the input earthquake motions and the structures. The concept of the TD method is verified with the measured data.

INTRODUCTION

The traditional equation of motion for single degree-of-freedom (SDOF) and multiple degree-of-freedom (MDOF) lumped-mass systems under earthquake excitation has the same general form (Refs.,1 to 4):

$$M\ddot{u} + C\dot{u} + Ku = P_{eff}(t) \qquad (1)$$

For a ground motion acceleration time history $\ddot{u}_g(t)$, the effective earthquake force vector of a MDOF system for a multistory building as shown in Figure 1 is

$$P_{eff}(t) = -M\{1\}\ddot{u}_g(t) \qquad (2)$$

In which {1} is a column vector of ones. Mathematically this implies that at a certain instant ti, all the masses in the structure are subjected to the same ground excitation without any spatial variations. Conceptually this means that the incident earthquake wave propagates throughout the entire structure instantaneously with an infinite speed, once it reaches the base of the structure. As is well known in classic mechanics, disturbances into a system of Eq.(1) will propagate with a finite speed into the solution domain, instead of an infinite speed as used in the traditional approach (Ref.5).

The TD method incorporates the attribute of finite wave speed in the forcing function, $P_{eff}(t)$ The following sections describe the conceptual and mathematical development, verification procedures using SMIP data, and application to calculate overturning moment of structures.

CONCEPTUAL AND MATHEMATICAL DEVELOPMENT

The two basic principles, equilibrium and compatibility, are the basis for the development of structural mechanics. When they are applied alone without wave propagation, the principles can be satisfied both locally and globally. With wave propagation, they are satisfied only locally or in the subdomain of solution and not necessarily the whole global domain of solution at certain instant.

An example, to illustrate the principle of compatibility coupling with wave propagation, is the motion of a hanging flexible string as shown in Figure 2(a). When one moves the string horizontally but slowly enough so not to generate any vibratory motion, as shown in Figure 2(b), an amount of translation at the top end produces the same amount of translation along the whole string at the same instant (i.e., there is rigid body motion compatible with the boundary condition of the string.). This apparently trivial case is actually of importance, since this is in fact the basis for developing the temporal variation only forcing function of earthquake motion, i.e. Eq.(2).

As one moves the string just fast enough to produce a swinging motion (or 1st mode of vibration), as shown Figure 2(c), the time variation of the input motion along the string is such that the spatial distribution of the effective force along the string is almost uniform. The characteristics of the motion are predominantly first mode. This therefore would be the case if one assumes a "temporal variation only" forcing function, and the motion of the hand is close to the fundamental mode of the string.

As one then vibrates the string faster , one can easily see the travelling of the motion along the string, and that the string vibrates in higher modes,as shown in Figure 2 (d). This last demonstration illustrates the main idea of this paper that a transient disturbance to the system will travel with finite speed, and the portion nearest the disturbance moves first followed very closely by the other portions. Namely, the top portion is subjected to force first, while the lower portions are subjected to the same force later. At a certain instant, there is nonuniform distribution of inertia force along the string. Therefore, the forcing function does not only have temporal variation but also spatial distribution.

Mathematically, the inertia force generated due to the wave motion thus has both spatial distribution and temporal variation. This spatio-temporal forcing function therefore has the following form in contrast to Eq.(2)

$$P(x,t)=M\ddot{U}g(x-ct) \qquad (3)$$

Pictorially, the existing method assumes the ground motion propagates through the structure instantaneously, and the masses at various levels are subjected to the same force simultaneously, as shown in Eq.(2) and in Figure 3(a). With the inclusion of finite wave speed, the masses at various levels are not subjected to the same force simultaneously as shown by Eq. (3), and in Figure 3(b).

VERIFICATION PROCEDURES FOR THE TD METHOD

A total of 10 buildings with earthquake records from California Strong Motion Instrumentation Program (CSMIP) were selected for study. One of the buildings was selected for detailed modeling and nine others were used for simplified analysis. The 10 buildings selected for analysis are listed in Table 1.

The building selected for detailed analysis was modeled as a 3-dimensional model as shown in Figure 4. All major structural frames were included; floor diaphragm was assumed rigid. A time-history analysis was then performed. This building was also modeled as a simplified model similar to Figure 1. The other nine buildings were modeled using simplified models only, similar to Figure 1. Time history analyses were performed for each building.

APPLICATION TO OVERTURNING MOMENT CALCULATION

The calculation of overturning of a building at its foundation seems simple, but is actually one of the most perplexing problems in design practice. Since the inclusion of seismic loads into the UBC (Ref.6), the overturning problem has been treated from a very conservative manner to relatively liberal manner, then adjusted to be conservative again as shown in Figure 5.

Initially the overturning stability problem was treated statically same as the lateral wind load with the requirement of overturning moment to be one-half (1/2) of the resisting moment. In the 1961 UBC, a J factor was introduced to reduce the overturning in recognition of the participation of higher modes of vibration. However, the code still required a comparison of wind and seismic overturning effects to select the governing load. In recent years, the requirement for comparing the wind load with seismic load for overturning effects was dropped. This essentially eliminates the requirement for overturning moment to be less than two-third (2/3) of the resisting moment. As the code stands today, UBC-91 does not require the overturning moment to be less than the resisting moment at the foundation-soil interface.

The new proposed TD method was used to study the overturning moment at the base of structure by comparing the maximum overturning moment (OTM) calculated from time history analyses with and without time delay.

SUMMARY OF FINDINGS

Table 2 summarizes the results in terms of predominant period of the input earthquake motions, the fundamental period of the structure, the displacement at the top of structure and overturning moment at the base of structure with and without time delay.

The results of the studies indicate that the input motion considering time delay tends to excite the higher modes of the structure. However, the effects on structural response depend on several factors including the relationship between the predominant period of the input earthquake motions and the structure, stiffness and height of the structure. Following are some major observations:

1. Where the predominant period of the input earthquake motions is close to the higher modes of structure, the structural response will be predominantly in the higher modes. For the case of 47-story office building, the input earthquake has the predominant period of 1 second, and the major response at 44th floor is also 1 second, Figures 6 (a) and (b). According to the analysis, this is close to the 3rd mode of the structure. The frequency analysis and the time history at the 44th floor indicate a fundamental building period of about 5.2 sec, Figure 6(c). For this case, the TD method shows higher responses than method without time delay, as shown in Case 5 of Table 2 and Figure 6. Case 1, the S.F. 18-story, has similar result.

2. Where the period of the input earthquake motions is close to the fundamental period of the structure, the TD method shows lower responses than method without time delay as shown in Cases 2,3,4,6 and 10 of Table 2.

3. Where the buildings damping ratio is unusually high, both the responses with and without time delay are very close. The Long Beach City Hall indicates a high damping ratio of 17%. This can also be seen from the damped-out roof displacement in the trailing portion of the recorded time history shown in Figure 7.

CONCLUSIONS

1. The TD method considering input motion with time delay tends to excite the higher modes of the structures. Therefore, the overturning moment calculated using the fundamental mode may be too conservative for high rise structures.

2. Where the predominant period of the input earthquake motion is in the higher modes of the structure, the TD method shows higher responses than method without time delay.

3. Where the period of the input earthquake motion is close to the fundamental period of the structure,the TD method shows lower responses than method without time delay.

4. For highly damped structures, the effects of time delay are negligible.

5. The TD method considering input forcing function with time delay is conceptually and mathematically sound. It effects may be significant depending on the predominant period of the input earthquake motion and the characteristics of the structures.

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TABLE 1

Buildings elected For Analyses

Name	EQ Record	PGA AT Base	Lateral Force Resisting System
1. S.F. 18-Story Commercial bldg.	Loma Prieta Oct.17, 1989	0.17g	Steel Moment-Resisting Frame
2. San Jose 13-Story Government Office Building	"	0.11g	Steel Moment-Resisting Frame
3. San Bruno 6-Story Office Building	"	0.14g	Concrete Moment -Resisting Frame
4. So. S.F. 4-Story Hospital	"	0.14g	Concrete shear wall at 1st. fl. Steel Moment-Resisting Frame above
5. S.F. 47-Story Office Building	"	0.13g	Steel Moment-Resisting Frame
6.San Jose 3-Story Office Building	"	0.10g	Concrete Moment-Resisting Frame and Shear Walls
7. Burbank 6-Story Calif. Federal Saving Building	Whitter, Oct. 1,1989	0.22g	Steel Moment Resisting Frame
8. Long Beach City Hall 15-Story	"	0.06g	Steel Moment-Resisting Frame
9. Long Beach Harbor Administration Building, 7- Story	"	0.07g	Steel Frame
10 Van Nuys Holiday Inn, 7-Story	"	0.17g	Concrete Moment-Resisting Frame

TABLE 2

SUMMARY OF RESULTS

Case	EQ Predominant Periods Second	Structural Periods Second	Displacement At Roof		O.T.M. Ratio Delay / No Delay
			W/o Delay	W/ Delay	
1. S.F. 18-Story	1.0 and less	$T_1 = 2.4$ $T_2 = 0.67$	3.3"	3.7"	1.47
2. San Jose 13-Story	2.8 and less	$T = 2.3$	16.6"	15.3"	0.89
3. San Bruno 6-Story	1.3 and less	$T = 0.85$	2.19"	2.05"	0.92
4. S.F. 4-Story	0.7 and less	$T = 0.7$	3.8"	4.4"	0.85
5. S.F. 47-Story	1.0 and less	$T_1 = 5.2$ $T_2 = 2.0$ $T_3 = 1.2$	3.1"	6.4"	1.19
6. San Jose 3-Story	1.0 and less	$T = 0.65$	3.1"	2.9"	0.86
7. Burbank 6-Story	1.0 and less	$T = 1.30$	1.57"	1.57"	1.06
8. Long Beach 15-Story	1.5 and less	$T = 3.2$	1.4"	1.53"	1.02
9. Long Beach 7-Story	1.0 and less	$T = 1.4$	1.2"	1.3"	0.97
10. Van Nuys 7-Story	0.5 and less	$T = 1.2$	1.19"	1.14"	0.97

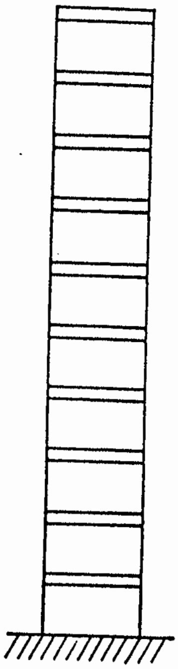


FIGURE 1 MDOF SYSTEM

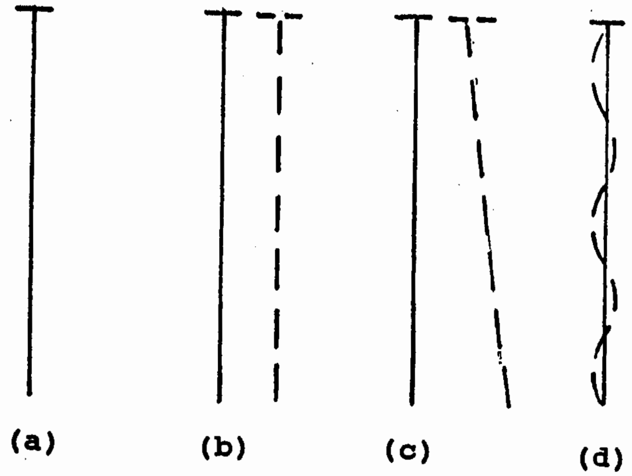
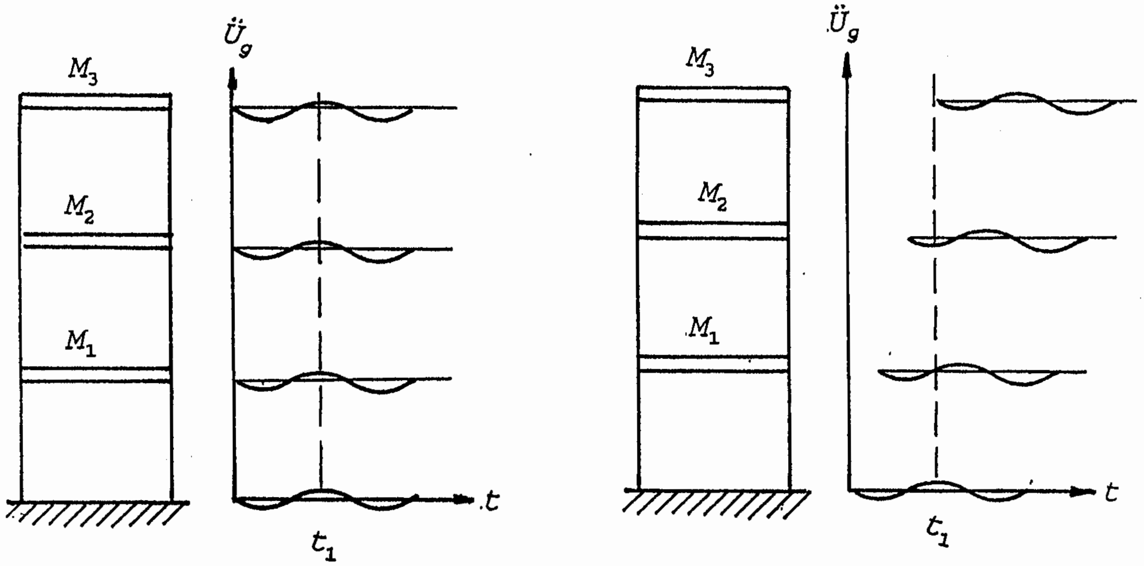


FIGURE 2 VIBRATION OF FLEXIBLE STRING



(a) INFINITE SPEED

(b) FINITE SPEED

FIGURE 3 INPUT MOTION PROPAGATION WITH INFINITE AND FINITE SPEED

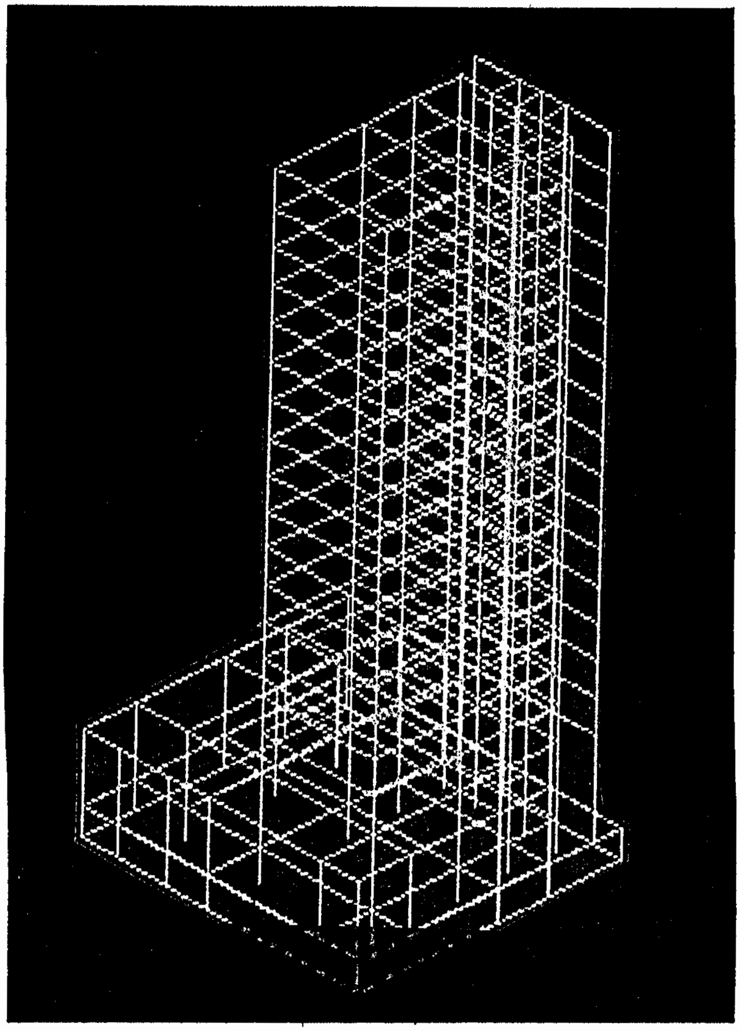


FIGURE 4 3-D MODEL OF S.F. 18-STORY BUILDING

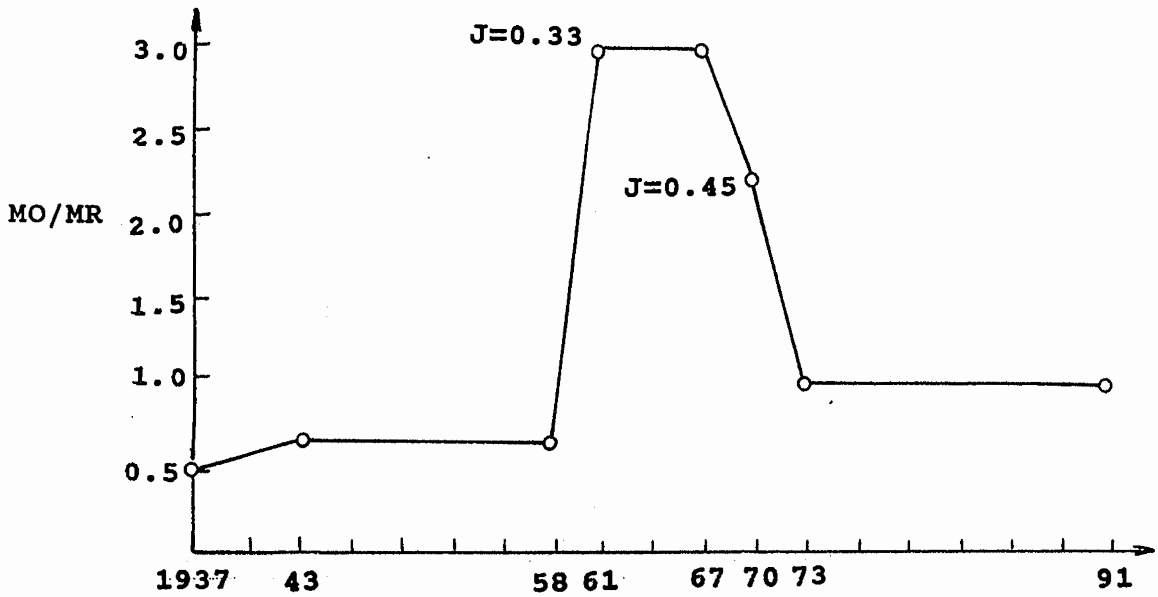
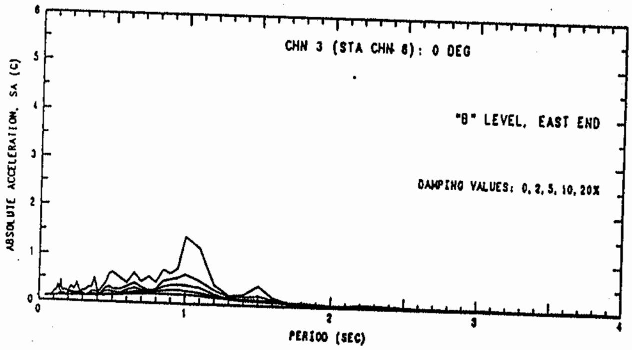
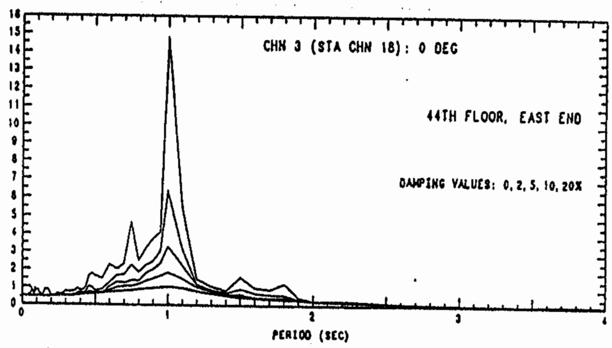


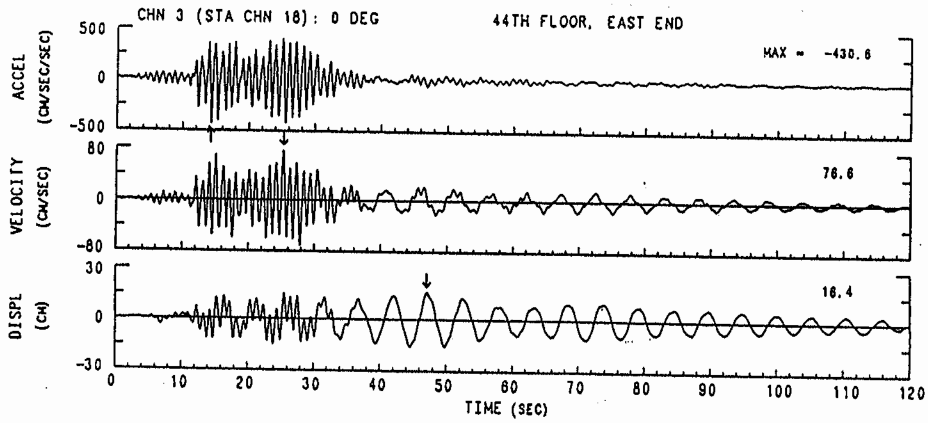
FIGURE 5 RATIO OF MO/MR, UBC



(a) RESPONSE SPECTRA AT "B" LEVEL



(b) RESPONSE SPECTRA AT 44TH FL.



(c) TIME HISTORY AT 44TH FL.

FIGURE 6 RECORDED DATA- S.F. 47-STORY OFFICE BLDG.
(FROM OSMS 91-07)

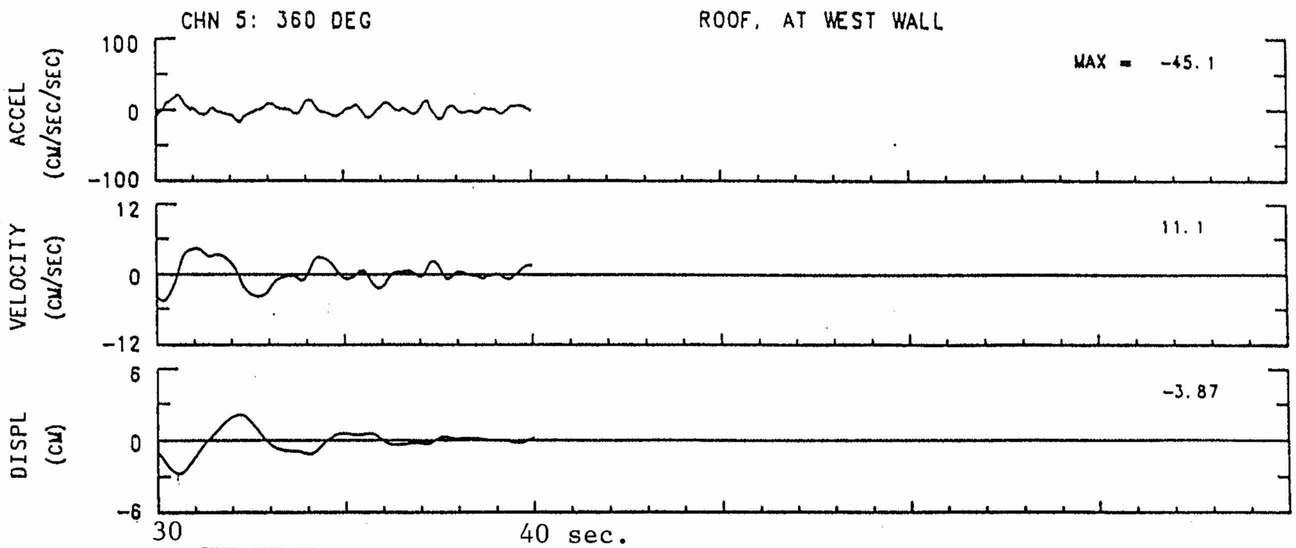


FIGURE 7 RECORDED DATA- LONG BEACH CITY HALL
(FROM CSMIP)