OBSERVED CHANGES IN THE NATURAL PERIODS OF VIBRATION OF A NINE STORY STEEL FRAME BUILDING

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SYNOPSIS

A case study is presented of the variable nature of the dynamic properties of a nine-story steel frame building based on vibration tests conducted during the last 12 years at the Jet Propulsion Laboratory in Pasadena, California. Observed changes of nearly 50 percent in the natural periods of vibration are attributed to observed changes in the mass and stiffness distribution of the structure caused by the addition of building materials during construction, by large relative displacements and dynamic forces experienced during the earthquake, and by the repair program conducted after the earthquake. The implications of these changes in the dynamic characteristics of the structure for those who perform dynamic analysis are discussed.

Introduction:

One particularly interesting observation based on studies of recorded motions in multistory buildings during the San Fernando earthquake is that these structures behaved as nonlinear softening dynamic systems. It was noted that the fundamental periods of vibration of virtually all instrumented buildings were longer than those determined before the earthquake during ambient vibrations at much lower levels of excitation. Also, studies of ambient vibrations undertaken after the earthquake revealed that the fundamental periods were longer than before the earthquake but shorter than during the event.

This nonlinear behavior is not surprising in itself, since forced vibration tests at levels of excitation much lower than those experienced during the San Fernando earthquake have revealed similar results in several buildings. The degree of nonlinear behavior during the earthquake demonstrated by buildings that experienced no structural damage is somewhat surprising however. The 15-story steel frame Kajima Building[3] in Los Angeles, experienced a change in its fundamental period of vibration of more than a factor of 2 which roughly corresponds to a loss in the overall stiffness of over a factor of 4. The fundamental period lengthened by a factor of 1.5 for the Millikan Library Building,[3] a 9-story reinforced concrete building located on the campus of the California Institute of Technology. These losses in stiffness have been attributed, for the most part, to changes in the behavior of nonstructural elements such as architectural precast curtain walls, plaster partitions, and others. A more accurate determination of the causes of these nonlinearities is usually not possible due to insufficient data.

In this paper, the authors consider various possible contributing factors to the nonlinear behavior of Building 180 of the Jet Propulsion Laboratory. This building is well suited for such a study since a considerable amount of data is available concerning the contribution to the dynamic behavior of various of its structural, as well as its nonstructural elements.

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Description of the Building

Building 180 was designed in 1961 and serves as an administration building for the Caltech Jet Propulsion Laboratory. The building is located in Pasadena, California approximately 15 miles from the center of energy release of the San Fernando earthquake. The nine-story steel frame structure has plan dimensions 40 by 200 feet. It measures 146 feet from foundation to roof with 114 feet above grade on the north side and 130 feet above grade on the south side.

The steel frame, which was designed to carry all loads, is atypical for southern California. In the E-W (longitudinal) direction, very rigid, welded steel trusses are connected to the columns by high strength bolts. These truss girders are typically 6'6" deep with 8-inch channel sections for top and bottom chords. In the N-S (transverse) direction, long span welded truss girders act compositely with 5 inch reinforced light weight concrete slabs. The 3'4" deep truss girders are also bolted to the columns with high strength bolts. The foundation system is composed of two continuous strip footings which run beneath the columns in the longitudinal direction and rest on firm alluvium. A dynamic analysis of the building indicated that the maximum stresses in the steel frame during the earthquake were not quite at yield point, even though the peak accelerations recorded were 20% g in the basement and 40% g on the roof.[2]

Observed Changes in the Periods of Vibration of Building 180

The natural periods of vibration of Building 180 at various times during its history are shown in Table 1. These periods were determined from measurements taken during forced vibration tests while the building was under construction,[1] from records of the response of the structure during the San Fernando earthquake[2] and from ambient tests conducted on three different occasions after the San Fernando earthquake. Due to this unusual amount of data presented in Table 1, an interesting picture of the variability of the dynamic characteristics of Building 180 is made available.

As expected, tests 8, 9, and 10 indicate that the natural periods of vibration of the building were longer during the earthquake than they were either before or after the event. An interesting observation can be made by comparing the period of vibration just after the earthquake with those characteristic of the earthquake response of the building. The lengthening of the fundamental periods by factors of 1.30 in the N-S direction and 1.23 in the E-W directions correspond to decreases in the overall stiffness by factors of 1.69 and 1.51 on the N-S and E-W directions respectively. Nielsen[1] observed very similar changes in the overall stiffness of the building between tests 1 and 4. The increases in overall stiffness computed by Nielsen were 1.75 in the N-S direction and 1.45 in the E-W direction and resulted from the addition of the concrete to the columns. These results seem to imply that changes observed during the earthquake could be the result of the loss in stiffness due to cracking of the encasing concrete of the columns. However, it does not seem appropriate to assume that the concrete provides zero added stiffness to the columns merely because it had cracked. More likely it is a combination of concrete cracking combined with nonstructural damage; but, since the cracked column concrete would be expected to influence the strong earthquake response of the structure but not the low level ambient response, one should see a difference in the periods of vibration for the two cases.

The structure seems to have regained some of its stiffness between July 1971 and July 1974. During this time the damage sustained by the structure during the earthquake was repaired. Also, during this time an inspection of the building indicated that 5 to 10 percent of the high strength bolts connecting the N-S truss girders to the columns were loose. The reason for this is unknown. The tightening of these bolts was completed just prior to test 11. It is unlikely that bolt tightening had a significant affect at these low levels of excitation. One would expect the N-S period to be influenced more than the E-W period; but the tests indicated a larger change in the E-W direction. The only other change in the building during this time was the removal of the 48,000 pound cooling tower from the roof. A simple calculation assuming the building to be a uniform shear beam with a concentrated mass at the end of the beam indicated that the removal of this weight would change the periods by less than 1 percent.

No changes in the periods of vibration were observed during the last year. Apparently the "healing" process has reached completion. However, an interesting coincidence is observed when the final measured periods of vibration are compared to those of test 4 which was conducted when all of the structural work had been finished and only nonstructural additions remained to be completed. The nearly identical periods of vibration measured in these tests do not imply that only structural elements are resisting motion during test 12 since the mass of the building was not the same for both tests. The mass was increased approximately 33 percent between tests 4 and 12 which implies that the overall stiffness has increased by a factor of approximately 1.75.

Concluding Remarks

It has been pointed out in this paper that various interpretations of the data presented here can lead to different conclusions regarding the source of nonlinearities observed in the behavior of the building. This indicates that a large amount of data does not guarantee that an exact representation of a structure can be made. On the contrary, these results indicate that even a simple multistory building is quite complex and one should not expect to describe it exactly regardless of the sophistication of the model. This is a valuable lesson since it demonstrates the fallacy of trying to formulate an extremely sophisticated finite element model. A prediction of the fundamental period of vibration of a particular structure may be 20 to 30 percent different from that observed during an earthquake, which implies estimates of stiffness and strain that are off by 40 to 50 percent. The authors believe, however, that the changes in stiffness that a structure experiences as indicated by the change in its fundamental period occurs quite early in its response to an earthquake and at relatively small deformation. Consequently, the energy absorbed in the process would be relatively small. Thus, a linear model should be adequate to describe the motion of a building up to the time of incipient yielding, or damaging of its structural members.

References

1. Nielsen, N. N., "Dynamic Response of Multistory Building," Ph.D. Thesis, California Institute of Technology, Pasadena, California (1964).

2. Wood, H. J., "Earthquake Response of Steel Frame Building," Earthquake Eng. and Str. Dynamics, Vol 4, 349-377 (1976).

3. Foutch, D. A., G. W. Housner and P. C. Jennings, "Dynamic Responses of Six Multistory Buildings During the San Fernando Earthquake," EERL 75-02, California Institute of Technology, Pasadena, California (1974).

4. Teledyne-Geotech, "Ambient Vibration Survey of Building 180, Jet Propulsion Laboratory," report for California Institute of Technology, Pasadena, California (1971)

Figure 1. View of South Elevation of Jet Propulsion Laboratory Building 180.

DISCUSSION

Jai Krishna (India)

The change of period due to change of strain was reported in the paper "Dynamic Behavior of Water Towers" by Krishna and Chandrasekaran in IV WCEE as a result of model tests in the laboratory. Authors have got this result during an actual earthquake is a distinct confirmation of this phenomenon. This may shake the faith in mathematical models since they tend to base their results on a fixed period and damping but possibly these changes in these important properties are more due to non linear behavior of concrete elements. The period in the above paper jumped from 0.9 sec to 1.4 sec due to large strains in concrete during earthquake but came back to 1.1 sec because the concrete recovered and had small residual strains resulting in a small increased in period after the earthquake.

Author's Closure

The comments of Dr. Jai Krishna are well taken. In the event of strong shaking large stresses and strains can produce changes in the effective stiffness and changes in the effective damping. It would be a mistake to think of structures behaving always in the linear elastic range during an earthquake. It is important to study more about how these changes occur and what the significance is as regards the ability of the structure to remain standing.