ANALYSIS OF A TWO STORY OAKLAND OFFICE BUILDING DURING THE LOMA PRIETA EARTHQUAKE

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ABSTRACT

A three-dimensional model of a two story Oakland office building was subjected to response spectra and time history analyses developed from the California Division of Mines and Geology Strong Motion Instrumentation Program (SMIP) Loma Prieta earthquake records. Although the building had a severe plan torsional irregularity, and was subjected to large peak ground, second floor and roof accelerations, the building suffered no damage. These dynamic analyses showed that the building was twice as stiff and strong as required by current 1988 Uniform Building Code provisions.

INTRODUCTION

The purpose of this study is to obtain a better understanding of the excellent performance of a two story building in Oakland which was subjected to large peak ground and spectral accelerations during the Loma Prieta earthquake. The building was instrumented by the State of California, Department of Conservation, Division of Mines and Geology Strong Motion Instrumentation Program in 1974. The building is designated as an Oakland two story office building, CSMIP Station No. 58224. The location of the ten sensors is shown in Figure 1. The acceleration records for these sensors are shown in Figure 2.

This report is written from the viewpoint of structural engineers who design new buildings and evaluate the performance of existing buildings. They are concerned with the maximum response of any particular element of the building, the duration of strong ground and building shaking, the periods of the significant modes of vibrations, and the maximum displacements. Strong motion records provide the data to be used with computer analyses to obtain the above information.

From the maximum response of an element, the structural engineer can design the member to resist the forces on the member. Duration and magnitude of shaking gives an indication of the number of cycles of large member forces and displacements. These data are important for the design and evaluation of ductile, semi-ductile and brittle members. The maximum displacement and inter-story drift give clues to possible damage to non-structural elements of a building. Designing a building with low inter-story drift criteria reduces the displacements of and damage to the non-structural elements.

BUILDING DESCRIPTION

The building is a two story office building located in Oakland, California, designed in 1964, and built in 1965. The plan dimensions are 150 ft. in the east-west direction and 163 ft. in the north-south direction with 14 ft. 6 in. story heights. See Fig. 3. The building was designed to have a future third story which was never built. It is located on the north-east corner of the street intersection with a three story building to the north and a one story building to the east.

The building is a structural steel moment frame structure with reinforced concrete fill over roof and second floor metal decking welded to the steel frame. The first floor is a reinforced concrete slab on grade. The foundations are reinforced concrete spread footings for the interior columns and continuous deep footings for the exterior walls and columns.

The building structure has a two story complete welded structural steel frame. All beam and girder to column connections are moment connections. The top and bottom flanges of each beam and girder to column connections are full penetration butt welded to stiffener and continuity plates which were full penetration butt welded to column flanges and webs. The beam and girder web connections consist of single web plates that are butt welded to the column web and flanges, and fillet welded to the beam and girder webs the full height of the web plates with return fillet welds top and bottom. There are no web doubler plates. These beam and girder to column welded connections are similar to "Special" steel moment-resisting space frame connections used in current construction, even though they were constructed over twenty-five years ago.

The north and east walls are on the property lines and are non-bearing four hour fire walls. They are constructed of 8" stack bond solid grouted reinforced concrete block. These concrete block walls are well anchored to the structural steel columns and beams, and the roof and second floor reinforced concrete fill. The exterior south and west walls are non-structural walls constructed of 6" nailable steel studs @ 16" o.c.

The north and east continuous 8" concrete block walls with the structural steel moment frames throughout the building produce a building with a severe torsional plan irregularity. The center of rigidity of the lateral force resisting system is located close to the intersection of the north and east continuous block walls. This torsional plan irregularity was recognized early in the architectural and structural design of the building. The architectural design was modified to allow closer column spacing in the west and south street window walls which increased the in-plane lateral force stiffness of these walls.

There are fifteen columns in the south wall frame as compared to six in the other east-west frames. Fourteen columns are located in the west wall frame as compared to seven columns in the other north-south frames. See Fig. 3.

The exterior west and south window walls are finished with interior plaster and exterior stucco. These walls are non-bearing walls and have almost continuous narrow height strip windows in the second story and large window openings in the first story. After the Loma Prieta earthquake, the interior plaster and exterior stucco south and west walls showed only minor cracks which could have been shrinkage cracks there before the earthquake.

The 1961 soils investigation report shows three soil borings to forty-two feet. The typical soil profile is firm silty sand to sand of the Merritt Formation underlie the site to depths of 30 to 40 feet. Below the Merritt Formation is sandy and clayey silt of the Alameda Formation extending to the depths explored. The static ground water, in 1961, was at 24 foot depth. According to the 1988 Uniform Building Code, Table 23-J, an S-2 soil with an S factor = 1.2 is appropriate for the site with a soil profile of dense or stiff soil conditions, where the soil depth exceeds 200 feet.

1964 BUILDING DESIGN

Frank E. McClure of Frank E. McClure and David L. Messinger, Consulting Structural Engineers, was the partner in charge, designer and engineer of record.

The building was designed to conform to the 1961 Oakland Building Code which followed the 1961 Uniform Building Code (UBC) lateral force provisions. It was designed as a three story office building with a mechanical space within the future third story. The design lateral base shear was calculated using the following formula: V = ZKCW, where Z = 1.0, K = 0.67, C = 0.05/ T**1/3 exp. power, T = 0.10 N = .30 sec., producing C = 0.075. V = 1.0 x .67 x .075 W = .05 W. The tributary weights of the future roof, future third, and second floor were 1,150 k, 2,840 k, and 2050 k, respectively, or a total weight of 6,040 k. Base shear V = .05 W = .05 x 6,040 = 302 k.

It is important to note that in the 1961 UBC, K = 0.67 could be used for buildings with a moment resisting space frame which, when assumed to act independently of any other more rigid elements, is capable of resisting 100 per cent of the total required lateral forces in the frame alone. There were no "Ordinary" and "Special" steel moment frame provisions.

The 1961 UBC drift requirements consisted of the following statement: "Lateral deflections or drift of a story relative to its adjacent stories shall be considered in accordance with accepted engineering practice." It was required to increase horizontal torsional moments resulting from an eccentricity between the center of mass and center of rigidity of not less than five per cent of the maximum building dimension.

With the plan torsional irregularity in the lateral force resisting system resulting from the stiff 8" concrete block walls on the east and north property lines, it was necessary to provide a stiff framing system in the south and west walls. A stiff structural steel moment frame was designed for these walls recognizing that it would be almost impossible for these frames to be as stiff as the concrete block property line walls.

The building was designed with 100% of the design lateral forces being resisted by the 8" reinforced concrete block wall parallel to the lateral forces. A separate design was prepared, wherein the 8" block walls were neglected, and 100% of the design lateral forces were resisted by the complete structural steel moment frames parallel to the lateral forces. All beam and girder to column connections were welded moment connections, not just in the south and west wall perimeter frames.

Based on very rough approximations of relative rigidities of the structural steel frames and engineering judgment, the south and west frames were designed for V = 116 k or about 40% of the total building lateral force parallel to each of these frames. This design base shear was much larger than would have been obtained by rigorous seismic analysis taking into consideration the relative rigidities of the concrete block walls and the steel moment frames, including the increase in the torsional moments due to a 5% additional accidental torsional eccentricity.

The 1961 UBC design requirements for structural steel frames were simpler than those in the 1988 UBC. Panel zone shear and drift calculations were not required. No column web doubler plates were provided. The steel frames as built do not conform with the panel zone shear requirements of the 1988 UBC. However, the beam and girder to column connections develop the flexural capacity of the beams and girders. Continuity and stiffener plates were provided which would conform to the 1988 UBC provisions for "Special" steel moment-resisting space frames. The structural steel frames in the south and west walls more than meet the drift and AISC unity check requirements of the 1988 UBC.

AMBIENT WIND AND FORCED VIBRATION MEASUREMENTS

In April 1965, the United States Coast and Geodetic Surveys measured the first mode period of the building using a Sprengnether Portable Seismograph and a Ranger Lunar seismometer. The wind excited period of the building in the north-south direction was 0.47 sec. and 0.48 sec. in the east-west direction. At the time of these measurements, the concrete block walls and the complete structure were in place. The south and west street exterior metal stud walls were in place but had not been plastered.

Later in 1965, forced vibration tests were performed on the building when the construction was almost complete. Only minor plastering of the mechanical penthouse walls and completion of the painting of the building remained to be completed. A complete description of these vibration tests and the results can be found in a paper, "Dynamic Response of a Two Story Steel Frame Structure," J. G. Bouwkamp and J. K. Blohm, Vol. 56, No. 6, Bulletin of the Seismological Society of America, December 1966.

Based on the forced vibration measurements, the first mode period was 0.426 sec. and the second mode period was 0.130 sec. The decrease in these periods, as compared with the ambient wind periods, can be attributed to the increase in the stiffness of the building due to the added stiffness of the exterior plastered south and west walls.

Having the first and second modes of vibration from the forced vibration measurements provided a unique opportunity to validate the computer modeling assumptions, if the computer model first and second modes of vibration were close to the forced vibration values.

COMPUTER MATHEMATICAL MODEL

A rigorous three-dimensional mathematical model of the building was prepared with 196 nodes, 321 beam-column elements and 88 membrane elements. This model included consideration of the flexibility of the roof and second floor diaphragms and the stiffness of the non-structural elements. The IMAGES-3D Finite Element Analysis Program developed by Celestial Software, Inc. was used.

In the development of a computer mathematical model to simulate the building's dynamic characteristics, the modeling of the non-structural elements can have an influence on the periods of the modes of vibrations for low levels of building excitation. Under the current study, computer analyses of the building without plastered exterior walls produced a first mode period = 0.63 sec. and a second mode period = 0.20 sec. as compared to a first mode period = 0.463 sec. and a second mode period = 0.172 sec. for the building with plastered exterior walls.

The model with the plastered exterior walls was considered a viable model when it produced a first mode period = 0.463 sec. and second mode period = 0.172 sec. as compared with the forced vibration first mode period = 0.426 sec. and second mode period = 0.130 sec.

Referring to Fig. 2. structural response record for south-east roof corner Sensor 3, the peak roof acceleration of 0.65 g occurred at about 14 seconds into the record. Prior to this peak acceleration, the period of the roof response was about 0.50 sec. and about 0.60 sec. after the peak acceleration. This period lengthening can be explained since after the peak response, the non-structural elements were not as well-connected to the structure and their lateral stiffness was diminished.

1988 UNIFORM BUILDING CODE SEISMIC ANALYSIS

Using the above computer model, a conventional 1988 UBC lateral force analysis was performed that took into account the flexibility of the roof and second floor diaphragms. The roof and second floor horizontal seismic forces were distributed throughout the building to the appropriate nodes based on their tributary nodal masses.

Based on the as-built two story building with a mechanical penthouse, the 1988 UBC lateral force base shear, V = ZICW/Rw, where Z = 0.40, I = 1.0, C = 1.25 S/T**2/3 exp. power, S for S2 soil = 1.2, T = Ct x (hn)**3/4 exp. power = 0.035 x (29)**3/4 exp. power = 0.437 sec. Therefore, C = 1.25 x 1.2/ 0.437**2/3 exp. power = 2.59, and V = .40 x 1.0 x 2.59 W/Rw. It was assumed that the structural steel frame met the requirements of a "Special" steel moment frame because of the detailing of the beam and girder to column connection with stiffener and continuity plates butt welded to the column web and flanges. Therefore, Rw = 12 was used. V = .40 x 1.0 x 2.59 W/12 = 0.086 W for Rw = 12.

The tributary weights of the roof and second floor were 2,200 k and 2,050 k, or a total weight of 4,250 k. Base shear V = 0.086 x 4,250 k = 366 k. This base shear of 366 k is about 20% greater than the 1961 UBC base shear = 302 k.

In order to compare the results of the 1988 UBC equivalent static lateral force analysis with the response spectra and time history analyses, it was decided to focus on the response of the south wall structural steel frame, designated as "Frame 7." Frame 7 is parallel to the orientation of Roof Sensor 3, Second Floor Sensor 5, and Ground Floor Sensor 7. The base shear forces on Frame 7 and its roof computed response were used to compare the results of the 1988 UBC equivalent static force analyses, the response spectra and time history analyses. These base shear forces, roof displacements and columns bending stresses are shown in Table 1.

The total 1988 UBC east west base shear = 366 k. The total base shear without accidental torsion for Frame 7 = 110 k or about 30% of the total building east west base shear. Accidental torsion added 9.8 k additional base shear to Frame 7, and amplified torsion required by Section 2312 e (6), 1988 UBC, added 2.78 times the accidental torsion of 9.8 k or 27 k base shear. The total Frame 7 base shear = 137 k which represents 37% of the total building base shear. It is interesting to note that the 1961 UBC Frame 7 base shear was about 40% of the total building base shear or 116 k without the use of computer analysis.

TABLE 1.

MAXIMUM RESPONSE OF SOUTH WALL STRUCTURAL STEEL FRAME

	INPUT	BASE SHEAR	ROOF DISPLACEMENT	COLUMN BENDING STRESS
1.	1961 UBC	116 k	0.40 in.	7,900 psi
2.	1988 UBC	137 k	0.47 in.	9,300 psi
3.	Response Spectra 5% Damping	203 k	1.19 in.	14,500 psi
4.	Time History 2% Damping	160 k	0.89 in.	12,200 psi
5.	Time History 5% Damping	133 k	0.80 in.	9,900 psi
6.	SMIP Sensor 7		0.80 in.	

LOMA PRIETA RESPONSE SPECTRA ANALYSES

Three-dimensional 5% damped response spectra analyses were performed, using Complete Quadratic Combination (CQC) combined 9 modes of vibration, which included 96 per cent of the participating mass of the structure. The requirements that orthogonal effects be considered was satisfied by using 100% of the east-west plus 30% of the north-south response spectra.

The total building base shear, V = 921 k or 22% of the total weight of the building. This building base shear resulting from the response spectra analysis is 921 k / 366 k = 2.5 times greater than the 1988 UBC base shear.

The base shear, V, for Frame 7 = 203 k. The Frame 7 columns members had bending stresses = 14,500 psi, or about one-half the 1988 UBC allowable bending stresses. The roof displacement = 1.19 in. or an inter-story drift of (.0036) or 70% of the 1988 UBC allowable inter-story drift of (.0050). See Table 1.

LOMA PRIETA TIME HISTORY ANALYSES

Three-dimensional time history analyses, using 2% and 5% damping, were performed, which included the time histories of the east-west, north-south, and vertical ground motions run concurrently. The largest total building shear, V = 608 k or 14% of the total weight of the building. This building base shear is 608 k / 366 k = 1.66 times greater than the 1988 Uniform Building Code base shear.

The 5% damped time history, maximum base shear, V, for Frame 7 = 133 k, and its roof displacement = 0.80 in. which corresponds to the Sensor 7 displacement of 0.80 in. This displacement represents an inter-story drift of (.0024) or one-half the 1988 UBC allowable inter-story drift of (.0050). The 2% damped time history responses were slightly greater and the column member bending stresses = 12,200 psi or 40% of the 1988 UBC allowable bending stresses. See Table 1.

FINDINGS AND CONCLUSIONS

1. A three-dimensional computer model was validated based on its first and second modes of vibrations which matched the forced vibration first and second modes. Using this model and the SMIP strong motion ground motion records, the time history analyses produced the displacements of the roof and second floor which were very close to the displacements shown in the strong motion records.

2. During the Loma Prieta earthquake, the roof, second floor, and ground were subjected to peak accelerations of 0.65 g, 0.39 g, and 0.26 g, respectively. However, according to the time history analyses, the total maximum base shear force on the building at any one time was 14% of the weight of the building.

3. Without performing response spectra and time history analyses, it is difficult to explain the excellent performance of the building when just comparing the design base shear, V equals 5.0% x the weight of the building with the maximum recorded peak ground acceleration equal to 26% g. Depending on the type of structural system and its configuration, buildings can resist peak ground accelerations many times greater than their design base shears, expressed as a percentage of gravity.

4. According to the time history analyses, the south wall structural steel frame was subjected to a lateral force about equal to the 1988 UBC lateral forces including the provisions for amplified torsion. The steel frame members were subjected to 34% of their maximum UBC allowable bending stresses. The frame deflected 50% of the 1988 UBC allowable drifts. In other words, the structural steel frame was designed to be more than twice as strong and twice as stiff as required by the 1988 UBC provisions.

5. This inter-story drift stiffness helps explain the lack of any damage in the building despite its severe plan torsional irregularity. Minimizing the inter-story drifts by designing a stiff structural steel frame along the two street window walls helped mitigate the stiffness of the opposite property line masonry walls. The use of over-strength and stiffness can mitigate severe plan torsional irregularities.

6. Time history analyses provide better information concerning the performance of an existing building than response spectra analyses. A better understanding of the variation with time of the displacements and stresses in the members is possible by "stepping" through the response of a building in 0.02 sec. time history intervals. Response spectra analyses, particularly using smoothed design response spectra, are more appropriate for the design ofnew buildings. Use of response spectra analyses, developed from on-site ground strong motion records, tend to overstate the response as compared to time history analyses, using the same ground strong motion records.

7. Multi-channel strong motion instrumentation records provide important data to validate the performance of a building during an earthquake. The records provide data concerning the acceleration, velocity and displacements at various locations of the building, which aids in the validation of the computer models used to evaluate the building. Computer analyses which can reproduce the measured responses in the building are more credible than computer analyses without such validation.

8. For buildings with plan torsional irregularities, the 1988 UBC provisions require inclusion of amplified torsion up to 3.0 times the accidental torsion. This amplified torsion provision requires increasing the design forces in the perimeter structural elements in the building. If damage control is a design goal, then reduction of the allowable inter-story drifts for these elements also should be considered. The excellent performance of the subject building can be attributed mainly to the stiffness in the street window wall structural steel frames.

ACKNOWLEDGEMENT

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SENSOR LOCATIONS

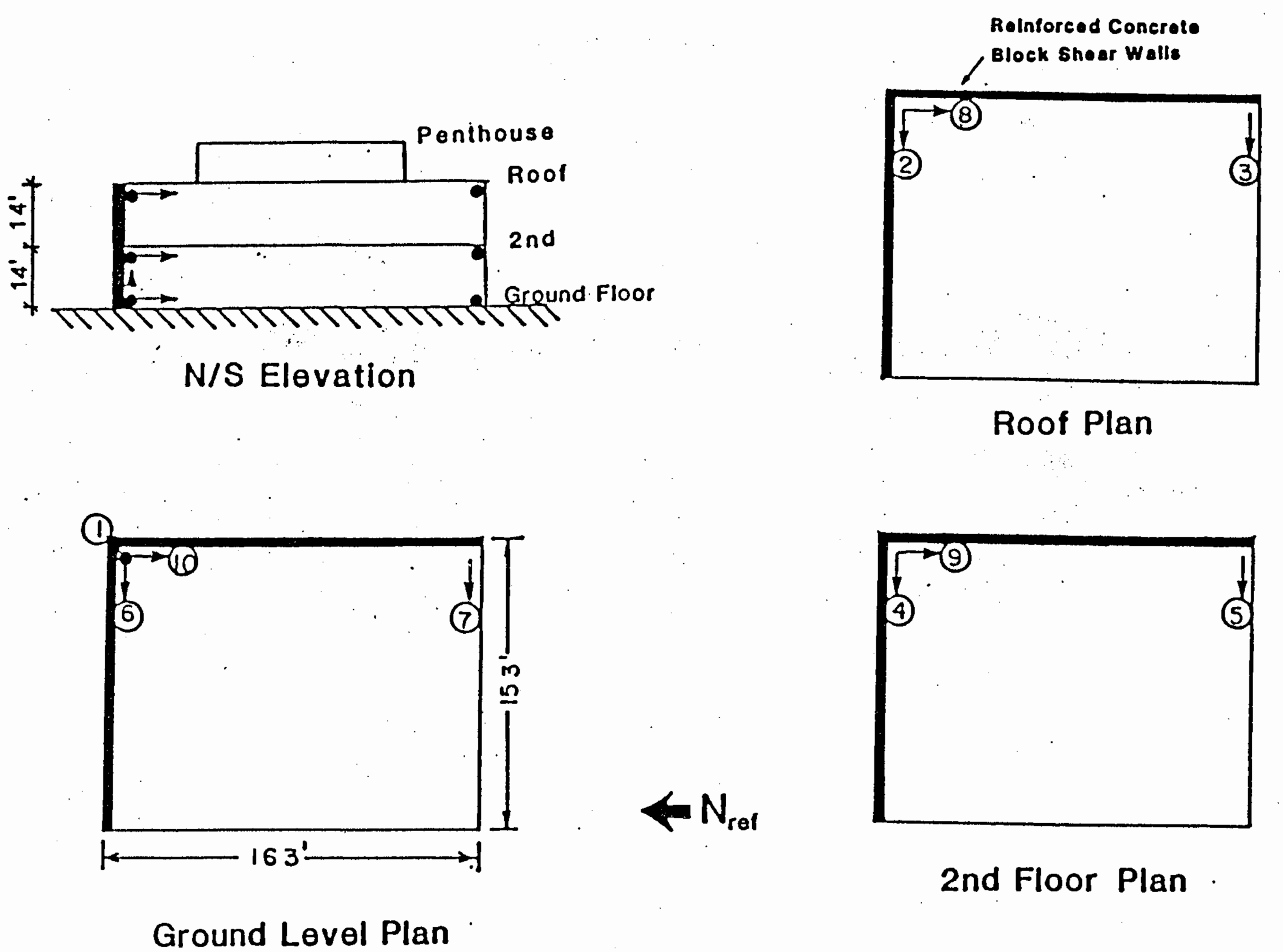

Figure 1

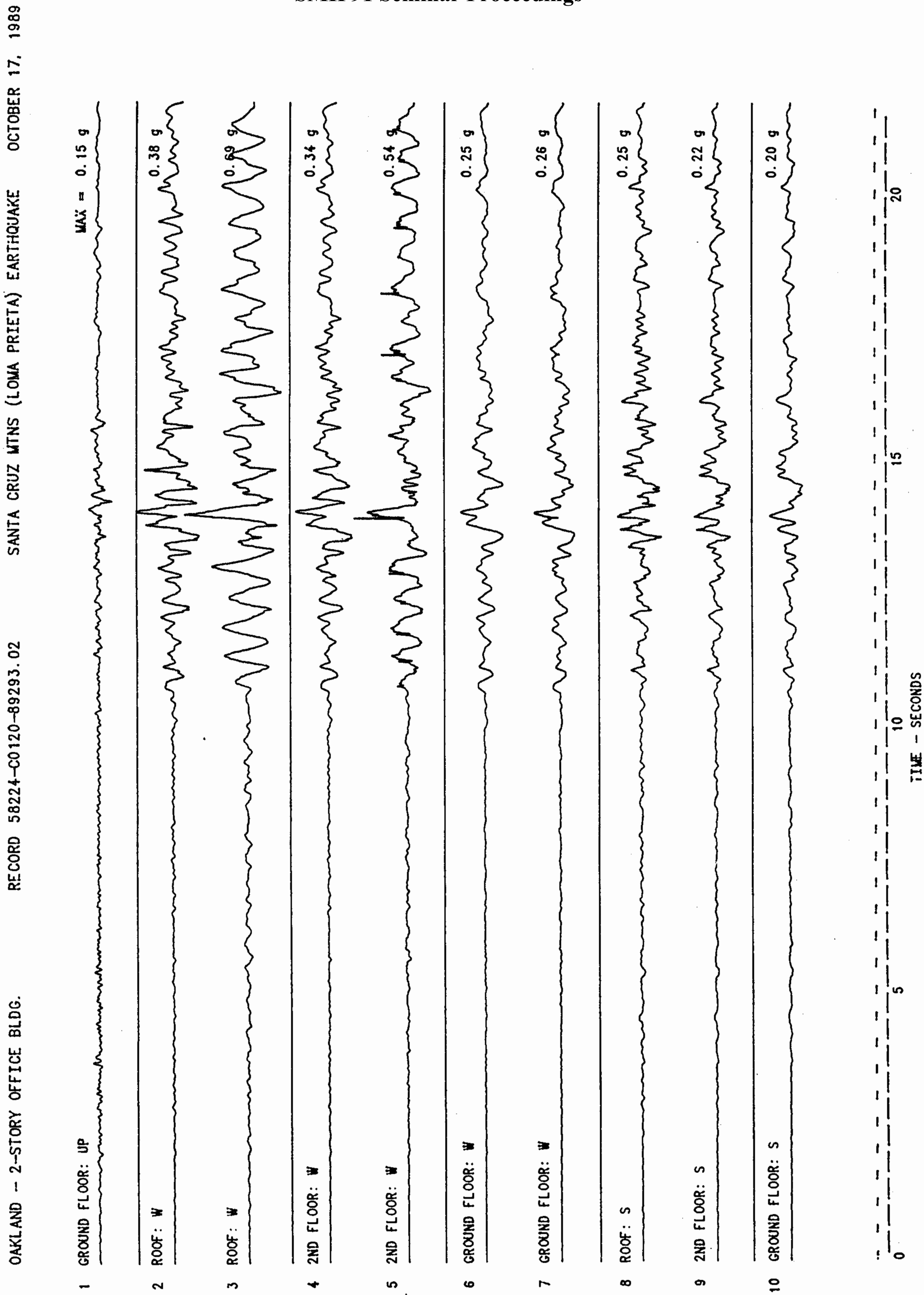
OAKLAND -- 2-STORY OFFICE BLDG.
RECORD 58224-C0120-89293.02
SANTA CRUZ MTNS (LOMA PRIETA) EARTHQUAKE
OCTOBER 17, 1989
1 GROUND FLOOR: UP
MAX = 0.15 g
2 ROOF: W
0.38 g
3 ROOF: W
0.69 g
4 2ND FLOOR: W
0.34 g
5 2ND FLOOR: W
0.54 g
6 GROUND FLOOR: W
0.25 g
7 GROUND FLOOR: W
0.26 g
8 ROOF: S
0.25 g
9 2ND FLOOR: S
0.22 g
10 GROUND FLOOR: S
0.20 g
0
5
10
15
20
TIME - SECONDS

Figure 2

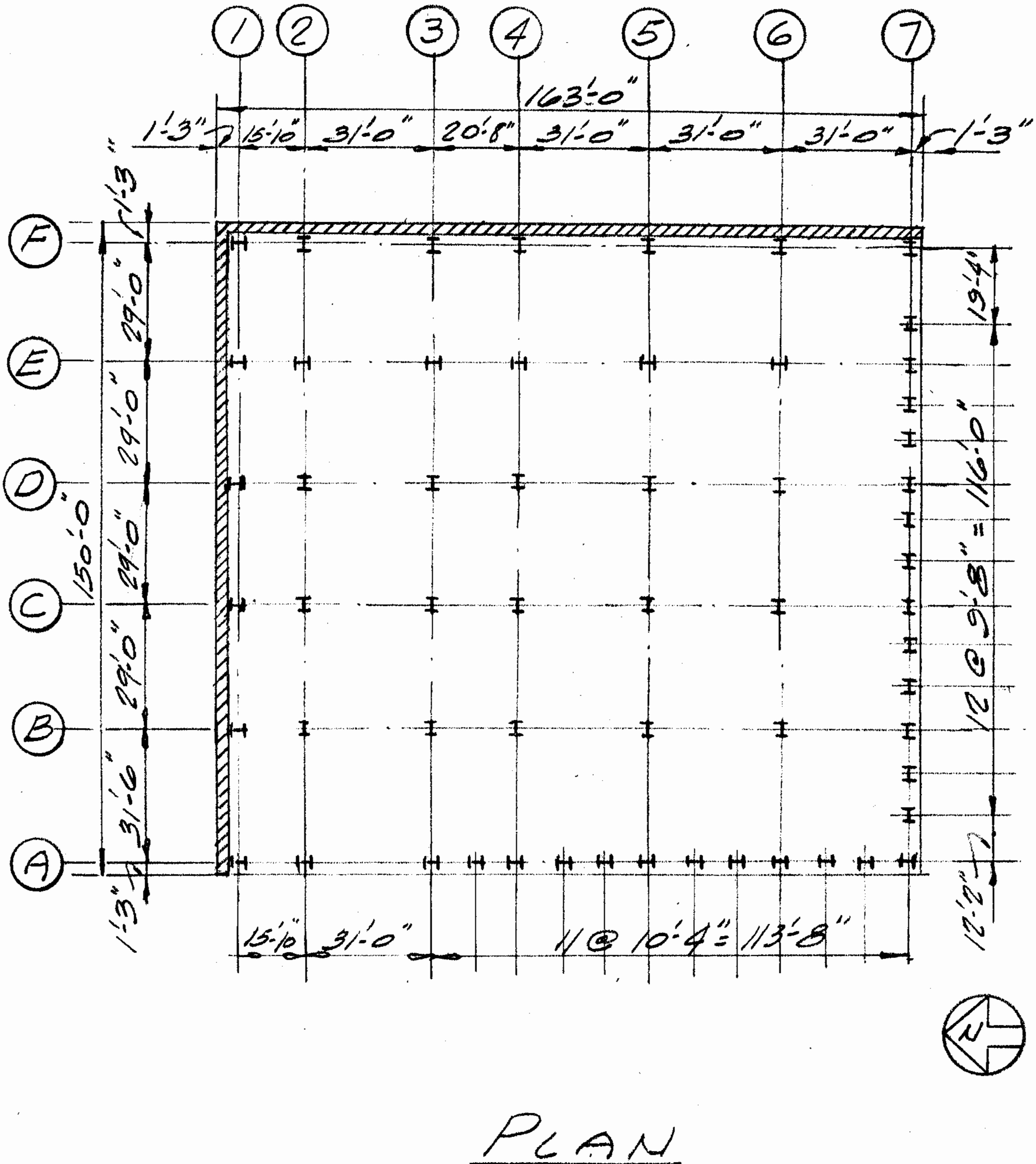
1
2
3
4
5
6
7
163'-0"
1'-3"
15'-10"
31'-0"
20'-8"
31'-0"
31'-0"
31'-0"
1'-3"
F
E
D
C
B
A
1'-3"
29'-0"
29'-0"
29'-0"
29'-0"
31'-6"
1'-3"
150'-0"
19'-4"
12 @ 9'-8" = 116'-0"
12'-2"
15'-10"
31'-0"
11 @ 10'-4" = 113'-8"
N
PLAN

Figure 3