ASEISMIC DESIGN OF TWO LONG-SPAN MULTI-STORY BUILDINGS

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INTRODUCTION

The problem of designing any multi-story building in a seismic region is a serious one, and should be approached with caution. Codes exist which guard against negligence on the part of the engineer and builder and serve as a minimum standard and guide. It must be kept in mind, however, that codes are based to a large extent on experience with existing structures. When considering the design of a structure which departs from conventional practice, engineering judgment would tend to require a more thorough investigation of the effects of earthquakes than may normally be necessary or justified. The most obvious manner in which to do this is to investigate the seismic history of the area in which the proposed structure is to be built and establish the effect of probable earthquakes on the proposed structural system.

A rational basis for this procedure is as follows: Thanks to the U.S. Coast & Geodetic Survey there are available numerous seismographs of strong-motion earthquakes recorded along the Pacific Coast. Many of these records are for earthquakes of magnitude 7 and over, which locally did damage to buildings corresponding to intensities up to VIII and IX. Although none of the records represent the maximum earthquake which some seismologists believe to be possible (in view of the strength of the earth's crustal mantle generally believed to be approximately M-9), some are rather close in magnitude and intensity to the 1906 San Francisco Earthquake (8.3).

It was felt by the authors that in view of the moderate magnitude of most Pacific Coast earthquakes and the relative infrequency of earthquakes 8.0 and over that a structure would perform satisfactorily if designed to withstand with little or moderate damage all moderate shocks and at least one major shock (8.0 or over) in its lifetime. The structure should also be designed so that in an earthquake of catastrophic proportions it would not imperil life, even though structural damage would be unavoidable.

DESIGN NO. I: THE NORTON BUILDING

The Norton Building, designed by Bindon & Wright, Architects, and Skidmore, Owings & Merrill, Consulting Architects and Engineers, is a 21-story office structure (Fig. 1), located in Seattle, Washington, in a region of moderate seismic activity. Between 1930 and 1951 there have been in Seattle approximately 25 earthquakes of intensity greater than MM III, of which two were MM VI, two were MM VII, and one was MM VIII (M-7.1). The building is notable for several reasons (Fig. 2): while the lower four stories of the building are conventional concrete flat slab construction the upper seventeen stories are composed of three 70'-0" bays spanning between steel frames. Except at the periphery of the floor the 70'-0" span is carried by 37" deep precast, prestressed simply supported concrete beams 10'-0" on center, each weighing 28,000# and containing holes in the web to permit the passage of air ducts, plumbing, etc. By threading mechanical equipment through the deep beams rather than under them, no increase in story height was needed despite the long spans. One half of the plan area of the center bay

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is occupied by a utility core. In the walls of the utility core are located concrete encased steel braced frames which extend to the foundation and which carry the major part of all lateral loads. A small part of the lateral loads is carried by the moment resisting steel frames on which the concrete beams rest. The framing as described results in a rentable area completely free of interior columns. All interior walls outside the utility core are prefabricated light metal partitions, designed for ease of rearrangement, (a highly desirable feature in office buildings) but completely lacking the structural strength and rigidity of traditional masonry block partitions. The building is sheathed by a glass curtain wall held by aluminum mullions, an earthquake hazard unless properly detailed.

Because of the large spans and movable partitions the following structural problems exist:

(1) There are fewer load resisting and energy dissipating elements than in conventional construction.

(2) The damping of such a structure will be lower than for a traditional structure, resulting in higher stresses and larger deflections in an earthquake.

(3) The lateral stiffness of the building might be insufficient for wind and seismic loads.

Not to be overlooked however, is the structural simplicity and symmetry of the design, (a consequence of the architectural philosophy of the designers of the building) which makes it possible to apply a rather exact structural analysis, too often a practical impossibility with set back and unsymmetrical buildings. This simplicity has important and rather obvious economic and aesthetic benefits as well.

PRELIMINARY DESIGN

A preliminary design was first prepared on the basis of a steel braced cantilever tower designed for a total base shear of $3\frac{1}{2}\%$ at code stresses with 33% stress increase. In addition, moment resistant frames were placed to support the prestressed beams at the short end-walls and to help in resisting torsional oscillations. (See Fig. 2). Then the spring constants were calculated for the frames by usual methods, and the periods of vibration for the various modes were obtained using Rayleigh's method.

At this stage, it was necessary to decide what methods to use to analyze the proposed structure for the effects of a strong-motion earthquake. As a preliminary step, it was decided to calculate the approximate response of the building by the use of the well known "spectrum response theory". Curves have been published by Professors Hudson and Housner of the California Institute of Technology for many recorded earthquakes in the U.S.A. which give the maximum velocities developed by a single mass, damped vibrator as a function of the period of vibration and the damping. Such a curve, called the velocity spectrum, for the El Centro Earthquake of May 7, 1940 is shown in Fig. 3. The curves can be used to predict the response of a multi-story building if one makes the assumption that it vibrates in the first mode. Actually, of course, other modes as well are excited, but if it is assumed that all of the energy of vibration is in the first mode, one obtains a reasonable estimate of the base shear, overturning moment, and kinetic energy of vibration. Referring to Fig 3 and noting that the calculated period of the building is 2.7 sec. it is found that the maximum response velocity is about 2.2 ft/sec@ 3% damping. The corresponding base shear is then equal to $\Sigma H = 5500^{K}$. It is seen that this value is several times as large as that specified by the code ($\Sigma H = 1600^k$). How this affected the design of the structure will be explained later. First it was deemed necessary to check these appropriate velocity spectrum calculations by a more exact dynamic analysis.

There are several well known mathematical methods suitable for making such an analysis, all of them depending on the integration of the differential equation of motion of a multi-mass elastic system with variable boundary conditions by one means or another, i.e.: Numerical (usually performed by a digital computer) graphical (such as the Delta-Phase Plane Method) mechanical (dynamic models) or analogous systems (analog computer).

Theoretically, all of these methods can yield accurate results. From a practical point of view, the choice of methods depends on the design being considered, the results desired, as well as on the time and cost involved. For this building Professor Housner of the California Institute of Technology was retained as consultant to perform the analysis on an analogue computer, which can be instrumented to represent a multi-story elastic system with any mass-stiffness and damping factors desired. The results of this analysis showed that the simple velocity spectrum calculations were somewhat too low, and that the expected shears and accelerations would be those corresponding to 15%g base shear (31% g at roof, 0 at ground level), $\Sigma H = 7150$. Similar forces were found to act on the building from the 1949 Olympia Washington ground motion ($\Sigma H = 6250^k$). These accelerations, shears and displacements are all much greater (by a factor of 4) than the code design values, and to design the structure to resist them without exceeding the elastic limit is very difficult and costly. This can be shown as follows: Assuming that all lateral load in the N-S direction is resisted by two K braced frames, 35' wide, 230' high, placed within the core walls: then for the El Centro earthquake, if the N-S base shear is 7150^k, the load on each truss is 3575^k and the force in each diagonal 2500^k. The code load for which the diagonal should be designed is 630^k and the area required is 22.5 in^2. The El Centro stress in the member is 110 ksi, which is much greater than the yield strength of the steel and the member has been stretched plastically an unknown amount, or has buckled. If a member is wanted which will just start yielding, its area is $\frac{2500}{33} = 76\ \text{in}^2$; this would be provided by a 14WF237 column section.

This last solution leaves much to be desired, however, in the way of economy as well as structurally because:

1. It is obviously uneconomical - requiring steel areas considerably in excess of those required by the code.

2. A member of this weight and compact section is very difficult to connect with rivets or bolts in such a way as to develop its yield strength.

3. Welding of the thick steel plates is difficult to do in the field, and is accompanied by uncertainties regarding the ultimate ductile behavior of an overstressed joint essential to the structural safety of the whole building.

A better approach would be to calculate the amount of plastic yielding and corresponding permanent set which would enable the structure to dissipate sufficient energy to remain stable. Many structures have stood up for this very reason - watertanks, refinery vessels, and even buildings. In many of these structures post-earthquake examination revealed that some plastic deformation had taken place, sometimes accompanied by failure of local character. In water tanks and refinery towers tie rods and anchor bolts have been stretched.

Several multi-story steel buildings in Japan and Mexico City have been observed to remain standing after quakes with horizontal dislocations of over an inch at some floor levels, indicating plastic bending and permanent set in the steel frame.

Today (1960) two difficulties stand in the way of a building designer trying to apply plastic energy absorption principles to a multi-story design - one is theoretical, the other one practical.

The theoretical problem is of great importance, and may be stated thusly; given a particular ground motion, and a proposed frame structure, what will be the location, sequence (both in time and space) and magnitude of the plastic deformations? The answer to this problem can be obtained, at least theoretically, using well known laws of mechanics. The computational work involved, however, makes the solution feasible only on a computer. Several research programs for the analysis of elasto-plastic frames are now finally being developed, but at the time of the design they were unavailable. This research activity could be of great benefit to the art of aseismic building design. It is to be hoped that methods of analysis will evolve from this work which will be as useful in predicting the response of elasto-plastic systems as the velocity spectrum is in predicting the response of elastic systems.

However, even assuming elasto-plastic behavior can be calculated, the practical problem cannot be avoided: What may be considered a tolerable deformation of the structure? This obviously will depend on the use and location of the building and has no sharply defined limit; implicitly or explicitly, in his design, the engineer solves this problem by judgment. Since the root of this problem lies in economics and psychology, it is very doubtful simple answers can be found.

The following example from the Norton Building can illustrate the difficulties involved in allowing the steel frame to yield; The preliminary design assumed a heavy moment resistant steel frame of built up members could be designed to take all seismic forces. It would coincide in plan with the walls around the core. Since the vertical load on a typical corner column in the core is 11,500k (dead load and code seismic), a heavy built up shape weighing over 2000 lb/ft is required. If this section were subjected to large bending moments, it could hardly be expected to develop its theoretical plastic moment capacity (already reduced by the vertical load) because there simply is not enough room for the required high strength bolts or rivets which carry the shear between the plates. However, assuming that the columns would not fail in shear, and that they all could develop their theoretical plastic moments M'_p simultaneously irrespective of some differences in relative rigidities, it is possible to calculate the effect of plastic hinges on the dissipation of energy in the structure. $\Sigma M'_p$ for the twelve moment-connected columns is $= 620{,}000^{K-IN}$. The base shear ΣH_{ULT} corresponding to this is $\frac{2 \cdot \Sigma M'_p}{h} = \frac{2 \cdot 620{,}000}{120''} = 5200^K$ (It is to be noted that this is considerably less than the maximum El Centro shear predicted for an infinitely elastic system with 3% damping.) The energy dissipated by the plastic hinges when a permanent dislocation or set of 1" occurs in the frame is $\Sigma H_{ULT} \cdot \Delta_p = 5200 \cdot 1^{K-IN} = 435^{K-FT}$. If it is assumed that such hinges occur twice in each cycle of vibration, then the average energy absorbed per second is equal to $e = \frac{435 \cdot 2}{2.7} = 324^{K-FT}/SEC$

This should be compared with the energy dissipated by 3% of critical damping, which for this structure and the calculated El Centro amplitudes of vibration* is found to be 1300^{k-ft}/cycle, or an average of 485^{k-ft}/sec. It is seen that the energy absorbed by hinges resulting in 1" sets is rather small, and therefore, the plastic sets would have to be quite large to give some effective damping.

* See sample calculation, page 12.

(The upper limit on the maximum possible single plastic set can be calculated approximately as follows: The maximum kinetic energy of the infinitely elastic structure at any time is $E_{max} = \frac{1}{2} M S_v^2 = \frac{2700 \cdot 17 \cdot 2.2^2}{2 \cdot 32.2} = 3450^{K\text{-}FT}$. If all of this energy were suddenly dissipated by plastic hinges between the 2nd and 3rd floor, the resulting set would be $\Delta_p = E_{max} / \Sigma H_{ULT} = 3450/5200 = 0.67^{FT} = 8\ in$. (Even much less than half of this set would probably render the structure dangerously useless.) In view of the questionable assumptions regarding the plastic capacities of the columns, andthe lack of a thorough elasto-plastic analysis to establish the magnitudes and locations of the probable permanent sets, it was decided to relieve the steel frame of all substantial moments by encasing it in an expendable concrete shearwall. Other problems also appeared to be solved by the introduction of concrete.

1. The concrete would crack and absorb energy before the steel frame would deform sufficiently to yield, and therefore damping would be increased.

2. The lateral stiffness would be improved, resulting in smaller deflections from windstorms and earthquakes.

3. The use of concrete would provide lateral stability to the columns, some of which carry very heavy axial loads during the El Centro earthquake, as the following table shows:

Loads on corner core column.

$D + L = 7460^k$	$f_a = 13.6$ ksi
$D + L +$ Code Seismic $= 11{,}560^k$	$f_a = 21.5$ ksi
D only $+$ El Centro Seismic $= 23{,}000^k$	$f_a = 44.0$ ksi

A steel frame was still retained, both to serve as a construction aid and more importantly to provide ductility and better ultimate strength to the structure. It was designed for a nominal value of 2.3% g base shear at code stresses. Fig. 5 shows the connections required for these forces alone.

One of the effects of the concrete is to increase the rigidity of the structure considerably. It then was necessary to consider what effect this would have on the response of the structure. The period of vibration in the first mode was found to be 1.6 sec. Referring again to the spectrum curve for El Centro it is seen that the base shear corresponding to this is 5350^k, i.e., there is no radical change in the base shear due to the shortening of the period. The shear walls were designed to carry these shears, while the steel columns would take the overturning moment.

The energy approach also throws some light on the nature of the overturning problem. It is only necessary to calculate the maximum kinetic energy available for overturning the building; this is obtained from the maximum velocity:

$$E_{max} = \frac{1}{2} M S_v^2 = \frac{1}{2} \cdot \frac{3300 \cdot 17 \cdot 1.0^2}{32.2} = 870^{K\text{-}FT}$$

This amount of energy is equivalent to raising the center of gravity of the building $870/3300 \cdot 17 \cdot 12 = 0.16^{IN}$. Obviously, unless the overturning can occur by failure of the compression columns or by a frame or shear failure, the overturning stability of most buildings could never be seriously endangered, even if full anchorage of the tension columns is not provided. Clearly, overturning is much more of a problem in light, slender structures such as stacks, chimneys and refinery towers, than in buildings.

Together the concrete and steel would be able to carry El Centro forces, the steel at stresses near yield point and the concrete at average stresses of about 500 psi in shear or past cracking. The concrete is well tied into the steel frame

frame, however, with shear connections and welded reinforcing bars, as Fig. 8 shows. It is expected that the structure will survive such a ground motion without substantial damage except some cracking of the concrete.

The shears and overturning moments from the tower are resisted by the very rigid and strong box shaped four story garage structure, which distributes these loads to the foundations, consisting of large spread footings. The core rests on a reinforced concrete mat approximately 80' x 80' and 7' thick (Fig. 9). The subgrade is fairly dense sand. The effect of subgrade compliance was found to be negligible in reducing the vibratory response of the building, the main effect being a lengthening of the period of vibration by about 5%.

DESIGN NO. II: PROPOSED HEALTH SCIENCES INSTRUCTION AND RESEARCH BUILDING FOR UNIVERSITY OF CALIFORNIA MEDICAL CENTER
Architects and Engineers: Reid, Rockwell, Banwell & Tarics

The proposed building is a 16* story steel-frame structure resting on 16 columns located at the periphery of the main unobstructed floor space which measured 90' x 90' (Fig. 10). This span is bridged by a gridwork of continuous 42" deep welded steel girders which are fixed at their supports by the columns, resulting in an effective space frame. The unobstructed floor spaces and the special mechanical systems lend themselves easily to frequent rearrangement of the laboratories, a constant problem in the rapidly changing field of medical research. The very extensive mechanical system (which can take care of bacteria, poisonous fumes and radioactive waste) requires more space than is usual in an office building, and to accomodate it efficiently, the deep floor girders have large holes in their webs at regular intervals. Because of relative rigidities, the floor gridwork participates only slightly in the resistance to lateral forces which is mainly provided by the spandrel frames on the periphery of the building. The architectural construction of the building is such that it does not add any elements of stiffness to the frame. (Fireproofing is of plaster or asbestos compounds, partitions are of the prefabricated light metal, self-braced type, and all stairs elevators, etc. are in a rigid tower separate from the main building). The frame rests partially on bedrock and partially on a very rigid concrete frame which goes down to bedrock. Because of the greater than usual forces on the main structural elements, all columns, girders and spandrels are fabricated from steel plates and some rolled shapes.

From economy in fabrication all columns and beams maintain their width and depth constant from top to bottom, although plate thicknesses change. Because of the summetry of the building, there are only two column and two girder types. The resultant savings in fabrication costs are expected to help offset the higher tonnage of steel required to span 93'. Members not participating in seismic resistance are connected with high-strength bearing bolts to reduce joint slippage, but seismic members are connected with ordinary high-strength bolts to permit slippage under seismic action. (This allows substantial energy dissipation to take place before any considerable yielding occurs). The aseismic design was based on a preliminary study of a model designed especially for the project.

PRELIMINARY SEISMIC DESIGN

The building is essentially a uniform mass, variable stiffness shear beam. According to the San Francisco Code, it should be designed for a total base shear of 3.5 g which in this case is about 800^k. The usual 33% increase in allowable stresses is permitted for aseismic design.

* later changed to 15 stories for architectural reasons.

It may be noted here that in computing the stiffness factors of the frame, it was found necessary to include the effect of shear deformation of the members. Because of the rather short and deep steel members, it was found that shear deformation reduced the effective stiffness of a joint by as much as 30% in some cases. To facilitate the analysis and design of the optimum frame, Mr. Ted York, a consulting engineer of San Francisco, was commissioned to prepare a computer program which would analyze a given frame for a given set of forces. The program assumes elastic action of the frame, but otherwise is completely general. (A 50 story frame 10 bays wide can be handled). It includes in the analysis the effect of axial column deformations on the stress distribution, and prints out all moments and forces acting on every member. In addition, it computes the deflected shape of the frame as well as the period of vibration associated with the deflected shape. The relegation of this numerical work to the computer permits the engineer to compare more alternative frame designs than would be feasible if some approximate but nevertheless slower method of analysis were used.

The period of vibration in the 1st mode was found to be 2.2 sec. According to the velocity spectrum theory, the approximate maximum base shear for the El Centro earthquake should be: 2740^{k}, or approximately 13.0% g; assuming 3% damping, the maximum energy attained by the structure is $1290^{k\text{-}ft}$ (this is equivalent to lifting the building .75" off the ground).

Since there are two frames resisting these loads, this means 1370^{k}/FRAME An analysis of the frames revealed that they would be able to resist these loads without plastic sets, but that certain portions of the crossection of some members would be very close to yielding. By proper design of the various elements it is possible to control the location of the initial yielding and to confine it. It is also important to insure that the elements which will yield first are able to absorb substantial amounts of energy without fracturing and that the frame will not be susceptible to elastic or plastic instability; therefore, a stability analysis was made for all important members, for both elastic and plastic conditions. Column to spandrel connections were designed to transmit the forces of El Centro at approximate yield stresses. Fig. 13 shows a typical frame detail near the base of the building.

Adjunct to the problem of stress in the frame is the question of deflection. The question of how to determine the right proportions of members of the frame involves several interesting problems of importance to the ultimate safety of the building. The most important member is of course the column. This is so because the column, in addition to carrying the lateral moment also carries large vertical load. These loads would contribute, in turn, to the moment if any large deformation occured due to the moments. The spandrels, by contrast, are carrying substantially no loads except the lateral, and they could accept moderate yielding. In many tall frame buildings, the factor which governs the sizes of the members in the lateral frame is not the lateral stress so much as the permissible drift (or horizontal deflection of the roof) due to lateral load from wind or earthquake. The practice of limiting the calculated deflection of the steel frame to two one thousands of the height under the code wind load is based on extensive experience with tall structures in the U.S.A. which indicates that buildings so designed do not sway objectionably in strong windstorms. A factor to be considered, however, is that masonry elements in traditional steel frame buildings added mass and stiffness to the building both of which would tend to reduce the amplitude of gust-induced swaying; many engineers estimate that the increase in stiffness of a typical traditional building due to the masonry may be as large as by a factor of 2 or 3. Modern frame buildings, lacking masonry, rely much more on the steel frame, which must supply the required rigidity. Fortunately, it is generally found that the columns contribute little to the overall deflection, so that for a small increase in girder size a substantial stiffening of the building is possible. In this building, enough fasteners are generally provided in the spandrel moment connection to develop the ultimate strength of the girder. This often results in a greater number of bolts than would be required for the code seismic moment especially in members having low D & L stresses. Since the increase in the safety factor is large compared with the cost of the additional bolts, the investment is felt to be worthwhile.

MODEL ANALYSIS AND DESIGN

For a more thorough dynamic investigation it was decided to build a simple model. The purpose of the model was to give a better intuitive and visual understanding of the behavior of a building in an earthquake than had been obtained by purely mathematical methods. The structural engineer depends very heavily on his visual sense of proportions to detect errors in design or to solve problems which often are not susceptible to analytical methods. It was felt that the very minor investment in a simple mechanical model would repay itself in the ability to visualize a physically simple process, the mathematics of which happens to be cumbersome. If, in addition to that, it were found that quantitative answers could be obtained with reasonable accuracy, the model would be completely justified. When designing the model, it was decided to make the horizontal deflections of the model equal to that of the prototype for identical ground motions. This was done because it is easy to relate deflections to stresses, (at least in the elastic range) and the deflections are easily measured on the model.

Since analysis of the proposed steel frame showed that 80% of the deflection is caused by bending and shear deformation of the frame, and only 20% by axial stresses causing rotation of the floors, it was clear that a very close dynamic approximation could be obtained by a "shear-type" model. The model consists of 16 plywood panels 18"x18" weighing 3.66 lb each mounted on a shaking table actuated by a groove cut to the proper scale from actual horizontal ground displacement-time curves as obtained from U.S. Coast and Geodetic Survey seismographs. The recording of the deflections is accomplished by mounting on a given floor very light smoked paper drums rotated by small electric motors at a constant speed, and having the relative motion between floors inscribed by a levered scratch arm. To duplicate the frame stiffness characteristics of the prototype, the distance between model floor masses is varied as shown in Fig. 17. In computing the stiffness factors (K_m) of model, the effect of vertical load on stiffness was taken into account. This factor, which proved to be quite troublesome had much to do with the final proportions of the model. Since it is physically very difficult to build a stable and reasonably small model with a period as long as the prototype's (2.3 sec), the model time scale was compressed 2.3 times. To retain dynamic similitude, the rate of introduction of the ground motion function had to be correspondingly increased. The dimensional relations between the model and prototype which allow the calculation of any variable in the prototype, are obtained by usual methods of dimensional analysis.

The damping of the model was studied and was found to increase with amplitude, as it undoubtedly would in the prototype. It was found that the model damping represents a prototype damping of 3% g @ deflections 1/8"/floor, 5% @ deflections $\frac{1}{4}$"/floor, 8% @ deflections $\frac{1}{2}$"/floor, 10% @ deflections 1"/floor.

The first question answered by the model had to do with the behavior of the lower floors during an earthquake. Although it was generally believed that the maximum relative displacements between stories do not occur at the beginning of an earthquake but rather build up as energy is accumulated in the building, it was nevertheless questioned whether some of the first large ground displacements would not be able to yield the lower floors. An examination of Fig. 14 showing the relative displacement between the ground floor and the 2nd floor shows that in the El Centro 1940 earthquake, at least, this does not occur. The maximum displacements and base shears are reached about 8 seconds after the start of ground motion. At that time a condition seems to be reached by the structure which for lack of a better term might be called a semi-steady state of forced vibration, since the maximum base shear periodically repeats itself, each time reaching approximately the same maximum value. This is of interest, since if the base shear is greater than the elastic limit shear of the frame, it seems possible for the frame to undergo a considerable number of reversals of plastic yielding. Fig. 14 also shows that at the beginning of the ground motion the base shear is quite irregular, but as energy begins to accumulate in the building the base shear starts conforming more or less with the first mode of vibration. A record of the 12th floor displacement shows that the shear at that level is affected relatively more by higher modes, as is expected. Especially predominant is the second mode. As at the ground floor, the maximum shear is repreated several times with little variation in magnitude. The measured deflection at the 2nd floor corresponds to a base shear of 5.3 lb in the model. The corresponding base shear in the prototype is 2800^{k}. This checks well with the base shear calculated using the velocity-spectrum theory: for T = 2.3 sec. $\Sigma H = 2740$. The maximum sidesway of the model during this earthquake was obtained by simultaneous recording at the top and bottom stories, and is seen from Fig. 14 to be about 2" which corresponds to 16" on the prototype (32" double amplitude) or about 1"/floor.

Fig. 15 shows the vibration records obtained on the model for the San Francisco earthquake of 1957. The smaller amplitude and higher frequency vibrations of the smaller earthquake are seen to excite the higher modes more than the El Centro earthquake did. The 2nd floor displacement still has the 1st mode evident, but strongly modified by 2nd, 3rd and higher modes. In the 1st floor displacement curve, it is very hard to see the 1st mode at all. The computed maximum shear at the base is 340^{k} or about 1.6% g.

CONCLUSIONS

While both the Norton and Medical Science Buildings have much in common, (they are both modern, glass curtain wall enclosed steel frame buildings with interior space uncluttered by masonry partitions), it can be seen that they achieve their required lateral seismic strength by different methods which are consistent with their respective framing patterns.

The Norton Building is framed essentially by simple beams resting on long span flexible steel frames. This means that the elements which carry vertical loads cannot be used to resist seismic load, and therefore a separate structural system for these loads is necessary. This separate system was severely restricted in size by the architectural requirement that the rentable area be clear of all columns. It was necessary, therefore, to use a lateral bracing system which in many ways is comparable to the lateral bracing system of more conventional type masonry filled steel framed structures. That is, a steel frame to carry the large stresses in major earthquakes and to tie together a masonry filler wall; and a filler wall which would provide the stiffness for small earthquakes and wind loads and provide damping and energy absorbtion for large earthquakes. It is true, the space requirement necessitated condensing this system, as it were, to only the core area, but in essence it remains comparable to the conventional masonry fill steel frame building.

The Medical Science Building by contrast has no core area within the building to be used for lateral strength. This requires a rather substantial departure from conventional design, that is, a frame which will resist adequately all lateral forces with minimum deflection and adequate ultimate strength. This requirement was not difficult to achieve, however, thanks to the way the vertical loads are carried; that is, with large span steel girders which are fixed to the columns, resulting in large column moments and therefore large column sections, and relatively short distances between the columns, due to the fact that all columns are near the outside of the building. To a scheme designed primarily to carry vertical loads, it was then only necessary to add a relatively small amount of steel in the form of spandrel beams and connections to achieve an extremely rigid and strong 3 bay frame, which could carry all conceivable lateral forces. The inherent strength of this frame can be shown by comparing the ultimate base shear capacity of the steel columns to the weight of the building, for both building (see page 12.) The Medical Science Building has approximately twice the base shear capacity of weight ration as the Norton Building. This difference is balanced of course by the strength and damping capacity of the concrete shear wall .

The concept and therefore the solution to the seismic strength of both buildings is different but, from a very basic point of view, the problem is the same. This problem, which is sure to confront the engineer more often in the future is; what criteria can be used for the seismic design of structures which do not conform to conventional building practices. In these cases codes and experience alone fail and extrapolation of past experience to unusual conditions must be accompanied by considerable intuition and judgment. Surely the criteria of tomorrow will in some degree be written by the failures of today. The seismologist and mathematician can help the engineer by developing more workable theories and in recent years much of value has been done by them. It must be remembered however that from the engineer's point of view any usable method of seismic analysis must be simple to apply. Many engineering decisions which involve considerable cost to the client and the safety of the public must be made in a relatively short time and the responsibility for these decisions rest heavily and squarely upon the engineer, as they should. It is with gratitude therefore that the engineer accepts the tools which have been given him.

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NOMENCLATURE

e = average rate of energy dissipation

$\bar{E}$ = maximum energy in the building

F = force per unit height of building which causes deflection

h = height of building

ΣH = total base shear

KE = kinetic energy

m = mass per unit height of building

S_v = velocity response, maximum velocity attained by single mass system acted on by given force function as a function of damping and period.

T = period of vibration

x = distance measured vertically on building

y = horizontal displacement of building due to imposed force, or shape of first mode.

Y = maximum horizontal displacement of building due to given force function

$\dot{Y}, \ddot{Y}$ = 1st and 2nd derivative of Y, maximum velocity and maximum acceleration.

SAMPLE CALCULATIONS FOR THE MEDICAL SCIENCE BUILDING

Properties of the building:

Height - 16 stories	210 feet
Weight - 1300 kips/story	90 k/ft

A uniformly varying load of 0 @ the base to 100 k/str. @ the roof gives a deflection of 3.8" @ the roof.

Period: $PE = \frac{1}{2}\int_0^h F \cdot y \cdot dx = \frac{1}{2}\int_0^{16} \frac{100}{16}x \, \frac{3.8}{16} x \, dx$

$$KE = \frac{1}{2}\int_0^h m(\dot{y})^2 dx = \frac{1}{2}\int_0^{16} \frac{1300}{g}\left(p\,\frac{3.8}{16}x\right)^2 dx$$

$$PE = KE \qquad p^2 = \frac{100}{1300}\,\frac{g}{3.8} \;;\; T = 2\pi \cdot \frac{1}{p} = 2\pi \cdot \frac{1}{2.83} = 2.22 \text{ sec}$$

The velocity response for El Centro @ 3% damping

For a single mass system $S_v = 2.0 \text{ FT/SEC}$

For a multi-mass system in the 1st mode

$$\dot{Y} = \frac{\int_0^h my\,dx}{\int_0^h my^2 dx}\; y\, S_v$$

Assuming the shape of the 1st mode is a straight line

$$\dot{Y} = \frac{\int_0^{16} \frac{1300}{g} x\,dx}{\int_0^{16} \frac{1300}{g}\frac{x^2}{16} dx} \cdot \frac{x}{16} S_v = \frac{3}{2}\frac{x}{16} S_v$$

@ the roof

$$\dot{Y} = 1.5\, S_v = 3.0 \text{ FT/SEC}$$

$$Y = \dot{Y}/p = 3.0(12)/2.83 = 12.7'' \text{ @ top}$$

$$\ddot{Y} = pY = 3.0 \cdot 2.83 = 8.5 \text{ FT/SEC}^2 = 26.4\,\%\,g$$

base shear

$$\Sigma H = \frac{1}{2}\, 16 \cdot 1300 \cdot 0.264 = 2740^{k}$$

Maximum energy in the building

$$\bar{E} = \frac{1}{2} mh(S_v)^2 = \frac{1}{2}\,\frac{1300 \cdot 16}{32.2}(2.0)^2 = 1290^{\text{K-FT}}$$

Maximum average energy dissipation

$$e = \frac{4\pi}{T}(\%\text{ damping}) \cdot \bar{E} = \frac{4\pi}{2.22}(0.03)1290 = 218^{\text{KFT/S}} = 400^{\text{HP}}$$

COMPARISON OF ULTIMATE BASE SHEAR TO TOTAL WEIGHT RATIOS FOR THE TWO BUILDINGS

Norton Building, Preliminary Design Steel Frame Only: $\frac{\Sigma H_{ult}}{W} = \frac{5200}{46{,}000} = 0.113$

Medical Science Building, Steel Frame: $\frac{\Sigma H_{ult}}{W} = \frac{5550}{21{,}000} = 0.265$

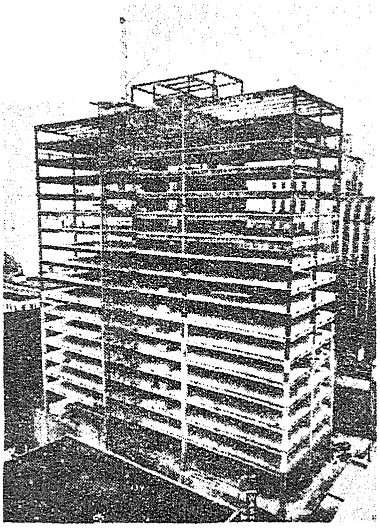


Fig. 1. Perspective of Norton Building under construction.

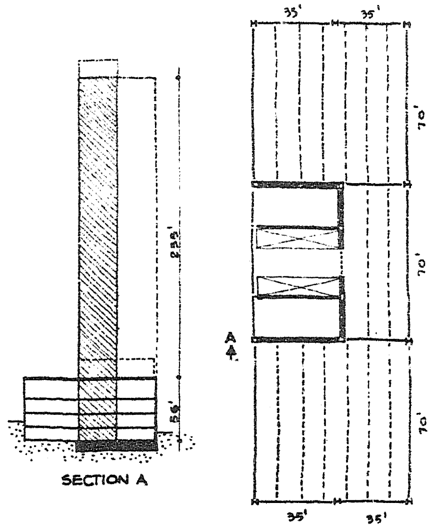


Fig. 2. Plan and Section of Norton Building

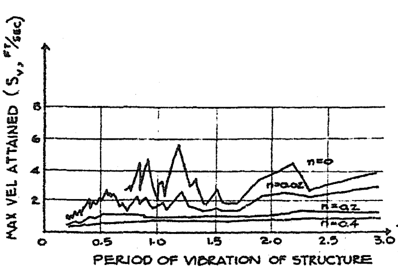


Fig. 3.

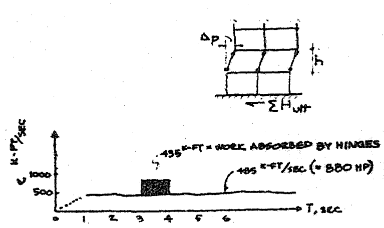


Fig. 4.

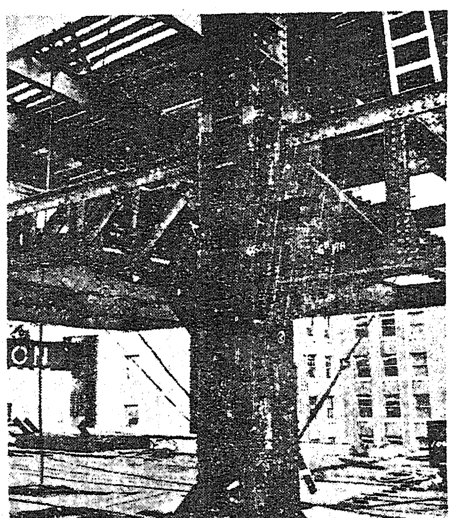


Fig. 5. Steel Frame detail on 5th floor

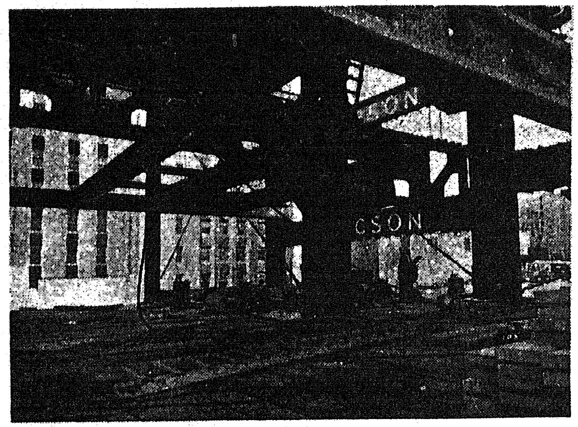


Fig. 6. View of Core framing - steel only

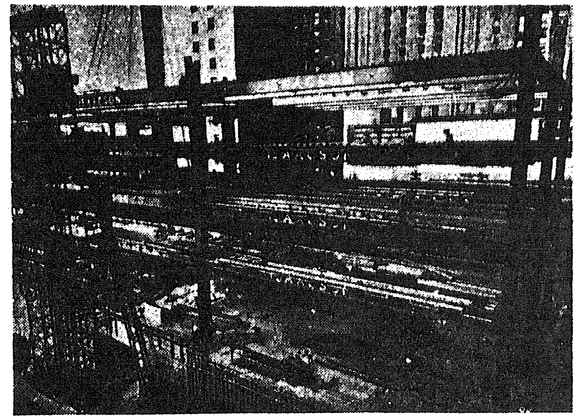


Fig. 7. View of frame going up

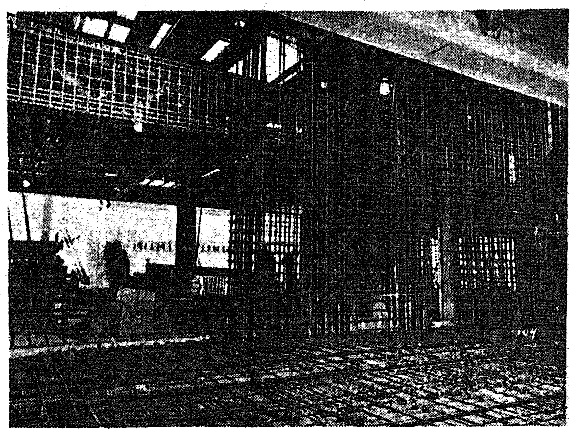


Fig. 8. View of core framing - reiforcing in place

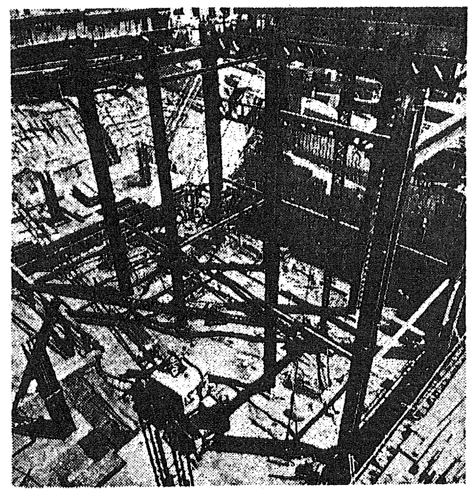


Fig. 9 View of core columns framing into foundation.

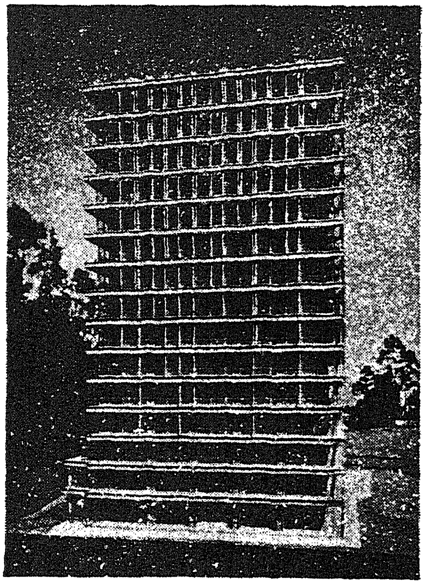


Fig. 10 Model of Health Science Research Building

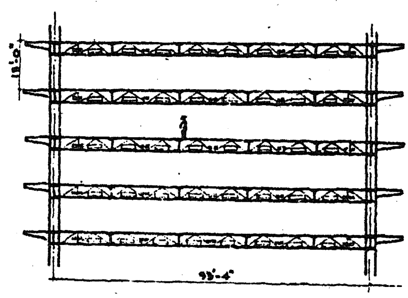


Fig. 12 Section through floor

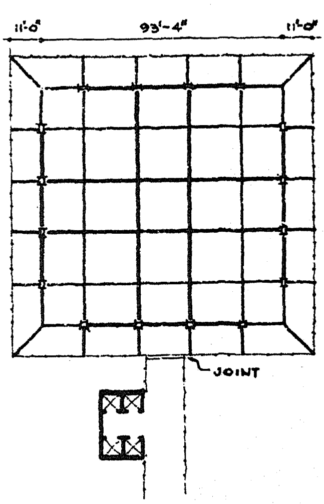


Fig. 11 Floor Plan

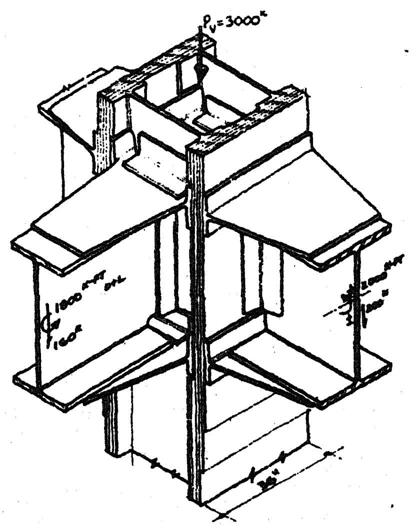


Fig. 13 Typical column and girder connection

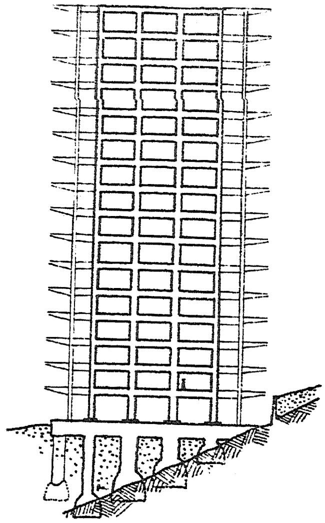
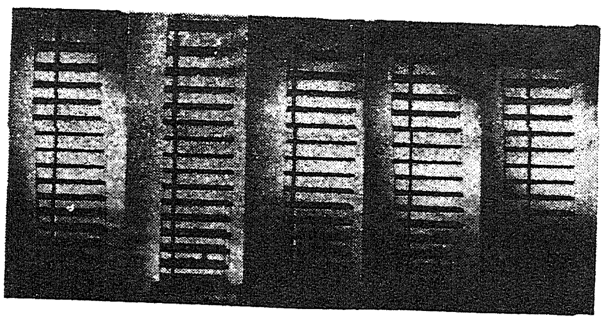


Fig. 16 Elevation of frame



T= 8 sec T=10 T=13 T=14 T=16

DEFLECTIONS ARE $\frac{1}{8}$ OF PROTOTYPE

T= 1sec T=3sec T=4sec T=7.5sec

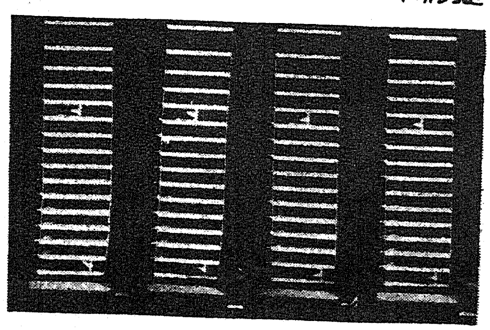


FIG 17. DEFORMATION OF MODEL DURING SHAKING TABLE TEST - EL CENTRO 1940 EQ

Fig. 17

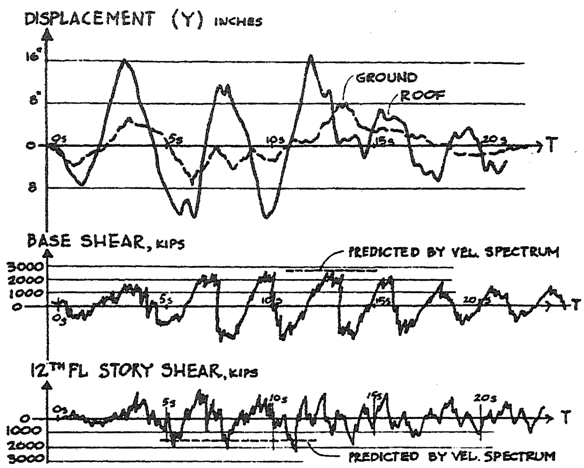


FIG 14: RESPONSE TO EL CENTRO 1940 GROUND MOTION

Fig.

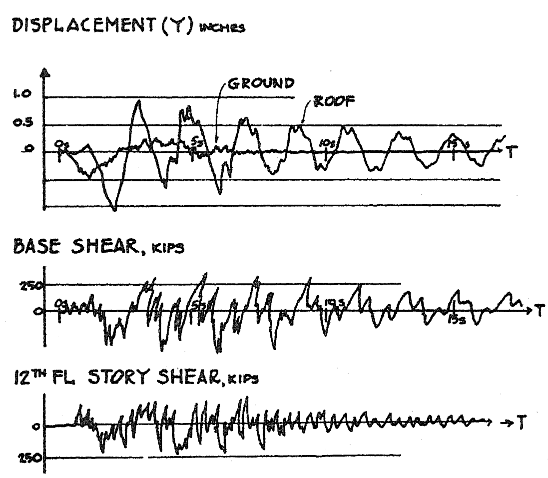


FIG 15: RESPONSE TO S.F 1957 GROUNDMOTION

Fig. 15