Recent Trends in High-Rise Building Design in Japan

By

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INTRODUCTION

1. Repeal of Building Height Control and Regulation for Antiseismic Design

Since 1921, Japanese Building Standard Law prohibited construction of buildings over 31 meters high in commercial zones, and houses and buildings over 20 meters in height in residential zones. Owing to extremely expensive cost of land in urban districts, however, movement to permit construction of taller buildings arose for more effective use of land and reurbanization purposes. Thus, the earthquake resistant design of high-rise buildings has become a serious problem for structural engineers.

The initial study in this field was made by a research committee headed by the author which was sponsored by the Japan National Railway. The National Railway proposed to construct a new Tokyo Central Station in 1958, to be 24 stories high, of a scale and form worthy of becoming the new center of transportation for Tokyo City. The structural design of such a tall building for earthquake forces would be entirely impracticable, if the severe requirements of current Building Standard Law were to be applied. The committee carried out extensive activities, and published the final report in 1962[1]. The most important conclusion of the work of this committee was the confirmation that the response of taller buildings to the earthquakes are in general less than the response of lower buildings designed according to the prevalent provisions in Japan, and accordingly, the design seismic forces could be reduced considerably for taller buildings.

Many other high-rise buildings were proposed in 1962 and thereafter, and structural design and analysis for earthquakes were further carried out. The possibility of constructing high-rise buildings in Japan was generally indicated.

The repeal of the building height control and revisions of the Building Standard Law were put into effect in 1963. The revisions permit the construction of buildings exceeding 31 m in height.

The structural design of buildings for earthquakes that are less than 45 m in height may be designed by the current provisions, whereas those more than 45 m in height may be designed by different suitable methods.

For the time being, no Law or Code is in force in regard to provisions for safety of buildings over 45 m in height against earthquakes in order not to hamper the promotion of research and development in antiseismic design and construction methods. High-rise building projects are reviewed and acted upon

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individually by the High-Rise Building Structure Examination Board in the Ministry of Construction. This Board began its activities in September 1964 under the author's chairmanship. It has already approved three high-rise buildings to date and they are now under construction and others are pending.

2. Ground Conditions and Earthquake Characteristics

In many parts of Japan, the ground is extremely soft and damage to buildings on softer ground has frequently been observed to be more extensive than on harder ground. This problem is regarded to be one of the most important subjects in the development of sound principles for antiseismic design.

Consequently, seismologists and structural engineers have carried out research on the various phases relating to the ground conditions and earthquake characteristics for five decades. Some of the research projects in this field are the following: investigations on the difference in earthquake motions by concurrent measurements at various locations at sites having different ground conditions, determination of predominant period characteristic of different sites, studies on the predominant periods and vibrations due to microtremors and earthquake motions, compilation of ground condition maps for different cities based on boring data, research on the velocity of wave transmission in various strata, studies on the relation of ground conditions and structural damage, studies on the relation of natural period of vibration of buildings and structural damage, etc.

There is another serious problem relating to antiseismic design. A prevalent practice in the design and construction of modern buildings in down town Tokyo is to provide basements, requiring the foundation to be constructed deep in the ground to reach hard strata. Consequently, investigations are required on the relation of earthquake motions at the ground surface, at the bottom of the foundation, and further down at bedrock. The design for earthquakes in Japan intends to take all the results of these researches into consideration.

3. Strength and Deformability

Sixty years ago, the late Dr. R. Sano correctly pointed out that the requirement on the strength was not sufficient for the structural materials and structural systems to be used in aseismic design, and that they should be deformable or ductile such that they would maintain resistance while taking up deformation after reaching the yield point. Ever since, in the research of structural materials and systems, emphasis has been placed particularly on the behavior beyond the elastic limit, and on the behavior under repeated loadings. Complete load deformation relationship and characteristics of hysteresis under reversals have been studied for various materials and structures.

It should be noted that in recent years, in connection with the introduction of new materials and new construction methods, considerable experimental studies have been made using full-scale or half-scale models of actual structures. These tests are performed to determine the resistance capacity of structures when big buildings are designed. These experimental works cover most construction systems, including steel, reinforced concrete, prestressed concrete and others. Tests on building elements such as beams, columns, shear walls with or without openings, beam-column connections, have also been made. Furthermore, vibration tests and static loading tests of existing buildings and model buildings up to the point of failure have been carried out.

Another important contribution is the research on the deformability of curtain walls and interior walls. Although they are usually not regarded as structural elements in the analysis, they may be hazardous when they crumble into fragments that may injure people, and fire prevention walls may become ineffective. From this point of view, the adequacy of building element designs is experimentally investigated by performing actual vibration tests on them. From these investigations, it is generally considered that the allowable story drift may be about 2 cm in Japan.

4. Study on the Response to the Earthquakes

In order to know directly what earthquake forces would act on buildings during earthquakes, SMAC accelerographs were developed in Japan since 1952. 180 instruments have been set up in Japan until the recent Niigata Earthquake. The importance of acceleration observation was widely recognized after the earthquake, and it is expected that more than fifty accelerographs will be installed in 1964 fiscal year.

SMAC's are usually placed in the basement of buildings as well as on the upper floors, and record concurrently the response at both locations to the earthquake motions. These data should be reduced to be used in the structural design, by means of analysis on computers. Researches in a wide scope have been made, using analog computers (for example SERAC computer) or digital computers, which can be used now for the analysis of multi-mass non-linear systems. These computers are also used in practical design.

5. Scope of This Paper

In order to realize the construction of high-rise buildings in Japan, research and development have been carried out as explained above and are being further pursued. The author accepted the request of the Organizing Committee of this World Conference to explain the general trends in Japan. However, in view of the diverse variety in research and developments, the author realizes that it is impossible to summarize them all.

Accordingly, this paper is limited to the introduction of those developments with which the author is intimately related. The author requests the participants to acquaint themselves with the trends in Japan by perusal of papers presented to this Conference. The author considers the recent developments in Japan as the consequence of international exchange of valuable knowledge.

I. PRESENT STATUS OF HIGH-RISE BUILDING DESIGN FOR EARTHQUAKES IN JAPAN

1. General

As mentioned previously, research on the development of design methods for high-rise buildings in Japan has been carried out recently by a large numbers of research workers with various differing philosophies in mind. It is almost impossible at present to predict how the results of these efforts will crystallize.

Since the revision of the Building Standard Law, the current provisions do not apply for the design of high-rise buildings over 45 m in height. The design may be carried out by any rational method. However, there arose a demand from structural designers for some authoritative guide in this respect. Responding to this, the Architectural Institute of Japan (AIJ) prepared a "Guide for Design of High-Rise Buildings" in 1964, under the chairmanship of Dr. K. Takeyama. This guide should be regarded as expressing the general opinion of structural engineers at that particular time[2)].

Before introducing the summary of this publication, however, the author wishes to introduce the current provisions specified in the Standard Law[3)], which are applicable to the design of buildings less than 45 m in height.

Current Provisions

The current provisions for the structural design of buildings for seismic forces are as follows (refer to Fig. 1).

1. Forces due to dead and live loads shall be combined with forces due to static horizontal seismic forces.

2. Allowable stresses for this combined forces shall be greater than the values for dead and live loads (2 times for concrete, and 1.5 times for steel).

3. Design horizontal seismic forces shall be obtained by multiplying seismic coefficient k to the loads of each floor, wherein k shall be 0.2 for the part of building less than 16 m in height, and 0.01 shall be added for increase of every 4 m in height in excess of 16 m.

4. The design seismic coefficient may be reduced to 90% or 80% according to the seismicity of the region. It may be further reduced to 90%, 80% or 60% according to the soil conditions and the type of construction, but not to be less than 50%.

2. AIJ Guide for Design of High-Rise Buildings

A. Suggestions for Structural Planning

1. The shape of building shall be simple.
   Buildings with complicated plans or elevations are apt to induce complicated vibration and hence they are undesirable.

2. It is recommended that the buildings be located on hard ground.
The characteristics of earthquake motions in the form of spectrum show that soft ground condition is unfavorable for high-rise buildings. Furthermore due attention should be given to the fact that soft ground often makes the earthquake motion itself more destructive.

3. The structure should consist of simple and clear systems from the viewpoint of mechanics.
A rectangular continuous frame is desirable, because it is possible to estimate accurately the strength, deformation and ductility, which in turn results in accurate determination of vibration period, drift, and yield strength.
The use of shear walls is in general very desirable for low buildings. However the adoption of shear walls into high-rise buildings should be examined carefully, as it would result in a very complicated state of deformation which requires advanced structural or vibrational analysis.

4. The frame shall have sufficient ductility in addition to strength.
The ductility or deformability of the structure has always been considered in Japan. A structure should be able to deform beyond yielding without losing its load-carrying capacity. Even when a building is subjected to a destructive earthqake, the building may be easily repaired and re-used if the frame is ductile and not seriously damaged.

5. The deformation of building shall be controlled from viewpoints of serviceability and safety.
Excessive drift in the building may result in the rupture of glass or curtain wall or partition wall. It is not desirable not only because fragments may injure people, but also because fire may expand easily. Accordingly the interior or exterior finish of building should be designed taking the deformability into account, and the drift in case of earthquake should be limited.

B. <u>Principles of Structural Analysis for Earthquakes</u>

1. The total base shear V_B shall be calculated by

$$V_B = C_B W,$$

where C_B = design base shear coefficient,
W = load for seismic analysis, i.e. sum of all dead loads and live loads as stipulated.

The base shear coefficient may be varied in the following range, less for longer natural period of vibration of the building:

$$0.2 \geqq C_B \geqq 0.05$$

The base shear coefficient less than the above lower limit may apply, in case it is proved that the response is substantially small considering the seismic intensity and natural period of vibration.
Some explanations will be given. AIJ considers that the dynamic analysis is indispensable for the design of high-rise buildings. However, for the time being, there are several problems, as to the choice of

earthquake motions, the assumptions involved in the simplification of structures for analysis, the treatment of damping and plastic deformation, the evaluation of the difference due to the different choice of earthquake motions or assumptions for the analysis. Nevertheless AIJ regards it is necessary to give general requirements, that the base shear coefficient shall be evaluated between the above-mentioned limits from the following expression,

$$C_B = \frac{0.36}{T} \text{ to } \frac{0.18}{T}$$

where T = natural period in sec., which may be estimated as 6~10% times the number of stories.

C_B shall be varied within the above range according to subsoil conditions and permissible plastic deformation. When the expected seismic intensity is small, above values may be reduced up to 60%.

2. The total base shear shall be distributed to upper stories, the method of which is not explicitly shown. It is recommended to take relatively large values for upper stories.

3. After completing structural design, it is recommended to make tests on the members and connections, in order to evaluate the stiffness, strength and deformation at yield, and the behavior beyond yielding.

4. The design shall be further investigated and improved, making dynamic analysis to earthquake motions and checking the elasto-plastic response values.

3. Comparison with SEAOC Code

The provisions of AIJ guide will be compared with those of SEAOC code[4)]

First, the design loads are compared. AIJ guide prescribes the use of specific live load for any type of building. On the other hand SEAOC code takes dead load only except for storage and warehouse, for which 25% of live load is added. In this regard AIJ guide provides higher safety.

Second, the allowable stress of steel in SEAOC code for combined loading condition is 1.33 times the usual value, whereas AIJ guide is to use 1.50 times the usual value. Here SEAOC provides greater safety. Hence it is concluded that the safety considering both effects of load and allowable stress would be approximately the same.

Finally, the base shear coefficient will be considered. For moment-resisting frames, SEAOC code adopts the following expression:

$$K \cdot C_B = 0.67 \times \frac{0.05}{\sqrt[3]{T}}$$

As seen in Fig. 2, AIJ base shear coefficient is about three to four times as great as the SEAOC base shear coefficient.

II. PROBLEMS ABOUT EARTHQUAKE EFFECTS RELATED TO THE DYNAMIC ANALYSIS

1. Ground Motion and Earthquake Effects on the Building

In order to study the behavior of buildings during earthquakes and to develop design method, it is most desirable to have measured data of the motion, particularly of the acceleration, both at the ground or foundation and upper portion of buildings. SMAC accelerographs were developed in Japan for this purpose, and were installed in many buildings all over the country. A recent contribution from the SMAC system was the recording of Niigata Earthquake, whereby accelerograms were obtained at the basement and roof of a four-storied apartment building in Niigata, and on the basement and sixth floors of Akita Prefectural Government Building.

From the data so far obtained, it was generally found that the response acceleration of upper portion of steel or reinforced concrete buildings was about three times the acceleration of the ground. An example will be shown in Fig. 3, which is the record taken at the basement and sixth floors of a steel framed six-storied building in Tokyo[5]. In this case the ground acceleration was amplified by the factor of three. In general this is the case for most buildings with natural period of vibration of less than one second, and which behaved within elastic range. However this may not be applicable to buildings with different structural or soil conditions.

2. Spectra and Soil Conditions

One of the most effective methods to study the characteristics of earthquake forces is to investigate the effect of these variables by constructing response spectra.

Response spectrum shows the relationship of acceleration and period. It has been recognized to be an effective measure of destructive dynamic characteristics of earthquake motions, and as is well known, it has been incorporated in the San Francisco Seismic Code. The research in Japan on this subject started about ten years ago by RAC Committee of which Professor Takahashi was the chairman. This work was succeeded by SERAC Committee under chairmanship of the author. Fig. 4 shows the comparison of acceleration spectra for various places with different soil conditions. Here the damping coefficient was taken as 5%, and spectral value was normalized taking the maximum value as 100%.

The period associated with the peak response is smallest for Sendai, Japan (1962), then in ascending order El Centro, U.S.A. (1940), Tokyo, Japan (1956), Osaka, Japan (1963), Seattle, U.S.A. (1949). For these earthquakes, periods were all less than 1 sec. Then Nagoya, Japan (1963) has a period greater than 1 sec., and Mexico (1962) shows a period of 2.5 sec. In general, harder the ground, shorter is the period. The longest period is for Mexico, where the ground consists of extremely soft and thick clayey soil.

A response spectrum gives, of course, the earthquake force to single storied structures directly. However the above-mentioned difference due to soil condition cannot be disregarded. The determination of base shear coefficient from spectrum has to take this fact into account. In this regard, the author cannot agree unconditionally with the proposed or specified formulas of the world.

3. Vibration of Ground Surface and Foundation Bed

One of the most important problems to be tackled for the antiseismic design of buildings is how to estimate the earthquake motions at foundation bed. Available data of earthquakes concern mostly with ground surface or shallow bed, and no reliable data are available yet as to the motion of hard gravel layers of diluvium, 15 to 20 m deep, on which high-rise buildings in Tokyo and others are to be grounded. Seismologists in Japan have attempted to solve this problem through measurement of microtremors at ground surface and simultaneous observation of earthquake at surface and depth in wells. It was found from this research effort, that such components of vibration are remarkably amplified at the ground surface that are associated with the predominant period obtained from the measurement of microtremors at the ground surface.

As an example, following is the summary of research made jointly by the Japan Atomic Energy Research Institute and Earthquake Research Institute of the University of Tokyo, on the site of the first atomic power station in Japan[6]. The predominant period was about 0.2 to 0.3 sec., and the amplifications of vibration components of this range of period at various depths to bed-rock at 21 m depth (mud-stone) were as follows.

Level	Amplitude ratio to -21 m level
Ground Level	5.0
- 7 m Level	2.0
-13 m Level	1.5
-21 m Level	1.0

The amplitude ratio at surface layers diminished rapidly as the period parted from the predominant period.

Thus, the measured earthquake at ground surface may be inadequate for design purpose, as it involves the amplification of vibration in the surface layers of soil. The study of this problem is in progress by Professors S. Nasu, H. Kawasumi and K. Kanai.

For the time being, we have to interpret the earthquake motion at foundation bed from the observed data at surface, taking into account the nature of soil stratification.

4. Dynamic Analysis and Adopted Earthquake Records

As mentioned earlier, AIJ guide postulates for the dynamic analysis in the second phase of the design. The selection of earthquake records for this purpose may raise serious problems to the structural designer. The available data and the method of the author will be outlined briefly.

As input data to SERAC analog electronic computer, we have two groups of data.

A. Earthquakes in the United States. These are adopted from digital data for IBM computers contributed by Professor Glen V. Berg of the University of Michigan.

El Centro, Calif.	NS and EW	1940
El Centro, Calif.	NS and EW	1934
Taft, Calif.	NS and EW	1952
Olympia, Wash.	NS and EW	1949

B. Earthquakes in Japan recorded by SMAC, as given in SERAC Report No. 4

As input data for use in digital electronic computers, we have, besides the above-mentioned American earthquakes, data on the following earthquakes.

Tokyo	NS	1956
Sendai	NS	1962
Osaka	EW	1963
Akashi	EW	1963

These were digitalized from SERAC input data, and will be published in SERAC Report No. 6.

In case of design of buildings founded on hard diluvium layers in Tokyo, the author is using data for El Centro NS 1940, Taft EW 1952, Tokyo NS 1956 and Sendai NS 1962.

III. DYNAMIC ANALYSIS AND THE DEVELOPMENT OF PRINCIPLES FOR THE DESIGN OF HIGH-RISE BUILDINGS FOR EARTHQUAKES

1. First Systematic Research in Japan

In 1959, a research committee was organized under the author's chairmanship, in order to investigate the possibility of constructing a new 24 storied Tokyo Central Station Building for the Japan National Railway[2]. The work was carried out for three years by four subcommittees, i.e. tentative design by Professor H. Umemura and Dr. H. Narita, seismic intensity of the ground by Professor H. Kawasumi, dynamic testing by Professors H. Kobayashi and M. Takeuchi, and response by Professor R. Tanabashi and Dr. T. Hisada.

The tentative design subcommittee made structural design of frames by steel construction and steel and reinforced concrete composite construction, based either on the seismic coefficient specified by Standard Law or taking 60% of it. The response subcommittee reduced them to five-degree-of-freedom systems, and analyzed them on SERAC computer using El Centro NS 1940 and Tokyo EW 1956 data. Following facts were found.

A. The structures designed to the specification of Standard Law behaved safely in terms of strength, under the earthquakes with maximum acceleration of 0.33 g, except for uppermost portion. Story displacements were also satisfactorily small.

B. The structures based on the 60% of Law also showed satisfactory behavior as far as strength is concerned, under the earthquake with maximum acceleration of 0.30 g, except for uppermost stories.

C. In either case, the uppermost portion showed whipping, which is very dangerous from the practical point of view.

Dr. Hisada and his group on the response subcommittee attempted to remove this difficulty, and found that the improvement was obtained by redesigning the structure with increased stiffness in the uppermost portion. This valuable finding, which may be referred to as the Hisada effect, suggested that better dynamic behavior would be obtained only by designs that take into consideration both strength and stiffness.

The author had doubts that the effect of short component of earthquake waves might be missed when reduced to five-mass system. Later Dr. Hisada and his group made analysis on a digital computer taking all 29 masses into account. It has been found that the general tendency of the response was satisfactorily predicted by the reduced system, although some stories had greater deformations (in some instances, twice) than the average deformations indicated by the 5-mass system.

This research indicated, for the first time, the possibility of high-rise construction, and helped to create a climate to repeal the building height limitation.

2. Activities of SERAC Committee

The SERAC Computer was installed in the University of Tokyo by the SERAC Committee headed by the author, with the financial support of Toyo Rayon Foundation for Promotion of Science and Technics.

The block diagrams are shown in Figs. 5, 6, 7, 8. The computer is a non-linear electronic low-speed analog computer capable of direct computation of response of five-degree-of-freedom shear system subjected to any ground disturbances.

Five SERAC reports have been published and distributed to date. Some topics of interest are mentioned briefly below.

A. Bi-linearity

The response of structures in the plastic range is sensitive to the assumptions of non-linearity. When negative bi-linear or perfect plastic characteristics are assumed, the deformation due to earthquakes is unstable, and the structures are apt to lose their resistance. When positive bi-linear characteristics are assumed, on the other hand, the deformation is always stable. This indicates that the ductility, preferably with strain hardening, is essential, besides the strength, in order to ensure the safety of structures to withstand earthquakes.

B. Analysis of the resistance of Toyo Rayon building against earthquakes

SERAC Committee made a response analysis of Toyo Rayon building. Using actual data of the structural design of this existing building, response was obtained for El Centro NS 1940 motion. Natural period of vibration of the building was 1.26 sec. When subjected to an earthquake with a maximum acceleration of 0.2 g, upper stories yielded and the average story displacement was about 1.0 cm. When subjected to a maximum acceleration of 0.33 g, middle stories and below behaved elastically, whereas upper stories experienced plastic deformation with ductility factor of 1.7 and the average upper story displacement was 3 cm. This example showed that a building designed according to the Japanese Building Standard Law started yielding, when it was subjected to an earthquake with El Centro wave pattern and maximum acceleration of 0.2 g.

C. Response of PCA 24 story RC building

A 24 storied reinforced concrete building in "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" published by the Portland Cement Association was analyzed for El Centro earthquake. Natural period of vibration of the building was 1.94 sec. Under maximum acceleration of 0.10 g, yielding started. Under maximum acceleration of 0.165 g, ductility factor of middle stories reached 3.0. Thus the bi-linear behavior was rather unstable and the author feels that a 65% increase in acceleration resulted in a 200% increase in the ductility.

3. Study of the Design of KM Building

In the summer of 1962, the author assumed the responsibility for the structural design of the KM Building, soon to commence construction. The preliminary investigations of a 30 story building has been published in SERAC Report No. 3, which, the author believes, gives a valuable suggestion to the design of high-rise buildings. Framing of the building is shown in Fig. 9.

As shown in Fig. 10, three designs were made. The first was based on the 50% of the requirement of Japanese Law. The second was based on the 25% of the same Law, with some modifications in the distribution of seismic coefficients so that better response would be obtained. The third was based on the 400% of SEAOC requirement in which case base shear was nearly same as for the second design. The schedule of framing members is shown in Fig. 11.

Three buildings were reduced to five-degree-of-freedom systems, and assuming damping coefficient of 5%, they were subjected to El Centro NS 1940, Taft EW 1952, and Tokyo EW 1956 motions with maximum acceleration of 0.33 g each. Results are shown in Figs. 12, 13, and Table 1

A. Effect of the difference in earthquake waves

El Centro earthquake gave the biggest response to all of these buildings. In general it would suffice to check the response against this earthquake record if the building is founded on the hard ground. For buildings on the soft ground, this earthquake may not be adequate as, for example, the Osaka earthquake record gives greater response to high-rise buildings.

B. Effect of the difference in structural design

In terms of ductility factor, it is seen that the building No. 1 shows extensive plastic deformation in upper stories. The building No. 3 also showed some plasticity. On the other hand, the building No. 2 remained in the elastic range in the three earthquakes.

It is striking to find that the building No. 1 had less capacity against earthquakes than No. 2 regardless of its high design shear which is almost twice as for building No. 2. The author arrived at the conclusion that this was due to the tapered proportioning of columns in building No. 1. Conforming with Dr. Hisada's experience in the Japanese National Railway committee that he succeeded in controlling the whipping by increasing the stiffness of upper stories, the current result again indicated that the traditional Japanese design of columns, i.e. thicker in lower stories and more slender in upper stories, was inadequate from the dynamic response point of view.

The author requested in the design of building No. 2 that the column sizes should be as uniform as possible from the bottom to the top and in addition the design base shear be reduced to one-half of building No. 1. The seismic coefficient distribution based on computer studies and the resulting story shears are modified as shown in Fig. 10 and it is evident that the dynamic response for building No. 2 is the best.

4. Vibration and Wave Transmission

The foregoing discussion will be better understood with the concept of wave transmission.

Recently it is the general opinion among seismologists that earthquake waves may be regarded as the superposition of random pulses. The author found from response curves that this is the case for most earthquake records. This concept encourages the treatment of earthquake response as wave transmission.

Many years ago (in 1927), the author proposed a solution of frame building response by substituting the frame with shear body and by adopting wave transmission theory. In this case a wave entering from the foundation is transmitted upwards, reflects at the free top end, then transmitted downwards and then superposed with incoming waves[8].

Extending this to a uniform frame with concentrated mass at each floor level, the author found that the wave velocity was not uniform, the shorter the wave length, the less is the wave velocity. For a wave length including n masses, the velocity c' is expressed by

$$c' = \frac{n}{\pi} \sin\frac{\pi}{n} \cdot c$$

where c = velocity of long wave.

In case of sharpest wave having two masses in a wave length, the minimum of n being 2, the lowest velocity is

$$c' = \frac{2}{\pi} c = 0.64 c$$

On the other hand, the stiffness of the open frames increases for sharper waves especially when the beam stiffness is low. This implies that sharper waves have higher velocity. (The results of the studies made in 1927 are summarized in reference 9)

In case of actual structures, these effects are further combined with non-uniform distribution of mass and stiffness. Consequently the earthquake waves are transmitted through the building, thereby changing their form in a very complicated manner.

A new principle of aseismic design may be deduced from these considerations, that is, "resistance of structures to earthquake motions consists in having earthquake waves pass the structures with the least disturbance". The principle of uniform proportioning of columns in building No. 2 is an applicable example. On the other hand such a design concept requires the use of digital computers by taking all of the masses into account.

Research towards the establishment of a new simplified design principle is strongly recommended based on the concept of wave transmission, as it appears to the author that this concept is well adapted to the case of very tall buildings over 30 stories high. In other words the design of such tall buildings should be based on the response analysis taking all masses into account. On the other hand the concept of vibration and modal analysis may be suitably applicable to lower buildings.

IV. AN EXAMPLE OF STRUCTURAL DESIGN FOR EARTHQUAKE

The author is continuing further studies on the revised 35 story KM building and the design procedure for this case will be briefly described as an example[10].

1. Design Procedure

Fig. 14 shows the procedure followed in the design of high-rise buildings adopted by the author's group. If we are given one particular earthquake record for which the building is to be designed, we can determine an optimum design. However, we cannot decide the most adequate earthquake record for a given site and location and soil condition. Thus, a sort of trial and error procedure as outlined in Fig. 14 has to be employed, in which tentative design is checked for several earthquake records of similar soil condition to make further modifications in the design.

Referring to Fig. 14, the preliminary design starts from assumed earthquake forces (1 - 1). Determining total story shear, structural analysis (2) is made, and bending moments, shears and axial forces are determined. For the combined loading, members are proportioned (5). Then establishing equations of vibration (6), the response was analyzed for various earthquakes. Deformations (7), story shear (8), overturning moment (9) are determined.

On the other hand, static analysis is made on the digital computer (6B) in order to improve the accuracy of the entire procedure. The stress due to axial deformation of columns for overturning moment is particularly important when frame includes core or shear walls. In the second design, the results of this static analysis are incorporated into the equations of vibration, thus improving the accuracy substantially.

The second design follows the same flow as the preliminary. In the meantime experimental works on the connection of framing members are made, which will be used in further modification of the equations of vibration.

2. Illustrative Example

Outline of the Building

Fig. 15 shows the framing of the building in the transverse direction. Columns and girders in the central span are thicker, forming a structural core. The distribution of lateral shear to exterior and interior columns is 1 to 3.5, according to the usual statical calculation.

Design Procedure

Procedure explained above was adopted. The most important thing in the design of high-rise buildings is to make the vertical distribution of stiffness as uniform as possible. This was mentioned in 1963 in the special lecture at the University of Tokyo commemorating the retirement of the author. After careful examination of recent research, the preliminarly design was made. Then the dynamic response and static structural analysis were carried out by a computer, and improving the defects and eliminating waste, the second design was made. It was again checked by using a computer, and the procedure was repeated.

Earthquake Motions

Both American and Japanese digitalized records are used as the input to the computer; the response of the preliminary and second designs to El Centro 1940 earthquake will be illustrated, using maximum acceleration of 0.33 g.

According to recent works (such as reference No. 11), steel structures usually have very low damping, occasionally as low as 1%. For additional safety, damping coefficient of 1% has been taken in this design.

The El Centro earthquake motion was applied at the level of the third floor. The lower stories are designed to be very strong and stiff. The effects of lower stories are to be considered later in the design.

Schedule of Columns and Girders of Second Design

Sections of columns and girders are shown in Fig. 16.

Assumptions in the Analysis

The author's practical D-value method was used in the preliminary design and vibration analysis. Considering the significant effect of total bending in case of frames having cores, however, the approximate method incorporating axial deformation of columns was used in the second design and analysis. The vibrational response for the second design is believed to be very accurate, because the structural analysis in the second design was compared satisfactorily with the accurate solution obtained from the digital computer.

Vibrational Modes of Second Design

Fig. 17 shows the vibrational modes of the second design after 2 sec. from the start of earthquake and thereafter with 0.2 sec. intervals. The movement of the building is not vibration in the usual sense, but it is a surprisingly complicated wave transmission. The short stubs in the figure denote the time associated with the maximum relative displacement in each story. Tenth story had a maximum drift at 2.4 sec., whereas for the 35th story, it was at 4.0 sec.

Design Shear and Response Shear

Assumed design shears and response values are compared in Fig. 18.

Overturning Moment

According to the preliminary design assumptions the overturning moment was 410,000 tm. However, the response values for preliminary and second designs are much smaller, being 330,000 and 240,000 tm, respectively (see Fig. 19).

Moment and Column Load due to Design Shear

Bending moments at the column heads and axial forces in the columns are shown in Fig. 20.

Relative Story Displacement

As shown in Fig. 21, the response value of story displacement - or story drift - is less than 2.0 cm in the lower stories; 2.1 cm in the 18th story; and less in 21 st, then 2.2 cm in 28th story which is the maximum.

Safety

The second design would behave within the allowable strength under El Centro 0.33 g earthquake condition and the maximum story drift of 2 cm would be produced by El Centro 0.29 g earthquake. The author believes that this is exceedingly safe for all practical purposes.

It is strongly recommended that an agreement on the magnitude of the design earthquake acceleration should be established for use in the dynamic design of buildings under varying conditions.

CONCLUSIONS

The following activities are deemed important and necessary.

Installation of Strong Motion Seismographs

It is most important to install strong motion seismographs at many places to obtain records of earthquake motions. At the UNESCO Intergovernmental Meeting on Seismology and Earthquake Engineering held in Paris, April 1964, it was agreed that a world-wide system of observation and exchange of data be implemented.

Analysis of Strong Earthquake Responses

The use of electronic computers for the analysis of earthquake response of tall buildings is very important. If such analyses are based on insufficient investigations, the solutions may be entirely different from the truth, hence misjudgment of the design may result. Following are some of the things to be considered: the choice of input earthquake waves, assumptions in the idealization of building into framing system for the analysis, introduction of the equations of motion, the choice of damping coefficient, energy dissipation to the ground, etc.

Concept of Wave Transmission and Multi-mass Analysis

The author mentioned the necessity of concept of wave transmission considering the fact that the earthquake waves consist of random pulses. In this regard he would like to recommend analysis taking all masses into account. In particular, when there is discontinuity of mass or stiffness distribution, involved singular reflection and transmission of waves will occur which may induce unexpected local stresses.

Utilization of Test Results

It is necessary to make tests to check and confirm the strength and stiffness of framing elements or connections. The most important is to make use of these test results in the modification of assumptions for the structural and vibration analysis. Taking steel construction as an example, it is occasionally noted that the deformations significantly exceed the conventionally calculated values even in the elastic range. It is very important to remember this fact in the analysis, especially in the response analysis.

Looking Ahead

It is very gratifying to note that earthquake engineering is emerging from chaos, and is developing with full vigor. The author is very happy that earthquake engineers and scientists of the world are cooperating for the benefit of mankind. With this, I wish to conclude my address at the Third World Conference on Earthquake Engineering.

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9. K. Muto, "Earthquake Resistant Design of Building", (Japanese), Tokyo, Japan, 1963.

10. K. Muto and others, "Study on Antiseismic Design of 35 Storied Office Building", (Japanese), Japan Science Council, Tokyo, Japan, 1964.

11. N.N. Nielsen, "Dynamic Response of Multistory Buildings", Calif. Inst. of Tech., U.S.A., 1964

Table 1

Comparison of several design

Elastro – Plastic Response   damping factor 5 %
max acc. = 330 gal

Bldg	Period (sec)	Earthquake	Cb	Max Ductility Factor of Story No.				
				1	2	3	4	5
No. 1 50% J.C.	2.95	El Centro	0.15	0.8	0.9	1.2	2.1	2.8
		Taft	0.10	0.7	0.6	0.6	0.7	0.8
		Tokyo	0.08	0.5	0.5	0.5	0.5	0.5
No. 2 25% J.C.	3.76	El Centro	0.09	0.9	1.0	1.0	1.0	0.9
		Taft	0.07	0.7	0.6	0.7	0.7	0.6
		Tokyo	0.05	0.4	0.5	0.4	0.5	0.4
No. 3 400%SEAOC	4.10	El Centro	0.09	1.0	0.9	1.1	1.1	1.4
		Taft	0.07	0.9	0.7	0.8	0.9	1.2
		Tokyo	0.06	0.7	0.6	0.5	0.5	0.5

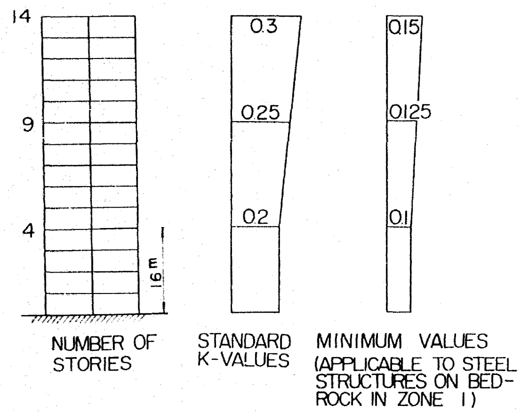

Fig. 1. Provisions of current building standard law.

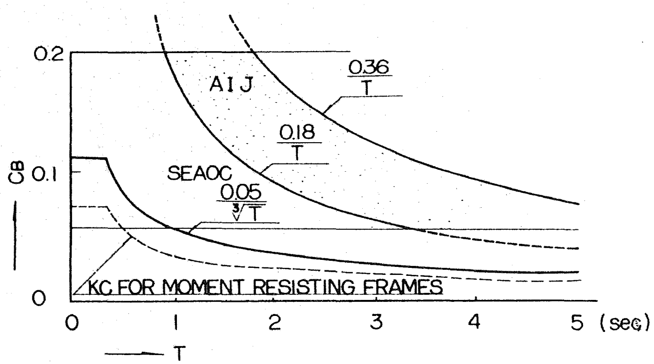

Fig. 2. Comparison of base shear coefficients.

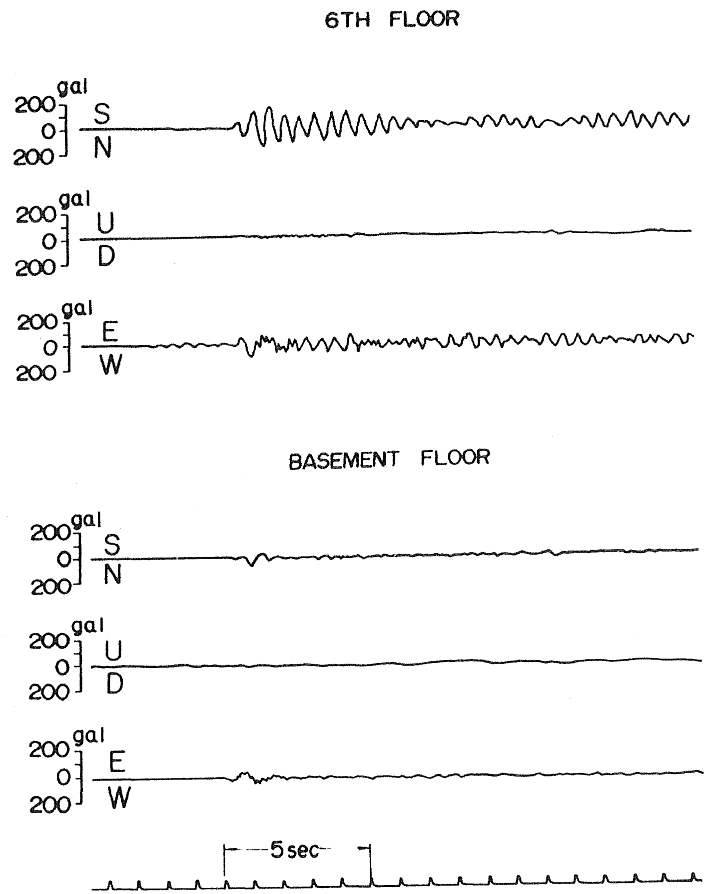

Fig. 3. Example of SMAC records for 6 storey steel frame bldg. in Tokyo.

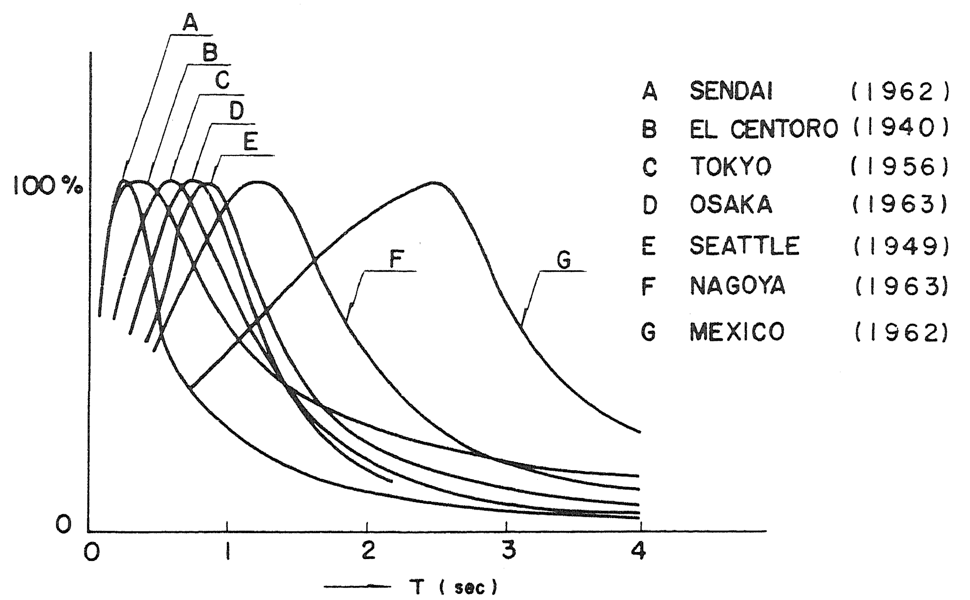

Fig. 4. Comparison of response spectra at various locations.

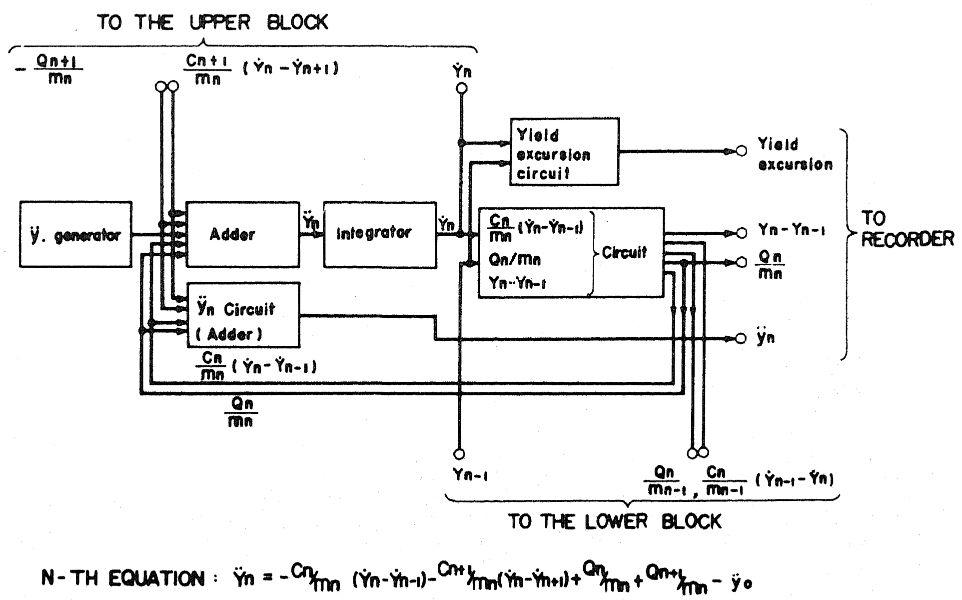

Fig. 5. Block diagram for solving n-th equation.

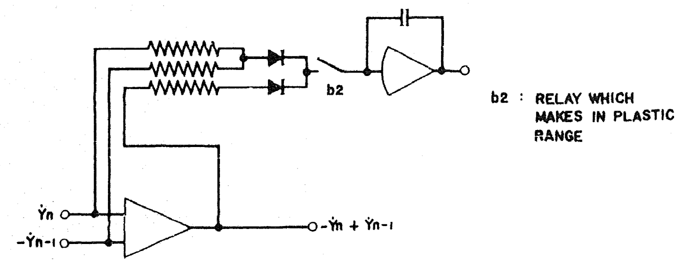

Fig. 6. Block diagram for yield excursion.

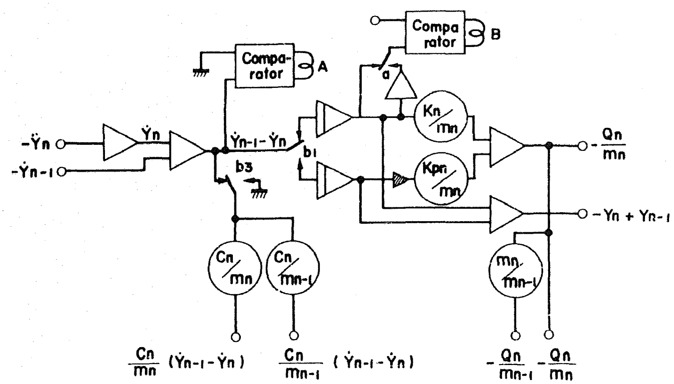

Fig. 7. General block diagram.

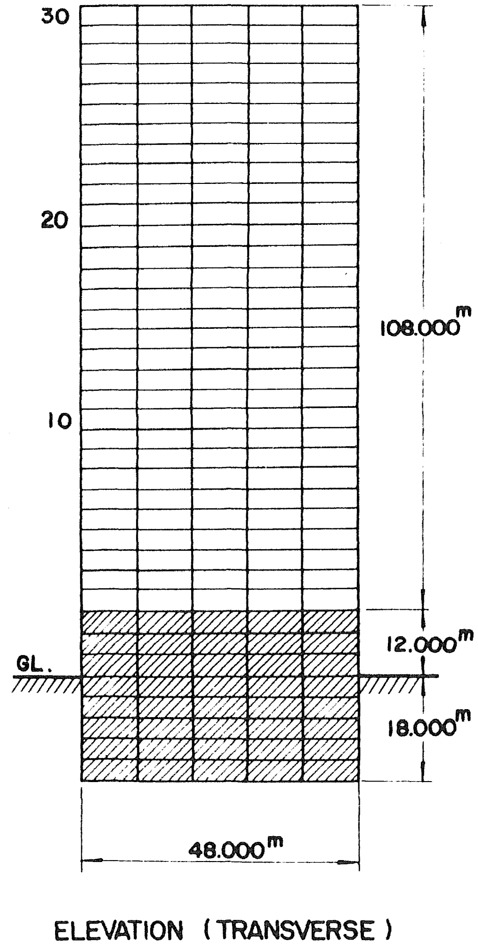

Fig. 9. Structural framing of KM building.

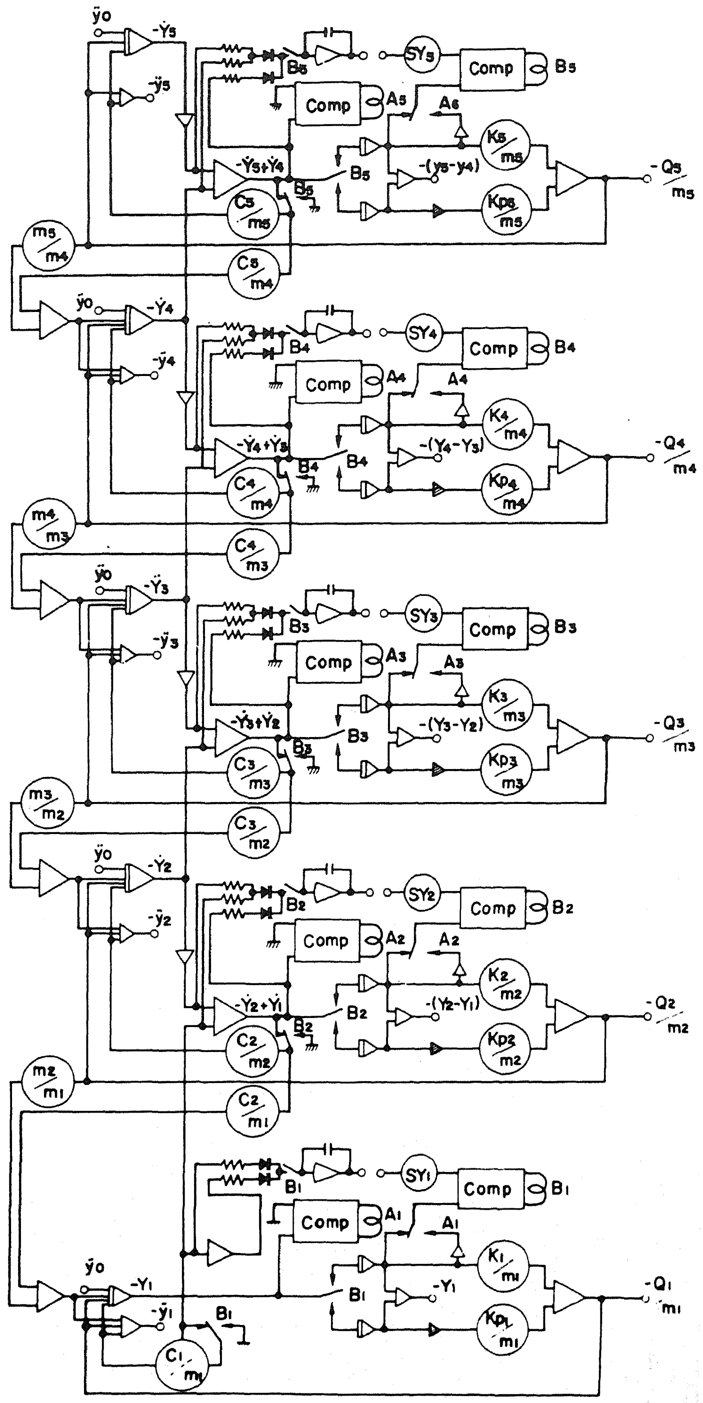

Fig. 8. Total block diagram for 5 mass system.

	No. 1 30S (50%)	No. 2 30S(25%)	No. 3 30S (400%SEAOC)
DESIGN SEISMIC COEFFICIENT	(STANDARD LAW) x50%	(STANDARD LAW) x 25%	(SEAOC) x 400%
ALLOWABLE STRESS	3300	3300	3300 kg/cm^2
STEEL DESIGNATION	SM50	SM50	SM50

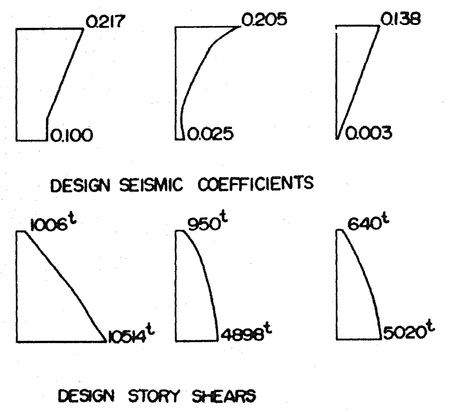

Fig. 10. Design seismic coefficients.

STORY	No. 1 30S (50%) COLUMN	No. 1 30S (50%) GIRDER	No. 2 30S (25%) COLUMN	No. 2 30S (25%) GIRDER	No. 3 30S (400%SEAOC) COLUMN	No. 3 30S (400%SEAOC) GIRDER
30 TH FL.	550, t = 12	300, 900, F.PL-300x12, W.PL-876x9	600, t = 12	400, 900, F.PL-400x12, W.PL-876x12	600, t = 12	300, 900, F.PL-300x12, W.PL-876x12
20 TH FL.	700, t = 25	450, 900, F.PL-450x22, W.PL-856x9	700, t = 22	400, 900, F.PL-400x18, W.PL-864x12	700, t = 20	400, 900, F.PL-400x14, W.PL-872x12
10 TH FL.	850, t = 28	500, 1000, F.PL-500x28, W.PL-944x12	700, t = 30	400, 900, F.PL-400x24, W.PL-852x12	700, t = 28	400, 900, F.PL-400x18, W.PL-864x12
1 ST FL.	1000, t = 36	500, 1150, F.PL-500x28, W.PL-1094x12	800, t = 36	400, 800, F.PL-400x26, W.PL-848x12	800, t = 36	400, 900, F.PL-400x20, W.PL-860x12

Fig. 11. Schedule of columns and girders.

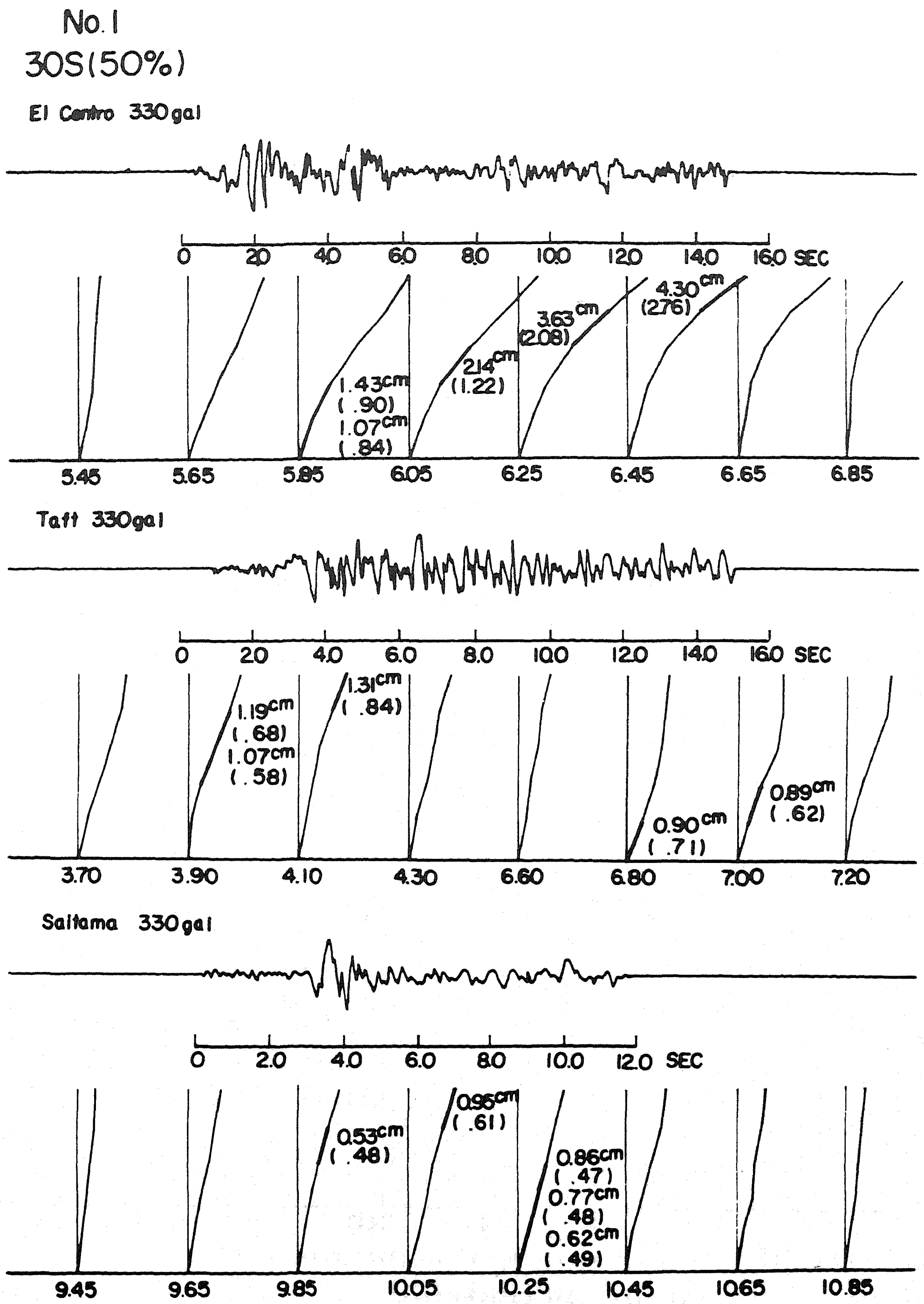

Fig. 12. Vibration forms with maximum drift and ductility factors (in brackets)

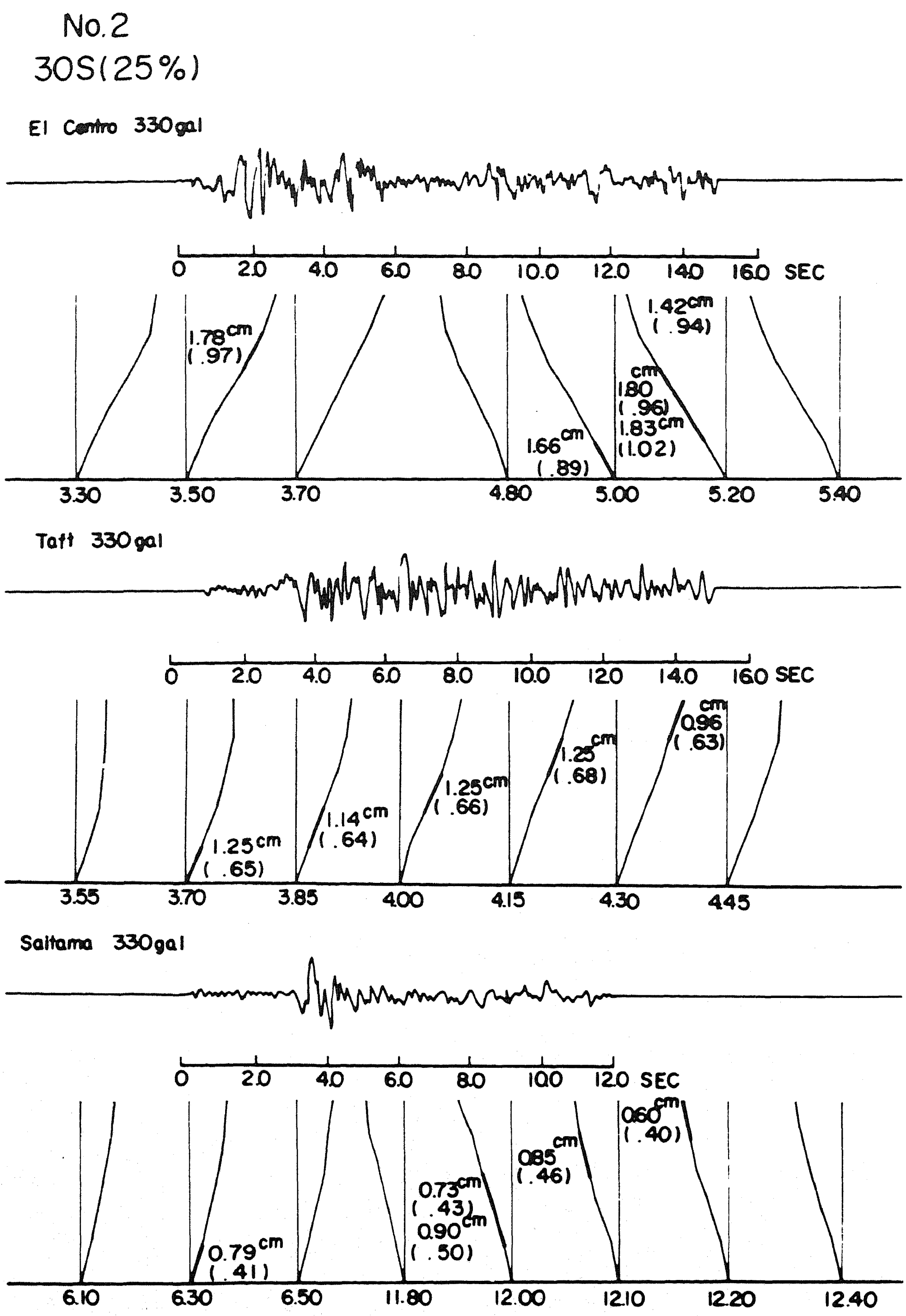

Fig. 13. Vibration forms with maximum drift and ductility factors (in brackets)

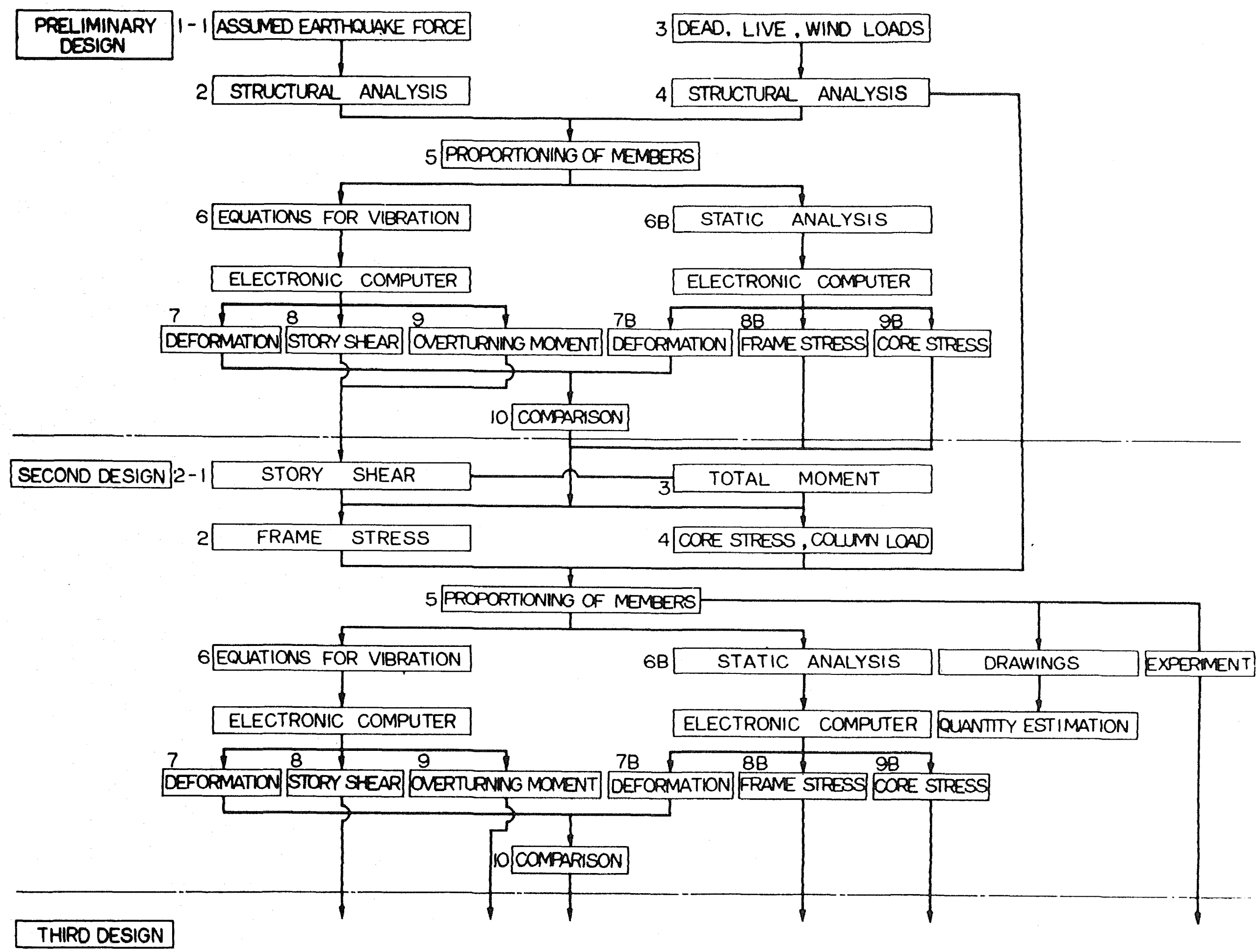

Fig. 14. Flow diagram of structural design for earthquakes.

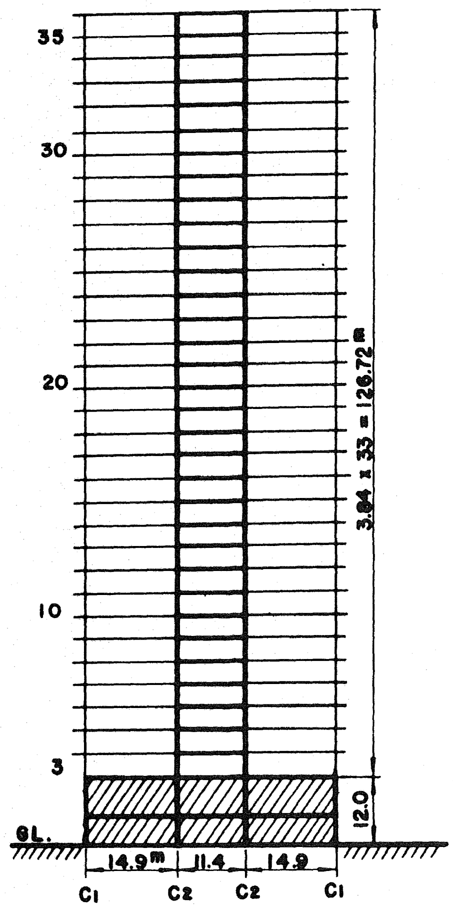

Fig. 15. Structural framing of the building.

	COLUMNS		GIRDERS	
	EXTERIOR	INTERIOR	EXTERIOR	INTERIOR
	D, Bx, By SM50A	D, Bx, By SM50A	D, B	D, B
35	D = 900 Bx = 300 By = 300 tF = 18 tW = 12	D = 1100 Bx = 300 By = 300 tF = 18 tW = 14	H-800x300 x 14 x 26 (SS50)	D = 1000 B = 400 tF = 24 tW = 18 (SS41)
25	D = 900 Bx = 300 By = 400 tF = 24 tW = 12	D = 1100 Bx = 400 By = 400 tF = 26 tW = 14	H-600x300 x14 x 26 C.P. 6 x 300 (SS50)	D = 1000 B = 400 tF = 24 tW = 18 (SS41)
15	D = 900 Bx = 300 By = 400 tF = 30 tW = 20	D = 1100 Bx = 400 By = 500 tF = 34 tW = 22	H-800x300 x14 x 26 C.P. 6 x 300 (SS50)	D = 1000 B = 400 tF = 26 tW = 16 (SS41)
5	D = 900 Bx = 300 By = 400 tF = 35 tW = 30	D = 1100 Bx = 400 By = 500 tF = 42 tW = 38	H-800x300 x 14 x 26 C.P. 6 x 300 (SS50)	D = 1000 B = 400 tF = 34 tW = 18 (SS41)

Fig. 16. Schedule of columns and girders.

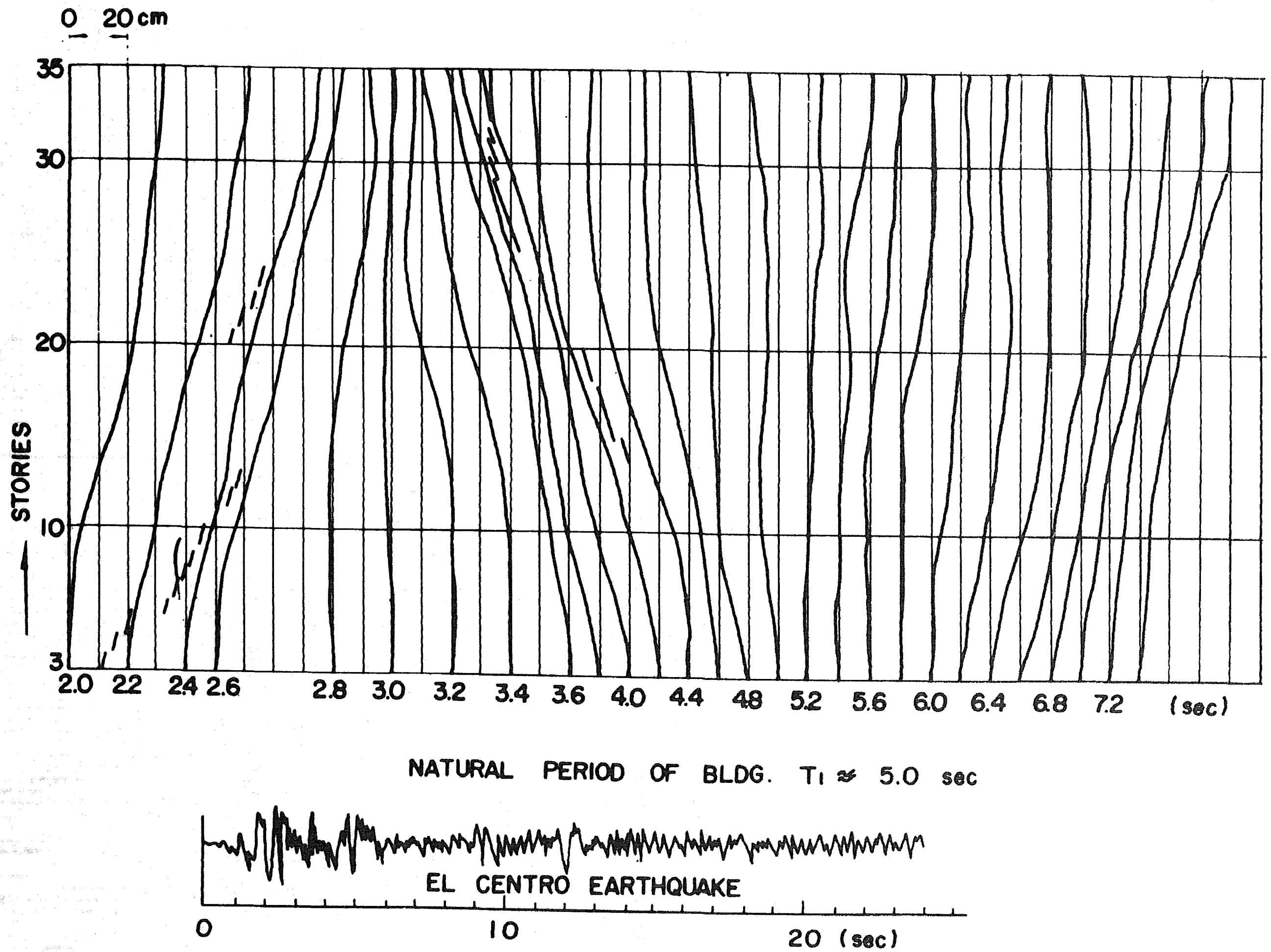

Fig. 17. Vibrational modes of second design.

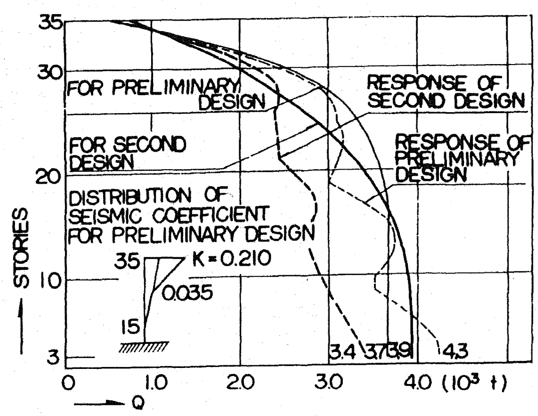

Fig. 18. Design shear and response shear distribution.

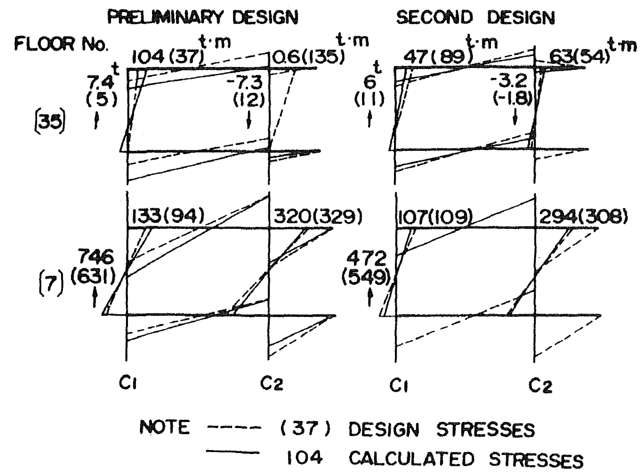

Fig. 20. Bending moments and axial stresses in columns in tm and t.

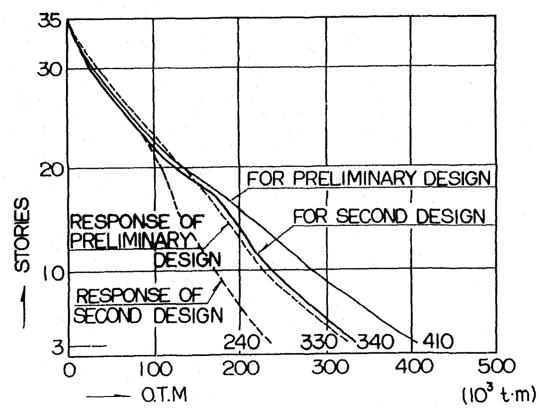

Fig. 19. Overturning moments.

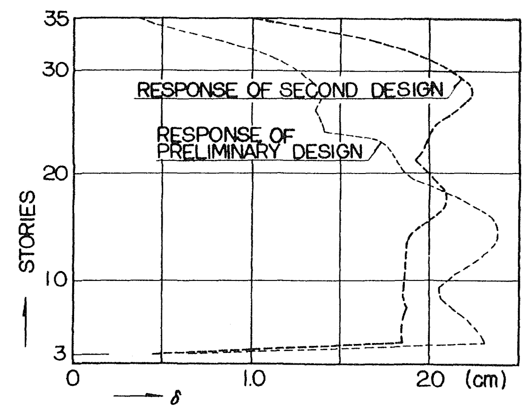

Fig. 21. Maximum relative story displacements of response.

RECENT TRENDS IN HIGH-RISE BUILDING DESIGNS

BY K. MUTO

D I S C U S S I O N

BY E. ROSENBLUETH

Without detracting from the value of this interesting paper, the writer wishes to refer to curve G in Fig. 4, which represents acceleration spectra for the earthquakes of 1962 in Mexico City. It is correctly pointed out that the maximum ordinates are associated with periods in the neighbourhood of 2.5 sec., but the curve fails to show the pronounced peaks in acceleration spectra that are systemically observed at about 1.8 sec, 1.1.sec., etc. These do not stand out in velocity spectra* but become apparent in acceleration spectra. The matter is important because the actual shape of earthquake spectra differ markedly from the much simplified curves that can be derived using formulas that contain a single maximum rather than the large number of prevailing periods noticeable save in strongly damped spectra.

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* Herrera I., Rosenblueth E., and Rascon D.A., "Earthquake Spectrum Prediction for the Valley of Mexico" 3 WCEE.