DEVELOPMENT OF EARTHQUAKE BUILDING CONSTRUCTION IN JAPAN

by YUKIO OTSUKI

INTRODUCTION

Long before the modern science of aseismic construction was developed, numerous earthquake resistant structures were built in Japan, namely, temples, pagodas and some special sections of various castles (1 and 2). It is evident that they are earthquake resistant because they have survived many disastrous earthquakes. Why they are earthquake resistant is not fully understood, even in light of our modern knowledge. About the only thing we can say, positively, is that such structures could not be approved under the provisions of the seismic codes now being enforced in Japan.

DEVELOPMENT OF THE JAPANESE SEISMIC CODE (3)

Near the end of the 19th century, there occurred a disastrous earthquake in central Japan, known generally as the Nobi-earthquake of 1891. The Tokyo area was badly shaken by the earthquake of 1894. As a result of the observation of the effects of these earthquakes, it was pointed out that spandrel girders and wall bracing were especially effective. The importance of establishing some standard for earthquake resistant structures was stressed. Acceleration seemed to be the major influencing factor.

Although a written form of aseismic construction requirements was not made available immediately, a common sense engineering approach was being made, especially by younger engineers in this field.

The San Francisco earthquake of 1906 created an accelerated pace by the younger engineers and some real thinking on the subject. Dr Riki Sano was one of these young engineers and he soon announced that an earthquake resistant structure could be built by assuming lateral force as proportional to the structures weight. The multiplier could be expressed in the form of equation 1.

$$K = \frac{\alpha}{g}, \qquad \text{..........} \quad (1)$$

Where α is the assumed seismic acceleration and g is the acceleration of gravity. To apply this concept to a multi-storied structure, the weight could be assumed to be concentrated at each floor level and be multiplied by K to obtain the design lateral force. This value "K" was called, "the Seismic Coefficient", and it is known in Japan as Sano's seismic coefficient. The use of this seismic coefficient became possible only with the designer assuming his own numerical value at his own risk. The value would be theoretically uniform in value along the entire height of

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M.S., Chairman Sub-Committee Vibration Testing, AIJ., Chief of Structural Section, Eng'r Dep't., Shimizu Construction Co., Ltd.

a building when it was very rigid, but would not be uniform in value for more flexible structures. It was about a decade before the catastrophic Kanto Earthquake of 1923 that Dr Sano published his aseismic design principle (4).

After Dr R. Sano formulated his aseismic design principles, Dr T. Naito developed aseismic design methods based on these principles. His foremost achievement was a method of analyzing walled frames by use of a "lateral force distribution coefficient". His concept was essentially based upon equal deformation assuming infinitely rigid floor slabs (10).

After the 1923 Kanto earthquake, the need for an aseismic building code was apparent. The K-value was given a value of 0.1 for general structures and 0.15 for chimney-like structures. This numerical value was based on the estimated value of 0.1 for the acceleration in the up-town area of Tokyo (Hongo) during the 1923 earthquake. It was believed that the acceleration in the down-town area of Tokyo (Marunouchi) might have reached twice or three times as much as the up-town area value, However, the counter-evidence of the Nippon Kogyo Ginko Building surviving with little damage in the down-town area, even though designed only with a K-value of 0.066, was a brake against application of a larger value for this area.

The proposals of the Imperial Earthquake Investigation Committee in 1924 were assimilated into the Building Code and have been enforced all over Japan since that time. At that time a dynamic approach to the problem was sought. Discussions of rigid structures versus flexible structures were published in the Architectural Institute of Japan Journal (5). Neither approach considered the unit deflection or deformation rate, which in the present thinking is considered important in such problems.

In the revised 1924 Building Code, the limitation of building height at 100 feet was kept unchanged. This limitation of height seems to have been determined solely to fit into the City Planning Scheme of that time (6).

The 1924 Building Code remained practically unchanged until the Japanese Engineering Standard, JES-3001, was published in 1948. The background for this Standard was as follows: With the progress in various fields of building research, ample statistical data was accumulated which made possible the specification of the design load at maximum possible values. The allowable stresses of the various building materials employed were determined to complement the determined loads. For long sustained loads, the creep limits were given, or for such materials which show no creep tendency under normal climate and loading conditions, the allowable stresses were specified to mainly keep the deformation within a certain maximum limit. For emergency loads, such as short time wind and seismic loads, it was the aim to define the probable maximum deformation in a certain time period. The wind force could be determined within fairly reasonable values. However, the seismic force could not be determined because of the lack of data on vibrational properties. Therefore, statistical ground acceleration data was used as the basis (see

Appendix -1). The allowable stresses for short time seismic loading conditions were determined at roughly twice the value allowed by the 1924 Code. Keeping step with the increase in the allowable stresses, the K-value was also doubled numerically to $K = 0.2$. The allowable stress under short-time loads was specified as the yield limit stress values for non-creeping materials. For materials exhibiting creep, a value of twice the long-time loading stress value was specified.

The underlying principle of JES-3001 was, insofar as the seismic provisions were concerned, that a structure built in compliance with specified provisions was not expected to resist an earthquake without damage but would sustain damage which could be repaired for a cost of not more than 10 percent of the original cost of construction (7).

After JES-3001 was published, it did not supersede legally the provisions of the 1924 Building Code. The Building Commissions, however, all required compliance with the newly published provisions of JES-3001. 1948-1950 were years of a post-war building boom in Japan. Many tall buildings were designed and built in all parts of Japan. Most of these were proportioned as rather more slender than in prewar years. On account of the doubled K-value, structural engineers had a difficult time to handle the up-lift forces at the column footings in many cases. The main complaint was that the large up-lift force was a result directly of the doubled K-value and could not be offset completely by use of the increased allowable stresses. It was argued further that this problem would not occur if the old 1924 Code provisions (still legally in effect) were used.

All this led to many heated discussions in the Architectural Institute. The result of the discussions was a deep disappointment to the practicing engineers as the effective K-values were not modified for an easier application to the taller buildings but instead were increased still more. The effect of flexibility was considered in the form of a gradual increase in the K-value, at a certain rate, for the portions above a certain height. It was confirmed also that a designer could use the old 1924 Code K-value of 0.1, but also must use the old 1924 Code allowable unit stress values.

An acceptable means of determining dynamically the effective K-value has been sought actively these past few years. The San Francisco Proposal on Lateral Forces (8) was an excellent start. Many papers on this subject have been published recently (14). Parallel to this effort, statistical studies on district probabilities of destructive earthquakes and on the damage rate relationship with the sub-soil condition made fair progress. The progress was considered effective enough to warrant making revisions in the Building Code. For the first time the K-value could be reduced to account for the reduced degree of the seismicity of the district. It could be reduced further in a direct relationship to the quality of the site subsoil. The Building Standard Law Enforcement Order of Nov. 1954 was modified in Aug. 1955 to account for these factors (see Appendix -2).

Very few engineers are satisfied with this Building Standard Law Enforcement Order as it now stands. Practicing engineers feel that the K-value hitherto specified might be unduly large; others wish to have a smaller K-value simply because of economical considerations; and the research men know that the overall seismic effect is not to be expressed as simply as specified by the Code provisions.

DEVELOPMENT OF JAPANESE EARTHQUAKE RESISTANT CONSTRUCTION

Until the end of the 19th Century, when Mr. Waters built a printing shop of brick masonry, Japanese building material consisted mainly of wood. Aseismic techniques employed by carpenters at that time were: 1. Excavated foundation, 2. Light roof construction, 3. Reinforcement of joints with metallic gussets, 4. Diagonal bracing or securely fastened sheathing, and 5. A base-less central column in tower-like structures. (See Fig. 1). However, these techniques were not known generally but were kept secret by the guilds and only members of those guilds were instructed in their use.

After the Meiji Era, Japan was eager to absorb foreign Culture and Sciences. Many brick buildings were constructed in the British Style but they were not earthquake resistant and most cracked or collapsed in the earthquakes of 1891 and 1894. It was learned that reinforcing of spandrel girders, improvement of the quality and workmanship of the cementing mortar, and rigid floor slabs did much to improve the earthquake resistance of these structures.

Wall bracing in the traditional wooden structures was found to be effective. Particular attention was paid to the foundation construction to prevent differential settlement as a means of increasing the earthquake resistance qualities of buildings.

Shortly after the turn of the century, the steel framed building was introduced into Japan and some small scale structures of this type were built. The Maruzen Building was the first large scale structure of this type constructed. It was a 3 story steel-framed structure with reinforced concrete floor slabs and reinforced brick exterior walls. Much was learned from the San Francisco Earthquake (9) concerning reinforced brick wall construction. This cage type construction proved fairly satisfactory in the 1923 Kanto earthquake, insofar as earthquake resistance was concerned, but the exposed steel framing was not fire-resistant and was badly damaged by the fires which followed the earthquake.

Though most of the brick buildings in the up-town (Hongo) area of Tokyo were damaged badly in the 1923 earthquake, the brick buildings in the Mitsubishi group in the down-town (Marunouchi) area suffered little damage. The demand for larger interior floor spaces and larger openings in exterior walls soon forced abandonment of the use of these brick structures.

Early in the 1920's, a new construction method was introduced into Japan. It consisted of a steel frame with hollow tile masonry curtain walls. It cut down the total weight of a building and hence reduced the static seismic force. The savings in materials and particularly the decrease in the construction time made it attractive from a cost standpoint. Many buildings, including the Marunouchi Building and the Nippon Yusen Kaisha (NYK) Building were built using this construction method. Most contractors seemed to be sold on its use. When the Marunouchi Building was almost completed, the Tokyo area was moderately shaken on April 26, 1921. The exterior walls of the Marunouchi Building were badly cracked. The partly constructed NYK Building suffered similar damage. The maximum acceleration in the down-town area during this earthquake was adjudged to be about 0.06 g. The toughness of reinforced concrete walls and diagonal bracing was observed again. The Marunouchi Building was reinforced immediately by interior reinforced concrete walls and diagonal bracing and as a result survived the 1923 earthquake.

In the 1923 earthquake, the damage survey revealed that the buildings with the reinforced concrete walls were for superior in resistance properties to those having the filled masonry walls. Brick buildings, with the exception of those mentioned previously, were found generally to be fragile. Most buildings which survived this earthquake had not been designed by methods which utilized a proper analysis of lateral force action but instead had been constructed fortunately utilizing stiffening wall panels placed judiciously enough to give the required resistance.

Japanese engineers learned much from the damage surveys of the 1923 earthquake. Together with the aseismic counter measures learned, the lessons of fire prevention techniques were not overlooked. Steel-framing was excellent in lateral load resistance because of its ductility but very poor in resistance against the high heat caused by after-shock fires. It was learned that the steel skeleton must be encased in fire-resistant materials. Thus was developed the construction method of casting the steel frame in concrete. In order to ensure proper bonding and some integral action, it was necessary to provide steel bar reinforcement. Steel in any form and particularly in structural sections was a very expensive building material. The desire was to count upon the added bar reinforcement as part of the structural member as a steel saving device. Thus the structural-steel skeleton, reinforced concrete-structure technique developed in our country. In this technique, the steel skeleton is composed of angles and channels in built-up sections utilizing the maximum overall section-modulus possible with a minimum of large rolled sections. Particular attention is paid to continuity at joints. This practice developed due to the fact that it was cheaper to fabricate a built-up section than it was to utilize a solid rolled section of equal section-modulus. Both the reduced cost of fabrication and the high cost of steel rolling facilities for large sections contributed to this practise.

In wholly bar-reinforced concrete practice,it was the initial custom to hook the ends of all bars. At the time that the use of steel framing with concrete floor slabs and hollow-masonry wall panel construction came into vogue, as previously mentioned, a new practice in bar reinforced concrete construction was introduced. This consisted of elimination of the end hook and the use of a deformed type bar. This was, of course, adopted to try and keep the cost of reinforced concrete construction competitive with the new steel frame construction. The majority of the research men were sceptical of this practice but it was welcomed by the contractors, partly because it saved some steel and cut construction time and partly because steel bars at that time were brittle enough so that cracks were generated when the bars were bent. The Naigai Building being constructed using this new practice was destroyed completely by the 1923 earthquake. It could have been caused partly by poor concrete and placement resulting in poor bonding. In those days, the specifications for structural concrete required no care in the water-cement ratio. Laboratory tests on bonding properties definitely proved the superiority of the end hooked bars in ultimate strength development. The use of end hooks again became an important earthquake resistant feature in reinforced concrete construction. The quality of steel reinforcing bars was improved to insure the proper ductility for making hooks. Concrete also was the subject of study; proportioning, mixing, method of placing and strength were studied to insure the specified uniform strength necessary in all portions of a structure. The effectiveness of the end hook in developing the ultimate strength of the bars was noticed later in the survey of bomb damage during the war years.

Methods for analysis of framed structures made fair progress based on the slope-deflection method. Studies on the elastic behavior of structures established the stiffness-ratio concept. The reasoning, "Stress in each member is exactly proportional to its stiffness ratio", was the concept that prevailed once in Japan. Though the wall panel was to be dealt with as a very stiff member, its stiffness value obtained from use of the elastic theory was overly large. Judgment indicated that a comparatively reduced value should be used in actual design. The hitherto generally used Portal Method for frame analysis was replaced by a method utilizing the stiffness ratio, because it was considered that only the latter could represent the actual distribution of stress. However, the Portal Method was much simpler to use in actual practice. The lateral load carried by the frame, in general, was only a fraction of the total and therefore any discrepancy resulting from the use of the simpler Portal Method, it was reasoned, would not be of great importance considering the entire structure.

Regardless of this, the AIJ Structural Standard established in 1933 was based on the stiffness ratio principle. It was considered that a structure proportioned according to this standard would be well balanced and reasonably earthquake resistant and, as such, would contain no individual members that might suffer premature failure. As time passed, and the memory of previous earthquake damages was dimmed, the average structural designer tended to follow the provisions of this standard in a token fashion only, forgetting the fact that a sound preliminary design consideration of all aspects was most important. One of the buildings designed in this period was the Daiwa Department Store Building which collapsed in the Fukui Earthquake of 1948 (11 and 12).

Briefly, this large building was designed in the conventional manner. A certain amount of the design lateral force was distributed to the exterior walled frames. The distribution coefficients selected were not very large as compared with some other designs. However, the exterior walled frames, as built, did not warrant the amount of the load distributed to them. Stress concentrations in members framed into the more rigid walls were overlooked. Worse, perhaps, the foundation construction was inadequate. Piling length was determined evidently upon driving resistance alone. The site ground condition was deep alluvial with alternating layers of saturated clay and silt and with only the overlying surface layer having any appreciable density. It was highly probably that the field engineer selected short piles because of the high driving resistance encountered in the overlying strata as well as the cost factor. The design of the structural members of the building was poor. For instance, the columns although somewhat small in section for the size of building, were reinforced with ample quantities of steel bars but the arrangement and placing of this reinforcing was inadequate. The column tie steel bars were small, spaced excessively and there were no sub-tie bars used. Many other discrepancies were observed from the damage surveys (see Fig. 2). Many lessons were learned bluntly from the collapse of this building. It cannot be denied that buildings similarly constructed perhaps still are in use somewhere in Japan.

As mentioned previously, the steel-skeleton reinforced concrete structure has become the most commonly used construction standard for large scale buildings in Japan. However, until quite recently, there has been no standard method for analysis of such a structure. The reinforcing has been regarded as composite. Some designers, however, regard the fabricated steel section as a portion of the reinforcing bars. Others regard the reinforcing bars as a part of the fabricated steel section and tend to disregard the concrete portion. Generally, when the overall fabricated steel section proportion is low the former seems appropriate and vice versa. Recommendations for this type of a structure have been formulated from the results of intensive study by a special committee of the AIJ (13) (also see Appendix 3).

Studies on seismic walls have been going on continuously to find the most appropriate values for the "Lateral Force Distribution Coefficient". The effectiveness of the seismic walls has been proven in every earthquake in which the results of their use has been surveyed. When these walled panels are provided properly in a framed structure, the overall deformation of the entire building is reduced. In other words, these walls take a large portion of the lateral force that otherwise would be carried by the framing members if the walls were omitted. The behavior of these walls during an earthquake is of critical importance in an earthquake resistant structure. The evaluation of the action of solid walls, walls with openings, and continuous walls in the horizontal and vertical directions has been one of the most intensive programs in earthquake resistant research in Japan (16).

As mentioned previously, the use of the deformed steel reinforcing bar without the use of end hooks for anchorage was discontinued after the 1923 Kanto earthquake. Recent research on the deformed bar coupled with the improvements in the bar deformations has led to its general use again without end hooks but only if the bonding properties are ensured by the proper concrete. It has been observed that the bonding power has been weakened by fire-damage to the concrete members.

In the post-war rehabilitation program, non-combustible construction materials for use in low-cost housing programs have been investigated. As mentioned previously (14), many new types of building structures constructed of these materials have been tested. Most of them have proven satisfactory in so far as vibration testing was concerned and some have been accepted officially for general use. However, none of them have yet been tested by an actual earthquake.

The problem of the foundation structure is receiving more and more study. Recent progress in the overall science of soils mechanics has taught us there are many important causes of structural damage to buildings other than caused directly by the vibrational action of earthquakes. Such an effect as uneven settlement which prestresses resisting members causes many buildings to lose much of their earthquake resisting capacity. The action of soils, such as saturated sands and sensitive clay, when subjected to violent earth vibrations is receiving more attention.

CONCLUDING REMARKS

The Japanese Islands have been subjected in the past to violent earthquakes and will again be so subjected in the future. Therefore, all buildings in Japan should be earthquake resistant. The majority of dwellings in Japan are made of wood. Most of them have heavy roof tiles that are good for rain protection and solar heat resistance but which make for a top-heavy structure. The general practice, when employed, to make these dwelling resistant to earthquakes is to provide diagonal bracing vertically and horizontally or to install a certain amount of wall panels in well-balanced locations. However, the desire of most Japanese to have wide openings in exterior walls on the sunny sides of the house works against this solution. The additional cost of the bracing and wall panels contributes to the difficulties of building officials in enforcing the proper degree of Code compliance. Reinforced concrete apartment houses and reinforced concrete block type dwellings are being built in increasing numbers as the cost of these structures decreases due to more and improved materials and better construction techniques.

The big question is still unanswered "What is a truly earthquake resistant building ?" When a building is designed in accordance with all the present Code requirements for aseismic construction, it is still only earthquake resistant to a certain degree. The Code value of seismic intensity is only a hypothetical one even though determined on the best available knowledge of probability and past experience. Since it is known that even the ground motion itself varies by a fair amount in any locality; that the effective acceleration or velocity differs according to the nature of the structure and the ground motion; and that no-one can foretell the exact intensity or nature of the next earthquake, it is natural to wonder how many of the presently constructed "earthquake resistant structures" will remain undamaged in any future violent shock.

Our current aseismic Code provisions are not reasonable fully. The uniform seismic coefficient, applied regardless of the nature of the structure, the foundation and the site soil condition, is considered obsolete now in view of present knowledge. The most urgent engineering problem with the use of the large K-value is the handling of the large overturning moments or up-lift forces, since an equilibrium of forces and moments must be established before entering into the stress analysis. Most structural engineers complain about the laborious analytical techniques concerned with meeting existing Code provisions as they know those same provisions disregard a reasonable mode of seismic distribution.

One most important task is to get structural designers to think. Competency to make an excellent and exact stress analysis of the framing members of the building is not enough to ensure it having a high degree of seismic resistance. We try to encourage the practising engineer to participate in programs concerned to vibration testing of structures, damage surveys, and etc., which will tend to give him a well rounded knowledge of his problems and the ability to plan as well as to analyze.

As stated previously, in order to save on the use of large rolled steel sections, the fabricated-steel-skeleton reinforced concrete structure seems to be the most suited to Japan. Whenever it is economically feasible, we make the structure as rigid as possible. The minimization of the unit deformation helps eliminate possible secondary effects. Though it is possible in theory to save cost from flexibility, it seems rather dangerous to make low buildings flexible. The reason lies in the fact that damage has a more direct relationship to the unit deflection than to the unit force. With the flexible structure, there is the possibility of phase reversal with consequent large unit deformation. However, when the building is rather tall, even in such cases the unit deformation will not be too large. Since our seismic Code provisions assume some permissible damage, the design methods for the structural members is not strictly elastic but rather is based on the ultimate strength theory.

The present tendency in Japanese architectural design is to follow the general trend abroad towards more interior spaciousness and large exterior wall window areas. Naturally, as the structural frame becomes more open, the flexibility increases. The non-structural materials that are installed in such buildings, such as glazing and finishing materials, will not be able to deform at the same rate or extent as the frame without considerable damage, even though the frame itself remains undamaged. Since we are destined to have frequent earthquakes, the architectural design will have to consider a practical damage limit level in the use of these materials.

Appendix 1. Seismic Probability Map in Japan (17)

1. Seismic probability: Let n(I) be the frequency corresponding to the intensity greater than I, the quotient n(I)/T, the average frequency, gives the fundamental quantity which represents the seismic probability. If a parameter which defines the functional form of n(I)/T can be found, then the parameter will be a kind of physical index to represent the probability. According to the author's data:

$$n(I) = C \times 10^{\beta I} \qquad \text{.......... (2)}$$

can be written for a certain location. In equation 2, β is a constant independent of locality. Therefore, $n(I_o)$:

$$S(I_o)/T = \sum_{I=I_o}^{\infty} n(I)/T \qquad \text{.......... (3)}$$

may be considered as an index to represent the seismic probability.

2. Fundamental Design Seismic Coefficient: It may be sufficient if a structure is so designed as to resist an earthquake whose intensity is the maximum that is to be expected during the lifetime of the structure. It is possible to define a limiting intensity, I_o, such that a structure may experience earthquakes whose intensities are less than but never greater. It then would seem reasonable to consider I_o as the expected maximum intensity for the structural life of the building. Therefore,

$$S(I_o)\,t/T = 1 \qquad \text{.......... (4)}$$

Fig. 4 is an example of the computed probability map assuming the duration of the structure to be 100 years. The maximum possible values of acceleration (gals.) entered in Fig. 4 were computed using equation 5.

$$\bar{a} = 0.45 \times 10^{0.5\,I} \qquad \text{.......... (5)}$$

Appendix 2. Seismic Force Regulations of the Building Standard Law Enforcement Order and The Construction Ministry Notification, Aug. 1955.

1. Basic Coefficient of Seismic Force (K_0) (See Fig. 5).

2. Multiplier used due to the combination of structural type and site soil condition (η_1) (See Table 1).

3. Multiplier due to the site District (η_2) (See Table 2).

4. Structural design coefficient, K, is calculated by means of equation 6.

$$K = K_o \times \eta_1 \times \eta_2 \qquad \text{.......... (6)}$$

5. Allowable unit stresses for short-time loading are given in Table 3.

Appendix 3. Basis of Analysis of Steel Skeleton Reinforced Concrete Structures (13).

1. Design Method for SSRC Structural Members: There are 3 types of methods for practical design: The reinforced concrete type, the steel frame type and the accumulative type. The steel frame type method stands on a common basis with the American and British practise where a heavy steel frame is employed disregarding the covering concrete. The R.C. type and the accumulative type count the concrete strength as a part of the overall member section. The R.C. method, appropriately explains the mechanics of application when the fabricated frame steel quantity is low and the fabricated steel section will give a proper bonding area per

effective unit area or while the deformation remains within the elastic limit. The accumulative method is convenient as a working mechanism at the maximum structural strength or at a state of large deformation beyond the elastic limit. Authors have noted that the SSRC structure is a far more ductile structure than a plain R.C. structure and recommend that the accumulative formula be used.

2. Accumulative Formula: This is a method to define the ultimate strength of a SSRC member as a sum of its elements: fabricated steel frame and concrete reinforced with bar steel.

For pure bending: $M = M_{fsf} + M_{c,r}$ .... (7)

For combined bending and axial force:

$$P = P_{fsf} + P_{c,r} \qquad (8)$$

Where, $$M = M_{fsf} + M_{c,r} \qquad (9)$$

$$\frac{M}{P} = e \qquad (10)$$

and, $$e_{fsf} \geq e \geq e_{c,r}$$

For shear: $$Q = Q_{fsf} + Q_{c,r} \qquad (11)$$

3. Merits of the Accumulative Formula: The accumulative formula may allow a small amount of error, but on the safe side. When the formula is applied in actual design, it exhibits the following merits:

a. With variation of the fabricated steel frame to reinforcing bar ratio, the formula automatically adjusts somewhere between steel-frame formula and plain R.C. formula.

b. The formula is applicable regardless of the ratio of fabricated steel frame to reinforcing bars.

c. The use of the formula allows a design to economically approach closely to the steel-frame structure design in ductility.

d. The formula gives priority to the fabricated steel frame. The fabricated steel frame on the compression side can be utilized fully to its allowable stress. Since the steel section on the tension side is excluded from the reinforced concrete bar steel ratio, the strength of the fabricated steel frame is never unduly limited by the concrete compressive strength.

BIBLIOGRAPHY

1. "Jishin-no-ma (Earthquake-room) and the concept on Aseismic Structures in Old Days" by Y. Okuma, AIJ Journal, Vol. 29, p.607-621, 1915.

2. "On the Vibration of 5-storied Pagodas" by F. Omori, AIJ Journal, Vol. 35, p.219-226, 1921.

3. "Earthquake-proof Construction in Japan" by T. Saita, Proceedings The 3rd Pan-Pacific Concgress on Sciences, 1926. (Most of the Papers published before 1926 are listed in this paper).

4. "Key Notes on Earthquake Resistant Buildings" by R. Sano, AIJ Journals, Vols.29 and 30, 1915-1916.

5. For example, in AIJ Journals:
"Theories on Earthquake Resistant Structures" by R. Sano, Vol.41, No. 491, 1927
"On Earthquake Resistant Structures" by K. Majima, Vol.41, No.491, 1927
"Discussion on Sano's 'Theories on Earthquake Resistant Structures'" by K. Majima, Vol.41, No.494, 1927
"Critique on Dr. Majima's Principle of Flexible Structures" by K. Muto, Vol.45, No. 543, 1931
"Reply to Muto's Criticism and Supplementary Discussion for Inviting Public Criticism" by K. Majima, Vol.45, No. 545, 1931
"Should an Earthquake Resistant Structure be Rigid or Flexible ?" by R. Oka, Vol.45, No. 551, 1931 and Vol.46, No. 556, 1932
"An Opinion on Destructive Power of Earthquakes and Earthquake Resistivity of Buildings" by R. Tanabashi, Vol.49, No. 599, 1935
"My Impression on Mr. Tanabashi's New Theory" by K. Majima, Vol.49, No. 604, 1935
"I Support the Rigid Structure Principle" by T. Kono, Vol.49, No.607, 1935
"Reply to Dr. Majima's Discussion" by R. Tanabashi, Vol. 50, No.609, 1936
"I do not Support Mr Kono's Rigid Structure Principle" by R. Tanabashi, Vol.50, No. 613, 1936
"Reply to Mr. Tanabashi on Rigid Structure Principle" by T. Kono, Vol.50, No. 614, 1936
"The Second Reply to Mr. T. Kono" by T. Tanabashi, Vol.50, No.618, 1936

6. Current Report; Journal, No. 452, Vol.38, 1924

7. "Structural Design Method" by K. Muto, Architectural Science Series, Vol.14, Shokoku-sha, 1954, p.22

8. "Lateral Forces of Earthquake and Wind" by Joint Committee of the San Francisco, California Section, ASCE, and the Structural Engineeers Association of Northern California, Proceedings ASCE, Vol.77, Separate No.66

9. Engineering Record, Aug. 5, 1911

10. "Earthquake Resistant Framed Building Structures" by T. Naito, AIJ Journal, Vols.36 and 37, Nos. 436, 437, 438, 440 and 441, 1922-1923

11. "Report on Damages due to the Fukui Earthquake of 1948", by Special Committee for Earthquake Damage Survey, 1950

12. "The Fukui Earthquake, Hokuriku Region, Japan, June 28, 1948, Part II, "Engineering", by the Engineer Section, Far East Command, U.S. Army.

13. "On the Basis of Analysis of Steel Skeleton Reinforced Concrete Structures" by T. Naka, S. Takada and M. Wakabayashi, Proceedings AIJ, Vol.33, 1955, (Also see App. 4)

14. "Building Vibrations in Japan (Presented on the second day of the present conference.) by H. Kawasumi and T. Hisada

15. Papers on Building Vibrations and Design Seismic Force:
"A Few Comments on Design Seismic Force Distribution" by Y. Otsuki, Proc. 2nd Japan National Conference for Applied Mechanics, 1952
"Vibration of a Sky-Scraper" by R. Tanabashi and T. Kobori, Proc. AIJ, Nos. 17, 18, 22, 24 and 27, 1952-1954
"Effective Seismic Force on Tall Buildings" by T. Taniguchi and H. Kobayashi, Proc. AIJ, No. 20, 1952
"The Worst Deformation of a Building during an Earthquake" by T. Taniguchi and H. Kobayashi, Proc. AIJ, No.31, May 1955
"Structural Responses to Irregular Ground Motions" by T. Taniguchi, H. Kobayashi and N. Yamashita, Proc. AIJ, No.33, Oct., 1955
"Deformation of Buildings due to Irregular Ground Motions and Equivalent Static Lateral Forces" by T. Taniguchi, H. Kobayashi and M. Yamashita, Proc. AIJ, No. 33, Oct., 1955

16. Papers on Seismic Wall:
"Experimental Studies on Reduction of Rigidity due to Opening" by R. Tanabashi, Trans. AIJ, 1934
"Lateral Force Distribution Coefficient, Deformation and Stress of Seismic Wall in Elastic Range" by S. Ban, Trans. AIJ, No. 4, Feb., 1937
"Lateral Force Distribution Coefficient for Rectangular Wall and Effect of Openings" by S. Ban, Trans. AIJ, No. 30, Sept., 1943
"Seismic Distribution Coefficients of Earthquake-Proof Wall" by T. Taniguchi, Trans., AIJ, No. 41, Aug., 1950
"On the Resisting Strengths and Properties of the Walls in Wooden Buildings" by T. Hisada, Trans. AIJ, No. 42, Feb., 1951
"Lateral Force Distribution Coefficients and Stress Analysis of Walled Frames" by D.W.Butler and K. Muto, 1951, 262 pages.
"Aseismic Analysis of Wall with Openings" by K. Muto and Y. Osawa, Trans. AIJ, No. 45, Oct., 1952
"Analytical Study on the Lateral Shear Distribution of Seismic Wall" by K. Muto and Y. Osawa, Trans. AIJ, No. 46, March, 1953
"Experimental Study on a Multi-storeyed Walled Frame" by T. Taniguchi and T. Hattori, Proc. AIJ, No. 29, Oct., 1954

17. "Seismic Probability Map in Japan" by H. Kawasumi, Journal AIJ, April, 1946

NOMENCLATURE

$\bar{a}$	Maximum possible seismic acceleration in gals.
α	Design seismic acceleration.
β	A constant.
C	A constant.
e	Overall eccentricity of the applied force.
e_{fsf}	Eccentricity of fabricated steel frame.
$e_{c,r}$	Eccentricity of concrete reinforced with steel bars.
g	Gravitational acceleration.
I	Seismic intensity.
I_o	Expected maximum seismic intensity.
K	Design seismic coefficient.
K_o	Basic seismic coefficient.
M	Overall bending moment.
M_{fsf}	Bending moment carried by fabricated steel frame.
$M_{c,r}$	Bending moment carried by concrete reinforced by steel bars.
$n(I)$	Frequency of earthquakes whose intensities are greater than I.
P	Overall axial force.
P_{fsf}	Axial force carried by fabricated steel frame.
$P_{c,r}$	Axial force carried by concrete reinforced with steel bars.
Q	Overall shearing force.
Q_{fsf}	Shearing force carried by fabricated steel frame.
$Q_{c,r}$	Shearing force carried by concrete reinforced by steel bars.
$S(I_o)$	Frequency of earthquakes whose intensities are not greater than I_o, during a period of T.
T	A certain time period, and generally chosen equal to the assumed lifetime of a building structure.
η_1	Multiplier due to the combination of structural type and soil.
η_2	Multiplier due to the district.

FIGURE CAPTION

Fig. 1. Sketch of 5-storied pagoda in Nikko.

Fig. 2. Two nominally equal strength column sections.
a) Actual column section of Daiwa Department Store Building.
b) Alternate column section as slightly larger size.

Fig. 3. Portion of the Daiwa Building which survived.

Fig. 4. Expected 100 year maximum Seismic Intensity.

Fig. 5. Basic Seismic Coefficient (K_o) specified in current Japanese Code.

Table 1. Multiplier, η_1, depending on type of structure and nature of soil.

Table 2. Multiplier, η_2, according to the districts.

Table 3. Allowable unit stresses for temporary loading cases.

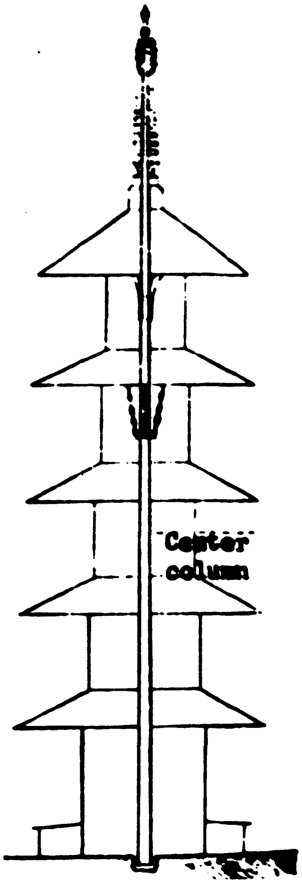

Fig. 1

Fig. 3

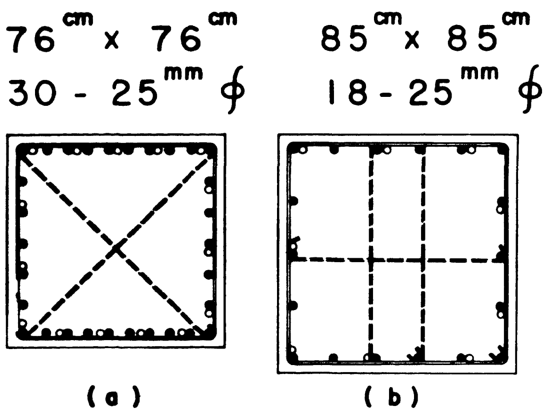

Fig. 2

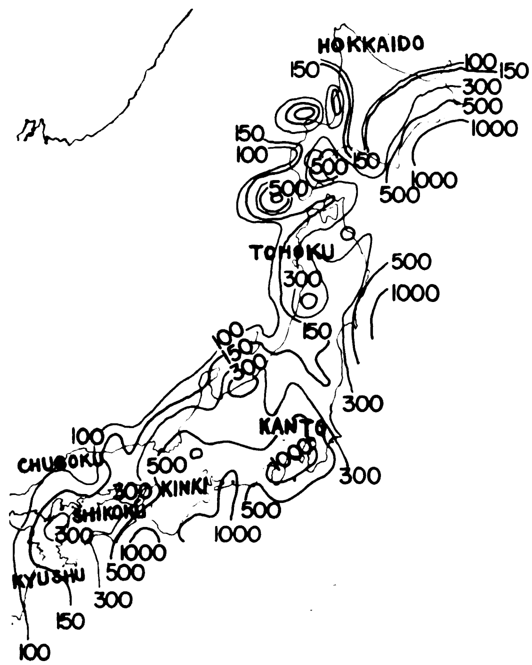

Fig. 4

TABLE 1

SOIL NATURE	MULTIPLIER η_1 — TYPES OF STRUCTURE: WOOD	STEEL	RC	MASONRY
TERTIARY OR OLDER	0.6	0.6	0.8	1.0
DELUVIUM OR ALLUVIAL GRAVEL	0.8	0.8	0.9	1.0
ALLUVIUM	1.0	1.0	1.0	1.0
VERY SOFT AND SATULATED SOILS	1.5	1.0	1.0	1.0

TABLE 2

DISTRICTS	MULTIPLIER η_2
CENTRAL KANTO AND KINKI	1.0
TOHOKU, SHIKOKU CHUGOKU AND SOUTHERN HOKKAIDO	0.9
NORTHERN HOKKAIDO AND KYUSHU	0.8

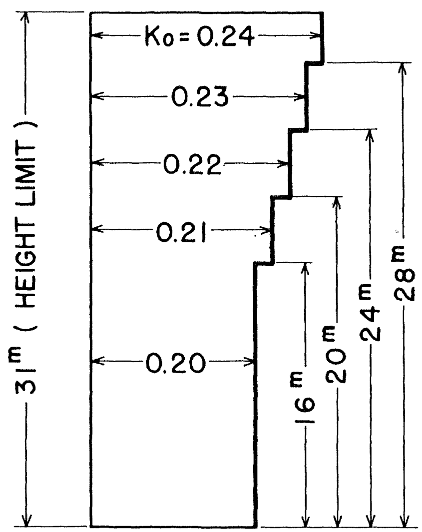

Fig. 5

TABLE 3

	COMPRESSION	TENSION
STRUCTURAL STEEL	2400 kg/cm^2 (34200 psi)	2400 kg/cm^2
CONCRETE	2/3 OF COMPRESSIVE STRENGTH	1/10 OF ALLOWABLE COMP. STRESS