Zeitschrift: IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

Band: 12 (1973)

Rubrik: Theme II: Studies on damping and energy absorption of structures

II

Studies on Damping and Energy Absorption of Structures

Etudes sur l'amortissement et l'absorption d'énergie de structures

Untersuchung über die Dämpfung und Energie-Absorption an Bauwerken

M. WAKABAYASHI
Professor of Disaster Prevention Research Institute
Kyoto University
Kyoto, Japan

1. INTRODUCTION

It has been well known for a long time that structures with large damping exhibit good performance against the earthquake excitation. A part of the energy caused by the earthquake is temporarily transformed to strain energy and kinetic energy of the structure. Input energy to the structure which is the remainder after the ground absorbed a part of total earthquake energy must be eventually dissipated by the visco-elastic action and elasto-plastic hysteresis of the main structural and cladding systems. Damping plays also an important role to reduce the various types of vibrational motion caused by the wind(63).

Damping can be divided into two types; internal damping and external damping. Visco-elastic damping, damping due to friction associated with inelastic slip occuring such as at bolted joints, and hysteretic damping due to large plastic deformation of members are examples of the internal damping. On the other hand, the external damping is related to the interaction between structures and surrounding objects. Damping due to air and to the interaction at foundations between the structure and the ground are examples of the external damping. The effect of the former is small. However, the latter, the effect of which is composed of the radiation loss of vibratory energy in the ground(3, 5, 8, 10, 11, 14, 27, 91) and the energy absorbed by the plastic deformation of the ground(90, 95), performs efficiently when the energy stored in the structure is released. The radiation loss of vibratory energy in the ground has a larger damping effect when the strucutre above ground is elastic and stiffer than the ground(92).

The elastic internal damping is often dealt with in the form of the equivalent viscous damping. The structure with large hysteretic damping has large energy dissipation capacity and thus may not reach its collapsed state even when it is subjected to strong earthquake motion. Therefore, it is very important to evaluate the hysteretic damping of the structure itself when checking the earthquake resistant ability of the structure.

If the earthquake motion is rather similar to the shock wave, the effects of viscous damping and of radiation damping are small, and the contribution from the hysteretic damping becomes more significant. In 1937, discussing the significance of the structural ductility in his velocity potential energy theory, Tanabashi indicated that the destructive force of the earthquake was proportional to the square of the maximum velocity and the resistance of the structure against the earthquake force was proportional to the potential energy conserved in the structure until failure(6).

In order to evaluate in general the deformability of the structure in the plastic range and how much plastic deformation a certain non-linear response of the structure corresponds, Newmark defined a ratio of the maximum deformation of the structure subjected to the earthquake excitation to the yielding deformation as a "ductility factor"(24). This definition then has been often used to discuss the response of the structure to the earthquake.

Figure 1 shows load-deflection curves of two different frames under monotonically increasing horizontal load. Maximum strength H_u of the frame may be determined based on the yield strength and/or stability limit load, and deformation capacity Δ_u may be determined according to the sudden decrease of the applied load or of serviceability caused by the rupture of steel material, local buckling of steel members and/or crash of concrete. Even though the maximum strengths of the frames shown in Figs. 1(a) and 1(b) are identical, the amount of absorbed energy by the frame shown in Fig. 1(a) before it reaches the limit deformation Δ_u is larger than that by the frame shown in Fig. 1(b), and the frame shown in Fig. 1(a) thus performs more efficiently than the frame shown in Fig. 1(b) in the sense of the earthquake resisting structure. Figure 2 shows examples of hysteresis loops of frames under repeated horizontal load. In these cases, the determination of the deformation capacity depends on the number of loading cycles in addition to the effects indicated above, since deformation phenomena associated with fatigue caused by the repeated loading have some effect on it. The frame shown in Fig. 2(a), which may dissipate a larger amount of energy before failure than the one in Fig. 2(b), may show better performance than the frame shown in Fig. 2(b) when subjected to the earthquake loading.

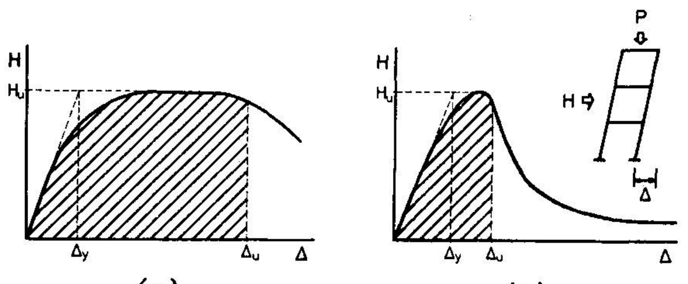


Fig. 1 Energy Absorption Capacity of Frames

In this report, viscous damping of various structures is first to be discussed. Then several kinds of methods to replace the structural damping by the equivalent viscous damping are to be introduced, and the hysteretic characteristics of materials, members, connections and overall structures are to be discussed.

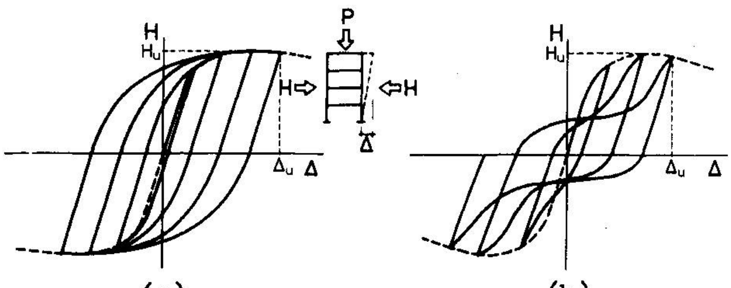


Fig. 2 Hysteresis Loops of Frames under Repeated Horizontal Loading

2. VISCOUS DAMPING AND HYSTERETIC DAMPING

2.1 Viscous Damping

It was shown that the approximate values of damping ratio υ, which was a ratio of viscous damping to critical damping, obtained from tests for several materials were; 0.05-0.1% for steel, 0.5% for reinforced concrete without cracks and 1.0-2.0% for cracked reinforced concrete(77). Prestressed concrete shows a value of 1.0% when the prestress is sufficiently applied and no crack is observed, and 2.0% is for prestressed concrete with microscopic cracks(37). Damping of steel structures is significantly affected by the type of connection system used. Reference (77) gave the following approximate values for the damping ratios of structures under the load of 70% of the design load; 0.5% for welded assemblies, 1.0% for high-strength bolted assemblies and 2.0% for riveted assemblies. Reference (25) indicated 1.0-3.0% for a usual bare steel frame, 1.0-8.0% for a concrete frame and 5.0-15.0% for a usual overall structure.

In order to evaluate the damping ratio υ of structural frames or bridges, a method based on the resonance curve obtained by the oscillation generator test and a method using quasi-static free vibration or free vibration caused by quick braking of vibration generator are often employed. Since some amount of energy is lost by radiation in the ground, it is rather difficult to obtain the internal damping characteristics of structural frame above ground only. The experimental data for υ obtained for the overall structural systems under the fundamental vibration including the structure-ground interaction showed the values scattering from 1.0 to 10.0%(12, 16, 17, 25, 37, 50, 51, 58, 60, 70, 77, 84, 85). Reference (51) gave an example where damping ratio υ of a bare steel frame was 2.0-3.0% while the same frame with cladding showed 5.0-6.0%. It is clear that the cladding has a significant effect on the internal damping of the structure. 2.0-3.0% for steel structures and 3.0-5.0% for reinforced concrete and steel reinforced concrete structures are the values of the damping ratio υ for fundamental vibration of these structures that are often used in the practical dynamic analysis performed for the design of tall buildings recently in Japan. Reference (85) contains experimental data obtained from dynamic tests of such tall buildings recently built in Japan.

Given in Ref. (47) are the general descriptions of damping behavior of bridges and experimental data for the damping ratio υ of existing bridges. Usual bridges show 0.5-2.0% as the values of the damping ratio υ(47, 77). Reference (22) obtained 0.4-0.6% from the experimental data of suspended bridges. In case of bridges, Coulomb's damping due to the movement of supports plays an important role. 0.2-0.5% for arch dams(46) and 2.0-3.0% for steel transmission towers(79) were also reported.

The damping ratio υ obtained from the micro-seismo tests, which is usually considered as viscous damping, is the result of the combined effect of friction at the surface of the crack, friction at connections and supports, radiation damping in the ground and many others. Therefore, unless the contribution from each of these effects are separately evaluated, use of the above shown values for the damping ratio υ must be limited to a certain extent.

2.2 Hysteretic Damping and Equivalent Viscous Damping

The internal damping of the structure in the elastic range is very small as already described. However, when the deformations of members and connections of the structure reach the plastic range, the hysteretic damping, that is considered to be equivalent to the energy-dissipation capacity, increases due to the large plastic deformation, and thus the structure may absorb and dissipate the input energy transmitted from the ground and be protected from the collapse. Such hysteretic damping has been often evaluated with the consideration of the so-called equivalent viscous damping coefficient. This idea, first proposed by Jacobsen(2), replaces an oscillator, which exhibits non-linear steady state vibration under sinusoidal excitation, by a linear oscillator with the identical resonance period and equivalent viscous damping coefficient which makes amounts of energy dissipated in one cycle by both oscillators identical. The relationship between restoring-force and deformation of an oscillator, as shown in Fig. 3, is not linear. This non-linear relationship is replaced by another oscillator whose restoring-force characteristic is linear given by a straight line AOC. The energy dissipation characteristic of the first oscillator is replaced by the equivalent viscous damping coefficient given to the replacing oscillator so that the area of the hysteresis loop ABCDA is equal to the effect of the viscous damping imposed in the replacing oscillator. Defining equivalent viscous damping coefficient as υ_{eq} it is given as

$$\upsilon_{eq} \equiv \frac{1}{2\pi} \cdot \frac{\text{Frictional Work Area(ABCDA)}}{\Delta OAE + \Delta OCF} \equiv \frac{1}{2\pi} \cdot \frac{\Delta W}{W} \qquad (1)$$

referring to Fig. 3. This energy ratio method non-dimensionalizes the area of hysteresis loop ΔW, which expresses the amount of the dissipated energy, by the another energy W.

Jacobsen obtained υ_{eq} by considering three types of skeleton curves; linear, soft spring and hard spring types(21). He concluded that the idea of the equivalent viscous damping coefficient could not be very easily applied on an arbitrary non-linear systems, and proposed an approximate formula to obtain υ_{eq} as

$$\upsilon_{eq} \equiv \frac{1}{2\pi} \cdot \frac{\text{Frictional Work Area}}{\text{Work Area under Skeleton}} \equiv \frac{1}{2\pi} \cdot \frac{\Delta W}{W} \qquad (2)$$

Referring to Fig. 4, Jacobsen proposed W_3 for the value of W in Eq. (2). On the other hand, W_1(45), W_2(35, 38) and W_4 have been also proposed. Reference (81) shows the values of W_1-W_4 when a Ramberg-Osgood type of hysteresis loop is used. Since the resonance period becomes longer as the vibration amplitude increases in the case of usual soft-spring type hysteretic systems, Caughey proposed a method to evaluate the equivalent stiffness and equivalent viscous damping by

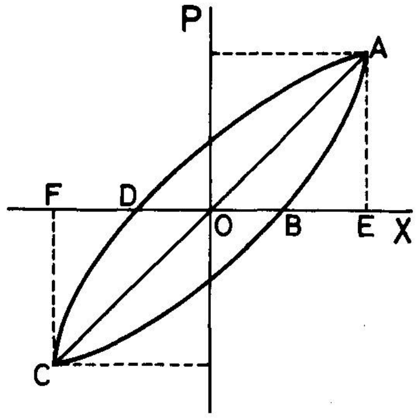


Fig. 3 Hysteretic Characteristics of a Nonlinear System

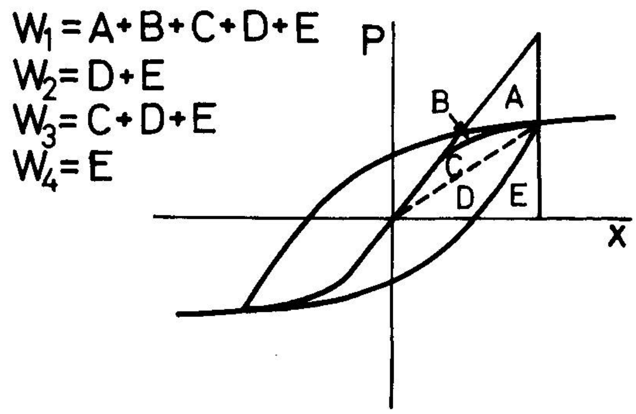


Fig. 4 Work Areas Taken for Computation of υ_{eq}

the least square method(19). This method is often used in the field of the response analysis.

In Reference (64), Jennings discussed the unique feature of the following methods to obtain υ_{eq}; 1° Resonant Amplitude Matching Method, 2° Dynamic Stiffness Method, 3° Dynamic Mass Method(45), 4° Constant Critical Damping Method, 5° Geometrical Stiffness Method(38), and 6° Geometical Energy Method(21). He commented that Dynamic Method(19) was useful in the theoretical investigation, that Geometrical Method was easy to apply and that Amplitude Method was understandable and gave the conservative value.

Fig. 5 shows the relationships between υ_{eq} and nondimensionalized deformation when the hysteresis loop is assumed to be elasto-plastic(64). Numbers identifying the curves in the figure refer to the above explained methods to obtain υ_{eq}. Since a large discrepancy is observed among the values of υ_{eq} computed based on the different definitions of υ_{eq}, it is necessary to clarify the definition of υ_{eq} when the experimental data are reduced. Although the idea of υ_{eq} is useful to clarify the qualitative behavior of the hysteretic damping of structures, the significance of this idea decreased because of the increasing error in the value of υ_{eq} when the hysteretic damping increases. In order to obtain accurately the response of the structure to a certain well-defined earthquake disturbance, a dynamic analysis may thus be carried out based on the real restoring-force characteristics. The ideas of equivalent viscous damping and equivalent stiffness might be efficient and useful, for example, when the response and earthquake resisting ability of a hysteretic system to random disturbances would be statistically dealt with(93, 94). Many references handled the energy dissipation behavior of connections and other structural elements with the form of υ_{eq} from the experimental data.

3.HYSTERETOC CHARACTERISTICS

3.1 Introduction

It is necessary to know the relationship between the applied load and deformation of a structure when investigating the dynamic behavior of the structure under earthquake loading. In an earlier time, when the non-linear vibration was analyzed, the real structure was replaced by a system of one-degree of freedom which was composed of a mass and a spring. Then, shear building type systems of multi-degree of freedom have gradually been employed to replace the real structure. In this case, the relationship between story shear and relative deformation in the story must be known to determine the spring constants of the replacing system. Recent development enables one to analyze the dynamic response of overall frame using its restoring-force characteristics which are composed of the relationship between the generalized stress and deformation of each structural elements, such as members, braces and connections(9, 18, 34, 43, 55, 56, 72, 73, 78, 89, 107). When evaluating the strain at each position of the structure from the results of the dynamic analysis, the response obtained by the analysis starting from the formulation of the load-deformation relationship of each member will give better results than the method using replacing multi-mass systems.

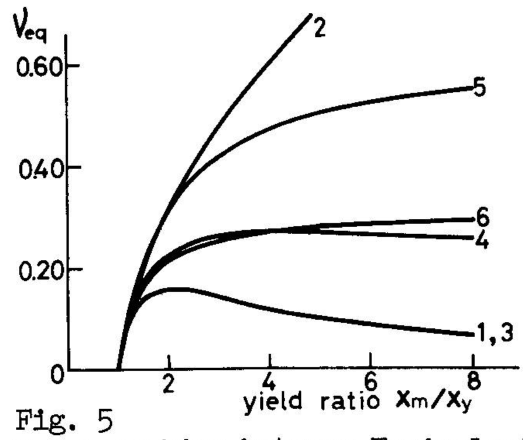


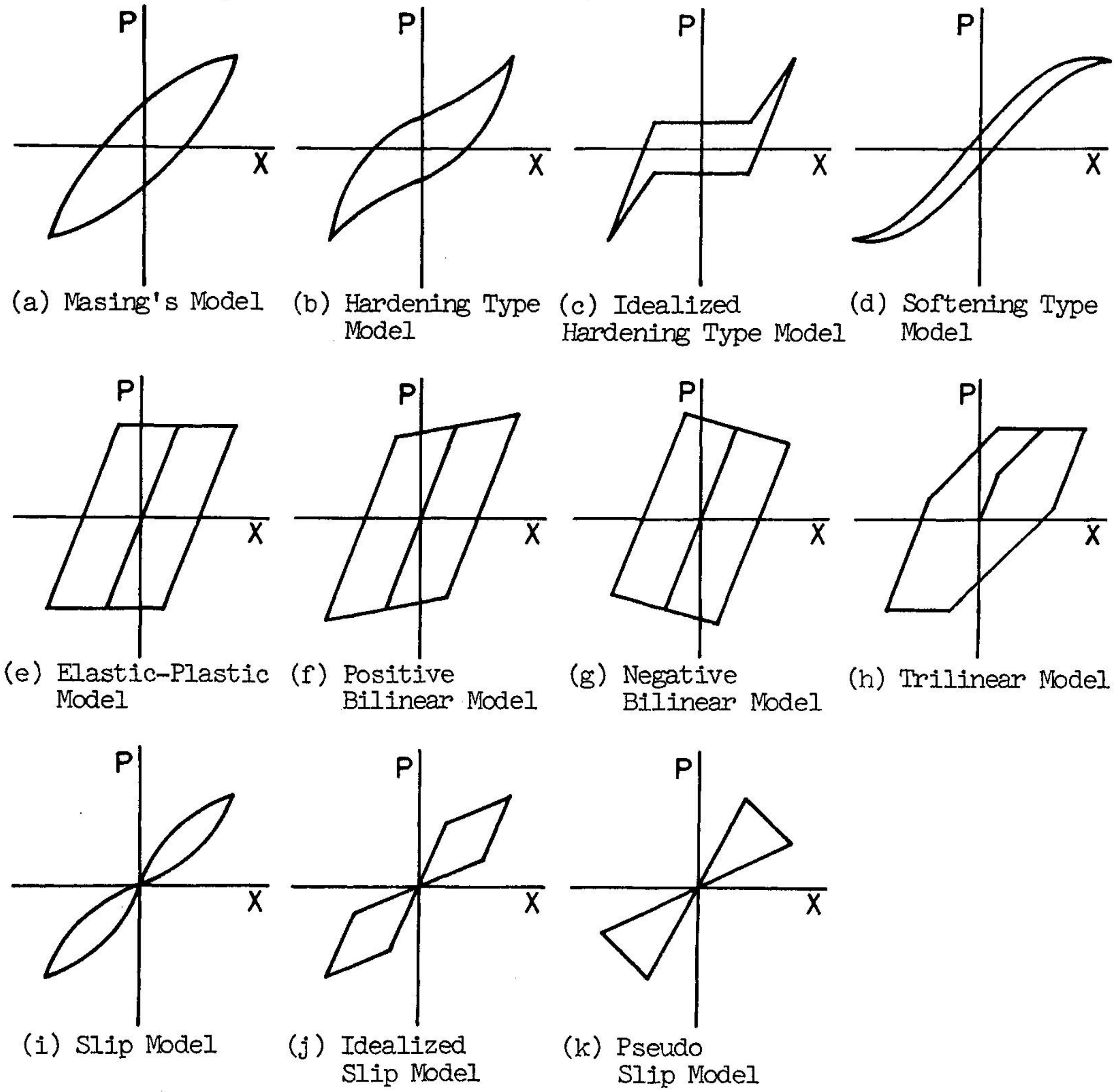
Fig. 5
Relationships between Equivalent Viscous Damping Coefficient and Deflection Amplitude

Furthermore, when the analysis starts from the formulation of the stress-strain relationship at each position of the structure and proceeds to formulate the load-deformation relationships of members, connections and finally overall structure, the results would be most accurate.

In this chapter of the report, first discussed are to be the general methods to formulate or idealize the hysteretic characteristics which are frequently used and applicable on an arbitrary generalized structure. Then, the formulations of hysteretic characteristics of materials, members, connections and overall structures are to be discussed.

3.2 Formulation of Hysteretic Characteristics

Fig. 6 shows several idealized hysteresis models frequently used for real loops of materials, members, connections and overall structures under repeated loading. A model shown in Fig. 6(a) usually represents a hysteresis loop of steel materials, steel members and frames, and welded connections. Shown in



(a) Masing's Model (b) Hardening Type Model (c) Idealized Hardening Type Model (d) Softening Type Model

(e) Elastic-Plastic Model (f) Positive Bilinear Model (g) Negative Bilinear Model (h) Trilinear Model

(i) Slip Model (j) Idealized Slip Model (k) Pseudo Slip Model

Fig. 6 Idealized Hysteresis Models

Figs. 6(b) and (c) are models for; 1)bolted connections in which slip occurs at a certain level of the applied load, 2)braced frame where brace is repeatedly subjected to the buckling deformation and plastic elongation, and 3)reinforced concrete members or shear walls with cracks. The hysteresis loop of prestressed concrete structures is often represented by model shown in Fig. 6(d). Figures 6(e) and (f) idealize the models in Fig. 6(a), neglecting and considering the effect of the strain hardening, respectively. When the gravity load on a frame is considerably large and P-Δ effect becomes significant, hysteretic horizontal load-deflection curve of the frame becomes as shown in Fig. 6(g). The model in Fig. 6(h) corresponds to the hysteresis loop of a structure which involves several frames with different stiffnesses, or of reinforced concrete member and frame whose stiffnesses significantly decrease when the cracks occur, compared with the state before the crack initiation. The models in Figs. 6(i), (j) and (k) are ficticious and used to compose a new hysteretic characteristic.

In 1935, Tanabashi described the hysteretic characteristics of reveted connection by combining the Masing's model(Fig. 6(a), 1) and his own slip model (4, 28). The load-deformation curve given by Masing's model is hogging when the load increases and sagging when it decreases. The unloading curve can be obtained by magnifying the skeleton curve twice in linear scale.

Most frequently used model for the nonlinear hysteretic characteristic is a bilinear type, which may be elastic-perfectly plastic type or elastic-hardening type. The stress-strain and moment-curvature relationships of steel and the relationship between horizontal load and deflection of a frame can be also replaced by the bilinear curves. Where the effect of the gravity load is not negligible, such as in the lower story of a tall building, a negative bilinear model as shown in Fig. 6(g) is used for the horizontal load-deflection relationship taking the P-Δ effect into account. In References (13, 15, 23, 24, 34, 48) and many others, dynamic analyses of structures were carried out, assuming bilinear horizontal load-deflection relationships, and the effect of the plastic deformation on the response was quantitatively evaluated. When the positive bilinear type of restoring force characteristic is assumed in the dynamic analysis, the obtained response is stable. On the other hand, the hysteresis loop of each cycle of the vibration gradually moves in one direction and the response finally diverges, when the negative bilinear type is assumed, and the phenomenon of incremental collapse appears(26).

Reference (76) assumed a model where a bilinear type hysteresis loop in each cycle gradually expanded as the number of cycles increased by the effect of the compressive strain cumulated in columns under the repeated bending.

References (29, 53, 97, 98, 101) assumed bilinear and trilinear models taking into account the effect of the stiffness reduction seen in the reinforced concrete frames and shear walls as the deformation increased.

Combining models shown in Figs. 6(f), (j) and (k), Ref. (96) idealized the hardening type hysteretic characteristic where the strength decreased with the increasing deformation, as shown in Fig. 2(b). In Ref. (83) the hysteretic characteristics of a prestressed concrete member as shown in Fig. 6(d) are idealized by a softening type polilinear model.

A representative model which idealized the hysteretic characteristics with mathematically expressed nonlinear curves was given by Jennings(33, 49). He drew a hysteresis loop for generalized yielding structure using a skeleton curve defined by Ramberg and Osgood(7) and Masing's hypothesis. This hysteresis loop is obtained by expanding the skeleton curve, determined by two constants, twice in linear scale. In many references, hysteretic characteristics of members, connections and frames have been idealized by Jenning's model(20, 54, 65, 80, 81, 108). In Reference (107), Ramberg-Osgood type moment-curvature relationship of members was used for the dynamic analysis of the overall frame. Another Masing type of formulation of the hysteresis loop was given by considering a class of models which are based on the idea of representing the structure as a continuously distributed collection of linear elastic and ideal slip elements(74).

Above introducted hysteresis loops are idealized models by Masing type formulation. Not many references can be found for the formulation of hardening type models as shown in Fig. 6(b) which are suitable for the hysteretic behavior of braced steel frames and reinforced concrete frames with shear-failing columns (112, 113). A model for the hysteresis loops of braced reinforced concrete frame was shown in Ref. (102), where the skeleton curve was determined by two constants and loops by ten. Reference (99) showed an idealized hysteresis loops by cubic curves.

3.3 Hysteretic Characteristics of Materials

In Reference (66), fundamental details concerning the hysteretic characteristics and damping behavior of metals and nonmetals are given and many references are also listed. Details of low cycle fatigue behavior of materials are given in Reference (30). Figure 7 shows a stress-strain relationship of steel material under monotonic loading and hysteresis loops under repeated loading with increasing the strain amplitude. As clarified from the figure, the area of each loop, that is the amount of dissipated energy in each cycle of loading, increases as the plastic strain amplitude increases. A linear relationship can be found between these two quantities plotted on the log-log scale. On the other hand, the total amount of the energy dissipated until the material reaches its plastic fatigue strength decreases as the plastic strain amplitude increases, and these two quantities are also linearly related on the log-log scale. Skeleton curve and hysteresis loops as shown in Fig. 7 can be idealized by Ramberg-Osgood formulation and Masing's hypothesis, respectively, and sometimes they are replaced by an elasto-plastic or a positive bilinear model in order to simplify the analysis.

In the case of concrete material, many factors governing the plastic behavior after concrete achieves its maximum strength are left unknown, even under monotonic loading. Very few experimental data are available to investigate the elastic-plastic behavior of concrete material under repeated loading.

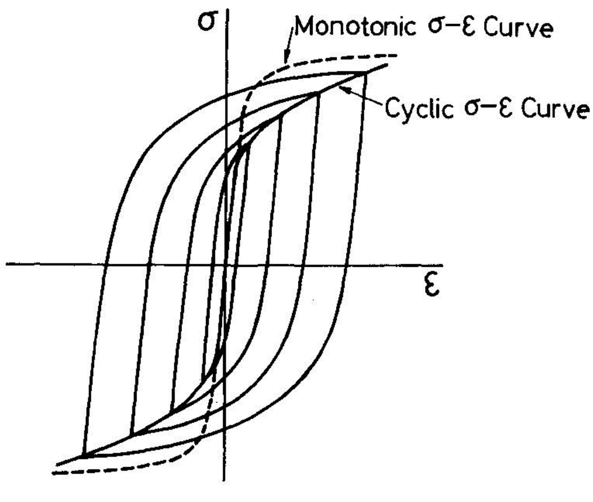


Fig. 7 Hysteretic Stress-Strain Relationship of Metal

Figure 8 shows an example of hysteresis loops in the large strain range of a concrete cylinder subjected to the compression repeatedly, and it is clearly observed from this figure that the slope of each unloading curve decreases as the strain amplitude increases(39, 75). Empirical formulas were proposed to express envelope curve, unloading curves and reloading curves of loops shown in Fig. 8(39, 75). Urgent investigations are needed on the behaviors of concrete under repeatedly applied tension and compression as in the case of beams, under repeated bi-axial stressing such as in the case of columns under constant axial thrust and repeated bending or shear, or hysteretic bond characteristics between concrete and steel bars.

3.4 Hysteretic Characteristics of Members and Connections

3.4.1 Steel Members and Connections

The hysteretic moment-curvature relationship of a steel cross-section under repeated bending can be theoretically obtained by dividing the cross-section into several elements which are assumed to behave according to the idealized Ramberg-Osgood type or bilinear type of hysteretic rules for steel. Then, using results of thus obtained moment-curvature relationship, the hysteretic load-deflection relationship of bending members can be obtained(68, 100). Reference (100) discussed the energy dissipation capacity, low cycle fatigue behavior and total dissipated energy of beams based on the results of above explained analysis. When a member is subjected to constant axial thrust and repeated bending, the consideration of the incremental collapse may be necessary(44). On the other hand, since the hysteretic moment-curvature relationship expanded by the strain hardening effect, it was assured that the hysteresis loops of a member under constant axial thrust and repeated bending expanded with the increasing loading cycle although the negative slope appeared in the loops due to the P-Δ effect(44, 76, 88, 100, 111, 113).

Very few researches can be found on the hysteretic characteristics of a member under bending or combined bending and axial thrust that may fail due to lateral buckling.

It is qualitatively known that beam-to-column connections built up by welding show hysteretic behavior similar to the one of a beam as shown in Fig. 6(a), and that bolted connections, in which slip may occur, show the hysteretic characteristics as shown Fig. 6(b) or (c)(59, 61, 67, 80, 108). Methods to obtain these hysteresis loops, especially those of the latter, theoretically have not yet been developed. In the case when the connection panel yields, a similar hysteretic characteristics as shown in Fig. 6(a) is obtained. When a bracing member is repeatedly subjected to the tension yielding and buckling, hysteresis loops become complicated and area enclosed by a loop in one cycle decreases as the slenderness ratio of the brace increases. The damping in stable loop cannot be expected in a very slender brace such as a round bar(103, 113). Investigation on such slender braces is still needed although there has been some work(71, 103).

3.4.2 Reinforced Concrete, Prestressed Concrete and Composite Structure

To theoretically obtain hysteretic behavior of reinforced concrete members under bending, a method based on the rules derived from the idealized model(31), and a method based on elasto-plastic hysteretic stress-strain relationship for steel and idealized relationship for concrete from the real hysteresis loops(40, 41, 62, 110), have been

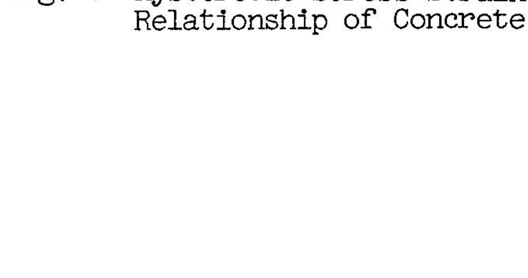


Fig. 8 Hysteretic Stress-Strain Relationship of Concrete

reported. The above method can be also applicable to reinforced concrete or steel reinforced concrete members subjected to constant axial thrust and bending (110). However, when the reinforced concrete member is subjected to repeated bending which causes the plastic deformation, it is not very suitable to assume that plane section remains plane after the member deforms, and that the stress-strain relationship of concrete inside stirrups is identical to that of concrete outside stirrups. Further investigations are needed on these points. The test results of prestressed concrete beams under dynamic repeated loading are shown and the energy dissipation is described in Ref. (57).

When reinforced concrete beams or columns shows shear failure, the hysteretic curves become as shown in Fig. 2(b), and no theoretical attack has been tried. In these cases the accuracy of the theoretical results would be reduced in comparison with the case without shear, since the direction of crack propagation is not necessarily normal to the longitudinal axis of the member.

In the case of reinforced concrete or prestressed concrete beam-to-column connections, panels of which would fail in shear, it was observed in some experiments that the hysteretic behavior was similar to that of a shear failing member. Many experimental investigations have been carried out on the reinforced concrete shear walls and the use of new structural system where steel braces are cast in the reinforced concrete shear wall have been recently initiated in Japan. Unfortunately, no research works to theorize the behavior of beam-to-column connections or shear walls have been performed.

3.5 Hysteretic Characteristics of Frames

3.5.1 Steel Frames

A few investigations on hysteretic behavior of unbraced frames under constant gravity and repeated horizontal loads can be found. Ref. (104) theoretically followed the experimentally obtained hysteresis loops of steel frames based on the assumed hysteretic moment-curvature relationships which were bilinear for beams and expanding positive bilinear for columns considering the effects of the strain hardening and cumulated compressive strain. Theoretical research works on braced frames are less. Assuming that a plastic hinge forms at mid-point of the brace after buckling occurs, and neglecting the interaction between the braces and the frame, the experimental hysteretic behavior can be theoretically followed to a certain extent(103). But more extensive investigations are necessary on this sucject.

3.5.2 Reinforced Concrete, Prestressed Concrete and Composite Frames

Experimental data of reinforced concrete, prestressed concrete frames are shown in Refs. (32, 36, 50, 52, 69, 82, 102, 105), and the hysteretic characteristics and energy dissipation of these frames are described.

It is expected that the hysteretic characteristics of reinforced concrete, prestressed concrete and steel reinforced concrete frames can be theoretically obtained when the hysteretic behaviors of structural elements such as members, connections and shear walls become known. However, the facts that the unstable portion of the stress-strain relationship of concrete shown in Fig. 8 makes it difficult to idealize the moment-curvature relationship of concrete by, for example, the Ramberg-Osgood method, and that the mechanism of shear failure has not yet been clarified leave the theoretical analysis method determining hysteretic behavior of these frames unknown. A practical method to obtain the restoring force characteristic is described in Ref. (86).

A few experimental investigations have been reported on the behavior of frames that involve shear failing columns(52, 87), but no theoretical works have been carried out.

4. CONCLUSION

Examples of referential subjects for the symposium discussion are as follows.

1. More data are necessary on the amount of the equivalent viscous damping. Also necessary are the data not only for the damping behaviors of various types of bare structures but also for the contribution from cladding to the damping. In order to evaluate the damping accurately that is usually handled as an equivalent damping, each contribution from viscoelastic damping, internal friction, plastic deformation and energy radiating into the ground should be separately investigated and evaluated.
2. Data for the hysteretic damping behavior of various kinds of structures are needed.
3. Extensive investigations on hysteretic characteristics of following materials, members and frames are needed.
   a. Materials, especially concrete, not only under uni-axial but also bi-axial and tri-axial states of stress.
   b. Steel beam-columns and steel beams that may fail by out-of-plane as well as in-plane buckling.
   c. Steel connections made by various kinds of fastening systems.
   d. Steel braces where the yielding due to tension and buckling due to compression are alternately repeated.
   e. Reinforced concrete, prestressed concrete and composite beams, beam-columns and connections. The hysteretic behavior of these structural elements should be investigated especially when they are expected to fail in shear.
   f. Reinforced concrete and composite shear walls.
   g. Steel braced and unbraced frames.
   h. Reinforced concrete, prestressed concrete and composite overall structures.
4. It is necessary from the viewpoint of aseismic design to develop methods to enable theoretically composing the hysteretic characteristics of overall structures starting from the local hysteretic behaviors under multi-axial state of stress.
5. The failure criteria for structures, considering the deterioration of structures due to low cycle fatigue, and the deterioration and transition of hysteresis loops, should be investigated, since safety of the structures against the earthquake can be discussed only when the results of response analysis are examined based on the failure criteria.

5. ACKNOWLEDGEMENT

The author wishes to express his sincere appreciation for the advice, comments and criticisms he received from Professors Ryoichiro Minai, Vitelmo V. Bertero and Tsuneyoshi Nakamura. Acknowledgements are also due his colleques, Messers Takeshi Nakamura and Shosuke Morino for the discussions and suggestions that were given when the manuscript was prepared.

6. BIBLIOGRAPHIC REFERENCES

1. Masing, G.:EIGENSPANNUNGEN UND VERFESTIGUNG BEIM MESSING, Proc, 2nd International Congress for Applied Mechanics, Zurich, 1926.

2. Jacobsen, L. S.:STEADY FORCED VIBRATION AS INFLUENCED BY DAMPING, Trans. ASME, Vol. 52, No. 22, Part 1, 1930, p. 169.

3. Sezawa, K. and Kanai, K.:VIBRATION OF BUILDING WITH DAMPING, Bull. Earthquake Research Institute, Tokyo University, Vol. XII, Part 4, Dec. 1934.

4. Tanabashi, R.:TESTS TO DETERMINE THE BEHAVIOR OF RIVETED JOINTS OF STEEL STRUCTURES UNDER ALTERNATE BENDING MOMENTS, Memoirs of the College of Engineering, Kyoto Imperial University, Vol. VIII, No. 4, 1935, p. 164.

5. Reissner, E.:STATIONARE AXIALSYMMETRISCHE, DURCH EINE SCHUTTELNDE MASSE ERREGTE SCHWINGUNGEN EINES HOMOGENEN ELASTISCHEN HALBRAUMS, Ingenieur-Archiv, Vol. VII. 1936, p. 381.

6. Tanabashi, R.:ON THE RESISTANCE OF STRUCTURES TO EARTHQUAKE SHOCKS, Memoirs of the College of Engineering, Kyoto University, Vol. IX, No. 4, 1937. p. 191.

7. Ramberg, W. and Osgood, W. R.:DESCRIPTION OF STRESS-STRAIN CURVES BY THREE PARAMETERS, NACA TN 902, July, 1943.

8. Arnold, R. N., Bycroft, G. N. and Waeburton, G. B.:FORCED VIBRATIONS OF A BODY ON AN INFINITE ELASTIC SOLID, J. Applied Mechanics, Vol. 22, Sept. 1955, p. 391.

9. Tanabashi, R., Kobori, T., Kaneta, K. and Muguruma, H.:ULTIMATE DEFLECTION DESIGN(PART 1 CONSIDERATION FROM THE VIEWPOINT OF EARTHQUAKE RESISTANT STRUCTURE), Proc. Symposium on Safety of Structure, Sept. 1955.

10. Toriumi, I.:VIBRATIONS IN FOUNDATIONS OF MACHINES, Technology Report of the Osaka University, Vol. 5, No. 146, 1955, p. 103.

11. Bycroft, G. N.:FORCED VIBRATIONS OF A RIGID CIRCULAR PLATE ON A SEMI-INFINITE ELASTIC SPACE AND ON AN ELASTIC STRATUM, Phylosophical Transactions of the Royal Society of London, Vol. 248, Series A, 1956, p. 327.

12. Hisada, T. and Nakagawa, K.:VIBRATIONS OF BUILDINGS IN JAPAN PART II, VIBRATION TESTS ON VARIOUS TYPES OF BUILDING STRUCTURES UP TO FAILURE, Proc. 1st World Conference on Earthquake Engineering, 1956, p. 7II-1.

13. Tanabashi, R.:STUDIES ON THE NONLINEAR VIBRATIONS OF STRUCTURES SUBJECTED TO DESTRUCTIVE EARTHQUAKES, Proc. 1st World Conference on Earthquake Engineering, 1956, p. 6-1.

14. Tajimi, H.:BASIC THEORIES ON ASEISMIC DESIGN OF STRUCTURES, Report of the Institute of Industrial Science, University of Tokyo, Vol. 8, No. 4, 1959, p. 170.

15. Ando, N.:NONLINEAR VIBRATIONS OF BUILDING STRUCTURES, Proc. 2nd World Conference on Earthquake Engineering, Vol. II, 1960, p. 1045.

16. Blume, J. A.:A RESERVE ENERGY TECHNIQUE FOR THE EARTHQUAKE DESIGN AND RATING OF STRUCTURES IN THE INELASTIC RANGE, Proc. 2nd World Conference on Earthquake Engineering, Vol. II, 1960, p. 1061.

17. Blume, J. A. and Meehan, J. F.:A STRUCTURAL-DYNAMIC RESEARCH PROGRAM ON ACTUAL SCHOOL BUILDINGS, Proc. 2nd World Conference on Earthquake Engineering, Vol. II, 1960, p. 1297.

18. Berg, G. V. and Dadeppo, D. A.:DYNAMIC ANALYSIS OF ELASTO-PLASTIC STRUCTURES Proc. ASCE, Vol. 86, EM2, Apr. 1960.

19. Caughey, T. K.:SINUSOIDAL EXCITATION OF A SYSTEM WITH BILINEAR HYSTERESIS, J. Applied Mechanics, Dec. 1960, p. 640.

20. Igarashi, S. and Taga, N.:HYSTERETIC CHARACTERISTICS AND STRUCTURAL DAMPING OF STEEL STRUCTURES UNDER ALTERNATE LATERAL LOADING, Trans. Architectural Institute of Japan, No. 120, Feb. 1966, p. 15.

21. Jacobsen L. S.:DAMPING IN COMPOSITE STRUCTURES, Proc. 2nd World Conference on Earthquake Engineering, Vol. II, 1960, p. 1029.

22. Kubo, K.:ASEISMICITY OF SUSPENSION BRIDGES FORCED TO VIBRATE LONGITUDINALLY, Proc. 2nd World Conference on Earthquake Engineering, Vol. II, 1960, p. 913.

23. Response Analyzer Committee:NON-LINEAR RESPONSE ANALYZERS AND APPLICATION TO EARTHQUAKE RESISTANT DESIGN, 2nd World Conference on Earthquake Engineering, Vol. II, 1960, p. 649.

24. Veletsos, A. S. and Newmark, N. M.:EFFECT OF INELASTIC BEHAVIOR ON THE RESPONSE OF SIMPLE SYSTEMS TO EARTHQUAKE MOTIONS, Proc. 2nd World Conference on Earthquake Engineering, Vol. II, 1960, p. 895.

25. Housner, G. W.:VIBRATION OF STRUCTURES INDUCED BY SEISMIC WAVES, Shock and Vibration Handbook, Vol. III, McGrow-Hill, 1961, p. 50-1.

26. Hisada, T., Nakagawa, K. and Izumi, M:EARTHQUAKE RESPONSE OF STRUCTURES HAVING VARIOUS RESTORING FORCE CHARACTERISTICS, Proc. Japan National Symposium on Earthquake Engineering, 1962, p. 63.

27. Thomson, W. T. and Kobori, T.:DYNAMICAL COMPLIANCE OF RECTANGULAR FOUNDATION ON AN INFINITE HALF-SPACE, Dept. of Engineering Report No. 62-9, University of California, 1962, p. 1.

28. Tanabashi, R. and Kaneta, K.:ON THE RELATION BETWEEN THE RESTORING FORCE CHARACTERISTICS OF STRUCTURES AND THE PATTERN OF EARTHQUAKE GROUND MOTIONS, Proc. Japan National Symposium on Earthquake Engineering, 1962, p. 57.

29. Takeda, T.:IDEALIZATION OF THE PLASTIC DEFORMATION OF REINFORCED CONCRETE BEAMS SUBJECTED TO THE ALTERNATE LOADING, Trans. Architectural Institute of Japan, Vol. 76, Sept. 1962, p. 98.

30. American Society for Testing and Materials:INTERNAL FRICTION, DAMPING, AND CYCLIC PLASTICITY, ASTM Special Technical Publication No. 378, 1964.

31. Aoyama, H.:MOMENT-CURVATURE CHARACTERISTICS OF REINFORCED CONCRETE MEMBERS SUBJECTED TO AXIAL LOAD AND REVERSAL BENDING, Proc. International Symposium of Flexural Mechanics of Concrete, 1964, p. 183.

32. Bertero, V. and McClure, G.:BEHAVIOR OF REINFORCED CONCRETE FRAMES SUBJECTED TO REPEATED REVERSIBLE LOADS, J. ACI, Vol. 61, Oct. 1964, p. 1305.

33. Jennings, P. C.:PERIODIC RESPONSE OF A GENERAL YIELDING STRUCTURE, Proc.ASCE, Vol. 90, EM2, Apr. 1964, p. 131.

34. Kobori, T. and Minai, R.:ASEISMIC DESIGN METHOD OF ELASTO-PLASTIC BUILDING STRUCTURES, Bulletin, Disaster Prevention Research Institute, Kyoto University, No. 68, Mar. 1964, p. 1.

35. Medearis, K. and Young, D. H.:ENERGY ABSORPTION OF STRUCTURES UNDER CYCLIC LOADING, Proc. ASCE, ST1., Feb. 1964, p. 61.

36. Odaka, T. and Saito, K.:EXPERIMENTAL STUDY ON THE RESTORING FORCE CHARACTERISTICS IN THE REINFORCED CONCRETE STRUCTURAL PORTAL FRAMES, Trans. Architectural Institute of Japan, No. 106, Dec. 1964, p. 1.

37. Penzien, J.:DAMPING CHARACTERISTICS OF PRESTRESSED CONCRETE, J. ACI, Vol. 61, Sept. 1964, p. 1125.

38. Rosenbluth, E. and Herrera, I.:ON A KIND OF HYSTERETIC DAMPING, Proc. ASCE, Vol. 90, EM4, Aug. 1964, p. 37.

39. Sinha, B. P., Gerstle, K. H. and Tulin, L. G.:STRESS-STRAIN RELATIONS FOR CONCRETE UNDER CYCLIC LOADING, J. ACI, No. 61-12, Feb. 1964, p. 195.

40. Sinha, B. P., Gerstle, K. H. and Tulin, L. G.:RESPONSE OF SINGLY REINFORCED BEAMS TO CYCLIC LOADING, J. ACI, No. 61-56, Aug. 1964, p. 1021.

41. Agrawal, G. L., Tulin, L. G. and Gerstle, K. H.:RESPONSE OF DOUBLLY REINFORCED CONCRETE BEAMS TO CYCLIC LOADING, J. ACI, No. 62-51, July 1965, p. 823.

42. Berg, G. V.:A STUDY OF THE EARTHQUAKE RESPONSE OF INELASTIC SYSTEMS, Proc. 34th Annual Convention of the Structural Engineers Association of California, Colorado, Oct. 1965, p. 63.

43. Clough, R. W., Benuska, K. L. and Wilson, E. L.:INELASTIC EARTHQUAKE RESPONSE OF TALL BUILDINGS, Proc. 3rd World Conference on Earthquake Engineering, Vol. II, 1965, p. II-68.

44. Fujimoto, M. and Hagura, H.:RESEARCH ON THE ELASTO-PLASTIC ANALYSIS OF STEEL SECTION SUBJECTED TO ALTERNATIVE LOAD, Trans. Architectural Institute of Japan, Part 1, No. 117, Nov. 1965, p. 10, Part 2, No. 121, Mar. 1966, p. 14, Part 3, No. 122, Apr. 1966, p. 1, Part 4, No. 125, July 1966, p. 8, Part 5, No. 133, Mar. 1967, p. 18.

45. Hudson, D. E.:EQUIVALENT VISCOUS FRICTION FOR HYSTERETIC SYSTEMS WITH EARTHQUAKE-LIKE EXCITATIONS, Proc. 3rd World Conference on Earthquake Engineering, Vol. II, 1965, p. II-185.

46. Hatano, T.:VIBRATION OF VISCO-ELASTIC BODY, Proc. 3rd World Conference on Earthquake Engineering, Vol. II, 1965, p. II-126.

47. Ito, M. and Katayama, T.:DAMPING OF BRIDGE STRUCTURES, Proc. Japan Society of Civil Engineers, No. 117, May 1965, p. 12.

48. Iwan, W. D.:THE DYNAMIC RESPONSE OF THE ONE DEGREE OF FREEDOM BILINEAR HYSTERETIC SYSTEM, Proc. 3rd World Conference on Earthquake Engineering, Vol. II, 1965, p. II-783.

49. Jennings, P. C.:RESPONSE OF YIELDING STRUCTURES TO STATICALLY GENERATED GROUND MOTION, Proc. 3rd World Conference on Earthquake Engineering, Vol. II, 1965, p. II-237.

50. Nakano, K.:EXPERIMENT ON BEHAVIOUR OF PRESTRESSED CONCRETE FOUR STORIED MODEL STRUCTURE UNDER LATERAL FORCE, Proc. 3rd World Conference on Earthquake Engineering, Vol. III, 1965, p. IV-572.

51. Watanabe, S., Kida, Y. and Higuchi, M.:THE VIBRATION ANALYSIS OF A STEEL STRUCTURES:THE VIBRATIONAL TEST OF OHBAYASHI-GUMI BUILDING, Proc. 3rd World Conference on Earthquake Engineering, Vol. II, 1965, p. II-695.

52. Aoyama, H. Endo, T. and Minami, T.:BEHAVIOR OF REINFORCED CONCRETE FRAMES SUBJECTED TO REVERSAL OF HORIZONTAL FORCES, Proc. 2nd Japan Earthquake Engineering Symposium, 1966, p. 315.

53. Clough, R. W. and Johnston, S. B.:EFFECT OF STIFFNESS DEGRADATION ON EARTHQUAKE DUCTILITY REQUIREMENTS, Proc. 2nd Japan Earthquake Engineering Symposium, 1966, p. 227.

54. Igarashi, S., et al.:PLASTIC BEHAVIOR OF STEEL FRAMES UNDER CYCLIC LOADING, Trans. Architectural Institute of Japan, No. 130, Dec. 1966, p. 8.

55. Kobori, T., Minai, R. and Fujiwara, T.:RESPONSE OF FRAMES WITH ELASTO-PLASTIC JOINTS TO EARTHQUAKE, Proc. 2nd Japan Earthquake Engineering Symposium, 1966, p. 215.

56. Kobori, T., Minai, R. and Suzuki, T.:EARTHQUAKE RESPONSE OF FRAME STRUCTURE HAVING ELASTO-PLASTIC JOINTS, Annuals, Disaster Prevention Research Institute, Kyoto University, No. 9, Mar. 1966, p. 165, (in Japanese).

57. Muguruma, H., Tominaga, M. and Okamoto, S.:MECHANICAL PROPERTIES OF SIMPLY SUPPORTED PRESTRESSED CONCRETE BEAMS UNDER REPEATED LOADINGS, RILEM International Symposium on the Effect of Repeated Loading of Materials and Structural Elements, Mexico City, Vol. V, Sept. 1966.

58. Nielsen, N. N.:VIBRATION TESTS OF A NINE-STORY STEEL FRAME BUILDING, Proc. ASCE, Vol. 92, EM1, Feb. 1966, p. 81.

59. Popov, E. P.:LOW-CYCLE FATIGUE OF STEEL BEAM-TO-COLUMN CONNECTIONS, RILEM International Symposium on the Effect of Repeated Loading of Materials and Structural Elements, Mexico City, Vol. VI, Sept. 1966.

60. Umemura, H., et al.:FORCED VIBRATION EXPERIMENT ON THE MULTI-STORY STEEL FRAMES APARTMENT BUILDING AND ITS ANALYTICAL STUDY, Trans. Architectural Institute of Japan, No. 129, Nov. 1966, p. 1.

61. Yamada, M. and Nakaza, I.:LOW-CYCLE FATIGUE TO RIVETED AND HIGH STRENGTH BOLTED JOINTS IN STEEL BEAMS, RILEM International Symposium on the Effects of Repeated Loading of Materials and Structural Elements, Mexico City, Vol. VI, 1966.

62. Yamada, M. and Kawamura, H.:STUDY ON THE ELASTO-PLASTIC FLEXURAL BEHAVIOR OF REINFORCED CONCRETE MEMBERS SUBJECTED TO AXIAL LOAD (Part III), Trans. Architectural Institute of Japan, No. 136, June, 1967, p. 15.

63. Davenport, A. G.:GUST LOADING FACTORS, Proc. ASCE, ST3, June 1967, p. 11.

64. Jennings, P. C.:EQUIVALENT VISCOUS DAMPING FOR YIELDING STRUCTURES, Proc. ASCE, Vol. 94, EM1, Feb. 1968, p. 103.

65. Kaldjiam, M. J. and Fan, W. R. S.:EARTHQUAKE RESPONSE OF A RAMBERG-OSGOOD STRUCTURE, Proc. ASCE, ST10, Oct. 1968, p. 2451.

66. Lazan, B. J.:DAMPING OF MATERIALS AND MEMBERS IN STRUCTURAL MECHANICS, Pergamon Press, 1968.

67. Popov. E. P.:PERFORMANCE OF STEEL BEAMS AND THEIR CONNECTIONS TO COLUMNS DURING SEVERE CYCLIC LOADING, Contributions to the Expanded Discussion, 8th Congress of IABSE, ThemeIII, New York, Sept. 1968.

68. Baron, F. and Venkatesan, M. S.:INELASTIC RESPONSE FOR ARBITRARY HISTORIES OF LOADS, Proc. ASCE, EM3, June, 1969, p. 763.

69. Bertero, V. and Bresler, B.:SEISMIC BEHAVIOR OF REINFORCED CONCRETE FRAMED STRUCTURES, 4th World Conference on Earthquake Engineering, Vol. I, 1969, p. B-2.109.

70. Funabashi, I., Kinoshita, K. and Aoyama, H.:VIBRATION TESTS AND TESTS TO FAILURE OF A 7-STORIES BUILDING SURVIVED A SEVERE EARTHQUAKE, Proc. 4th World Conference on Earthquake Engineering, Vol. I, 1969. p. B-1 26.

71. Fujimoto, M., Segawa, T. and Matsumoto, Y.:ELASTIC-PLASTIC ANALYSIS OF A BRACED FRAME UNDER REPEATED LOADING, Proc. of Annual Meeting of Architectural Institute of Japan, Aug. 1969. p. 1213.

72. Goel, S. C.:P-Δ AND AXIAL COLUMN DEFORMATION IN ASEISMIC FRAMES, Proc. ASCE, ST8, Aug. 1969,

73. Hanson, R. D. and Fan, W. R. S.:THE EFFECT OF MINIMUM CROSS BRACING ON THE INELASTIC RESPONSE OF MULTI-STORY BUILDINGS, Proc. 4th World Conference on Earthquake Engineering, Vol. II, 1969, p. A-4 15.

74. Iwan, W. D.:THE DISTRIBUTED-ELEMENT CONCEPT OF HYSTERETIC MODELING AND ITS APPLICATION TO TRANSIENT RESPONSE PROBLEMS, Proc. 4th World Conference on Earthquake Engineering, Vol. II, 1969, p. A-4 45.

75. Karsan, I. D. and Jirsa, J. O.:BEHAVIOR OF CONCRETE UNDER COMPRESSIVE LOADINGS, Proc. ASCE, ST12, Dec. 1969, p. 2543.

76. Kato, B. and Akiyama, H.:THE ULTIMATE STRENGTH OF THE STEEL STRUCTURES SUBJECTED TO EARTHQUAKE, Proc. 4th World Conference on Earthquake Engineering, Vol. II, 1969, p. A-4 59.

77. Katayama, T.:DAMPING CHARACTERISTICS OF BUILDING STRUCTURES, Trans. Architectural Institute of Japan, No. 157, Mar. 1969, p. 11.

78. Kobori, T., Minai, R. and Fujiwara, T.:EARTHQUAKE RESPONSES OF A FRAME STRUCTURE HAVING ELASTO-PLASTIC JOINTS, Annuals, Disaster Prevention Research Institute, Kyoto University, No. 12A, Mar. 1969, p. 321.

79. Nielsen, N.:DYNAMIC RESPONSE ON A 90-FT STEEL FRAME TOWER, Proc. 4th World Conference on Earthquake Engineering, Vol. I, 1969, p. B-1 126.

80. Popov, E. P. and Pinkney, R. B.:RELIABILITY OF STEEL BEAM-TO-COLUMN CONNECTIONS UNDER CYCLIC LOADING, Proc. 4th World Conference on Earthquake Engineering, Vol. II, 1969, p. B-3 15.

81. Rea, D., Clough, R. W., Bouwkamp, J. G. and Vogel, U.:DAMPING CAPACITY OF A MODEL STEEL STRUCTURE, Proc. 4th World Conference on Earthquake Engineering, Vol. I, 1969, p. B-2 63.

82. Shiga, T. and Ogawa, J.:THE EXPERIMENTAL STUDY ON THE DYNAMIC BEHAVIOR OF REINFORCED CONCRETE FRAMES, Proc. 4th World Conference on Earthquake Engineering, Vol. I, 1969, p. B-2 165.

83. Spencer, R. A.:THE NONLINEAR RESPONSE OF A MULTISTORY PRESTRESSED CONCRETE STRUCTURE TO EARTHQUAKE EXCITATION, Proc. 4th World Conference on Earthquake Engineering, Vol. II, 1969, p. A-4 139.

84. Tamura, R., Murakami, M., Osawa, Y. et al.:VIBRATION TEST OF A LARGE MODEL STEEL FRAME WITH PRECAST CONCRETE PANEL UNTIL FAILURE, Proc. 4th World Conference on Earthquake Engineering, Vol. I, 1969, p. B-2 15.

85. The Group for Dynamic Tests of High-Rised Buildings :SUMMARIZED REPORT ON DYNAMIC TESTS OF HIGH-RISED BUILDINGS AND COOPERATIVE PLAN FOR LARGE-SCALE VIBRATION TESTS IN JAPAN, Proc. 4th World Conference on Earthquake Engineering, Vol. I, 1969, p. B-1 111.

86. Umemura, H. and Aoyama, H.:EVALUATION OF INELASTIC SEISMIC DEFLECTION OF REINFORCED CONCRETE FRAMES BASED ON THE TESTS OF MEMBERS, Proc. 4th World Conference on Earthquake Engineering, Vol. I, 1969, p. B-2 91.

87. Higashi, Y. and Ohkubo, M.:STATIC AND DYNAMIC LOADING TESTS OF REINFORCED CONCRETE FRAMES WITH THIN SPANDREL OR WING WALLS, Proc. U. S.- Japan Seminar on Earthquake Engineering, 1970, p. 225.

88. Igarashi, S., et al.:INELASTIC BEHAVIOR OF STRUCTURAL STEEL SECTIONS UNDER ALTERNATIVE LOADING, Trans. Architectural Institute of Japan, No. 170 Apr. 1970, p. 39.

89. Kobori, T., Minai, R. and Fujiwara, T.:TORSIONAL RESPONSE OF FRAMED STRUCTURES CONSIDERING A YIELD SURFACE IN ELASTO-PLASTIC JOINTS, Annuals, Disaster Prevention Research Institute, Kyoto University, No. 13A, Mar. 1970, p. 287.

90. Kobori, T. and Setogawa, S.:NUMERICAL ANALYSIS ON FOUNDATION VIBRATION BY FINITE ELEMENT TECHNIQUE, Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 501.

91. Kobori, T. and Suzuki, T.:FOUNDATION VIBRATIONS ON A VISCOELASTIC MULTI-LAYERED MEDIUM, Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 493.

92. Kobori, T., Minai, R. and Inoue, Y.:ON THE EARTHQUAKE RESPONSE OF STRUCTURAL SYSTEMS CONSIDERING THE INTERACTION EFFECTS OF THE GROUND, Bull. of Disaster Prevention Research Institute, Kyoto University, Vol. 20, Part 2, Dec. 1970, p. 105.

93. Minai, R.:ON THE ASEISMIC SAFETY OF BUILDING STRUCTURES, Annuals, Disaster Prevention Research Institute, Kyoto University, No. 13A, Mar. 1970, p. 5.

94. Minai, R. and Suzuki, T.:RELIABILITY ANALYSIS OF ASEISMIC SAFETY OF ELASTO-PLASTIC STRUCTURES CONSIDERING RANDOM FATIGUE, Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 723.

95. Minai, R. and Inoue, Y.:ON EARTHQUAKE RESPONSE OF GROUND-STRUCTURE SYSTEMS, Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 41.

96. Murakami, M., Nishikawa, T., Aoyama, H. and Osawa, Y.:A STUDY ON THE CAUSE OF DAMAGE TO THE HACHINOHE TECHNICAL COLLEGE DUE TO 1968 TOKACHI-OKI EARTHQUAKE (Part 2), Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 803.

97. Nielsen, N. N. and Imbeault, F. A.:VALIDITY OF VARIOUS HYSTERETIC SYSTEMS, Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 707.

98. Omote, Y., Takeda, T., Moritaka, I. and Yoshioka, K.:EXPERIMENTAL STUDIES ON RESPONSE OF STRUCTURES TO SHOCK WAVE-PART 1 REINFORCED CONCRETE STRUCTURES, Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 579.

99. Shiga, T., Ogawa, J., Shibata, A. and Shibuya, J.:THE DYNAMIC PROPERTIES OF REINFORCED CONCRETE FRAMES, Proc. U. S.- Japan Seminar on Earthquake Engineering, 1970, p. 346.

100. Tanabashi, R., Yokoo, Y., Nakamura, T. et al.:LOAD-DEFLECTION BEHAVIORS AND PLASTIC FATIGUE OF WIDE-FLANGE BEAMS SUBJECTED TO ALTERNATING PLASTIC BENDING, Trans. Architectural Institute of Japan, Part I, No. 175, Sept. 1970, p. 17, Part II, No. 176, Oct. 1970, p. 25, Part III, No. 177, Nov. 1970, p. 35.

101. Takeda, T., Sozen, M. A. and Nielsen, N. N.:REINFORCED CONCRETE RESPONSE TO SIMULATED EARTHQUAKES, Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 357.

102. Tani, S., Nomura, S., Nagasaka, T. and Hiramatsu, A.:STUDY ON RESTORING FORCE CHARAVTERISTICS OF REINFORCED CONCRETE STRUCTURES(STATIC ANALYSIS) Proc. 3rd Japan Earthquake Engineering Symposium, 1970, p. 691.

103. Wakabayashi, M.:THE BEHAVIOR OF STEEL FRAMES WITH DIAGONAL BRACINGS UNDER REPEATED LOADING, Proc. U. S. Japan Seminar on Earthquake Engineering 1970, p. 328.

104. Wakabayashi, M., Matsui, C., Minami, K. and Mitani, I.:INELASTIC BEHAVIOR OF FULL SCALE STEEL FRAMES, Annuals, Disaster Prevention Research Institute, Kyoto University, No. 13A, Mar. 1970, p. 329.

105. Blakeley, R. W. G. and Park, R.:SEISMIC RESISTANCE OF PRESTRESSED CONCRETE BEAM-COLUMN ASSEMBLES, J. ACI, Vol. 68, Sept. 1971, p. 677.

106. Brown, R. H. and Jirsa, J. O.:REINFORCED CONCRETE BEAMS UNDER LOAD REVERSALS, J. ACI, Vol. 68, May 1971, p. 380.

107. Kobori, T., Minai, R. and Fujiwara, T.:RESPONSE ANALYSIS OF ELASTO-PLASTIC FRAME STRUCTURE WITH RAMBERG-OSGOOD CHARACTERISTICS, Annuals, Disaster Prevention Research Institute, Kyoto University, No. 14A, Apr. 1971, p. 301.

108. Krawinkler, H., Bertero, V. and Popov, E. P.:INELASTIC BEHAVIOR OF STEEL BEAM-to-COLUMN SUBASSEMBLAGES, Earthquake Engineering Research Center Report No. EERC 71-7, University of California, Berkely, Oct. 1971,

109. Tanabashi, R., Yokoo, Y., Wakabayashi, M., Nakamura, T. and Kunieda, H.: DEFORMATION-HISTORY DEPENDANT INELASTIC STABILITY OF COLUMNS SUBJECTED TO ALTERNATING LOADING, Preprint for RILEM Symposium on Experimental Analysis of Instability Problems on Reduced and Full-Scale Models, Buenos Aires, Sept. 1971.

110. Wakabayashi, M. and Yamaguchi, T.:ELASTO-PLASTIC BEHAVIOR OF STEEL REINFORCED CONCRETE MEMBERS, Trans. Architectural Institute of Japan, Extra, Nov. 1971, p. 777.

111. Yamada, M., et al.:ELASTO-PLASTISCHE BIEGEFORMANDERUNGEN VON STAHLSTUTZEN MIT I-QUERSCHNITT, Der Stahlbau, Vol. 40, Ht. 3, Mar. 1971, p. 65.

112. Naka, T., Wakabayashi, M. and Kato, B.:ELASTO-PLASTIC BEHAVIOR OF ENCASED STRUCTURES, Invited Discussion to Symposium on Planning and Design Tall Buildings, ASCE-IABSE Joint Committee, 1972.

113. Wakabayashi, M.:FRAMES UNDER STRONG IMPULSIVE, WIND OR SEISMIC LOADING, State of Art Report to Symposium on Planning and Design of Tall Buildings, ASCE-IABSE Joint Committee, 1972.

SUMMARY

The significance of damping and energy absorption of structures and structural elements under earthquake excitation is first described, and then the details of these phenomena are discussed in relation to the equivalent viscous damping coefficient. Several mathematical models representing the hysteretic behavior of materials, members, connections and frames are introduced. The state-of-art is outlined for each of these subjects, referring to the recent publications. Finally, some important problems are indicated for the symposium discussions.

RESUME

La capacité d'absorption d'énergie et les facultés d'amortissement de constructions sous l'effet de tremblements de terre sont définies, puis ces phénomènes sont analysés en détail en fonction du coefficient d'amortissement visqueux équivalent. Plusieurs modèles mathématiques représentant le comportement hystérèse de matériaux, d'éléments, d'assemblages et de structures sont présentés. L'état des connaissances actuelles est précisé pour chacun de ces sujets, en se référant aux publications récentes. D'importants problèmes sont enfin suggérés pour les discussions du colloque.

ZUSAMMENFASSUNG

Zunächst wird die Bedeutung der Dämpfung und Energieabsorption an Bauwerken und Bauelementen unter Erdbeben-Erregung beschrieben; sodann werden die Einzelheiten dieser Phänomene in ihrer Beziehung zum äquivalenten viskosen Dämpfungskoeffizienten diskutiert. Dabei gelangen eine Reihe mathematischer Modelle in Form des Hystereseverhaltens der Materialien, der Elemente, Verbindungen und Bauwerke zur Darstellung. Der gegenwärtige technische Stand wird für jeden dieser Gegenstände, unter Bezugnahme auf die neuesten Veröffentlichungen umrissen. Schliesslich wird auf einige wichtige Probleme zur Symposiums-Diskussion hingewiesen.