Passive damping technology for buildings in Japan

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Summary

This paper first reviews the concept of the damage-controlled structure (DCS) which is a kind of passive damping technology, proposed before the Hyogoken-Nanbu Earthquake in Japan and the Northridge Earthquake in the USA. The philosophy, the necessity and the potential of the damage-controlled structure are stated in the first two sections of this paper. Second, a modified shear-bending beam model and a rational dynamic analysis method of three-dimensional frame for the damage-controlled structure with passive energy dissipation devices are reviewed. Thirdly, a series of dynamic loading test results of modeled damage-controlled steel frame with hysteretic dampers are presented. Finally, a number of actual example building projects which exemplify the current seismic design trend using the passive damping technology in Japan are reviewed.

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The Loma Prieta earthquake in the USA in 1989 highlighted the need to restore as soon as possible the inherent functions of buildings. In earthquake-resistant design of building structures, priority is of course given to protection of human lives against extremely infrequent large earthquakes. However, the above need arises from the large impact of economic loss associated with the stoppage of economic activities. This is especially so where the damage occurs in large cities. This problem has been considered more seriously by many people not only in the USA but also in Japan since the Northridge Earthquake in 1994 and the Hyogoken-Nanbu Earthquake in the subsequent year.

Building damage due to earthquakes is classified according to the damage magnitude from light to heavy as follows: negligible, light, medium, large and complete collapse. From the building user's viewpoint, the middle three categories correspond to continuous function, asset integrity and human-life preservation. More than 100 years have passed since the Nobi Earthquake, and nearly 100 years have passed since the San Francisco Earthquake at the beginning of the 20th century. Since then, a lot of research has been carried out in the area of earthquake-resistant design of building structures. As a result, the same concept has been employed in Japan and the USA in design against extremely large earthquakes. Based on this concept, structures are designed to behave plastically to utilize their ability to absorb energy. The structure is not intended to result in complete collapse, but is based on planned damage to building structures. It includes the concept of human-life protection, but does not consider asset integrity and continuous function.

Needless to say, the standard of earthquake resistance of building structures is determined by the economic and technological strength of the country. In this regard, the earthquake resistance cannot be considered as an absolute value. In Japan, the minimum requirement is specified in Kenchiku Kijun-ho (standard law of architecture). However, this law does not provide sufficient requirements. It is also necessary to establish earthquake-resistant design methods that incorporate repairability after a large earthquake. These needs arise when considering city-level earthquake resistance and continuity of the city's economic activities. The conventional concept can be applied to extremely infrequent large earthquakes that can be classified as natural disasters. The following concept is effective for large earthquakes assumed by the conventional design. The behaviors of the main structural elements such as columns and beams are suppressed within the elastic range. Damage is suppressed by elasto-plastic dampers, mounted in parallel with the main structural elements, aimed at energy absorption.

Abbreviations
DTS = damage-tolerant structures
SSD = slit steel dampers
DCT = damage-controlled structures

This paper first reviews the concept of the damage-controlled structure (DCS) which is a kind of passive damping technology, proposed before the Hyogoken-Nanbu Earthquake in Japan and the Northridge Earthquake in the USA. The philosophy, necessity and the potential of the damage controlled structure are stated in the first two sections of this paper. Secondly, a modified shear-bending beam model and a rational dynamic analysis method of three-dimensional frame for the damage-controlled structure with passive energy dissipation devices reviewed. Thirdly, a series of dynamic loading test results of modeled damage-controlled steel frame with hysteretic dampers are presented. Finally, a number of actual example projects which exemplify the current seismic design trend using the passive damping technology in Japan are reviewed.

Philosophy of damage-controlled structures

Since the spring of 1992, the concept of damage-tolerant structures (DTS) or damage-controlled structures (DCS) has been proposed[1]. Since the Northridge Earthquake, passive damping technologies have been increasingly taken into attention in the USA[2–6]. Similarly in Japan, buildings have been increasingly designed with dampers since the Hyogoken-Nanbu Earthquake[7]. The word 'Sacrifice' was used in the short explanatory notes that appeared together with a conceptual picture of a damped structure on the cover of the engineering news records (ENR) in the summer of 1997. These notes explained that energy absorption by axial yielding of the damping brace became the sacrifice. Thus, high-rise building structures were saved even under a large earthquake. The current law of earthquake-resistant design was promulgated in June 1981. This law specified the structural characteristic factor D_s relating to the calculation of horizontal load-carrying capacity. A larger D_s was required for buildings with a larger number of earthquake resisting walls or braces. This gives preference to pure framed structures. For pure framed structures, the sacrifice becomes the flange welded part of the beam ends for steel structures. The Northridge Earthquake and the Hyogoken-Nanbu Earthquake clearly demonstrated that little energy absorption could be expected from plastic deformation of beam ends at the time of an earthquake. This is because that the plastic deformation of beam ends is equivalent to the method of mounting elasto-plastic dampers in series in a part of an elastic frame, leading to large deformation of the whole frame after it becomes plastic.

The number of buildings in the Northridge Earthquake in which the beam ends became plastic has been increasing as the investigation has proceeded. According to information from engineers in Los Angeles, the number had exceeded 200 in the summer of 1998[8–11]. Furthermore, in experimental research to find ways of avoiding this phenomenon, there appear to be problems that cannot be made public. In the Hyogoken-Nanbu Earthquake in Japan, the number of buildings having fractures at beam ends is reported to be about 40[12,13]. However, there seems to be an unwillingness to pursue a study to clarify the problem for the next generation. No comprehensive inspection seems to have been carried out. In both the USA and Japan, most of the problematic buildings were of pure framed structure containing almost no braces. It has often been reported that earthquake-resisting walls play an important role in earthquake resistance even in reinforced concrete buildings. Although it cannot be said that the braces and earthquake-resisting walls have been added for a definite purpose, they are a kind of 'Sacrifice' in earthquake-resistant design.

Studies carried out in the 1970s would probably have determined the direction of policy, and given preference to the moment resistant framed structures without braces[14–16]. There seems to have been an aversion to rapidly decreasing strength due to the brace buckling and shear failure of earthquake resisting walls. Braces and earthquake-resisting walls provide excellent earthquake resistance, but they have somewhat troublesome aspects in terms of re-utilization of buildings after an earthquake because of buckling and cracking. Thus, future earthquake-resistant design is directed toward determining a lucid method of placing damping materials at the correct locations, aimed at energy absorption during an earthquake.

Torahiko Terada wrote an essay entitled 'Clavicle' in the Kuramae Newspaper in 1933. It started with the sentence: 'A child was injured after falling down the stairs', and was followed by a description of man's structure and mechanism. 'It is said that the 'clavicle' is designed to break in such a case. It plays the role of a safety valve. Because it preferentially breaks, more important parts are saved by its sacrifice'. The essay continued with: 'My layman's thinking led to the idea, why not design and construct a 'clavicle of the house''. We could design this part to break if a large earthquake occurs, and thus save the other vital parts. This would be the original concept of a damage control structure.

In 1933, which was the 10th year after the Great Kanto Earthquake, Heigaku Tanabe published 'Taishin Kenchiku Mondou (questions and answers on earthquake-resistant architecture)'. This book is famous for its idea of constructing seismic isolation systems, and claims that it is better not to tightly fix the sill and the foundation. It also mentions the usefulness of the brace. The preface says: 'For *Shinkabe* (wall with exposed columns), the architecture is not as freely designed in terms of structure and appearance as western-style architecture when using braces, since the columns are exposed to the wall surface'. Then, it says 'In earthquake countries like ours, it is rather natural to lay a bare thick brace and to leave the wall divided into triangles as it is. Rectangular frames made up of only columns and beams in the conventional way are unreasonable and unnatural'.

Many kinds of damping members have been developed. Some use elasto-plastic deformation, some use viscous materials, and some use viscoelastic materials. They are mounted in a frame to resist story deformation of the buildings.

Here, let us consider a brace placed at 45° in a frame consisting of columns and beams shown in Fig. 1. Compared with shear deformation occurring in the frame, the expansion and contraction of the brace becomes $1/\sqrt{2}$. Because the brace length is $\sqrt{2}$ times the column length, the axial strain in the brace becomes 1/2 of the shear deformation angle of the frame. Taking into account the fact that the joints at the brace ends are of high rigidity and strength and are thus not made plastic, the findings are as follows. After converting the yield strain of the steel member to be a little higher than 0.1%, it is found that the story deformation angle is as small as 1/500 when the brace yields.

For a steel plate shear wall, the shear yield strain is about 1.5 (= 2.6/1.732) times the axial yield strain, since the yield shear stress is $1/\sqrt{3}$ time the yield stress and the shear elastic modulus is 1/2.6 times Young's modulus. It is found that story deformation angle becomes about 1/600 and it is made plastic at almost the same level as the small story deformation angle for the brace.

It is possible to make the story deformation angle at the beginning of plastic deformation smaller for the brace and steel plate wall by using ultralow yield steel to locally concentrate the plastic-deforming part, etc.

Let us consider the yield deformation angle of a rigid joint frame comprised of columns and beams. For the steel structure frame, discussion is focused on the rotational distortion occurring at the beam ends, which receive an asymmetric bending moment. This is because the bending deflection in the beam comprises nearly half of the structure deformation. Let the span and the depth of the beam be L and D, respectively. Then, the deformation angle for the stress of the flange at the beam ends to reach the yield point σ_y becomes $(\sigma_y/3E)(L/D)$. The span L of the frame is predetermined and Young's modulus E is constant. Therefore, it is found that the deformation at the yield point of the frame can be increased by using steel having a high yield point σ_y and members having a smaller depth D than conventional ones.

As discussed above, the following conclusions can be drawn. The yield deformation of the moment resistant frame can be easily determined by selecting the materials and cross sections. On the contrary, the yield deformations of the damping components such as braces and shear walls are determined from the

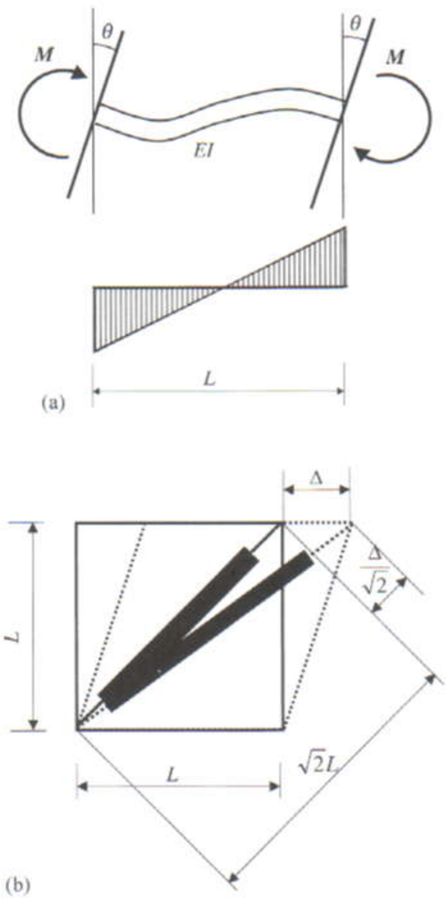


Fig. 1 Deformation of a frame structure: (a) deformation angle at beam ends of a rigid frame; (b) axial deformations of brace and shear deformation of frame

overall configuration and the material selection. Thus, yield deformation can not be changed by adjusting the plate thickness and the local configuration. When the damping materials are combined with the moment resisting frame, neglecting problems such as brace buckling, shear failure of shear wall, etc., the following difficulties occur. Deformation where strength takes effect comes earlier than that in a moment resisting frame, so strength deterioration occurs thereafter. Therefore, it is difficult to rationally add both of the resistant strengths.

Research and development have increased the deformation capability of damping components that yield early. A buckling confining brace has thus been developed. For the steel shear wall, research has been carried out to find a rib installation method to prevent strength decrease due to buckling. These methods make it possible to assemble a parallel structure of an elastic frame and an energy absorbing frame. The brace and wall-shaped damping components have an overwhelmingly higher ratio of resistance against horizontal force to consumed steel volume than the moment resisting frame structure. This means a large increase in the economic viability of the damage-control structure.

A lot of attention has been paid to structure life in relation to global environmental issues. In design, efforts tend to be made to establish the best conditions at the time of completion. However, when considering the structure use for periods from a few tens of years to a few hundreds of years, the optimum solution at completion would not be that for the whole life of the building. Taking the difference between the lives of the interior finishing, individual facilities such as air conditioning, etc., into account, a design enabling partial renewal becomes necessary. By taking this necessity into account in early years, designs have already been put into practice that separate the skeleton and infill, and that clearly separate the space occupied by the facility piping from that occupied by the structure. It is also desirable to employ the above concept in earthquake-resistant design. Conventional structures are based on the other way of thinking in terms of the addition of plastic deformation capability of the columns and beams. An attempt was made in the design of conventional structures to give columns and beams the role of resisting earthquake loads in addition to their primary role of supporting vertical load. However, the problems of this method were made evident by the damage to rigid pure framed structures in the Northridge and the Hyogoken-Nanbu earthquakes. The ideal damping structures refer to the method where the frame supporting the vertical load guarantees elastic behavior even in an earthquake and energy absorption during the earthquake is expected at the damping members only. This design method has the characteristic that the vertical-load-supporting mechanism has differing requirements and functions which are clearly separated from the earthquake-energy-absorption mechanism.

Examination of administration should be undertaken when the building is built not only in Japan but also in other countries. As discussed above, the damping structure is more rational than the conventional structure from the standpoint of earthquake-resistant performance and cost. The seismic isolation structure is similarly evaluated. However, it is unfortunate that they are treated as special structures just because the current law does not include them, even though they are the most rational structures. For damping structures, the following problems arise. Special steel materials and viscoelastic materials are employed as damping members. While the steel building structure has to be based on allowable stress design in the primary design determined in the Kijun-ho Shiko-rei (standard law enforcing rule), the damper yields to this level of earthquake force.

Since the Kenchiku Kijun-ho is soon to be revised, the degree of freedom is expected to be increased. It is thus desired to introduce an environment that will popularize such damping structures and seismic isolation structures. On this occasion, the judging also needs to have knowledge of this new technology. Since it is said that the judging will also be open to the private sector, this can be expected.

To apply a damping structure to an actual project, a wall-shaped space needs to be provided in a building in order to mount the damping members. Buildings cannot be made only with floors, columns and windows, walls are also necessary. In this regard, architects will need to have an understanding of this.

In addition, construction activities raise new environmental problems such as ruining rain forests and increasing CO_2 by requiring the production of cement and steel. These problems can be reduced by lengthening the building's life span. It must be meaningful that large buildings can still be used after large earthquakes instead of reconstructing them. The concept of damage-controlled structure with passive damping technology is a good solution to this problem.

The technology of DCS is theoretically based on Connor's 'Performance-based design'[17,18]. This concept suggests that a more rational design can be achieved if each design parameter individually corresponds to each design requirement. The design strategy of DCS is composed of two independent requirements. One is to design the primary structure resisting only the vertical load, another is to design the passive damping devices resisting the earthquake load.

Dynamic analysis method for DCS

The procedure of a practical structural design for a project usually includes three design phases as follows: conceptual design, preliminary structural design and final detailed design.

Selection of structural system, plan layout and configuration of the structure, as well as the structural design criteria are usually determined in the conceptual design stage. In the stage of preliminary structural design, distribution of the global shear stiffness and bending stiffness throughout the building height, the inter-story deformation criteria and response acceleration requirement, as well as the energy equilibrium which means that the energy supply of the structure should be greater than the energy demand due to the earthquake should be considered. It is also very important to do the preliminary structural design to achieve the economical and rational design solution. Simplification of the complicated actual building structure into a simple shear bending beam model is usually very effective in this design stage.

In order to verify the strength requirement of each structural member, to consider the interaction effects of various members, to check the performance of each damper, dynamic response analysis using three dimensional frame model is usually necessary during this design stage.

In this section, a simplified shear-bending model[19,20] for the preliminarily design and a rational dynamic analysis method of a three dimensional frame structure[21] for the detailed member design are reviewed as follows.

MODIFIED SHEAR-BENDING BEAM MODEL

In the conventional shear-bending beam model, the components of shear and bending deformation contributing to the lateral translational deformation at each lumped mass are considered in total. The influence of shear force on the bending moment is considered by introducing a parameter that is related to the ratio of bending stiffness to shear stiffness. The static force equilibrium equation is written in the form of eq. (1).

$$\mathbf{F}_i = \mathbf{K}_i\mathbf{U}_i \tag{1}$$

where $\mathbf{F}_i$ is a force vector including the shear forces and bending moments at the two ends of ith beam element; $\mathbf{K}_i$ is the stiffness matrix including the components of shear and bending stiffness of ith beam element; $\mathbf{U}_i$ is a deformation vector including the lateral translational deformation (u_i, u_{i+1}) and rotational deformation (θ_i, θ_{i+1}) at two end nodes of ith beam element.

However, various dampers are effective only in the direction of shear deformation. It is very difficult for a conventional shear bending beam model to consider the use of energy dissipation devices in the global structure system.

A modified shear bending beam model shown in Fig. 2 has been proposed[19,20]. In this model, the lateral translational deformation divided into independent shear and bending components is expressed in the form

$$u_{i+1} = u_i + \delta_{si} + h_i(\theta_i + \theta_{i+1})/2 \tag{2}$$

where u_i and u_{i+1} are the translational deformation at ith and $(i+1)$th lumped mass; δ_{si} is the shear deformation between ith and $(i+1)$th lumped masses subtracted by the rotation due to the bending deformation; θ_i and θ_{i+1} are the rotational deformation at ith and $(i+1)$th lumped masses; h_i is the inter-story height between ith and $(i+1)$th lumped masses.

Therefore, the shear force and bending moment of ith beam element are expressed as

$$Q_i = k_{si}\delta_{si}$$
$$B_i = k_{bi}\phi_i \tag{3}$$

where Q_i and B_i are the shear force and bending moment acting at the center of ith beam element; k_{si} and k_{bi} are the shear and bending stiffness of ith beam element; ϕ_i is the rotation difference between $(i+1)$th and ith lumped masses.

The forces of various dampers acting in the shear direction are calculated from the shear deformation δ_{si} and other related factors as

$$F_d = f(\delta_{si}, \dot{\delta}_{si}, T, A, \omega, E, G) \tag{4}$$

where the damper force is dependent not only on the shear deformation but also on the velocity, temperature, frequency, geometrical size and material properties corresponding to the type of damper. Details of eq. (4) corresponding to various types of damper have been given by many researchers[2–4,22–25].

A RATIONAL DYNAMIC ANALYSIS METHOD FOR 3D FRAME

Based on the conventional approaches of structural analysis, members with material nonlinearity or mechanical nonlinearity such as elasto-plastic dampers and viscoelastic dampers are considered as a part of the whole structure. The dynamic equations used in many existing general application programs are usually written in the form

$$\mathbf{M}\ddot{\mathbf{X}} + \mathbf{C}\dot{\mathbf{X}} + \mathbf{K}_T\Delta\mathbf{X} + \mathbf{F}_T = -\mathbf{M}\ddot{\mathbf{X}}_g \tag{5}$$

where $\mathbf{M}$ is the lumped mass matrix; $\mathbf{C}$ is the natural damping matrix assumed to be proportional to the

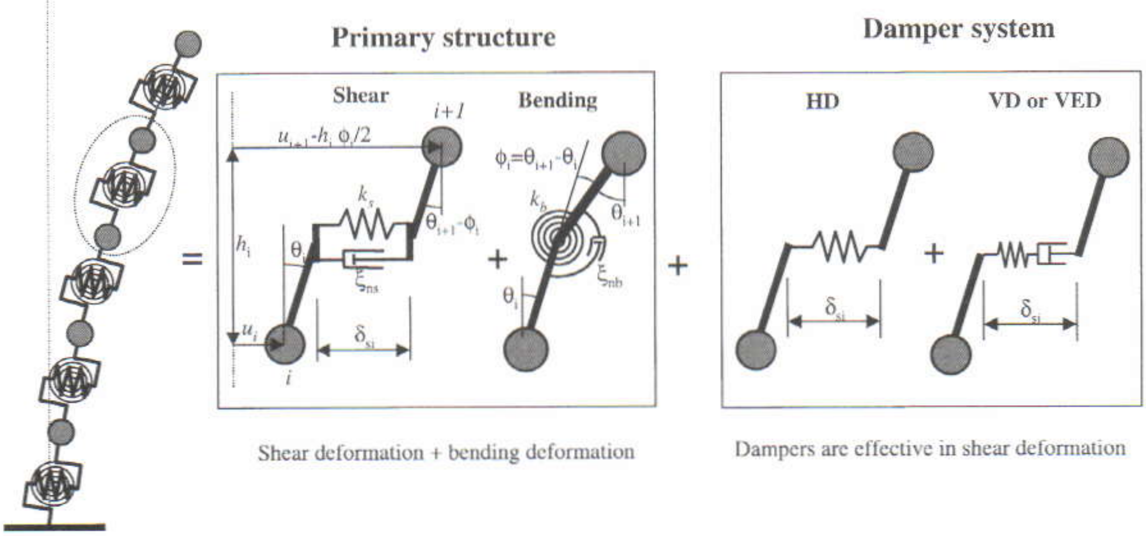


Fig. 2 Shear-bending beam model for the dynamic analysis of DCS

mass and stiffness matrix; $\mathbf{K}_T$ is the transient stiffness matrix including the elastic and inelastic properties of the structure; $\mathbf{F}_T$ is the internal reaction force vector at previous calculating step; $\mathbf{X}$ is the response displacement vector including rotational deformation; $\dot{\mathbf{X}}$ is the response velocity vector; $\ddot{\mathbf{X}}$ is the response acceleration vector; $\Delta\mathbf{X}$ is the displacement increment vector from the previous calculating step to the current step; $\ddot{\mathbf{X}}_g$ is the acceleration vector of ground motions including the components of x, y, and z directions as

$$\ddot{\mathbf{X}}_g = \{\ddot{x}_g, \ddot{y}_g, \ddot{z}_g\}^{\mathrm{T}} \tag{6}$$

For the conventional structures, it is impossible to predict which members or elements will suffer plastic deformation under the action of extreme earthquake. The transient element stiffness of all members should be included in the global stiffness matrix $\mathbf{K}_T$. Therefore, the global transient stiffness matrix $\mathbf{K}_T$ has to be reestablished which will consume a huge amount of computation time during the step by step calculation of eq. (5). Especially in the case of using viscous dampers or viscoelastic dampers (see Fig. 7b and c), the transient stiffness of the dampers will occur infinite large value when the displacement reaches maximum which makes the time integration calculation impossible. However, for the damage-controlled structures, the inelastic members or elements with mechanical nonlinearity (the damper system shown in Fig. 3) are specified separately by the designers. The elastic and inelastic parts of the global stiffness matrix $\mathbf{K}_T$ can be divided into two independent parts. The primary structure contributes to the elastic part of $\mathbf{K}_T$. The inelastic part of the structure due to the action of dampers can be expressed by the force vector and moved into the right side of the dynamic equation. Therefore, eq. (5) can be rewritten in the form of eq. (7).

$$\mathbf{M}\ddot{\mathbf{X}} + \mathbf{C}\dot{\mathbf{X}} + \mathbf{K}\mathbf{X} = -\mathbf{M}\mathbf{E}\ddot{\mathbf{X}}_g - \mathbf{F}_d \tag{7}$$

where, $\mathbf{K}$ is the elastic stiffness matrix of the primary structure and remains constant during the whole time integration. $\mathbf{F}_d$ is the force vector of the damper system and depends on the displacement, velocity, temperature and the material properties used in the dampers. $\mathbf{F}_d$, therefore, is a complicated function depending on various variables expressed in the form of eq. (4).

The flow chart of a computer program for the three-dimensional dynamic analysis is shown in Fig. 4. The most important advantages of this approach is (1) to avoid recalculation of the global transient stiffness matrix, so that a great amount of calculation time can be saved (2) to avoid the difficulty of overflow in transient stiffness of dampers like viscous or viscoelastic dampers, so that the stable time integration can be ensured (3) since the damper forces are subtracted from the external earthquake force, the concept of damage controlled structure is obviously expressed in mathematics, and (4) it is easy to treat any newly developed damper system as long as the hysteresis of force and displacement is given.

In the numerical time integrations of eq. (7), the force equilibriums are set on the latest time step $t + \Delta t$. However, the force vector of dampers $\mathbf{F}_d$ at time $t + \Delta t$ is unknown before the acceleration $\ddot{\mathbf{X}}_{t+\Delta t}$ at time $t + \Delta t$ is resolved. Therefore, the deformation of the damper at time step $t + \Delta t$ is proposed to be predicted from the values of the previous 3 steps using the Lagrangian interpolation method. If the time increment Δt is assumed to be a constant, the second-order Lagrangian interpolation formula is written in the simple form of eq. (8).

$$\delta_{d,t+\Delta t} = \delta_{d,t-2\Delta t} - 3\delta_{d,t-\Delta t} + 3\delta_{d,t} \tag{8}$$

The force vector $\mathbf{F}_{d,t+\Delta t}$ of dampers at time step $t + \Delta t$ can be calculated from the deformation $\delta_{d,t+\Delta t}$ and the given hysteresis model of the dampers. The unbalanced forces produced in the dampers

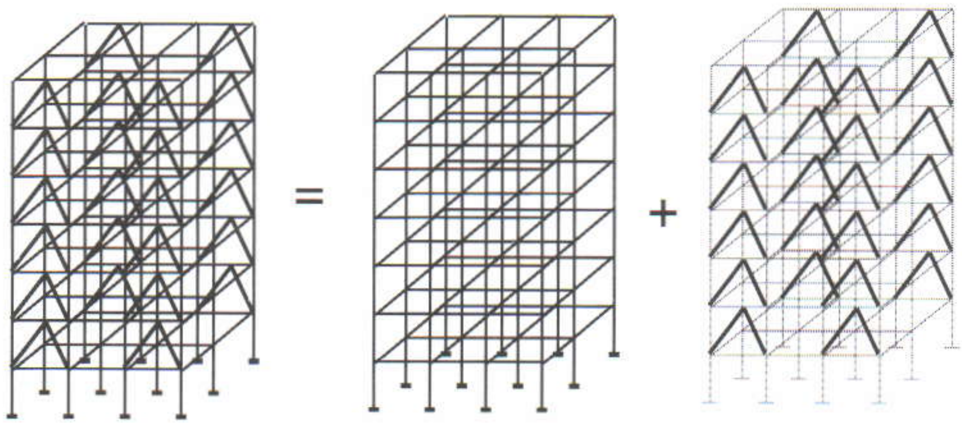


Fig. 3 Three-dimensional frame model for the dynamic response analysis of DCS

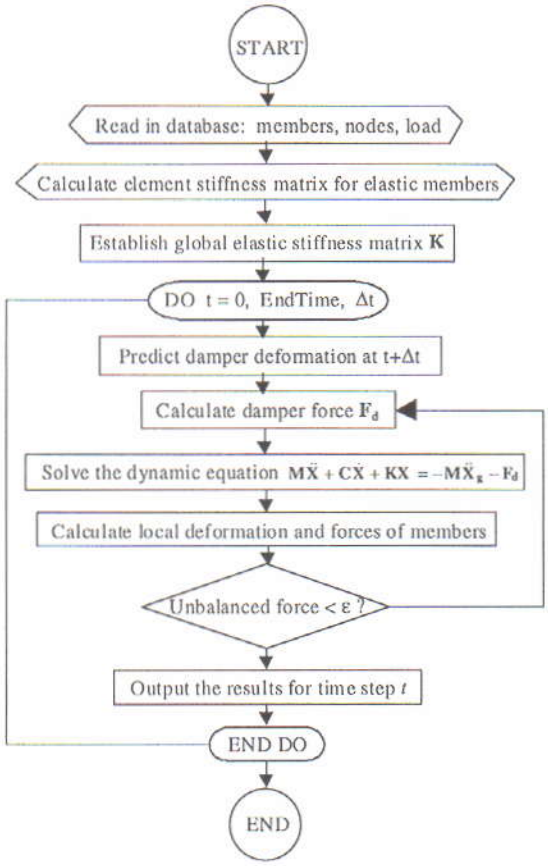


Fig. 4 Flow chart of a rational dynamic analysis computer program for a three dimensional frame structure

should be eliminated through the iterative calculation shown in the flow chart of Fig. 4.

Mechanical models of various dampers

There are various dampers used as energy dissipation devices[2–6, 22–25]. The primary dampers are categorized as in Fig. 5. For the purpose of dynamic response analysis using eq. (7), the relationship between force and deformation of the dampers is needed and usually obtained from the experimental results of an individual damper test or calculated from the constitutive law of the damper materials. However, the deformation of the damper greatly depends on the stiffness of the members connecting it into the primary structure. Figs. 6a and b show two typical damper installations in a primary structure. Fig. 6a shows a brace-type damper connected to a primary structure with two elastic connecting members. This kind of damper is subjected to axial force and axial deformation only. Fig. 6b shows a shear-deformed-type damper connected to a primary structure by two short elastic columns which are also called connecting members. In all of these damper systems, an elastic spring connecting the damper to the primary structure in series has to be considered. Figs 7a–c show models of three different dampers: a hysteretic damper, a viscous damper, and a viscoelastic damper.

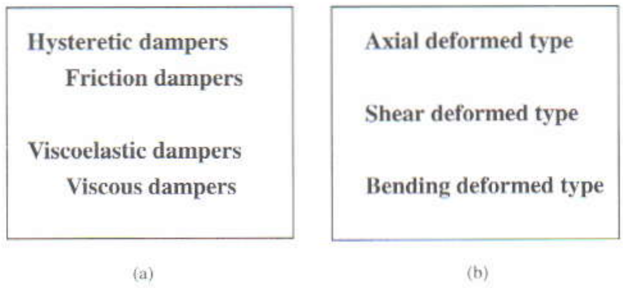


Fig. 5 Category of dampers: (a) based on the hysteresis type; (b) based on the deformation type

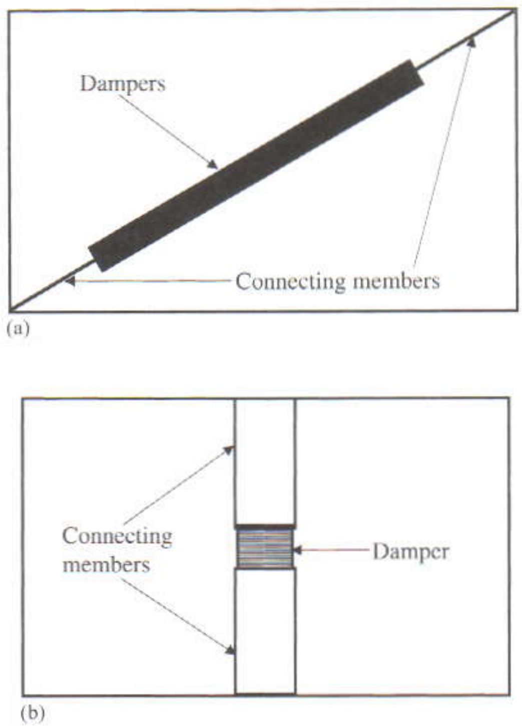


Fig. 6 Installation of various dampers: (a) in the center of brace; (b) between two short columns connecting to two adjoining floor beams

All the dampers can be modeled as multiple directional inelastic springs, as shown in Fig. 8[21]. The axial spring denotes the brace-type damper whose axial force is denoted by $\mathbf{N}_d$. The shear spring denotes the shear deformed-type damper whose shear force is denoted by $\mathbf{Q}_d$. The bending spring is considered as elastic spring denoting the bending elastic deformation of connecting members. Its bending moment $\boldsymbol{B}$ can be calculated from the bending stiffness and the rational angle produced from the rotational deformation $\bar{\theta}_i$ and $\bar{\theta}_j$ at each end of the ith damper.

All the dampers are assumed to be effective only in the $\bar{X}\bar{O}\bar{Z}$ plane of the local coordinate system shown in Fig. 9. It means that the forces $\bar{Y}_i$, $\bar{Y}_j$ along with $\bar{y}$ axial direction and the bending moments $\bar{M}_{\bar{x}i}$, $\bar{M}_{\bar{z}i}$, $\bar{M}_{\bar{x}j}$, $\bar{M}_{\bar{z}j}$ about the $\bar{x}$- and $\bar{z}$-axis are all zero. The forces at two end nodes i and j of the

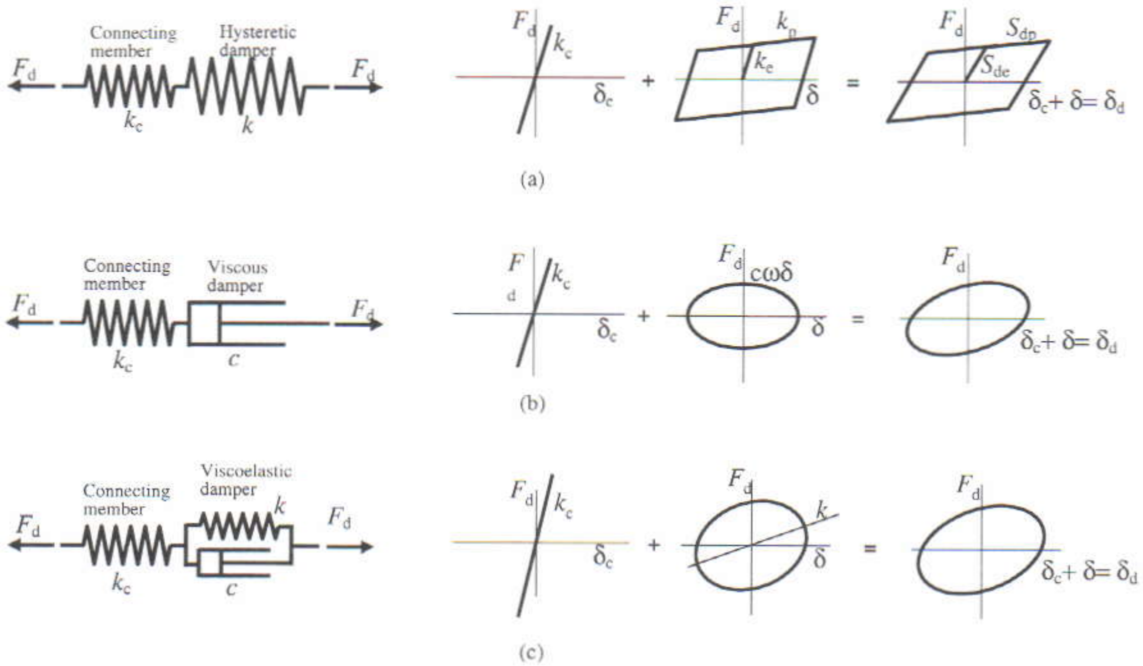


Fig. 7 Brace-type dampers: (a) combination of elastic spring and hysteretic damper; (b) combination of elastic spring and viscous damper; (c) combination of elastic spring and viscoelastic damper

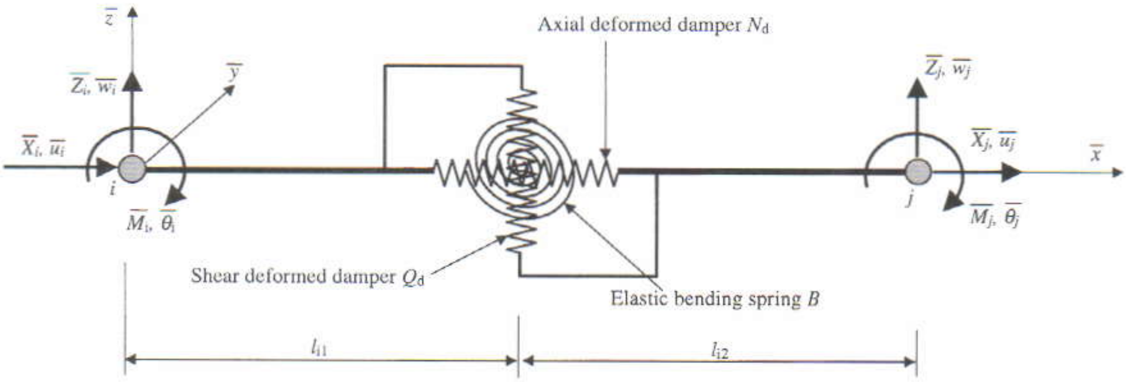


Fig. 8 Single damper model

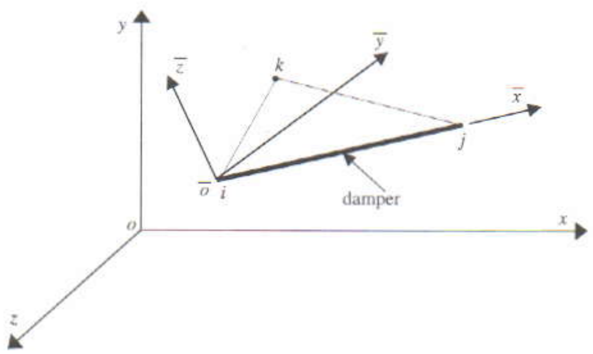


Fig. 9 Local coordinate and global coordinate system for damper model

damper in the local coordinate system can be calculated by

$$\bar{X}_i = -N_d, \quad \bar{Z}_i = -Q_d, \quad \bar{M}_{\bar{y}i} = l_{i1} Q_d - B \tag{9}$$

$$\bar{X}_j = N_d, \quad \bar{Z}_j = Q_d, \quad \bar{M}_{\bar{y}j} = l_{i2} Q_d + B \tag{10}$$

The bending moment at the elastic bending spring is calculated from the bending stiffness and the rotation angle by

$$B = k_b(\bar{\theta}_j - \bar{\theta}_i) \tag{11}$$

where k_b is the elastic stiffness of the bending spring. The stiffness k_c of the connecting members is given below in respect to different installation methods.

BRACE-TYPE DAMPER (FIG. 6a)

If the damper is installed in the center of an elastic brace like in Fig. 6a, the stiffness of the connecting members can be determined from the relationship between the force and deformation of the elastic brace as

$$k_c = E\frac{A_1 A_2}{l_1 A_2 + l_2 A_1} \tag{12}$$

where A_1, A_2, l_1, and l_2 are the areas and lengths of the two elastic braces at two ends of the damper.

SHEAR-TYPE DAMPERS (FIG. 6b)

For the dampers installed midway between two short columns connecting the upper and lower floor beams shown in Fig. 6b, the stiffness of the connecting members is determined based on that the short column is considered as a cantilever beam. From the relationship of the force acting at the tip of the cantilever beam and the shear, bending deformation, the stiffness of connecting spring is given by

$$k_c = \frac{1}{l_{i1}/GA_{si1} + l_{i2}/GA_{si2} + l_{i1}^3/3EI_{i1} + l_{i2}^3/3EI_{i2}} \tag{13}$$

where A_{si1}, A_{si2}, l_{i1}, l_{i2} are the shear areas and lengths of the lower and upper connecting members shown in Fig. 6b.

Calculation of damper force at next step

Damper forces are usually calculated from the hysteretic models. Different types of damper, of course, have different hysteretic models. A bilinear model is usually used for the hysteretic dampers which is usually called elasto-plastic dampers or metallic dampers, an elliptical hysteretic model is usually used for the viscous dampers, and a complicated non-linear elliptical hysteretic model with inclined axes relying on the temperature and frequency is usually used for the viscoelastic dampers. The three different models shown in Fig. 7 with elastic members connecting to the dampers are considered in the following.

FOR THE HYSTERETIC DAMPER (FIG. 7a)

The stiffness of the elastic connecting spring is denoted by k_c. The elastic stiffness of the hysteretic damper is denoted by k_e and the second stiffness of the damper by k_p. The hysteretic model of the combined connecting member and hysteretic damper is bilinear. The elastic stiffness and the second stiffness of the combined hysteretic damper are given by

$$S_{de} = \frac{k_c k_e}{k_c + k_e}, \quad S_{dp} = \frac{k_c k_p}{k_c + k_p} \tag{14}$$

The force at time step $t + \Delta T$ can be calculated by eq. (15), where the deformation of the damper $\delta_{d,t+\Delta t}$ is predicted by

$$F_{d,t+\Delta t} = \begin{cases} S_{de}\delta_{d,t+\Delta t} & (\delta_{d,t+\Delta t} < \delta_{dy}) \\ F_{dy} + S_{dp}[\delta_{d,t+\Delta t} - \delta_{dy}] & (\delta_{d,t+\Delta t} > \delta_{dy}) \end{cases} \tag{15}$$

FOR THE VISCOUS DAMPER (FIG. 7b)

The deformations of the connecting member and the viscous damper are assumed to be δ_c and δ, respectively. The differential equation for the force and deformation acting on the two ends of the connecting member and the viscous damper is expressed as

$$c\dot{F}_d + k_c F_d = ck_c\dot{\delta}_d \tag{16}$$

where k_c is the stiffness of the connecting member, c is the damping coefficient of the viscous damper, F_d and δ_d are the force and deformation of the damper system shown in Fig. 7b, respectively.

In the numerical time integration, the damper force F_d at time step $t + \Delta t$ can be calculated from the results at time step t by

$$F_{d,t+\Delta t} = e^{-k_c\Delta t/c}\left(F_{d,t} + k_c\int_0^{\Delta t}\dot{\delta}_d(\tau)\,e^{k_c\tau/c}\,d\tau\right) \tag{17}$$

If the time increment Δt used in the time integration is sufficiently small, the velocity $\dot{\delta}(\tau)$ during the time increment Δt can be considered as a constant and $\dot{\delta}(\tau)$ can be moved out from the integration. Therefore, $F_{d,t+\Delta t}$ can be solved from eq. (17) and expressed as

$$F_{d,t+\Delta t} = F_{d,t}e^{-k_c\Delta t/c} + c\frac{\delta_{d,t+\Delta t} - \delta_{d,t}}{\Delta t}(1 - e^{-k_c\Delta t/c}) \tag{18}$$

FOR THE VISCOELASTIC DAMPER (FIG. 7c)

The viscoelastic damper with elastic connecting members is modeled as in Fig. 7c. The relationship between force and deformation can be expressed in the form

$$c\dot{F}_d + (k_c + k)F_d = ck_c\dot{\delta}_d + k_c k\delta_d \tag{19}$$

Integrating eq. (19) from time step t to $t + \Delta t$, the damper force at time step $t + \Delta t$ can be obtained from eq. (20).

$$F_{d,t+\Delta t} = e^{-(k_c+k)\Delta t/c}\left[F_{d,t} + \int_0^{\Delta t}\left(k_c\dot{\delta}_d(\tau) + \frac{k_c k}{c}\delta_d(\tau)\right)e^{(k_c+k)\tau/c}\,d\tau\right] \tag{20}$$

If the deformation $\delta(t)$ of the damper at time increment Δt is assumed to be a linear function so that the velocity $\dot{\delta}_d(\tau)$ keeps constant during the time increment Δt, the damper force at time step $t + \Delta t$ can be expressed by the state variables at time step t. The calculation formula is given by

$$F_{d,t+\Delta t} = e^{-(k_c+k)\Delta t/c}F_{d,t} + \frac{k_c k}{k_c + k}(\delta_{d,t+\Delta t} - e^{-(k_c+k)\Delta t/c}\delta_{d,t}) + c\left(\frac{k_c}{k_c + k}\right)^2\frac{(\delta_{d,t+\Delta t} - \delta_{d,t})}{\Delta t}(1 - e^{-(k_c+k)\Delta t/c}) \tag{21}$$

If the elastic stiffness k of the viscoelastic damper is very small compared to the stiffness k_c of the connecting member, it can be taken as 0. Thus, eq. (21) can be reduced to eq. (18), which is the hysteresis formulation of the viscous damper with a connecting member. However, if the stiffness k_c of the connecting member is infinitely large compared with the stiffness k of the viscoelastic damper, eq. (21) can be reduced to eq. (22), which is the formulation of the Kevin model for the viscoelastic material.

$$F_{d,t+\Delta t} = k\delta_{d,t+\Delta t} + c\frac{\delta_{d,t+\Delta t} - \delta_{d,t}}{\Delta t} \quad (22)$$

CUMULATIVE DEFORMATION OF A DAMPER

The cumulative deformation or its ratio is usually used to evaluate the damage extent of a hysteretic damper. It should be noted that a damper's deformation must be subtracted from the deformation of the connecting member. Therefore, the damper's cumulative deformation should be calculated by

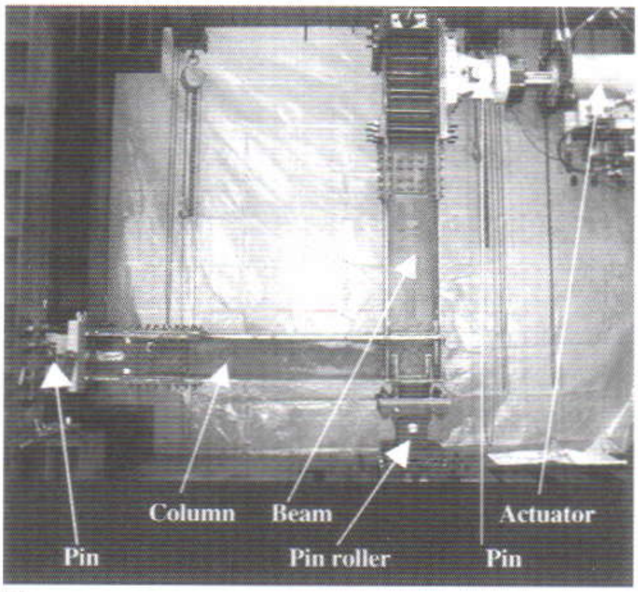
$$\delta_{p,max} = \delta_{d,max} - \frac{F_d}{k_1} \quad (23)$$

Experimental studies on DCS

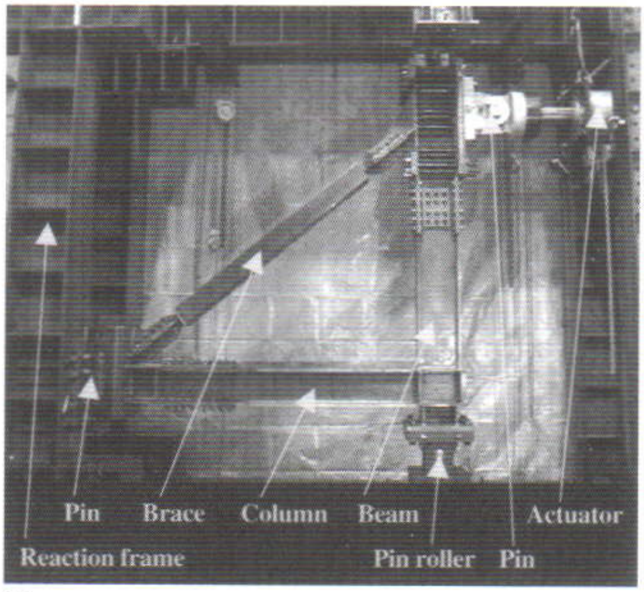
Many fractures were found in or near the welded connections between beam and column of the steel frame following the Hyogoken-Nanbu Earthquake. There are many fracture causes considered such as the welding quality, loading speed, ambient temperature, toughness of the steel, etc. However, the most important factor is likely the concentration of excessive plastic strains at the beam ends near the welded connections, because the conventional steel structures were designed based on the principle of strong column and weak beam, which allows the beam ends to yield largely to absorb most of the input earthquake energy. As mentioned previously, the beam ends become the 'Sacrifice' during the extreme earthquake.

For the damage-controlled structures, much of the input energy of the earthquake is absorbed/dissipated by the specific members so that the damage is limited to certain members. In order to verify experimentally the difference between conventional steel frame and DCS steel frames, a series of static cyclic loading and dynamic loading tests were carried out[26–28]. It has been confirmed from the experimental studies that the DCS is much better than the conventional steel structure in the energy dissipation capacity. Based on the weight calculation of the specimens shown in Table 1, the DCS can achieve better economic effect than the conventional structures.

As an example of the experimental study of DCS, Figs 10a and b illustrate two types of tested steel frame. Specimen MRF1 is designed as a conventional steel frame without dampers and MRF2 is designed based on the concept of DCS which has a hysteretic damper (unbonded brace). Two different strength steels, high-strength steel HT590 and mild-strength steel SM490A were used for the test specimens. The



(a)



(b)

Fig. 10 Two types of specimen: (a) MRF1 (Moment Resistant Frame); (b) Slender MRF2 with unbonded brace

Table 1. Sectional size and weight ratio of the tested specimens

Steel	Frame type	Beam (length = 200 cm)	Column (length = 245 cm)	Weight (kN)	W_1/W_2
SM 490A	MRF1	H-380 × 200 × 9 × 12	H-320 × 300 × 19 × 22	$W_1 = 4694$	1.52
(mild)	MRF2 + brace	H-300 × 170 × 9 × 12	H-260 × 260 × 12 × 16	$W_2 = 3097$	
HT590	MRF1	H-380 × 160 × 6 × 9	H-320 × 300 × 19 × 22	$W_1 = 4234$	1.58
(High strength)	MRF2 + brace	H-300 × 130 × 6 × 9	H-260 × 260 × 12 × 16	$W_2 = 2685$	

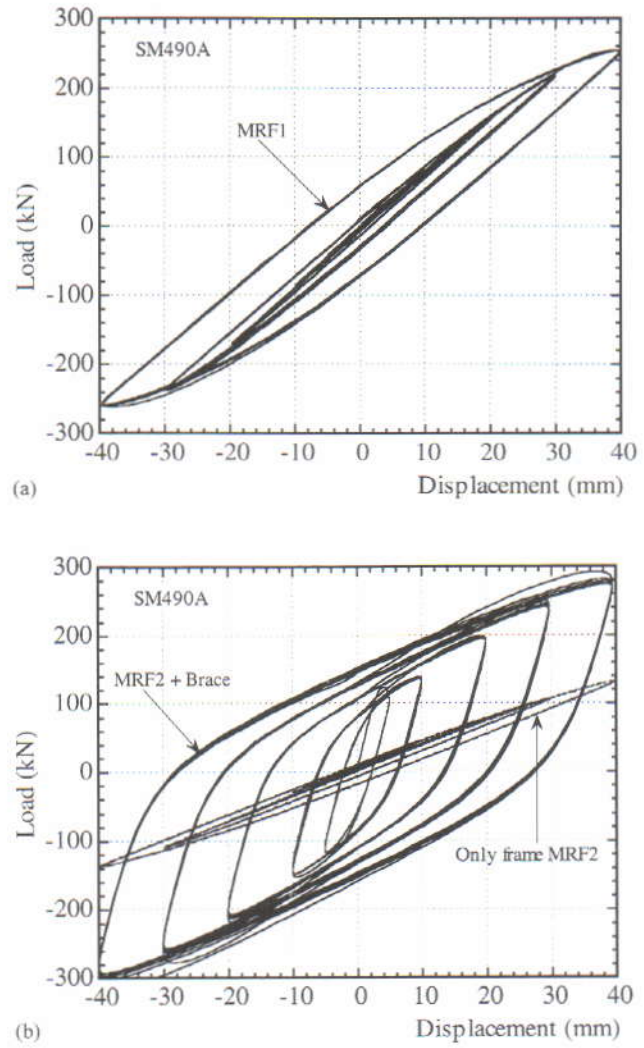


Fig. 11 Hysteresis of load and deformation of the specimens made of mild-strength steels: (a) MRF1 specimen; (b) MRF2 specimen

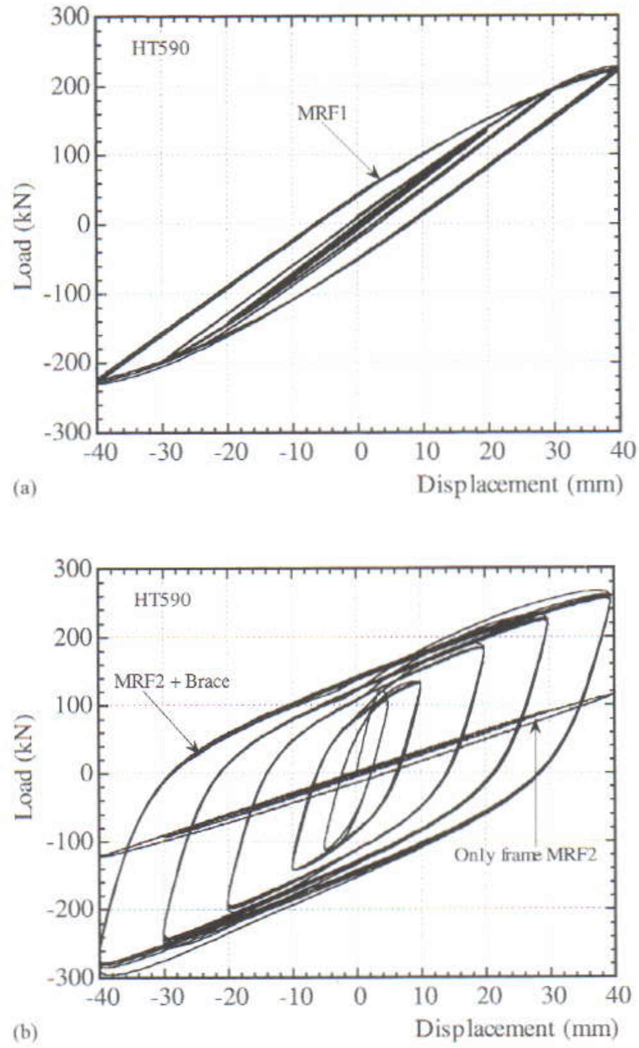


Fig. 12 Hysteresis of load and deformation of the specimens made of high-strength steels: (a) MRF1 Specimen; (b) MRF2 specimen

sectional size and the weight ratio of the two specimens are shown in Table 1. For the purpose of comparison, two test specimens were designed with the same ultimate strength. Due to the effect of the damper, the sectional size of specimen MRF2 is smaller than that of MRF1 so that the weight of MRF2 is less than that of MRF1. The weight w_2 does not include the weight of the brace.

The dynamic loading was supplied by the actuator at 0.65 Hz frequency. After the first 2 cycles under 5 mm amplitude, 5 cycles for each amplitude were applied until the amplitude reach 40 mm which is equal an inter story angle of 1/50. After that, cyclic dynamic loading was continuously applied until the ultimate load decrease to 90% of the maximum load.

Figs 11a, b and 12a, b show the hysteresis of load and deformation of the specimens made of mild-strength steel SM490A and the high-strength steel HT590, respectively. Figures (a) and (b) of both Figs 11 and 12 show the results of specimens MRF1 which were designed based on the conventional structure and those of MRF2 which were designed based on the concept of damage-controlled structures. Compared to the plastic extent of MRF1, the primary structure of beam and column of MRF2 maintains almost in elastic even under the 1/50 inter story deformation angle (seen the central loops of Figs 11b and 12b). On the contrary, it is obviously that the energy dissipation capacity of MRF2 specimen including the brace is much more than that of MRF1. In this experimental study, sinusoidal excitations with the same displacement amplitude were applied to the specimens of MRF1 and MRF2. In fact, the response of the displacement of MRF2 will be smaller than that produced in MRF1 because MRF2 system dissipated much more energy than that of MRF1. It can be concluded that the seismic performance of MRF2 is much better than that of MRF1.

Actual example DCS projects

After the Hyogoken-Nanbu Earthquake, a lot of building projects designed based on the concept of damage controlled structures have been examined by

Table 2. Tall steel buildings designed based on the same concept of DCS in recent years

Year	Project's name	Location	Usage	Height (m)	Structure type	Dampers	Ductility ratio
1995.6	International Congress	Osaka	Congress	104	S_F	HD_B	0.95
1995.7	Todai Hospital	Tokyo	Hospital	82	S_F	VD_S	0.93
1995.7	Tohokudai Hospital	Sendai	Hospital	80	S_F	VD_S	0.97
1995.8	Central Government	Tokyo	Office	100	S_F	HD_B + VD_S	0.78
1995.10	Harumi I Chome	Tokyo	Office, Shop	175	S_F	HD_B	0.88
1996.2	Toranomon 2 Chome	Tokyo	Office, Shop	94	S_F	VD_S	0.94
1996.3	Passage Garden	Tokyo	Office	61	S_F	HD_B	0.88
1996.4	Shiba 3 Chome	Tokyo	Office	152	S_F	HD_B	0.97
1996.6	Art Hotel	Sapporo	Hotel	96	S_F	HD_BD	0.85
1996.8	Kanto Post Office	Saitama	Office	130	S_F	VD_S	0.87
1996.10	Nakano Urban	Tokyo	Office, Shop	96	S_F	VD_S	0.68
1997.7	DoCoMo Tokyo	Tokyo	Communication, etc.	240	S_F	VD_S	0.79
1997.10	Minato Future	Yokohama	Hotel, Shop, Office	99	S_F	HD_BD	0.98
1997.11	Nishiguchi Shintoshin	Yamagata	Office, Hotel, etc.	110	S_F	HD_B	1.00
1998.2	DoCoMo Nagano	Nagano	Communication	75	S_F	VD_S	0.89
1998.4	East Osaka City	East Osaka	Office	120	S_F	HD_S	1.00
1998.5	Kouraku Mori	Tokyo	Office, Shop	82	S_F	HD_B	1.00
1998.7	Harumi I Chome	Tokyo	Office, Shop, etc.	88	RC_F	HD_B	1.00
1998.11	Adago 2 Chome	Tokyo	Office, Shop	187	S_F	VD_B	0.71
1998.11	Gunyama Station	Fukushima	Shop, School, etc.	128	S_F	HD_B + VD_S	0.98

Notes: Year — Time of project design
Structure type — Type of primary structure
S_F — Steel frame structure
RC_F — Reinforced-concrete structure
HD_B — Brace-type hysteretic damper made of steels like unbonded braces
HD_S — Shear panel-type hysteretic damper made of steel
HD_BD — Bending-type hysteretic damper like slit damper and honeycomb damper
VD_S — Shear wall-type viscous damper like oil-filled shear walls
VD_B — Brace-type viscous damper like oil piston damper

the Japan Building Center. Some of them have already been constructed. Table 2 gives a list of these projects. Some typical building projects are introduced here.

CENTRAL GOVERNMENT BUILDING

The Central Government Building located in Chiyoda-ku, Tokyo, whose conceptual drawing of the building is shown in Fig. 13 was designed by the Architecture Department of the Ministry of Construction and Kume Sekkei Co. Ltd. The total height is 144.5 m including a 55 m antenna tower on the roof of the building. The superstructure above the ground level is a moment-resisting steel frame with the combined use of HDs (steel walls made of extra-low-yield point steels) and VDs (viscous damping wall), while the underground structure is a steel reinforced concrete frame with reinforced concrete shear walls. Columns and beams of the primary structures use SN490B steel (maximum strength is 490 MPa). In the dynamic response analysis of the structure, the modified shear bending beam model using multi-springs and multi-dashpots shown in Fig. 2 was used. This is a very important building since it is used for the headquarters of the Government

Fig. 13 Central government building (with HD + VD) (Courtesy of Kume Sekki Co. Ltd.)

Police Board. The primary structure is designed to behave elastically even under a large intensity earthquake whose maximum velocity is 50 cm/s. Most of the earthquake energy is designed to be absorbed by the combined use of HDs and VDs. The HDs are steel walls made of extra-low yield point steel (yield point is 100 MPa). The yield shear force level of HDs is assumed to be 5% of the total building weight. The distribution of yield shear force throughout the height of the building is assumed to be proportional to the distribution of yield shear force of the primary structure. On the other hand, the VDs consist of two movable steel plates and three fixed steel plates. The space between the movable steel plates and the fixed steel plates is filled with viscous liquid like silicone oil.

ART HOTEL IN SAPPORO

The Art Hotel Sapporo, designed and constructed by Kumagai Corporation, is another DCS building. The conceptual drawing is shown in Fig. 14. The total height is 96 m. There are 26 stories above ground and one story underground. The primary structure is a moment resisting steel frame that is designed to support only the vertical load. Two thousand slit steel dampers (SSD) whose details are shown in Fig. 15 made of mild-strength steel (SN490B) are installed in the building. When the SSD subjects the shear force through the top and bottom bolts, the slender bars between the slits experience bending deformation. Because the cross section of the slender bars is very small, they yield quickly even under very small shear deformation, which makes this type of damper a good choice for absorbing earthquake energy. However, since the yielded parts of this kind of damper are easily concentrated on the small end parts of the slender bars, the slits should be manufactured very carefully so as to avoid excessive local strain concentration.

PROJECT OF HARUMI 1 CHOME

Fig. 16 shows the conceptual drawing of the project in Harumi 1 Chome in Tokyo, which was designed by Nikken Sekkei Ltd. There are three buildings in this project. The total floor area of the three buildings is more than 500 000 square meters. Hysteretic dampers (unbonded braces) are installed on the exterior moment resisting frames and in every story.

Fig. 14 Art Hotel in Sapporo (with HD) (Courtesy of Kumagai Corporation)

PASSAGE GARDEN IN SHIBUYA, TOKYO

Passage Garden located in Shibuya, Tokyo whose conceptual drawing is shown in Fig. 17 was designed by Plantec Design Office (structural design was by Alpha Structural Design Office and Nippon Steel Corporation). The structural system of this office building has no vertical columns. The vertical and lateral loads are supported by the inclined column system. The DCS building is 61.4 m high and has 14 stories. The entire structural system is composed of two independent structural systems; elastic column system and unbonded member system that acts as hysteretic dampers. The unbonded member system has an equivalent damping coefficient of about 8%. Since the entire structural system is divided into two independent systems, damage to the building in case of an extremely large earthquake would be confined to the unbonded members, which are designed to be able to be easily replaced.

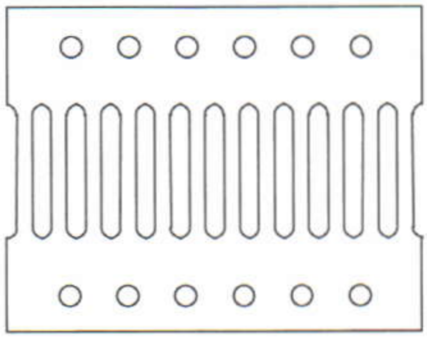


Fig. 15 Details of the steel slit damper (SSD)

Fig. 16 Harumi 1 Chome Project (with HD) (Courtesy of Nikken Sekkei Ltd.)

Fig. 17 Passage Garden in Shibuya (with HD) (Courtesy of Nippon Steel Corporation)

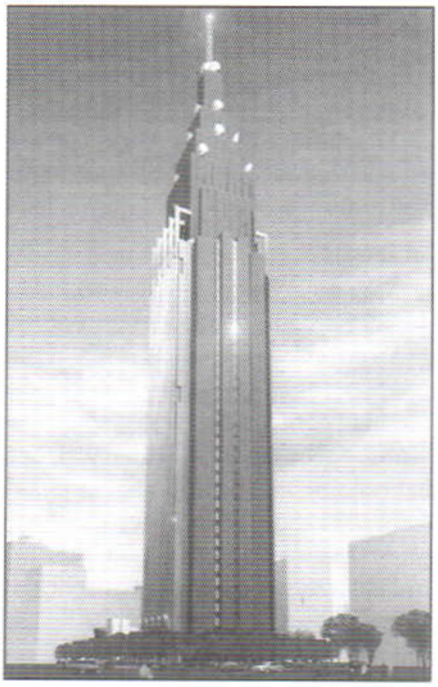


Fig. 18 DoCoMo Tokyo Building (with VD) (Courtesy of NTT Power and Building Facilities Inc.)

DoCoMo Tokyo building

This building shown in Fig. 18 is located in the metro area of Tokyo, near Yoyogi station. The structural design was made by NTT Power and Building Facilities Inc. This building has two parts. The lower part has 27 stories and is mainly used for the office and the upper part of the building is the antenna which has 23 stories used for the mobile communication. There are also three stories under ground. The total height of the building is 240 m. The building part of 27 stories is steel frame structure with 76 viscous damping walls both in X and Y directions. The supplemental viscous damping wall system has the same energy dissipation capacity of 5% equivalent viscous natural damping in both directions. The antenna part of 23 stories is a steel frame structure with steel braces. The viscous damping wall is a kind of high-quality highly stable damping system and has been used in more than 10 tall building projects since it was first used in a seismic buildings in 1988. Due to the use of additional viscous damping wall, the primary steel structure is designed within the elastic region even under the second level of earthquake (maximum velocity is 50 cm/s).

Fig. 19 Minato Future in Yokohama (with HD) (Courtesy of Kume Sekkei Co. Ltd.)

Commercial building in Minato Mirai

The building project shown Fig. 19 locates in the central Yokohama. It is mainly used for commercial facilities such as hotels and shops. The superstructures above the ground level are steel frame structures. The height of the building is about 100 m with 25 stories above ground level and 2 stories underground. The structural design was made by Kume Sekkei Co. Ltd. About 570 steel slit dampers whose shape is shown in Fig. 15 were installed from the first story to the top story. Half of the dampers were in the X direction and another half in the Y direction. The energy dissipation capacity of one SSD is about 240 kN m. About 20–30% of the lateral shear force of on the earthquake is designed to be resisted by the SSDs and the rest is designed to be resisted by the primary structures. The maximum plastic ratio of the primary structure is less than 1.0 and is in agreement with the requirement of damage controlled structures.

DoCoMo Nagano building

This building shown in Fig. 20 is located near Nagano Station of Nagano City and is used both as office and communication facilities. The structural design was made by NTT Power and Building Facilities Inc. The project includes two parts—67 m high building part and 39 m high antenna part. There are also two stories underground. Both the building part and the antenna part are of steel frame structure. In order to reduce the response of inter-story drift and the floor acceleration, many additional viscous dampers are included in the building. The design target of the energy dissipation capacity of the additional viscous dampers is about 20% equivalent viscous natural damping.

Conclusions

For a number of decades, the strong column weak beam system based on the conventional design method has mainly been used in a seismic structures. It was assumed that local plasticity at beam ends connected to the column was allowed and severe damage of the whole building was permitted as long as human life could be guaranteed during an extreme earthquake. However, the lessons learned from the Hyogoken-Nanbu Earthquake and Northridge Earthquake indicated that, large plastic deformation concentrated on the beam ends resulted in many failures in the welded connections between beams and columns. It resulted not only in huge economic loss, but also the loss of human lives. It is very difficult to check and expensive to repair such steel building failures even if the building does not completely collapse. In Japan, the seismic design trend for tall buildings after the Hyogoken-Nanbu Earthquake is to design based on the concept of damage controlled structures. According to this concept, the primary structure of beams and columns is always assumed to behave elastically, or only a small amount of plasticity is permissible during an extreme earthquake. Most of the input energy of an earthquake is assumed to be absorbed by the additional dampers, not by the local damage and large plasticity of beams and columns. The number of tall buildings designed based on this concept in Japan is increasing. DCS with passive damping technology promises to become the new direction in the field of earthquake resistant design of buildings.

Fig. 20 DoCoMo Nagano (with VD) (Courtesy of NTT Power and Building Facilities Inbc.)

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* Special interest

** Exceptional interest

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