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INFLUENCE OF HYSTERETIC DAMPERS ON THE SEISMIC RESPONSE OF HIGH-RISE BUILDINGS

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1. Introduction

Since the concept of performance based design to the structure has been proposed by J.J.Connor and A.Wada (1992), a lot of experimental and theoretical research works about the study of Damage Tolerant Structures (Noted as DTS here) under the action of earthquake, have been done by the writers and some other authors, such as M.Iwata, H.Kawai and A.Watanabe. The basic assumptions of DTS are, (1) the main structure made of high strength steel supports the vertical loads (including dead loads and live loads) and remains elastic always, (2) the input energy of earthquake should be dissipated by the shear-resistant members like braces that act as hysteretic dampers (Noted as HD in this paper here after), (3) the buildings, therefore, could be able to work continuously after the earthquake finished as long as the damaged braces are replaced by the new ones.

A building structure (Fig. 1 (a)) could be separated into main structure (Fig. 1 (b)) and brace system (Fig. 1 (c)) independently. According to the design technique of energy concentration proposed by H.Akiyama (1985), it is assumed that the energy brought from the earthquake should be dissipated completely by the brace system because the main structure could not be allowed to plunge into the yield for its dangerous to link with the overturn collapse of the whole building.

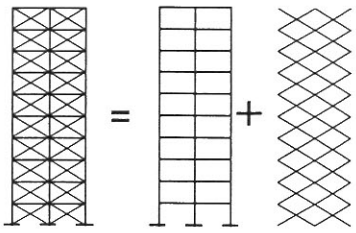
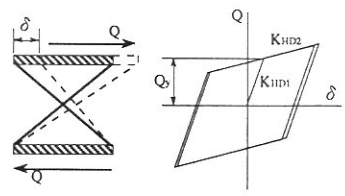


Fig. 1 Concept of DTS



(a) Hysteretic Damper (b) Q-δ relationship of H.D.

Fig.2 Mechanism of HD

The energy dissipating capacity and the response subduing capacity of the HD, however, have not been completely understood yet. The purpose of this paper is, therefore, to investigate the influence of some factors relating to the HD on the seismic response of high-rise buildings, such as the distribution shape and the fluctuation of yield level or second stiffness of the HD.

The braces in some floor might be thought as one HD shown in Fig. 2 (a). Relationship between the restoring shear force Q and displacement δ of the HD is considered as bi-linear model illustrated in Fig. 2 (b). The yield level is denoted as Q_y, elastic stiffness is denoted as K_{HD1}, second stiffness after yield is denoted as K_{HD2}. The yield level of Q_y used in the numerical response analysis here is taken as *0.5%W*, *1%W* and *2%W*, where W is the total weight of the building. The elastic stiffness of K_{HD1} is taken as 4 times of the shear stiffness of main structure that will be stated in section 3. The second stiffness K_{HD2} of the HD after yield is taken in the scope of 0 to *20%*K_{HD1}. The influence of different yield level and second stiffness on the response of shear strain and energy dissipating capacity will be discussed in the following sections 6 to 9.

2. Simplified model for numerical response analysis

In order to make the numerical response analysis simply for the high-rise buildings, we have proposed a multi-spring and multi-lumped-mass analysis model shown in Fig. 3 based on the bending-shear pole model proposed by H.Tajimi (1965), in which the shear and bending deformation could be divided into two independent parts in translation and rotational direction respectively.

In this analysis model, there are three springs and three dash pots to be considered to connect two adjoining lumped-masses. One elastic spring in translational direction stands for the elastic shear stiffness of the main structure. Another one is inelastic spring in translational direction and stands for the shear-resistant inelastic members such as braces (or HD). The third one is rotational elastic spring standing for the bending stiffness of the main structure. Two dash pots in translational direction and one in rotational direction are considered as the natural and artificial damping effect. The mechanism of the springs and dash pots is illustrated in Fig. 3 (c). The relationships between the restoring force and displacement of the elastic springs and

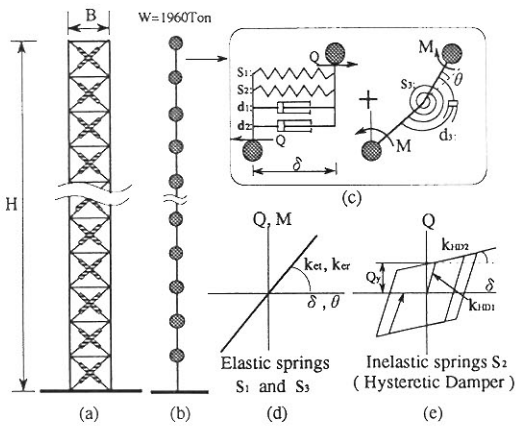


Fig.3 Analysis model

HD are shown in Fig. 3 (d) and (e) respectively.

3. Optimum stiffness distribution of the main structure

In order to make the material be used as most benefit as possible, it is necessary to have the shear strain and bending curvature maintained constant throughout the height of the buildings. It is obviously meaningful that an optimum stiffness distribution should be used in the design of main structures, because the response of shear and bending strain is mainly depended on the shear and bending stiffness of the main structure. A continuous optimum stiffness distribution to consider the shear and bending deformation separately based on the first vibration mode has been developed by J.J. Connor and A. Wada (1992). That is, however, certainly not the optimum distribution as for the MDOF system due to the effect of higher order vibration mode. We adopt therefore an approximate optimum stiffness distribution proposed by the writers (1993), that are summarized in the following equations.

$$D_T = \frac{m\Omega^2 H^2}{2}\left(1+\frac{A}{3}\right) * \tilde{D}_T$$
$$D_B = \frac{m\Omega^2 H^4}{2}\left(\frac{1}{4}+\frac{2}{3A}\right) * \tilde{D}_B \qquad (1)$$

$$\tilde{D}_T = \alpha_0 + \alpha_1\bar{x} + \alpha_2\bar{x}^2 + \alpha_3\bar{x}^3$$
$$\tilde{D}_B = \beta_0 + \beta_1\bar{x} + \beta_2\bar{x}^2 + \beta_3\bar{x}^3 + \beta_4\bar{x}^4 \qquad (2)$$

where,

A : structure parameter (H/fB)
$\bar{x}$: dimensionless coordinate (x/H)
H : total height of building
B : width of building
f : constraint of shear strain to column strain
m : mass per unit height of building
Ω : first circular frequency of structure in radians

The values of coefficient α_i and β_i are listed in the table 1.

4. Energy deposited in the vibration system

The motion equation of a general vibration system model shown in Fig. 3 might be expressed in the following form,

$$M\ddot{y} + (C_n + C_a)\dot{y} + F_e(y) + F_p(y) = F_{eq} \qquad (3)$$

where,

$M\ddot{y}$: inertia force item of the system,
$C_n\dot{y}$: natural damping force item,
$C_a\dot{y}$: artificial damping force item,
$F_e(y)$: elastic restoring force item,
$F_p(y)$: inelastic restoring force item,
F_{eq} : seismic force item.

Multiplied by dy on both sides of Eq. (3), and integrated from beginning 0 to time t, Eq. (3) will be transferred into the following energy form.

$$M\int_0^t \ddot{y}\dot{y}dt + C_n\int_0^t \dot{y}^2dt + C_a\int_0^t \dot{y}^2dt + \int_0^t F_e(y)\dot{y}dt + \int_0^t F_p(y)\dot{y}dt = \int_0^t F_{eq}\dot{y}dt \qquad (4)$$

or

$$E_k + E_{dn} + E_{da} + E_{se} + E_{sp} = E_{eq} \qquad (5)$$

where,

$E_k = M\int_0^t \ddot{y}\dot{y}dt$ denotes the kinetic energy of the system,
$E_{dn} = C_n\int_0^t \dot{y}^2dt$ denotes the natural damping energy,
$E_{da} = C_a\int_0^t \dot{y}^2dt$ denotes the artificial damping energy,
$E_{se} = \int_0^t F_e(y)\dot{y}dt$ denotes the elastic strain energy ,
$E_{sp} = \int_0^t F_p(y)\dot{y}dt$ denotes the hysteretic energy in HD,
$E_{eq} = \int_0^t F_{eq}\dot{y}dt$ denotes the input energy of earthquake.

In the vibration system without any artificial dampers, the third term of Eq. (4) will be disappeared. On the contrary, if there is no inelastic members (or HD) occurred in the system, there will be no fifth term in the Eq. (4). If some structural members are plunged into the inelastic range, the inelastic members will act as HD and similar to the artificial viscous dampers. Let the energy dissipated by the inelastic springs (or HD) be equivalent to the energy consumed by the artificial viscous dampers, that is,

$$E_{sp} = \int_0^t F_p(y)\dot{y}dt = E_{da} = C_a\int_0^t \dot{y}^2dt \qquad (6)$$

From this equilibrium condition, the prediction formula of the equivalent viscous damping coefficient h_e could be derived on the basis of assumption that the artificial damping term C_a be proportional to the shear stiffness of the main structure, that is

$$C_a = \frac{2}{\omega_1}h_e D_T \qquad (7)$$

After it is substituted into Eq. (6), the equivalent viscous damping coefficient h_e could be expressed by the following formula for a MDOF system.

$$h_e = \frac{\omega_1\int_0^t\left[\sum_{i=1}^{n} F_p(y_i)\dot{y}_i\right]dt}{2\int_0^t\left[\sum_{i=1}^{n} D_{Ti}\dot{y}_i^2\right]dt} \qquad (8)$$

where, ω_1 is the first circular frequency of the building structure.

Table 1

α_0=	1.1412+0.0906A
α_1=	-0.4737-0.3256A+0.0289A²
α_2=	-0.0422+0.4990A-0.0833A²
α_3=	-0.4388-0.2013A+0.0533A²
β_0=	1.10304-0.0061A+0.0022A²
β_1=	-1.6279-0.0415A-0.0022A²
β_2=	0.5408+0.3844A-0.0481A²
β_3=	-0.3113-0.5247A+0.0924A²
β_4=	0.2971+0.1864A-0.0442A²

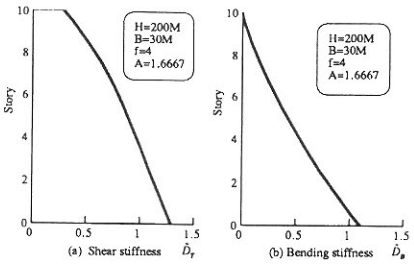
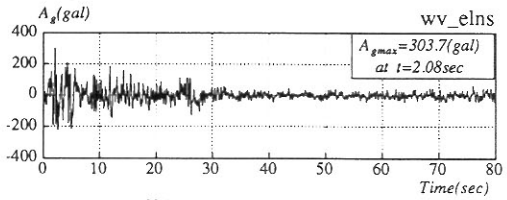


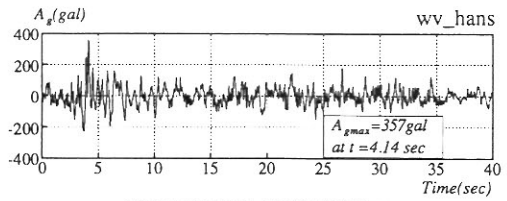
Fig.4 Dimensionless stiffness distribution used here

5. The ground motions used in the response analysis

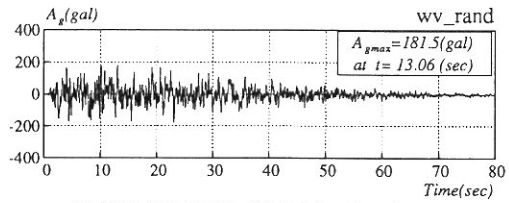
In the dynamic response analysis, we utilized three kinds of artificial ground motions that are generated by ourselves to fit for a given target design spectrum of equivalent velocity based on the earthquake energy (H.Akiyama 1985). These artificial ground motions are created by using FFT method and using the phase extracted from the famous EL-CENTRO and HACHINOHE real earthquake ground motion as well as uniformly distributed RANDOM phase. The history diagrams of acceleration of these artificial ground motions are shown in Fig. 5. Even though the artificial ground motions are fit for the same spectrum the maximum values of acceleration are quite different because the quite different phases are used.



(a) Using the phase of EL-CENTRO ns



(b) Using the phase of HACHINOE ns



(c) Using the uniformly distributed random phase

Fig. 5 Artificial ground motions

6. Influence of HD place position

In the next sections, we will discuss the influence of various factors relating to the HD on the shear strain response and energy dissipating capacity. The height of the example building used in response analysis is 200M and the total weight is 1960 ton. The natural periods under 3rd vibration mode are 6s, 2.1s and 1.2s, respectively. The yield level Q_y of HD is taken as $0.5\%W$ and the second stiffness of HD is taken as $5\%K_{HD1}$ here.

We at first discuss the influence of HD place position throughout the building height on the seismic response. Let us take one HD as an example shown in Fig. 6, which is arbitrary placed on the i story. Fig. 7 and 8 illustrate the displacement response at top floor and the energies dissipated by the HD under three different ground motions. The vertical axis denotes the HD place position that varied from the ground floor to the top floor. The lateral axis of Fig. 7 shows the displacement ratio of $x_{HD}^{top}/x_{ND}^{top}$, where x_{HD}^{top} denotes the displacement at top floor when a HD is placed on the i floor and x_{ND}^{top} denotes the displacement without any HD. The lateral axis of Fig. 8 shows the energy dissipated by the HD.

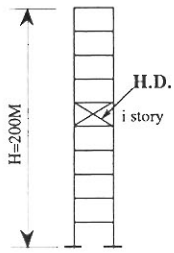


Fig.6 Place position of one H.D.

In the legend of the figures, *elns* denotes the **wv_elns** ground motion be used, *hans* denotes the **wv_hans** ground motion be used, *rand* denotes the **wv_rand** ground motion be used. From these figures, it is obvious to understand that the HD placed on the lower half part of the height is quite efficient in reducing the displacement response and dissipating the earthquake energy. The characteristics of

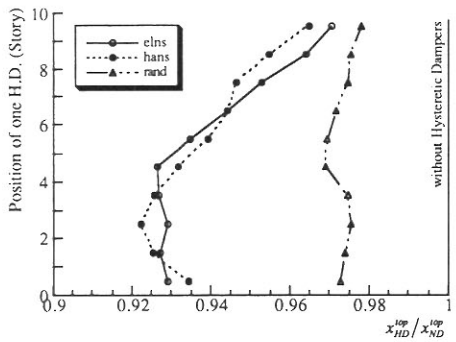


Fig. 7 Response of displacemen at top story

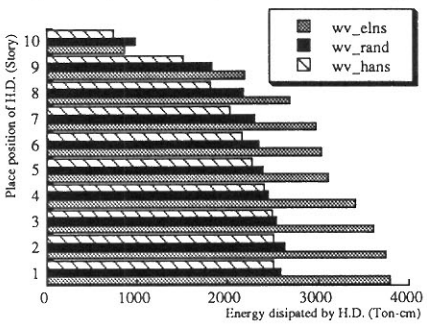


Fig. 8 Influence of place position of H.D. on the energy disipating capacity

the ground motions greatly affect the validity of HD. For example, the content of the displacement response reduced is very small when **wv_rand** ground motion is used by comparing with two other ground motions. On the other hand, the energy dissipated by HD is quite larger when **wv_elns** is used than that two other ground motions are used.

7. Influence of the HD distribution shape on the shear strain response

Even if every story is equipped with HD, there is also a problem of distribution shape of the HD stiffness and yield level. Two types of distribution shape shown in Fig. 9 (a) and (b) are considered here. Case A considers the proportional distribution to the shear stiffness of main structure, it is noted as P.D. of H.D. simply, case B considers the uni-

form distribution throughout the height and noted as U.D. of H.D. The shear strain responses of these two cases of distribution shape are illustrated in Fig. 10. The result of the case without any HD is also plotted in the figure for the purpose of comparison.

It has been seen from Fig. 10 that there is great benefit in both cases in reducing the shear response compared with the case without any HD. Although the shear strain in the lower half part has no difference between case A and case B, however, the shear strain in the upper half part in case A is less than that in case B because the stiffness of case A is larger than that of case B in the upper half part.

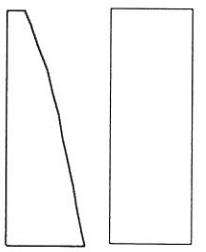


Fig.9 Two types of distribution of H.D.

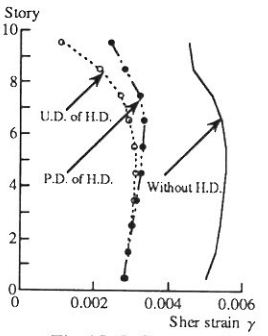


Fig.10 Influence of distribution shape.

Another kind of distribution shape of HD illustrated in Fig. 11 have been analyzed next. We here consider ten cases in which the HD are distributed in only first floor, under 2nd floor, ••••••, under 9th floor and all the floors. The shear strain responses in these ten cases have been illustrated in Fig. 12. It is obvious to know from this figure that the shear strain response is abruptly decreased in the floor where the HD is certainly attached to the main structure. The validity of HD on reducing the shear strain response could also be confirmed here.

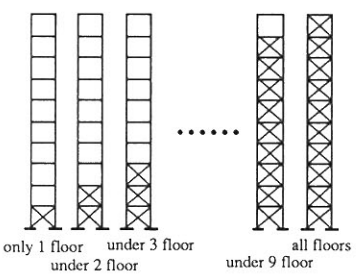


Fig.11 Various distribution of H.D.

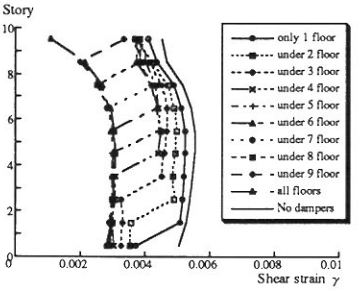


Fig.12 Influence of H.D. distribution on the response of shear strain

8. Influence of yield level of HD on the shear strain response

In this section, we will discuss the influence of different yield level Q_y of HD on the shear strain response and energy dissipating capacity. The validity of energy dissipating would be appreciated by transferred into the equivalent viscous damping coefficient by means of Eq. (8) stated in section 4.

Three kinds of yield level Q_y, *0.5%W*, *1.0%W* and *2%W* have been taken into the consideration, where W denotes the total weight of the building. Ground motion **wv_elns** has been used in the numerical response analysis.

The shear strain responses by using different yield level are plotted in Fig. 13. The shear strain response is greatly subdued when HD is used, but the influence of using different yield level is not so great compared with the decreased content from that without any HD. We know that it is sufficient to use the low yield steel with 0.5%W yield level from the figures.

The input earthquake energy and that dissipated by HD corresponding to different yield level, has been shown in Fig. 14. It is known that 50% to 70% of input earthquake energy is dissipated by the HD corresponding to the different yield level. Although the yield level of HD varies from *1%W* to *2%W*, the energy dissipated by the HD is quite near the same under this kind of ground motion.

The equivalent viscous damping coefficient obtained in line with the equivalent energy principle are shown in Fig. 15. The values of equivalent viscous damping coefficient are 5.19%, 9.43% and 14%, corresponding to the *0.5%W*, *1%W* and *2%W* yield level respectively.

9. Influence of HD second stiffness on the shear strain response

While the second stiffness of HD (Refers K_{HD2} in Fig. 2) is varied from (0 ~ 20)% of elastic stiffness K_{HD1}, the shear strain response, energy dissipating ca-

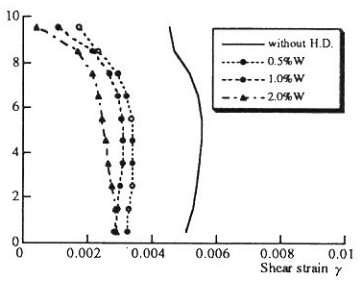


Fig.13 Influence of yield level of H.D. on the response of shear strain

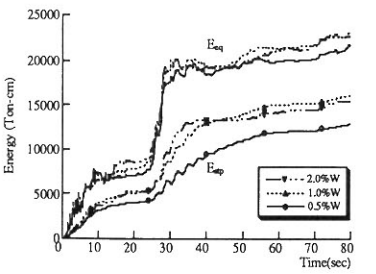


Fig. 14 Energy disipated by the H.D with different yield level

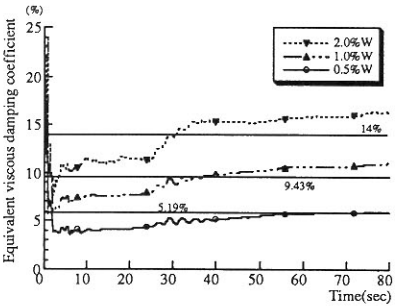


Fig. 15 Equivalent viscous damping coefficient with different yield level

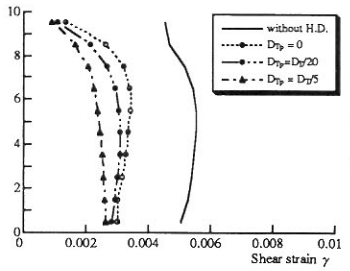


Fig. 16 Influence of second stiffness on the response of shear strain

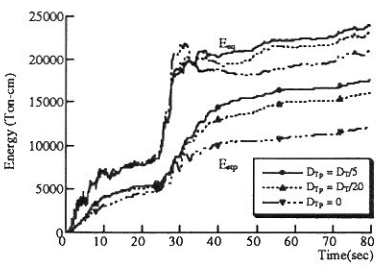


Fig. 17 Energy disipated by the H.D. with different second stiffness

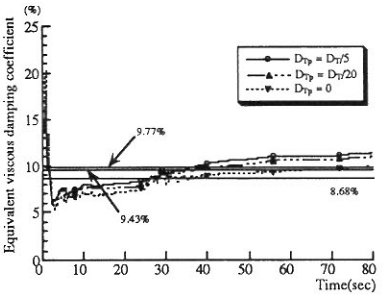


Fig. 18 Equivalent viscous damping coefficient with different second stiffness

pacity and the equivalent viscous damping coefficient have been illustrated in figures 16, 17 and 18.

Similar to the influence of yield level, the second stiffness of HD has some influence on the shear strain response, but the quantity is not so large comparing with the decreased content from that without any HD. The results said that the subduing capacity of shear strain response increases with the second stiffness of HD increases.

Fig. 17 shows the input earthquake energy and the energy dissipated by HD varied with the time. The HD dissipated energy is relatively lower if the second stiffness is 0. That will, however, increases largely and not changes extremely as long as there is some second stiffness in HD. Even that, the percentage of HD dissipated energy over the input earthquake energy seems to be near.

It does not seem from Fig. 18 that the equivalent viscous damping coefficient is influenced greatly by the second stiffness of HD, even the energy dissipated by HD is so much different (see Fig. 17) by using different second stiffness. The difference value of equivalent viscous damping coefficient is only 1% approximately, when the second stiffness of HD varies from (0% ~ 20%) of elastic stiffness.

10. Conclusion

We have confirmed that the following conclusions could be drawn from the above numerical response analysis.

(1) The HD such as shear-resistant members like braces made from extra low yield point steel have sufficient energy dissipating capacity and response subduing capacity acted like the same way of viscous damping.

(2) The HD placed on the low half part of the high-rise building is more efficient in reducing the response of displacement at the top floor than those placed on the upper half part of the buildings because the low half part is dominated by the shear deformation.

(3) It is very important that the HD should be continuously placed throughout the height of the buildings, in order to make the shear strain distribution be near the constant for the purpose of optimum design.

(4) The distribution shape of yield level and stiffness of HD throughout the height is not so important. It is suitable to adopt the uniform distribution for the safety and convenient of manufacture.

(5) The HD with 0.5%W yield shear force level is sufficient to make HD pose enough energy dissipating capacity and subduing capacity of shear strain response.

(6) It is proposed that the HD with some second stiffness are used for the sake of energy dissipating capacity, although the second stiffness of HD is not so important in reducing the shear strain response.

(7) At last, we also have to say that different ground motion has a great influence on the various response even the used artificial ground motions are created to fit for the same design spectrum of energy velocity.

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高層建物の地震応答に与える履歴ダンパーの効果

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目的：　現代社会の発展に伴い、超高層建物の社会的・経済的な価値がますます高くなっている。超高層建物の構造設計は人命を守ることだけでなく、建物の被害を最小限にすることが重要となってきた。この要求に応じて、J.J.Connor、和田章、岩田衛らの研究グループは、建物の被害レベル制御設計法を提唱している。本論文では、被害レベル制御法によって設計した超高層建物を対象にして、履歴ダンパー（HDと略記する）の地震応答に与える影響を数値解析により調べる。

解析モデル：　フレーム構造の建物を柱と梁から構成される主体構造とブレース系の二つ部分に分けて考える。被害レベル制御設計法の概念に基づき、主体構造は常に弾性状態を保たせ、地震入力エネルギーは降伏変位の小さな筋違い（HDと見なす）によって吸収させる。本論文の数値解析には、せん断と曲げ変形を独立的に分離して、複数バネと複数ダッシュポットを有する多自由度振動モデルを用いる。HDの復元力と変形との関係はバイリニア・モデルを用いる。

超高層建物の地震応答に影響を与える要因は地震動の性質や減衰機構のほか、主体構造の剛性分布が主要な要因である。建築材料を十分有効に利用できる最適設計に適用可能なため、本論の著者らによってすでに提案している高次振動モードまで考慮した最適剛性分布を用いる。

人工地震動：　数値応答解析に用いる地震動には、目標とする設計スペクトルに適合するように作成した三種類の模擬人工地震動を用いる。実際の地震動に近づけるため、人工地震動を作る際に用いた位相はEL-CENTROとHACHINOHEの地震動からFFTにより抽出した位相および一様ランダム分布位相である。設計スペクトルは超高層建物に対してより不利な長周期成分を多めに考慮して、秋山によって提案されたエネルギー等価速度スペクトルを用いた。三種類の人工地震動の最大加速度はそれぞれ303.7(Gal), 357(Gal)と181.5(Gal)である。

HDにより吸収されたエネルギーと剛性に比例する人工粘性ダンパーにより吸収されたエネルギーとの等価性の基づいて、HDの等価粘性減衰係数を表わす計算式を導いた。HDのエネルギー吸収効果は時系列エネルギーと等価粘性減衰係数により評価することができる。

履歴ダンパーの影響解析：　履歴ダンパーによる超高層建物の地震応答に与える効果を、主に以下のような４つの方面から解析した。

１）HDの建物の高さ方向における配置の影響　高さ方向の任意なi層にHDを一つ設けた場合に、頂上の横水平変位応答およびHDに吸収されるエネルギーを解析した。HDは建物の中下部に配置した場合には中上部に配置した場合より、水平変位応答を抑えることやエネルギーを吸収することに、より効果的に働く。これは超高層建物の中下部には主にせん断変形が支配されていると考えられるからである。しかし、用いる地震動の性質による影響も大きく、HDの配置影響については定量的にまだ断定できない。

２）HDの建物の高さ方向における分布形状の影響　すべての層にHDがあると仮定し、各層のHDの降伏レベルと剛性が主体構造のせん断剛性に比例する場合と均一分布する場合の二種類の分布形を考慮して応答解析を行った。高さ方向におけるHDの降伏レベルと剛性分布の違いによる影響は小さいので、部材の統一と安全面を考えると、均一分布をとることが妥当であると思われる。また、HDが1層目だけ、2層目以下、3層目以下、……、9層目以下、及び全部層に配置する10ケースを検討した。HDを配置した層と配置しない層の間にひずみの急激な変化があるため、すべての層に配置することが適当である。

３）HDの降伏レベルの変動による影響　HDにおいて、0.5%W、1%W、2%Wの三種類の降伏レベルを考慮して、せん断ひずみの応答、HDの時系列エネルギー及び等価粘性減衰係数に及ぼす影響を解析した。等価粘性減衰係数は降伏レベルが増加すると共に増える一方、せん断歪と時系列エネルギーに与える影響はあまり大きくない。(0.5～1)%Wの降伏レベルが妥当であると思われる。

４）HDの２次剛性の変動による影響　HDにおいて、0、5%K_{HDI}、20%K_{HDI}の三種類の二次剛性を考慮して、せん断ひずみの応答と時系列エネルギーおよび等価粘性減衰係数に及ぼす影響を解析した。せん断ひずみと等価粘性減衰係数に及ぼす影響はあまり大きくないが、時系列エネルギーに及ぼす影響が大きいため、5%K_{HDI}の二次剛性を有することが妥当であると思われる。

結論：　フレーム構造の超高層建物に設けた降伏変位の小さな筋違いが履歴ダンパーの機能を十分果たすことを、地震応答の低減、地震入力エネルギーの吸収について数値解析から論じた。