

EVALUATION OF SEISMIC-RESISTANT CAPACITY OF EXISTENT HIGH-RISE BUILDINGS UNDER EXTREMELY STRONG EARTHQUAKE EXCITATIONS TAKING VARIOUS DETERIORATION EFFECTS

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<https://doi.org/10.15017/1398306>

出版情報：九州大学, 2013, 博士（工学）, 課程博士
バージョン：
権利関係：全文ファイル公表済



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RISE BUILDINGS UNDER EXTREMELY STRONG EARTHQUAKE
EXCITATIONS TAKING VARIOUS DETERIORATION EFFECTS

A DISSERTATION
SUBMITTED TO THE DEPARTMENT OF ARCHITECTURE
GRADUATE SCHOOL OF HUMAN-ENVIRONMENT STUDIES
OF KYUSHU UNIVERSITY
IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE DEGREE OF
DOCTOR OF ENGINEERING

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June 2013

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ABSTRACT

The researches addressed in this thesis are mainly about the evaluation on the seismic-resistant capacity of various existent high-rise buildings subjected to extremely strong earthquake ground motions. In particular, we specifically incorporate the influences of $P-\Delta$ effect and member deteriorations induced by local failure.

Six chapters are included in this dissertation, beside the Chapter 1 for introduction and Chapter 6 for summary and conclusions, the core of the thesis consists of two parts, Chapters 2~3 are expected to calibration the accuracy of unified constitutive models for concrete and steel members by incorporating strength and stiffness deteriorations. Chapter 4~5 focus on the seismic safety and collapse capacity of various existent high-rise buildings subjected to severe seismic excitation. Details of these four chapters are introduced below.

In Chapter 2, the unified constitutive models for steel and concrete fiber materials for fiber element analysis are developed or modified in accordance with the fully calibrated and recognized models as mentioned above. Due to the effects of geometric-nonlinear and material-nonlinear ought to be fully considered in the assessment of the seismic-resistant capacity and collapse behavior of high-rise buildings under severe seismic excitations. The unified constitutive models incorporating the strength and stiffness deteriorations of structural components are respectively quantified and evaluated on the basis of an extensive database of the experimental results on H-shaped steel members, hollow steel tubular members and concrete filled steel tubular members. Calibrated results in this chapter are expected to prove the feasibility and accuracy of such deteriorating constitutive models, and support the reliability of the dynamic analyses on the seismic-resistant capacity of various existent high-rise steel and concrete filled steel tubular buildings that tend to be conducted the Chapters 4~5.

In Chapter three, a full-scale shaking table test of 4-story steel moment-resistant frames subjected to incremental earthquake loadings was used for the calibration of stress fiber model analysis employing the stress-strain models incorporating strength and stiffness deteriorations, in order to predict the collapse capacity of the steel frame under severe seismic excitation. An E-Defense shaking table test of a high-rise steel building subjected to long-period ground motions is also used for the validation of the unified constitutive model. In addition, a CFT frame specimen subjected to cyclic horizontal loadings with constant axial loading is also simulated. Although differences are included in the modelling of three dimensional frame specimen by two-dimensional plan frame model, the two dimensional frame models by adopting the unified constitutive models are reliably capable to predict the collapse and seismic responses of low-rise and high-rise steel and concrete filled steel tubular frame buildings.

In Chapter 4, the seismic-resistant capacity of existent high-rise steel buildings with various story levels subjected to long-period ground motions, near-fault ground motions and artificial ground motions are numerically investigated. The deterioration behavior of such building models is evaluated by using the unified constitutive models and taking the $P-\Delta$ effect into account. The deterioration criteria are also quantified on the basis of maximum and residual story drift angles, structural component ductility and energy dissipation. Various high-rise building models incorporating and ignoring deterioration effect are compared with each other to further assess the safety margin until collapse. The $P-\Delta$ effect and member deterioration have been proved very

significantly influence the seismic-resistant capacity and collapse behaviour of high-rise steel buildings.

In Chapter 5, we replace the square hollow steel tubular columns in high-rise steel building with the associated square concrete filled steel tubular columns by equivalent flexural stiffness, in order to assess the seismic-resistant capacity of the existent high-rise concrete filled steel tubular building that is analogous to the associated high-rise steel buildings. Extreme earthquake waves with long-period ground motions (i.e. Art-Hachi) are selected because of the larger seismic responses have been obtained in Chapter 4. In comparison with the analogous high-rise steel buildings, the safety margin until collapse of concrete filled steel tubular building structures is assessed. Larger safety margin until collapse of high-rise CFT buildings than that of high-rise steel building is obtained.

Finally, summary and conclusive remarks of the findings in Chapter 2~5 are respectively achieved in Chapter 6. According to above achieved findings, by considering the influences of $P-\Delta$ effect and strength and stiffness member deterioration effect, the existent high-rise buildings have higher probability of collapse when under long-period ground motions and pulse-like ground motions, than under earthquake waves which are used in structural design so far. $P-\Delta$ effect tends to reduce the seismic resistant capacity of the building with higher story level based on Uetani theory. Strength and stiffness member deterioration make the high-rise building suddenly collapse and reduce the safety margin until collapse of the high-rise buildings. Hence, the influences of $P-\Delta$ effect and member deterioration effect should be incorporated not only for the seismic evaluation on various existent building structures, but also for the performance-based seismic design of new constructing building structures in the future.

ACKNOWLEDGEMENTS

Firstly, I would like to express the most sincere gratitude to my thesis advisor, Professor Akihiko Kawano. His profound knowledge, modest personality and enthusiastic attitude have brought huge improvements on my confidence, level of expertise. Every time when I was in confusion and difficulties on research and life, his kindness and responsibility always help me to go through these difficult barriers. You always taught me to make things as simple as possible, which lead me to find the truth in right and efficient way. I would say ‘Thank you very much’ for everything.

I would also like to express my deep appreciation to my sub-advisors and also the examiners of my doctoral thesis, Professor Toshihiko Ninakawa, Professor Tatsuo Kanno, for giving me their invaluable and constructional comments for this dissertation. Meanwhile, I also wish to deeply appreciate Professor Koji Uetani, Professor Yoshikazu Araki, who provide research grants and patiently discuss the subjects and objectives of research program with us. Doki San and assistant professor Matsuo is also very grateful for their warm helps and sincere exchanges on daily life and research perspectives.

I would like to thank the High-Rise Building Group Crew: Yoshiyuki Yamamoto, Keita Odawara, Masayuki Fujii, Shoichi Egashira, Kouhei Hayashi, who spent the past three years to promote our research quality together. I also wish to thank other undergraduate and graduate students of Kawano laboratory who have been sharing ideas and having fun with me. I would also like to thank international group fellows, Wei Li, Nasruddin, Zamani and Qiao et al., for learning specialty knowledge from each other and good international communications.

I specially wish to thank my former advisor, Professor Guoliang Bai from Xi’an University of Architecture and Technology. Without your supports, I would have not been interested in this field.

Finally, my family members are appreciated for their selfless supports during my Doctoral period and not only years. The most regret thing is I couldn’t accompany him when my grandfather passed away. I hope he will feel pleased at my success somewhere.

Yongtao Bai

June, 2013

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CHAPTER 1

INTRODUCTION

1.1 Background

Extraordinary big earthquake induced in subduction zone attacked (Tohoku earthquake, 2011) Eastern Japan, and subsequently caused huge losses of human life and infrastructural engineering. Meanwhile, as the subduction-zone earthquake, Tokai, Tonankai, and Nankai consolidated type earthquake has also been predicted where scales of ground motions and the Tsunami are far beyond those of the past estimation. Besides, local earthquakes directly underneath inland by active faults which causes extremely strong pulse-like ground motions near the fault are predicted in Tokyo and Osaka regions in Japan these days. The same situation can be found in interior regions of China as well. Tokyo, Osaka, and Nagoya are big cities and a number of super high-rise buildings have been constructed there since 1970's. These cities are located on thick and soft alluvial beds which make the ground predominant period longer up to 2 to 5 seconds. Therefore, when strong ground motions that distinguish long period components are generated by the big subduction-zone earthquakes, the high-rise buildings are likely to resonate with such long-period ground motions. It was confirmed that the long-period ground motions were observed in Tokyo, Osaka and Nagoya during the Tohoku Earthquake (AIJ 2011), and many high-rise buildings in these regions were shaking in their predominant periods for a long time such as over five minutes. Since the early constructed high-rise buildings have not been checked against the long-period ground motions and seismic risks of such existing buildings subjected to extremely strong inland earthquakes have not been investigated as well, the seismic-resistant capacity of existing high-rise buildings should be evaluated to reconfirm the seismic safety and collapse risks of such buildings under long-period ground motions and severe seismic excitations.

For a high-rise building, the $P-\Delta$ effect due to the weight of building causes the story deformation concentration in the lower part even for the weak-beam and strong-column structural system (Uetani 1996). Moreover, the reduction of strength and the negative slope of stiffness after the maximum strength (i.e., peak point) in a structural component further cause the concentration of deformation in very local part and the serious overall collapse may be generated. Thus, both of the $P-\Delta$ effect and the strength reduction in a structural component produce synergetic effect, which may cause the fatal collapse consequence of high-rise buildings when subjected to extreme ground motions.

The mechanisms of strength reduction in steel structural components (e.g., H-shaped steel and hollow steel tube) are induced by the local buckling of plate elements and brittle fracture of component connections. The brittle fracture at welded beam-column joints is one of the serious problems in steel structures, and such a damage have been observed in a number of lower flanges

in steel girders at the column joints during Hyogoken-Nanbu Earthquake (17th, January, 1995). Although there still remains a difficulty to avoid the fracture from steel structures made of heavy steel plates, the accumulated plastic deformation of a structural component near the joint may be thought as one of the reliable indexes. A number of studies on the post-buckling behavior of steel and concrete filled steel tubular members have been addressed (e.g., Matsui et al. 1980; Nakashima et al. 1983; Yasui and Inoue 1993, etc.). In contrast, the mechanism of strength reduction in concrete filled steel tubular (CFT) components is commonly initiated by the local buckling of confined steel tube and crushing of confined concrete. However, compared with the post-buckling behavior of hollow steel tubular components, the corresponding CFT components more hardly buckle and subsequently demonstrate more stable behavior after local buckling since the confining effect of in-filled concrete. Several researches on the post-buckling behavior of CFT components are also conducted by Matsui et al. (1983); Morino et al. (1992); Uy (1998); Kimura, Kawano and Matsui (1999); Kawaguchi and Morino (2001), etc.

For the existent high-rise buildings under severe earthquake excitations, the strength reduction and stiffness deterioration of the corresponding structural components are expected to demonstrate primary influence on their seismic resistant capacity and dynamic stability. Thus, the influences of P- Δ effect and strength deterioration should be incorporated in the evaluation on the seismic-resistant capacity of various existent high-rise buildings during 1970~1990 (AIJ 2007) subjected to severe earthquake loadings, since that such high-rise buildings were seismically designed without considering the P- Δ effect and member deterioration.

1.2 Objectives

The deterioration behavior of various structural components is induced by different mechanisms as introduced in section 1.1, e.g., the local buckling of structural steel and the crushing of concrete. These deterioration factors tend to be initiated when high loading or deformation are achieved, local buckling of a steel plate element is generated by the out-of-plane instability level of local compressive stress, in contrast, fracture is tend to be produced by tensile stress (e.g., upper and lower flanges of H-shaped steel easily occur tensile fracture). Thus, the deterioration mechanisms of such structural members realistically happen in the corresponding existent high-rise building structures under severe earthquakes, and such effect should be quantified and investigated.

Generally, two aspects are expected to be addressed in this dissertation. Firstly, an unified constitutive model to simulate the deterioration behavior of various structural components which are feasible for the steel and concrete materials in associated structural components is expected to be developed in order to predict the collapse behavior of structural frames composed of various materials or structural systems with precision at the same level. As a frequently employed analysis method, stress fiber model analysis is capable of simulating the elasto-plasticity of various structural members. The accuracy of stress-strain models for structural members significantly influences the analytical results, especially in the case of utilizing the model without incorporating any deterioration effects which has a risk to overestimate the safety of frames. Thus, various strength and stiffness deteriorations are considered in stress-strain models and calibrated on the basis of extensive experimental database of structural components. The feasibility and accuracy of such unified constitutive models on simulating the global behaviour of structural frame is also validated by realistic experimental studies on frame structures.

Secondly, by utilizing the stress fiber model based on the unified constitutive model, various existent high-rise buildings are dynamically analysed on the basis of time-history analysis method

taking the geometric nonlinearity (P- Δ effect) into account. Typical structural systems of steel structure, concrete filled steel tubular structure are popular structures as high-rise buildings (Fukushima 1999), e.g., before 1980, steel structures were preferred, while after 1980, CFT structures merged and became more and more popular. Therefore, in this thesis, high-rise steel and concrete filled steel tubular buildings are analysed. The feasible unified constitutive models are employed into the corresponding high-rise building planar models, to evaluate the deterioration behavior and collapse capacity of such building structures subjected to severe seismic excitations.

Finally, based on the expected findings of this study, the evaluation on the seismic risks of existent high-rise buildings subjected to severe ground motions should simultaneously incorporate the effect of member deterioration and P- Δ effect since the seismic-resistant capacity of various high-rise buildings incorporating such effects tends to significantly soften and the collapse capacity of such building structures subsequently deteriorates which has not been taken into account in practical structural design so far. Moreover, the effect of member deterioration on the seismic performance of high-rise buildings is expected to be an important and essential influential factor that should be considered in the performance-based seismic design of high-rise buildings under unanticipated severe earthquakes.

1.3 Organization

Six chapters are included in this dissertation, beside the Chapter 1 for introduction and Chapter 6 for summary and conclusions. The core of the thesis consists of two parts, Chapters 2~3 is expected to establish and calibrate the unified constitutive model for the stress-strain relations of concrete and steel taking the effect of deterioration behavior. Chapters 4~5 focus on the seismic safety and collapse capacity of various existent high-rise buildings subjected to severe seismic excitations. Details of these four chapters are introduced below.

In Chapter 2, various stress-strain models of steel and concrete materials for stress fiber model analysis are developed or modified in accordance with the fully calibrated and recognized models as mentioned above. Due to the effects of geometric-nonlinearity and material-nonlinearity ought to be fully considered in the assessment of the seismic-resistant capacity and collapse behavior of high-rise buildings under severe seismic excitations, the stress-strain models incorporating the strength and stiffness deteriorations of structural components (e.g., H-shaped steel, hollow steel tubular members, CFT members and RC members) are respectively quantified and evaluated on the basis of an extensive database of the experimental results on H-shaped steel members, hollow steel tubular members, CFT members, and RC members. Calibrated results in this chapter are expected to prove the feasibility and accuracy of such deteriorating stress-strain models, and support the reliability of the dynamic analyses on the collapse-resistant capacities of various existent high-rise steel, CFT and RC buildings that tend to be conducted in the Chapters 4~5.

In Chapter 3, the feasibility and accuracy of such stress-strain models incorporating member deterioration on frame structures necessitate simulating the frame structures under seismic excitations, to evaluate the effect of member deterioration on the seismic-resistant capacity of structural frame. (write the overview of chapter 3)

In Chapter 4, existent high-rise steel buildings with various heights are analysed subjected to long-period ground motions, near-fault ground motions, as well as artificial ground motions. The deterioration behavior of such building models is evaluated, and the deterioration criteria are also quantified on the basis of maximum and residual drift and component ductility.

In Chapter 5, we only replace the square hollow steel tubular columns in high-rise steel building with the associated square CFT columns by equivalent flexural stiffness, in order to assess the seismic-resistant capacity of the existent high-rise CFT building that is analogous to the associated high-rise steel buildings. A comparative study on the collapse margin of such two building systems is conducted to further quantify the effect of member deterioration between square hollow steel tubular columns and CFT columns on the corresponding high-rise buildings.

In Chapter 6, due to the confining effect on the core concrete and longitudinal steel bar by hoop/stirrup reinforcement is significantly influence the crushing of confined concrete and the buckling of longitudinal steel bar, a parametric study based on the hoop/stirrup ratio (p_w) is conducted to evaluate the seismic safety of existent high-rise RC buildings subjected to severe earthquakes. As mentioned in Chapters 4 and 5, the seismic safety of high-rise RC buildings is comprehensively judged by engineering demand parameters and earthquake intensity.

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CHAPTER 2

UNIFIED CONSTITUTIVE MODELS OF VARIOUS STRUCTURAL COMPONENTS

2.1 Introduction

The elasto-plastic behavior of various structural members comprehensively determines strength, deformation, ductility and local failure as well. The uncertainty of the local-failure of structural members seems significantly large, unlike those of the strength and deformation. For example, for the sectional steel members [e.g., H-shaped steel, hollow steel tube, and concrete filled steel tube (CFT)], local buckling could be seen as a limit-state that if the steel member could be further capable of resisting forces (e.g., static and dynamic loads). Hence, the strength deterioration of above steel members after local-buckling occurring would have important effect on their seismic resistant capacity. Meanwhile, in terms of the concept of deformability in steel members, it is important how small is the deformation level when the local buckling occurs, how steep is the negative slope in load-deformation relations after local buckling, and how small the residual strength level after the deterioration has completed. Similarly, concrete portion caused by the flexural buckling of longitudinal steel bars and the crush of concrete induces similar phenomenon of deterioration. Thus, the deformation level until peak strength, the post-peak residual strength and the gradient of negative slope of various structural members (i.e., H-shaped steel member, hollow steel tubular members and CFT members) should be evaluated and quantified, in order to accurately predict the deformability and the deterioration behavior of structural members subjected to various loading procedures.

As a frequently adopted analysis method, stress fiber model analysis based on a method that discretizes the beam-column into longitudinal stress fibers, so that the sectional force-deformation relation is derived by integration of the stress-strain relations of the fibers under the assumption that the plane remains plane in cross section. Thus, the nonlinear stress-strain relation models of steel and concrete stress fibers determine the elasto-plastic behavior of each beam-column element. Structural components such as beams and columns consist of a number of beam-column elements. The length of an element being expected to deform plastically is adjusted to be the same as the depth of cross section D , which enable to calculate the deterioration behavior in a stable manner. In other words, the stress-strain relation models of stress fibers calibrated to express the strength deteriorations in a structural member is based on the condition that the length of a beam-column element is assumed as the plastic region D .

The seismic-resistant capacity of framed structures composed of beams and columns is further entirely affected by the constitutive model taking the effect of strength deterioration. The geometric nonlinearity is also taken into account in the frame analysis, in order to consider the P - Δ effect.

Several researches on the nonlinear behaviors of plain and confined concrete have been

conducted, e.g., Popovics (1973) proposed a basic stress-strain relation of plain (unconfined) concrete covering the range of elasticity to plasticity. Since then, Mander et al. (1988) theoretically and experimentally studied the confinement effect of concrete by lateral steel bar hoop/stirrup in RC members, and modified the Popovics model of concrete to be feasible for confined concrete. On the other hand, Sargin et al. (1971) studied the effects of lateral reinforcements upon the stress and deformation behavior of concrete and proposed stress-strain model of confined concrete by adopting a normalized parameters ($X=\varepsilon/\varepsilon'_{cc}$, $Y=\sigma/\sigma'_{cc}$). Sakino and Sun (1994) defined the parameters (A, D) of Sargin stress-strain model into W and V based on experimental researches, by considering the confinement effects based on effective confined stress (σ_{re}) in order to be feasible for defining the confined concrete by various lateral confinements (steel hoop/stirrup, rectangular and circular steel tubes).

Likewise, with respect to the constitutive model of steel member, several important researches have been conducted by the associated researchers. Initially, Ramberg and Osgood (1943) suggested that the stress-strain curve of steel could be described by three parameters, namely, Young's modulus and two secant yield strengths. Menegotto & Pinto (1973) proposed a nonlinear formula to define the skeleton and hysteresis stress-strain relation for steel, while the unloading stiffness deterioration and Bauschinger effect were not incorporated in his model. Kato and Akiyama (1973) claimed that the hysteresis curve of steel should be subdivided into three components, namely, a part of skeleton curve, the elastic unloading curve and softened curve based on Bauschinger effect. Besides, Wakabayashi and Shibata (1976) proposed a bi-linear stress-strain model which is feasible for sectional steel and steel bar, by considering Bauschinger effect. Meng and Ohi (1992) introduced a parameter Ψ to define a rule of the shift of skeleton curve to be connected with unloading curve in hysteresis curve of steel.

In Chapter 2, a sequence of unified constitutive models for stress-strain relations of steel and concrete are defined by referring to the fully calibrated and recognized models as mentioned above. The effects of geometric nonlinearity and material nonlinearity should be fully considered in the assessment of the seismic-resistant capacity and collapse behavior of high-rise buildings under severe seismic excitations. Hence, the stress-strain models incorporating the strength and stiffness deteriorations of structural components are respectively quantified and evaluated based on an extensive database of the experimental results of H-shaped steel members, hollow steel tubular members, CFT members. The calibrated constitutive models will prove the feasibility and accuracy of such deteriorating stress-strain relations, and support the reliability of the dynamic analyses on the collapse-resistant capacities of various existent high-rise steel and CFT buildings that tend to be conducted the Chapters 4~5.

2.2 Analysis Theory and Fiber Discretization

2.2.1 Theory and method of analysis

Fiber element analysis program developed by Kawano (1995) is employed to carry out the nonlinear analysis of various structural members under various loading procedures. Details of the analysis algorithm could be referred to (Kawano 1995). The relation between generalized stress and strain is briefly introduced as follows.

Plane assumption of the section makes the element satisfy the following equation referring to Figure 2.1 (a):

$$\varepsilon = \varepsilon_0 - y\kappa \quad (2.1)$$

The stiffness of cross sections is numerically integrated by dividing the section into a number of layers which comprise the stress fibers. The stress in the j -th fiber is defined by instantaneous stress-strain relation, which is a function of the elasto-plastic strain ε_{ep} as follows:

$$\sigma_j = \sigma_j(\varepsilon_{epj}) \quad (2.2)$$

Where, σ_j is the stress at the j -th stress fiber, and $\sigma_j(\varepsilon_{epj})$ indicates the instantaneous stress-strain relation for j -th fiber in the section. With respect to ε_{epj} , the elasto-plastic stress at the j -th stress fiber, due to that ε incorporates three components (i.e. elasto-plastic strain ε_{ep} , creep strain ε_{cr} and shrinkage strain ε_{sh}), we ignore the effects of creep and shrinkage, therefore, ε only includes ε_{ep} as follows:

$$\varepsilon_{epj} = \varepsilon_0 - y_j K \quad (2.3)$$

The stress resultants are numerically derived by summing the stress of all fibers in the section:

$$S = \begin{bmatrix} N \\ M \end{bmatrix} = \begin{bmatrix} \sum_j A_j \sigma_j \\ -\sum_j A_j y_j \sigma_j \end{bmatrix} \quad (2.4)$$

Where, N is the axial force, M is the bending moment, and A_j is the sectional area of the j -th fibre. S is the vector of generalised stresses. The assumed stress of the section [see Figure 2.1 (b)] is finally concentrated on centroid point.

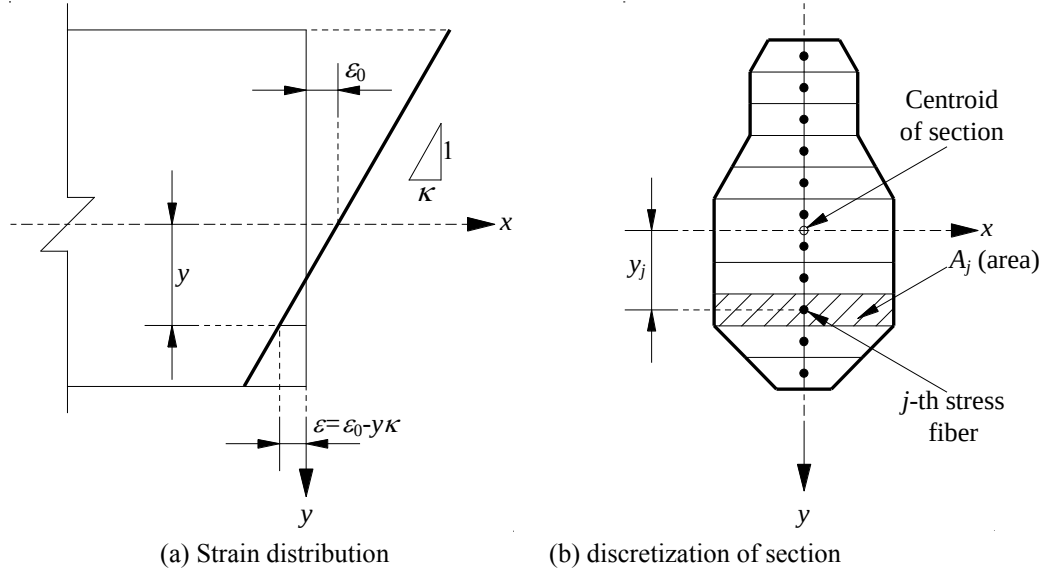


Figure 2.1 Plane section assumption and fiber discretization of section

Geometric nonlinearities are considered using an updated Lagrangean formula with a local coordinate axis for each element, which moves along with the element within the global coordinate axis system. The element stiffness is evaluated by the means of Gaussian numerical integrals using three Gaussian points (sections).

2.2.2 Fiber discretization

(a) Block sectional discretization

Rectangular sectional members could be subdivided into block sectional elements. The stress fiber is defined to be located at the centroid of an element and to have the sectional area of the

elements. Considering the simplicity of model and running duration of analysis program, fiber layers are only subdivided along with horizontal direction. The horizontal fibers of the steel webs of steel tube and H-shaped steel and the concrete core in rectangular CFT section are subdivided into eight layers. The horizontal fibers of the steel flange of steel tube and H-shaped steel are subdivided into two layers. In particular, the fiber discretization for the rectangular hollow steel tubular and CFT sections and the H-shaped steel section are demonstrated in Fig. 2.2, respectively. Where, B is the outer width of steel tube, t is the thickness of steel tube.

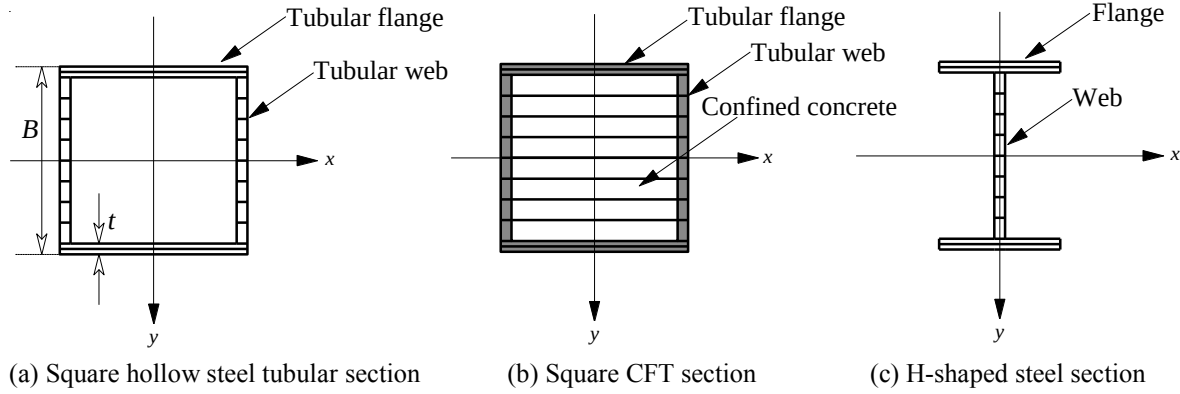


Fig. 2.2 Block sectional discretization

(b) Ring sectional discretization

For circular sectional member, the rule of ring sectional discretization is adopted to subdivide fiber layers along with the thickness of ring and the circular hoop of ring. As shown in Fig. 2.3, the discretization of the ring fibers for the circular steel tubes in hollow steel tubular and CFT sections, are subdivided into eight layers in circular hoop direction, and two layers in thickness direction. For the concrete core in circular CFT section, four fiber layers are subdivided.

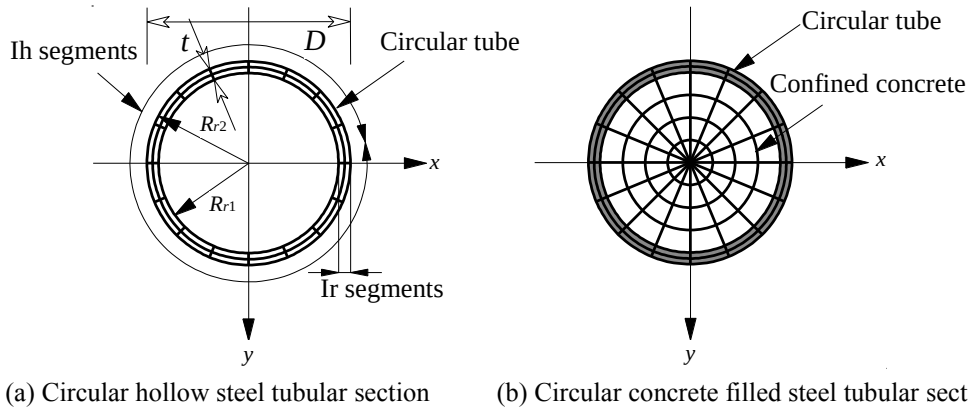


Fig. 2.3 Ring sectional discretization

2.2.3 Longitudinal element discretization

The plastic deformation may be thought to concentrate at the regions of beam ends and/or column ends in high-rise multi-story buildings. The beam-column element at the plastic region should have an appropriate length, so that the deterioration behavior strongly affected by the element length. Ideally, the length of the element should vary according to the sectional shape, a kind of material and the structural system (i.e. H-shaped steel section, square steel tubular section, circular steel tubular section, square CFT section, circular CFT section), because the mechanism of

the strength deterioration, that is the size of local instable deformation region, are widely different from each other (i.e., local buckling of a steel plate element, crush of concrete, and confining effect between concrete and steel). It is well known that the axial force ratio affect the size of local instable deformation region as well. After having considered these complexities enough, the length of an element in plastic region is assumed as large as the depth of the member cross section D . In other words, the stress-strain relation models of various cross sections and various axial force ratios are calibrated to express the strength deteriorations in structural members and frames under the condition of the element length of D as shown in Figure 2.4~2.5.

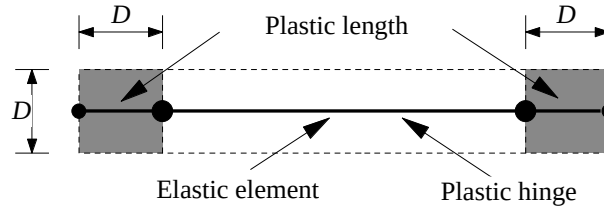


Figure 2.4 Longitudinal discretization of element for S and CFT sections

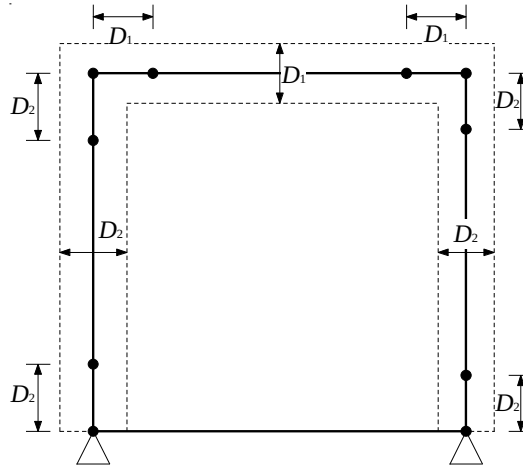


Figure 2.5 Example of longitudinal discretization of elements in steel and CFT frames

2.3 Constitutive Model for Concrete

Confining effect on the confined concrete of CFT members improves the strength or/and deformability to certain degree, as shown in Figure 2.6. In particular, two types of confined concrete exist according to lateral confining intensity, namely, highly confined concrete such as in-filled concrete in circular steel tube whose strength and deformability are both improved, and partially confined concrete such as in-filled concrete in a square steel tube whose only deformability is increased.

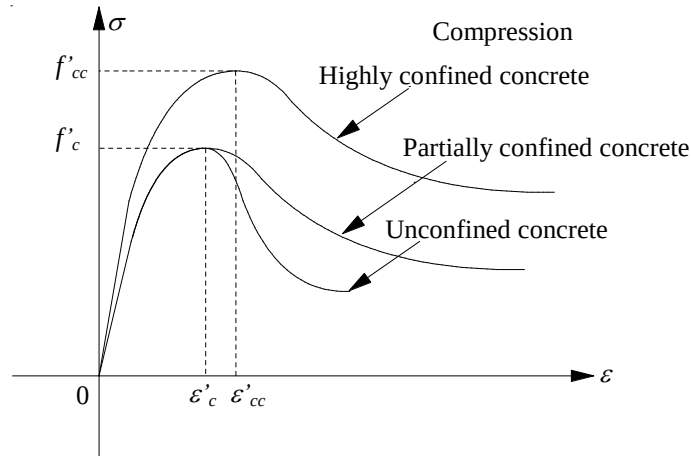


Figure 2.6 Stress-strain relation of confined and unconfined concrete

2.3.1 Skeleton and hysteresis rules for stress-strain relation of concrete

The skeleton curve for stress-strain relation of concrete is assumed to consist of three branches, which are the ascending pre-peak curved line, the straight line for post-peak deterioration, and the glowing straight line for residual strength after completing deterioration as shown in Figure 2.7 (a). The reason why two straight lines are used to express after the post-peak behavior is to keep the simplicity with certain acceptable accuracy. From the numerical study, there is no significant difference whether the post-peak behavior is assumed by curved line or multi straight lines. It is unified that the skeleton is approximated by one curve line and following two straight lines for not only concrete but also steel. The skeleton in Figure 2.7 (b) denotes the case without the strength deterioration which is used for the comparative discussion in the alter chapters.

The hysteresis rule of confined concrete model is based on the relation that residual strain (ε_{re}) at the intersection point between unloading curve and strain coordinate ($\sigma=0$) is equal to a half of the unloading strain (ε_{un}) at unloading point (see Figure 2.7). According this rule, the unloading stiffness deterioration of concrete material can be simulated in the hysteretic curve. Furthermore, the stress-strain model incorporating strength deterioration [(Fig. 2.7 (a))] shows larger unloading stiffness deterioration than that of ignoring strength deterioration [(Fig. 2.7 (b))].

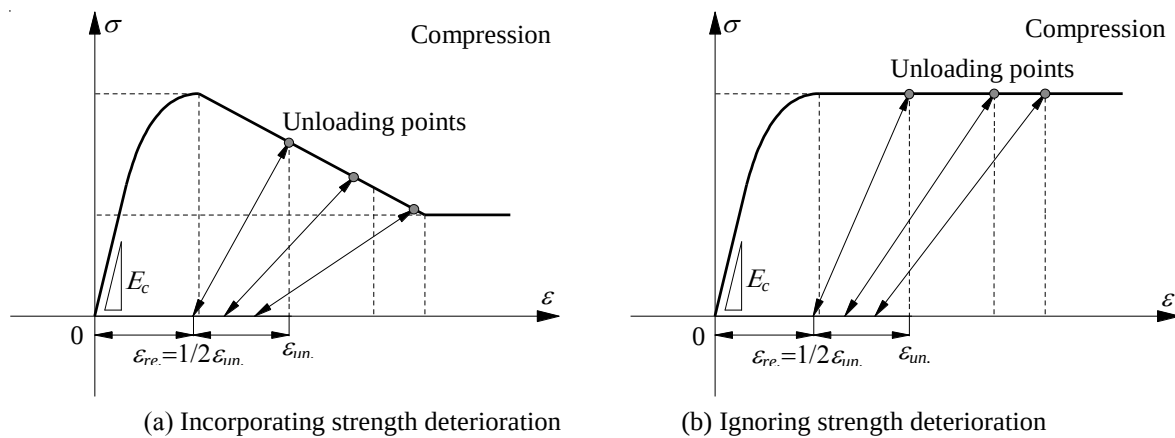


Figure 2.7 Skeleton and hysteresis rule of the concrete model

2.3.2 Pre-peak branch

The stress-strain relation model of concrete is normalized by the peak-point (f'_c, ε'_c) of partially concrete, or the peak-point ($f'_{cc}, \varepsilon'_{cc}$) of highly confined concrete, into X ($=\varepsilon/\varepsilon'_c$ or $\varepsilon/\varepsilon'_{cc}$)- Y ($=\sigma/f'_c$ or σ/f'_{cc}) relation, in order to simply describe the confinement effect on strength, deformability and strength deterioration. Normalized model of concrete is shown in Fig. 2.8.

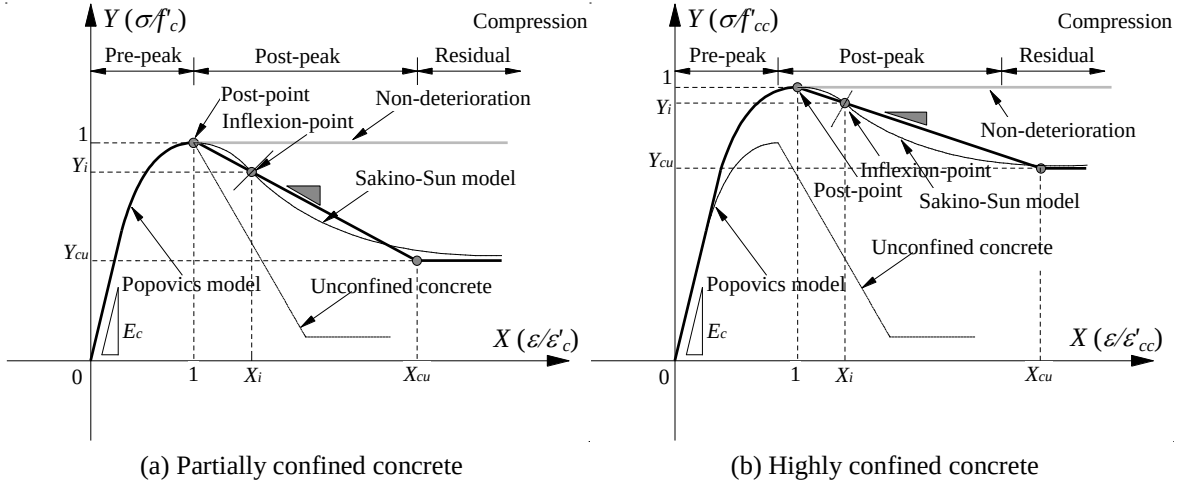


Figure 2.8 Normalized constitutive model of concrete

With respect to the pre-peak branch in the stress-strain relation of the unconfined or partially confined concrete ($X < 1$), Popovics's proposal (1973) is assumed as Eq. (2.5) which is based on

$$\sigma = f'_c \cdot \frac{r_c (\varepsilon/\varepsilon'_c)}{r_c - 1 + (\varepsilon/\varepsilon'_c)^{r_c}}, \quad r_c = \frac{E_c}{E_c - (f'_c/\varepsilon'_c)} \quad (2.5)$$

Normalizing Eq. (2.5) into X - Y coordinate to be

$$Y = \frac{VX}{1 + (V-1)X^{\frac{V}{V-1}}} \quad (2.6)$$

$$\text{Where, } X = \frac{\varepsilon}{\varepsilon'_c}, Y = \frac{\sigma}{\sigma'_c}, V = \frac{E_c \cdot \varepsilon'_c}{f'_c}$$

Where, ε'_c and f'_c ($=0.85\sigma_B$) respectively are the strain and stress at the peak point of partially confined concrete model. E_c is the Young's modulus of concrete. X ($\varepsilon/\varepsilon'_c$) is a coefficient normalized by the strain of the peak point (ε'_c). Y ($=\sigma/f'_c$) is a coefficient normalized by the strength of the peak point (f'_c). V is a function of the secant modulus and tangent modulus of concrete, based on Popovics model.

The ending point of pre-peak branch, namely, peak-point, is calculated by the formulae as shown in Table 2.1 from unconfined concrete to highly confined concrete. In particular, the effective lateral confined stress (σ_{re}) is considered on the increasing of the strength and the corresponding strain at strength [i.e., the peak-point ($f'_{cc}, \varepsilon'_{cc}$)] of the highly confined concrete in circular CFT members based on the formulae (Sakino and Sun, 1994) by

$$\frac{f'_{cc}}{f'_c} = K = 1 + 23 \cdot \frac{\sigma_{re}}{f'_c} \quad (2.7)$$

$$\frac{\varepsilon'_{cc}}{\varepsilon'_c} = \begin{cases} 1.0 + 4.7(K-1) & , \quad K \leq 1.5 \\ 3.4 + 20(K-1) & , \quad K > 1.5 \end{cases} \quad (2.8)$$

$$\sigma = f'_{cc} \cdot \frac{r_c (\varepsilon / \varepsilon'_{cc})}{r_c - 1 + (\varepsilon / \varepsilon'_{cc})^{r_c}} \quad , \quad r_c = \frac{E_c}{E_c - (f'_{cc} / \varepsilon'_{cc})} \quad (2.9)$$

$$\text{Where, } X = \frac{\varepsilon}{\varepsilon'_{cc}}, Y = \frac{\sigma}{\sigma'_{cc}}, V = \frac{E_c \cdot \varepsilon'_{cc}}{f'_{cc}}$$

However, the confining effect on improving the peak point for the confined concrete in square CFT member could be ignored (Sakino 2004). So with in pre-peak branch, the behavior of the confined concrete in square CFT member could be treated the same as that of plain concrete.

Table 2.1 Parameters for unconfined and confined concrete

Parameters	Unconfined concrete	Confined concrete in CFT member	
		Square CFT	Circular CFT
X		$\varepsilon / \varepsilon'_{cc}$	
Y		σ / f'_{cc}	
V		$E_c \times \varepsilon'_{cc} / f'_{cc}$	
$\varepsilon'_{cc} / \varepsilon'_c$	1	1	$\begin{cases} 1.0 + 4.7(K-1) & , \quad K \leq 1.5 \\ 3.4 + 20(K-1) & , \quad K > 1.5 \end{cases}$
$K = f'_{cc} / f'_c$	1	1	$\frac{f'_{cc}}{f'_c} = 1 + 23 \cdot \frac{\sigma_{re}}{f'_c}$
σ_{re}	0	$2 \cdot \sigma_y \left(\frac{B}{t} - 1 \right) \cdot \left(\frac{B}{t} - 2 \right)^{-3}$	$0.068 \cdot \sigma_y \left(\frac{D}{t} - 2 \right)^{-1}$
W	$1.50-17.1 \times 10^{-3} f'_c$	$1.50-17.1 \times 10^{-3} f'_c + 2.39 \times \sigma_{re}^{0.5}$	$1.50-17.1 \times 10^{-3} f'_c + 1.59 \times \sigma_{re}^{0.5}$

$$f'_c = 0.85 \sigma_B$$

$$\varepsilon'_c = 2.62 k_p^{0.25} \times 10^{-3}$$

$$E_c = (6.90 + 25.72 k_p^{0.5}) \times 10^3$$

$$k_p = f'_c / 60 (\text{MPa})$$

Note: σ_B is cylinder strength of concrete; f'_c is strength of plain and partially confined concrete; f'_{cc} is strength of highly confined concrete; ε'_c is strain at strength of plain and partially confined concrete; ε'_{cc} is strain at strength of highly confined concrete; σ_{re} is effective lateral confined stress provided by steel tube; B is outer width of rectangular steel tube; D is outer diameter of circular steel tube; t is thickness of steel tube.

2.3.3 Post-peak branch

The stress of concrete declines after achieving the peak point (f'_{cc} , ε'_{cc}), as shown in Fig. 2.8. Even though the confining effect on the confined concrete by steel tube or hoop/stirrup could have growth on the strength and deformability of confined concrete, the strength and stiffness also overtly deteriorate under large deformation range and/or cyclic loadings. Thus, defining the post-peak branch of stress-strain skeleton model of concrete is important to be capable to capture its deterioration behavior.

(a) Model simplicity

As shown in Fig. 2.8, in order to describe the deterioration of confined concrete, the gradient of the descending slope (τE_c) is simplified into bi-linear curve based on nonlinear Sakino-Sun model (1994) proposed by Sargin formula (1973) expressed below:

$$Y = \frac{VX + (W - 1)X^2}{1 + (V - 2)X + WX^2}, \quad (X \geq 1, Y \geq 1) \quad (2.10)$$

Thus, from the Fig. 2.8, the post-peak branch is determined by two characteristic points which are the peak point (1, 1), and the inflexion point (X_i, Y_i) based on Eq. (2.10). The deterioration slope is calculated by the peak point (1, 1) and the inflexion point (X_i, Y_i) as shown in Fig. 2.9 (a), as expressed by

$$\tau E_c = \frac{1 - Y_i}{1 - X_i} \quad (2.11)$$

The deterioration slope could be defined by bi-linear curve, which are expressed by

$$\begin{cases} Y - 1 = \tau E_c (X - 1) \\ Y_{cu} = \frac{\sigma'_{cu}}{f'_{cc}} = \lim_{X \rightarrow \infty} \frac{VX + (W - 1)X^2}{1 + (V - 2)X + WX^2} = 1 - \frac{1}{W} \end{cases} \quad (2.12)$$

Hence, the crossover point C (X_{cu}, Y_{cu}) of the bi-linear curve defined by Eq. (2.12) can be determined. In Eq. (2.12), the inflexion point (X_i, Y_i) have to be derived firstly to determine the τE_c . As shown in Fig. 2.9 (b), the lowest point of $Y'(X)$ versus X is the inflexion point (X_i, Y_i).

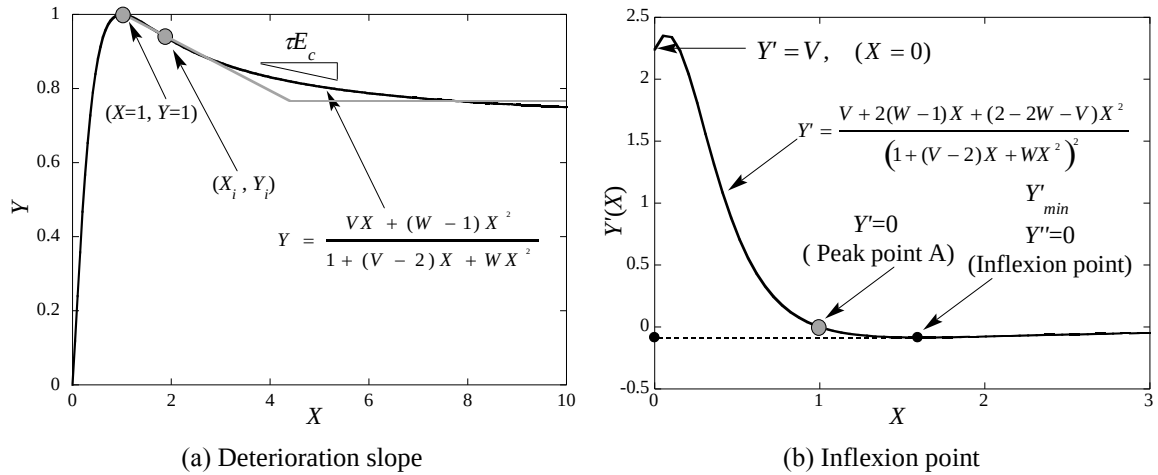


Figure 2.9 Typical curves of the Sargin's equation

(b) Regression analysis

Multivariable regression analyses are conducted, in order to propose a formula for defining the X_{cu} ($=\varepsilon'_{cu}/\varepsilon'_{cc}$) value of the confined concrete. For confined concrete, the regression parameters W and V are the coefficients in Sakino-Sun proposal based on Sargin's equation. According to the concept of inflexion point in the curve of Sargin's polynomial equation as shown in Fig. 2.8, the derivative of Eq. (2.10) is to be

$$\frac{dY}{dX} = \frac{V + 2(W - 1)X + (2 - 2W - V)X^2}{[1 + (V - 2)X + WX^2]^2} \quad (2.13)$$

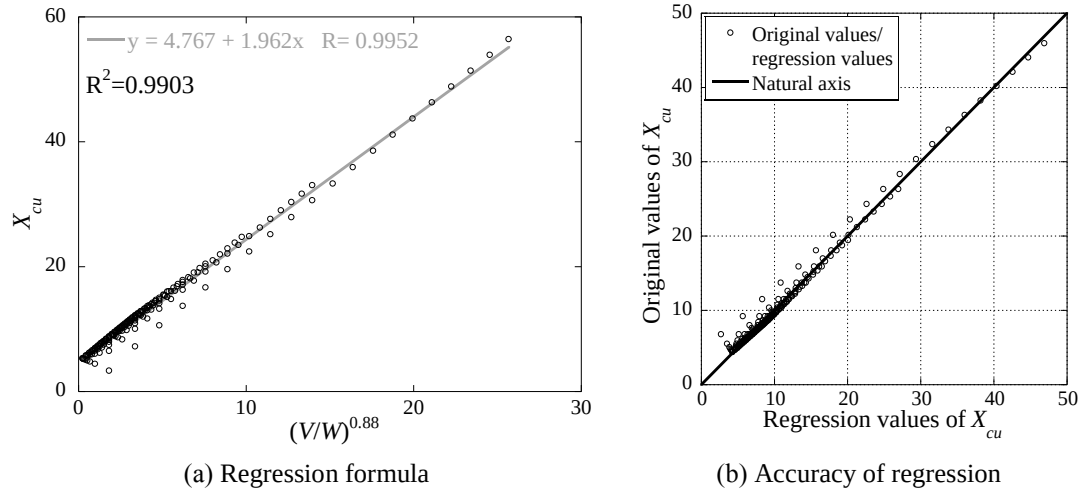
The applied range of the regression parameters of W, V needs to be determined. According to the relations between W to f'_c, σ_{re} and V to f'_c , the applicable ranges of W and V are calculated by the corresponding range of f'_c, σ_{re} as shown in Table 2.2.

Table 2.2 Application range of regression parameters of W and V

Regression parameter		Confined concrete							
		Square CFT member				Circular CFT member			
W	V	f'_c	σ_{re}	W	V	f'_c	σ_{re}	W	V
0.5	1	100N/mm ²	0.052 ($f_v=200\text{N/mm}^2, B/T=90$)	0.49	1.23	100N/mm ²	0.14 ($f_v=200\text{N/mm}^2, D/T=100$)	0.63	1.36
5	20	20N/mm ²	1.85 ($f_v=800\text{N/mm}^2, B/T=30$)	4.66	2.24	40N/mm ²	1.23 ($f_v=600\text{N/mm}^2, D/T=35$)	2.98	23.84

As shown in Table 2.2, the multiple regression analysis is conducted under the ranges of $W=0.5\sim 5$ and $V=1\sim 20$. The regression formula and its accuracy are shown in Fig. 2.10. As shown in Fig. 2.9 (a), based on Eq. (2.12), the formula of value X_{cu} which represents the ultimate strain of confined concrete is proposed to be

$$X_{cu} = \frac{\varepsilon_{cu}}{\varepsilon_{cc}} = 1.96 \cdot \left(\frac{V}{W}\right)^{0.88} + 4.77 \quad (2.13)$$

**Figure 2.10** Regression analysis and accuracy of regression result

Thus, the point (X_{cu}, Y_{cu}) can be determined by Eq. (2.11) and Eq. (2.13). The whole unified stress-strain models for the confined concrete in circular and square CFT columns considering the effect of strength deterioration are proposed by the methods of multiple regression and comparison between experimental and analytical results.

2.3.4 Residual branch

After descending post-peak branch, a horizontal residual branch is the second part of the bi-linear curve, as shown in Fig. 2.8. As we discussed in Sec. 2.3.3, the limit value of Y when $X \rightarrow \infty$ could be considered as the Y_{cu} of the horizontal line of residual branch.

2.4 Constitutive model for Steel Members

2.4.1 Local buckling of steel plate element

The local buckling patterns of hollow steel tubular and concrete filled steel tubular components are obviously different due to the confinement effect by in-filled concrete. For unfilled rectangular or square bare steel tubes under axial compression, the local buckling pattern consists

of inward and outward buckles (Bridge and O'Shea 1999). For steel tubes being in-filled by concrete, the inward buckling is effectively prevented, as shown in Fig. 2.11. Thus, the local buckling pattern of steel tubes and its influence on strength deterioration with or without internal confinement needs to consider.

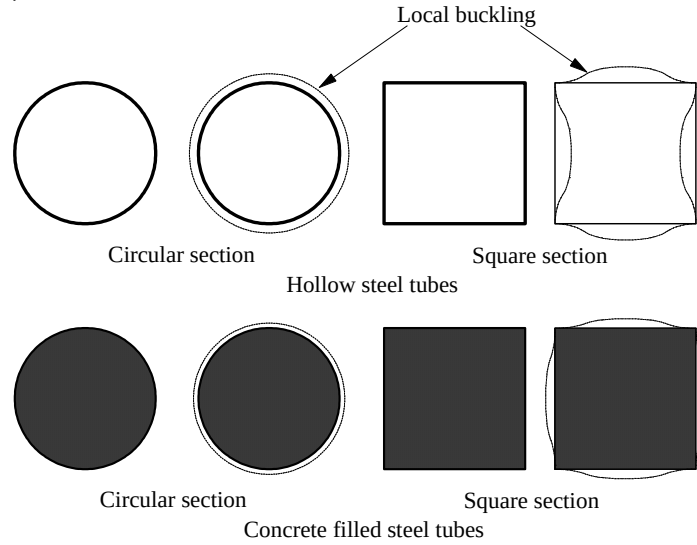


Figure 2.11 local buckling patterns of hollow steel tubular and CFT columns

2.4.2 Skeleton of stress-strain relation

With respect to the stress-strain model of steel components, several proposals have been conducted, e.g., Ramberg-Osgood model (1943) and Menegotto-Pinto [denoted as M-P model (1973)], etc. M-P model is adopted here, and the formula of M-P model is expressed by

$$\frac{\sigma}{\sigma_y} = \frac{\left(1 - \frac{E_{s,\infty}}{E_s}\right) \cdot \frac{\varepsilon}{\varepsilon_y}}{\left[1 + \left(\frac{\varepsilon}{\varepsilon_y}\right)^R\right]^{\frac{1}{R}}} + \left(1 - \frac{E_{s,\infty}}{E_s}\right) \cdot \frac{\varepsilon}{\varepsilon_y} \quad (2.14)$$

Based on Eq. (2.14), two M-P skeleton curves respectively being scaled by the yield strength (σ_y) and ultimate strength (σ_u) are combined together to describe the yield point elongation and stress hardening after steel components achieve their yield strength, see Figure 2.12.

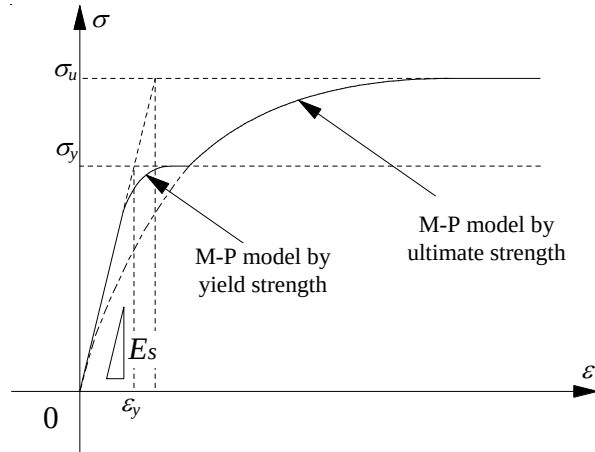


Figure 2.12 Composition of Two M-P curves in skeleton

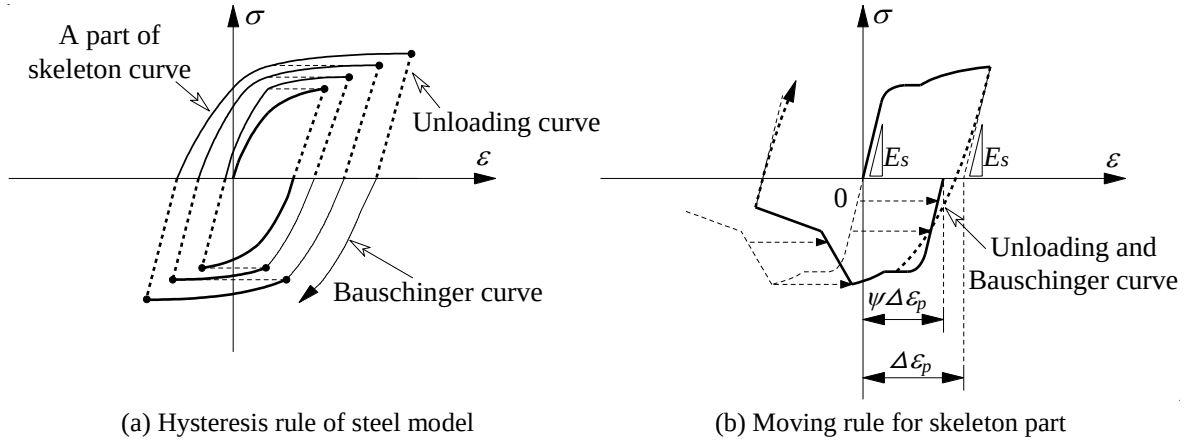


Figure 2.14 Constitutive relation models for steel tubular

2.4.4 Post-peak branch for H-shaped steel

Local buckling of sectional H-shaped steel under bending or axial loading induces a strength and stiffness deterioration of such steel components. The strain at the local buckling (ε_{lb}) of H-shaped steel component is calculated on the basis of the formulae as shown in Table 2.3 proposed by Yamada et al. (1993), in which, post-peak deterioration is controlled by parameters α_w and α_f which are based on width-to-thickness ratio of flange B/t_f and web $(D-2t_f)/t_w$, and the yield strength of steel σ_y . Thus, three deterioration parameters (i.e., ε_{lb} , γ_{spm} , τ_{lb}) are respectively calculated by Yamada's proposal (1993). Besides, on the basis of statistical data on the residual stiffness factor after full deterioration (τ_{re}) of steel (-0.0025~-0.0075), the stiffness factor τ_{re} is assumed to be -0.003 for simplicity.

Table 2.3 Stress-strain model of H-shaped steel and steel bar

Deterioration Parameter	H-shaped steel
α_w	$\alpha_w = [(D-2t_f)/t_w]^2 \times \varepsilon_y$
α_f	$\alpha_f = [B/t_f]^2 \times \varepsilon_y$
ε_{lb}	$\max \left[\frac{0.18}{\alpha_f} + \frac{2.6}{\alpha_w} + 0.3, \frac{0.5}{\alpha_f} + \frac{5.7}{\alpha_w} - 4.0 \right] \cdot \varepsilon_y$
γ_{spm}	$\gamma_{spm} = -0.062\alpha_w - 0.56\alpha_f + 0.98$
τ_{lb}	$-0.0046\alpha_w^2 - 0.57\alpha_f^2 - 0.0005$
τ_{re}	$\tau_{lb2} = -0.003$

Note: B is width of H-shaped steel; D is height of H-shaped steel; t_f is thickness of the flange of H-shaped steel; t_w is thickness of the web of H-shaped steel; ε_{lb} is strain at local buckling; γ_{spm} is strength deterioration factor after local buckling; τ_{lb} is deterioration stiffness ratio of negative slope after local buckling; τ_{re} is residual stiffness ratio of after deterioration.

2.4.5 Post-peak branch for hollow steel tubes

Steel tubular components are also frequently adopted as columns in existent high-rise buildings (AIJ 2007). Thus, the deterioration parameters of various hollow and concrete filled steel

tubular structural components are needed to be determined to simulate the post-peak deterioration of such members.

Table 2.4 Deterioration parameters of the stress-strain model of various steel tubular components

	Square steel tube		Circular steel tube	
	Hollow	Concrete filled	Hollow	Concrete filled
n	-	-	-	$n < 0.4$
ε_{lb}	$[(8.7/\alpha) - 1.2] \times \varepsilon_y$	$[6.20 \times (\alpha^2 \times \varepsilon_y)^{-1.4} + 1] \times \varepsilon_y$	$0.205 \alpha^{-1.39} \times \varepsilon_y$	$(0.18 \alpha^{-1.40} + 1) \times \varepsilon_y$
γ_{spm}	$-0.079\alpha + 0.81$	$-0.079\alpha + 0.81$	$3.37 \alpha^{-0.07} - 3.576$	$3.37 \alpha^{-0.07} - 3.576$
τ_{lb}	$-0.014\alpha^2 - 0.005$	$-0.014\alpha^2 - 0.005$	$-0.12 \alpha^{-0.48} + 0.011$	$-0.12 \alpha^{-0.48} + 0.011$
τ_{re}	-0.005	-0.001	-0.005	-0.001
α	$(B/t)^2 \times \varepsilon_y$	$[(1/1.32) \times (B/t)]^2 \times \varepsilon_y$	$(D/t) \times \varepsilon_y$	$(D/t) \times \varepsilon_y$

Note: B is outer width of rectangular steel tube; D is outer diameter of circular steel tube; t is thickness of the steel tube; ε_y tensile yield strain of steel material; ε_{lb} is strain at local buckling; γ_{spm} is strength deterioration factor after local buckling; τ_{lb} is deterioration stiffness ratio of negative slope after peak point; τ_{re} is residual stiffness ratio of after deterioration.

(a) *Rectangular hollow steel tube*

Various deterioration parameters of the rectangular hollow steel tubes can be determined on the basis of the proposal by Yamada et al. (1993), as shown in Table 2.4.

(b) *Circular hollow steel tube*

The strain at the local buckling ε_{lb} of circular hollow steel tubes is proposed by Ogawa et al. (1995) as shown in Table 2.4. The strength deterioration of circular steel tube after local buckling ($\varepsilon \geq \varepsilon_{lb}$) is simplified into bi-linear relation as shown in Figure 2.15. Based on Ochi's proposal (1988) on the post-buckling behavior of circular hollow steel tube, A bilinear stress-strain relationship after local buckling can be derived as shown in Table 2.4.

2.4.6 Post-peak branch for concrete filled steel tubes

(1) *Rectangular concrete steel tube*

For the local-buckling of rectangular CFT members, Kawano (2006) focused on the criteria for local buckling of CFT member under cyclically repeated loading. Being different from square hollow steel tube, the restraint of in-filled concrete in CFT member would avoid the inward local buckling of steel tube. Therefore, according to the investigation Suzuki (1996), the post-buckling behavior of square hollow steel tubular member (B/t) is approximately equal to that of square tube ($1.32 \times B/t$) in CFT member, the normalized factor α as shown in Table 2.4 is correspondingly modified to be

$$\alpha = \left(\frac{1}{1.32} \cdot \frac{B}{t} \right)^2 \cdot \varepsilon_y \quad (2.17)$$

The parameters of r_{spm} , τ_{lb} are correspondingly modified by α . Besides, according to the basic equation proposed by Yamada et al. (1993), the strain at the local buckling of square CFT is modified by Kawano (2006) to be

$$\frac{\varepsilon_{lb}}{\varepsilon_y} = 6.20 \cdot \alpha^{-1.4} + 1 \quad (2.18)$$

(2) *Circular concrete steel tube*

The strain at the local-buckling of circular CFT members is also derived based on Kawano's modification (2006) of Ochi's equation (1988) as shown in Table 2.4 by

$$\frac{\varepsilon_{lb}}{\varepsilon_y} = 0.18 \cdot \alpha^{-1.4} + 1 \quad (2.18)$$

2.5 Calibration on Constitutive Models of Various Structural Components

2.5.1 Calibration procedures and methodology

(a) Test specimens and loading procedures

Test specimens subjected to various loading procedures are simulated to verify the accuracy of the stress-strain models of concrete and steel developed in the Sections 2.3~2.4. Extensive test specimens as shown in Appendix A are simulated and compared with the corresponding stress-strain models. The Loading types of these test specimens are monotonic axial loading, monotonic and cyclic horizontal loadings with axial constant loading as shown in Figure 2.15. The boundary conditions are fixed supports at both ends of specimens, and fixed support at the base end and free at the top. For the specimens subjected to axial load, the geometric nonlinearity, especially the $P-\Delta$ effect is also incorporated in the analysis.

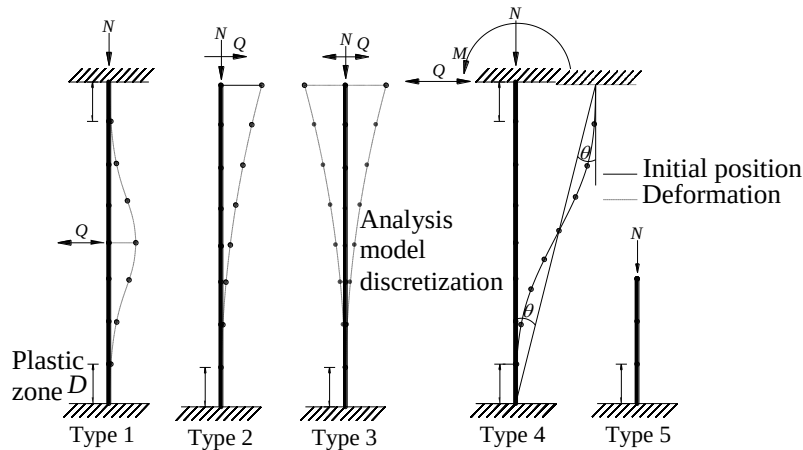


Figure 2.15 Load types of test specimens

(b) Calibration Methodology

Since that not only the strength and deformability of various structural members, but also evaluate the deterioration of structural members are expected to be calibrated, the peak point (incorporating the maximum strength and the deformation at the strength) of load-deformation relation and the deterioration slope after peak point are compared between analysis and experiment. Figure 2.16 demonstrates the method to pick up the above three parameters under monotonic and cyclic loading.

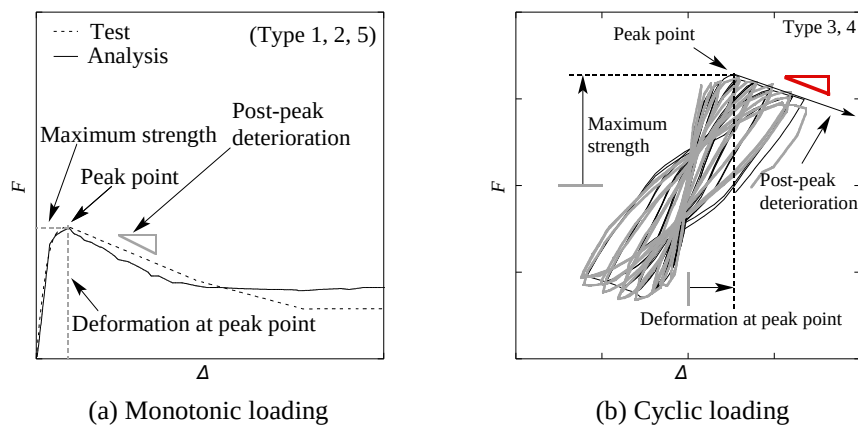


Figure 2.16 Comparison methodologies of load-deformation relations

2.5.2 H-shaped steel member

With respect to the high-rise steel buildings constructed during 1980s~90s period, H-shaped steel have been mainly utilized as the girders (AIJ 2007). Therefore, calibration on the constitutive model of the H-shaped steel components is conducted, where the analytical results using the constitutive models are compared with the corresponding experimental results (Mitani 1980, Suzuki et al. 1991, Yoshiyuki 1991, Yamada et al. 1992, Akiyoshi et al. 1992). Geometric and material properties and the loading conditions of H-shaped steel specimen are shown in Table 2.5.

Table 2.5 Geometric and material parameters of H-shaped steel specimens

Researchers	Specimen numbers	D (mm)	B (mm)	D/t_w	B/t_f	σ_y (N/mm ²)	n ($=N/N_y$)	Loading types
Yamada et al.	19	157~291	168~264	14.8~29.7	14.0~22.0	356~499	-	Type1
Suzuki et al.	8	150	150	22.0	16.7	291~526	-	Type1
Mukai et al.	10	138~258	108~324	20.0~40.0	12.0~36.0	300	-	Type1
Mitani	7	100~137	76~100	20.3~39.0	12.4~22.2	300~456	0.30~0.60	Type2
	5	100~136	99~100	26.6~39.1	15.2~22.1	300~320	0.30, 0.60	Type3
Matsui et al.	12	150	150	22.0, 23.0	16.7, 25.0	292~378	0.10~0.60	Type3

A rigorous accuracy verification of the constitutive models is carried out by summarizing test/analysis ratio of calibration parameters against the width-to-thickness ratio (D/t_w), which are respectively shown in Figure 2.17. Parameters of peak point and post-peak deterioration show different deviations as seen in Figure 2.17, especially for the calibration ratio of post-peak deterioration, the dispersion of test/analysis ratio shows wider distribution than those at the peak point.

However, the standard deviation of the ratios between experiment and analysis is around 10%, which may be thought that the accuracy of the constitutive model satisfies practical engineering use. Specifically, the test/analysis ratios of the post-peak deterioration for various type loading procedures show highest deviations which are 0.113 for Type 1 loading procedure, 0.113 for Type 2 loading procedure, and 0.101 for Type 3 loading procedure. This indicates that post-peak deterioration is the most difficult behavior to be predicted, compared with those of strength and deformation at peak point.

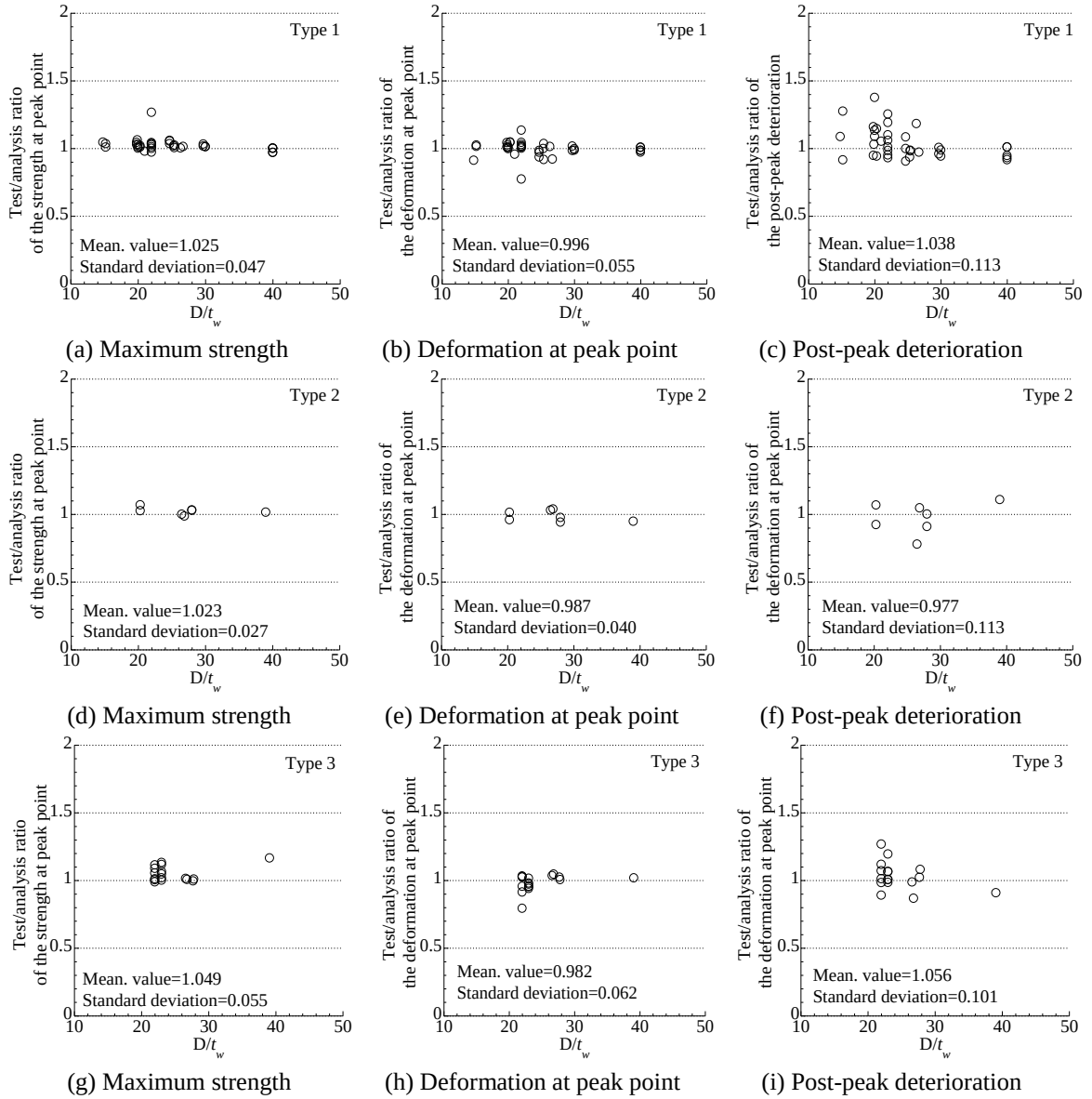


Figure 2.17 Accuracy calibrations of specimens with D/t_w (all data)

Figure 2.18 illustrates a typical example of the cyclic deterioration of H-shaped steel component under monotonic and cyclic loadings, where it is found that when the load applies cyclically, the deterioration behaviour may be accelerated.

As shown in Figure 2.18, both of the post-peak deteriorations in skeleton curve and in cyclic hysteretic curve could be predicted by the proposed constitutive model of H-shaped steel components. Strength deterioration caused by the cyclic loadings up to the same rotation angle ($\theta/\theta_p=5$) is also predicted by constitutive models. In addition, the unloading stiffness deterioration can be also well simulated because of the moving the unloading and Bauschinger curve based on hysteresis rule.

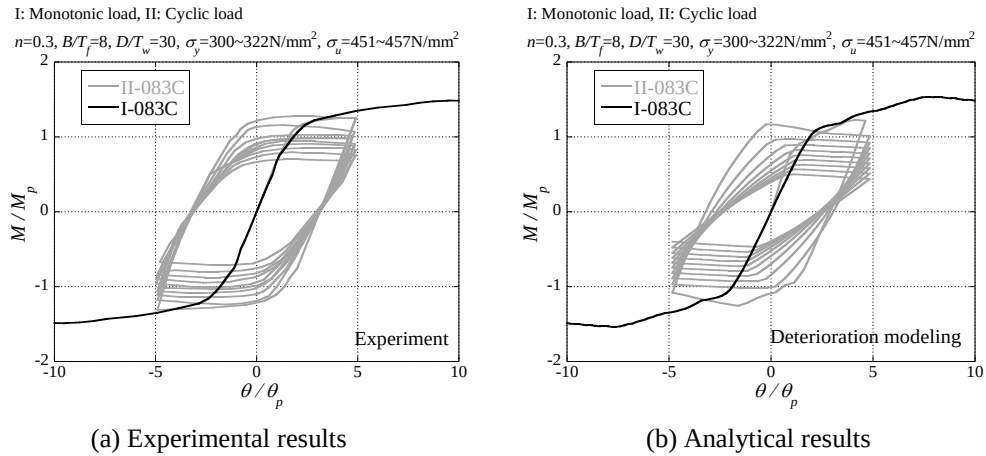


Figure 2.18 Cyclic deterioration of H-shaped steel component

2.5.3 Circular hollow steel tubular member

Monotonic and cyclic tests of type 2, 3, 4 and 5 (Sakai et al. 1992, Ueno 1991, Tsuda and Matsui, 1998, Tsutomu et al. 1987, Yasui 2002) for circular hollow steel tubular columns are also simulated to verify the prediction accuracy by the proposed constitutive model considering strength deterioration. As shown in Table 2.6, 48 circular hollow steel tubular specimens are modeled to verify the prediction accuracy, various parameters of f'_c , σ_y , D/t and n etc. are also summarized.

Table 2.6 Parameters of the circular hollow steel tubular test specimens

Test	Specimen numbers	Geometrical properties			Material properties		Axial load level	Loading type
		D/t	$t(\text{mm})$	$L(\text{mm})$	$\sigma_B(\text{N/mm}^2)$	$\sigma_y(\text{N/mm}^2)$	n	
Ochi et al.	4	29.9~37.5	4.39~5.50	847~853	-	377~549	0.0	Type2
	2	41.9, 43.6	3.80, 3.95		-	474~484	0.0, 0.4	Type3
Tsuda et al.	9	39.9~87.4	1.60~3.50	750	-	343~415	0.07~	Type2
	6				-		0.35	Type3
Tsuji et al.	10	25.4, 32.7	3.50, 4.50	400, 600, 800	-	329, 356	0.0, 0.2	Type2
	6				-			Type3
Yasui	4	17.0~42.4	3.27~12.7	550, 900	-	345~382	0.0, 0.3	Type4
	7				-			Type5
Summary	48	17~87.4	1.60~12.7	-	-	276~549	0.0~0.35	Types 2~5

Note:

D is the depth of circular CFT member;

t is the thickness of the steel tube in circular CFT member;

L is the length of the test specimen;

σ_B is the cylinder strength of concrete;

σ_y is the yield strength of steel;

n is axial load ratio.

Generally speaking, the overall comparisons between analytical and experimental results of each calibration parameter are respectively shown in Figure 2.19, it is noted that the peak point of circular hollow steel tubular members can be accurately predicted by the corresponding stress-strain models, e.g., the mean value of the test/analysis ratios for the strength and deformation at peak-point are 1.002 and 1.012, with standard deviation of 1.3% and 3.3%. However, in the case of the test/analysis ratio for deterioration stiffness, mean value is 0.953 and standard deviation is up to 8.7%.

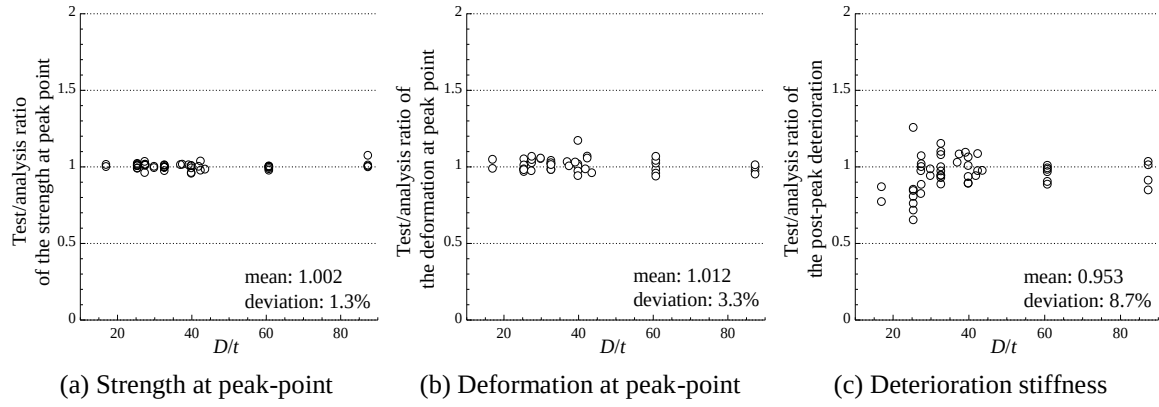


Figure 2.19 Comparison results of circular hollow steel tubular columns

Besides, typical examples of the test-to-analysis comparisons related to axial load level n are picked and illustrated in Figures. 2.20~2.21. Acceptable error is observed between test and analysis results for the circular hollow steel tubes under monotonic and cyclic loadings. Overall, stable performance of the unified constitutive models is captured when the specimens under low ($n=0.07$) and high ($n=0.21$) axial load ratios.

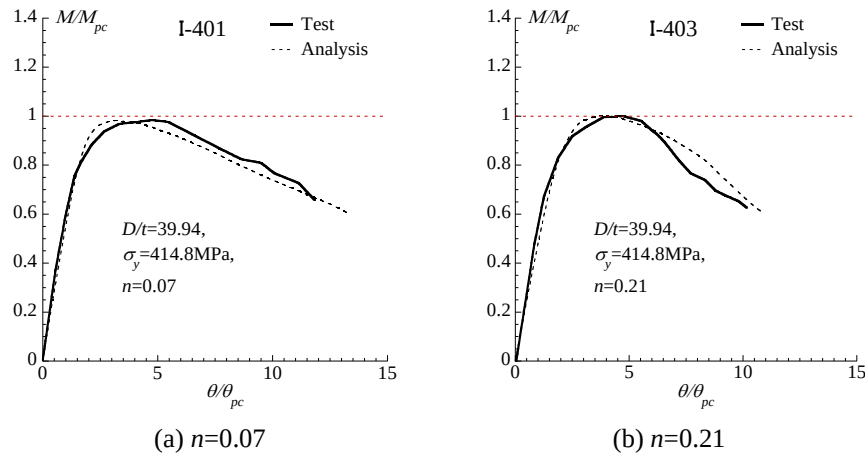


Figure 2.20 Comparison results of circular hollow steel tubular columns under type 2 loading

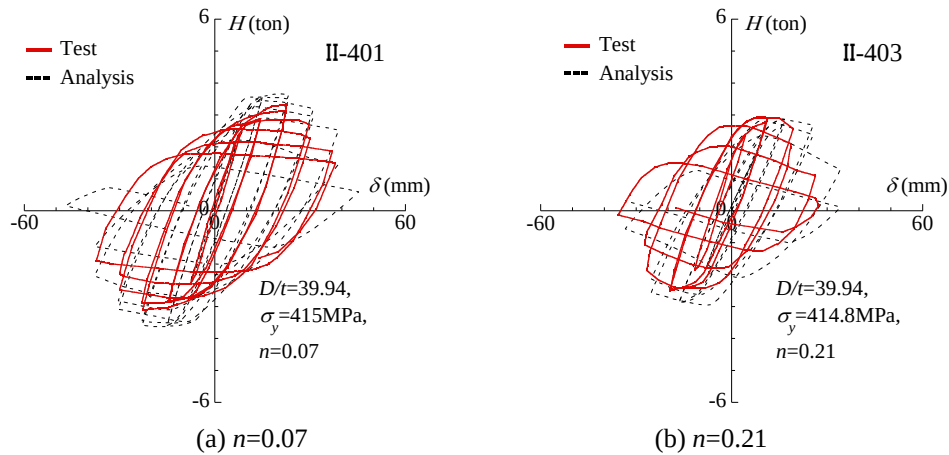


Figure 2.21 Comparison results of circular hollow steel tubular columns under type 3 loading

2.5.4 Circular concrete filled steel tubular member

Monotonic and cyclic tests of types 1, 2, 3 and 4 (Elremaily and Azizinamini 2002, Xiao et al. 2005, Inai et al. 2004, Fujinaga and Matsui et al. 1997) for circular CFT specimens are simulated to calibrate the prediction accuracy by the proposed constitutive models by considering strength deterioration.

Table 2.7 Parameters of the circular CFT test specimens

Test	Number	Geometrical properties			Material properties		Axial load raio	Loading types
		D/t	$t(\text{mm})$	$L(\text{mm})$	$\sigma_B (\text{N/mm}^2)$	$\sigma_y (\text{N/mm}^2)$	n	
Elremaily et al.	6	34/51	6.4/9.5	914	40/70/100	372	0.2~0.4	Type 1
Xiao et al.	1	112	3	1500	39.1	303	0.47	Type 2
Inai et al.	2	53	4.5	1440	40/90	276	0.4	Type 4
Fujinaga et al.	21	33/39/77	3.01/2.55/1.25	500	30.9~33.5	406/391/410	0.2~0.8	Type 3
	6						0.3, 0.5, 0.7	Type 2
Summary	36	33~112	1.25~9.5	500~1500	30.9~100	276~449	0.2~0.8	Types 1~4

Note:

D is the depth of circular CFT member;

t is the thickness of the steel tube in circular CFT member;

L is the length of the test specimen;

σ_B is the cylinder strength of concrete;

σ_y is the yield strength of steel;

n is axial load ratio.

As shown in Table 2.7, 36 circular CFT specimens are adopted to verify the prediction accuracy of the constitutive models. According to the above verifications, comparisons of strength, strain at the strength and deterioration stiffness obtained by experimental results and analytical results are summarized as presented in Figure 2.22. The comparison results are observed that the analytical results show high agreement with the test results according to the mean ratios and deviations, based on various D/t or B/t (22~112 for circular CFT) and various loading procedures.

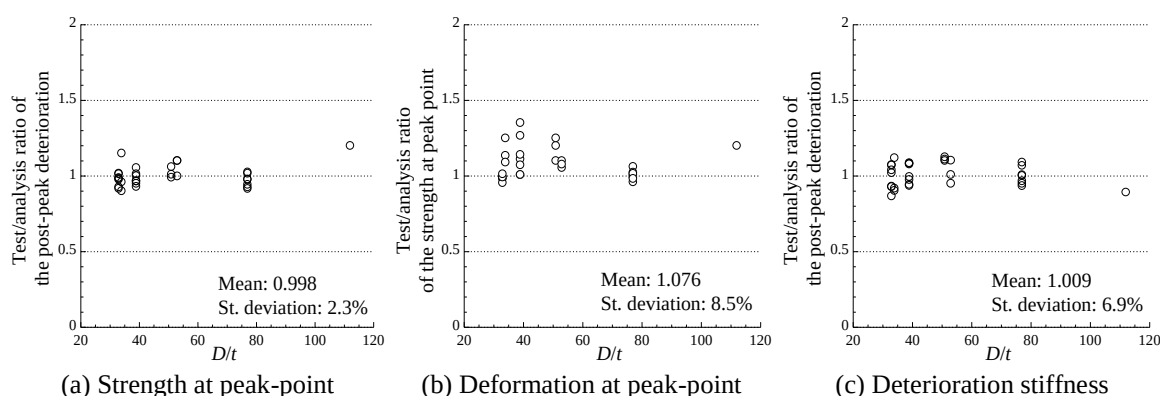


Figure 2.22 Comparison results of circular CFT columns

Comparison results of circular CFT specimens with different axial load ratios $n=0.4$ and 0.8 to verify the dependence of n on the deterioration of circular CFT columns (Fujinaga and Matsui et al. 1997) as showed in Figures. 2.23~2.24. As observed in Figures. 2.23~2.24, the confined effects

considerably depend on axial load ratio n . The strength deterioration gradually decreases along with the increasing of n .

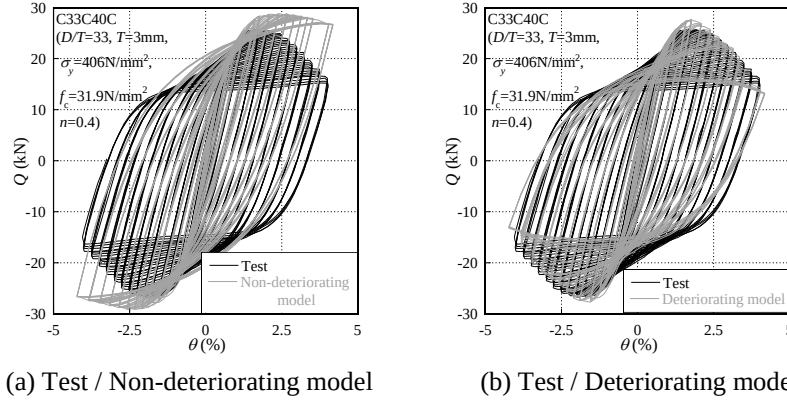


Figure 2.23 Comparison between experiment and analysis of circular CFT ($n=0.4$)

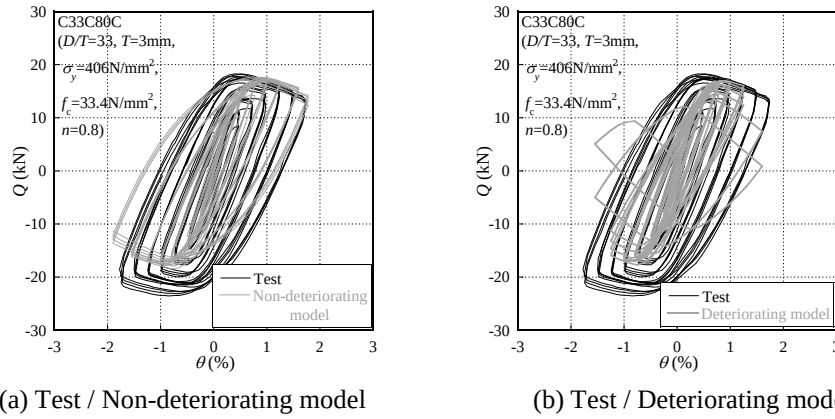


Figure 2.24 Comparison between experiment and analysis of circular CFT ($n=0.8$)

Figures 2.25~2.26 show the comparison between analytical and experimental horizontal load-deflection curves with different diameter-to-thickness ratio and concrete strength. It is observed that predicted curves mainly have good agreement with test results by using degradation model of steel and no-degradation model of confined concrete.

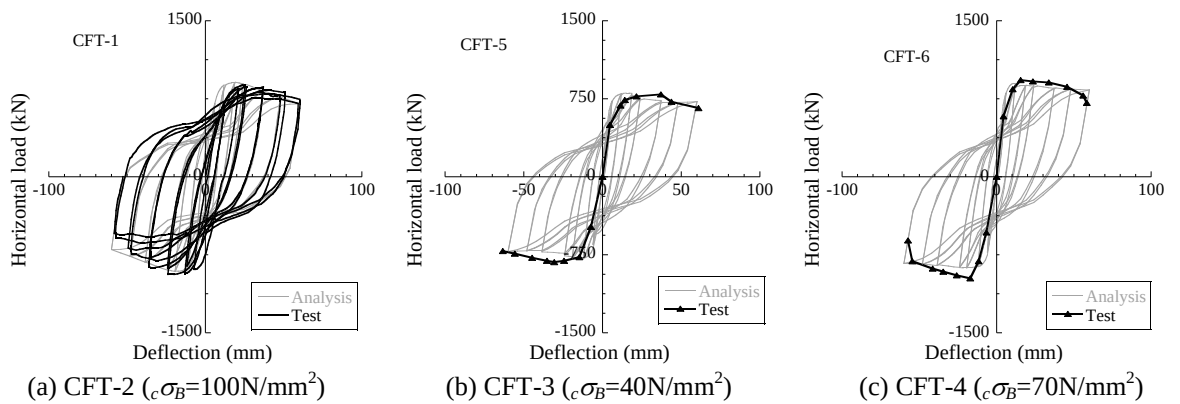


Figure 2.25 Specimens with the diameter-to-thickness ratio $D/t=51$

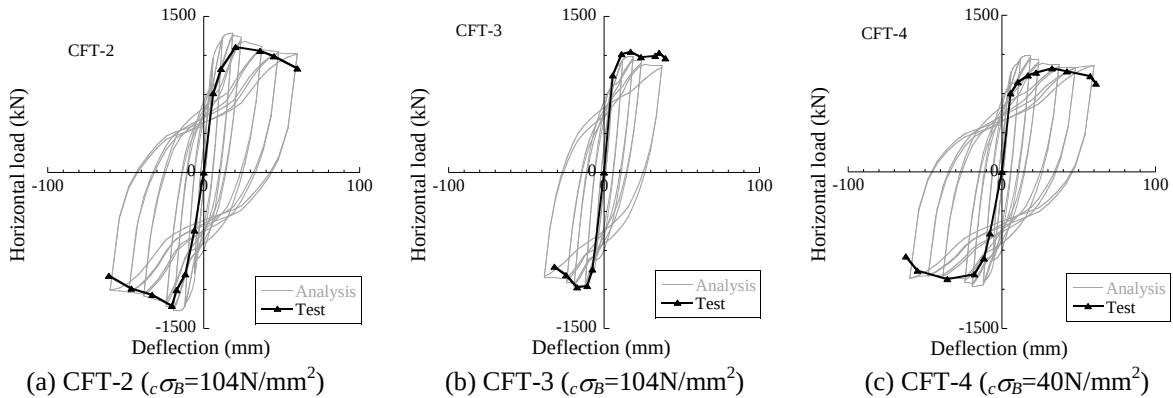


Figure 2.26 Specimens with the diameter-to-thickness ratio $D/t=34$

2.5.5 Square hollow steel tubular member

Various tests of square hollow steel tubular columns conducted by references (Yoshida et al. 1996, Kato and Akiyama et al. 1978, Tsuda and Matsui 1998) are adopted to verify the accuracy of analytical results by considering the strength deterioration. Properties of verified experimental hollow steel tubular specimens are shown in Table 2.8.

Table 2.8 Parameters of square hollow steel tubular specimens

Test specimens	Numbers	Geometrical properties			Material properties		Axial load level	Loading types
		B/T	$t(\text{mm})$	$L(\text{mm})$	$c\sigma_B(\text{N/mm}^2)$	$\sigma_y(\text{N/mm}^2)$		
Yoshida et al.	6	23.2~44.0	5.7~8.6	400~1000	-	351~395	-	Type 5
Kato et al.	5	25.0~41.7	6.0	300~750	-	319~338	0.0	Type2
Tsuda et al.	12	22.5~47.6	3.1~5.8	750	-	325~456	0.1~0.5	Type2
	6	25.9~47.6	3.2~5.8		-	325~456	0.1, 0.45	Type3
Summary	29	22.5~47.6	3.1~8.6	300~1000	-	319~456	0.0~0.5	Types 2, 3, 5

Note:

- B is the width of square CFT member;
- t is the thickness of the steel tube in square CFT member;
- L is the length of the test specimen;
- $c\sigma_B$ is the cylinder strength of concrete;
- σ_y is the yield strength of steel;
- n is axial load ratio.

Comparisons of test/analysis results of strength, deformation at peak-point and post-peak deterioration and the corresponding mean ratios and standard deviations are shown in Figure 2.27, respectively.

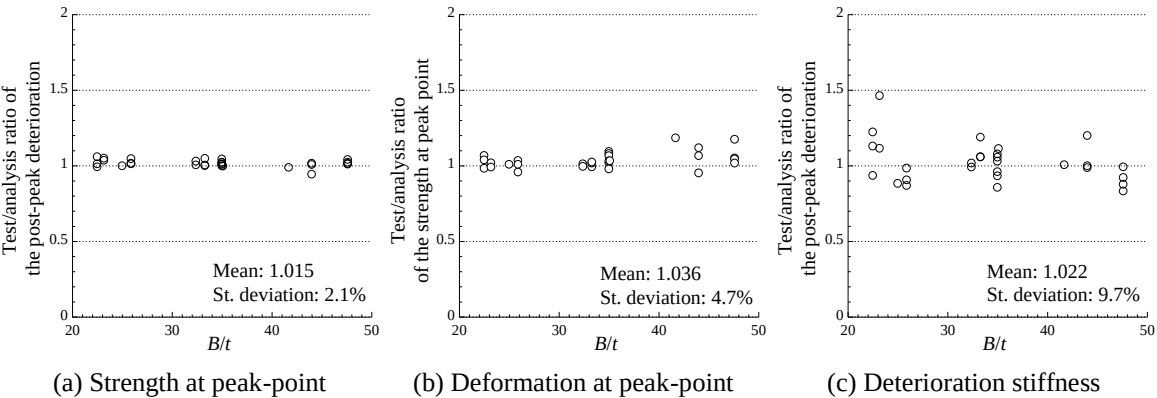


Figure 2.27 Comparison results of square HST columns

As examples, typical comparisons of the square hollow steel tubular columns under monotonic and cyclic loading procedures between experimental and analytical results are demonstrated in Figures 2.28~2.29. As for square hollow steel tubes under monotonic horizontal loading with constant axial load ratio, post-peak deterioration behavior is well traced, while the residual branch until the end of the skeleton curve gradually shows differentials. On the other hand, for the specimen under cyclic loadings (see Figure 2.29), cyclic deterioration behaviors of the specimen under various axial load ratios are accurately predicted by the unified constitutive models for square hollow steel tubular members.

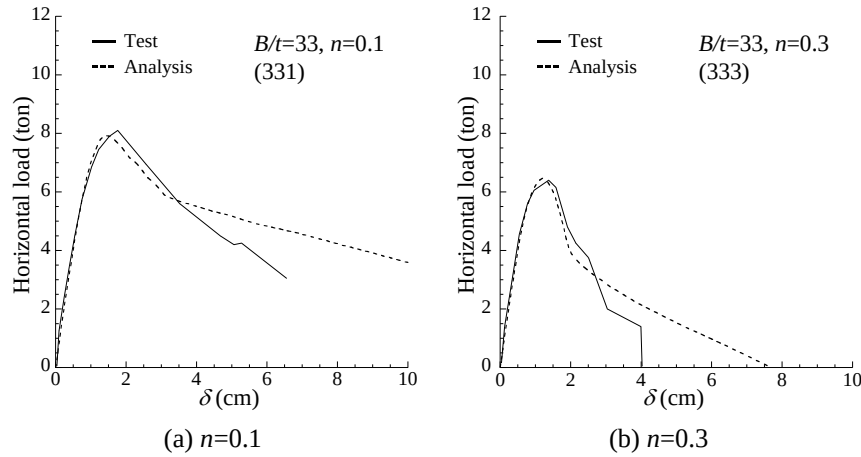


Figure 2.28 Comparison results of square hollow steel tubular columns under type 2 loading

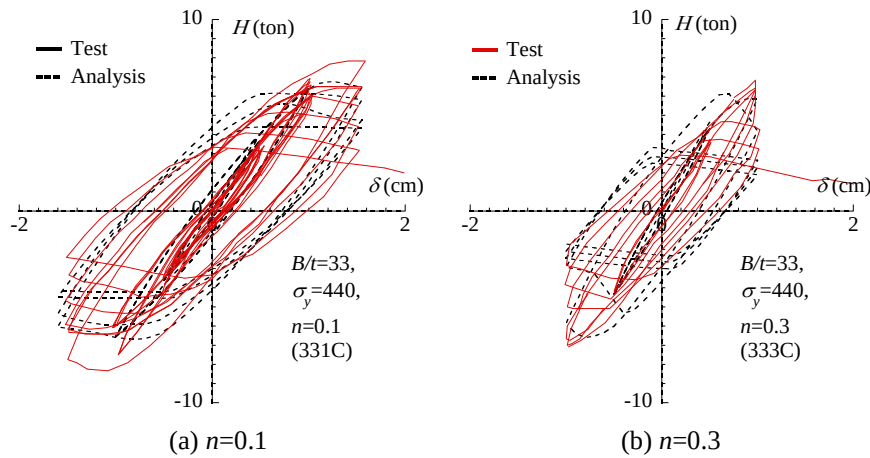


Figure 2.29 Comparison results of square hollow steel tubular columns under type 3 loading

2.5.6 Square concrete filled steel tubular member

Considering that the effects of depth (D) and width (B) on the confinement effects and seismic behavior of rectangular CFT columns are little to be considered. Several tests (Ricles et al. 2004, Han 2002, Yamaji and Matsui 1997) of square CFT columns are simulated by adopting proposed square models with considering the strength deterioration of confined concrete and square steel tubes. Properties of verified square CFT specimens are shown in Table 2.9.

Table 2.9 Parameters of square CFT test specimens for verification

Test	Specimen numbers	Geometrical properties			Material properties		Axial load level	Loading procedures
		B/t	t (mm)	L (mm)	$c\sigma_B$ (N/mm ²)	σ_y (N/mm ²)	n	
Ricles et al.	4	32/48	8.9/6.1	1500	110	600/660	0.1/0.2	Type 3
Inai et al.	4	35/47	6.0/4.5	1260	35.5~42.4/ 84.5~94.5	295/276	0.4	Type 4
Han	2	35/42	2.86	300/360	48.3	228	-	Type 5
Matsui et al.	21	22/31/43	4.40/3.21 /2.24	500	34.1~36.3	407/449/415	0.2~0.8	Type 3
	6						0.3, 0.5	Type 2
Summary	37	22~48	2.24~8.9	300~1500	34.1~94.5	228~660	0.1~0.8	Types 2~5

Note:
 B is the width of square CFT member;
 t is the thickness of the steel tube in square CFT member;
 L is the length of the test specimen;
 $c\sigma_B$ is the cylinder strength of concrete;
 σ_y is the yield strength of steel;
 n is axial load ratio.

Likewise, the comparisons of the peak-point and post-peak deterioration between experiment and analysis are respectively shown in Figure 2.30. All of the standard deviations of the test/analysis ratios of the strength, deformation at peak point and post-peak deterioration are stabilised at 6.6%, 7.9% and 6.9%, respectively.

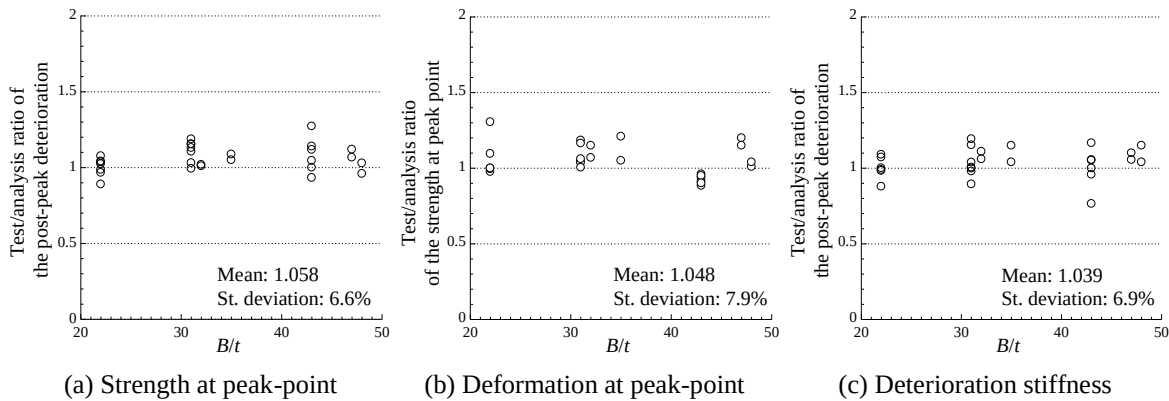


Figure 2.30 Comparison results of square CFT columns

In particular, in order to consider various deterioration characteristics of the confined concrete, steel tube in the square CFT members, a typical specimen with width-to-thickness ratio of 32 (Ricles et al. 2004) is modelled by considering only concrete, only steel tube, and fully incorporating both of them. The corresponding comparisons of such modelling and tests are shown in Figure 2.31.

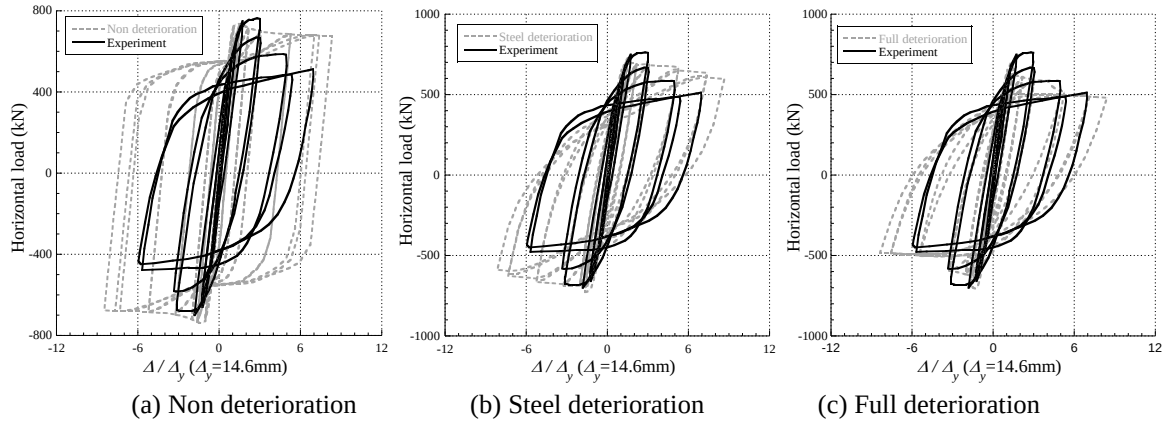


Figure 2.31 Comparison between analysis and experiment of horizontal load-displacement relations of square CFT specimen ($B/t=32$, $n=0.2$)

It is noted that ignoring the steel deterioration which is equal to only incorporating the concrete deterioration tend to overestimate the post-peak deterioration of the corresponding square CFT columns, and also significantly overestimate the post-peak deterioration stiffness comparing to the experimental results, as shown in Figure 2.31 (a). In contrast, when only considering the effect of steel deterioration, although the post-peak deterioration stiffness is also slight overestimated, the unloading stiffness deterioration can be well predicted by such model. Therefore, in the case of full incorporating the deterioration of steel tube and confined concrete, various deterioration characteristics of the square CFT under cyclic loading could be well predicted, e.g., post-peak deterioration of the hysteresis loop, and unloading stiffness deterioration. Moreover, the occurrence conditions of member deterioration along with width-to-thickness ratio B/t and axial load ratio n can be assessed by analysis, as shown in Figures 2.32~2.34.

According to the comparison results as shown in Figures 2.32~2.34, for the member with small width-to-thickness ratio $B/t=32$, axial load ratio ($n=0.1, 0.2$) does not show significant correlation with member deterioration, but for the large width-to-thickness ratio $B/t=48$, axial load ratio $n=0.2$ demonstrates significantly sensitive to the member deterioration, because the local-buckling of this member is induced when member deterioration is incorporated as shown in Fig. 2.34 (b).

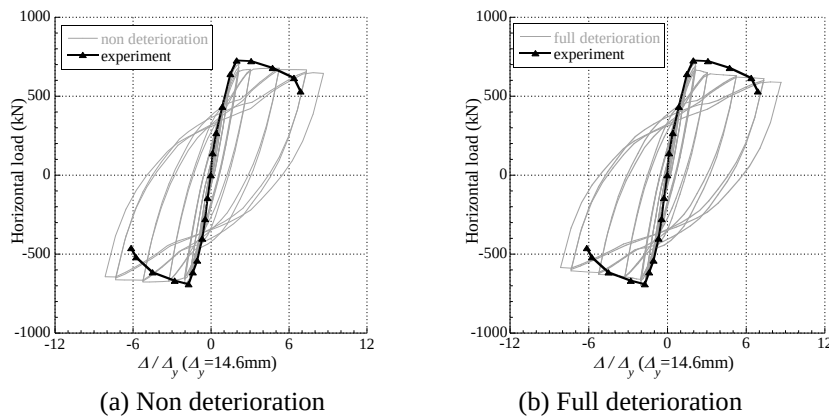


Figure 2.32 Comparison between analysis and experiment of horizontal load-displacement relations of square CFT specimen ($B/t=32$, $n=0.1$)

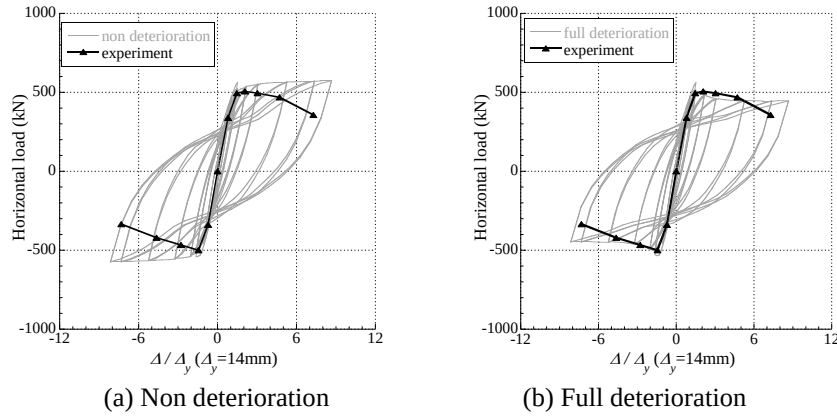


Figure 2.33 Comparison between analysis and experiment of horizontal load-displacement relations of square CFT specimen ($B/t=48$, $n=0.1$)

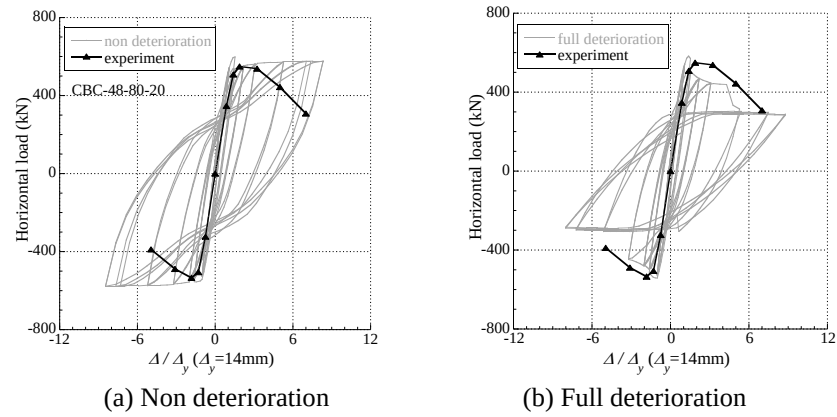


Figure 2.34 Comparison between analysis and experiment of horizontal load-displacement relations of square CFT specimen ($B/t=48$, $n=0.2$)

2.6 Conclusions

In this Chapter, the deterioration characteristics of the unified stress-strain models for the concrete and steel materials specifically in various associated structural members are developed. Confining effect on the concrete of CFT member was considered and the post-peak deterioration of concrete induced by crush and slip was also quantified based on a unified stress-strain model. Moreover, the strength and stiffness deteriorations of steel members with various cross sections are incorporated to evaluate the deterioration behavior of such members, e.g., local buckling of H-shaped steel and square steel tubular components may deteriorate the strength in a small interval of deformation. After the unified stress-strain models for concrete and steel elements being determined, the accuracy of these models to predict the occurrence of deterioration was calibrated on the basis of a database including extensive realistic experimental specimens. In particular, some calibration results can be concluded as follows:

(1) Unified Constitutive Models: A series of unified constitutive models for concrete and steel members have been developed by incorporating various deterioration effects of the unloading stiffness deterioration, Bauschinger effect, post-buckling strength deterioration and stiffness deterioration of negative slope. These constitutive models with deterioration effects are expected to accurately predict the deterioration behavior of various structural components

(2) H-shaped steel: High accuracy of the validation on the H-shaped steel under various loading procedures has been derived. The standard deviations of the test/analysis ratios of the

strength and deformation at peak point are under 10%, and the standard deviation of the test/analysis ratios of the post-peak deterioration is around 10% which indicates that deterioration behavior is the most difficult part to be traced. The calibration accuracy of the unified constitutive models for H-shaped steel well satisfies the engineering application.

(3) Hollow steel tubes: As for the circular hollow steel tube specimens under various loading procedures, the standard deviations of the test/analysis ratios of the strength and deformation at peak point are both 1.3% and 3.3% and the standard deviation of the test/analysis of the post-peak deterioration is 8.7%. Besides, the circular hollow steel tubes under low and high axial load ratios can be stably predicted by the unified constitutive models. The analogous validation results are also obtained in case of the square hollow steel tubes.

(4) Concrete filled steel tubes: Circular concrete filled steel tubes under ultra-high axial load ratio ($n=0.8$) still shown stable hysteresis behavior, and the analytical results by using the unified constitutive models incorporating deteriorations of concrete and steel demonstrated instable behavior which is not march up to the test results. Hence, the confining effect tends to strengthen the concrete to prevent crush and slip and the deterioration behavior of the confined concrete in circular CFT should be ignored. In contrast, for the square CFT under cyclic loading, ignoring the steel deterioration would significantly overestimate the stiffness, and ignoring the concrete deterioration would overestimate the post-peak behavior. These characteristics of CFT were included in the constitutive model.

Overall, the unified constitutive models for the concrete and steel member of various structural components under various loading procedures, is capable to simulate the strength, deformation at peak point and also post-peak deterioration behavior on the premise of high accuracy. The deterioration behavior of concrete and steel members should be rigorously considered in the seismic design of these structural components.

Nomenclature

B	Outside width of square steel tube [mm]
D	Outside diameter of circular steel tube [mm]
T	Thickness of steel tube wall [mm]
X, Y	stress and strain normalized by peak values of stress–strain relations for concrete
V	Coefficient of the Sargin's function defined by Sakino & Sun
W	Coefficient of Sargin's function defined by Sakino & Sun
E_c	Young's modulus of concrete
E_s	Young's modulus of steel
σ_c, ε_c	Stress and strain of concrete
f'_c	Compressive strength of unconfined concrete [N/mm ²]
ε'_c	Strain at maximum stress of unconfined concrete
f'_{cc}	Compressive strength of confined concrete [N/mm ²]
ε'_{cc}	Strain at maximum stress of confined concrete
f'_{cu}	Ultimate strength of confined concrete after strength deterioration [N/mm ²]
ε'_{cu}	Ultimate strain of confined concrete after strength deterioration
σ_B	Cylinder strength of concrete [N/mm ²]
σ_{re}	Effective lateral confining stress provided by steel tube
σ_y, ε_y	Tensile yield stress and strain of steel
σ_u, ε_u	Tensile ultimate stress and strain of steel
τ	Deterioration factor of confined concrete
τE_c	The deterioration modulus of confined concrete after peak
ε_{lb}	The strain at local buckling of steel tube
α	Normalized value of $B(D)/T$ and steel yield strain
τ_{lb}	Rate of the buckling negative slope to Young's modulus
τ_{lb2}	Rate of the post-buckling negative slope to Young's modulus
γ_{spm}	Rate of the post-buckling strength to the buckling strength
R_{ini}	The R (radian) for the first M&P skeleton curve
R_u	The R (radian) for the second M&P skeleton curve
ψ	Ratio to move the skeleton curve
k_p	Normalize ratio of the compressive strength of plain concrete to 60MPa
n	Axial load level ($n=N_0/N_u$)
N_0	Axial load on the HST and CFT columns
N_u	Ultimate compressive resistance of HST and CFT columns

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CHAPTER 3

SIMULATION OF DETERIORATION BEHAVIOR OF VARIOUS STRUCTURAL FRAMES

3.1 Introduction

In the Chapter 2, we discussed the member deterioration on various structural components (i.e., S, CFT sectional members) based on the calibration and validation of developed stress-strain models for concrete (i.e., confined concrete) and various steel components (i.e., H-shaped steel, circular and rectangular hollow steel tubes, circular and rectangular confined steel tubes) through extensive experimental database of structural members. Furthermore, in this chapter, the member deterioration on the seismic-resistant capacity of frame structures are also calibrated, especially on collapse resistant capacity of frame structures subjected to severe earthquakes, by utilizing the proposed unified constitutive models of concrete and steel in the simulation of experimental frame specimens.

3.2 Shaking Table Test of Full-Scale 4-Story Steel Building

A shaking table test of a full-scale 4-story steel building was conducted on the September, 2007, by Suita et al. (2009), in order to reproduce the process of collapse of the steel building under the oversized earthquake excitation that is larger than seismic criteria. According to the experimental results, the process leading to collapse mechanism and the corresponding collapse behaviour were captured. Since then, this experiment was analysed by some researchers, e.g., Lignos et al. (2012) have analysed this E-defence shaking table test to explore the so called ‘sidesway collapse mechanism’ of moment frame structure, based on two dimensional analysis program.

We employ two-dimensional frame analysis with stress fiber models by considering the effect of member deterioration, to calibrate the accuracy of the corresponding constitutive models for fiber elements. Furthermore, the feasibility of analysis program to simulate experiments also needs to be investigated. The full-scale 4-story steel building is designed as a moment-resistant frame with square hollow steel tubular columns and H-shaped steel girders, and the beam-column connections employed the through diaphragm. The details of the test specimen can be referred by Suita et al. (2009).

In particular, the cross sections of H-shaped steel for girders were selected as the most deformable ones and those of square steel tubular members for columns were the moderate deformable ones with the $B/t=33.3$. They are specified in Enforcement Ordinance of Construction Standard Law as the rank of FA and FB, respectively. The deformability ranks of structural members satisfy the requirement of low-rise buildings in the practical design.

The Takatori-JR ground motions recorded in 1995 Hyogoken-Nanbu earthquake (Nakabura 1996) applied to the frame specimen on a shaking table test. The intensity of the ground motions is progressively increased in a sequence of 20%, 40%, 60% and 100% of the original record, in which 20% and 40% of the intensity may be thought to correspond to the level 1 and 2 of ground motions for the high-rise building design in Japan, respectively. Therefore, it has to be said that the intensity of Takatori-JR ground motions exceed the earthquake level of Japanese design standards. The prototype frame specimen is a four-story three-dimensional moment resistant frame [see Figure 3.1 (a)] with one span in X direction [see Figure 3.1 (b)], and two spans in Y direction[see Figure 3.1 (c)].

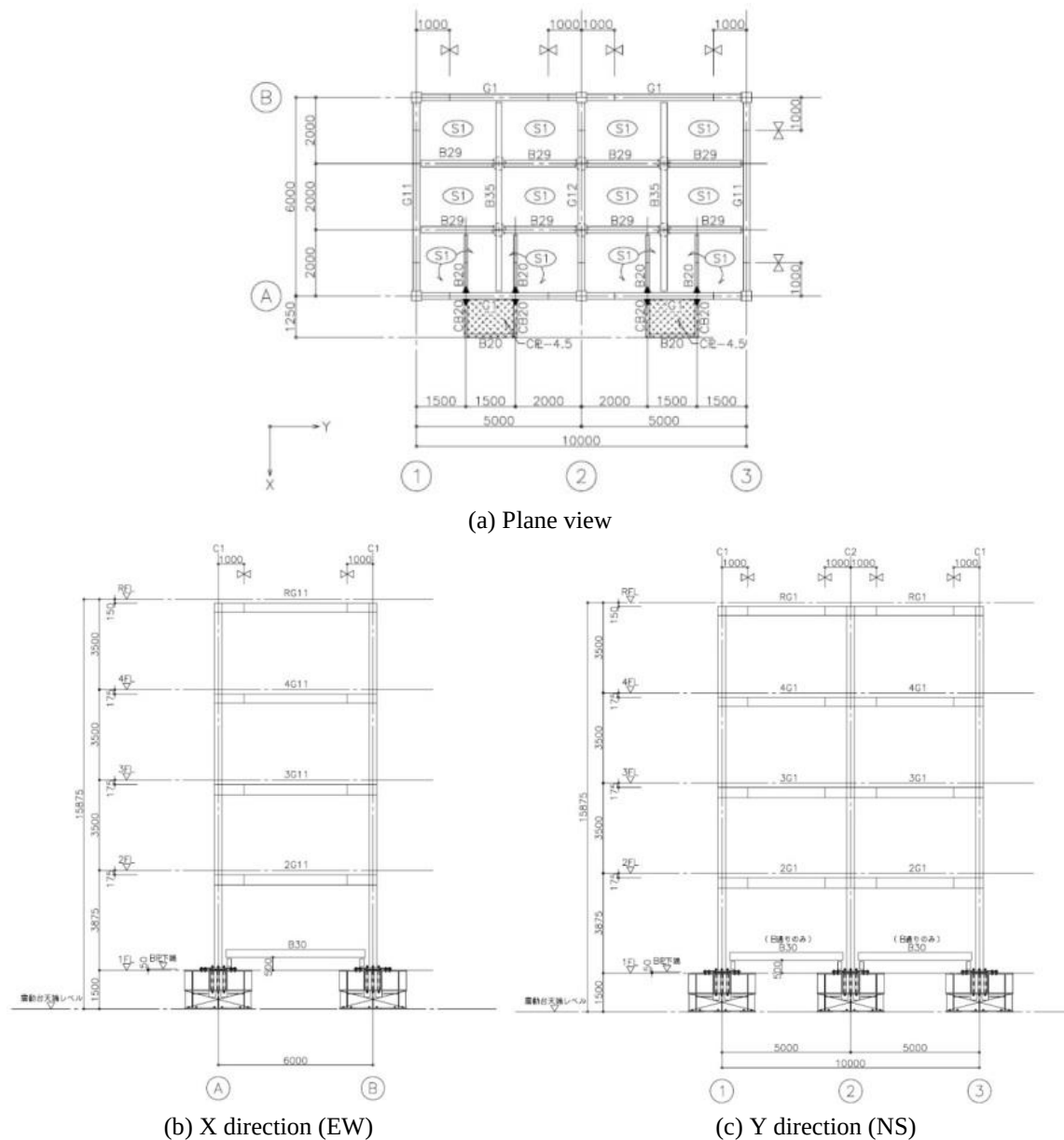


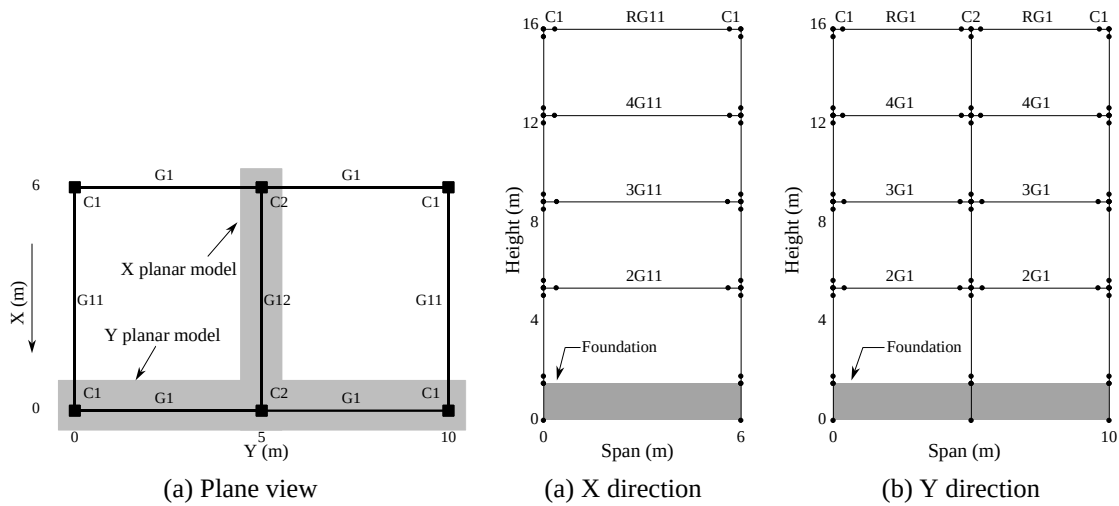
Figure 3.1 Prototype building of full-scale frame specimen

Corresponding to the girders and columns of the X-directional and Y-directional frames models (see Figure 3.2), the cross sections of them are shown in Table 3.1. Interior and exterior square steel tubular columns are the same with $B=300\text{mm}$ and $t=9\text{mm}$.

Table 3.1 Steel components of frame specimen

Floor	G1 (SN400B)	G1 (SN400B)	G1 (SN400B)	Story	C1, C2 (BCR295)
R	H-346×174×6×9	H-346×174×6×9	H-346×174×6×9	4	□-300×300×9
4	H-350×175×7×11	H-350×175×7×11	H-340×175×9×14	3	□-300×300×9
3	H-396×199×7×11	H-400×200×8×13	H-400×200×8×13	2	□-300×300×9
2	H-400×200×8×13	H-400×200×8×13	H-390×200×10×16	1	□-300×300×9

By adopting a two-dimensional analysis program as introduced in Chapter 2, the analytical model is a planar frame model for each of X and Y directions as shown in Figure 3.2. The east-west component (EW) and north-south component (NS) were applied to the frame specimen in X direction and Y direction, respectively, so that the analytical frame model in X direction and Y direction are inputted the EW and NS components, respectively. Even though the up-down component (UD) of the motions is also applied to the specimen, it is ignored in this analysis. The vertical gravity weights of the planar frame models (X and Y directions) are applied to be the one third of those of prototype specimen. Beams and columns are discretized into a number of beam-column elements and the element length at the member ends are the same as the corresponding member depths as mentioned in the section 2.2. In the same way, the cross section of each element is further discretized into a number of layers forming stress fibers in which the deterioration effects are introduced as mentioned in the Chapter 2.

**Figure 3.2** Modelling of full-scale planar frame specimen

Thus, we respectively validate the seismic responses and collapse phenomenon of the specimen in X and Y directions, and meanwhile, prove the accuracy and feasibility of the unified constitutive model and frame analysis program used for predicting seismic resistant capacity of steel frame structures.

3.2.1 X direction simulation (X-EW)

With respect to the simulation of the test specimen in X direction, the X direction model is subjected to the EW component of Takatori-JR. Figure 3.3 shows the comparisons between the results of analysis and experiment of the maximum story drift angle subjected to progressively increasing seismic excitations. It is notable that the analytical results closely follow the tracks of experimental results from 20% to 100% of the intensity. When the intensity of JR-Takaotiri-EW is increased from 40% to 60% of its original wave, local failure of columns at bottom story is

preliminarily initiated because large width-to-thickness steel tubular column rarely has non-ductile capacity to prevent local-buckling.

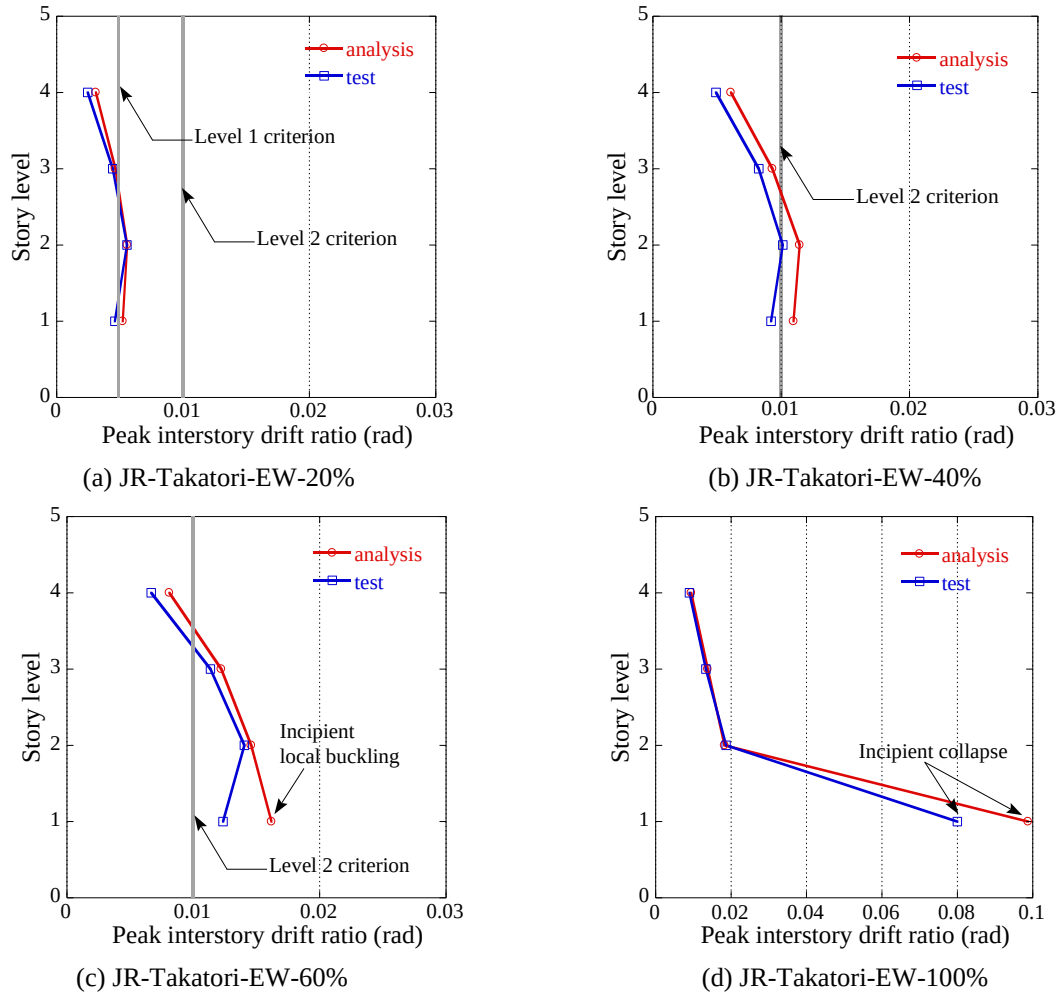


Figure 3.3 Comparison of the peak interstory drift angle at longitudinal (two span) direction

3.2.2 Y direction simulation (Y-NS)

Likewise, the Y-direction of test specimen is also simulated. As shown in Figure 3.4, analytical results is capable to capture the collapse capacity of the response of the specimen in Y direction, e.g., when subjected to 100% of ground motion, the maximum story drift angle of analysis and experiment are 0.139rad and 0.186rad. It indicates that the stress-strain model incorporating member deterioration behavior is capable to simulate the collapse behavior of frame structure under severe ground motions.

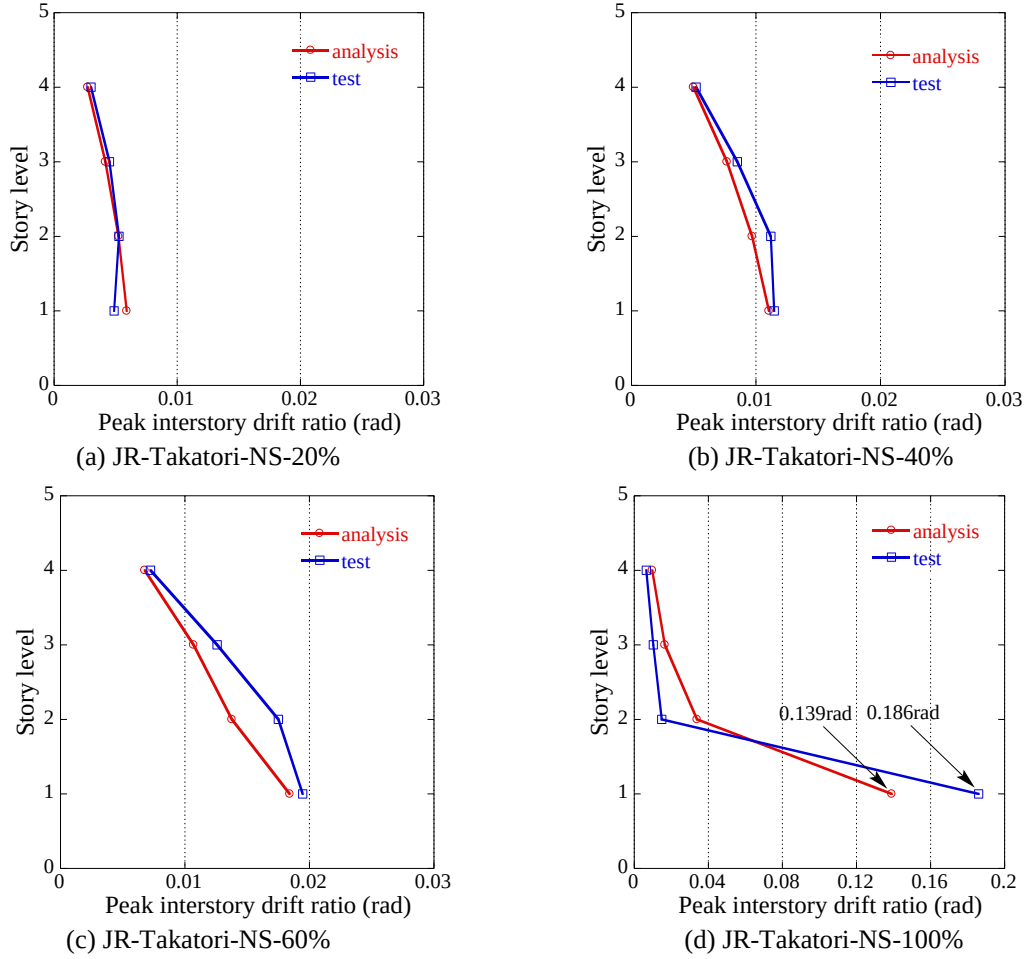


Figure 3.4 Comparison of the peak interstory drift ratio at longitudinal (two span) direction

3.2.3 Energy dissipation

In this section, the collapse process of the test specimen is investigated on the basis of energy dissipation which is calculated by

$$W_{ck} + W_{es} + W_h + W_p + W_{pd} = E_{in} \quad (3.1)$$

Where, E_{in} is input energy at horizontal direction which is based on

$$E_{in} = -\int \sum_{i=1}^4 m_i \ddot{x}_0 \dot{x}_i dt - \int \sum_{i=1}^4 m_i \ddot{y}_0 \dot{y}_i dt \quad (3.2)$$

Where, m_i is the gravity at i -story, $\ddot{x}_0$, and $\ddot{y}_0$ are the ground acceleration at X and Y directions; $\dot{x}_i$, $\dot{y}_i$ are the relative velocity compared with ground velocity at X and Y directions. In addition, W_{ck} is kinetic energy; W_{es} is elastic strain energy; W_h is damping energy; W_p is hysteretic energy; W_{pd} is energy caused by P- Δ effect (reduction of potential energy of barycenter). Figure 3.4 shows the distribution factors of the amount of energy being dissipated between by the first to fourth stories of a frame model in X direction.

The ground motions are Takatori-JR EW component with the intensity from 40% to 100%. It is observed that when the intensity of ground motions is at low level such as from 40% to 60% [see Figure. 3.5 (a) to (b)], the analysis overestimates the energy distribution factor of the first story. The may be caused by the degree of fixed supports at the column bases of the first story, so that in the analytical frame model has been assumed the complete fixed support at the bases, even though

the complete fixed supports are impossible in the experiment. The differences decrease gradually as the intensity of ground motions increase. Figure 3.5 (c) and (d) show the distribution factors for stories until 3.5 seconds and until 6.0 seconds at the ground motion intensity of 100%, respectively. The energy distribution factors at the intensity of 100% indicate almost no differences between in experiment and analysis. The deterioration of column bottoms at the first story early occurs by the local buckling at the intensity of 100%, which relieve the supports of the fixed condition at the column bases, in other words, the conditions of frame supports in analytical model might become similar to those in experiment. From these consideration, it may be said that the frame analysis successfully simulate the behavior of frame specimen up to the collapse. Figure 3.6 illustrates time history of the cumulative dissipated energy of a whole frame in experiment and analysis, where almost good agreement can be found.

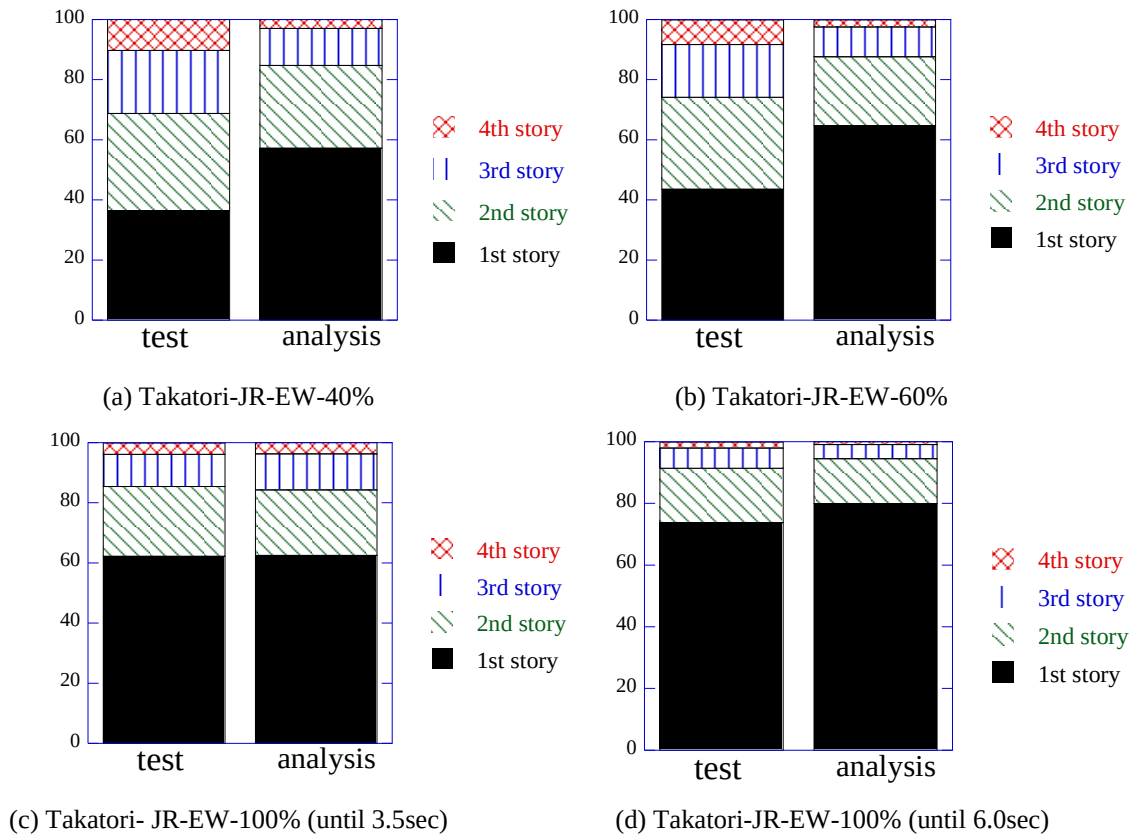


Figure 3.5 Hysteretic energy distribution of the 4-story steel frame

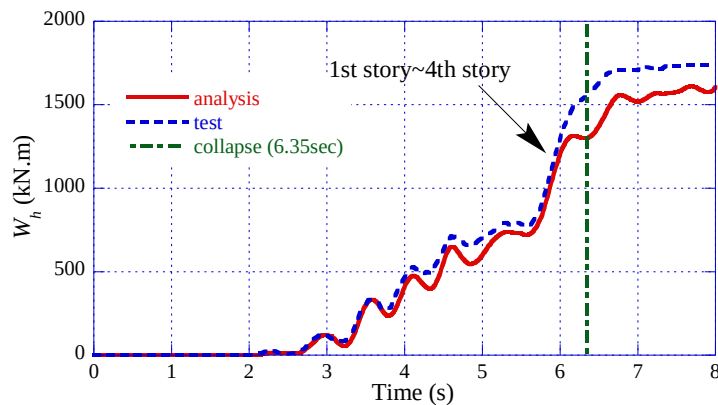


Figure 3.6 Collapse progress of the 4-story frame under 100% of Takatori-JR

3.3 Shaking Table Test of High-Rise Steel Building Subjected to Long-Period Ground Motions

A series of E-Defense shaking table tests on a high-rise steel building based on substructure method are simulated on the basis of the corresponding stress-strain models of steel components incorporating strength and stiffness deteriorations. Several related publications (Nagae et al. 2009, Chung et al. 2010, Matsumiya et al. 2010) have addressed on the seismic-resistant capacity of the high-rise building structure and the beam-column connections in the corresponding high-rise building.

Due to the test specimen is designed on the basis of the equivalent modified model of a 21-story generic high-rise steel building. The modified model as shown in Figure 3.7 consists of steel moment frame and three substitute layers instead of upper stories with the equivalent gravity and stiffness. These three substitute layers are respectively located at 9F, 14F and 19F of the prototype model. The substitute layers are constructed by concrete slabs, and the gravity is sustained by rubber bearings which are placed at the column positions of each layer. Besides, steel dampers are also adopted to simulate nonlinearity and energy dissipation of the upper stories with substitute layers (Chung et al. 2010). The analytical models adopted here in longitudinal and transverse directions are simulated on the basis of modified model (Figure 3.7) as shown in Figure 3.8.

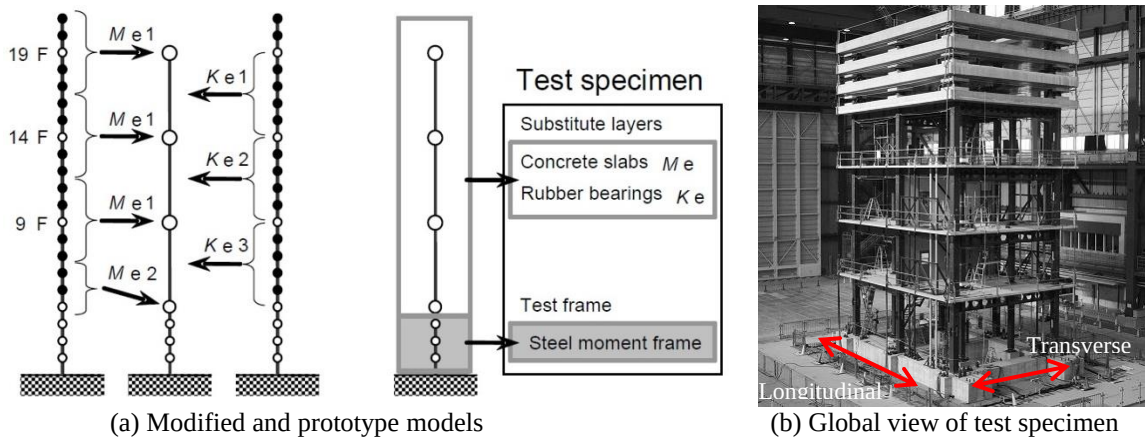


Figure 3.7 Prototype and test specimen of high-rise steel building (Chung et al. 2010)

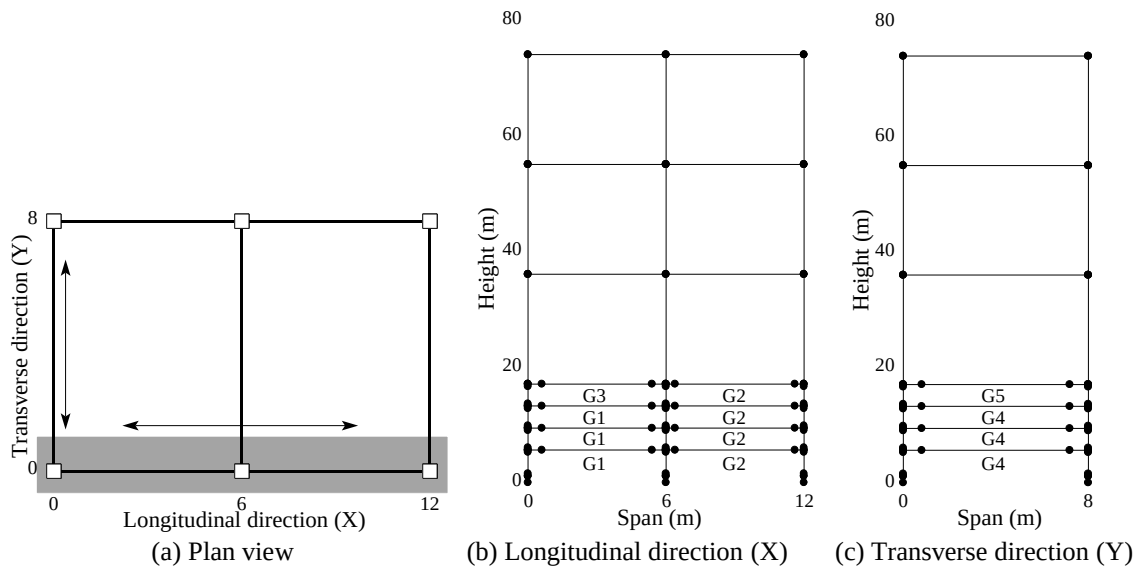


Figure 3.8 Plan and elevation views of analytical models

The geometric and material properties of girder and column structural components of high-rise building specimen are demonstrated in Table 3.2. Generic steel materials are used existent high-rise steel buildings in Japan. Besides, the El Centro earthquake waves (NS, EW) scaled to level two criteria with $PGV=0.5\text{m/s}$ and long period ground motions of San waves (NS, EW) are set as the input ground motions (see Figure 3.9).

Table 3.2 Geometric and Material properties of girder and column members

Components		Cross section	σ_y	σ_u
Column (SM490A)		$\square 400 \times 400 \times 25$	343	515
Girder (SM490A)	G1	H $600 \times 200 \times 9 \times 19$	357	532
	G2	H $400 \times 200 \times 8 \times 13$	426	576
	G3	H $500 \times 200 \times 9 \times 16$	366	532
	G4	H $800 \times 199 \times 10 \times 15$	377	526
	G5	H $650 \times 199 \times 9 \times 14$	344	514
Concrete			30	

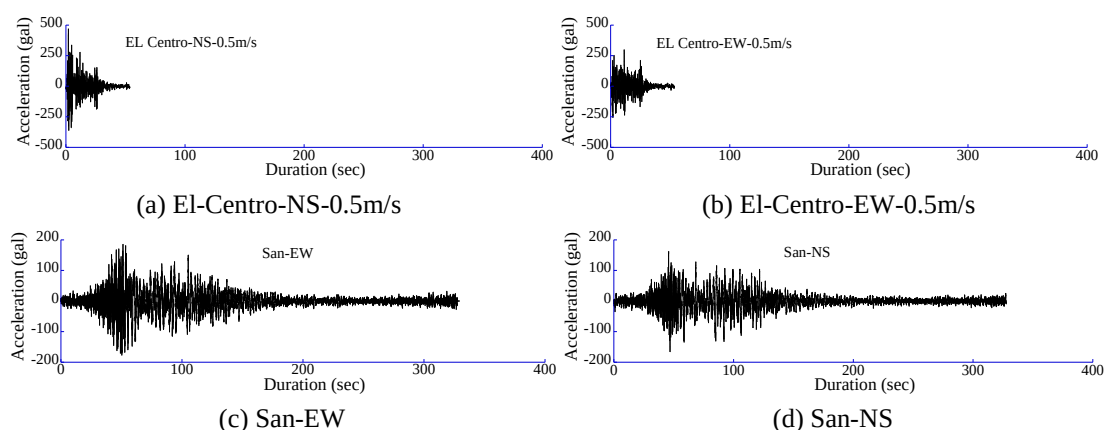


Figure 3.9 Time histories of input ground motion waves

According to the test results from white noise vibrations, the first mode natural periods of longitudinal (two span) and transverse (one span) directions are 2.13sec, 2.24sec, respectively. According to the eigen-value analysis of the two dimensional model, the comparisons of natural periods at first three modes of the test specimens are shown in Table 3.3. It can be seen that a quite good agreement is found between experiment and analysis.

Table 3.3 Comparisons of natural periods of the test specimens to analytical models

Direction	Method	1 st period (sec)	2nd period (sec)	3rd period (sec)
Longitudinal direction	Experiment	2.13	0.80	0.53
	Analysis	2.13	0.71	0.46
Transverse direction	Experiment	2.24	0.82	0.51
	Analysis	2.24	0.74	0.45

3.3.1 Longitudinal direction (X)

Figure 3.10 shows the comparisons between analytical and experimental results on the maximum story drift angle responses of the test specimen under various levels of ground motions on the longitudinal direction. It is noted that normal stories from first to fourth of a frame can be well simulated by analysis, even though some differentials are observed between experiment and analysis in the substitute stories. This is mainly because of the uncertainty of the story stiffness

couldn't be perfectly equalized. Therefore, with respect to the normal stories from the first to the fourth, it can be concluded that the analysis and the analytical model is capable to accurately simulate the experimental results.

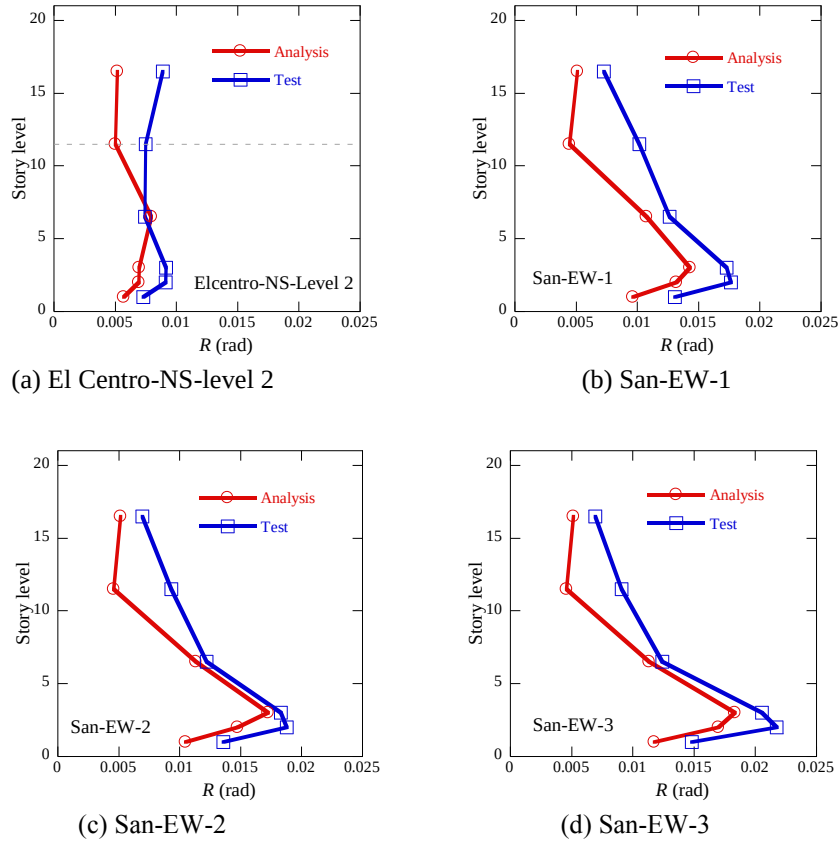


Figure 3.10 Maximum story drift angle in the longitudinal direction

Besides, the shear force distribution, velocity and acceleration responses of the test specimen are also simulated on the longitudinal direction, as shown in Figures 3.11~3.13. The maximum velocity responses of analytical results show higher correlated to those of test results in comparison with acceleration, e.g., the acceleration response at the bottom story under San-EW-1 [Figure 3.12 (b)] has been significantly overestimated by analysis program. According to the distributions of shear force as shown in Figure 3.13, the distribution of shear force is going to be better predicted when the test specimen under long-period ground motion (San-EW). Overall, the seismic responses of test specimen could be predicted relatively accurate, even if the characteristics of ground motions highly affect to the prediction.

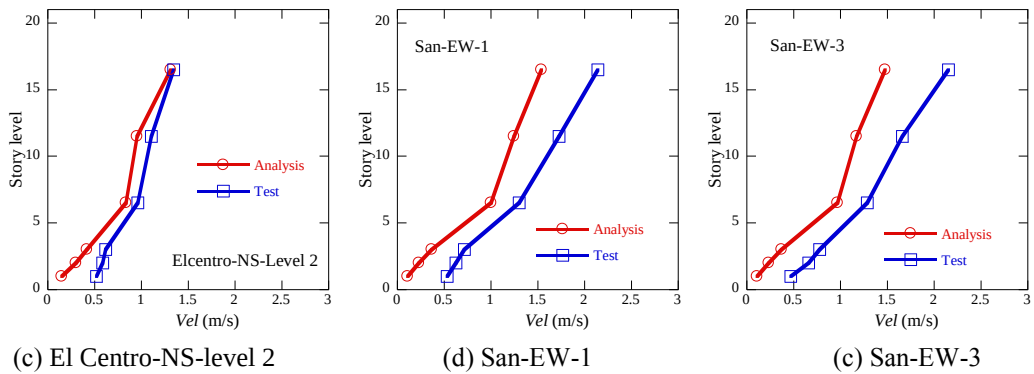


Figure 3.11 Velocity responses on the longitudinal direction

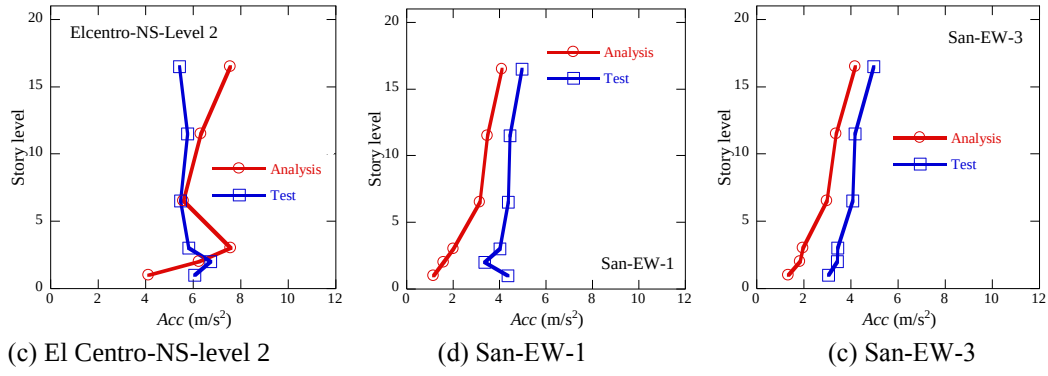
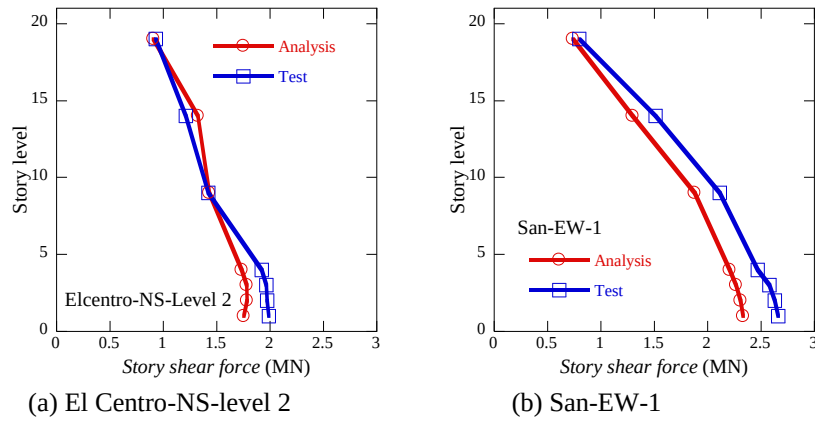
**Figure 3.12** Acceleration responses on the longitudinal direction**Figure 3.13** Story shear force responses on the longitudinal direction

Figure 3.14 shows the energy dissipation of each end of girders from the second to fourth floors. Firstly, from the comparisons between the energy dissipation of the left girder (G1) and right girder (G2), it is obvious that asymmetric responses of girder energy dissipation were noted in all three floors under both El Centro-NS-level 2 and San-EW-1 loadings. This phenomenon indicates that the side-sway deformation of the test specimen has been induced, and left girders may tend to absorb more and more energy than right girders until the one side-sway collapse occurs because asymmetric fractures cause the unbalance of the left and right girders. Analytical results relatively underestimate the asymmetric energy dissipation. Fractures at the lower flange of girders occurs at the test specimen may be a reason for above observations, because the fracture has not been incorporated in analytical program. Besides, two-axis buckling of steel members happens in three-dimensional specimen is the also the mainly reason that simulated results is smaller than those of experimental results. Anyway, this is a difficult point needs to be further thought, especially on the basis of the geometric and material nonlinearity have already been taken into account by analytical program.

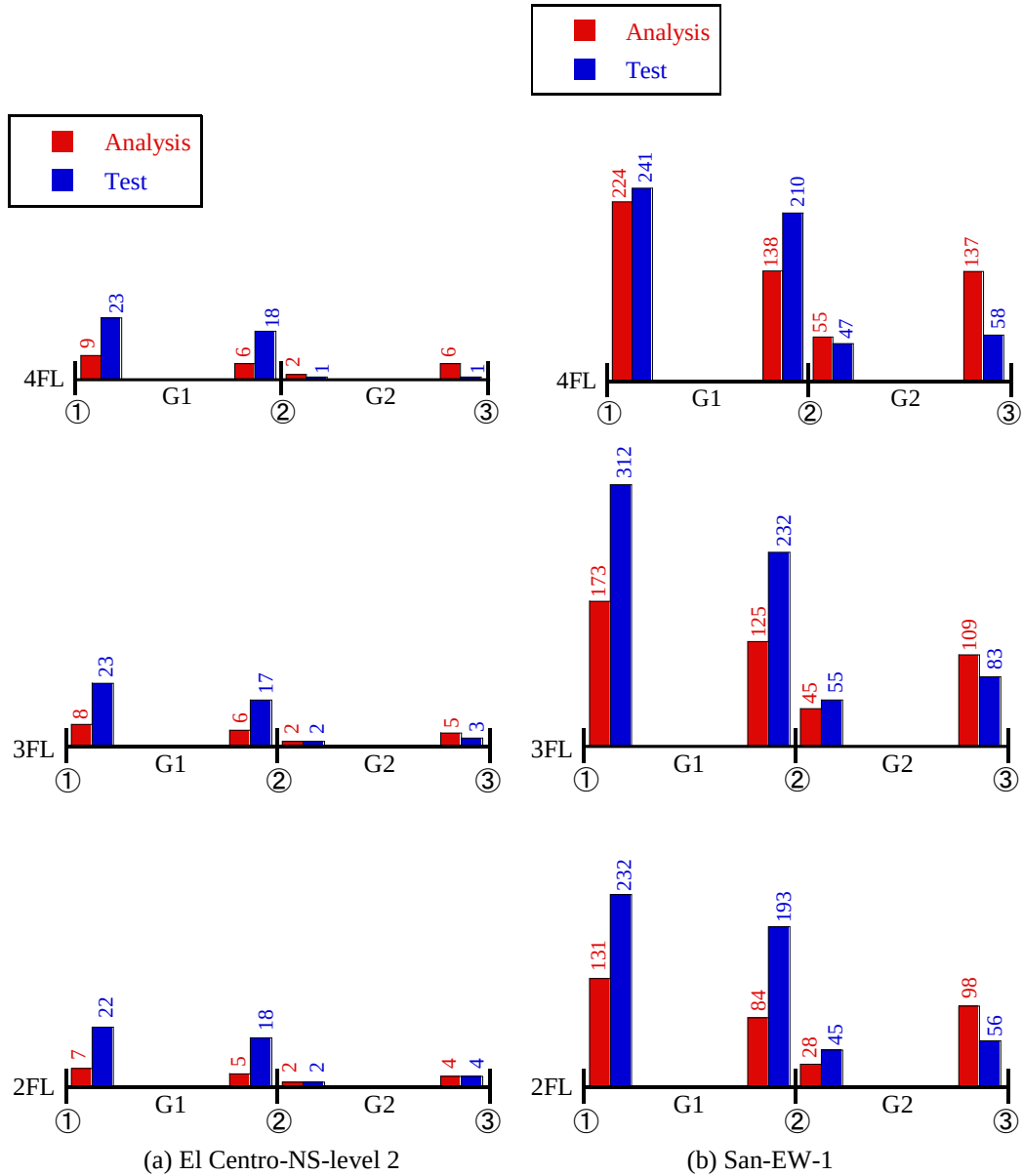


Figure 3.14 Hysteresis energy responses on the longitudinal direction

Figures 3.15~3.17 (a) show the test results of the story shear force and drift angle relationships of the lower three stories under El Centro-NS-level 2, San-EW-1, San-EW-3, and the corresponding analytical results are also demonstrated in Figures 3.15~3.17 (b). Shear force coefficient is derived as the time history acceleration normalized by the floor weight of the upper stories. Interstory drift angle is supposed to be expressed as θ .

Based on the test and analytical results obtained from the El Centro-NS-level 2 (see Figure 3.15), high agreement of hysteresis responses is achieved, which indicates the seismic performance of the longitudinal test specimen during elasto-plastic range (level 2 criteria allows the certain extent plasticity of structure) can be accurately predicted. Nevertheless, in accordance with the discussions addressed previously with respect to the hysteresis energy dissipation, differentials between analytical and test results in the case of San-EW-3 significantly grew. According to the phenomenon observed during experiment in longitudinal direction (Chung et al. 2010), four fractures being occurred at the right girders at third and fourth floors deteriorate the story stiffness

by 44% at most make the drift increase correspondingly. The effect of fracture is not incorporated by program, and no stiffness decreasing was derived as well.

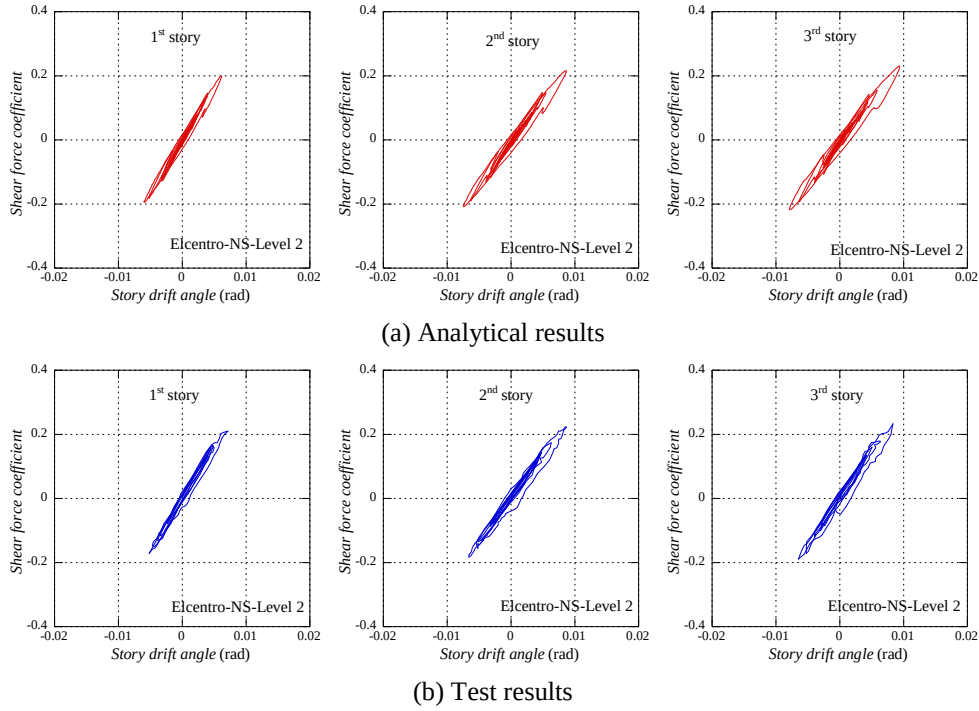


Figure 3.15 Story shear force coefficient and drift angel subjected to El Centro-NS-level 2 loading

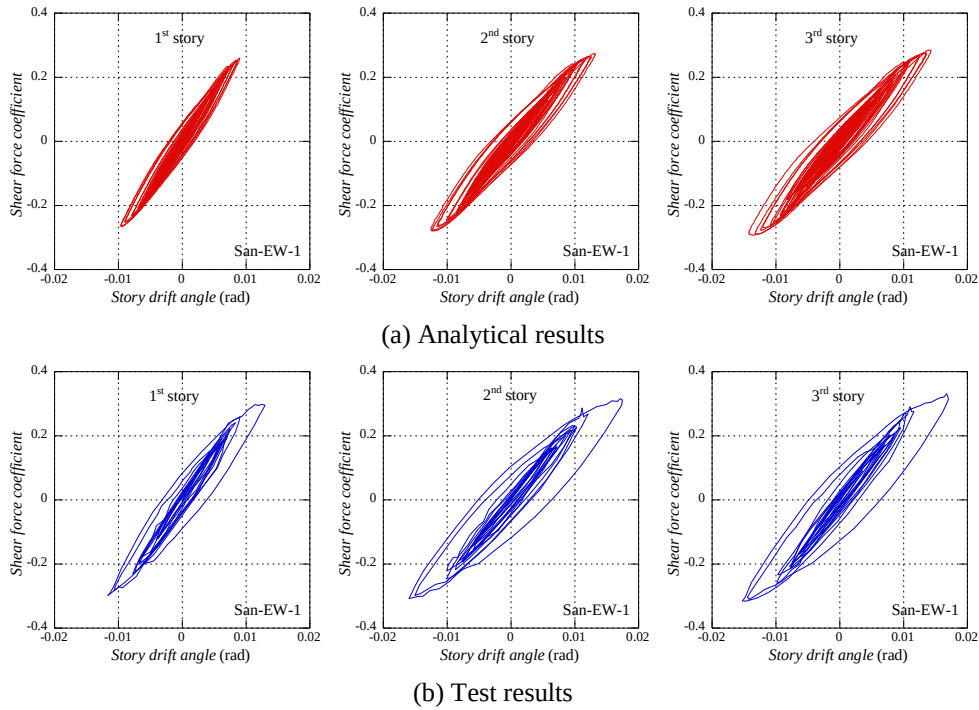


Figure 3.16 Story shear force coefficient and drift angel subjected to San-EW-1 loading

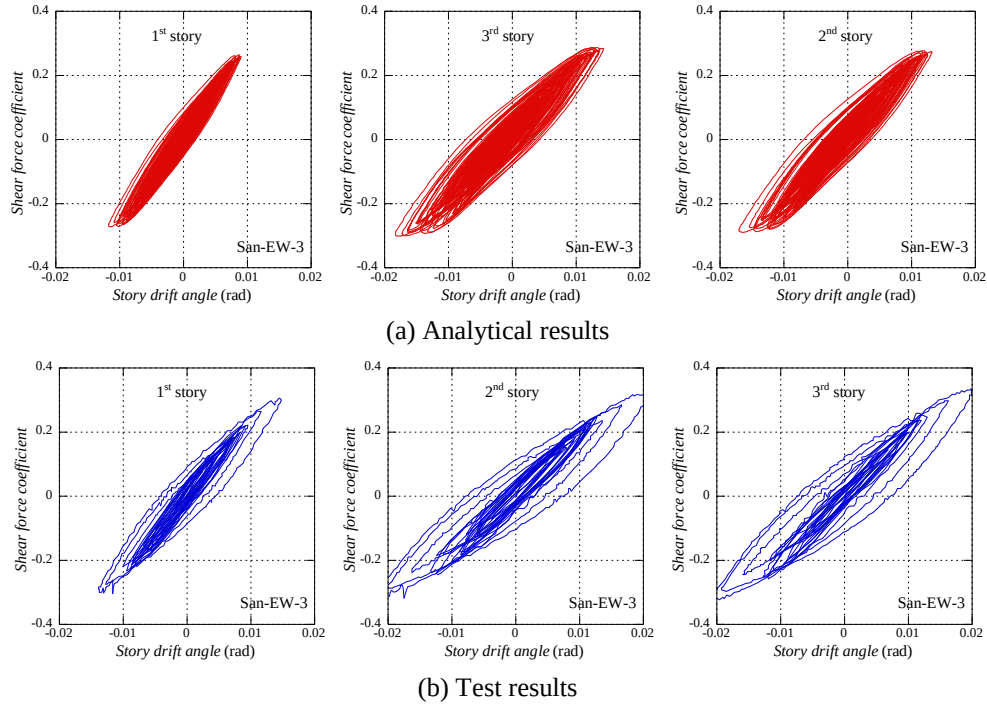


Figure 3.17 Story shear force coefficient and drift angel subjected to San-EW-3 loading

3.3.2 Transverse direction (Y)

Likewise, transverse direction of the test specimen is also modeled by stress fiber model analysis program, as shown in Figure 3.8 (b). The maximum interstory drifts of transverse test specimen are analogously predicted by analysis program as shown in Figure 3.18. Together with the comparison of the maximum drift responses (in Figure. 3.18), the velocity, acceleration, and story shear force distribution of the transverse test specimen are respectively predicted by program as shown in Figures 3.19~3.21. Among various seismic responses of the transverse test specimen simulated by program, story drift and shear force distribution are more accurately calibrated. Velocity and acceleration responses from first story to third story relatively underestimated.

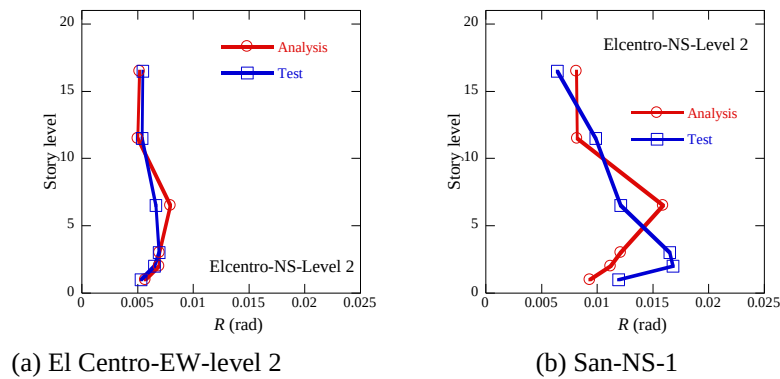
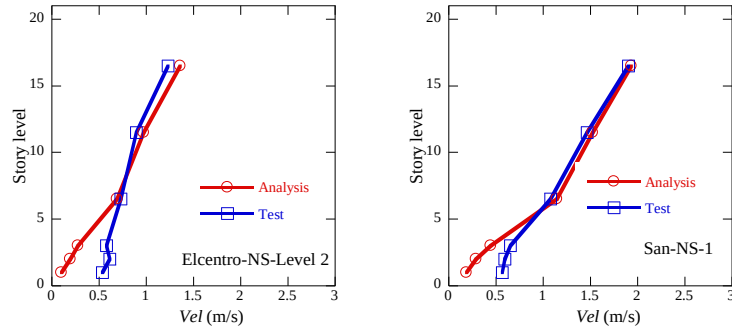
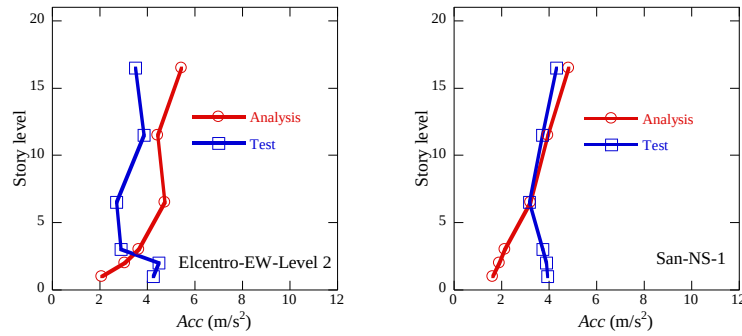


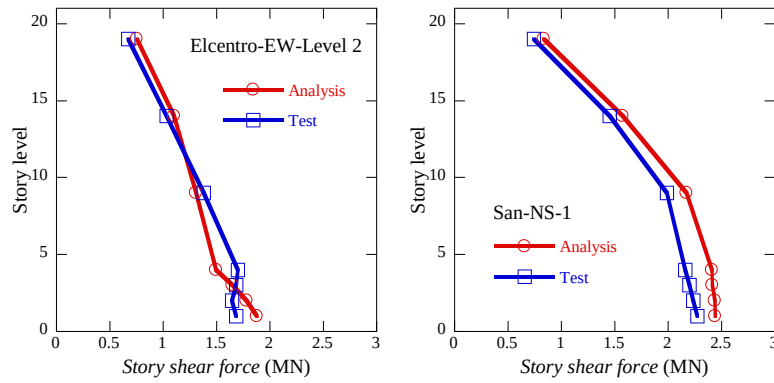
Figure 3.18 Peak interstory drift ratios on the transverse direction



(a) El Centro-EW-level 2 (b) San-NS-1
Figure 3.19 Velocity responses on the transverse direction



(a) El Centro-EW-level 2 (b) San-NS-1
Figure 3.20 Acceleration responses on the transverse direction



(a) El Centro-EW-level 2 (b) San-NS-1
Figure 3.21 Story shear force responses on the transverse direction

Moreover, the story force and drift angle relationship subjected to El Centro-EW-level 2, San-NS-1 are also calibrated as shown in Figures 3.22~3.23. In the transverse direction, the fractures being occurred at the bottom flange of the left girders from second to fourth floors significantly deteriorate the story stiffness in the positive direction [see Figure 3.23 (b)]. In addition, two-axis directional deformation of experimental specimen is also the important factor of increasing the component damages which is not considered in two-dimensional analytical simulation.

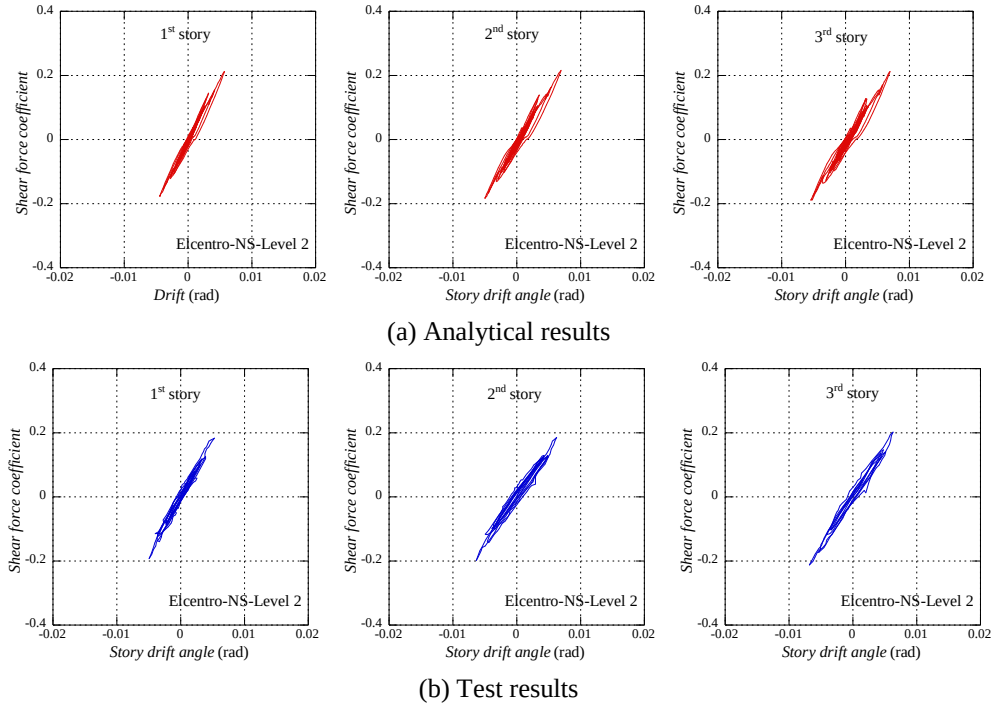


Figure 3.22 Story shear force and drift angel subjected to El Centro-EW-level 2 loading

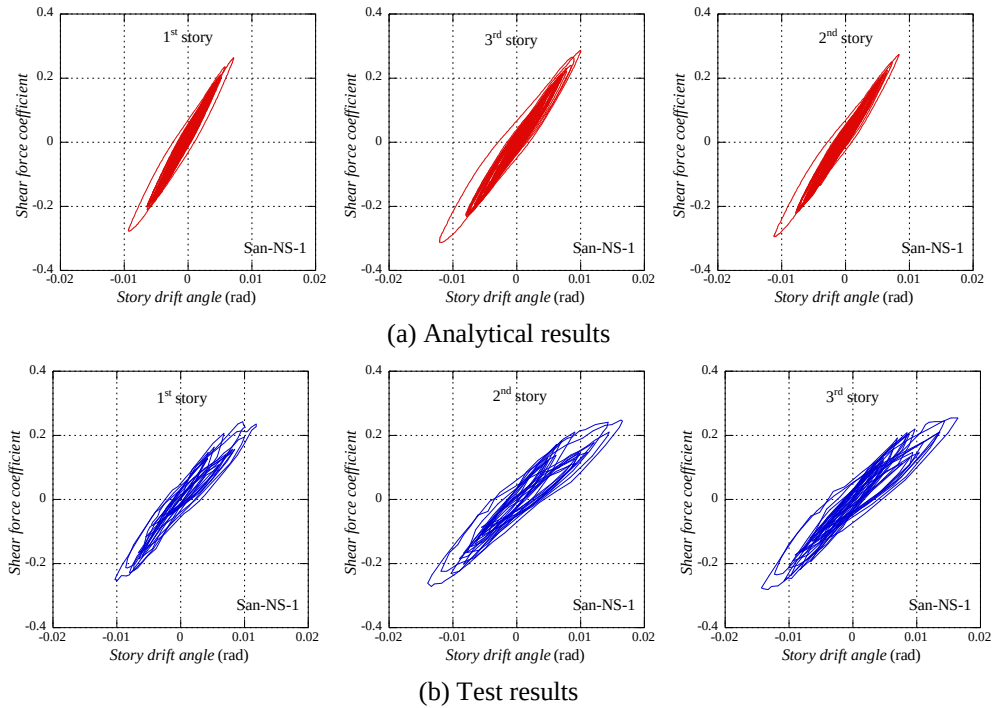


Figure 3.23 Story shear force and drift angel subjected to San-NS-1 loading

3.4 Concrete Filled Steel Tubular Frame

Similarly, the deterioration behaviour of concrete filled steel tubular frame ought to be validated to prove the accuracy and feasibility of the unified constitutive models for CFT frames. The reference on the experiment of CFT frame structures has been rarely reported, so a static cyclic experiment (Sakai and Matsui 2001) of a planar CFT frame (square CFT columns and H-shaped

steel girders) is simulated. Figure 3.24 shows the elevation view of CFT frame specimen and the two-dimensional modelling of this frame specimen.

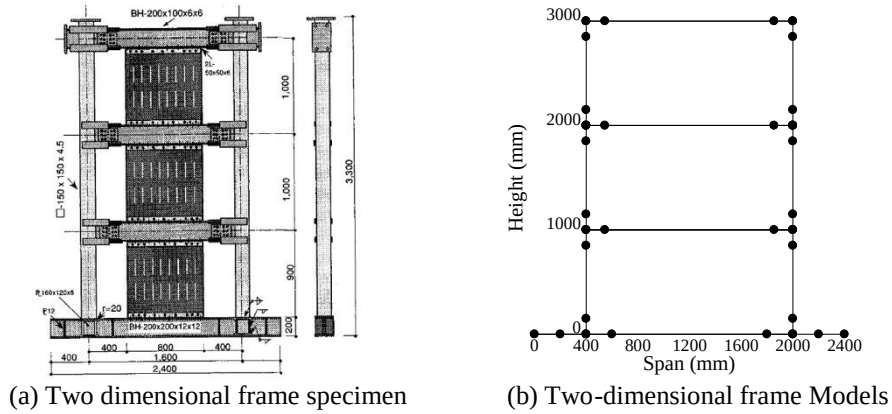


Figure 3.24 Modelling of CFT frame specimen

The cross section of the column of CFT frame specimen is uniformly fabricated to be $150 \times 150 \times 4.5$, with steel material STKR400 $\sigma_y = 409 \text{ N/mm}^2$ and $\sigma_u = 477 \text{ N/mm}^2$. Likewise, BH-200 $\times$ 100 $\times$ 6 $\times$ 6 H-shaped steel of steel material SS400 $\sigma_y = 337 \text{ N/mm}^2$ and $\sigma_u = 444 \text{ N/mm}^2$, is adopted to be the girder of the CFT frame specimen. The strength of the confined concrete in CFT columns is 31.3 N/mm^2 based on four concrete cylinder test results. Loading procedure of this experiment is cyclic horizontal loading applied on the top of frame specimen, and constant axial loadings with axial load ratio $N = 0.2 \cdot N_u \times 2$ of two CFT columns is also applied simultaneously.

Comparisons of hysteretic load (H) versus roof drift ratio (R) curve between experimental and analytical results are shown in Figure 3.25~2.26. The comparison when $R \leq 2\%$ is compared in Figure 3.24, where visible but slight influence of deterioration is obtained. The simulation without considering deterioration effect significantly overestimates the strength and deterioration behavior of CFT frame specimen [see Figure 3.26 (a)], but the analytical results incorporating deterioration effect shows stable and close correlation with that experimental results [see Figure 3.25 (b)].

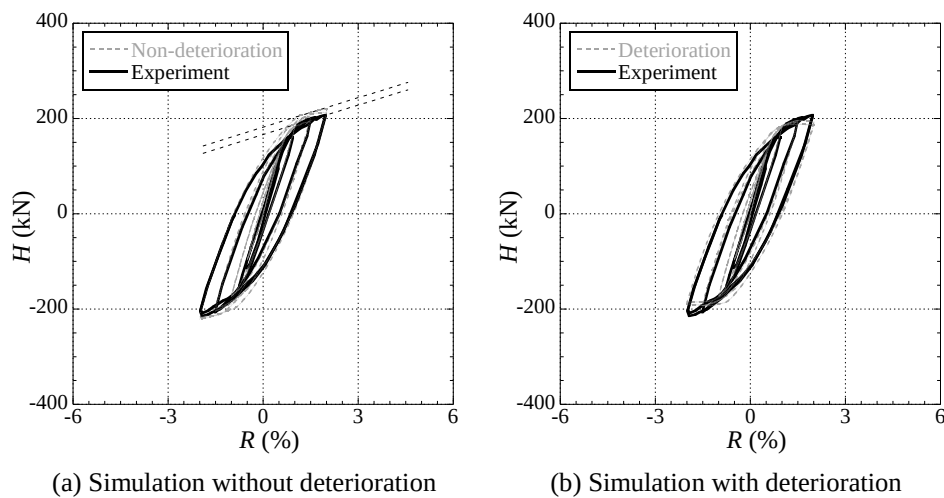


Figure 3.25 Comparisons between simulation results with experimental results until $R \leq 2\%$

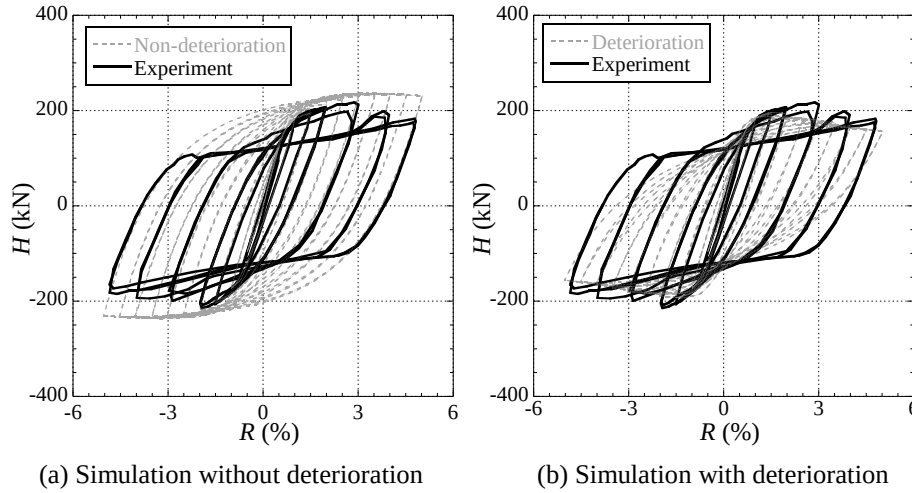


Figure 3.26 Comparisons between simulation results with experimental results until collapse

Through simulating a cyclic test of CFT frame structures, the unified constitutive models with deterioration effect can accurately predict the deterioration behaviour of the CFT frame structure. Ignoring the deterioration effects of steel tube and confined concrete will significantly overestimate the seismic resistant capacity if CFT frame structures. Hence, deterioration effect should be rigorously considered in simulating the seismic performance of high-rise CFT building structures.

3.5 Conclusions

In this Chapter, a full-scale shaking table test of 4-story steel moment-resistant frames subjected to incremental earthquake loadings was used for the calibration of stress fiber model analysis employing the stress-strain models incorporating strength and stiffness deteriorations, in order to predict the collapse capacity of the steel frame under severe seismic excitation. Besides, due to the assessing on the collapse safety of existent high-rise buildings are urgent to be evaluated because the long-period ground motions tend to produce a lot more responses on such high-rise buildings, another E-Defense shaking table test of a high-rise steel building subjected to long-period ground motions is also used for the validation of the unified constitutive model. In addition, a CFT frame specimen subjected to cyclic horizontal loadings with constant axial loading is also simulated in this chapter.

(1) Overall, the analysis and analytical model is a plane frame takes the geometric nonlinearity into account, and calibration on the test specimen of 4-story steel frame by X-direction and Y-direction based on two dimensional modelling shown feasible to predict the collapse. The story drift of bottom story of the test specimen is the most difficult part to be accurately predicted, because the local buckling of the columns at first story makes the seismic response calculation instable, especially subjected to 100% Takatori-JR loading. However, the collapse phenomenon of the test specimen was accurately predicted on both X and Y directions. Based on percentage of hysteresis energy absorption at each story, bottom story dissipates around 80% energy under 100% Takatori-JR loading.

(2) With respect to the shaking table test on the high-rise steel buildings subjected to long-period ground motions, story shear stiffness plays an important role on the predicting the seismic responses. According to the test phenomenon during loading, fractures occurred at the bottom flanges of girder at connections deteriorate the story shear stiffness of the test specimen. Subsequently, the story shear force and story drift angle relationship is significantly affected by the

reduction of story shear stiffness subjected to San-NS/EW-3 loading. Due to the fracture is not incorporated in the analytical frame model and the two-axial buckling happened in the three dimensional test specimen could not be simulated in two-dimensional planar frame model. So reasonably errors are observed in the comparisons between experimental and analytical results.

(3) Besides, through simulating a cyclic test of CFT frame structures, the unified constitutive models with deterioration effect can accurately predict the deterioration behaviour of the CFT frame structure. Ignoring the deterioration effects of steel tube and confined concrete will significantly overestimate the seismic resistant capacity of CFT frame structures. Hence, deterioration effect should be rigorously considered in simulating the seismic performance of high-rise CFT building structures.

Although somewhat differences must be included in the modelling by two-dimensional plan frame model with respect to three dimensional frame test. The two dimensional frame models by adopting the unified constitutive models are reliably capable to predict the collapse and seismic responses of low-rise and high-rise steel and concrete filled steel tubular buildings.

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CHAPTER 4

EVALUATION OF SEISMIC-RESISTANT CAPACITY OF EXISTENT HIGH-RISE STEEL BUILDINGS

4.1 Introduction

Tokachi-oki Earthquake ($M_w=8.0$) on September 26th of 2003 generated serious fire disasters on oil-tanks, due to resonance effects at long-period branch of structures (oil-tanks) and the earthquake ground motions (AIJ 2007). Afterwards, new understanding on the threat of severe earthquakes with long period components are recognized on the seismic safety of building structures with long natural periods such as high-rise buildings. The 2011 Great East Japan earthquake and induced Tsunami (AIJ 2011) have taken devastating losses on human life, buildings and infrastructures near seismic epicentre, and simultaneously, long period ground motions are observed at Tokyo, Nagoya and Osaka regions, where high-rise buildings resonate over five minutes. Meanwhile, high probability of severe earthquakes with magnitude class over 8.0 along the Tokai, Tonankai and Nankai sea troughs of Japan have also been predicted, in next certain years. Particularly, high-rise buildings located in basin area whose period margin between 3sec~6sec are expected to face extreme challenges and damages by severe earthquakes occurring nearby.

According to the nearly 40-year constructional history of high-rise buildings in Japan, destructive severe earthquake has rarely been experienced. Thus, various extensive existent high-rise buildings face urgent challenges of seismic safety under probable severe ocean-ridge earthquakes and near-fault earthquakes. For the high-rise buildings under earthquake excitations, the P- Δ effect and strength deterioration induced by local buckling of steel components and/or crush of concrete could lead to extreme permanent damages or even collapse, especially when it resonates with long period ground motions generated by severe earthquakes with high level magnitude. In this chapter, typical existent high-rise buildings composed of steel members are investigated.

Several studies on seismic performance and collapse behavior of steel frame were also addressed by some representative researchers, e.g. symmetry limit theory for steel beam-columns originally revealed the steady-state path of cantilever members (Nakamura and Uetani 1989, 1990). Since then, the symmetry limit theory has been extended to reveal the phenomenon of deformation concentration at lower stories in the process of dynamic excitation even for the weak-beam-type frames (Uetani et al. 1996). Probability of collapse and design for collapse safety for structural systems based on the study of simplified performance based earthquake engineering were assessed (Zareian and Krawinkler 2007). Influence of P- Δ effect on dynamic response of high-rise steel buildings under extreme seismic excitations was analytically studied (Kim and Araki 2009). Seismic resistance capacity of high-rise buildings subjected to long-period ground motions was

studied on the basis of E-Defence shaking table tests (Chung et al. 2010). Sidesway collapse of deteriorating structural systems under seismic excitations was analytically and experimentally addressed (Lignos and Krawinkler 2011). However, the strength deterioration such as local buckling has not been fully investigated in the evaluation of seismic-resistant capacity of high-rise building.

In Chapter 4, existent high-rise buildings are simulated by two dimensional model of moment-resistant steel frame with H-shaped steel girders and square hollow steel tubular columns. Both deteriorating and non-deteriorating stress-strain models for such steel components are respectively employed to investigate the seismic-resistant capacity of high-rise steel buildings with consideration of only P- Δ effect, and both of P- Δ effect and member deterioration, respectively.

Accurate instantaneous stress-strain models for the steel members in high-rise building models are based on validations in Chapters 2~3. Meanwhile, as the basis of Chapter 4, preliminary studies on the calibration and validation of stress-strain models for various components including steel members with consideration of strength deterioration were published (Bai and Kawano et al. 2011).

4.2 Statistics of Existent High-Rise Steel Buildings in Japan

Since the limitation on the height of building by building standard law in Japan is abolished on 1963, high-rise buildings started to be constructed for nearly 50 years by far. Nowadays, development of high-rise buildings in various stages reflects technology progress and economic growth, e.g., 1963~1971 as initial stage, 1972~1981 as second stage, 1982~1991 as third stage and 1992~ as the fourth stage (AIJ, 2007). Each constructional decade of high-rise buildings has different characteristics of structural system, material type, overall and story heights, use function of building, etc. Fukushima et al. (1999) statistically investigated existent high-rise buildings until 2000 in Japan. According the statistic database (Fukushima 1999), high-rise steel buildings existing in Japan are mainly used as office buildings by function, and Figure 4.1 shows statistical database of existent high-rise steel office buildings.

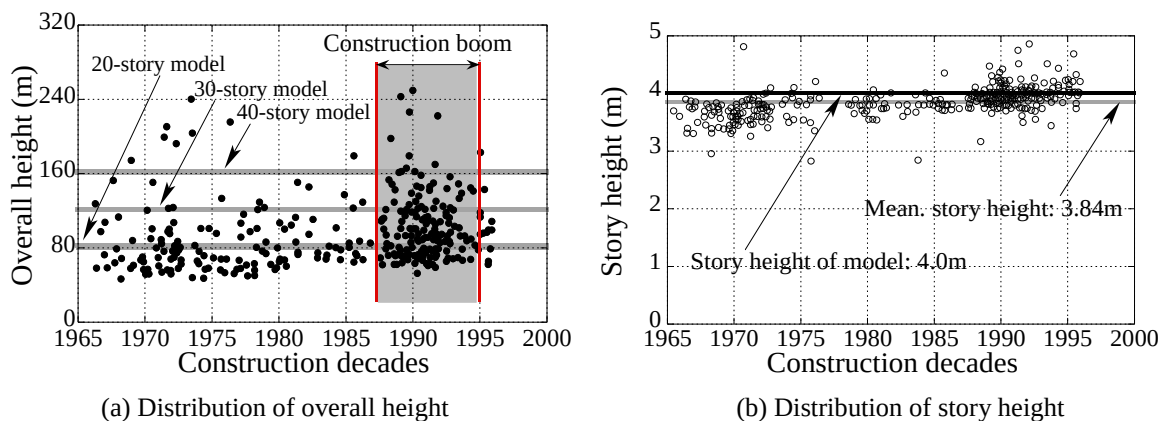


Figure 4.1 Statistical databases on existent high-rise steel office buildings (Fukushima 1999)

According to the distributions of overall and story heights of existent high-rise steel office building, three overall heights are considered to be 81meters (20 story model), 121 meters (30 story model), and 161meters (40 story model), in order to basically cover the ranges of overall heights as shown in Figure 4.1(a). Story height for each story model, as shown in Figure 4.1(b), is considered as 4.0 meter. With respect to span of high-rise steel office buildings, 8m is assumed as standard span in high-rise building models, in accordance with the common knowledge that the range of

span is centered on 6m~9m. Furthermore, steel materials of SM490 and SN490 are the most frequently used in existing steel office buildings, therefore the steel material SM490 and SN490 with the nominal strengths of $\sigma_y=320\text{N/mm}^2$ (σ_y is yield strength) and $\sigma_u=490\text{N/mm}^2$ (σ_y is ultimate tensile strength) are considered as the priority in the modelling of high-rise steel buildings in this chapter.

Challenges on seismic safety of these existing high-rise buildings appear, especially for the buildings subjected to severe long-period ground motions, because such long-period ground motions induced by severe ocean-ridge earthquakes has rarely occurred at the big cities where a lot of high-rise buildings have been built. Furthermore little attention has been paid on the resonance probability with the long-period pulse-like ground motions which are highly expected to be occurred at Hanshin district.

4.3 Deterioration Characteristics of Steel Members

Accurate stress-strain models for rectangular steel tube and H-shaped steel members associated with strength deterioration caused by local-buckling, developed and calibrated in Chapter 2, are utilized to assess the effect of strength deterioration of steel members on the seismic safety and collapse behavior of high-rise steel buildings, so that rectangular steel tubular column and H-shaped steel girders are predominantly adopted in existent high-rise steel buildings during 1980~1990s. According to Chapter 2, a stress-strain model for steel members incorporating strength deterioration (i.e. local buckling) is developed and calibrated, as shown in Figure 4.2. Local buckling of H-shaped steel members is significantly affected by parameters like width-to-thickness ratio of web $[(D-2t_f)/t_w]$, where D , t_f and t_w denote depth of cross section, thickness of flange and thickness of web, respectively] and flange $(B/2t_f)$, where B denotes width of cross section), and yield strain $\varepsilon_y (= \sigma_y / E_s)$, where σ_y and E_s denote the yield stress and Young's modulus, respectively), as shown in Table 4.1 (Yamada et al. 1993). Simultaneously, local buckling of hollow steel tubular members is affected by such as width-to-thickness ratio (B/t) , where B and t denote outside width of tube and tubular wall thickness, respectively) and ε_y .

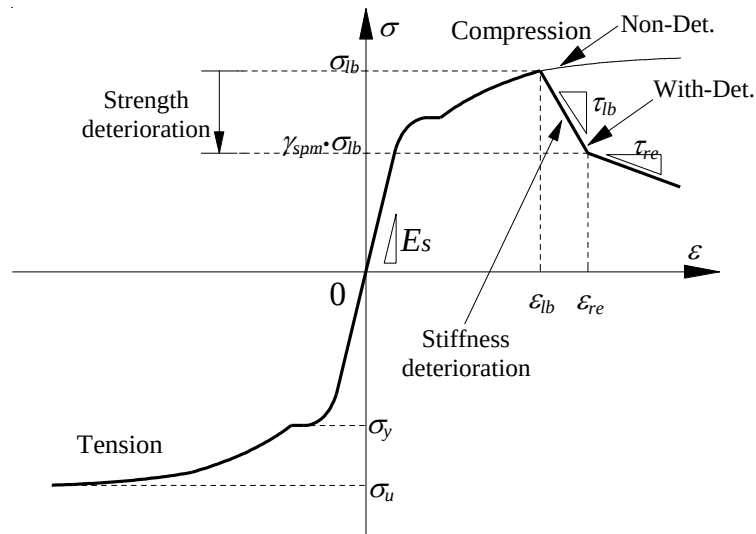
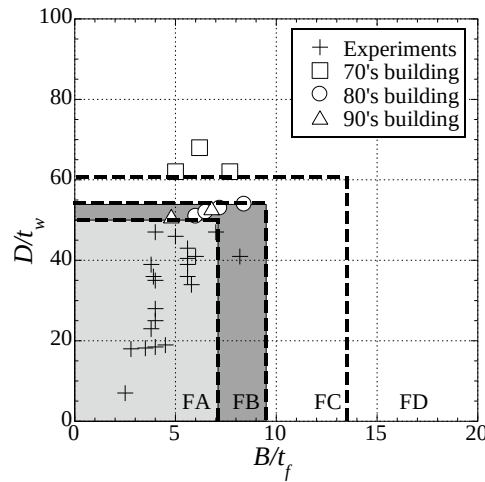


Figure 4.2 Stress-strain deteriorating model for steel member

Table 4.1 Deterioration parameters of steel members in high-rise building models

Deterioration parameters	H-shaped steel girder		Square hollow steel tubular column
ε_{lb}	$\text{Max.}[(0.18/\alpha_f)+(2.6/\alpha_w)+0.3, (0.5/\alpha_f)+(5.7/\alpha_w)-4.0] \cdot \varepsilon_y$	ε_{lb}	$[(8.7/\alpha)-1.2] \cdot \varepsilon_y$
γ_{spm}	$-0.062\alpha_w-0.56\alpha_f+0.98$	γ_{spm}	$-0.079\alpha+0.81$
τ_{lb}	$-0.0046\alpha_w^2-0.57\alpha_f^2-0.0005$	τ_{lb}	$-0.014\alpha^2-0.005$
τ_{re}	-0.003	τ_{re}	-0.005
α_w	$[(D-2t_f)/t_w]^2 \cdot \varepsilon_y$	α	$(B/t)^2 \cdot \varepsilon_y$
α_f	$(B/2t_f)^2 \cdot \varepsilon_y$		

According to the illustration in Table 4.1, width-to-thickness ranks [i.e. $(D-2t_f)/t_w$ and $B/2t_f$ for H-shaped steel member, B/t for square hollow steel tubular member] of steel members have influences on stiffness deterioration ratio (τ_{lb}) and strength deterioration ($\gamma_{spm} \cdot \sigma_{lb}$) after local-buckling. On the basis of the statistics on the compactness ranks of steel members in existent high-rise steel buildings as shown in Figure 4.3 (AIJ 2007), H-shaped steel girders for buildings existing in 80's~90's are mainly FA and FB ranks in terms of the Enforcement Ordinance of Construction Standard Law published by the Ministry of Land, Infrastructure and Transport and Tourism of Japan (2007). Therefore, the effects of deterioration on the seismic safety and collapse resistant capacity of high-rise steel buildings are investigated by adopting different ranks of width-to-thickness ratios of FA and FB for H-shaped steel girders.

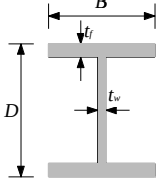
**Figure 4.3** Width-to-thickness ratios of 490MPa H-shaped steel girder in existent high-rise buildings

With respect to square steel tubular columns, the most compactness rank of FA is popularly used for the high-rise buildings in those days. However, in order to know the influences of the compactness of tubular column sections on frame behavior, frame models made of FB ranked columns as well as those of FA ranked columns are numerically investigated.

4.3.1 H-shaped steel girder

According to the statistical database on existent high-rise steel buildings in Japan, H-shaped steel is predominantly applied as the girders. Compactness ranks of H-shaped steel members based on the Enforcement Ordinance of Construction Standard Law published by the Ministry of Land, Infrastructure and Transport and Tourism of Japan (2007), as shown in Table 4.2.

Table 4.2 Compactness ranks of H-shaped steel members

Section	Member	Part	Width-thickness ratio	
			FA	FB
	Column	Flange (B/t _f)	$9.5\sqrt{235/F}$ (8.08 for $F=325\text{N/mm}^2$)	$12\sqrt{235/F}$ (10.2 for $F=325\text{N/mm}^2$)
		Web (H/t _w)	$43\sqrt{235/F}$ (36.6 for $F=325\text{N/mm}^2$)	$45\sqrt{235/F}$ (38.3 for $F=325\text{N/mm}^2$)
	Beam	Flange (B/t _f)	$9\sqrt{235/F}$ (7.65 for $F=325\text{N/mm}^2$)	$11\sqrt{235/F}$ (9.35 for $F=325\text{N/mm}^2$)
		Web (H/t _w)	$60\sqrt{235/F}$ (51 for $F=325\text{N/mm}^2$)	$65\sqrt{235/F}$ (55.25 for $F=325\text{N/mm}^2$)

Where, F is yield strength of steel materials

SN490 and SM490: $F=325\text{N/mm}^2$

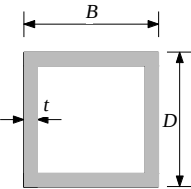
For SN490 and SM490: $\sqrt{235/F} = 0.85$

H-shaped steel girders with FA and FB ranks are assumed in terms of manufacturing version of corresponding steel members (JFE, 2010), as shown in Appendix B (Table B.1). For the deterioration model of H-shaped steel girders, parameters can be calculated by the formulae in Table 4.1. The strain at local-buckling (ε_{lb}) at the starting point of deterioration of FB H-shaped steel girders (0.009~0.014) obviously shows smaller than that of FA girders (0.004~0.013). This indicates that earlier occurring of local buckling for FB girders shows lower ductility capacity than FA girders to avoid the generation of strength and stiffness deterioration. The stiffness of post-peak deterioration ratio (τ_{lb}) of FA girders demonstrate smaller values than that of FB girders, which further means that the descending rate of FA girders shows milder descending gradient than FB girders. For SM490 and SN490 steel material, yield strength $\sigma_y=325\text{N/mm}^2$, ultimate strength $\sigma_u=490\text{N/mm}^2$ and elastic modulus $E_s=205000\text{ N/mm}^2$.

4.3.2 Square hollow steel tubular column

Similarly, the compactness ranks (FA, FB) of square hollow steel tubular columns specified in the Enforcement Ordinance as shown in Table 4.3, are also considered as the levels of deterioration of interior columns and exterior columns in high-rise steel building models.

Table 4.3 Width-thickness ratio criteria of rectangular HST member

Section	Member	Criteria of Width-thickness ratio (B/t)	
		FA	FB
	Column	$< 33\sqrt{235/F}$ (28.1 for SM490 and SN490)	$33\sqrt{235/F} < B/t < 37\sqrt{235/F}$ (31.5 for SM490 and SN490)
	τ_{lb}	< 0.04	< 0.06
	ε_{lb}	> 0.0086	> 0.0064
	$\varepsilon_{lb} / \varepsilon_y$	> 4.31	> 3.18
	γ_{spm}	> 0.69	> 0.65

* For SN490 and SM490: $\sqrt{235/F} = 0.85$

A variety of deterioration parameters (as shown in Table 4.4) have obvious differentials between FA and FB members, e.g., for the post-peak stiffness deterioration ratio τ_{lb} , FA member

(0.04) likely to demonstrate nearly 50% smaller stiffness than that of the FB member (0.06), which means FA members more mildly deteriorate after local-buckling. Besides, the other deterioration parameters (e.g., ε_{lb} : strain at local-buckling, γ_{spm} : strength deterioration ratio) of FA member show more ductile behavior than that of the FB member altogether. Thus, width-to-thickness ratio (B/t) can be considered as a reasonable parameter to represent member deterioration of square hollow steel tubular column. Based on the B/t criteria of square hollow steel tubular members (as shown in Table 4.3), the square hollow steel tubular columns satisfying FA and FB criterion are assumed as the corresponding columns in high-rise steel building models, by utilizing the existing sections in industrial manufacture (JFE, 2010). Details including section, parameters of deterioration model of the assumed interior and exterior square hollow steel tubular columns are respectively shown in Appendix B (Tables B.2~3). For instant, as a represented parameter, the stiffness deterioration ratio (τ_{lb}) to story levels is shown in Figure 4.4.

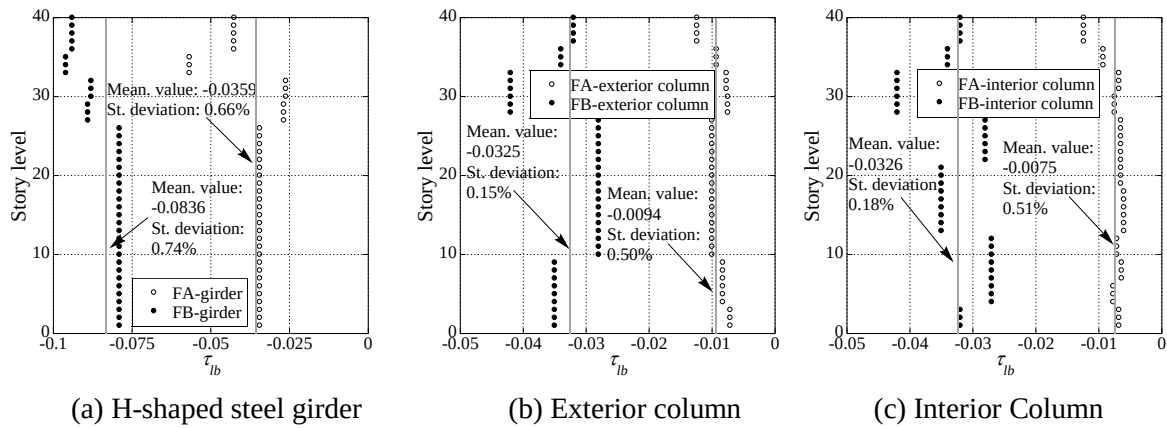


Figure 4.4 Stiffness deterioration ratio (τ_{lb}) of the steel members in high-rise steel building models

4.4 Seismic Design of Building Models

4.4.1 Design criterion

Due to that the objectives in this chapter are various existent high-rise steel buildings, current seismic design codes should be followed in the seismic design of high-rise building models. Thus, according to current Japanese seismic design standards (AIJ 2007, Kim 2010), the design base shear coefficient C_B is obtained by

$$C_B = Z \cdot R_t \cdot C_0 \quad (4.1)$$

Where, C_0 is the standard shear coefficient that is assumed as $C_0=0.3$ which indicates median seismic demand for buildings. Z represents the seismicity of the region which is assumed as 1.0 where Tokyo, Nagoya and Osaka are located in. R_t indicates the type of site that is a fraction of T_1 and T_c , by in which, the site period $T_c=0.6$ based on the assumption of type 2 soil that denotes median hardness ground.

The damping factor of steel-framed buildings is empirically assumed as identical value of 2%, in order to eliminate the effect of damping factor on the seismic responses varied high-rise steel buildings. The floor load is set to be 7.84kN/m^2 , and the composite beam action stiffened by floor slabs with 150mm thickness and $b_a=0.8$ effective width is also taken into account (AIJ 2010). Moreover, for high-rise building, story levels significantly affect eigen value (vibration mode), base shear coefficient (C_B), P- Δ effect, and also axial force ratio of column members. Thus, 20, 30

and 40 story high-rise steel buildings are seismically designed by seismic design code (1981~1995);

Seismic design of various existent high-rise steel building in accordance with certain criterion (BCJ 2002) as follows:

(a) Due to the strong-column weak-beam basic concept is followed in moment-resisting frames, high-rise buildings with weak-beam mechanism should be guaranteed. Thus, the column over-strength factor (COF) of interior beam-column connections should satisfy the demand of $COF=1.5\sim2.0$, and $COF=3.0\sim4.0$ of exterior beam-column connections, in which, the column base at bottom story of building and the joint at roof story of building are excepted. The definitions of interior and exterior COF are shown in Figure 4.5

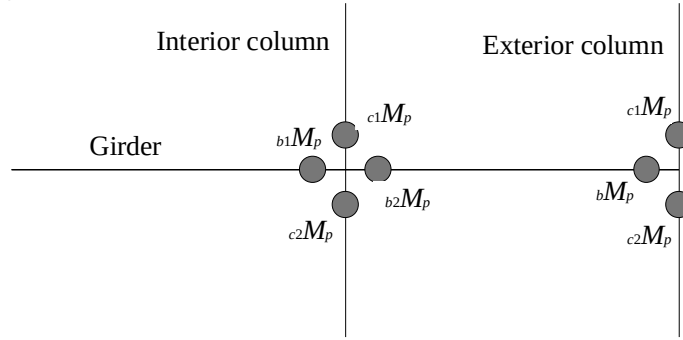


Figure 4.5 COF of interior and exterior beam-column connections

For the exterior beam-column connection, the column over-strength factor COF_{ex} is calculated by

$$COF_{ex} = \frac{c1M_p + c2M_p}{bM_p} \quad (4.2)$$

Similarly, for the interior beam-column connection, the COF_{in} is also calculated by

$$COF_{in} = \frac{c1M_p + c2M_p}{b1M_p + b2M_p} \quad (4.3)$$

Where, c_iM_p is the full plastic bending moment capacity of column that is calculated by considering the effect of axial load ratio, b_iM_p is the full-plastic bending moment capacity of H-shaped steel.

(b) Static design should be conducted, in order to evaluate the allowable stress of structural members, and demand of maximum story drift angle. In the static design, the horizontal force is based on the A_i distribution specified in Enforcement Ordinance by Construction Standard Law, and the design criterion that the maximum story drift angle θ_{max} is less than 1/200 is calibrated based on the allowable stress design method are expressed as follows;

Allowable stress design: $\sigma_b \leq f_b$, stress is equal to or less than the allowable stress, where σ_b is stress response of steel member, f_b is short-term allowable stress of steel member;

Performance demand: $\theta_{max} \leq 1/200$, where θ_{max} is maximum story drift angle.

(c) Since that the importance of earthquake velocity has been focused from 1980s in the dynamic design, where the peak ground velocity (PGV) is adopted to specify the intensity of design earthquakes of level 1 and level 2. The inputting ground motions for dynamic design incorporate artificial wave of Art-Hachi and observed waves of El-Centro (NS), Taft (EW) and Hachinohe (NS). The seismic demands in dynamic design are shown as follows:

(i). Level 1, PGV=0.25m/s (the earthquake associated with 10% likelihood of being exceeded in 50 years)

Allowable stress design: $\sigma_b \leq f_b$;

Performance demand: $\theta_{max} \leq 1/200$.

(ii). Level 2, PGV=0.50m/s (the earthquake associated with 2% likelihood of being exceeded in 50 years)

Performance demand: $\theta_{max} \leq 1/100$

Plastic deformation ratio μ_b of girder should be less than 4: $\mu_b \leq 4$

The details of the seismic design of each high-rise steel building model are introduced in Appendix B.

4.4.2 Structural-scale models

(a) Plan and elevation view

According to the statistical assessment on the structural system, member, and material in existent high-rise buildings, various existent heights (number of stories) [i.e. 20, 30 and 40 story levels (denoted as 20S, 30S and 40S)], width-to-thickness ratio ranks of columns and beams (denoted as FA_c - FA_b , FA_c - FB_b and FB_c - FB_b , where FA_c and FB_c denote the ranks of FA and FB for columns, and FA_b and FB_b denote the ranks of FA and FB girders, respectively) with planar moment-frame models are seismic designed by a series of methods mentioned in Sec. 4.4.1. The elevation and plan view of the planar models are shown in Figure 4.6.

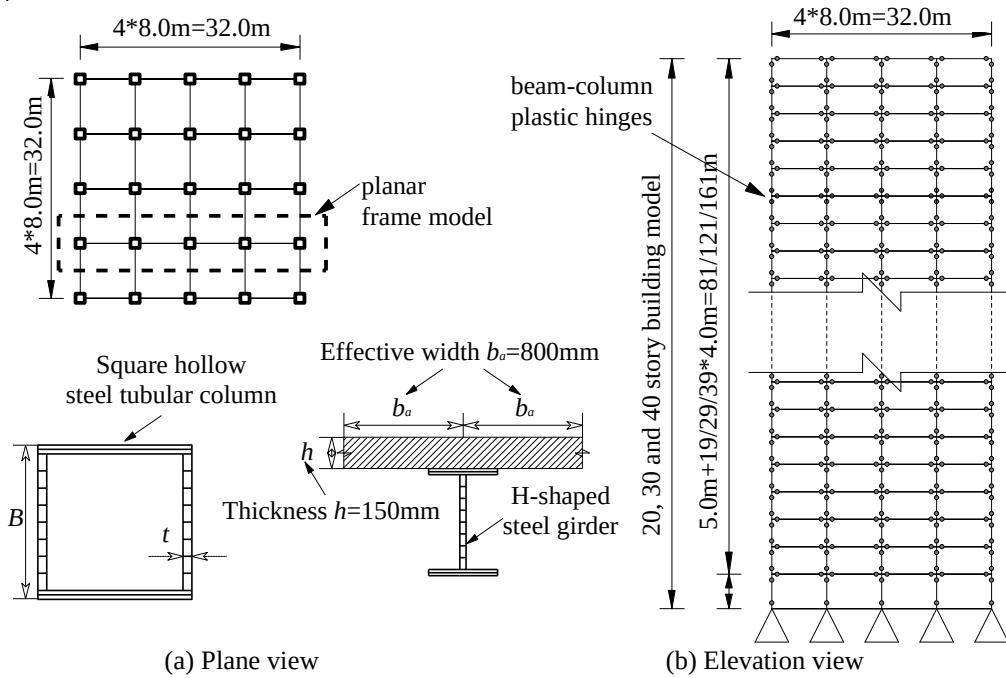


Figure 4.6 Various high-rise steel moment frame models

(b) Natural periods

Eigen value analyses for each building model are conducted to find the natural period of the frame model, and the first-three modes of each models are shown in Table 4.4. The natural periods of the first three modes are roughly close, which indicate for building models to have

almost the same horizontal stiffness for each height. Hence, the building models with the same story level are mainly treated to have analogous seismic criteria.

Table 4.4 Details of high-rise steel building models

Model	Height (Story)	Compact ranks	R_t	C_B	1 st mode (s)	2 nd mode (s)	3 rd mode (s)
20S-FA _c -FA _b	81m (20)	FA column FA girder	0.40	0.12	2.36	0.81	0.47
20S-FA _c -FB _b		FA column FA girder			2.38	0.81	0.47
20S-FB _c -FB _b		FA column FA girder			2.32	0.78	0.45
30S-FA _c -FA _b	121m (30)	FA column FA girder	0.26	0.08	3.61	1.21	0.69
30S-FA _c -FB _b		FA column FA girder			3.57	1.20	0.68
30S-FB _c -FB _b		FA column FA girder			3.54	1.17	0.66
40S-FA _c -FA _b	161m (40)	FA column FA girder	0.20	0.06	4.67	1.48	0.81
40S-FA _c -FB _b		FA column FA girder			4.53	1.42	0.78
40S-FB _c -FB _b		FA column FA girder			4.53	1.36	0.73

4.4.3 Deterioration behavior

The ‘FA’ and ‘FB’ compactness ranks of steel members (MLIT of Japan 2007) are respectively utilized in low-deteriorating building models (i.e. 20S-FA_c-FA_b, 30S-FA_c-FA_b, 40S-FA_c-FA_b) that use FA rank girder and column for seismic design, median-deteriorating building model (i.e. 20S-FA_c-FB_b, 30S-FA_c-FB_b, 40S-FA_c-FB_b) which use FA rank column and FB rank girder for seismic design and high-deteriorating building model (i.e. 20S-FB_c-FB_b, 30S-FB_c-FB_b, 40S-FB_c-FB_b) which use FB rank girder and column for seismic design.

Figure 4.7 shows the stress-strain models of ‘FA’ and ‘FB’ girders and column members adopted in the associated high-rise steel building model. It should be noted that the local-buckling of FB girder with width-to-thickness ratio of $D/t_w=52.6$ and $B/t_f=16.0$ happens much earlier than that of FA girder with width-to-thickness ratio of $D/t_w=42.1$ and $B/t_f=9.7$ [see Figure 4.7 (a)]. Post peak strength deterioration after the local buckling of FB girder proceeds within a small margin which leads the negative slope to be very steep. The average constitutive models of FA column and FB column are also compared with each other [see Figure 4.7 (b)], where the deterioration margin of FA column clearly shows larger than that of FB column. In addition, column members demonstrate higher capacity of deterioration prevention than that of the corresponding girder members.

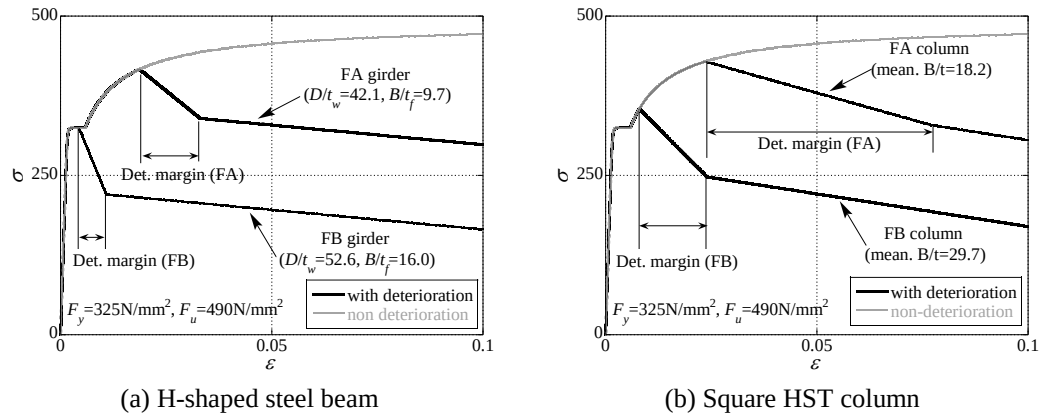


Figure 4.7 Deteriorating levels in the stress-strain model of member in 40S building models

4.5 Seismic Responses under Long-Period Ground Motions

Synthetic long period ground motions induced by ocean-ridge earthquakes is featured by long-duration, large seismic excitation on long-period branch of spectrum. Therefore, the seismic responses of existent high-rise steel buildings subjected to synthetic long-period ground motions (C-San-EW, YKL-NS, WOS-EW) generated in basin areas by subduction zone earthquakes is urgent to be assessed (AIJ, 2007). Moreover, extreme ground motions were also captured in Great East Japan Earthquake, 2011 (AIJ, 2011). Hence, we also evaluated the seismic responses of existent high-rise steel buildings under observed ground motions (MYG004-NS, CHB009-NS) in Great East Japan Earthquake, in order to assess the earthquake resistant capacity and deterioration behavior of various building models incorporating deterioration or not. Various seismic responses like the maximum story drift angle, residual story drift angle, plastic deformation ratio and cumulative plastic deformation ratio (based on hysteresis energy) are calculated in this section.

4.5.1 Characteristics of ground motion

The seismic responses of various building models subjected to synthetic long-period waves and the motions captured in Tohoku earthquake are analysed. The representative values of inputting ground motions are presented in Table 4.5.

Table 4.5 Details of Input ground motions

	Ground motion	Location	PGA (gal)	PGV (kine)	Time (sec)
Synthetic severe long period waves	C-San-EW	Nagoya	185.9	49.4 (Level 2)	329.3
	YKL-NS	Yokohama	499.2	70.0 (over Level 2)	210.0
	WOS-EW	West-Osaka	69.3	24.8 (Level 1)	301.4
Observed waves in Tohoku earthquake	MYG004-NS	Sendai	2670	153.3 (over Level 2)	300.0
	CHB009-NS	Chiba	178.6	47.9 (Level 2)	300.0

The typical time-history acceleration and the pseudo velocity spectrum of these waves are shown in Figures 4.8~4.9. C-San-EW is the synthetic long-period ground motion induced by subduction zone earthquake near Nagoya area. WOS-EW is the synthetic long-period ground motion induced near western Osaka area. YKL-NS is the synthetic long-period ground motion induced near Tokyo area. MYG004-NS is the largest ground motion captured in Great Eastern Japan earthquake 2011. CHB009-NS is the long period ground motion captured at Kanto area. In addition, larger seismic velocity excitations of long-period ground motions are observed as shown in Figure 4.9.

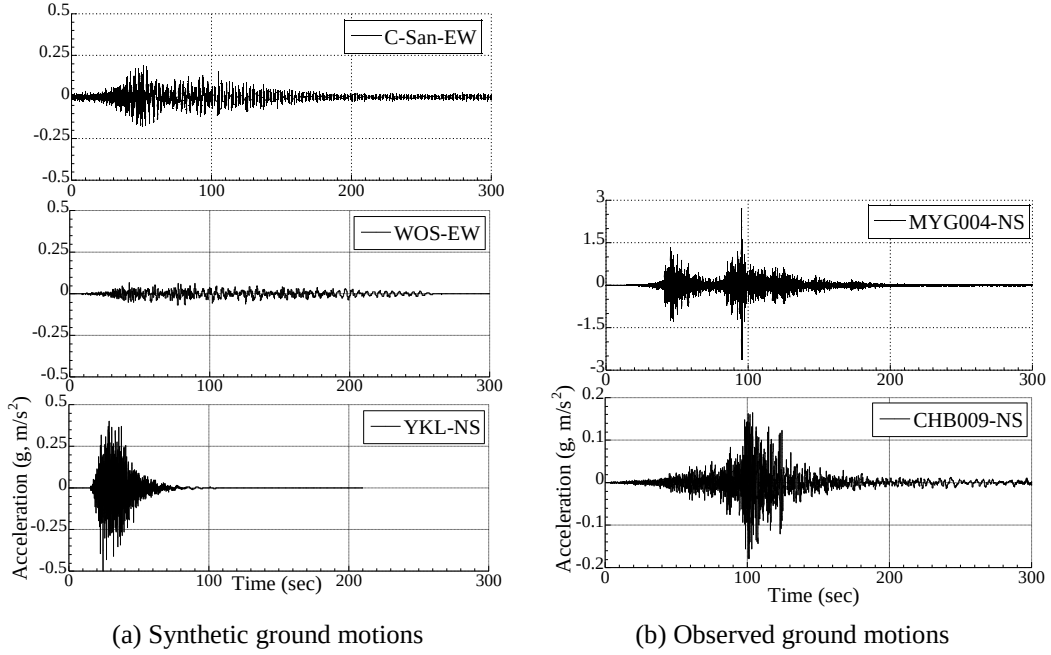


Figure 4.8 Time-history acceleration long-period ground motions

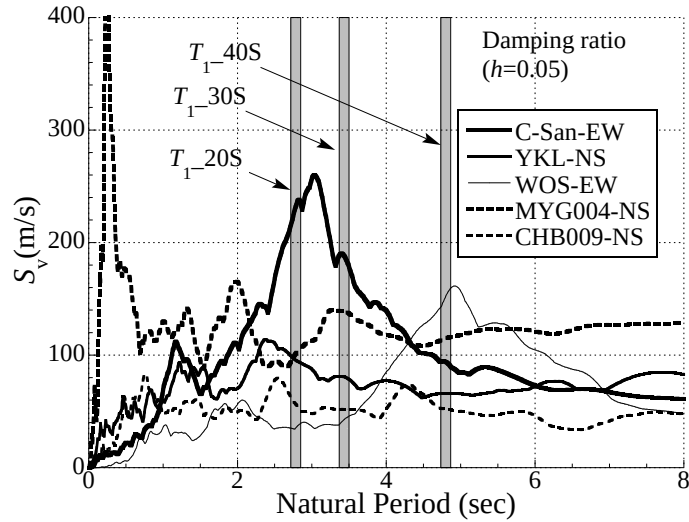


Figure 4.9 Pseudo velocity response spectra of various long-period ground motions

4.5.2 Response of maximum story drift angle

Firstly, we conduct time-history dynamic analysis on various high-rise steel building models under severe earthquakes that incorporating the synthetic long-period ground motions (C-San-EW, YKL-NS, WOS-EW) and the observed severe ground motions (MYG004-NS, CHB009-NS) in Great East Japan earthquake, 2011. Maximum story drift angle response of the i -story of moment-frame structures are calculated by

$$\theta_{max,i} = \frac{\delta_{max,i} - \delta_{max,i-1}}{h_i} \quad (4.5)$$

Where, $\delta_{max,i}$ and $\delta_{max,i-1}$ are respectively instantaneous maximum horizontal drifts of i and $i-1$ story, h_i is the story-height of i story. Thus, according to Eq. (4.5), the responses of maximum story drift angle of various high-rise steel buildings is computed as shown in Figures 4.10~4.18. Figures

4.10~4.18 (a) demonstrate the maximum story drift angles of building models under the earthquakes (Art-Hachi, El-Centro-NS, Hachinohe-NS, Taft-EW and WOS-EW) scaled to Level 1 (PGV=0.25m/s). Figures 4.10~4.18 (b) show the maximum story drift angles of building models under the earthquakes (Art-Hachi, El-Centro-NS, Hachinohe-NS, Taft-EW, CHB009-NS and C-San-EW) scaled to Level 2 (PGV=0.5m/s). Figures 4.10~4.18 (c) demonstrated the maximum story drift angles of building models under the earthquakes (MYG004-NS and YKL-NS) over Level 2 (PGV=0.5m/s).

According the maximum story drift angle responses of various high-rise building models demonstrated in Figures 4.10~4.18, several relations between seismic resistant capacity and structural performance can be founded below.

(a) We could briefly find out that seismic demands of level 1 and level 2 ($\theta_{max} \leq 0.005$ for level 1 demand, $\theta_{max} \leq 0.01$ for level 2 demand) could not be satisfied by when subjected to synthetic long-period ground motions, e.g., C-San-EW (near PGV of Level 2) excited relatively large θ_{max} on 20S and 30S building models ($\theta_{max} > 0.01\text{rad}$). In contrast, WOS-EW (near PGV of Level 1) tends to have larger excitations on 30S and 40S building models ($\theta_{max} > 0.005\text{rad}$). These seismic response observations indicate that the existent high-rise steel buildings (height: 81m~160m) might have large risks to satisfy seismic demands under severe earthquakes with long period component.

(b) As for the relation between story level (20, 30, 40) and maximum story drift angle response, large variability is captured, because the characteristics of each ground motions has different influence on the seismic capacity of the correspondingly different models that have various natural vibration features. This finding further proves that long-period ground motions tend to have larger uncertainty on the seismic response of high-rise buildings. Furthermore, long-period ground motions induced in varied basin-areas may have associated different seismic excitations, e.g., C-San-EW wave shows extremely large excitation on the structure with ‘first-mode’ natural period (T_1) between 2~4sec, and WOS-EW likewise influences the structure with T_1 between 4.5~5.5sec (AIJ 2007).

(c) By comparing the maximum story drift angle response of each building model subjected to observed long-period ground motion (CHB009-NS) and largest observed ground motion in great east Japan earthquake (2011). As shown in Figures 4.10~4.18 (c), with respect to the earthquake over Level 2, period of vibration is short. In this case, θ_{max} responses are relatively small even with long period component.

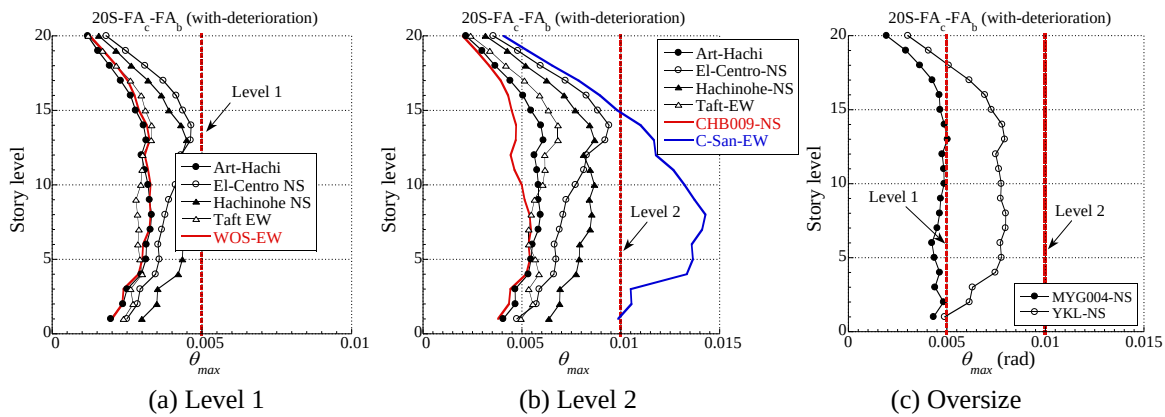
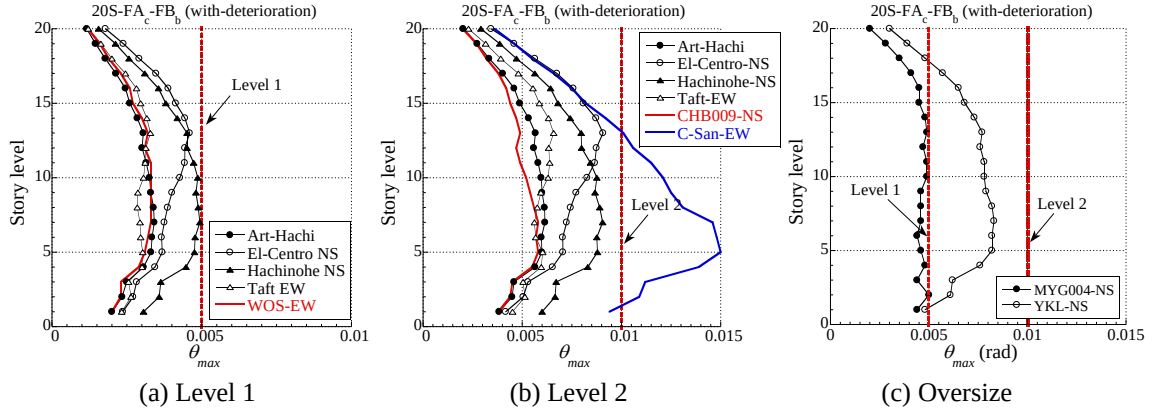
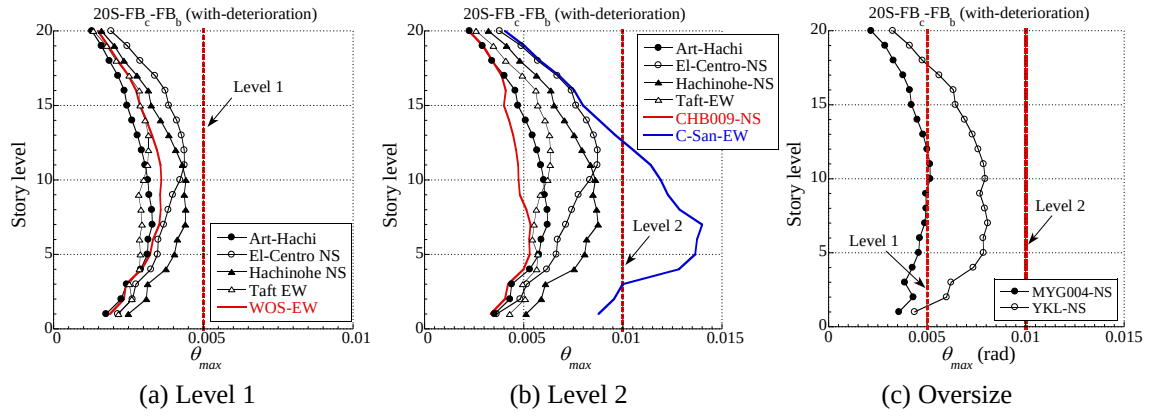
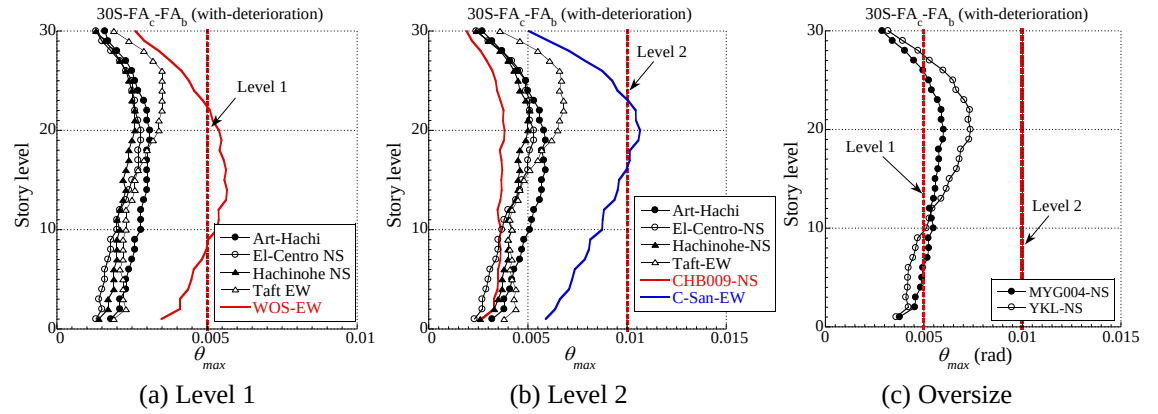
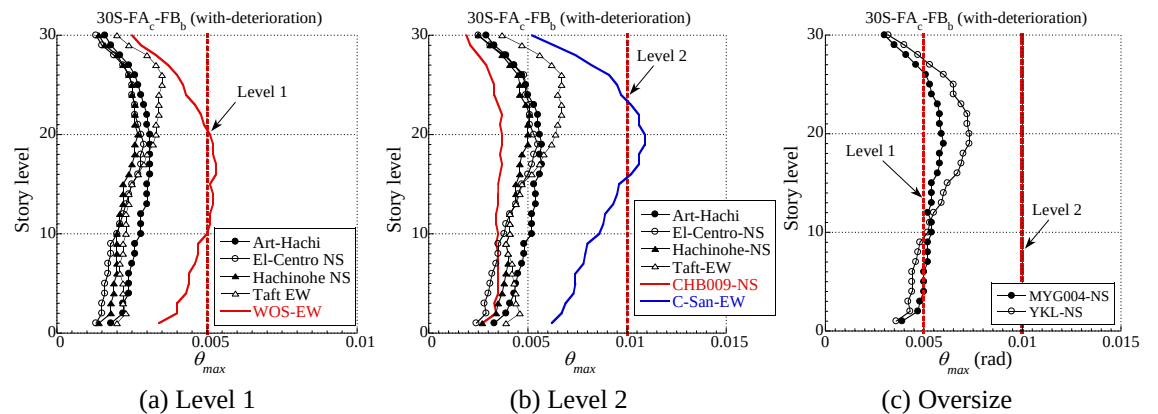
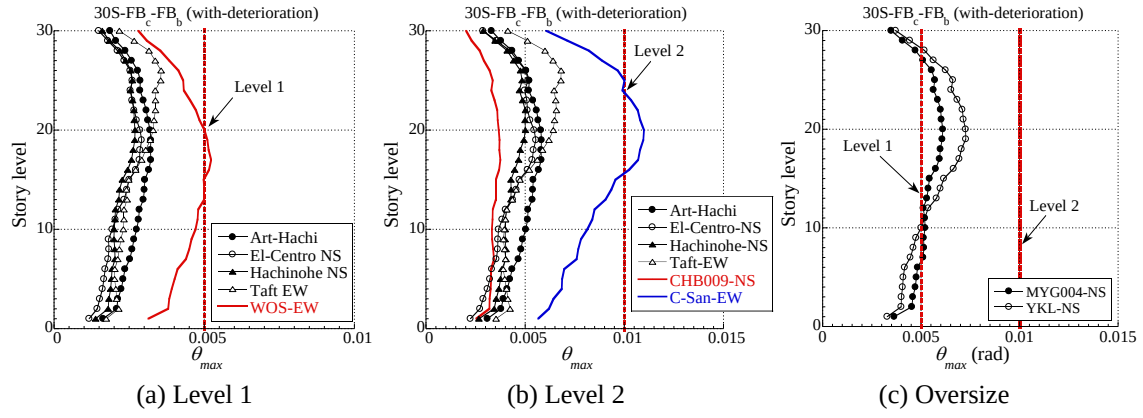
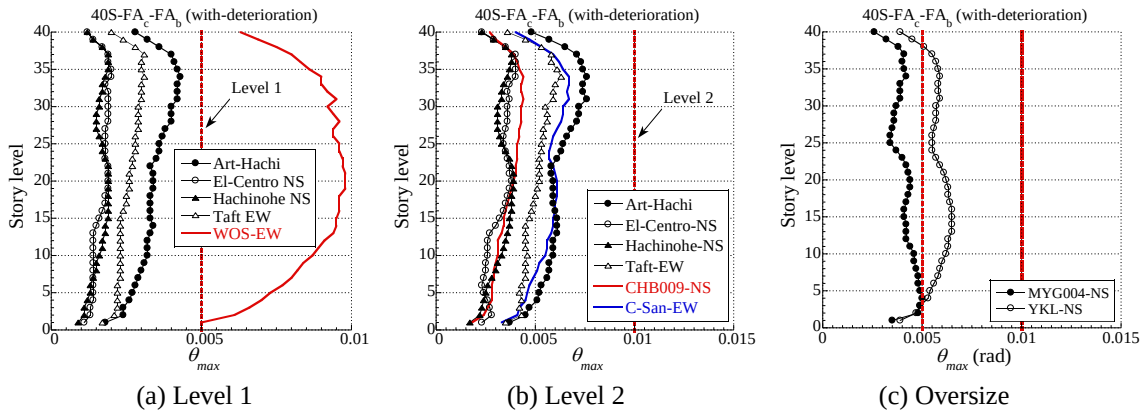
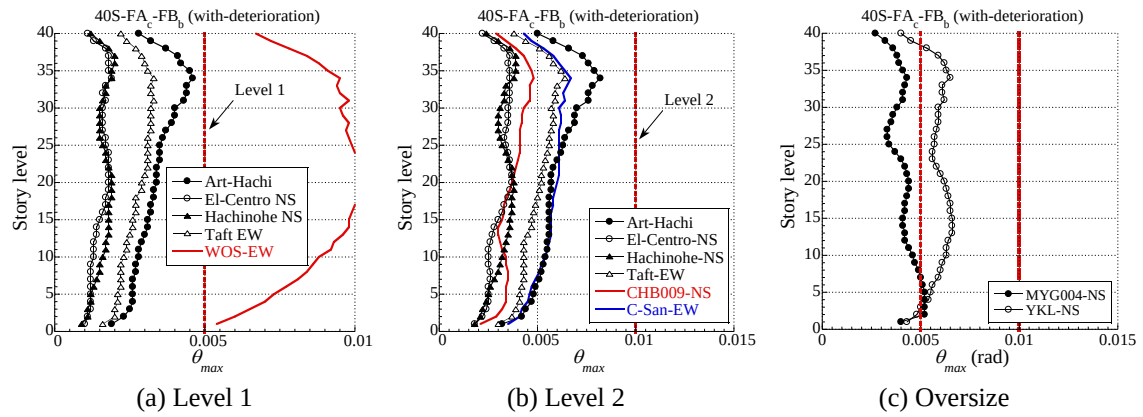
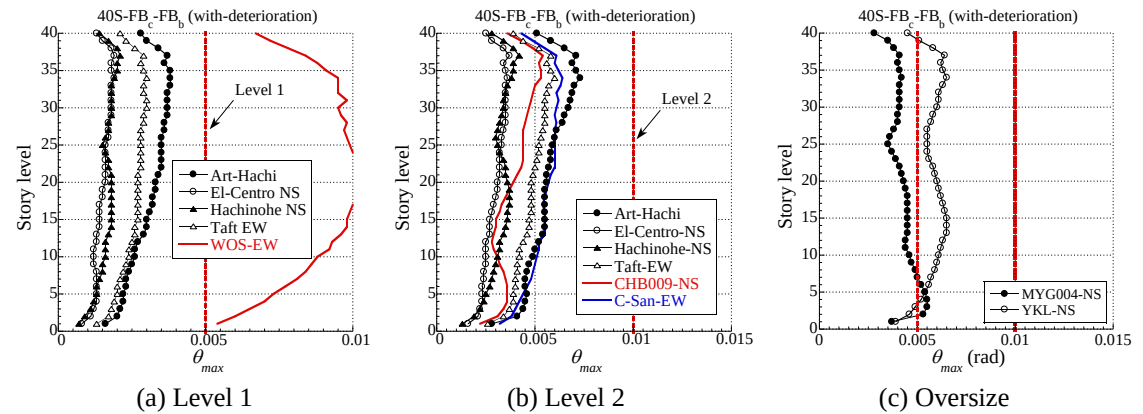


Figure 4.10 20S-FA_c-FA_b model


 Figure 4.11 20S-FA_c-FB_b model

 Figure 4.12 20S-FB_c-FB_b model

 Figure 4.13 30S-FA_c-FA_b model

 Figure 4.14 30S-FA_c-FB_b model


 Figure 4.15 30S-FB_c-FB_b model

 Figure 4.16 40S-FA_c-FA_b model

 Figure 4.17 40S-FA_c-FB_b model

 Figure 4.18 40S-FB_c-FB_b model

In addition, the effect of member deteriorations (FA, FB for girder and column) on the maximum story drift angle responses of various building models (20S, 30S and 40S) is not overtly observed, by comparing the maximum story drift angle of the building model with deterioration and that without deterioration. However, certain slight effect of deterioration on maximum story drift angle is produced, which means that local-buckling of steel members is initiated by strength and stiffness deterioration. This phenomenon becomes more and more notable from the toughest building model FA_c-FA_b to intermediate building model FA_c-FB_b and finally to weakest building model FB_c-FB_b. For instance, mild effect of member deterioration is derived in 40S-FA_c-FA_b model under WOS-EW and MYG004-NS, as shown in Figure 4.19. This amplified effect on θ_{max} may be increased by seismic excitations with larger intensity.

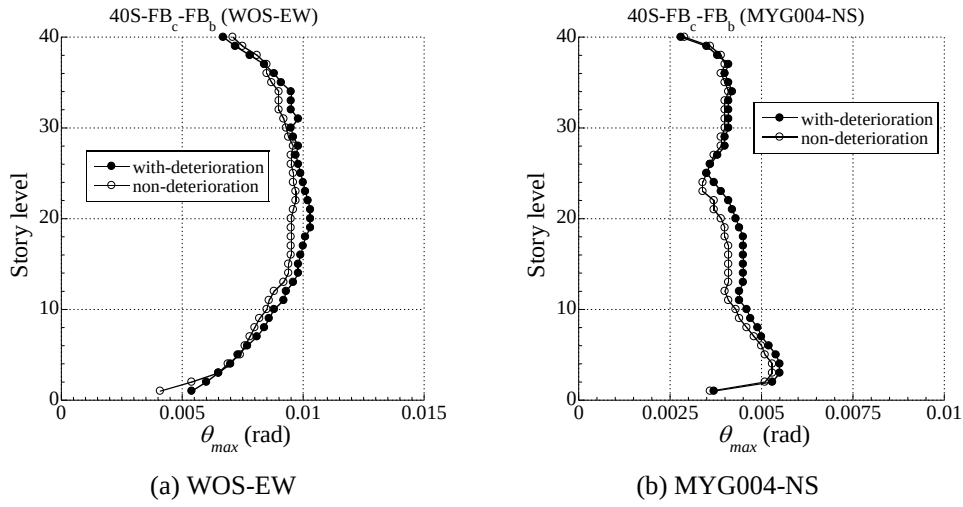


Figure 4.19 Effect of member deterioration on the θ_{max} of 40-story model

Seismic responses of the FA_c-FA_b building models with various heights under C-San-EW, WOS-EW and YKL-NS are demonstrated in Figure 4.20, based on the normalised height in vertical direction. The dependence of the relation between long-period ground motion characteristics and the natural period of high-rise building structures on the seismic responses is clarified since each long-period ground motion has its special effective range. Hence, for the existent high-rise steel buildings, seismic assessment should be specifically conducted by considering the characteristics of long-period ground motions.

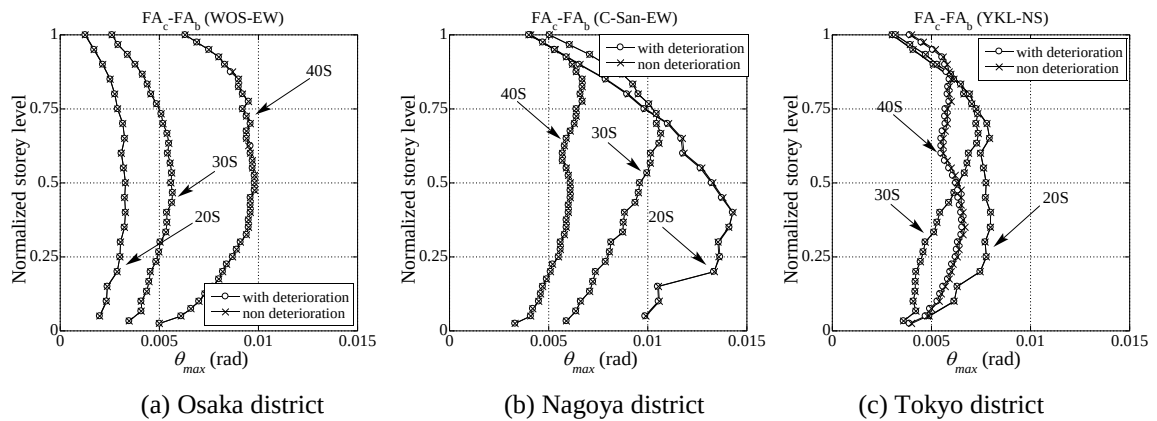


Figure 4.20 Effect of the characteristics of ground motions on θ_{max} of 20S, 30S and 40S building models

4.5.3 Response of residual story drift angles

Permanent deformation of the structural member or system remained after earthquake occurring, i.e., the residual story drift angles of building structures are considered as an important indicator to assess the functionability and servability of existent high-rise steel buildings suffered from earthquake. Many studies considering the residual drift of building structures have been conducted by Kawashima et al. 1998, Erochko and Christopoulos et al. 2011. Besides, residual story drift angle has been treated as a seismic performance indicator by several researchers, e.g., Mahin and Bertero (1981) have considered maximum and permanent drifts in the inelastic response of single degree-of freedom systems. MacRae and Kawashima (1997) have addressed the post-earthquake residual displacement based on bi-linear oscillators. Christopoulos et al. (2003) examined the relationship between residual displacement and peak displacement by considering the positive-yield hardening of structural components. Ruiz-Garcia and Miranda (2005) assessed the seismic performance of existing structures by accounting for residual displacements. It should be noted that the residual deformations likely to increase when the maximum deformation incorporating inelasticity develops.

Along with the loading cycles increase, the strain of various steel fiber elements would achieve a certain level that is probably larger than the yield strain (ε_y) and local-buckling strain (ε_{lb}) of steel fibers. Thus, considering the member deterioration would become important to predict how much the residual drift remains after earthquake ended. To be similar to the calculation of θ_{max} , the residual story drift angle (θ_{res}) of the high-rise steel building models can be calculated by

$$\theta_{res,i} = \frac{\delta_{res,i} - \delta_{res,i-1}}{h_i} \quad (4.6)$$

Where, $\delta_{res,i}$ and $\delta_{res,i-1}$ are respectively instantaneous residual horizontal drifts of i and $i-1$ story, h_i is the story-height of i story. According to the discussions in section 4.5.2, C-San-EW (nearly equal to Level 2 seismic criterion) and WOS-EW (nearly equal to Level 1 seismic criterion) clearly influences on the maximum story drift angle of high-rise buildings because of long-period characterized spectral shape.

Differentials of the residual story drift angle between various building models with and without deterioration effect are respectively calculated and compared with each other. The building model without member deterioration effect includes P- Δ effect to only evaluate the P- Δ effect on the response of residual story drift angles. Hence, we respectively evaluate the residual story drift angles of various high-rise building models under C-San-EW and WOS-EW waves, as shown in Figures 4.21~4.22.

(a) As mainly observed in Figures 4.21~4.22, not being similar to the maximum story drift angle, residual story drift angle tends to maintain in the buildings with larger story level and long-period spectral shape shows little influence.

(b) Residual story drift angle existed in 20S-FA_c-FB_b building model under C-San-EW is rarely extensively induced compared with that of 30S-FA_c-FB_b and 40S-FA_c-FB_b building models. Nevertheless, C-San-EW excited larger maximum story drift angle on 20S-FA_c-FB_b building model than those of the corresponding 30S-FA_c-FB_b and 40S-FA_c-FB_b building models.

(c) With respect to the effect of member deterioration, it is noted that larger residual story drift angle does not indicate the correspondingly larger or more extensive member deteriorations have happened [see Figure 4.21 (a), Figure 4.21 (c)]. Low extent of residual story drift angle as shown in Figure 4.22 (a) seems have more extensive influence potential of member deterioration.

(d) As for the comparisons as shown in Figure 22, analogous phenomenon to those revealed previously has been found as well. In addition, residual story drift angle tends to maintain more in higher building model (40S) than that in lower building model (20S, 30S).

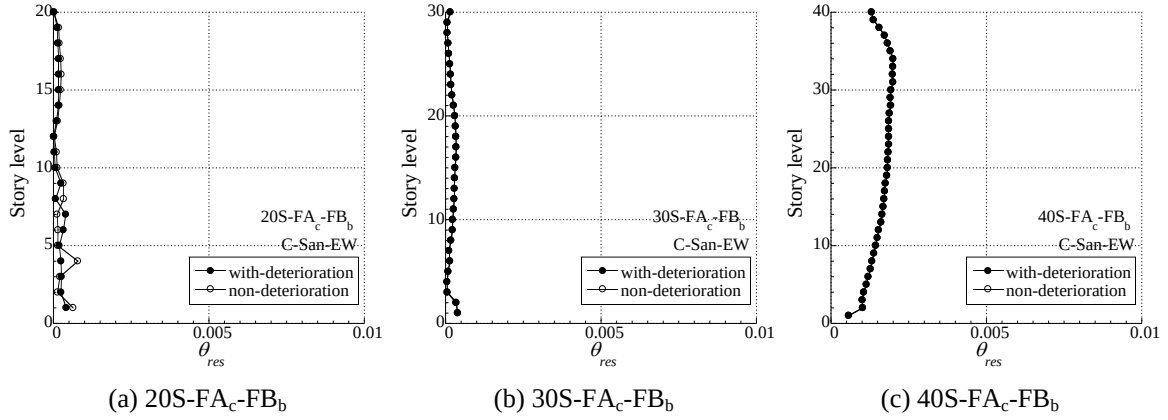


Figure 4.21 Residual story drift angle of building models under C-San-EW

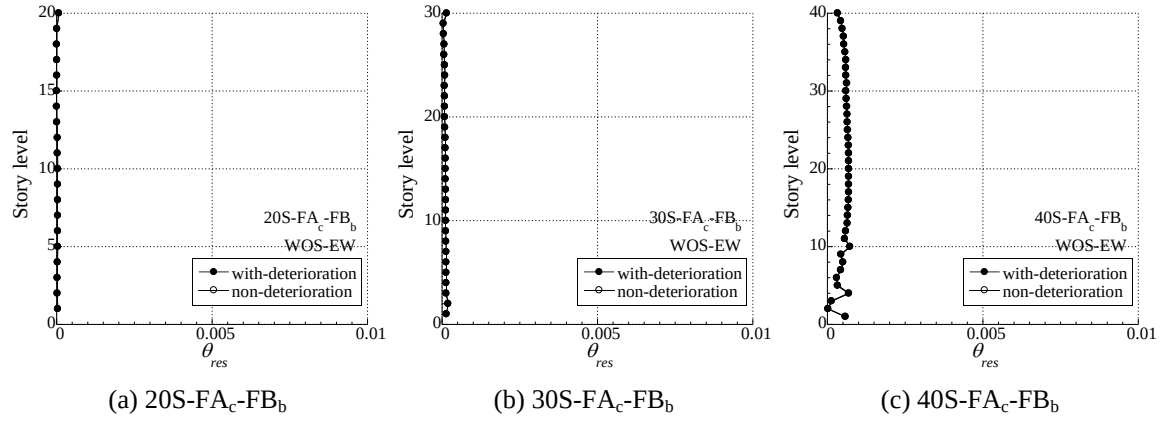


Figure 4.22 Residual story drift angle of building models under WOS-EW

4.5.4 Plastic deformation ratio

(1) Definition of plastic deformation ratio

Beside the seismic responses at global-level (maximum story drift angle and residual story drift angle) of high-rise steel building structure, the plastic-hinge rotation at the ends of beam and column members are also initiated. Plastic-hinge rotation is an index for evaluating the plastic-deformation capacity of local regions (Wakabayashi, 1986). Local buckling and damage of steel members can be examined by normalized rotation of steel member, i.e., plastic deformation ratio.

$$\begin{cases} \mu_b = \frac{\theta_{\max}^b}{\theta_y^b} \\ \mu_c = \frac{\theta_{\max}^c}{\theta_y^c} \end{cases} \quad (4.7)$$

Where, the yield rotation of beam and column is calculated by

$$\theta_y = M_y \cdot \frac{l}{6EI} \quad (4.8)$$

Where, the yield bending moment of beam and column is calculated by

$$M_y = Z\sigma_y \quad (4.9)$$

Besides, θ_{max}^b and θ_{max}^c are the maximum responses of the deflection angle of structural element at the each end of steel beam and column (element length is equal to cross-section depth D); θ_y^c and θ_y^b is the rotation at yield point of column and beam, respectively; l is length of member; E is Young's modulus; Z is the section modulus; σ_y is yield stress of steel. Hence, μ_b and μ_c is nearly equal to plastic deformation ratio because small level of elastic rotation is included.

(2) Responses of plastic deformation ratio

As an example, the girders' plastic deformation ratio μ_b of 20, 30 and 40 story building models with the correspondingly various deterioration levels subjected to C-San-EW and WOS-EW waves which indicate the largest responses among the synthetic earthquake here, are compared with each other to explore the effect of long-period seismic excitation on various heights of high-rise steel buildings as shown in Figure 4.23~Figure 4.28. The Level 2 seismic criteria that require the μ_b should be under 4, to indicate the extent of structural damage. It is noted that the dependence of the beam responses on story level (20S, 30S and 40S) and member deterioration has been variously demonstrated.

(a) As shown in Figure 4.23, the largest plastic deformation ratios at the beam end are observed in 20S building model under C-San-EW: the μ_b of various deteriorating models of 20S-FA_c-FA_b, 20S-FA_c-FB_b and 20S-FB_c-FB_b are respectively 4.01, 3.76 and 3.68. 30S building model comes second (see Figure 4.24): μ_b of various deteriorating models of 30S-FA_c-FA_b, 30S-FA_c-FB_b and 30S-FB_c-FB_b are respectively 1.93, 2.11 and 2.13. 40S building models come last (see Figure 4.25): μ_b of various deteriorating models of 40S-FA_c-FA_b, 40S-FA_c-FB_b and 40S-FB_c-FB_b are respectively 1.15, 1.33 and 1.34.

(b) By comparing the plastic deformation ratio at the beam end of various models between ductile FA beam and non-ductile FB beam (e.g., between 20S-FA_c-FA_b and 20S-FA_c-FB_b, or 20S-FA_c-FB_b and 20S-FB_c-FB_b), member deterioration of the FA beam in 20SFA_c-FA_b model has not been visibly induced [see Figure 4.23(a)], while the member deterioration of the FB beam in 20SFA_c-FB_b model obviously amplified the plastic deformation ratio at the beam end of lower stories, as shown in Figure 4.23(b).

(c) However, the FA column in 20S-FA_c-FB_b model has not efficiently strengthened the plastic hinge rotation of FB beam compared with the FB column in 20S-FB_c-FB_b building model, as shown in Figures 4.23(b) and Figure 4.23(c). This because the plastic hinge of FA column is stronger than FB column. Hence, the weaker FB column leads no smaller plastic deformation ratio of FB beam [Figures 4.23~4.24(b) and Figure 4.23~4.24(c)]. As for the 40S building models (see Figure 4.25), small plastic deformation ratio is induced and deterioration effect has not been obtained neither.

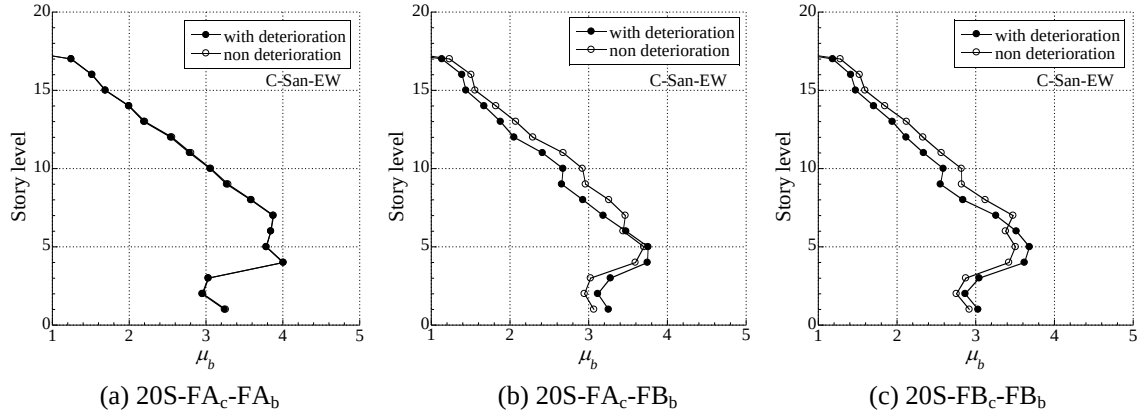


Figure 4.23 Plastic deformation ratio at the beam end μ_b of 20S building models under C-San-EW

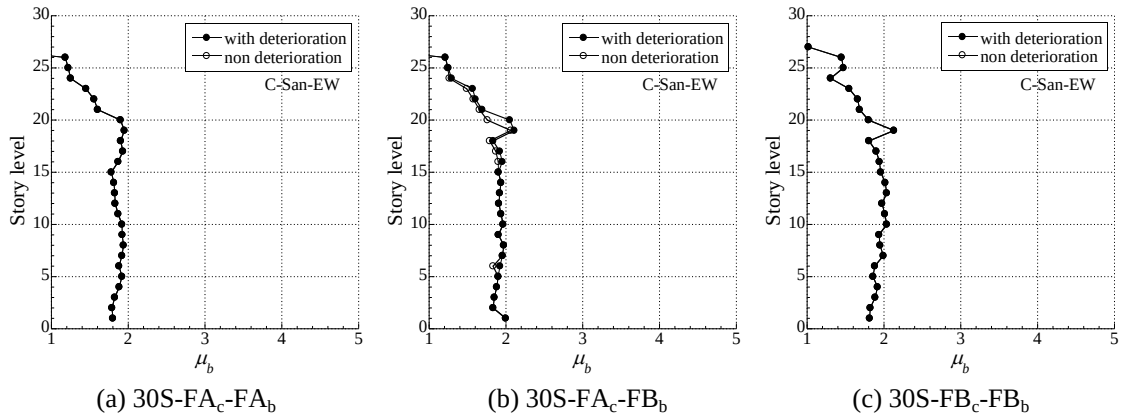


Figure 4.24 Plastic deformation ratio at the beam end μ_b of 30S building models under C-San-EW

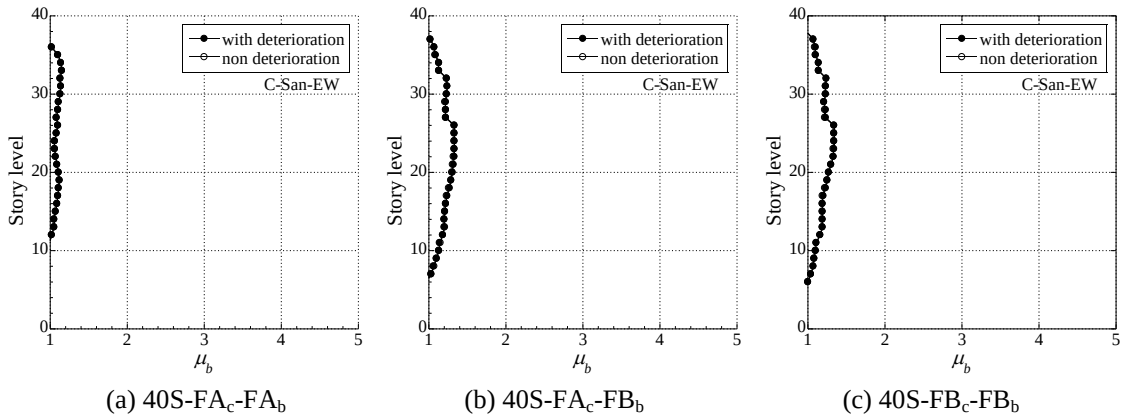


Figure 4.25 Plastic deformation ratio at the beam end μ_b of 40S building models under C-San-EW

As shown in Figures 4.26~4.28, the plastic deformation ratio of various building models subjected to long period ground motion (WOS-EW) near Level 1 shows smaller than those under Level 2 ground motion (C-San-EW). However, the μ_b responses of 40S building models under WOS-EW (Figure 4.28) show larger than 1.0 which is the Level 1 criteria. This indicates that current seismic criteria could not be satisfied for existent 40S high-rise buildings.

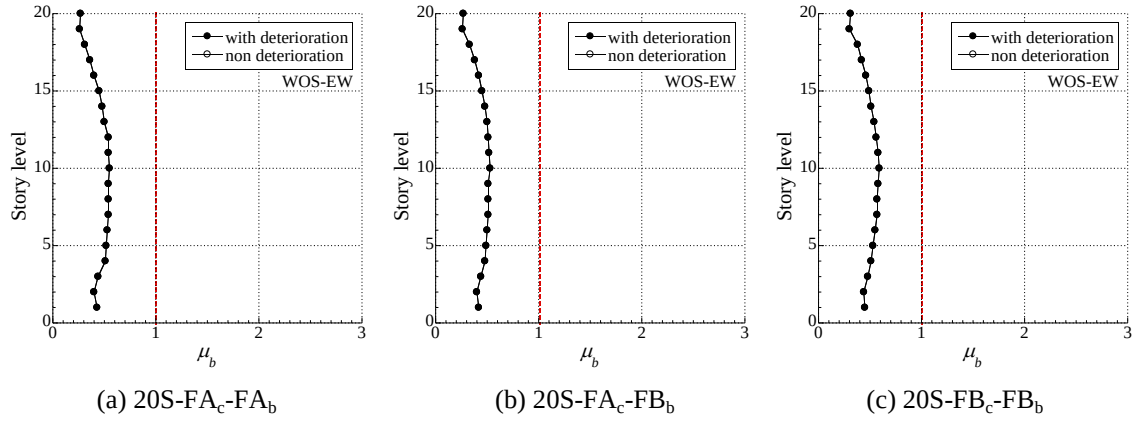


Figure 4.26 Plastic deformation ratio at the beam end μ_b of various 20S building models under WOS-EW

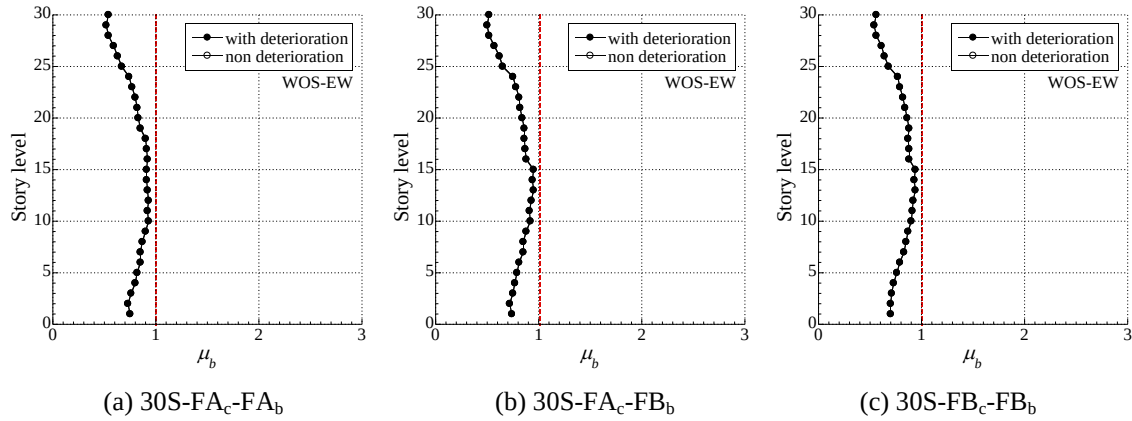


Figure 4.27 Plastic deformation ratio at the beam end μ_b of various 30S building models under WOS-EW

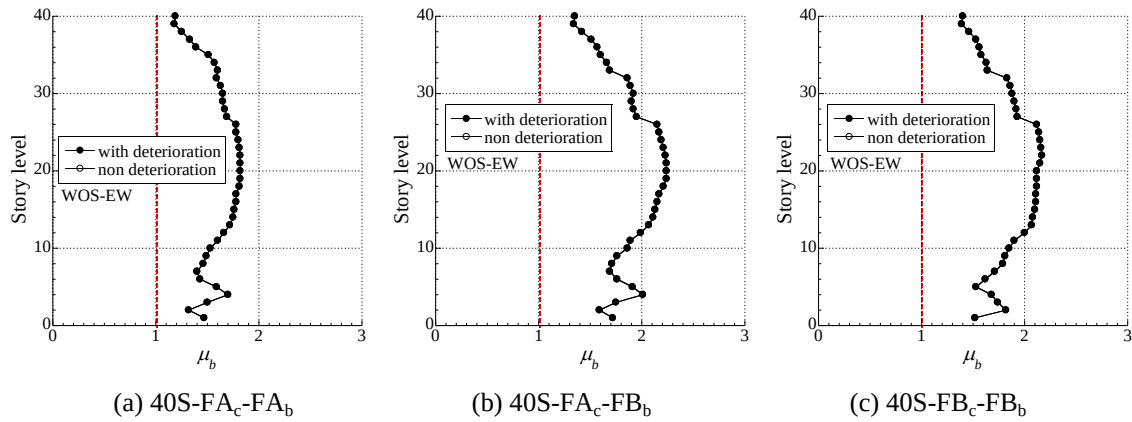


Figure 4.28 Plastic deformation ratio at the beam end μ_b of various 40S building models under WOS-EW

In addition, comparing with the maximum story drift angle responses of various height building models under WOS-EW and C-San-EW (in Section 4.5.2), the plastic deformation ratio at the beam end of the corresponding building models are also compared with each other as shown in Figure 4.29. It should be noted that various long period ground motions generate correlative responses of the plastic deformation ratio at the beam ends of various high-rise building because of the characteristics of ground motions.

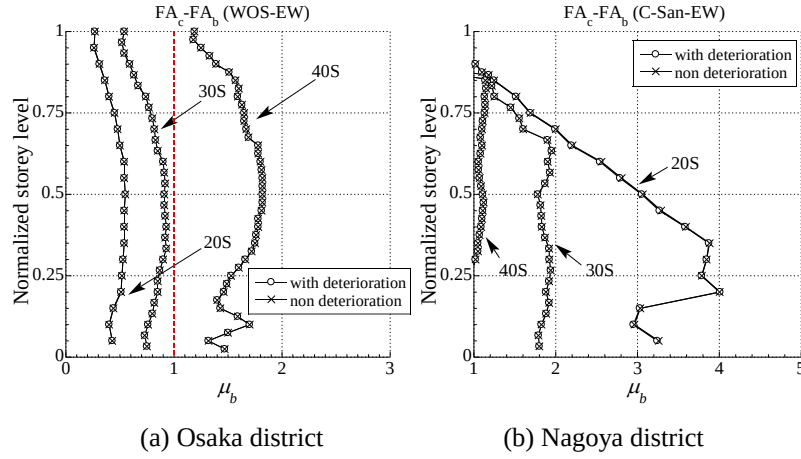


Figure 4.29 Effect of ground motion on μ_b of 20S, 30S and 40S building models

(3) Plastic mechanism

In order to particularly demonstrate the range and level of the plastic damages at column base and beam end, the distributions of the plastic hinges of various height building models (20S-FA_c-FB_b, 30S-FA_c-FB_b, and 40S-FA_c-FB_b) are illustrated in Figures 4.30~4.33, based on different levels of plastic deformation ratios ($1 < \mu < 2$, $2 < \mu < 4$, $4 < \mu < 7$, and $7 < \mu$). It is noted that

(a) The plastic hinges of the beam ends at exterior span are more extensively generated than that at interior span, which indicates that when the building structure is subjected to cyclic earthquake loadings, the beams at right and left exterior spans stand more bending moments than interior spans on positive and negative directions.

(b) With respect to the influence of member deterioration effect on the distributions of plastic hinges [see Figure 4.30(a) and Figure 4.31(a)], a story collapse mechanism is formed at sixth floor by deterioration effect. Only incorporating $P-\Delta$ effect [see Figure 4.30(a)] increases the height of plastic deformation ratio within the range of 2~4.

(c) Based on the comparison as shown in Figure 4.32 (a) and Figure 4.32 (b), member deterioration does not extend the range of plastic hinge, but aggravate the level of member damage based on μ , e.g., all of the beams at the fifth-story of deteriorating building model have been produced larger plastic deformation ratio (between 2~4 with circle mark) than that of non-deteriorating building model. Hence, the plastic collapse mechanism is not affected by deterioration effect.

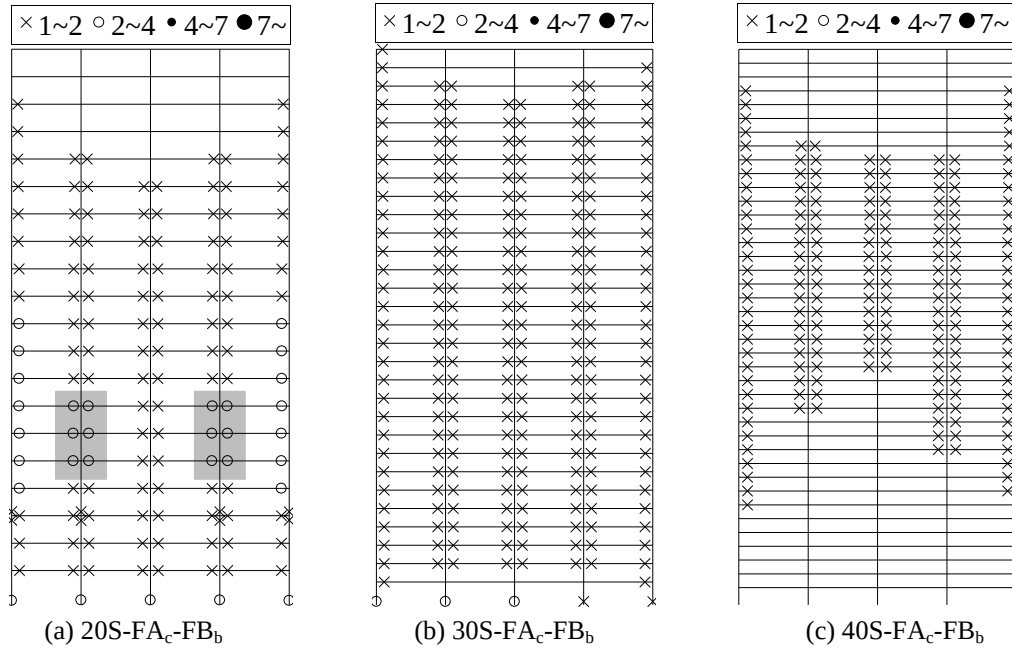


Figure 4.30 Distribution of the plastic hinges of various FA_c-FB_b models under C-San-EW (non-deterioration)

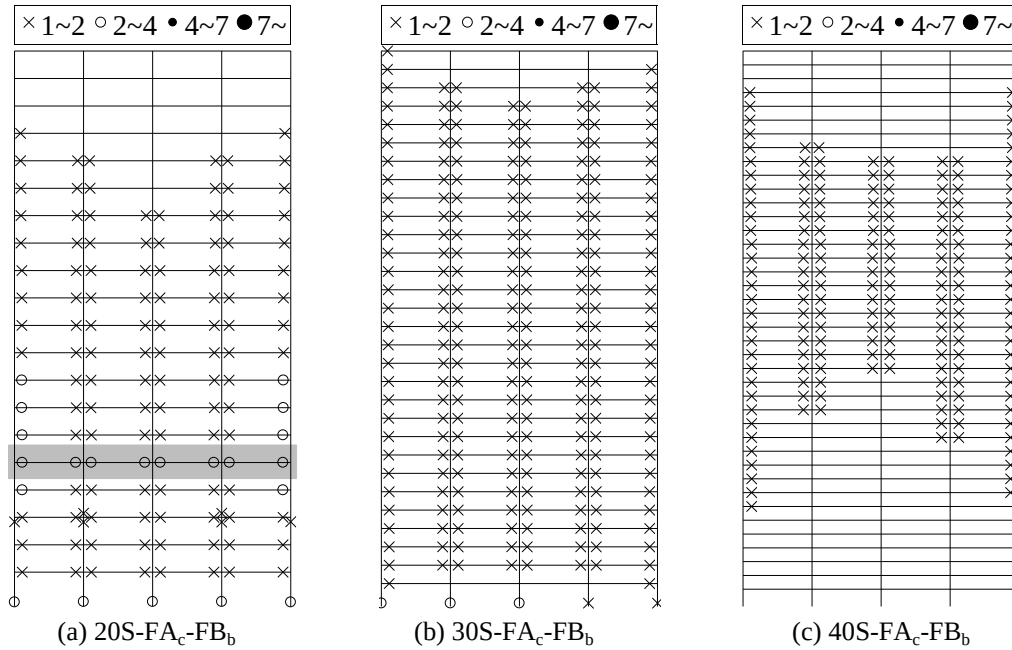


Figure 4.31 Distribution of the plastic hinges of various FA_c-FB_b models under C-San-EW (with deterioration)

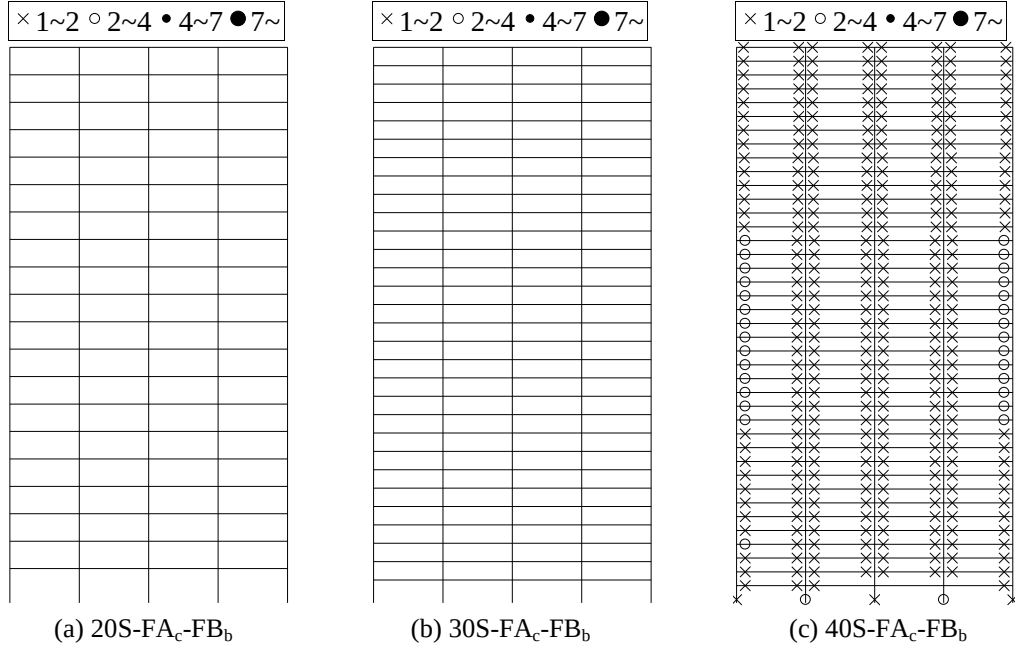


Figure 4.32 Distribution of the plastic hinges of various FA_c-FB_b models under WOS-EW (non-deterioration)

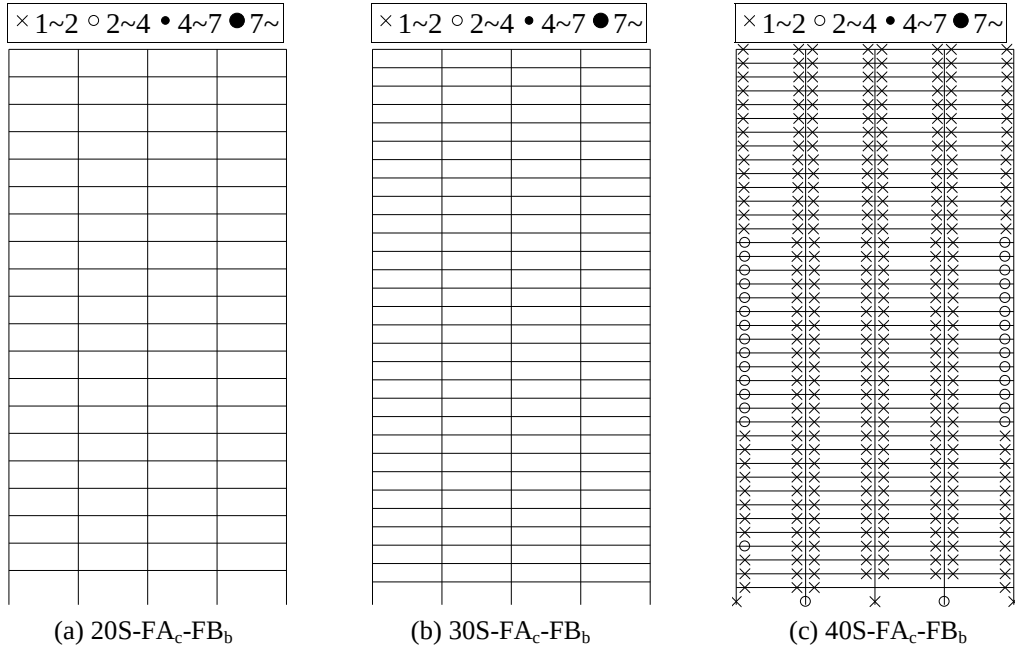


Figure 4.33 Distribution of the plastic hinges of various FA_c-FB_b models under WOS-EW (with deterioration)

4.5.5 Cumulative plastic deformation ratio

Kato and Akiyama (1975) have proposed a hypothesis in which the term cumulative plastic deflection is introduced. Hysteretic energy dissipation implies cumulative plastic energy which is dissipated in structures, in the form of uncovered damages in structural components that incorporate girder and column in moment frames. The cumulative hysteretic energy absorbed by beam and column members during earthquake divided by the elastic limit energy of the

corresponding members is defined as the member's cumulative plastic deformation ratio η_b for beams and η_c for columns, which are expressed by the following equations:

$$\begin{cases} \eta_b = \frac{W_h}{{}_gM_p \cdot {}_g\theta_p} \\ \eta_c = \frac{W_h}{{}_cM_p \cdot {}_c\theta_p} \end{cases} \quad (4.10)$$

Where, W_h is cumulative hysteretic energy dissipation at the ends of girder and/or column which is calculated by

$$W_h = \sum_i W_i \quad (4.11)$$

In addition, ${}_gM_p$ and ${}_cM_p$ are the full-plastic bending moment of beam and column respectively, ${}_g\theta_p$ and ${}_c\theta_p$ are the elastic rotation angles at the ends of girder and column corresponding to the full plastic moments, respectively. The calculation of the ultimate energy-dissipating capacity ($M_p \cdot \theta_p$) is demonstrated in Figure 4.33. The cumulative hysteretic energies of each cycles (W_i) is demonstrated Figure 4.34.

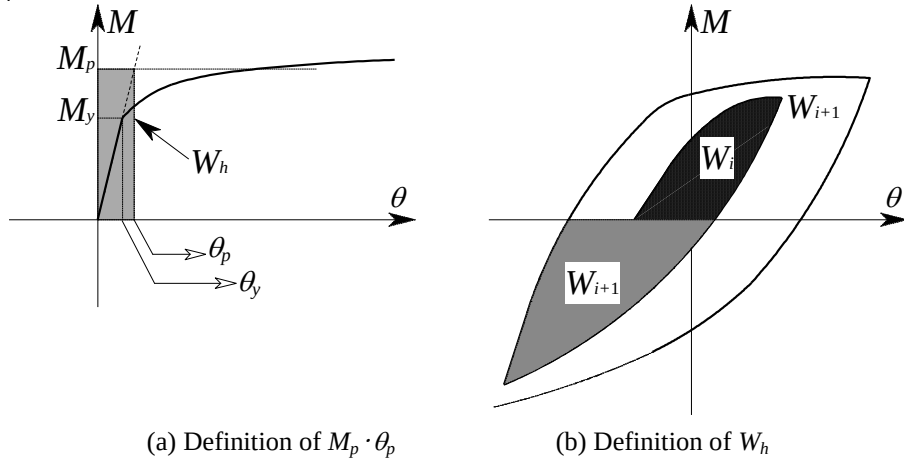


Figure 4.34 Definition of cumulative plastic deformation ratio

Figures 4.35~4.36 respectively illustrate the maximum responses of cumulative plastic deformation of the beam member of various high-rise steel building models (i.e., 20S-FB_c-FB_b, 30S-FB_c-FB_b, as well as 40S-FB_c-FB_b) under severe synthetic long-period ground motions. In which, it is observed that

(a) The η_b responses of each building model under such long-period ground motions is analogous to that of maximum story drift angle and μ_b responses, e.g., the largest η_b response 20S-FA_c-FB_b and 30S-FA_c-FB_b models is excited by C-San-EW, and the largest η_b response 40S-FB_c-FB_b models is excited by WOS-EW, which is the same as the maximum story drift angle and μ_b responses in above cases.

(b) However, there is one more phenomenon should be noted, which is the η_b goes much faster than that of maximum story drift angle or μ_b parameters, e.g., in case of 20S-FA_c-FB_b model, C-San-EW wave the η_b responses this model a lot larger than subjected to the other motions. This indicates that η_b is more significantly related to the characteristics of predominant period of such long-period ground motions than other parameters.

(c) In detail, for the 20S-FA_c-FB_b model under C-San-EW, both deteriorating and non-deteriorating μ_b and μ_c responses are respectively compared with each other as shown in Figure 4.35(b), in which, being different from that member deterioration (beam) tend to increase the η_b to the extent of lower stories (e.g., from bottom story to 7th story in 20S building model), the member deterioration (column) amplifies η_c mainly at bottom-story where the column-base may absorb substantial inelastic strain energy based on associating plastic rotations.

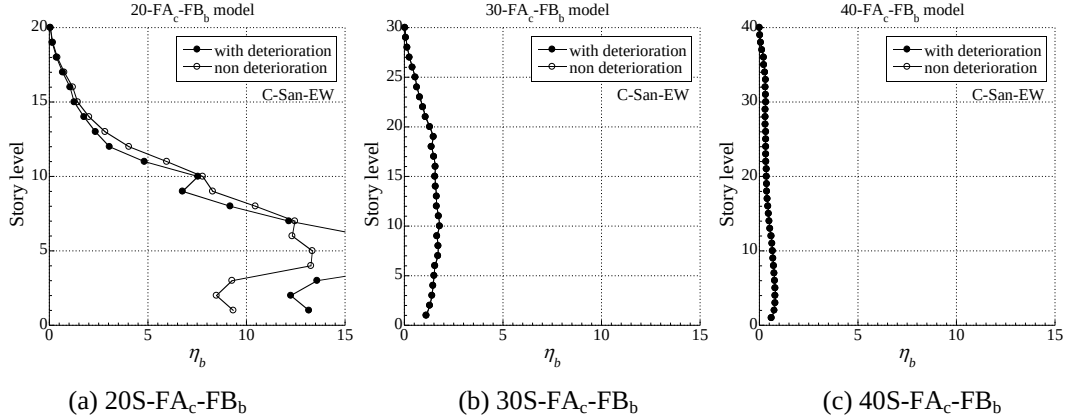


Figure 4.35 η_b of various height building models under C-San-EW

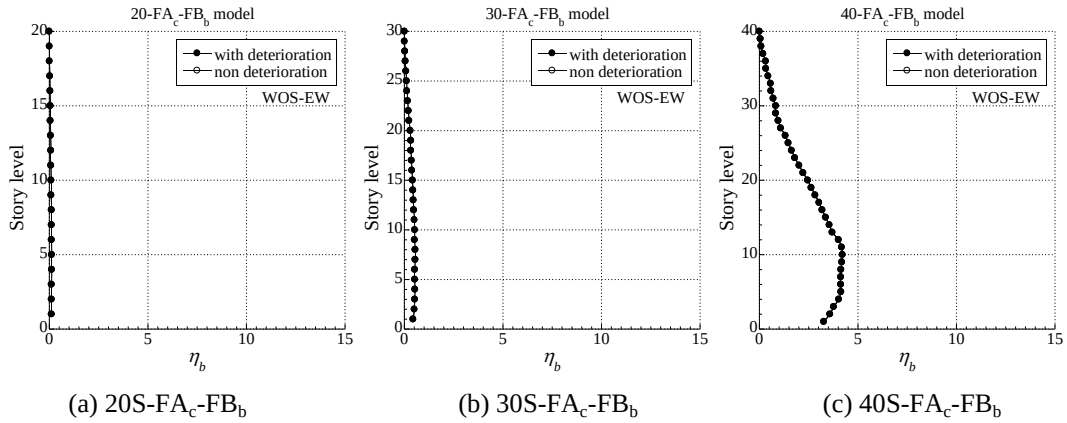


Figure 4.36 η_b of various height building models under WOS-EW

4.6 Seismic Response under Near-Fault Ground Motions

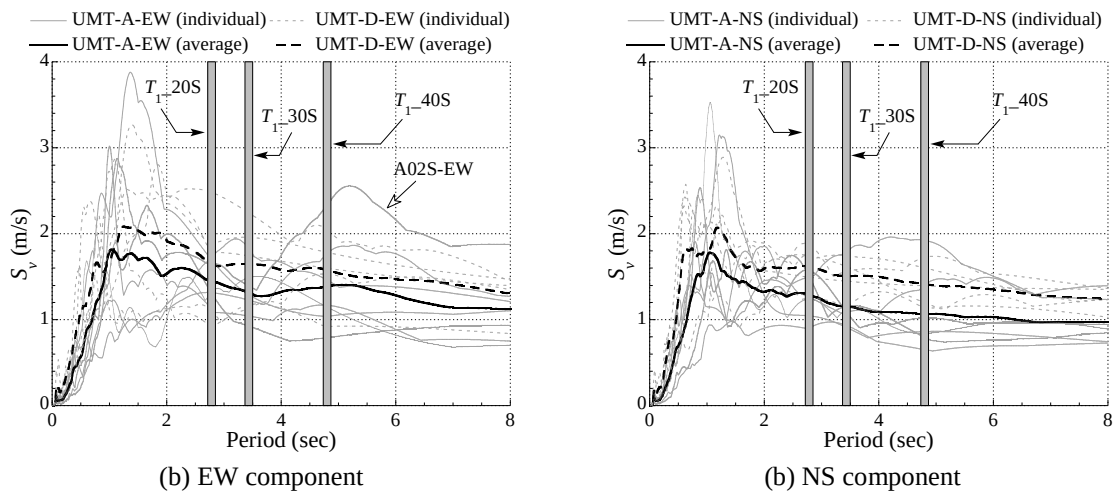
4.6.1 Characteristics of ground motions

The effect of near fault pulse-like ground motions on seismic performance of frame structure have been considered by several researchers, e.g., Alavi and Krawinkler (2004) evaluated the behavior of moment-resisting frame subjected to near-fault ground motions. A series of synthetic pulse-like ground motions are proposed by the prediction of Uemachi near-fault earthquake (UMT) in Osaka (Taga 2011), as shown in Table 4.6. Two components of 'NS' and 'EW' are consisted in this series of near-fault earthquake.

Table 4.6 Parameters of synthetic Uemachi near-fault ground motions

Area	Component	Name	PGA (gal)	PGV (cm/s)
A area	EW	UMT-A02S-EW	387.3	118.4
		UMT-A03S-EW	524.4	155.2
		UMT-A04S-EW	736.2	258.6
		UMT-A05S-EW	636.8	176.7
		UMT-A08S-EW	320.8	62.2
		UMT-A09S-EW	521.5	96.0
		UMT-A10S-EW	353.5	226.7
	NS	UMT-A02S-NS	261.9	74.4
		UMT-A03S-NS	531.0	150.4
		UMT-A04S-NS	548.0	144.7
		UMT-A05S-NS	506.3	154.0
		UMT-A08S-NS	335.2	96.6
		UMT-A09S-NS	620.5	103.4
		UMT-A10S-NS	551.2	148.3
D area	EW	UMT-D02S-EW	768.9	110.0
		UMT-D04S- EW	1044.4	123.1
		UMT-D06S- EW	885.6	163.2
		UMT-D08S- EW	530.1	74.4
		UMT-D10S- EW	947.5	226.6
	NS	UMT-D02S-NS	598.3	102.6
		UMT-D04S-NS	1187.2	129.7
		UMT-D06S-NS	818.0	128.5
		UMT-D08S-NS	912.5	93.0
		UMT-D10S-NS	831.6	252.0

Figure 4.37 gives pseudo-velocity spectrum of various UMT waves divided by different component directivities (EW, NS) and areas (A, D), and first three mode natural periods of various high-rise building models with 20, 30, and 40-story levels are also indicated in Figure 4.37. Beside, Figure 4.38 showing the time history of UMT-A02S-EW exhibits obvious pulse characteristics of ground motions. Two representative waves of UMT-A02S-EW and UMT-D06S-EW which have large seismic pulse in long-period range (see Figures 4.38~4.39) are selected to investigate the effect of member deterioration on seismic-resistant capacity of high-rise steel buildings. As shown in Figures 4.38(b) and 4.39(b), obviously large long period component of these two near-fault waves are captured.

**Figure 4.37** Pseudo velocity spectrum of near-fault ground motions induced in UMT earthquake

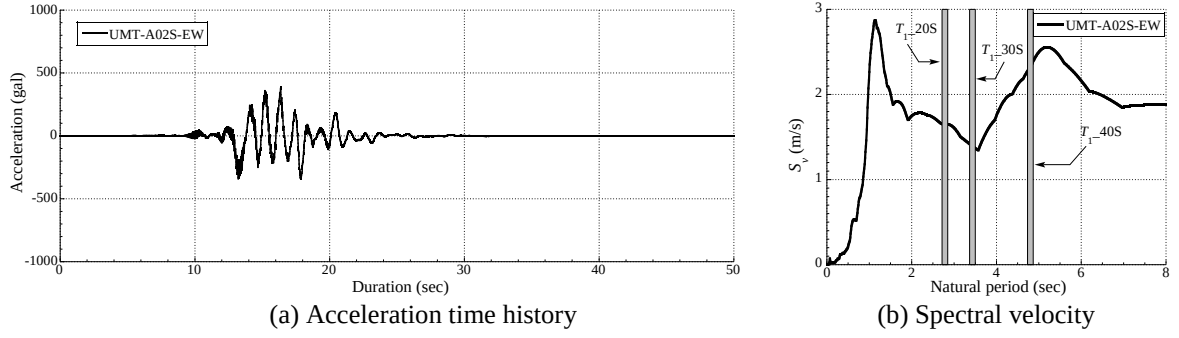


Figure 4.38 Acceleration Time history and spectral velocity of UMT-A02S-EW pulse-like waves

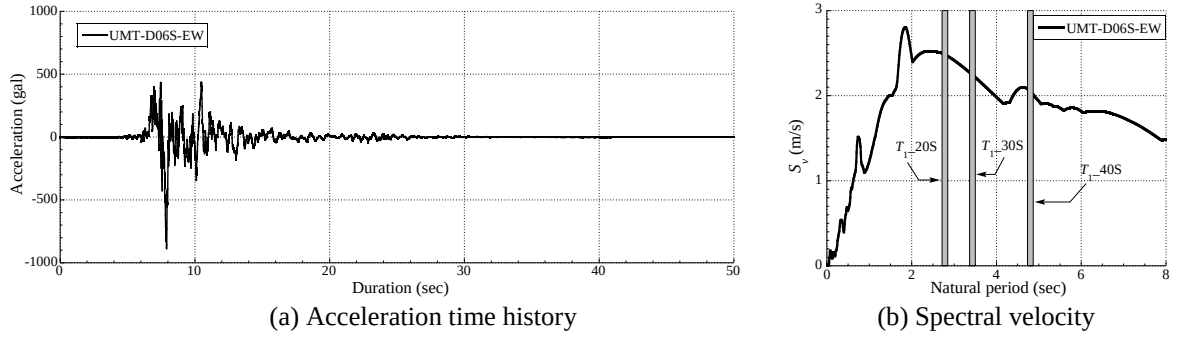


Figure 4.39 Acceleration Time history and spectral velocity of UMT-D06S-EW waves

4.6.2 Response of maximum story drift angle

Maximum story drift angle of various height building models under two representative waves (i.e., UMT-A02S-EW and UMT-D06S-EW) is calculated. For the building models with the same height, different extent deteriorations of structural components are compared with each other, see Figures 4.40~4.41.

According to the observation (see Figure 4.40) of various story S building models, the deterioration zone (height) occurs with obviously different levels, e.g., little visible performance deterioration is achieved in 20S building model [see Figure 4.40(a)], while which of the 30S/40S building models go more obvious. Further, the amplified effects on 40S building model become more severe within smaller height, compared to 30S building model. When various building models subjected to UMT-D06S-EW (see Figure 4.41), 20S building models show large level of story drift angle responses. But as for 40S building models, lower stories are produce extensive story drift angle than those at upper stories.

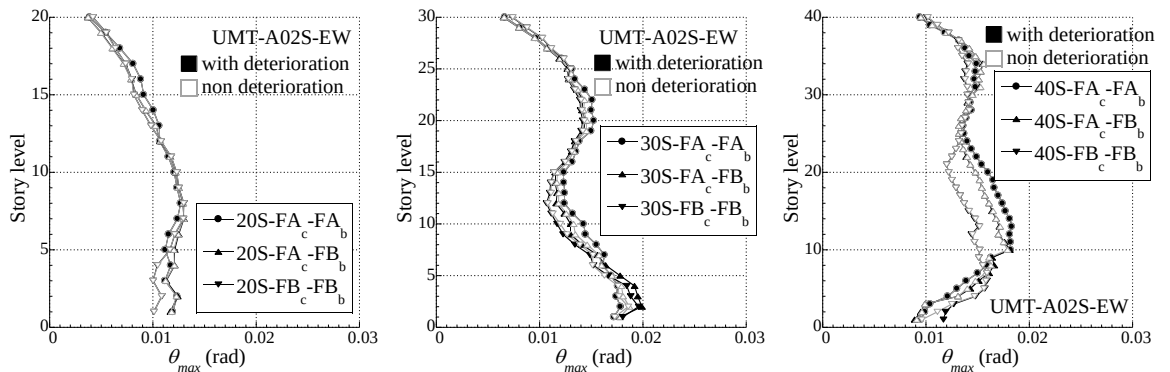


Figure 4.40 Maximum story drift angle of various story building models under UMT-A02S-EW

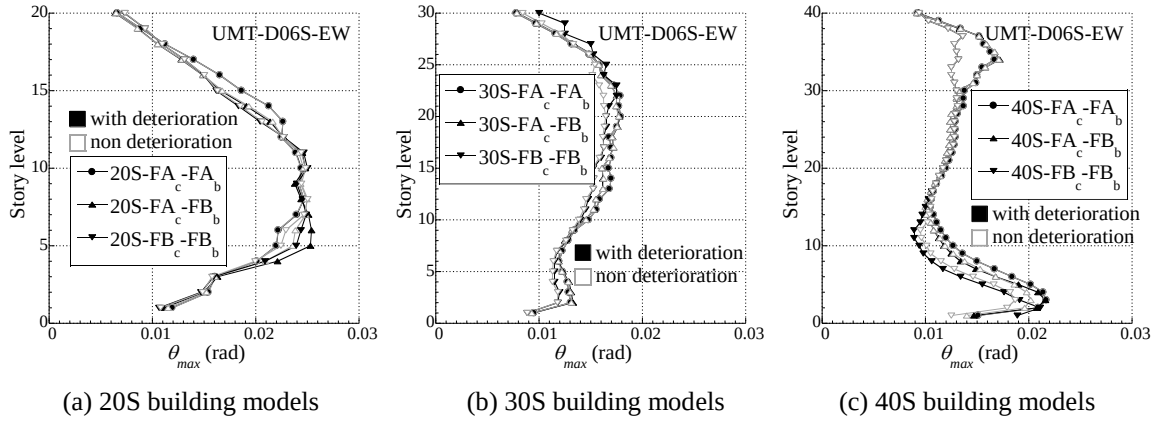


Figure 4.41 Maximum story drift angle of various story building models under UMT-D06S-EW

4.6.3 Response of residual story drift angle

Figures 4.42~4.43 show the seismic responses of residual story drift angle of various high-rise steel building models under UMT-A02S-EW and UMT-D06S-EW waves. The deterioration effects are incorporated or not to explore the influences on residual story drift angle after earthquake ended. As the comparisons we observed in Figures 4.42~4.43, some findings can be achieved below.

As shown in Figure 4.42, residual story drift angle is mainly concentrated in lower story, overall. The influence of member deterioration is also mostly induced at lower stories. Beside, in contrast, when the building models are subjected to UMT-D06S-EW with larger PGV, deformation concentrate on lower stories corresponding to that of maximum story drift angle.

But according to Figure 4.43, extensively large residual story drift angle remains, especially for 20S-FA_c-FB_b building model. As for 40S-FA_c-FB_b building model under UMT-D06S-EW, member deterioration effect further amplifies the deformation concentration of residual story drift angle within the height from bottom story to tenth story.

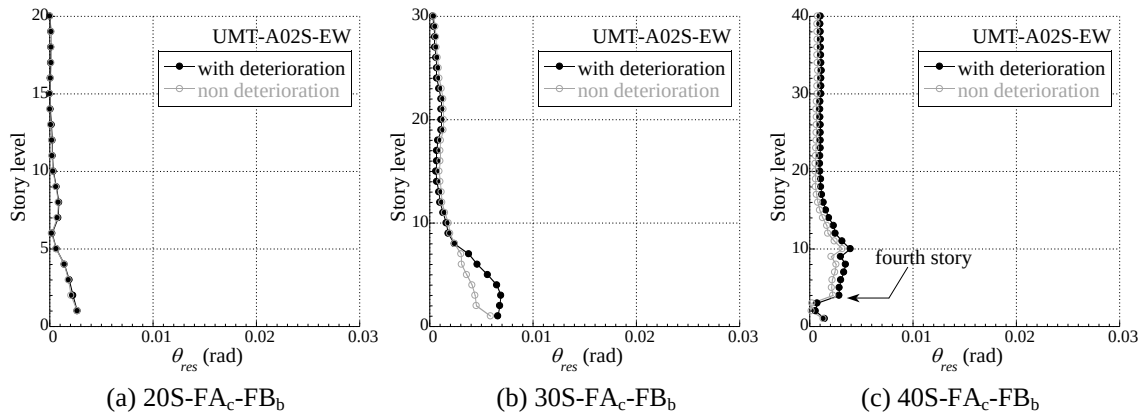


Figure 4.42 Residual story drift angle of various story building models under UMT-A02S-EW

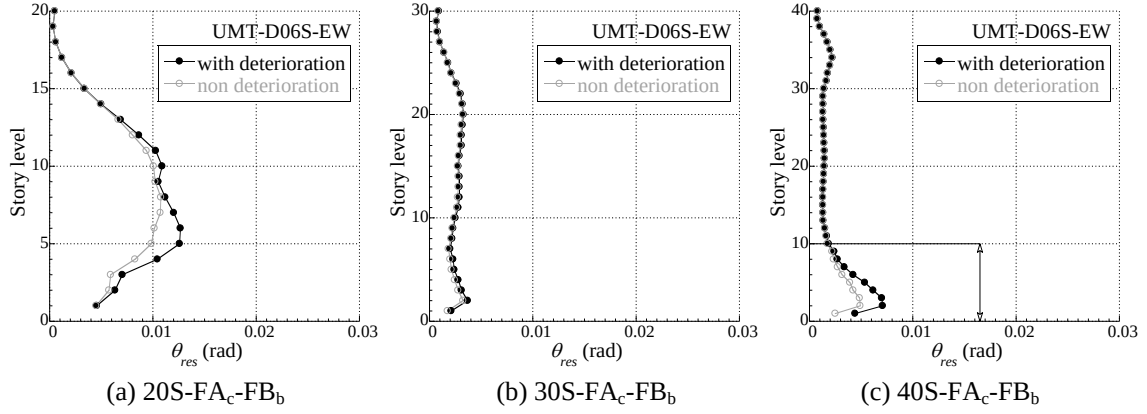


Figure 4.43 Residual story drift angle of various story building models under UMT-D06S-EW

Besides, a sudden increasing of residual story drift angle at the fourth story of 40S-FA_c-FB_b building model under UMT-A02S-EW and the second story of 40S-FA_c-FB_b building model under UMT-D06S-EW. Hence, the time history story drift angle responses of these weak stories and their adjacent stories are compared with each other as demonstrated in Figure 4.44.

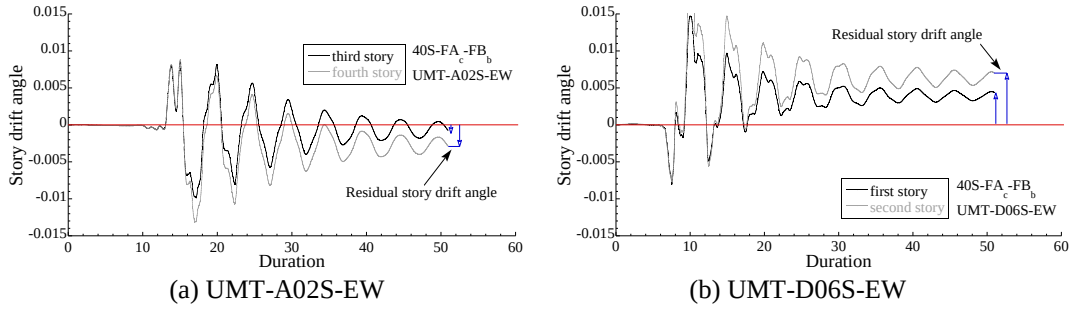


Figure 4.44 Time history story drift angle of 40S-FA_c-FB_b building model

In addition, the dependence of residual story drift angle on maximum story drift angle of 40S-FA_c-FB_b building model under a series of UMT near-fault ground motions (see Figure 4.45). An increasing trend of the $\theta_{res}/\theta_{max}$ ratio is observed in Figure 4.45. The $Y(\theta_{max})$ value of the point where cuts Y -axis and the trend line (red dashed line) can be derived from the results of both deteriorating and non-deteriorating buildings models. The non-deteriorating building model overestimates the capacity of residual story drift angle occurring prevention. A coefficient that is expressed by

$$\kappa = \frac{C_{det}}{C_{non}} \quad (4.12)$$

Where, C_{det} is a criterion of deteriorating building model where residual story drift angle starts to show accelerated growth; C_{non} is a criterion of non-deteriorating building model where residual story drift angle starts to show accelerated growth. Hence, in the case of the comparison of 40S-FA_c-FB_b building model (see Figure 4.45), $\kappa=0.617$. Based on this coefficient, we could quantify the deterioration behaviour of the building models on generating the residual story drift angle after extraordinary earthquakes.

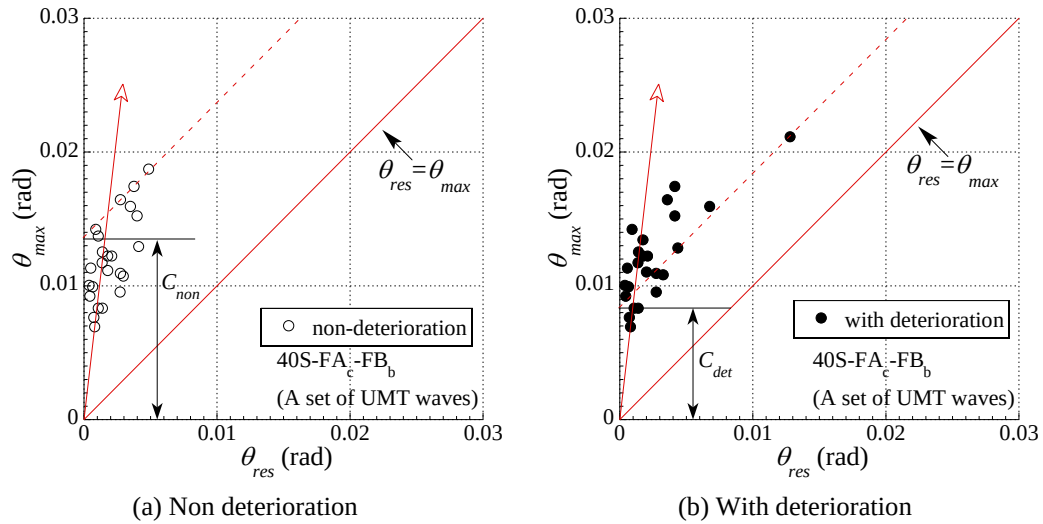


Figure 4.45 Dependence of θ_{res} on θ_{max} response of 40S-FA_c-FB_b building model

Therefore, as for the pulse-like earthquakes with long period component likely to excite larger differentials between the building models with and without deterioration effect. Meanwhile, the residual story drift angle is also more obviously remained in those building models after pulse-like earthquakes than after the long period ground motions induced by subduction-zone earthquakes. Hence, pulse-like ground motions should be more severe for existent high-rise buildings to prevent collapse during earthquake and extensive residual story drift angle after earthquake.

4.6.4 Plastic deformation ratio

As introduced in section 4.5.4, the identical calculation of the plastic deformation ratio at the beam end and column base (μ_b , μ_c) of high-rise building models is utilized to measure the seismic response and plastic damages of the inside components in high-rise building models.

The responses of the plastic deformation ratios at the beam and column ends of building models are calculated on the basis of the maximum plastic rotation at the beam and column ends during seismic excitations. The beam and column sections have different plastic rotating capacities before producing plastic hinges. Hence, the responses of plastic deformation ratios at the beam ends and column ends are demonstrated in Figures 4.46~4.47, and Figures 4.48~4.49, respectively.

(a) Only 40S-FB_c-FB_b building model shows deterioration behavior on the plastic deformation ratios at the beam ends caused by member deterioration of H-shaped steel beam [Figure 4.46~4.47 (c)]. Furthermore, due to the characteristics of pulse-like ground motions, the distribution of the plastic deformation ratio varied with story level.

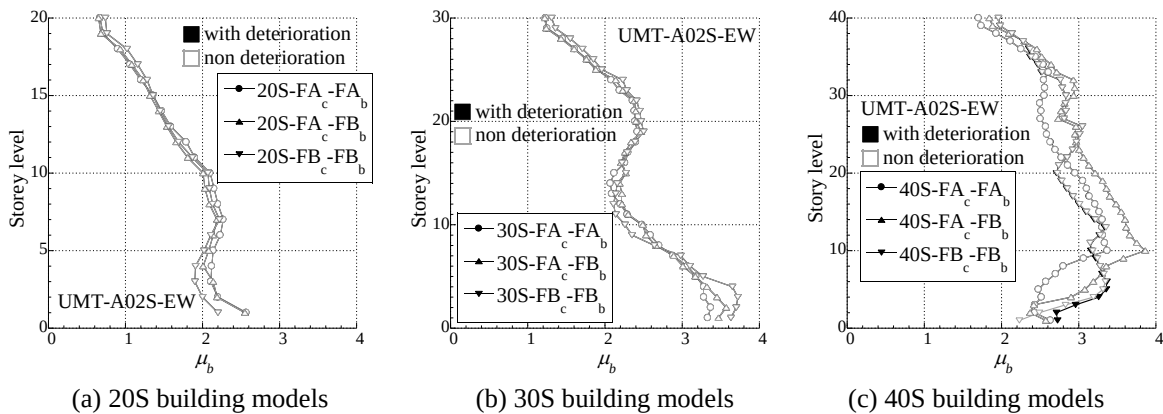


Figure 4.46 Plastic deformation ratio at the beam ends of various building models under UMT-A02S-EW

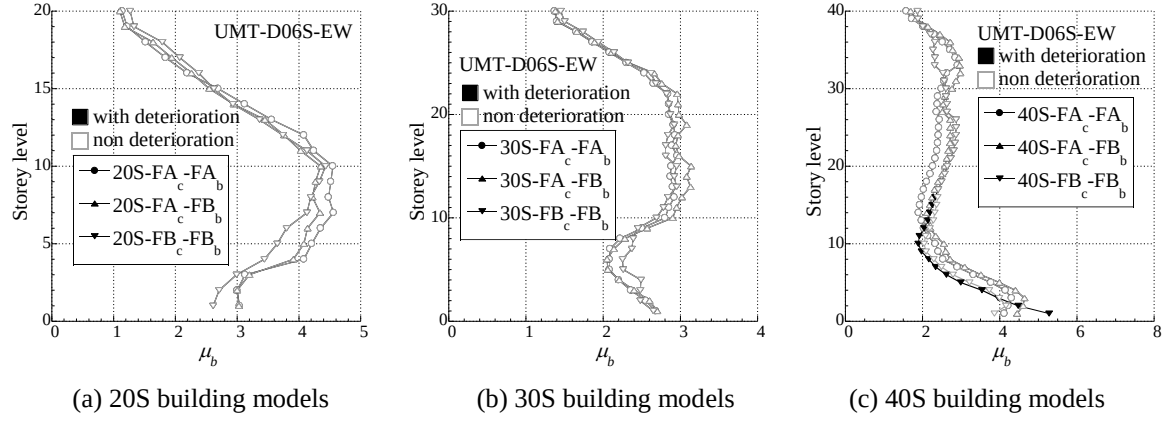


Figure 4.47 Plastic deformation ratio at the beam ends of various building models under UMT-D06S-EW

(b) With respect to the plastic deformation ratio at the column ends (see Figures 4.48~4.49), it is observed that certain lower stories induce plastic rotations beside the column-bases at bottom story. However, only the column bases at bottom story is significantly affected by member deterioration and the maximum plastic deformation ratios exceed 4 which indicates the median damage of the columns has been induced.

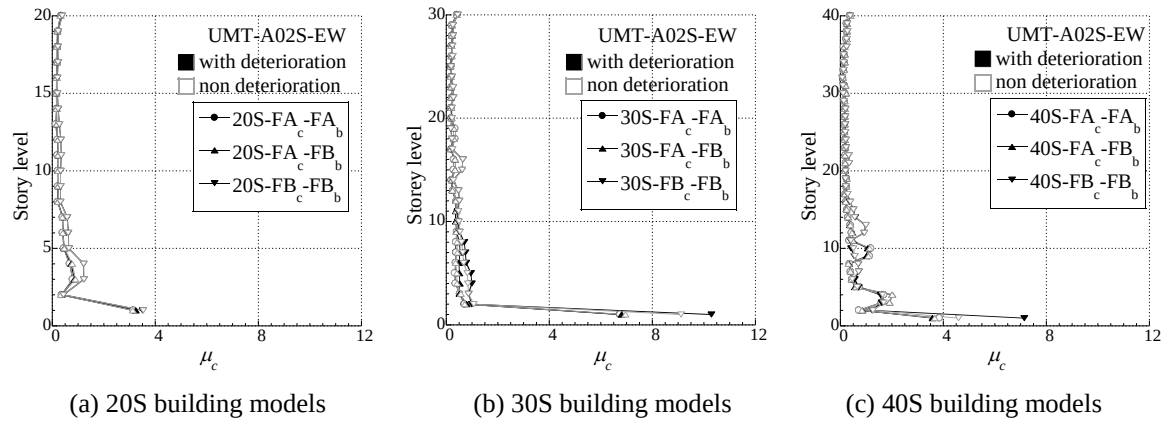


Figure 4.48 Plastic deformation ratio at the column ends of various building models under UMT-A02S-EW

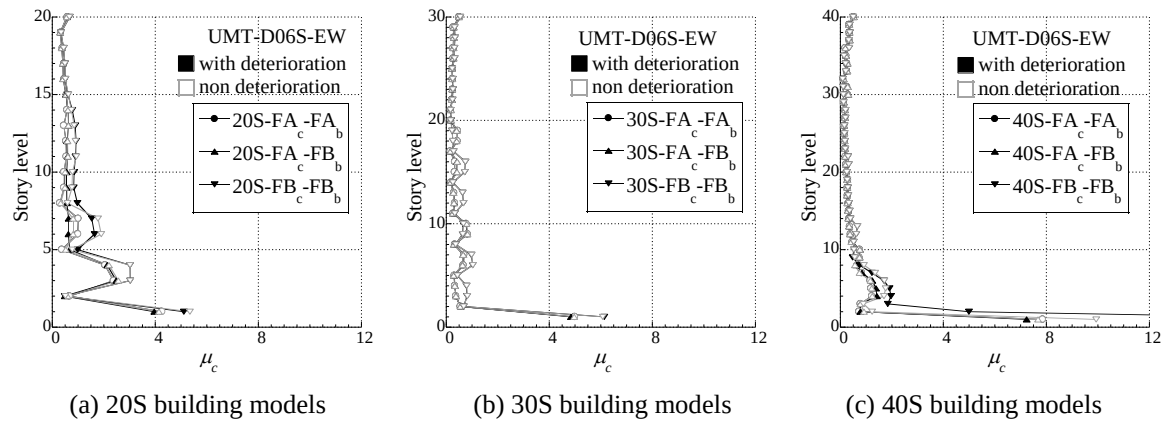


Figure 4.49 Plastic deformation ratio at the column ends of various building models under UMT-A02S-EW

4.6.5 Cumulative plastic deformation ratio

Likewise, the cumulative plastic deformation ratios at the beam ends and column ends (η_b , η_c , respectively) are also calculated by the formulae introduced in Section 4.5.5. The cumulative plastic deformation ratio of various height building models under a series of UMT waves are also analysed to imply the effects of strength deterioration on the hysteretic energy dissipation of 40S model.

Figures 4.50~4.53 compared the results of the maximum η (η_b , η_c) between deteriorating and non-deteriorating building models. In particular, the comparison of η_b , η_c responses between the building models with and without deterioration effect subjected to UMT-A02S-EW and UMT-D06-EW along the story direction demonstrated in Figures 4.50~4.51.

(a) As shown in Figures 4.50, 4.51, for the η_b of beam components, the influence of story (20S/30S/40S) on η_b could be significantly observed, and the strength deterioration increasingly occurs along with the story increases. Quantitatively speaking, the η_b responses of various models under UMT-A02S-EW respectively are 4.25, 2.97 and 5.00, and the η_b responses of the corresponding models under UMT-D06S-EW are 2.11, 6.31 and 6.61. It implies that low level of cumulative plastic energy has been generally dissipated by beam components.

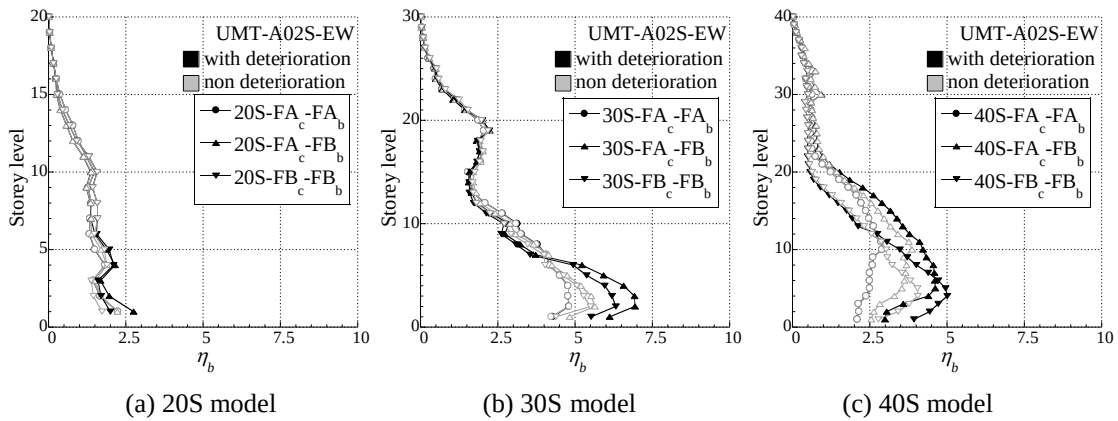


Figure 4.50 Cumulative plastic deformation of beam (A02S-EW)

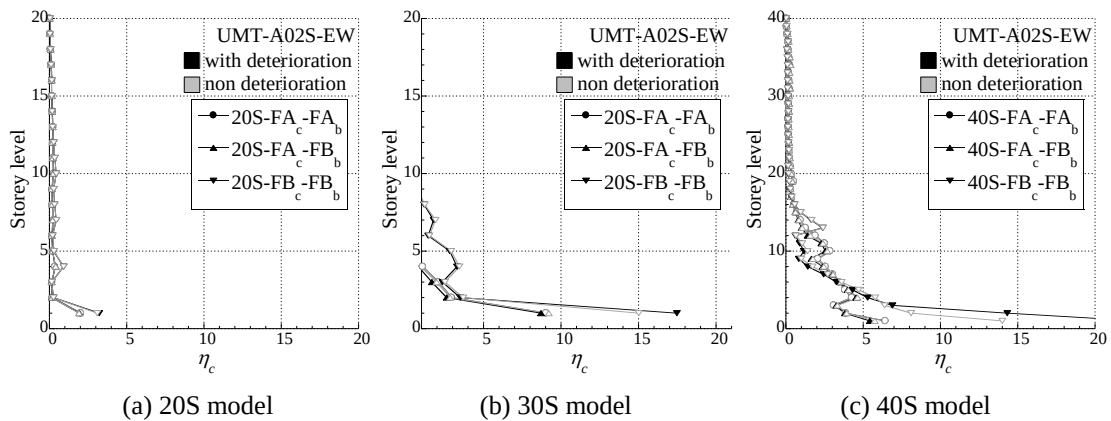


Figure 4.51 Cumulative plastic deformation of column (A02S-EW)

(b) According to Figures 4.52, 4.53, η_c of column components has more energy dissipation than that of beam components. According to the comparison of η_c , η_c is mainly concentrated at lower story, especially at the column-base of the bottom-story of various building models.

Moreover, η_c of deteriorating building model at lower-story achieves significantly larger than 20, which indicates high level of cumulative plastic energy are absorbed at this position.

(c) Lower stories have more amplified influences of strength deterioration, in terms of the comparisons on η of deteriorating and non-deteriorating models. Meanwhile, the deterioration heights of η_b and η_c show varied patterns. This phenomenon could be simply concluded that, on one hand, deterioration height of η_b is larger than that of η_c due to that for weak-beam frame, yielding of beam earlier than that of column would make more beam components be occurred cumulative plastic deformation. On the other hand, effects of deterioration on η_c at the bottom-story of models show significantly larger than that of η_b .

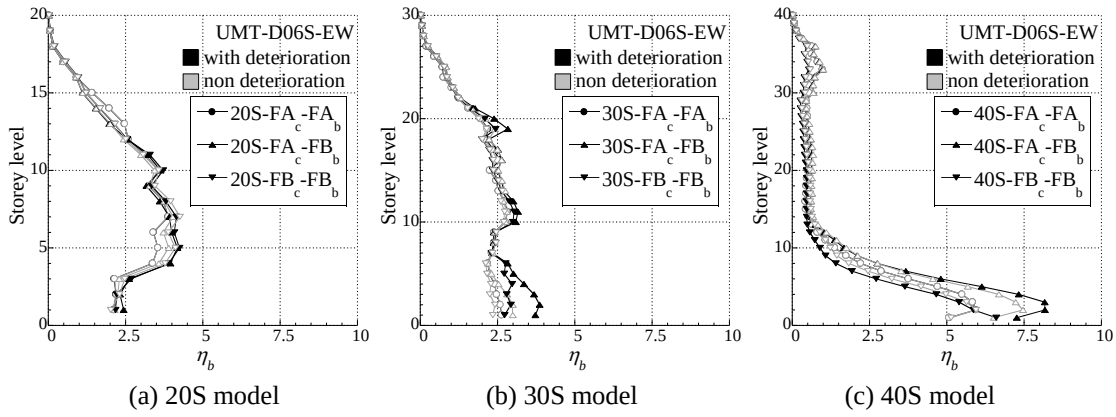


Figure 4.52 Cumulative plastic deformation of beam (D06S-EW)

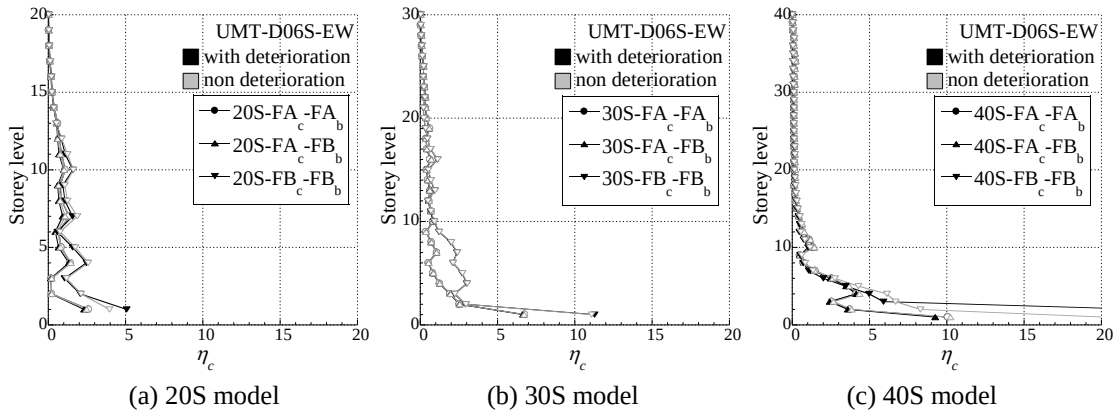


Figure 4.53 Cumulative plastic deformation of column (D06S-EW)

Furthermore, the time-history cumulative hysteretic energy locally absorbed in the most damaged element is also evaluated. The column-base element at bottom story (see Figure 4.54) which tends to dissipate a large amount of hysteretic energy is captured.

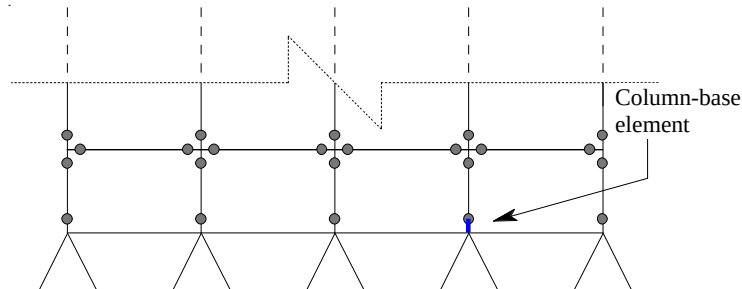


Figure 4.54 Captured column-base element at bottom story

Comparisons of time history energy accumulation of the bottom story column-base element in various high-rise buildings are conducted as shown in Figures 4.55~4.57. In which, by comparing with the criterion of $\eta=1, 5$ and 10 , the member deterioration of 40S-FA_c-FA_b model has not been induced within the range of $5 < \eta < 20$. However, in the case of 40S-FA_c-FA_b model, the incipient deterioration begins to be induced at the point ($\tau=9.67\text{sec}$, $\eta=12.9$, where τ is instantaneous time during earthquake).

Be aimed at the initiation and developing of the deterioration effect on cumulative energy dissipation in high-rise buildings, three stages could be achieved below.

(a) First stage: in this stage, deterioration effect for this column member has not been initiated during the whole duration of seismic excitations [see Figures 4.55~4.57(a)]. This is because that these compact column sections (FA) have ductile capacity to resist local buckling happening.

(b) Second stage: the column member with deterioration effect dissipates less hysteretic energy than that of the column member without deterioration [see Figures 4.55~4.57 (b) and Figure 4.55(c)]. This reverse relation indicates that deterioration effect is induced in beam members prior to in the corresponding columns. Hence, more energy absorptions of the beam members caused by deterioration effect will reduce the columns absorb less energy for the time being.

(c) Third stage: the deterioration effect is developed from beam to column [see Figures 4.56~4.57(c)]. Deterioration behavior of columns being started to be produced means the collapse potential of column members may happen.

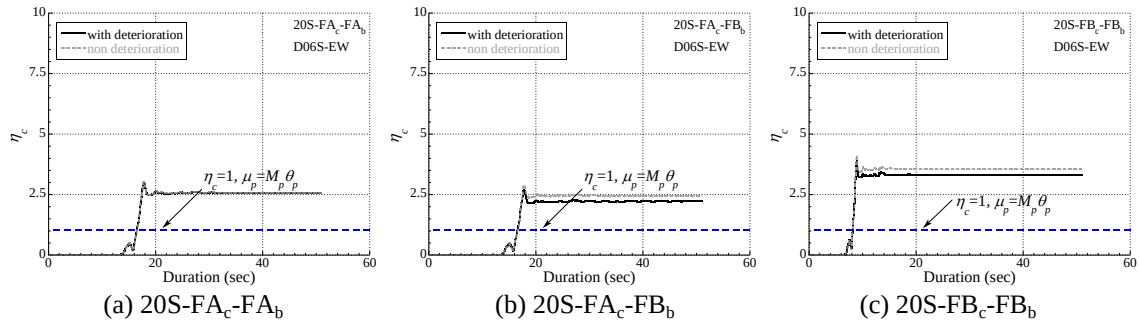


Figure 4.55 Maximum story drift angle of various story building models under UMT-D06S-EW

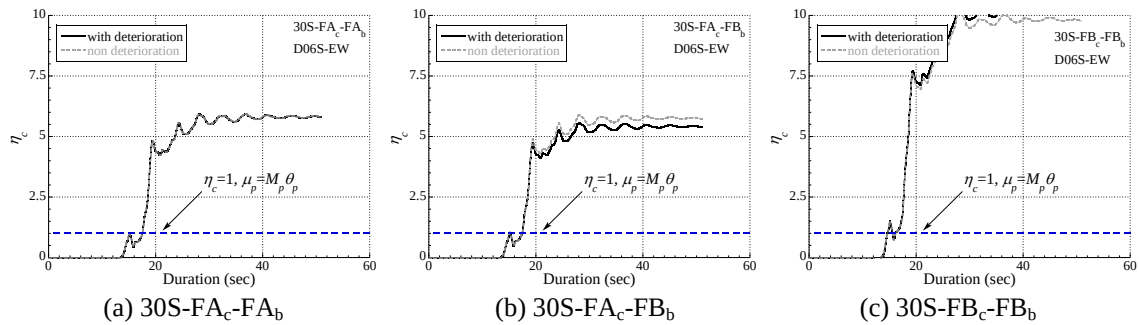


Figure 4.56 Maximum story drift angle of various story building models under UMT-D06S-EW

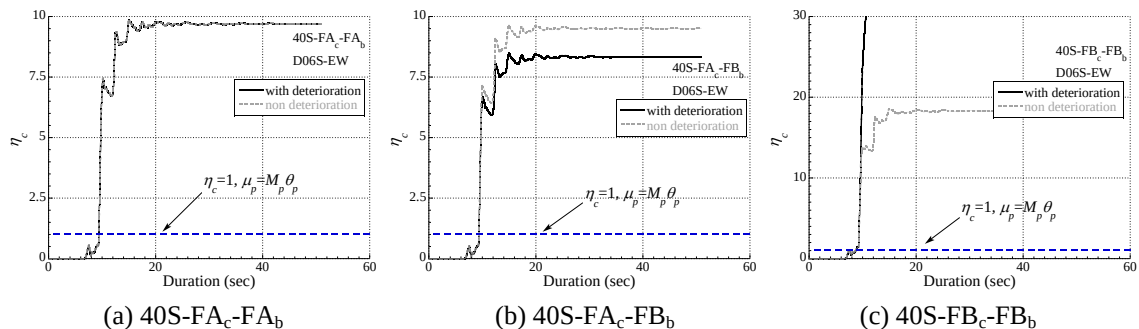


Figure 4.57 Maximum story drift angle of various story building models under UMT-D06S-EW

4.7 Responses of Incremental Dynamic Analysis

Incremental dynamic analysis (IDA) method based on scaling the seismic intensity of earthquake, is proposed by the analogy with the incremental static push-over (SPO) analysis. IDA concept that originally mentioned by Bertero (1977), has been adopted and developed in several research works by Vamvatsikos and Cornell (2002), Zareian and Krawinkler (2007), Liel and Deilerlein (2008) et al. Due to the IDA method is capable to estimate the seismic performance of building structures covering the whole process from elastic to ultimate seismic capacity and until global collapse eventually. Hence, we employ IDAs to predict the whole-process seismic behavior of high-rise buildings, and further discuss the effect of member deterioration on global behavior and safety margin until collapse of building.

According to the discussion in section 4.5, the increase of peak ground velocity (PGV) of ground motions, are likely to have higher correlation as an intensity parameter with the performance parameters (such as maximum and residual story drift angles, plastic deformation ratio and cumulative plastic deformation ratio) of high-rise building structures. Moreover, there is no doubt that the characteristics of seismic ground motions is one of the uncertainty source that should not be ignored in performance-based seismic design, therefore, in this study, a series of artificial waves (i.e., Art-Hachi, BCJ-L2, JSCA-Kobe and Yokohama) that have the closely flat velocity spectrum in the range of long period branch, as shown in Figure 4.58 (a). Besides, based on the details of each input artificial ground motion as shown in Table 4.7, the duration of such artificial waves are $D_{\text{Art-Hachi}}=163.8\text{sec}$, $D_{\text{BCJ-L2}}=120\text{sec}$, $D_{\text{JSCA-Kobe}}=60\text{sec}$, $D_{\text{Yokohama}}=80\text{sec}$, respectively. Thus, the input energies of such earthquakes correspondingly demonstrate differentials as shown in Figure 4.58 (b) on the basis of equivalent pseudo-velocity [$V_E=(2E/M)^{1/2}$] proposed by Akiyama (2006), because that the duration is significantly related to the input energy for frame model (Uang and Bertero 1992).

Table 4.7 Details of the input artificial ground motions

Types	Ground motion	PGA (gal)	PGV (kine)	Duration (sec)
Artificial waves	Art-Hachi	466.7	70.1	163.8
	BCJ-L2	355.7	80.4	120.0
	JSCA-Kobe	469.5	58.7	60.0
	Yokohama	350.2	62.0	80.0

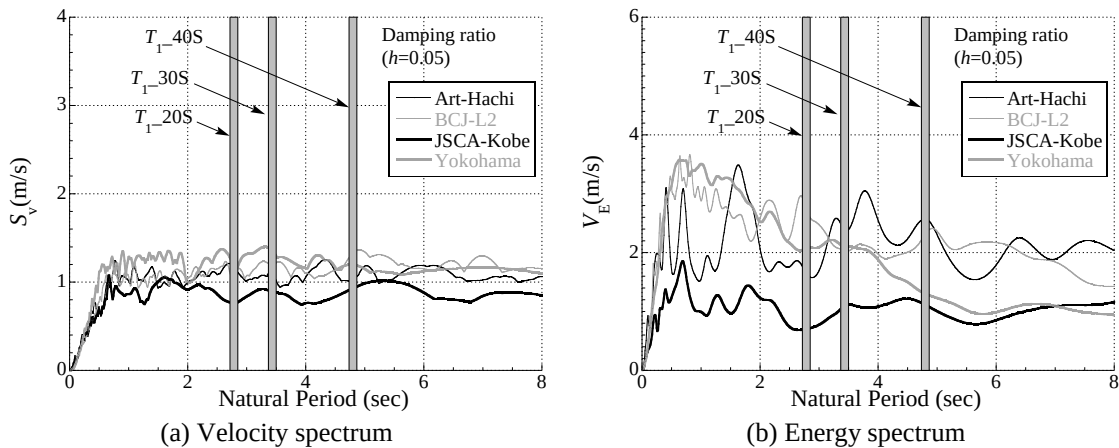


Figure 4.58 Pseudo velocity and energy response spectra of long-period ground motions

4.7.1 Maximum and residual story drift angles

We compare the maximum story drift angle responses of 20-story, 30-story and 40-story high-rise building models incorporating deterioration with those of ignoring deterioration based on IDA method (e.g., Art-Hachi), to reveal the developing of the member deterioration effect on the maximum story drift angle responses of various building models, as shown in Figures 4.59~4.61.

(a) According to the incremental dynamic responses of various building models with different levels of deterioration (FA girder and column represent low deterioration, correspondingly, FB beam and column indicate high deterioration), various non-deteriorating building models shows the similar seismic responses under incremental factor intensities. Such phenomenon indicates that ignoring the local buckling of the steel members in high-rise steel buildings tend to overestimate their seismic behavior [e.g., low-deteriorating and median-deteriorating models: 40S-FA_c-FA_b, 40S-FA_c-FB_b, or small incremental factors ϕ of earthquake, shown in Figures 4.61 (a)~(b)] and collapse capacity [e.g., high-deteriorating: 40S-FB_c-FB_b, or large incremental factors ϕ of earthquake, shown in Figures 4.61 (c)] to some extent, especially for high-deteriorating models that utilizing FB rank members.

(b) With respect to the amplified effect of member deterioration on θ_{max} response, the 40S-FA_c-FA_b building model shows little differentials under the IDA of Art-Hachi in Figure 4.61 (a). On the other hand, as shown in Figure 4.61 (b), FB girder produces a little amplified effects of deterioration on the θ_{max} responses of 40S-FA_c-FB_b model under Art-Hachi ($\phi=2, 3$). Furthermore, the FB column significantly deteriorates the seismic behavior of 40S-FB_c-FB_b model [in Figure 4.61 (c)] until collapse occurred.

Thus, for the seismic safety of various existing high-rise steel buildings under high-level earthquakes, member deterioration is an important factor that should be fully considered. Besides, high deteriorations (i.e., FB rank) of beam and column show most overtly effect on the seismic behaviour of high-rise steel buildings, in particular, collapse phenomenon of the 40S-FB_c-FB_b model [in Figure 4.61 (c)] is captured in the case of Art-Hachi ($\phi=2, 3$).

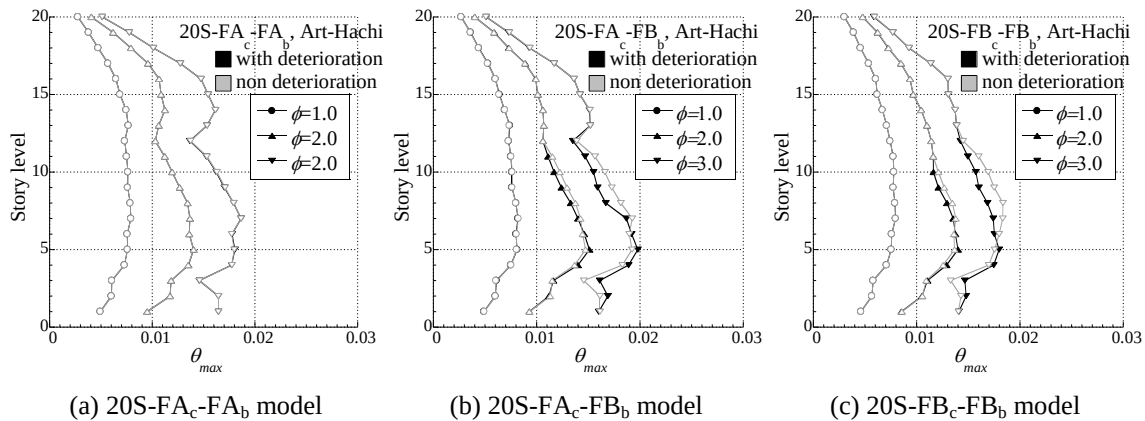


Figure 4.59 Incremental dynamic responses of various 20-story building models under Art-Hachi

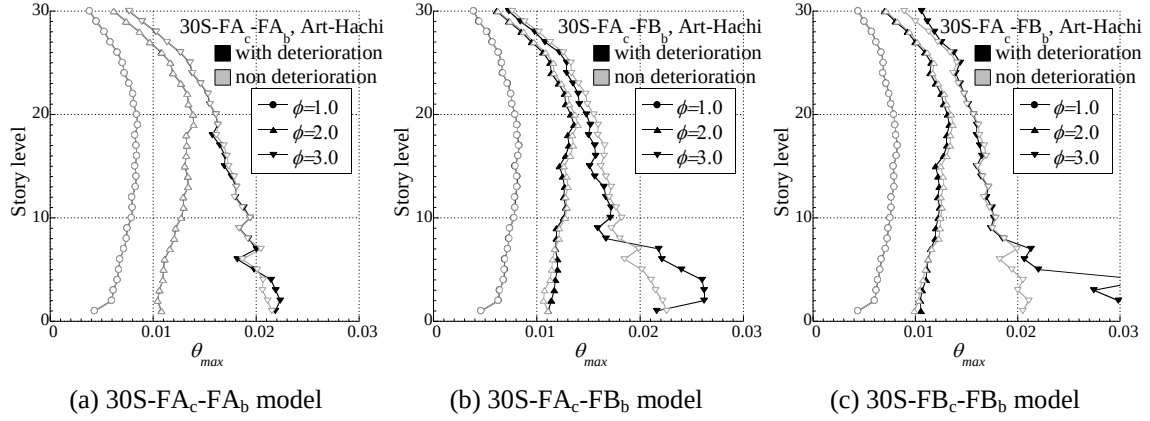


Figure 4.60 Incremental dynamic responses of various 30-story building models under Art-Hachi

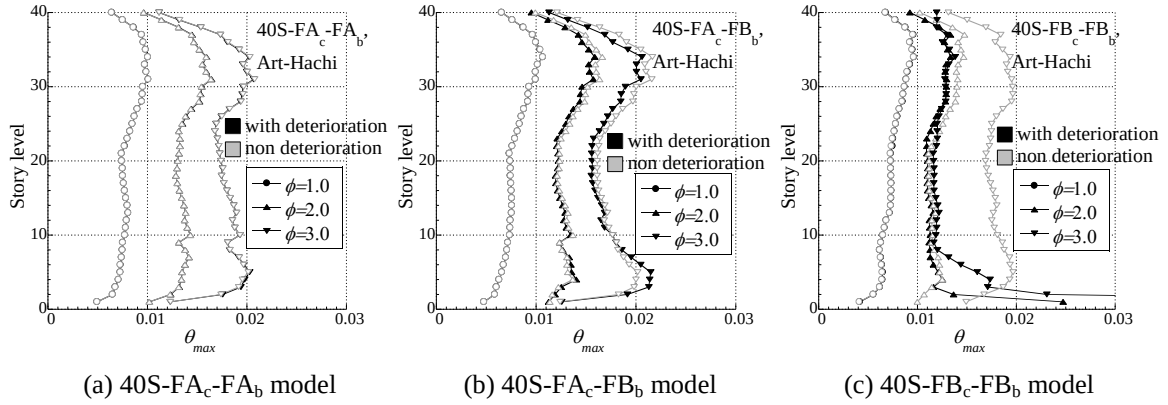


Figure 4.61 Incremental dynamic responses of various 40-story building models under Art-Hachi

Moreover, as the indicators of seismic performance, maximum and residual story drift angles of the high-rise steel buildings versus incremental factor (ϕ) are compared with each other to investigate the ratio of residual story drift angle to maximum story drift angle, as shown in Figures 4.62~4.64.

(a) As for 20S building models, residual story drift angle more feasible to indicate the member deterioration effect on performance-based assessment than maximum story drift angle. Because the story has largest member deterioration coincides with the story has largest residual story drift angle. However, this is not feasible for maximum story drift angle. Thus, as shown in Figures 4.62(b) and (c), the maximum story drift angle of the building without deterioration effect shows smaller than those with deterioration effect.

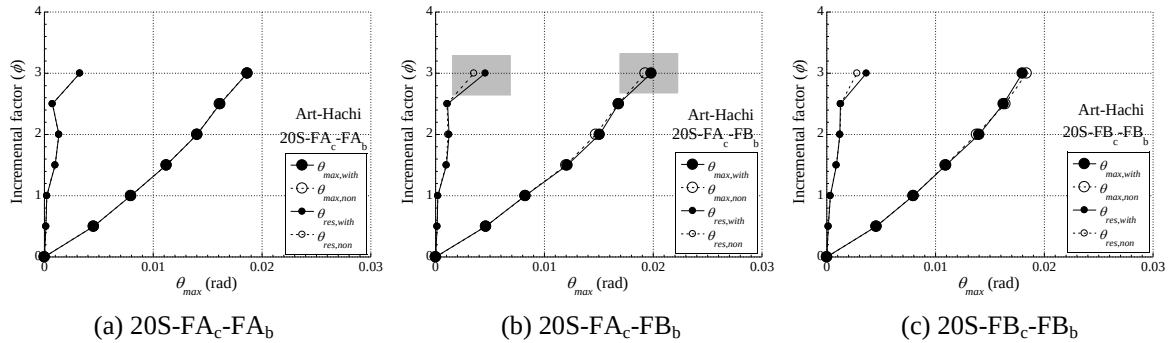


Figure 4.62 Incremental maximum and residual story drift angles of 20S building models

(b) Residual story drift angle stays at a low deformation level (i.e. 0.01rad), and then becomes very sensitive to the intensity of ground motions [see Figures 4.63~4.64(c)]. Thus it is more difficult to utilize the residual story drift angle to evaluate the seismic-resistant capacity of high-rise buildings after earthquake, especially to judge the criteria of residual story drift angle before collapse happening.

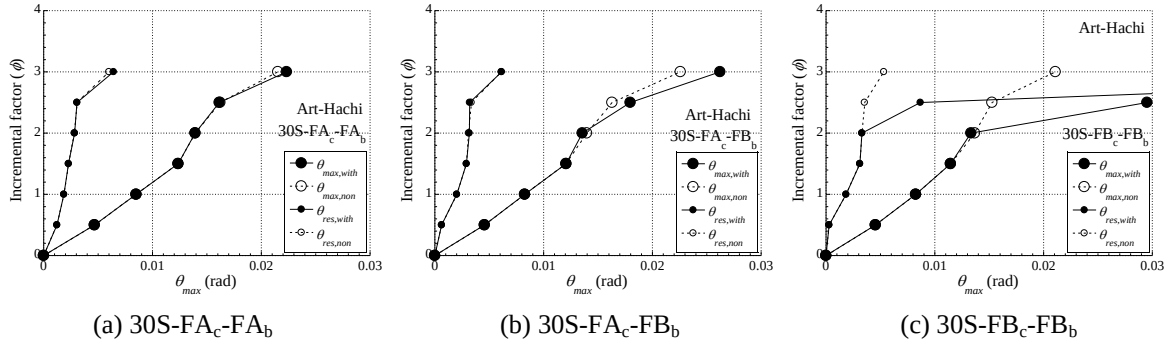


Figure 4.63 Incremental maximum and residual story drift angles of 30S building models

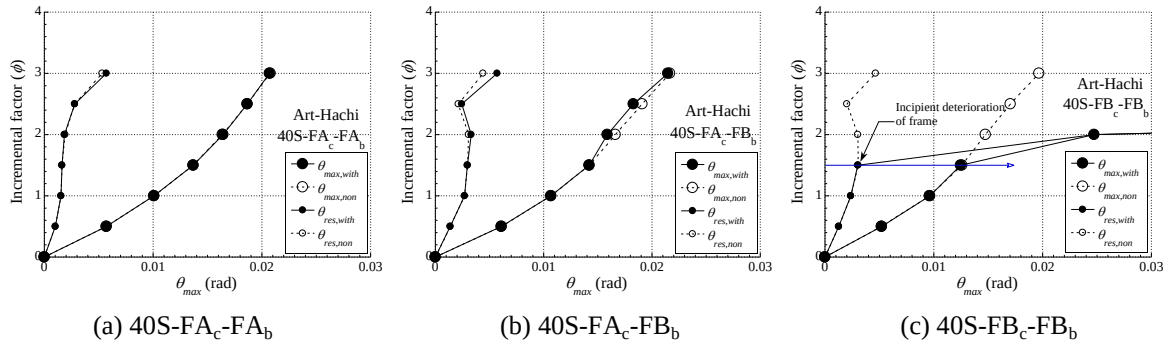


Figure 4.64 Incremental maximum and residual story drift angles of 40S building models

4.7.2 Plastic deformation ratio

Beam and column-base plastic-hinge mechanism of high-rise steel building is assumed happening in structural design of frame models as shown in Figure 4.65, according to weak-beam strong-column philosophy of moment-resisting frame structures. Plastic hinges at the ends of weak beams preliminarily occurs in frame structure [see Figure 4.65 (a)], and the plastic hinges at the column base of bottom story is subsequently induced. Besides, after plastic hinges at column base of the bottom story is induced as shown in Figure 4.65 (b), deformation concentration at lower story portion will gradually occurs (Uetani 1996).

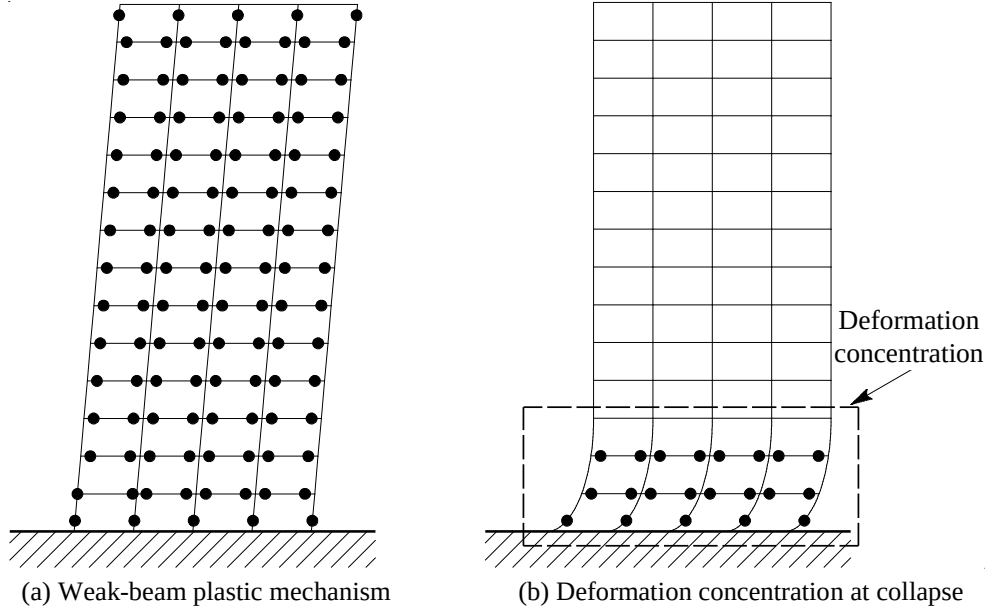


Figure 4.65 Deformation and plastic collapse mechanism of high-rise steel building

The member-level responses of the plastic deformation ratios of at the ends of beam and column-base of various high-rise building models subjected to incremental seismic excitations as shown in Figures 4.66~4.74 are compared between deteriorating and non-deteriorating high-rise building models.

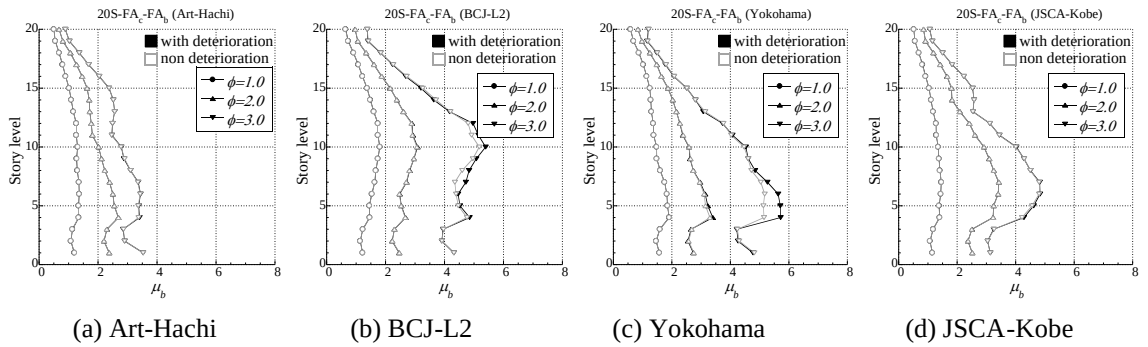


Figure 4.66 Incremental response of the plastic deformation ratio at the beams ends of 20S-FA_c-FA_b

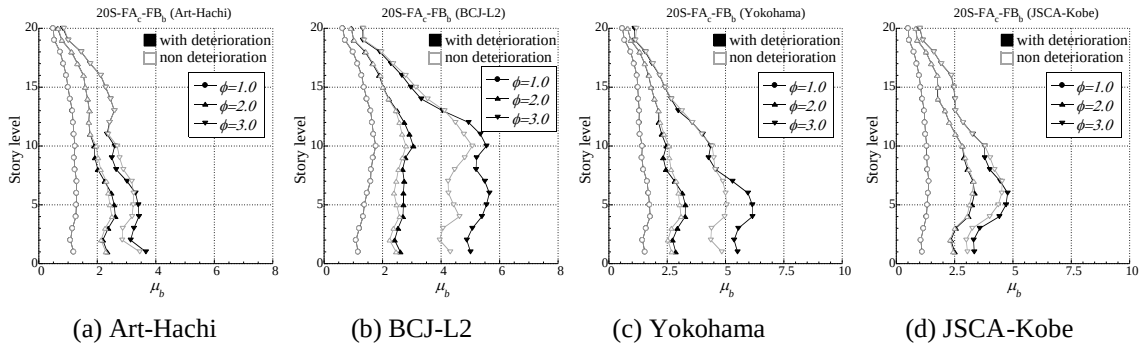
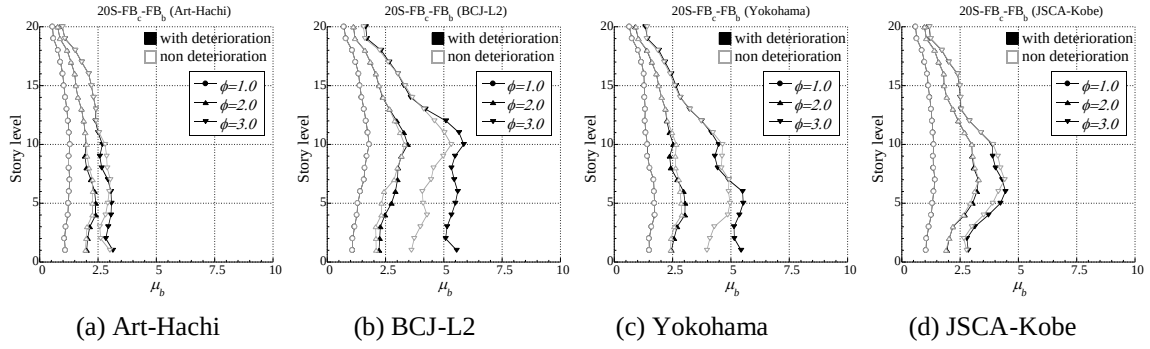
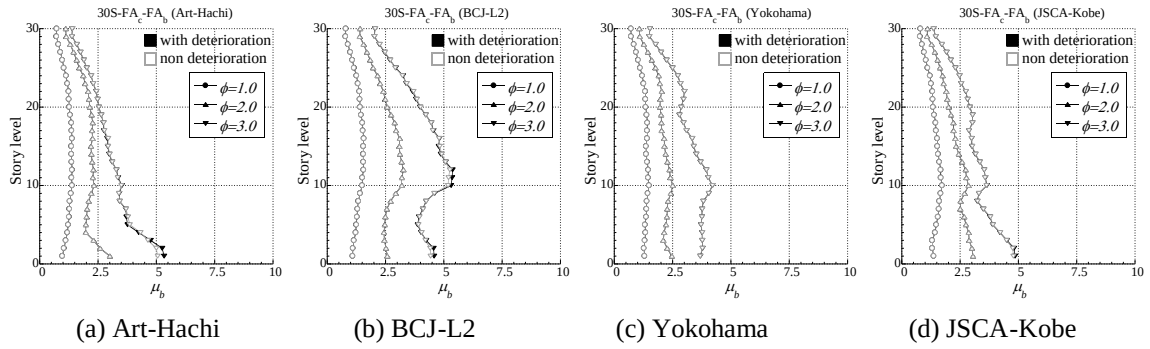
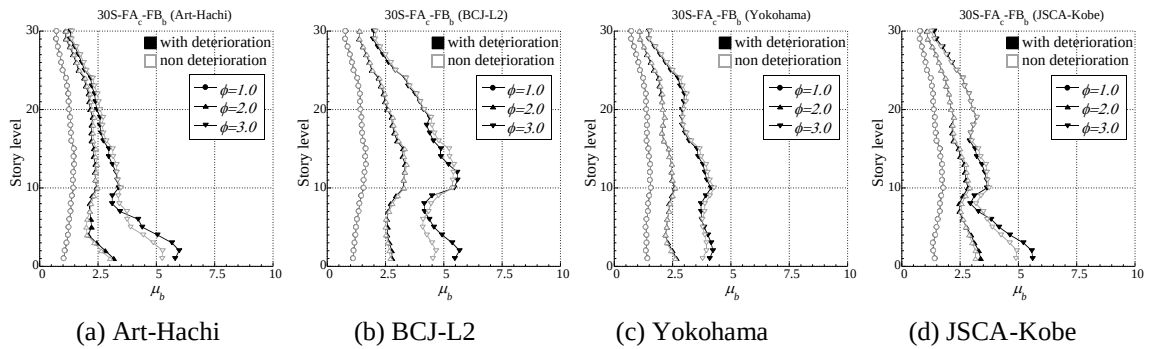
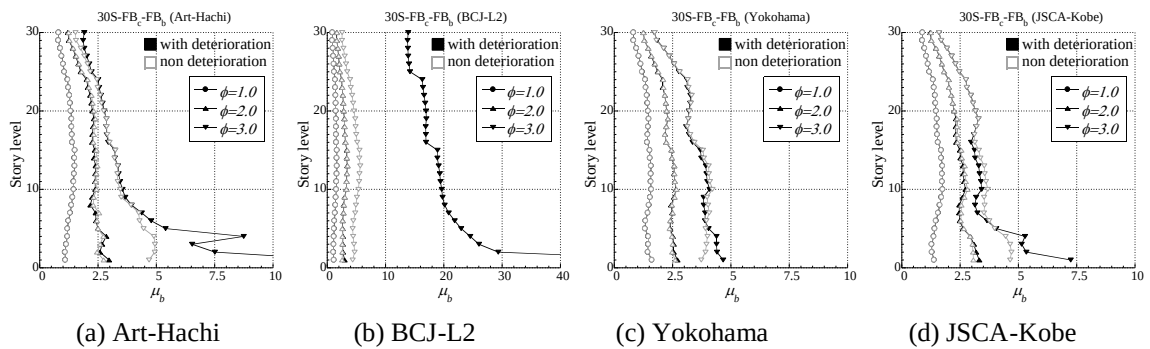


Figure 4.67 Incremental response of the plastic deformation ratio at the beams ends of 20S-FA_c-FB_b


Figure 4.68 Incremental response of the plastic deformation ratio at the beams ends of 20S-FB_c-FB_b

Figure 4.69 Incremental response of the plastic deformation ratio at the beams ends of 30S-FA_c-FA_b

Figure 4.70 Incremental response of the plastic deformation ratio at the beams ends of 30S-FA_c-FB_b

Figure 4.71 Incremental response of the plastic deformation ratio at the beams ends of 30S-FB_c-FB_b

By ignoring the member deterioration, the plastic deformation ratios of upper-story beams of 40S-FA_c-FA_b model tend to proportionally increases as the incremental factor of BCJ-L2 ground motions increases, as shown in Figure 4.72 (b), and the plastic deformation ratios at the lower-story beams increases faster than the increasing of incremental factor ϕ . This indicates the ductility

responses tend to be more and more sensitive to the intensity of ground motions as the inelastic-rotation increases, and the plastic deformation ratios induced at lower-story shown higher sensitivity than that at the upper-story of high-rise building structures. However, the member deterioration of 40S-FA_c-FA_b model is considered as shown in Figure 4.72 (b), a tremendous increasing of the μ_b at both upper-story and lower-story as the incremental factor is increased from 4.0 to 5.0 is observed, which indicates that incipient collapse of 40S-FA_c-FA_b model may be induced under BCJ-L2 seismic excitation with $\phi=5.0$.

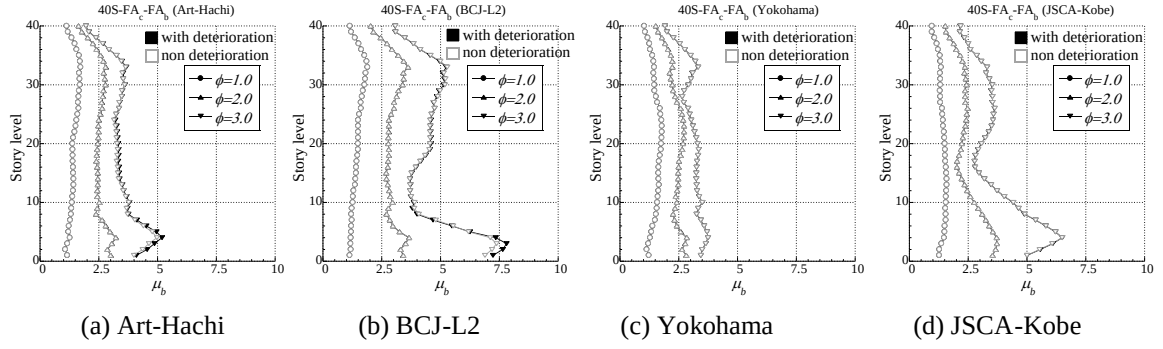


Figure 4.72 Incremental response of the plastic deformation ratio at the beams ends of 40S-FA_c-FA_b

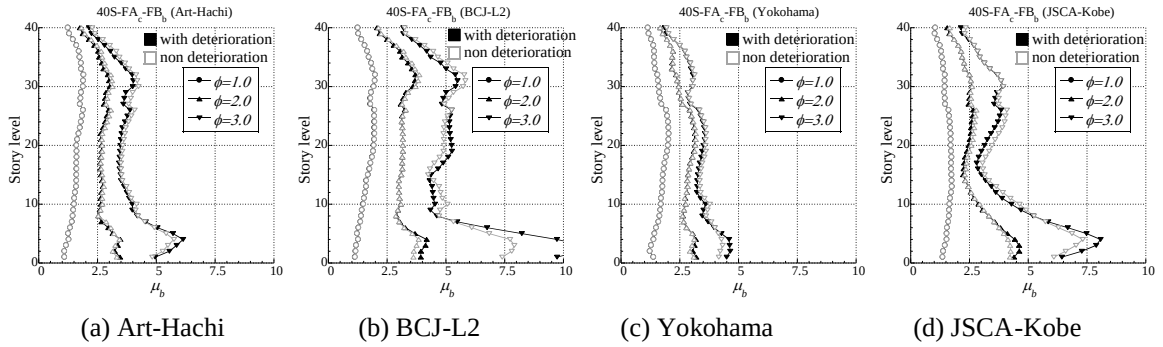


Figure 4.73 Incremental response of the plastic deformation ratio at the beams ends of 40S-FA_c-FB_b

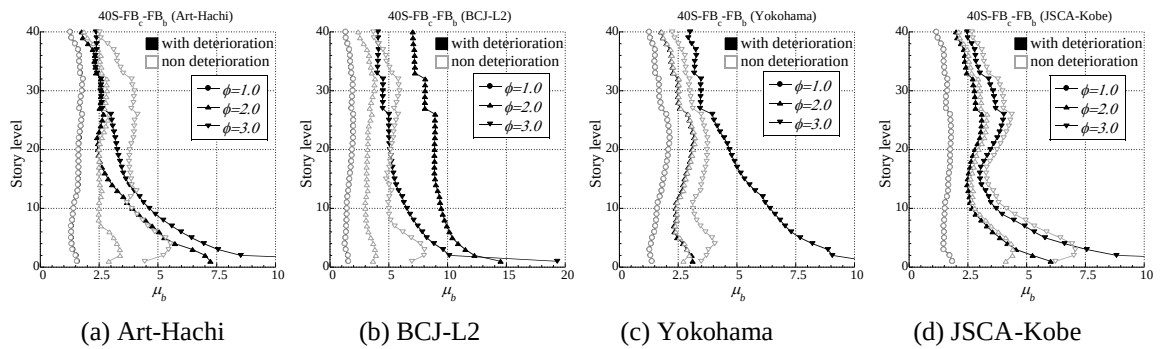


Figure 4.74 Incremental response of the plastic deformation ratio at the beams ends of 40S-FB_c-FB_b

4.7.3 Cumulative plastic deformation ratio

The effect of member deterioration on hysteretic energy dissipation shows differential to that of maximum and residual story drift angle responses as discussed in section 4.5~4.6. Besides, the cumulative plastic deformations of beam and column components also demonstrate various results. Thus, in order to be comparative to the comparison results of plastic deformation ratio (in section

4.7.2), the cumulative plastic deformations of beam and column of the 40S-FA_c-FA_b building models with and without deterioration effect subjected to incremental BCJ-L2 are also evaluated as shown in Figures 4.75~4.76, where generally homogenous developing process of η_b and η_c along the story direction is obtained. Nevertheless, the incremental η_b response is suddenly significantly amplified along the overall story height when incremental factor is increased from $\phi=4.0$ to $\phi=5.0$, but the sensitive increasing of η_b from $\phi=4.0$ to $\phi=5.0$ only concentrates on lower-story as shown in Figure 4.76 (b).

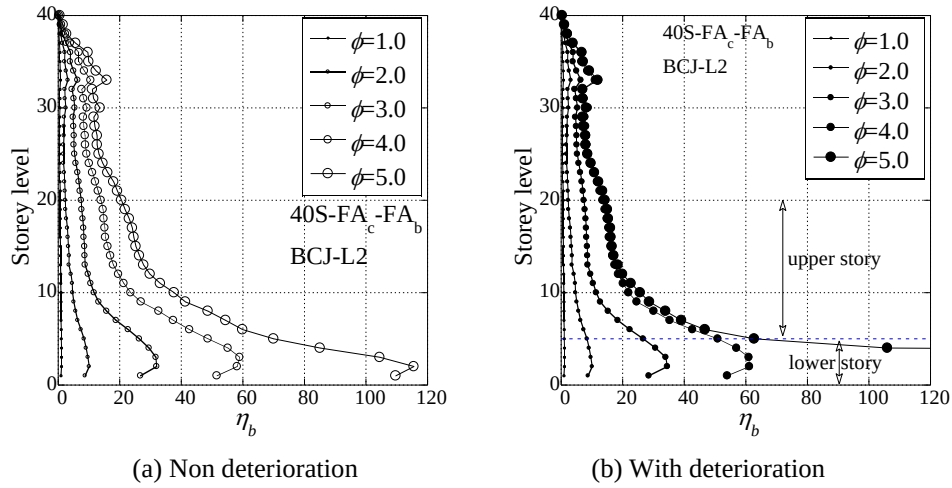


Figure 4.75 Beam cumulative plastic deformation of 40S-FA_c-FA_b model (e.g., under BCJ-L2)

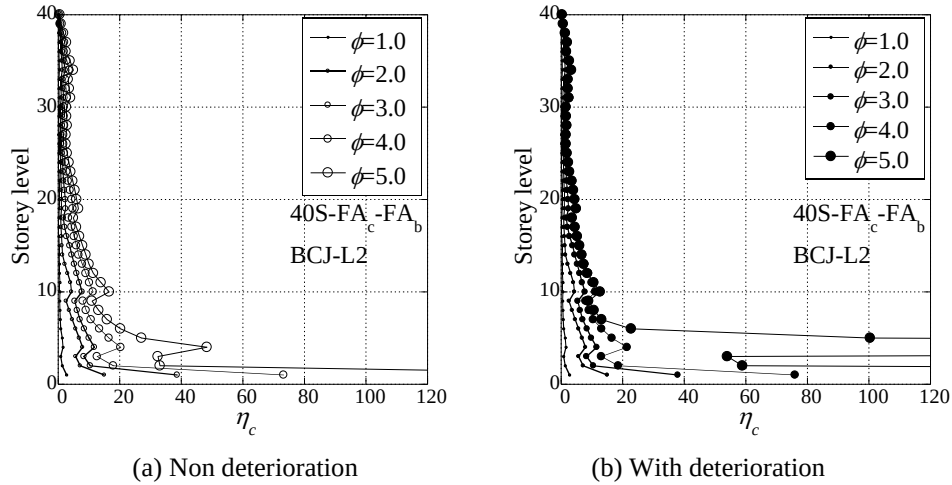


Figure 4.76 Column cumulative plastic deformation of 40S-FA_c-FA_b model (e.g., under BCJ-L2)

Besides, in order to evaluate the sensitivity of η to the incremental factor of ground motions, the beam and column hysteretic energy dissipations of the deteriorating and non-deteriorating 40S-FB_c-FB_b models versus intensity of various artificial ground motions are also compared against each other, as respectively presented in Figures 4.77~4.78.

According to various comparisons in Figures 4.77~4.78, the amplified effect of strength deterioration on η_b at the beam end is larger than that of column. However, the η_b is also extensively developed, e.g., results of η_b reach to be larger than 20 that is regarded as a criteria of severe plastic damages, under various incremental artificial waves with $\phi=3.0$. Moreover, due to

that flat curves of deteriorating model are derived in Figure 4.54, which indicate that η_c shows more and more sensitive when ϕ becomes larger.

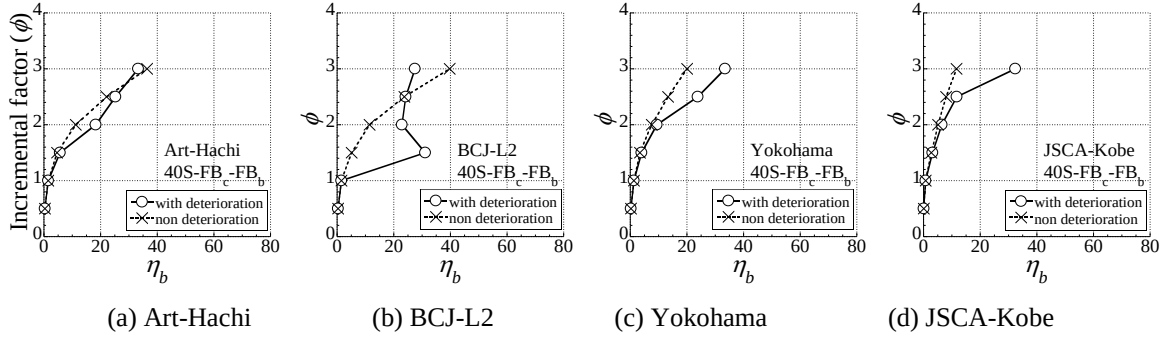


Figure 4.77 Incremental responses of the max. η_b of 40S-FB_c-FB_b building model

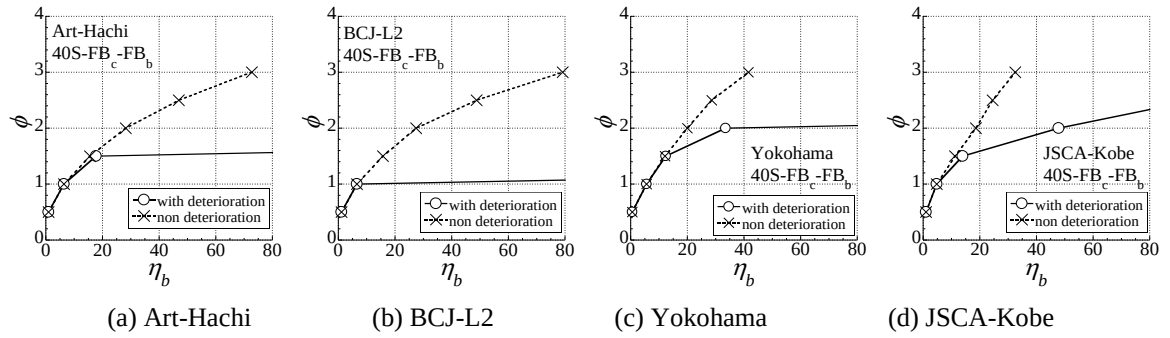


Figure 4.78 Incremental responses of the max. η_c of 40S-FB_c-FB_b building model

Figure 4.79 specifically presents the deterioration concentration on η . Effect of member deterioration increases the cumulative plastic deformation ratio from bottom story up to a certain story. Therefore, for each practical case of high-rise buildings, the height of member deterioration should exist which may collapse in this portion.

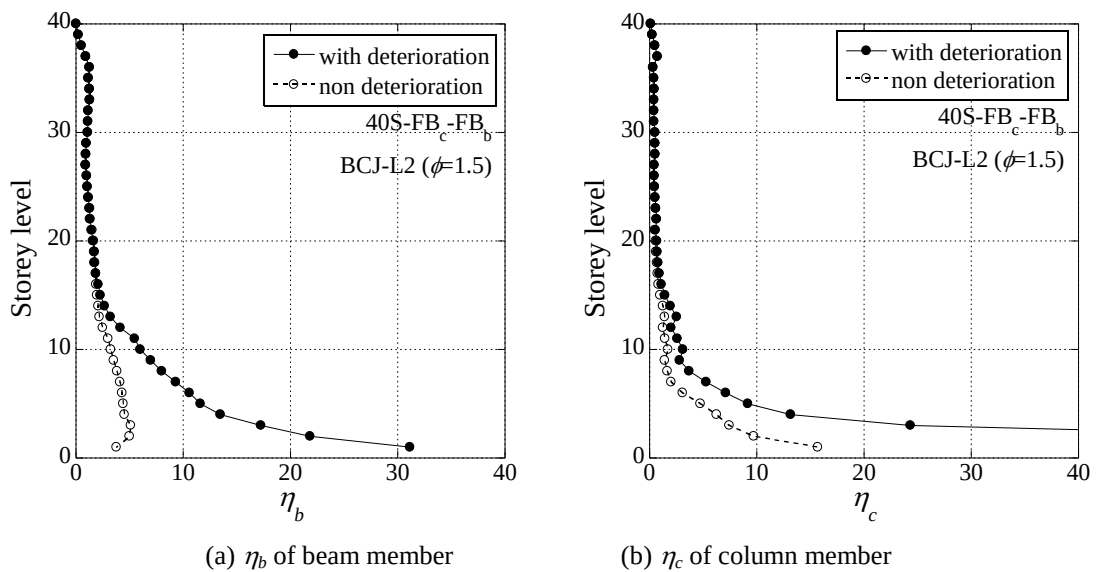


Figure 4.79 Cumulative plastic deformation ratio concentrations of beam and column members

4.7.4 Deterioration transfer of frames

In order to be capable of capturing the deterioration of structural members in existent high-rise buildings, we respectively employ two types of stress-strain models that are the one incorporating deterioration and the other one ignoring deterioration. It should be noted that yielding and local-buckling of the steel members of high-rise buildings could not be simultaneously produced under seismic excitation, while gradually occurred and transferred to adjacent members. Thus, developing of the limit-state of the member deterioration occurring in high-rise steel buildings necessitate the assessment on the effect of member deterioration on seismic-resistant capacity of various high-rise steel buildings subjected to the incremental ground motions based on IDA method.

(1) Only $P-\Delta$ effect

Firstly, we consider the deterioration transfer at the frame level based on the θ_{max} responses of the 40S-FA_c-FA_b building model. $P-\Delta$ effect is only considered in this part to reveal the effect of geometric nonlinearity on seismic responses of high-rise buildings on positive and negative sway directions. Three limit-states which are symmetric state and asymmetric state and instable state can be achieved as shown in Figure 4.80.

(a) It is noted that when the incremental factor of seismic excitation (i.e., Art-Hachi) is small ($\phi \leq 2$), corresponding to Figure 4.80 (a), no difference of horizontal drift between the positive and negative directions is observed and the horizontal sway is under the symmetry limit.

(b) As shown in Figure 4.80 (b), the seismic resistant capacity of building model reduces at side direction because of $P-\Delta$ effect, and lower part starts to come into asymmetry state.

(c) When the asymmetric state develops further, instable state which lead the building structures to collapse as shown in Figure 4.80 (c).

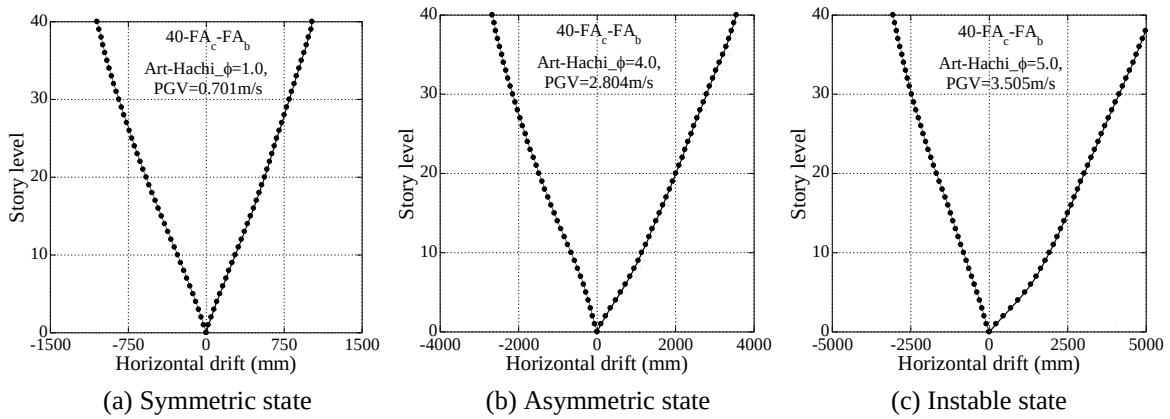


Figure 4.80 Process of deterioration transfer on horizontal drift of 40S-FA_c-FA_b model

(2) $P-\Delta$ effect and member deterioration

Corresponding to the above discussions on the influence of only $P-\Delta$ effect on seismic responses of high-rise buildings under incremental seismic excitations, the influence of member deterioration effect should be also investigated. Likewise, three states are also derived and named as non-deterioration state, partial-deterioration state and full-deterioration state. Figure 4.81 shows the process of deterioration transfer based on the horizontal drift of 40S-FA_c-FA_b model.

(a) When the incremental factor of seismic excitation (i.e., Art-Hachi) is small ($\phi \leq 2$), corresponding to Figure 4.81 (a), no difference of horizontal drift between the deteriorating building model and non-deteriorating building model is observed and the horizontal sway of building model is still under the non-deterioration state.

(b) As shown in Figure 4.81 (b), the seismic resistant capacity of deteriorating building model tends to reduce at the lower part (i.e., partial deterioration is induced), along with the building comes into partial-deterioration state.

(c) When the building achieves the dynamic-instability state, as shown in Figure 4.81 (c), deterioration of member transfers from partial story to whole story. Hence, full-deterioration state is achieved and collapse happens under high intensity level of ground motion.

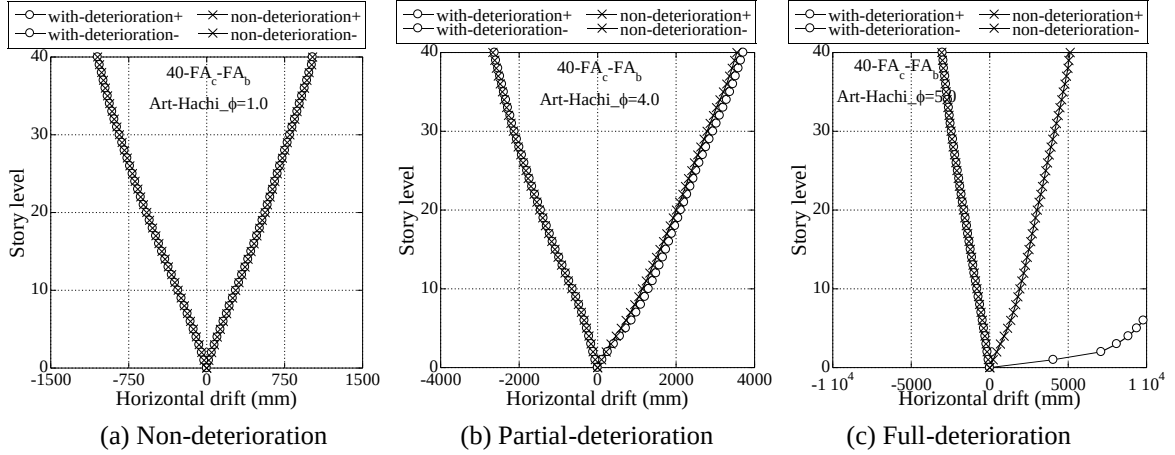


Figure 4.81 Process of deterioration transfer on horizontal drift of 40S-FA_c-FA_b model

The assumed process of deterioration transfer is also proved by the θ_{max} response of the corresponding building model, as shown in Figure 4.82. Deformation concentration of θ_{max} gradually produces when the intensity of ground motion increases.

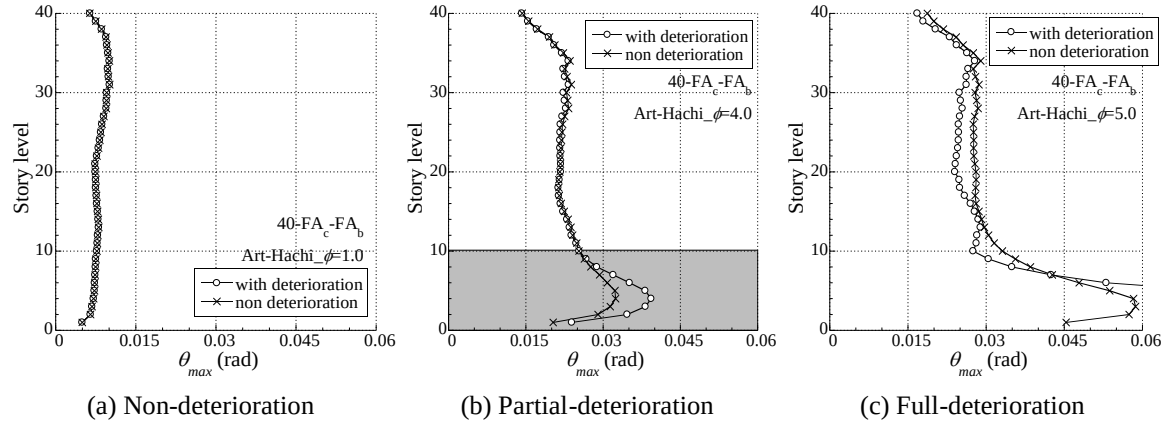


Figure 4.82 Process of deterioration transfer on θ_{max} response of 40S-FA_c-FA_b model

As an index for the moment-curvature capacity of critical regions (e.g., plastic hinge, joint), the plastic deformation ratio of beam (μ_b) is capable to more directly indicate the deterioration transfer inside the high-rise steel buildings, which develops from local portion to global range along with the vertical direction of building structures. The transfer of member deterioration is initiated and extended, as the intensity of ground motions is incrementally increased. So the relation between μ_b and story level of 40S-FB_c-FB_b model under the incremental seismic-excitations of Yokohama are shown in Figure 4.83.

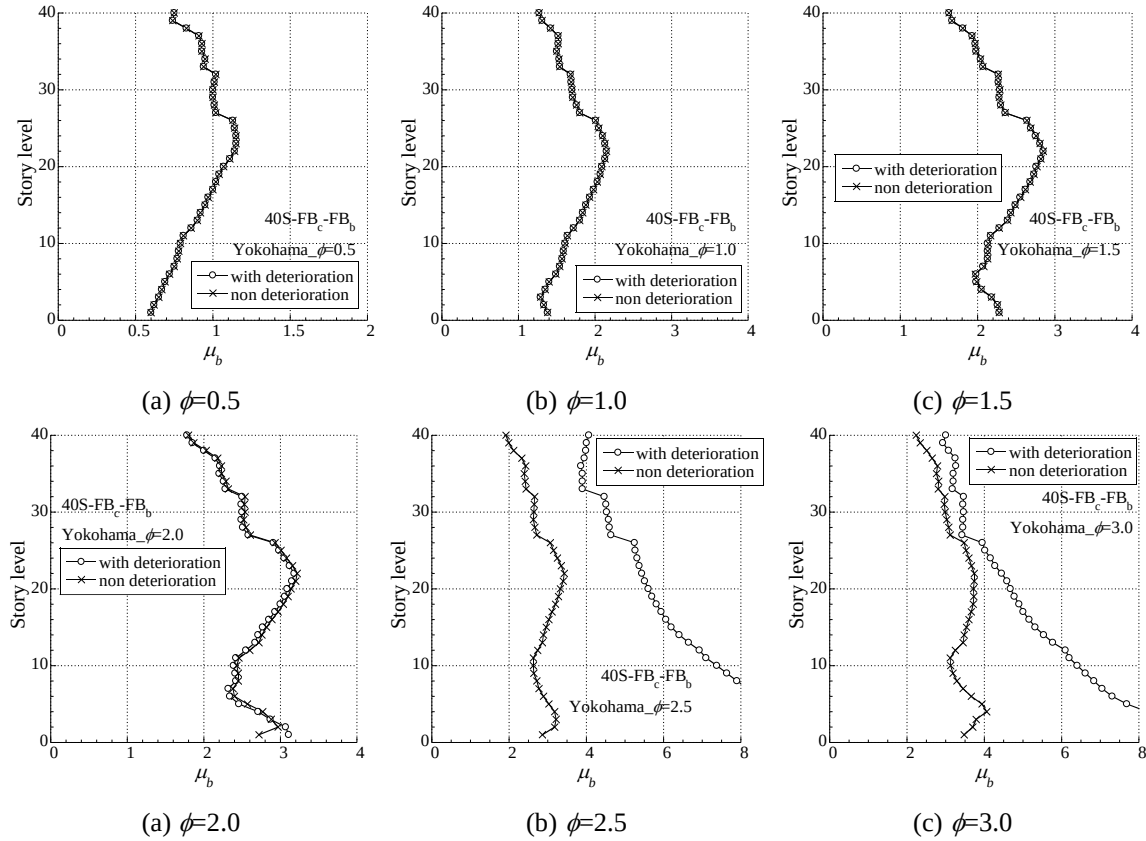


Figure 4.83 Deterioration transfer of beam ductility μ_b response of 40S-FB_c-FB_b model

The commentary in the American Society of Civil Engineers Standard 7-02 (ASCE 2002) “Minimum Design Loads for Buildings and Other Structures” describes progressive collapse as “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it”. Beside the previous conclusions on the collapse-resistant capacity derived on the basis of the instantaneous responses of high-rise buildings, the time-history response is also needed to be evaluated to explore the collapse process of such building structures under severe seismic excitations. Due to the incipient collapse of the high-rise buildings indicates the initial condition of collapse occurring, the 30S-FB_c-FB_b building model incipiently collapse under Art-Hachi wave with incremental factor $\phi=3.0$ and so as the 40S-FB_c-FB_b building model under Art-Hachi wave with incremental factor $\phi=2.0$.

Thus, the time-history story drift angle responses at the bottom-story of above building models are respectively illustrated in Figure 4.84. For the 30S-FB_c-FB_b model under Art-Hachi ($\phi=3.0$) as shown in Figure 4.84 (a), the member deterioration significantly slide the time-history story drift angle response from around 35.0 sec, and gradually glide on one side until collapse occurs (around 69.2 sec). However, for the 40S-FB_c-FB_b model under Art-Hachi ($\phi=2.0$) as shown in Figure 4.84 (b), the incipient slide occurs even later (around 59.2 sec), but the stable side-sway did not sustain for a long-time until collapse is induced (around 68.4 sec). This indicates the process of the incipient collapse of 30S-FB_c-FB_b building model shows ductile behavior to resist side-sway developing (lasting around 34.2 sec), while inductile incipient collapse of 40S-FB_c-FB_b model has been progressed only lasting around 9.2 sec.

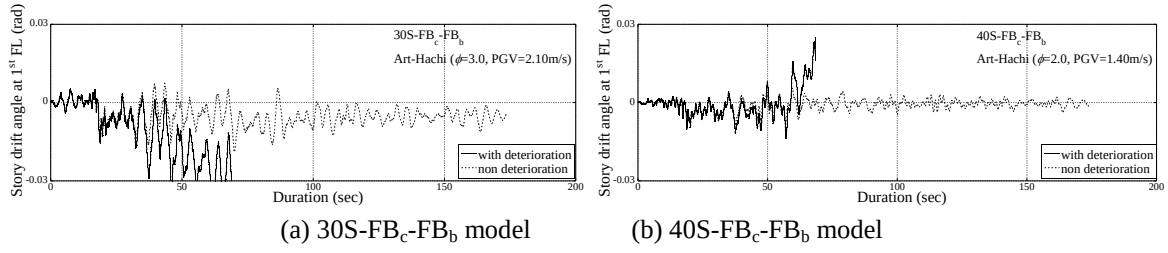


Figure 4.84 Progress of the incipient collapse of 30S/40S building models

Besides, due to Figure 4.85 only revealed the collapse process at the bottom-story of high-rise steel buildings, the details of the localization and globalization of such buildings during collapse process need to be focused. Thus, as shown in Figure 4.85, for the 40S-FB_c-FB_b building model incorporating member deterioration under severe seismic excitation (Art-Hachi with PGV=2.10m/s), the localization collapse is suddenly induced at local stories, e.g., significantly amplified seismic responses of the deteriorating model is captured.

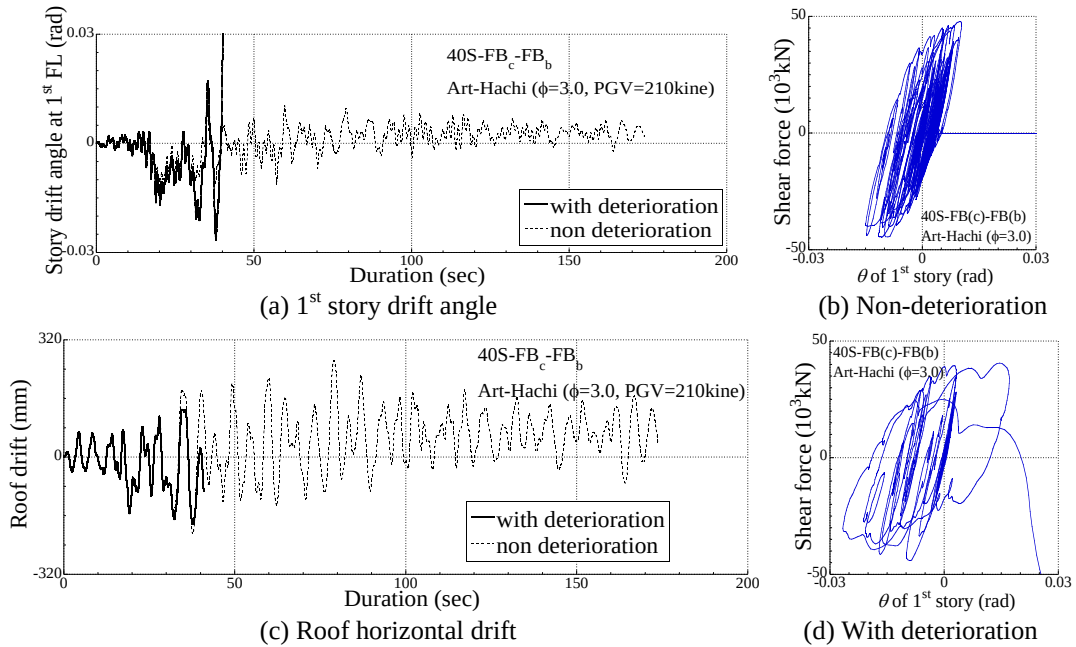


Figure 4.85 Collapse of structural system under extremely large ground motion (Art-Hachi, $\phi=3.0$);

Besides, according to the stress-strain responses of the elements that absorbed most hysteresis energy as shown in Figure 4.86, post-peak strength deterioration significantly increase the deformation (strain) to an instable condition.

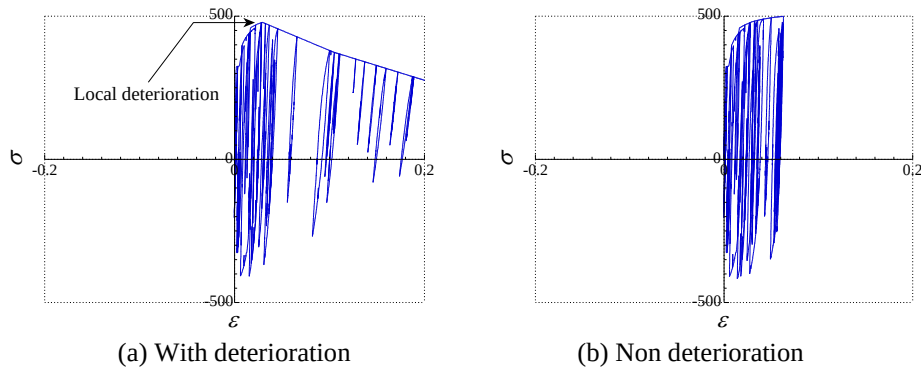


Figure 4.86 Effect of column deterioration on stress-strain relationship of 40S model

Structural collapse mechanisms including non-ductile progressive collapse and ductile side-sway collapse are significantly affected by seismic behavior (e.g., deformation capacity, ductility and hysteretic energy) of the components in structural system, i.e., high-rise steel buildings with moment resisting frame, as shown in Figure 4.87. Therefore, in this section, we specifically consider the effects of member deterioration on the collapse mechanism of high-rise steel buildings under severe earthquakes, based on IDA of artificial waves which incorporate Art-Hachi, BCJ-L2, JSCA-Kobe and also Yokohama.

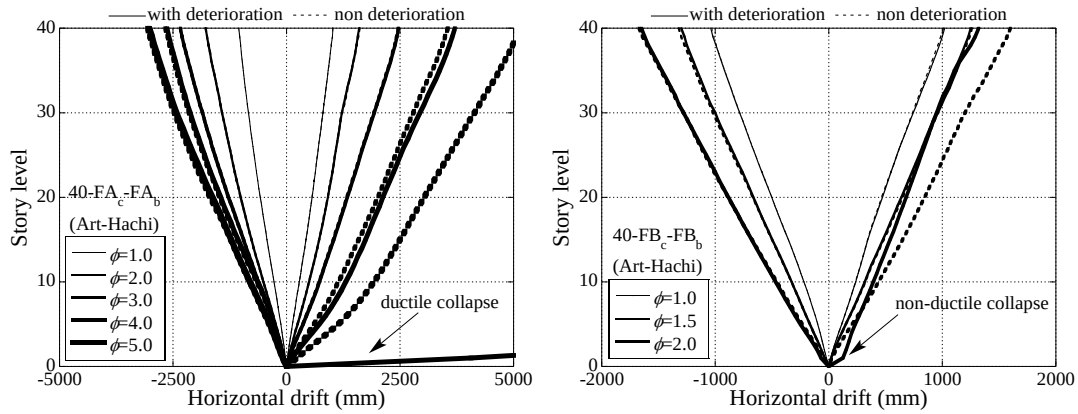


Figure 4.87 Collapse modes of various high-rise steel buildings

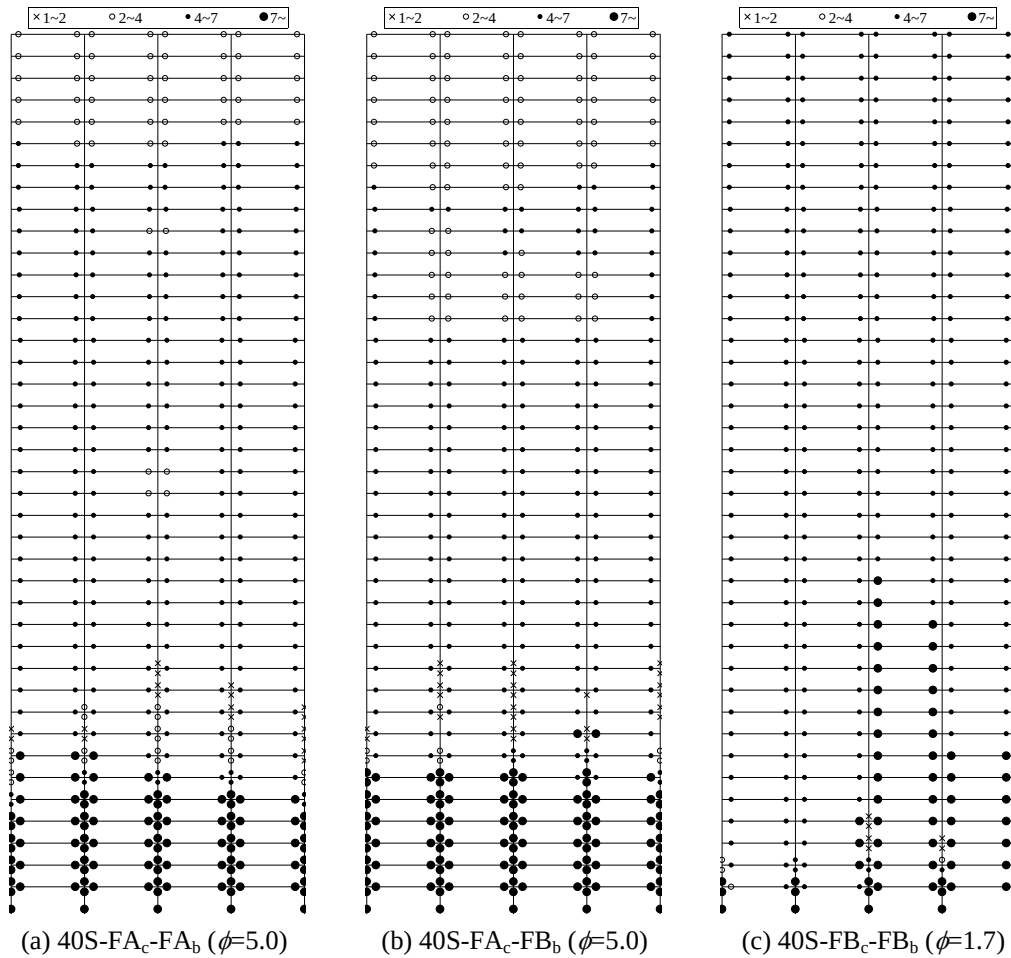


Figure 4.88 Collapse mechanisms of various 40S high-rise steel buildings (e.g., under Art-Hachi)

4.7.5 Collapse-resistant capacity

Effect of member deterioration on seismic performance of high-rise steel buildings is initiated and transferred from local region (i.e., member level and partial story) to global region (overall story), as discussed in section 4.8. Thus, in this section, in order to globally indicate the effects of member deterioration on the earthquake-resistant capacity and collapse potential of high-rise steel buildings, we further analyse the global deteriorating performance of high-rise steel buildings under large earthquake motions, by using IDAs based on various artificial waves (i.e., Art-Hachi, BCJ-L2, JSCA-Kobe and Yokohama). Such results are expected to support the instability-resistant design methodology for high-rise steel buildings incorporating the effect of member deterioration. The limit-state of collapse we assume in this chapter is that the performance indicator (e.g., maximum and residual story drift angles, μ and η) increases extensively larger than the increasing of the intensity of ground motions.

In this section, the member deterioration is induced by the local buckling of steel components, and the structural deterioration cumulated by member deterioration tend to be more and more important for the seismic-resistant capacity of high-rise steel buildings as the intensity of ground motions increases P- Δ effects. Thus, the definition for deterioration criteria, collapse criteria and dynamic instability and the corresponding ranges are proposed as follows,

- a. Non-deterioration range: in which there is no effect of member deterioration is observed;
- b. Deterioration criteria: a limit state after which the incipient collapse starts to merge;
- c. Deterioration range: member deterioration partially amplified the seismic response of building structure, and the amplified factor proportionally increases as the incremental factor of ground motions increases;
- d. Collapse range: collapse is a process that is regarded to begins from deterioration criteria to dynamic instability is induced;
- e. Instability criteria: seismic response suddenly increases to an extensive level when the intensity of ground motion increases by a little increment, e.g., the θ_{max} versus PGV curve softens to nearly horizontal direction.

Collapse risk of structural system may be induced by extensively large story drifts, which may be amplified by P- Δ effect and member deterioration. As introduced in the section 4.7, IDA method has been proved to be capable of determining the collapse-resistant capacity of building structures, e.g., IDA has been adopted by U.S. Federal Emergency Management Agency guidelines (FEMA 2000) to be the state-of-the-art method to determine the collapse capacity. Therefore, the θ_{max} versus PGV relationship curves of various building models subjected incremental seismic-excitations as shown in Figure 4.89, respectively reveal the collapse-resistant capacity of the 40S-FA_c-FA_b model and 40S-FB_c-FB_b model.

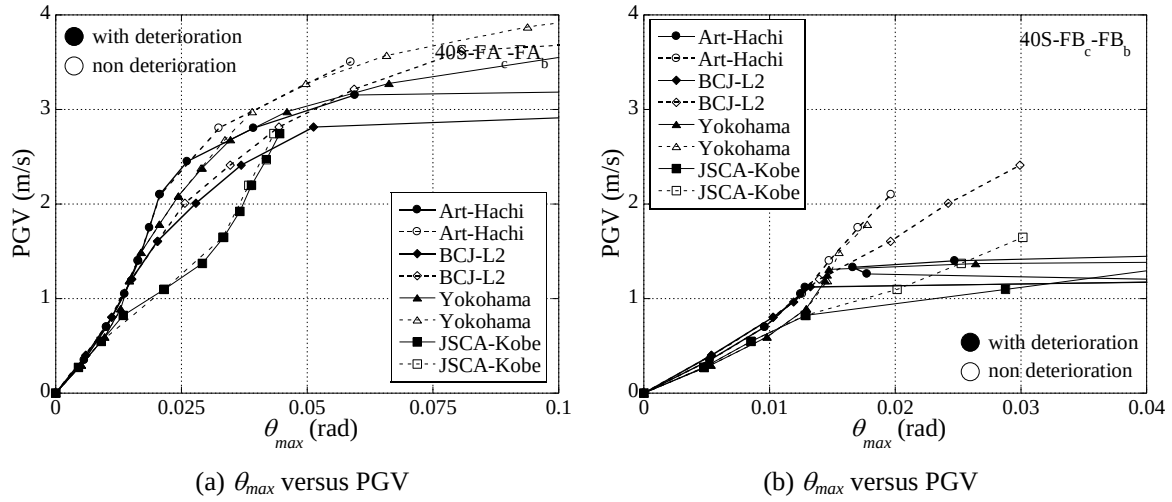


Figure 4.89 Incremental maximum story drift angles of various 40S high-rise building models

As shown in Figure 4.90, firstly, various earthquake waves induced similar incremental responses of the η of beam and column, but the aleatory uncertainty caused by earthquakes is still not avoided due to the various energy inputting of each wave. Besides, the amplified effect of member deterioration on the η_b of beam component is significantly different from that of column (η_c) from the initial range of deterioration is observed.

In particular, energy-dissipating capacity of beam members deteriorates stably, and in the case of BCJ-L2 waves [as shown in Figure 4.90 (a)], the strengthening of energy-dissipating capacity of beam is even observed, which is owing to the extreme softening (collapse) of column, as shown in Figure 4.90 (b). In contrast, η of the column is suddenly increased from initial deterioration to final collapse. This process is mainly because that the developing of column deterioration produces more cycles and larger hysteresis loops under incremental earthquake excitations. Meanwhile, η_b and η_c responses are related each other, and a lot of energy has been absorbed by column lead beam dissipate less hysteretic energy.

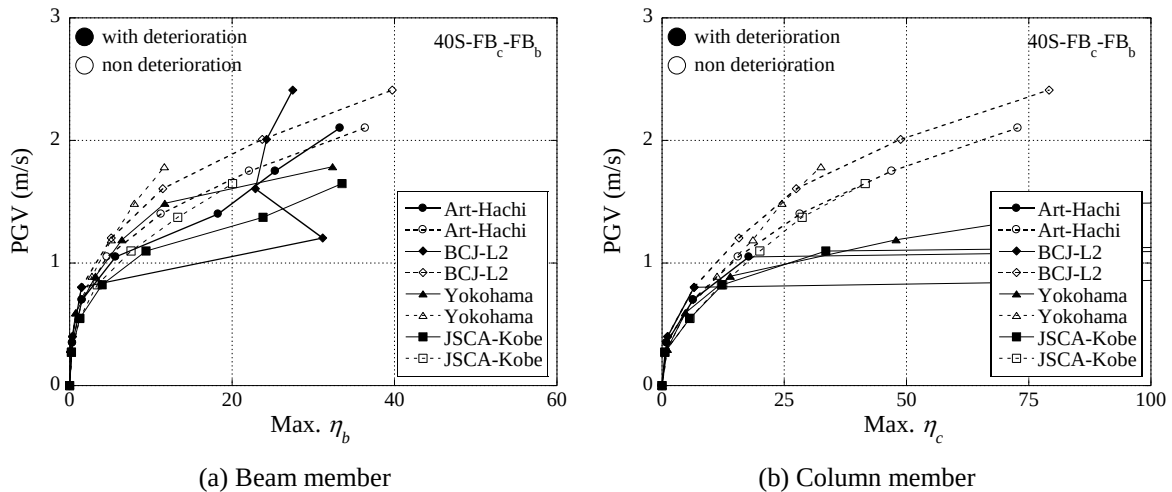


Figure 4.90 Incremental responses of maximum cumulative plastic deflection of beams and columns

4.7.6 Safety margin until collapse

The seismic-resistant capacity softens from incipient deterioration being induced, and collapse occurs if significantly increasing of seismic responses of high-rise steel buildings is obtained as a

little increasing of the intensity of ground motion. The range from incipient deterioration to collapse occurring can be treated as the safety margin until collapse. Figure 4.91 shows the safety margins until collapse of low-deteriorating and high-deteriorating building models under progressively increasing Art-Hachi.

Based on the comparisons in Figure 4.91 (a) and Figure 4.91 (b), the incipient deterioration of 40S-FA_c-FA_b building model occurs around 2.10 times higher than that of 40S-FB_c-FB_b model, and the incipient collapse 40S-FA_c-FA_b building model correspondingly occurs around 2.81 times higher than that of 40S-FB_c-FB_b building model. The safety margin until collapse of 40S-FA_c-FA_b building model is even 10.52 times larger than that of 40S-FB_c-FB_b building model.

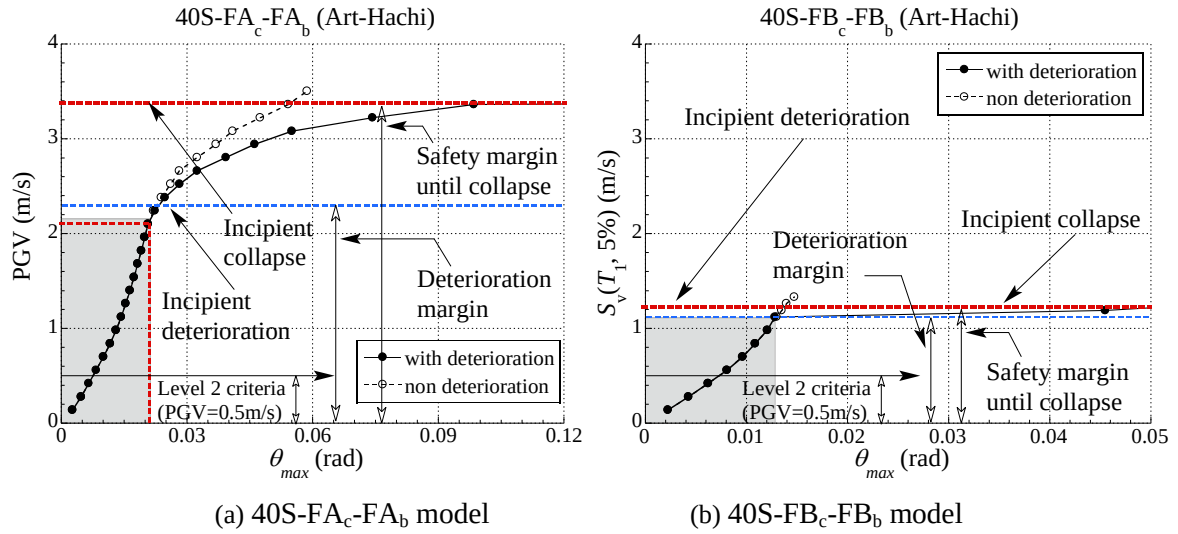


Figure 4.91 Effect of column deterioration on θ_{max} of 40S model

4.8 Conclusions

In this Chapter, the effect of member deterioration on the collapse capacity of existent high-rise steel buildings subjected to severe seismic excitations is conducted through a time-history analysis using stress fiber models consist of unified constitutive models. Herein, the characteristics and the intensity of ground motions and story levels are especially incorporated to evaluate the seismic risk of such buildings.

(1) According to the time-history analysis of various existent high-rise steel buildings with different story levels subjected to long-period ground motions, the effect of predominant period of such long-period ground motions have significant influences on the seismic responses of high-rise building models. For the existent high-rise steel buildings, the seismic safety is greatly dependent on the predominant period of ground and it should said that the corresponding synthetic ground motions should be incorporated in the assessment on the seismic safety of high-rise buildings. The seismic resistant capacity tends to be more affected by the deterioration of columns than that of girders. Due to the weak beam strong column system, the member deterioration proceeds from beam to column. Hence, even though the seismic design based on Level 1 PGV criteria and Level 2 PGV criteria is well satisfied for various existent high-rise buildings, long period ground motions induced unsatisfactory of those seismic criteria should be considered in seismic assessment and design and only PGV is not sufficient treated as the intensity criteria.

(2) Pulse-like ground motions with long-period component and short duration of vibrations excite larger seismic responses and deterioration behavior of high-rise building models. This

indicates that near-fault inland ground motions have large safety risks on those buildings and deterioration effect should be considered in the evaluation of existing high-rise steel buildings.

(3) With respect to the performance indicators and component indicators of the high-rise buildings based on incremental dynamic analysis, residual story drift angle is rarely induced when the intensity of ground motions stays at lower level. However, residual story drift angle increases and develops faster than maximum story drift angle when subjected to high-level intensity of ground motions, and eventually becomes very sensitive until collapse. Hence, how to judge the extent of residual story drift angle is important to evaluate the functionability and serviceability of the existent high-rise steel buildings after earthquakes.

(4) The local member deterioration causes to make the story shear stiffness of high-rise steel buildings softened. Even though the uncertainty of ground motion tends to affect the deterioration behavior of high-rise steel buildings, the incipient deterioration of the identical building model merges relatively close. As a matter of fact, from the deterioration behavior of such steel buildings, the effect of member deterioration starts to play a more and more important role on the seismic-resistance deterioration developing, e.g., the deterioration behavior of FB_c - FB_b building model greatly suddenly than that of

(5) P - Δ effect causes the high-rise building sway asymmetrically in one-side direction, and until instable state. In contrast, member deterioration effect further amplifies this asymmetric state and makes the building structures deteriorates and eventually collapse.

(6) After comprehensively evaluating the seismic-resistant capacity of the high-rise building models with different story levels and compactness steel members. Deterioration behavior of those building structures is significantly affected by the compactness ranks of steel members and the story level. The compact section demonstrated stable behavior after local buckling and higher story level (40S) shown most severe safety risks compared with others.

(7) The safety margin until collapse for existent high-rise steel buildings is proposed on the basis of the deterioration criteria and incipient collapse, in order to predict the degree of the collapse safety of various high-rise steel buildings subjected to incremental seismic excitations. The safety margin until collapse shown a lot more correlated to the level of member deterioration, e.g., low deterioration building models demonstrated collapse criterion around 10 times higher than that of the corresponding high deteriorating building models.

All in all, the P - Δ effect and member deterioration have been proved very significantly influence the seismic-resistant capacity and collapse behaviour of high-rise steel buildings. Those efficient factors should be rigorously considered in seismic assessment of high-rise steel buildings subjected extraordinary earthquakes.

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CHAPTER 5

EVALUATION OF SEISMIC-RESISTANT CAPACITY OF EXISTENT HIGH-RISE CONCRETE FILLED STEEL BUILDINGS

—in Comparison with High-Rise Steel Buildings—

5.1 Introduction

Since 1950s, concrete filled steel tubular (CFT) column has been studied and the design methods were developed, e.g., Klöppel's proposal (1935, 1957) for CFT columns were taken into account for German steel regulations (DIN 1050 in 1954), Naka and Kato (1961) experimentally studied the column strength of concrete casted steel tube. Thereafter, various CFT columns have been more and more frequently studying as the column in steel building structure and the pier and arch in bridge structures.

With respect to the CFT structural system (e.g., moment-frame, truss), certain studies have been conducted. Kawano and Sakino (2003) involved in the seismic resistance of CFT trusses. Kawakami et al. (2004) studied a method to improve distribution of story drift angle responses in CFT moment-resistant frames under severe earthquakes. Kawano et al. (2005) further conducted the research on seismic-resistant capacity of steel multi-story frames with CFT truss beams. Xu and Kawano (2005) revealed the dynamic stability and the elastic-plastic deformation of yielding columns of the CFT moment-resistant frames permitting partial columns to yield. Kawano and Sakino et al. (2006) addressed a study on the stress transfer capacity between steel tubes and infill concrete around beam-to-column connections of a CFT frame installed with diagonal bracing members. Tort and Hajjar (2007) proposed a performance-based design approach for rectangular concrete-filled steel tube (RCFT) frames under seismic loading. Nakahara, Sakino and Kawano (2008) conducted a research on the bond behavior of concrete filled steel tubular frames under cyclic horizontal load. Han et al. (2009) experimentally studied the seismic behavior of concrete-filled steel tubular frame to RC shear wall high-rise mixed structures, Nenavit and Hajjar (2012) addressed nonlinear seismic analysis of circular concrete-filled steel tube members and frames.

Nevertheless, the research on deterioration behavior and collapse-resistant capacity of high-rise CFT buildings, especially considering the effect of member deterioration, has not been fully involved before. Moreover, taking advantage of the opportunity to assess the seismic safety of various existing high-rise buildings subjected to long-period ground motions (AIJ 2007), the seismic performance and collapse-resistant capacity of existent high-rise CFT building is planned to be analytically investigated, and quantification of the effect of member deterioration on the deterioration behavior of such high-rise CFT building subjected to severe earthquakes is expected to be revealed as well.

According to the conclusions on the effect of member deterioration on the seismic performance of existent high-rise steel buildings (in Chapter 4), the effect of column deterioration likely to shown significant influence on the seismic damage and collapse-resistant capacity of high-rise steel buildings. Thus, in Chapter 5, we only replace the square hollow steel tubular columns in high-rise steel building with the associated square CFT columns by equivalent horizontal stiffness, in order to assess the seismic-resistant capacity of the existent high-rise CFT building that is analogous to the associated high-rise steel buildings. A comparative study on the collapse margin of such two building structures is conducted to further quantify the effect of member deterioration between square hollow steel tubular column and CFT columns on the corresponding high-rise buildings.

5.2 Statistics of High-Rise Concrete Filled Steel Tubular Buildings in Japan

As mentioned in Chapter 4, extensive existent high-rise steel buildings are constructed from 1965~2000, especially during the construction-boom (i.e., 1980s~1995s) of high-rise buildings. Furthermore, CFT columns are more adopted in high-rise buildings in Japan since 1992 (AIJ, 2007). Figure 5.1 demonstrates a statistical database (AIJ, 2007) based on 300 existent high-rise buildings higher than 100 meters in Japan, during 1966.2~1998.12. The cases of high-rise CFT buildings shows stably ascending tendency since 1981, as shown in Figure 5.1. However, according to the statistical database in section 4.2, high-rise CFT buildings higher than 80 meters are always incorporated in high-rise steel buildings. Thus the real existing high-rise CFT building cases could be more than the database as shown in Figure 5.1.

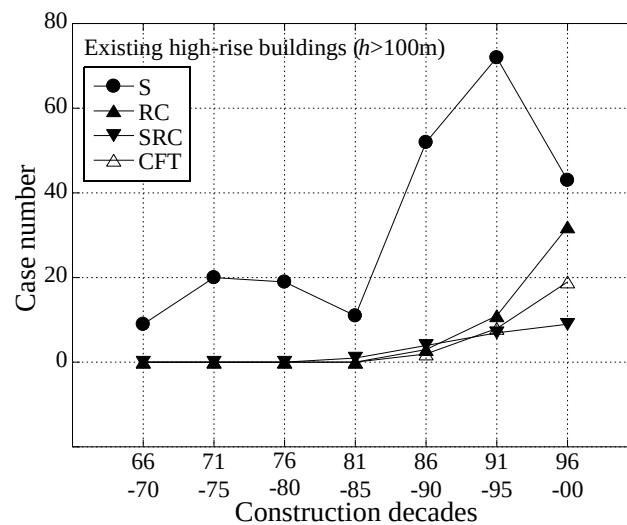


Figure 5.1 Statistical databases of various existent high-rise buildings

5.3 High-Rise Concrete Filled Steel Tubular Building Models

Based on the various statistical databases on the existent high-rise CFT buildings, increasing cases of high-rise CFT building necessitate the assessment on their collapse-resistant capacity under severe seismic excitations. Furthermore, in order to compare and quantify the seismic responses and collapse-resistant capacity of high-rise CFT buildings to analogical high-rise steel buildings, only the square hollow steel tubular column in high-rise steel building models are replaced by the square CFT column based on equivalent horizontal stiffness.

5.3.1 Equivalent square concrete filled steel tubular column

As mentioned above, the square CFT columns in high-rise CFT building are decided on the basis of the equilibrium of horizontal stiffness of analogical hollow steel tubular column. As shown in Figure 5.2, the equivalent square CFT column which is analogous to square hollow steel tubular column, can make the tubular section be more compact by keeping the same width-thickness ratio (B/t) than associated hollow steel tubular column, or save the steel material by reducing the thickness of steel tube.

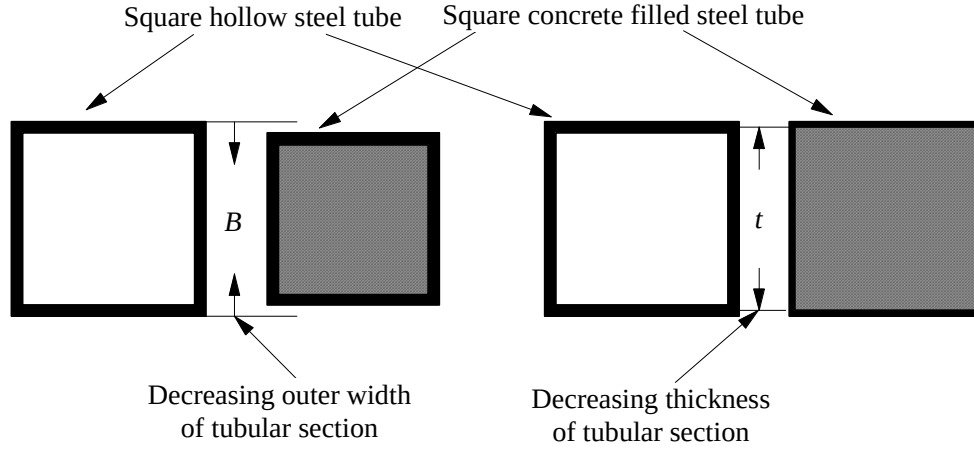


Figure 5.2 Equivalent square CFT columns with respect to square hollow steel tubular columns

Both of above advantages show different deterioration for CFT column due to the local-buckling of steel tube and confining effect to core-concrete are overtly affected by B/t of tube, as discussed in Chapter 2. Therefore, we respectively employ the compact CFT section and slender CFT section in high-rise CFT building models, in order to be analogous to FA_c - FA_b steel building model and FB_c - FB_b steel building model. Due to little effect of beam deterioration (FA_c - FB_b building model) on collapse-resistant capacity of high-rise steel building has been testified in Chapter 4, so in the case of the assessment on collapse-resistant capacity of high-rise building, we focus on collapse potentials of low-deteriorating model and high-deteriorating model.

According to the Recommendations for Design and Construction of Concrete Filled Steel Tubular Structures (AIJ 2008), being different from the ranks of the width-thickness ratio (FA , FB) on square hollow steel tubular members by Building Standards Act of Japan (MLIT 2007), the range of width-thickness ratio (B/t) of CFT members are differently required as shown in Table 5.1.

Table 5.1 Criterion for B/t of CFT member

FA	FB	FC	FD
$0.02 \leq R_u$	$0.015 \leq R_u < 0.02$	$0.01 \leq R_u < 0.015$	$R_u < 0.01$

For rectangular CFT member, the ultimate rotation angle is calculated by

$$R_u = \frac{\gamma_r}{0.15 + 3.79 \frac{N}{N_0}} \cdot \frac{t}{B} \cdot {}_R\alpha \quad (5.1)$$

where, B is the outer width of rectangular steel tube, t is thickness of steel tube, N is axial loading, N_0 is the axial yield strength of CFT section which is equal to $A_s \cdot F_y + A_c \cdot F_c$, (where, A_s is the

sectional area of steel tube, F_y is yield strength of steel, A_c is the sectional area of concrete, F_c is nominal strength of concrete), γ_r is a coefficient defined by buckling length (l_k) of CFT members, (if $l_k/D \leq 10$, $\gamma_r = 1.0$, if $l_k/D > 10$, $\gamma_r = 0.8$), and $R\alpha$ is defined by

$$R\alpha = 1.0 - \frac{F_c - 40.3}{566}, \text{ if } R\alpha \geq 1, R\alpha = 1 \quad (5.2)$$

For instant, the ranks of the width-thickness ratio of square hollow steel tubular and CFT members with the steel material (SM490 and SN490) and in-filled concrete ($F_c = 60 \text{ N/mm}^2$) are compared in Figure 5.3.

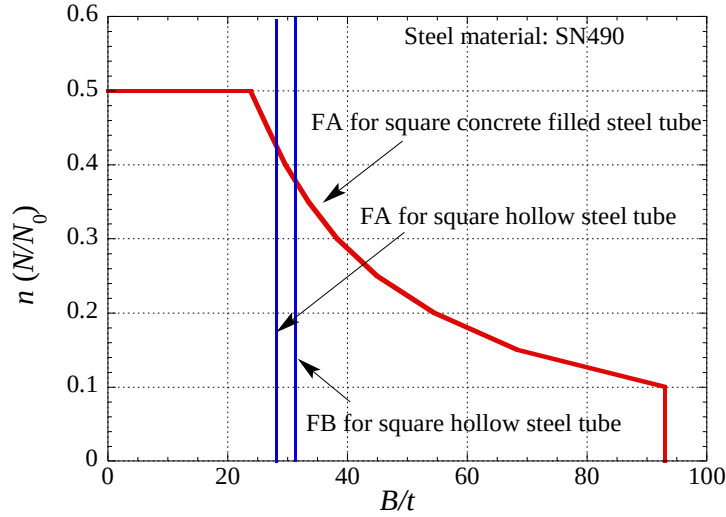


Figure 5.3 Dependence of B/t and $n (N/N_0)$ on the ranks of square HST and CFT columns

Thus, based on Eq. (5.1), Eq. (5.2), the ranges of R_u and the corresponding B/t and $n (N/N_0)$ for compact FA CFT column and slender FA CFT column are respectively shown in Appendix C. Hence, based on the relation between axial load ratio and width-to-thickness ratio (B/t) of CFT section, the compact FA CFT columns and slender FA CFT columns are selected and used in the generic high-rise CFT building models.

5.3.2 Deterioration behavior of square concrete filled steel tubular column

Occurrence conditions of the local-buckling of square CFT columns are based on width-thickness ratio (B/t), and ε_y (σ_y/E_s), according to the discussions in Chapter 2. Considering that in the realistic design of high-rise buildings, identical strength steel material is preferred to be utilized, so the yielding strain (ε_y) of steel material is not realistic to be treated as the deterioration parameter of square CFT columns. Thus, likewise to Chapter 4, B/t is also employed as the parameter to control the levels of deterioration for the square CFT columns in existent high-rise CFT buildings. A sequence of rigorous validation on the deterioration behavior of CFT structural components based on experimental database has been conducted by Bai et al. (2012).

Figure 5.4 shows the comparisons between deteriorating and non-deterioration models of the ductile and non-ductile FA CFT columns (i.e., exterior and interior columns, respectively) assumed for the corresponding columns in high-rise CFT building models. Where in Figure 5.4, significant differentials between deteriorating and non-deteriorating models for various CFT columns, specifically, the strain (ε_{lb}) at local buckling for the ductile FA CFT columns (e.g., for exterior column, $B/t=15.8$; for interior column, $B/t=12.9$) is much larger than that of the non-ductile FA

CFT columns (e.g., for exterior column, $B/t=37.2$; for interior column, $B/t=37.5$). In particular, for the ductile FA interior column, the local buckling is not even occurred before $\varepsilon=0.1$, which means the member deterioration may not be initiated in realistic structures even under severe earthquake excitation. Correspondingly, the deterioration ductility ratio (λ_{det}) that is a fraction of the strain at local buckling to the strain at the ending of local buckling by

$$D_{det} = \frac{\varepsilon_{re}}{\varepsilon_{lb}} \quad (5.3)$$

Thus, the deterioration margins of various non-ductile CFT exterior and interior columns are respectively $\lambda_{det}=3.02, 3.07$, and for that of the ductile CFT columns, the deterioration could be approximately treated to be infinite: $\lambda_{det} \sim \infty$, because the strains at local-buckling of compact CFT columns is capable to achieve very large level (e.g., for exterior CFT column, the average ε_{lb} is around 0.08) that is considered not realistic in existent engineering case.

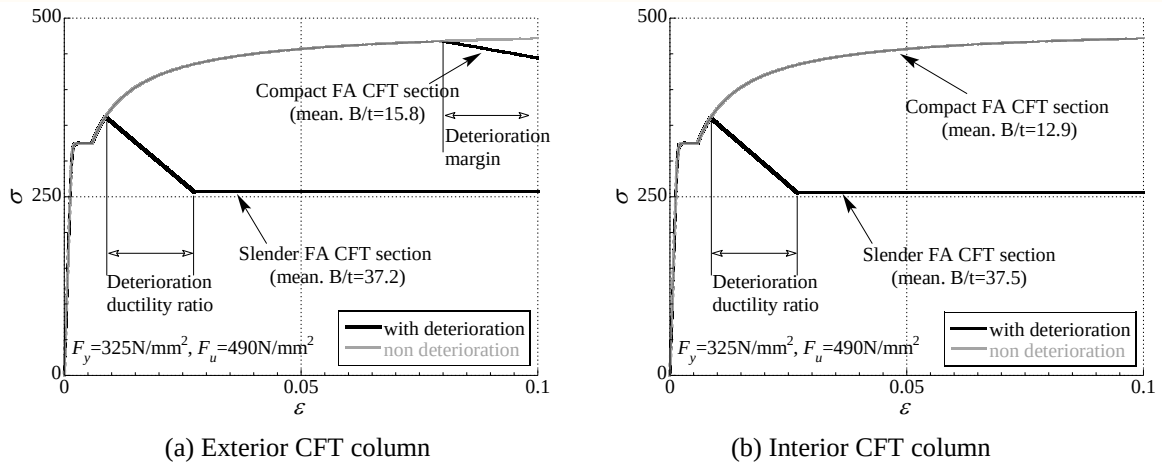


Figure 5.4 Deteriorating levels of CFT columns in 40S CFT building model

As demonstrated in Figure 5.4, the compact CFT columns are expected to show more ductile deformability to avoid the local buckling being induced than that of the slender FA CFT section. Nevertheless, comparing to the member deterioration of the corresponding hollow steel tubular columns (i.e., compact indicates FA rank column with average $B/t=18.2$; slender: FB rank column with average $B/t=29.7$), the residual stiffness (τ_{re}) of CFT columns ($\tau_{re}=0$) shows more stable behavior during residual branch than that of the hollow steel tubular column ($\tau_{re}=-0.005$). This advantage may improve the collapse-resistant capacity of the corresponding high-rise CFT buildings. According to the collapse is considered to occur when the strength of member achieves zero (e.g., stress deteriorate at the strain-axis), CFT columns only could achieve collapse within the deterioration branch due to the flat residual branch of the CFT column could not develops to strain-axis, while the hollow steel tubular column may achieve collapse state during deterioration and residual branches. Therefore, as discussed above, the collapse of CFT column occurs later and milder than that of the hollow steel tubular column based on microscopic stress-strain model.

5.3.3 Seismic design of building models

The elevation and plan view of high-rise CFT building is analogous to that of high-rise steel building, and the seismic design for existent high-rise CFT buildings is also conducted on the basis of Japanese building standard which is identical to the seismic design for existent high-rise steel buildings, as shown in Chapter 4. Table 5.2 shows the details of various high-rise CFT building

models, in which, the high-deteriorating and low-deteriorating CFT building models have close ‘first mode’ natural periods against each other. However, comparing with CFT models and the associated steel models, it is noted that the fundamental period CFT model is slightly smaller than that of the corresponding steel model. Likewise to the design of high-rise steel building models, various high-rise CFT buildings are also seismically designed on the basis of seismic design code of Japan. Details of the seismic design of high-rise CFT building models are illustrated in Appendix C.

As shown in Table 5.2, for high-rise CFT building models, C_c indicates the FA ranked square CFT columns with small width-to-thickness ratio, and S_c indicates the FA ranked square CFT columns with large width-to-thickness ratio.

Table 5.2 Parameters of high-rise CFT and analogous steel building models

CFT building	Analogous steel building	Height (story)	Square CFT column	H-shaped steel girder	T_1 of CFT building	T_1 of steel building
20CFT- C_c -FA _b	20S-FA _c -FA _b	81m	Compact-FA	FA	2.32s	2.36s
20CFT- S_c -FB _b	20S-FB _c -FB _b	(20S)	Slender-FA	FB	2.26s	2.32s
30CFT- C_c -FA _b	30S-FA _c -FA _b	121m	Compact-FA	FA	3.47s	3.61s
30CFT- S_c -FB _b	30S-FB _c -FB _b	(30S)	Slender-FA	FB	3.32s	3.54s
40CFT- C_c -FA _b	40S-FA _c -FA _b	161m	Compact-FA	FA	4.38s	4.67s
40CFT- S_c -FB _b	40S-FB _c -FB _b	(40S)	Slender-FA	FB	4.12s	4.53s

5.4 Maximum and Residual Story Drift Angles

Incremental seismic responses of the high-rise CFT buildings with various deterioration levels (high-deterioration and low-deterioration) are analytically evaluated and the effects of member deterioration on the earthquake-resistant capacity of the deteriorating and non-deteriorating models are compared against each other. Besides, in order to directly compare the CFT building model with the associated steel building model and also try to eliminate the record to record uncertainty in the range of long-period based on the ‘first mode’ spectral velocity [$S_v(T_1)$], identical flat-like artificial ground motions (i.e., Art-Hachi, BCJ-L2, JSCA-Kobe and Yokohama) are selected as input ground motion, as shown in section 4.7.

5.4.1 Responses of incremental dynamic analysis

The member (H-shaped steel girder and square CFT column) deteriorations on strength and stiffness would deteriorate the seismic-resistant capacity of high-rise CFT buildings from local regions to global responses. Thus, likewise to section 4.7, the maximum and residual story drift angles of various high-rise CFT building models by considering and ignoring member deteriorations are also compared on the basis of IDAs (Art-Hachi, BCJ-L2, Yokohama, JSCA-Kobe) method. Figures 5.5~5.8 show a typical result of maximum and residual story drift angles of various high-rise CFT building models under incremental Art-Hachi waves. Some findings through comparing various effective factors (i.e. deterioration behavior, story level) can be derived and described below.

(1) Maximum story drift angle:

(a) Story level (i.e. 20-story, 30-story and 40-story) of various building models does not significantly affect the level of maximum story drift angle responses, due to the flat spectral shape of Art-Hachi wave. However, deterioration effects on the maximum story drift angles of 30CFT- FC_c -FA_b [see Figure 5.5 (b)] and 40CFT- FC_c -FA_b [see Figure 5.5 (c)] are captured in lower stories.

In contrast, with respect to the building models with $S_c\text{-FB}_b$, the CFT columns with large width-to-thickness ratio decrease the deterioration behaviour of the corresponding building models.

(b) As shown in Figure 5.6, all of the CFT building models are likely to show deterioration behavior from $\phi=2.0$. Maximum story drift angle at bottom story of 40CFT- $S_c\text{-FB}_b$ building with deterioration is larger than that of second story [see Figure 5.6 (c)] under Art-Hachi $\phi=3.0$. It indicates that story collapse induced by deterioration effects happens from bottom story of this building model.

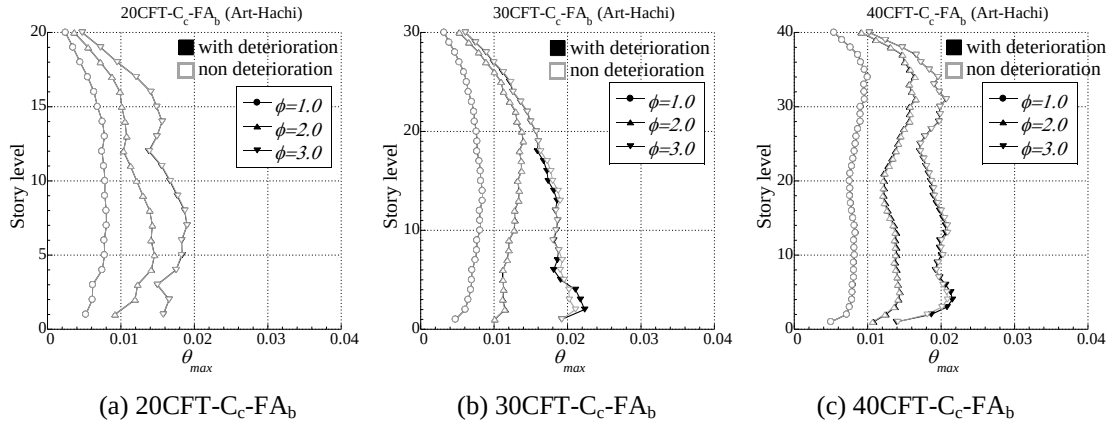


Figure 5.5 Maximum story drift angle of high-rise CFT building models with low deterioration behavior

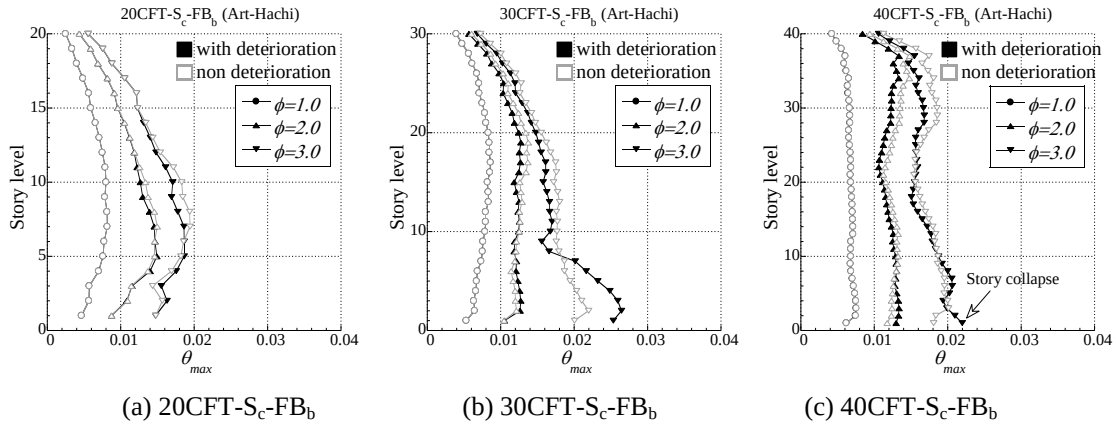


Figure 5.6 Maximum story drift angle of high-rise CFT building models with high deterioration behavior

Hence, corresponding to the 40CFT- $S_c\text{-FB}_b$ building model, the time history story drift angle of the first story of the building model with and without deterioration effect is shown in Figure 5.7. Considering member deterioration starts to be induced from an instantaneous time and since then, the asymmetric state develops prior to ignoring member deterioration.

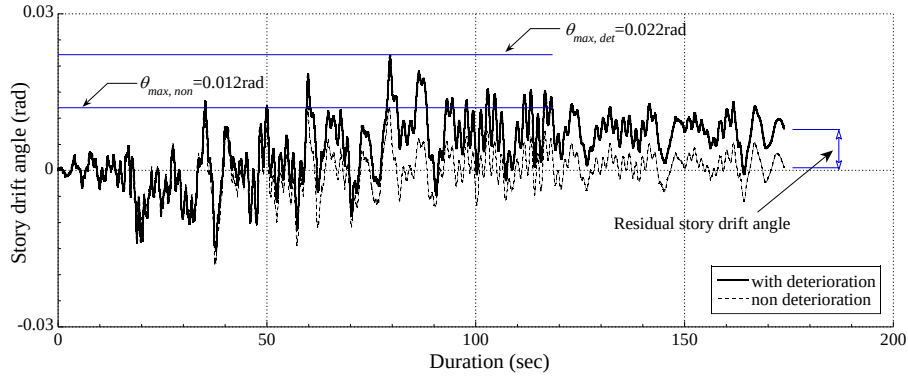


Figure 5.7 Time history story drift angle of 40CFT-S_c-FB_b model under Art-Hachi $\phi=3.0$

In order to compare with the time history seismic responses of the corresponding steel building model (40S-FB_c-FB_b) to the CFT building model (40CFT-S_c-FB_b), the incremental time history responses of the story drift angle of the first story of 40S-FB_c-FB_b building model under Art-Hachi ($\phi=1\sim3$) is demonstrated in Figure 5.8. Where in Figure 5.8, instable state is induced when subjected to Art-Hachi with $\phi=3.0$, while the analogous CFT building model can stay in stable symmetric state without collapse.

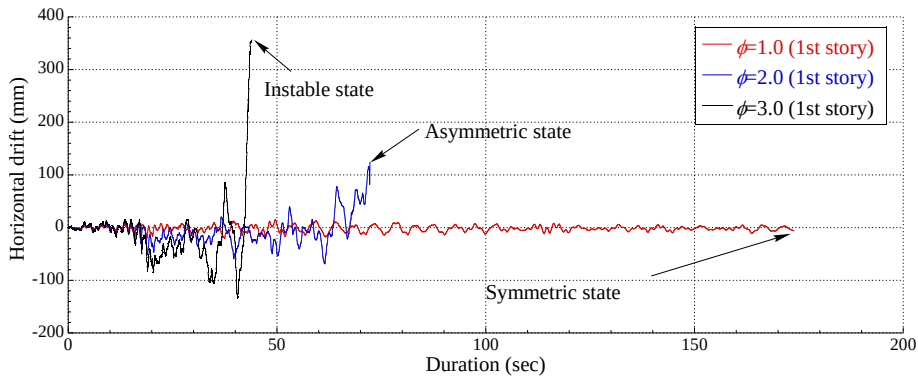


Figure 5.8 Time history story drift angle of 40S-FB_c-FB_b model under Art-Hachi $\phi=3.0$

(2) *Residual story drift angle:*

Comparing to the maximum story drift angle, residual story drift angle is induced within a low level as shown in Figures 5.9~5.10.

(a) The building models which demonstrated deterioration effect on maximum story drift angle are correspondingly affected on residual story drift angle as well. Concentration of residual story drift angle of 40CFT-C_c-FA_b building model is clearly captured as shown in Figure 5.9 (c).

(b) Story level of building model has correlation with residual story drift angle after earthquake: 40CFT building maintained largest residual story drift angle, 30CFT building models comes second and 20CFT building maintained lest residual story drift angle.

(c) Effect of member deterioration is primarily observed in high-deteriorating 20, 30, 40CFT-S_c-FB_b building model (see Figure 5.10).

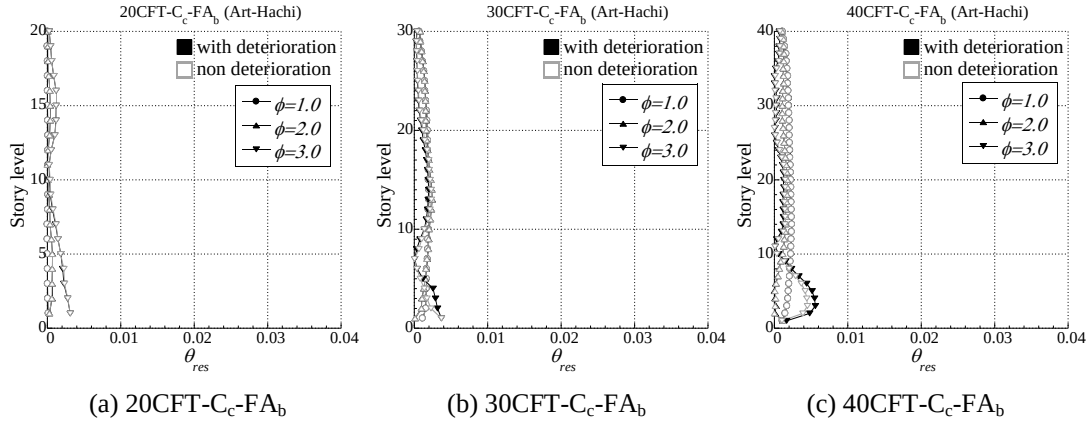


Figure 5.9 Residual story drift angle of high-rise CFT building models with low deterioration behavior

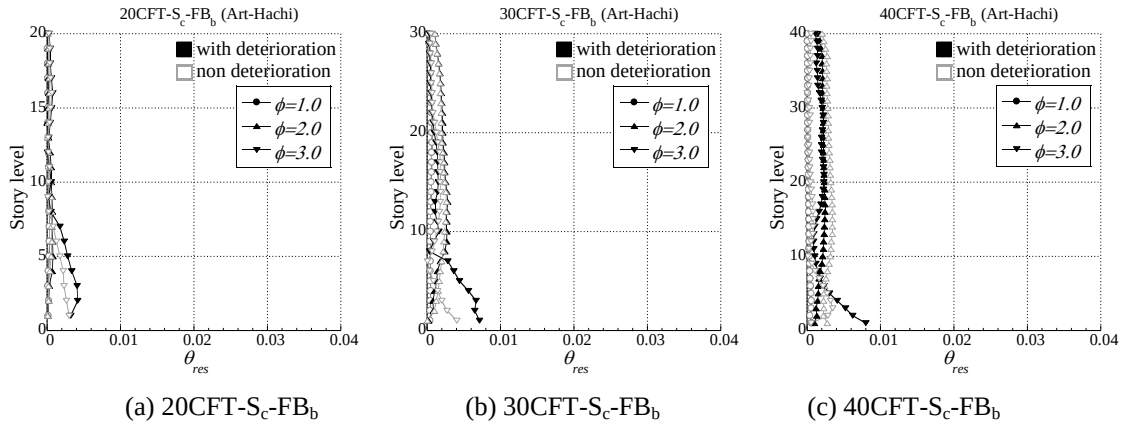
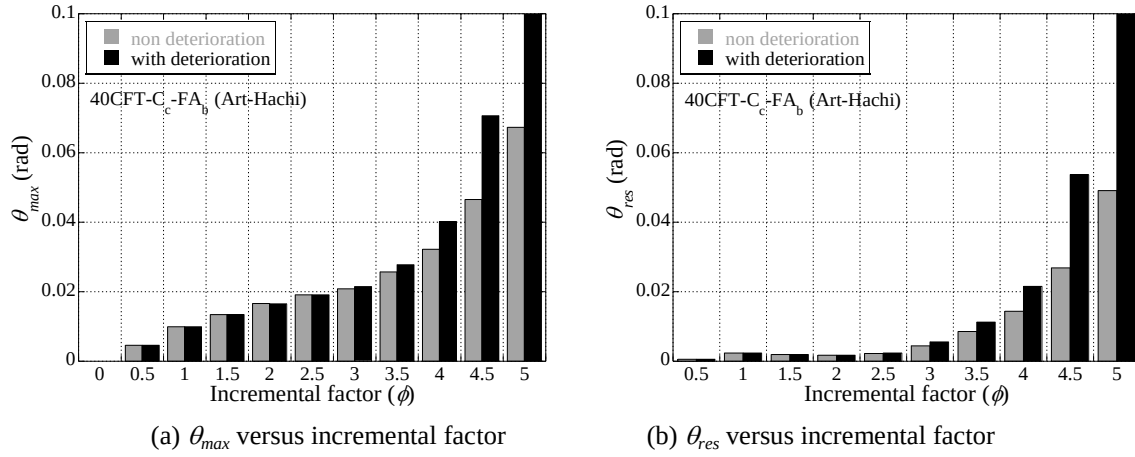


Figure 5.10 Residual story drift angle of high-rise CFT building models with high deterioration behavior

The deterioration effects on both maximum and residual story drift angle responses of various high-rise CFT buildings are gradually induced, along with the intensity of ground motion incrementally increased. Even though the member deteriorations cumulated in lower-story portion are obviously observed for high-rise CFT building, the responses of maximum and residual story drift angles are not occurred at the bottom-story while at the adjacent upper story. This phenomenon indicates that the building structure is still capable to avoid collapse occurring that should be initiated from bottom story. Overall, based on evaluation of maximum and residual story drift angle, the higher building model has the larger influence of deterioration effect on its seismic resistant capacity.

Correspondingly, the maximum and residual story drift angles are picked to globally show the proceeding of the 40CFT-C_c-FA_b building model with deterioration, compared with the 40CFT-C_c-FA_b building model without deterioration, as shown in Figure 5.11. The member deterioration effect on maximum and residual story drift angle starts to produce when intensity factor $\phi \geq 3$.



(a) θ_{max} versus incremental factor (b) θ_{res} versus incremental factor
Figure 5.11 Effect of member deterioration on incremental dynamic responses

As discussed above, the member deterioration tends to amplify the maximum and residual story drift angle responses of high-rise CFT buildings, which indicates that the local member deterioration plays an important role in global seismic performance of building structures, e.g., seismic-resistant capacity of high-rise CFT buildings tend to be reduced because of the member deterioration initiated by local failure. Therefore, a direct comparison of the maximum story drift angles of high-rise CFT building models with and without deterioration effects (see Figures 5.12~13) is conducted under various incremental earthquake excitations. The dashed diagonal is a criteria line indicating the equilibrium between deteriorating and non-deteriorating responses. Some relationship could be briefly captured according to the comparisons as shown in Figures 5.12~5.13. Where in Figures 5.12~5.13, $\theta_{max,det}$ denotes the maximum story drift angle of building model with deterioration effect; $\theta_{max,non-det}$ denotes the maximum story drift angle of building model without deterioration effect.

According to the developing of these four individual lines from equilibrium to softening branch as shown in Figures 5.12, the extent of deterioration should be proportionally ranked as Art-Hachi, BCJ-L2, Yokohama and JSCA-Kobe, which is mainly correlated to the duration of each ground motion (T) because the intensity (PGV) of ground motions is not considered in this comparison.

Besides, the incipient deteriorations of the 40CFT-FA_c-FA_b building model and 40CFT-S_c-FB_b building model under the earthquake excitations of BCJ-L2, Yokohama and JSCA-Kobe (except for Art-Hachi) demonstrate the same level ($\theta_{max,det} > 0.02$) in the range of $\theta_{max} < 0.08$ rad. However, in the case of such models under Art-Hachi with duration of 163.1sec as shown in Figure 5.12, the capacities of 40CFT-FA_c-FA_b building model and 40CFT-S_c-FB_b building model to prevent deterioration effect from being occurred are 0.021rad, 0.015rad [the 40CFT-S_c-FB_b building model shows 1.4 times higher], respectively. This priority is mainly controlled by the deterioration of the corresponding buildings, because the ‘weak-beam’ is yielded in advance comparing to the ‘strong-column’.

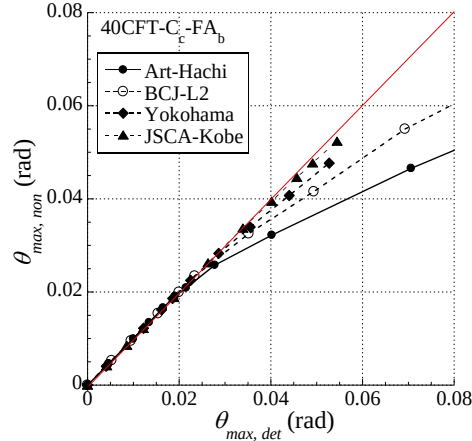


Figure 5.12 Maximum story drift angle of 40CFT-C_c-FA_b building models with high deterioration behavior

In contrast, the local deterioration of columns tends to affect the performance of various building models after incipient deterioration is induced. Moreover, 40CFT-S_c-FB_b building model subjected to Art-Hachi collapsed (e.g., $\theta_{max,det}/\theta_{max,non} \rightarrow \infty$) after a limit-state that $\theta_{max,det}=0.036\text{rad}$ divided by $\theta_{max,non}=0.026\text{rad}$ is 1.38 (see Figure 5.13, while the 40CFT-C_c-FA_b building model is still showing ductile capacity even at a state that $[\theta_{max,det} (0.72\text{rad})/\theta_{max,non} (0.46\text{rad})=1.57]$. This indicates that the priority of low level of member deterioration is mainly merged after the equilibrium is broken at the branch point, and plays a more and more important role in the softening branch in which performance deterioration and even collapse may be induced in the case of the high-rise CFT buildings with high level of member deterioration.

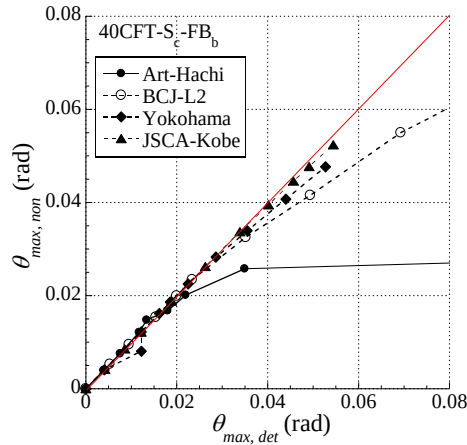


Figure 5.13 Maximum story drift angle of 40CFT-S_c-FB_b building model with high deterioration behavior

5.4.2 Correlation between maximum and residual story drift angles

In this section, the residual deformation represented by residual story drift angle indicates the permanent story drift angle remained in the high-rise building structures after earthquake excitation, which is also analogous to the residual strain after unloading at the stress-axis in the stress-strain models of the corresponding fiber elements (as shown in Figure 4.16). As we introduced previously as well as the discussions in section 4.5.3, as the important engineering indicators of the building structures subjected to earthquake excitations, the percentage of θ_{res} to θ_{max} represents the extent of permanent deformation is induced inside high-rise buildings.

In order to evaluate the collapse process of various high-rise CFT building models subjected to incremental seismic loadings, the relationship between residual story drift angle θ_{res} and maximum story drift angle θ_{max} of each high-rise CFT building models are respectively analyzed and compared with each other in the engineering indicator (maximum and residual story drift angle) versus intensity measure (PGV) relation curve based on IDA method, as shown in Figure 5.14. In which, the deterioration margins of θ_{max} and θ_{res} curves can be captured, on the basis of the comparison between deteriorating and non-deteriorating curves which are respectively demonstrated by solid-line and dashed-line with circle marks.

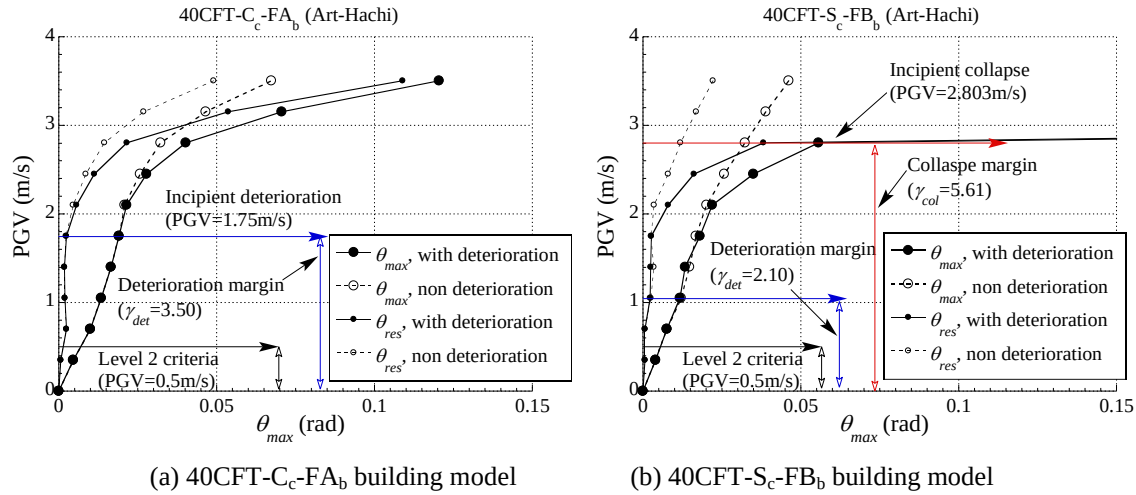


Figure 5.14 Maximum story drift angle versus PGV of various high-rise CFT buildings

According to ending points of each curve (top points at the highest PGV), amplified factor (α_{det}) of the θ_{max} and θ_{res} of 40CFT-FA_c-FA_b building model are $\theta_{max,det}/\theta_{max,non}=1.79$ and $\theta_{res,det}/\theta_{res,non}=2.22$, respectively. Moreover, the difference value (D-value) between θ_{max} and θ_{res} gradually reduces when the intensity is increased after the incipient deterioration is induced (i.e., post branch point). However, the reducing ratios of the D-value of the deteriorating and non-deteriorating models tend to show greatly different, e.g., from the branch points to the corresponding ending points with identical PGV in the case of 40CFT-C_c-FA_b building model, the $\theta_{res,det}/\theta_{max,det}$ increases from 12% to 90%, and the $\theta_{res,non}/\theta_{max,non}$ ratio increases from 0.12rad to 0.73rad. Likewise, from the branch points to the corresponding ending points of 40CFT-S_c-FB_b building model, $\theta_{res,det}/\theta_{max,det}$ increases from 18% to nearly 100%, and the $\theta_{res,non}/\theta_{max,non}$ increases from 0.18rad to 0.48rad. These findings indicate that the D-value between θ_{max} and θ_{res} of various building models with and without deterioration is significantly dependent on the collapse-resistant capacity of them, e.g., the $\theta_{res}/\theta_{max}$ ratio at the branch point.

On the other hand, based on the θ_{res} versus θ_{max} relationship curves of various high-rise CFT building models as shown in Figures 5.15~5.16, it is observed that in the range of 0.10rad of story drift angle, which is extremely large for the displacement demand of building structures, member deterioration hasn't significantly amplify the $\theta_{res}/\theta_{max}$ ratio within the same level of the story drift angle, e.g., assuming the θ_{res} is equal to 0.025rad, the associated θ_{max} values of various models (i.e. 40CFT-C_c-FA_b and 40CFT-S_c-FB_b building models) with and without deterioration, are 0.05rad, 0.048rad, as well as 0.051rad and 0.44rad, respectively. This

phenomenon indicates the member deterioration rarely soften the stiffness of θ_{res} - θ_{max} relation curves after the yield point as pointed out in Figure 5.15 (a). However, the softening of θ_{res} - θ_{max} curves incorporating member deterioration as shown in Figure 5.15 (b) and Figure 5.16 (b) tend to amplify the acceleration of θ_{res} response which is finally expected to be equivalent to the corresponding θ_{max} response (i.e., approaching the blue equilibrium line).

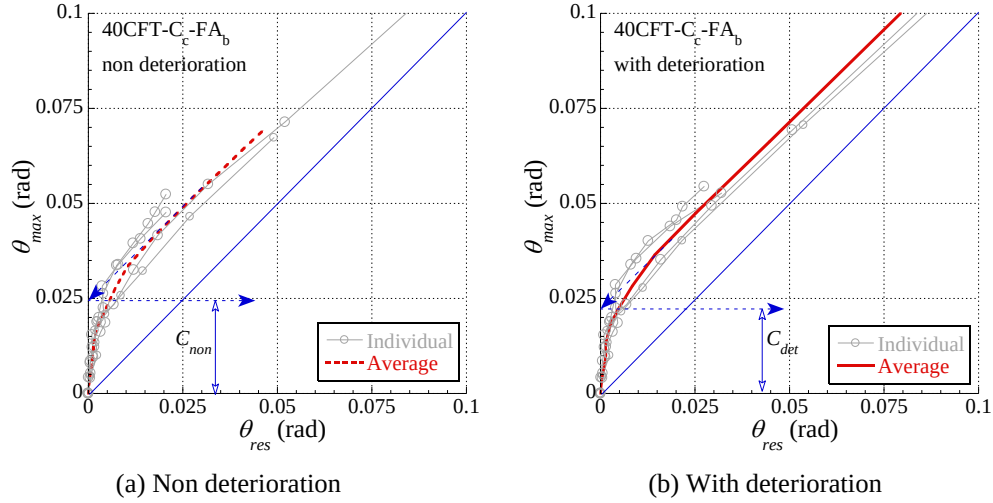


Figure 5.15 Comparisons between θ_{max} and θ_{res} of 40CFT-C_c-FA_b building model

According to the comparisons between various 40CFT-S_c-FB_b building model with and without deterioration effects as shown in Figure 5.16, on one hand, the high level of member deterioration softens θ_{res} - θ_{max} curve a little more than that of low-level of member deterioration, the 40CFT-S_c-FB_b building model will subsequently collapse earlier, the more θ_{res} - θ_{max} curve softens, the faster the level of maximum story drift angle develops. In addition, by comparing of θ_{res} versus θ_{max} relation curve of the building models with and without deterioration (see Figure 5.16), an approximate bilinear relation can be briefly captured.

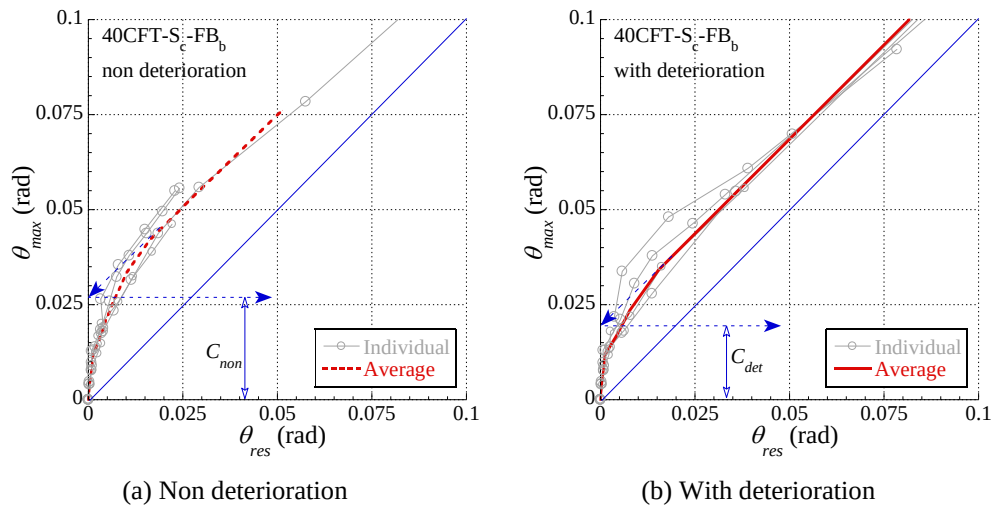


Figure 5.16 Comparisons between θ_{max} and θ_{res} of 40CFT-S_c-FB_b building model

5.4.3 Comparative assessment of concrete filled steel tubular buildings with steel buildings

CFT column always shows better seismic stability than hollow steel tubular columns in high-rise buildings (Bai et al. 2011), since that the confined effects improves the stability of CFT columns after local buckling occurring. Therefore, existing high-rise CFT building models are also analysed by considering the member deterioration of steel and confined concrete. Figures 5.17~5.18 illustrate the seismic responses of 40CFT-C_c-FA_b building model and the corresponding 40CFT-S_c-FB_b building models under incremental dynamic analysis, based on the same labels.

(1) Deterioration effect

According to the comparisons of incremental dynamic responses between S and CFT models in Figures 5.17~5.18, low-deteriorating S and CFT models (e.g., 40S-Cc-FA_b and 40CFT-C_c-FA_b) show excellent performance to resist deterioration being occurred. While, for the high-deteriorating S and CFT building models [e.g., 40S-FB_c-FB_b, 40CFT-S_c-FB_b], the member deteriorations of both S and CFT building models are captured with different effect ranges. In particular, the horizontal displacements at deterioration zone (i.e., localization of lower-story) are amplified by deterioration in high-deteriorating CFT building model under incremental dynamic loading (Art-Hachi with $\phi=2, 3$), on the other hand, sway collapse is captured in the corresponding high-deteriorating steel building model.

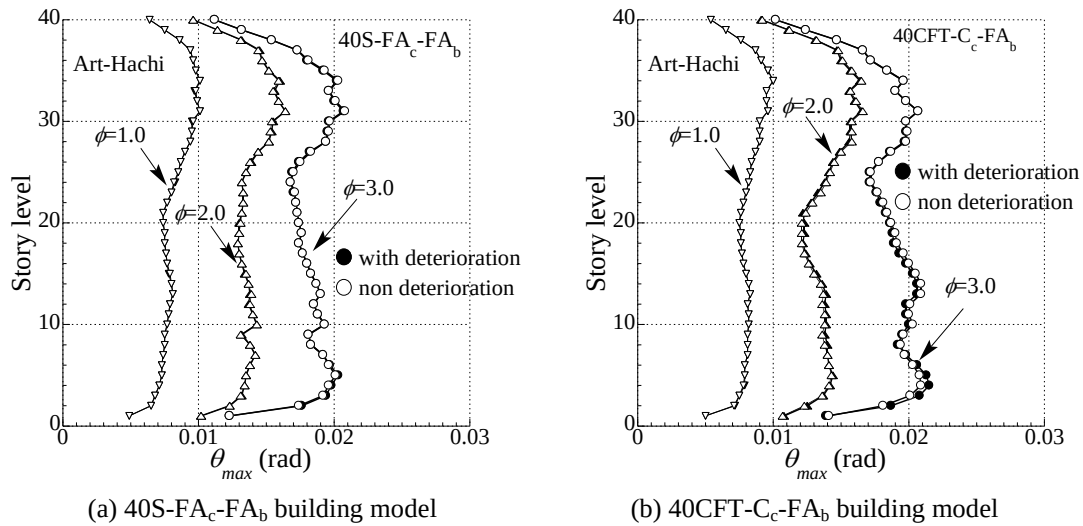


Figure 5.17 Incremental dynamic responses of low-deterioration building models under Art-Hachi ($\phi: 1\sim 3$)

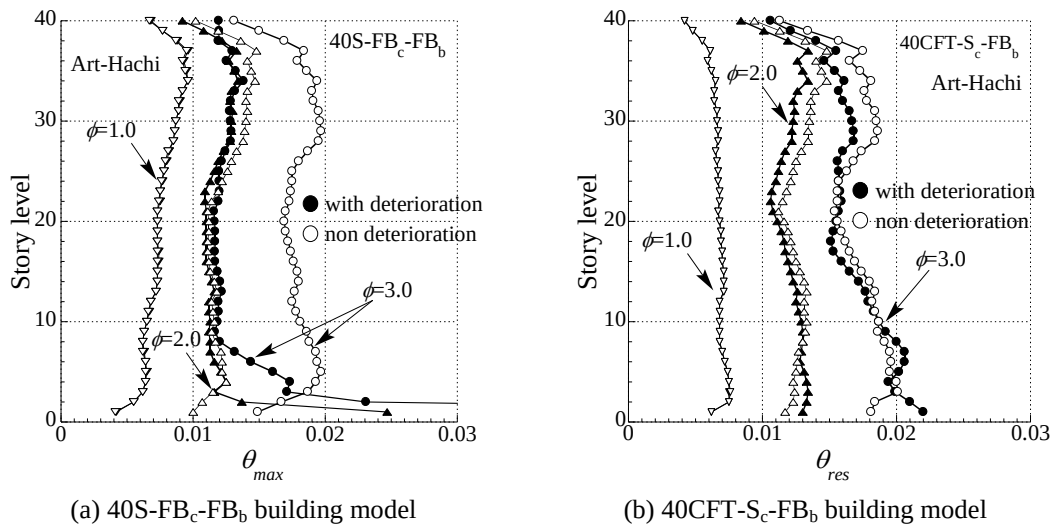


Figure 5.18 Incremental dynamic responses of high-deterioration building models under Art-Hachi ($\phi: 1\sim 3$)

(2) Earthquake effect

The comparisons of incremental dynamic responses between 40 story S and CFT models with high-level of member deterioration (FB for steel components, high-deterioration for CFT components) are shown in Figure 5.19. Capacity deterioration of S model is more obviously derived than that of CFT model, subjected to IDA with large incremental factors (ϕ). However, the uncertainty of various artificial waves has been clearly observed as well.

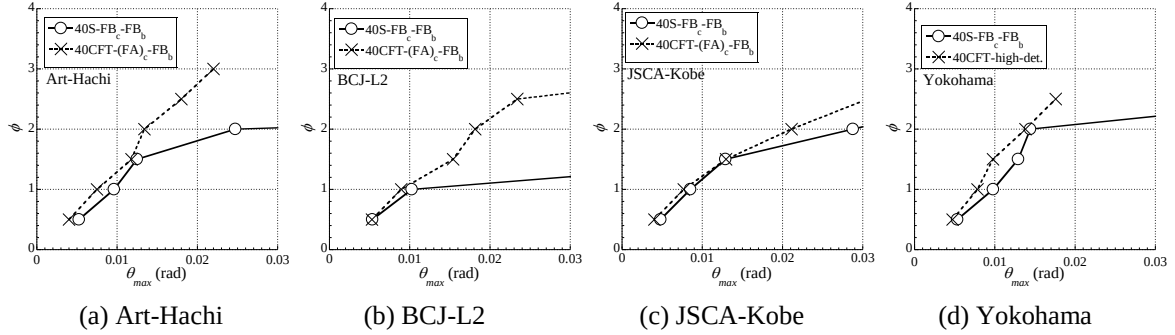


Figure 5.19 Comparisons of incremental seismic responses between S and CFT building models

(3) Height effect

Various high-rise building models (S, CFT) with 20, 30 and 40 stories behave different deterioration during the IDA process, since that higher story has larger $P-\Delta$ effects and make the strength deterioration be induced at lower stories. Figures 5.20~5.21 present the effects of deterioration on the maximum story drift angle responses of S and CFT building models with different stories (20S, 30S, 40S) under Art-Hachi. For S building models, the effects of story-level on the deterioration are captured on the basis of maximum story drift angle. The capacity of S model to resist deterioration occurring tend to descend when story-level increases (20S→30S→40S). On the contrary, CFT building models has little correlation between story-level and strength deterioration, due to better capacity of CFT columns to resist the local buckling occurring than that of square hollow steel tubular columns.

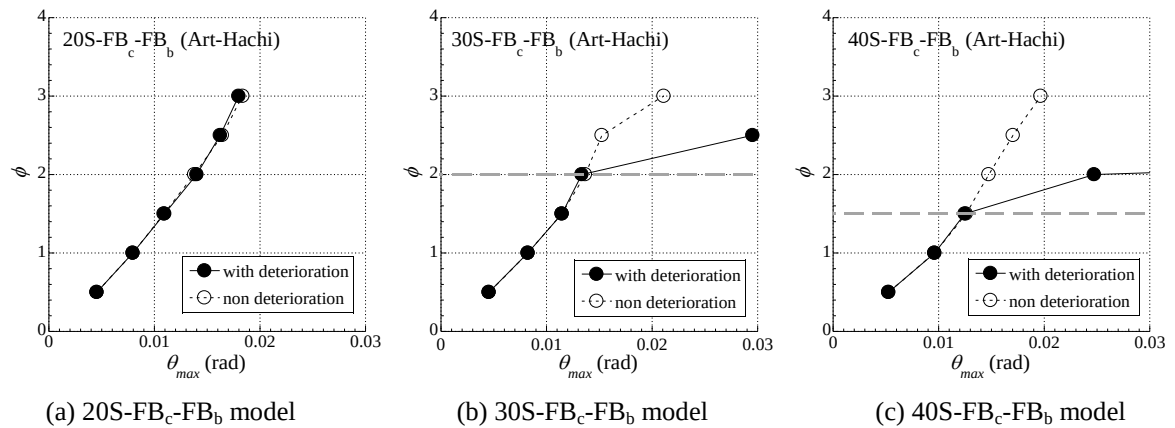


Figure 5.20 Dependence of story numbers on seismic response of various steel building models

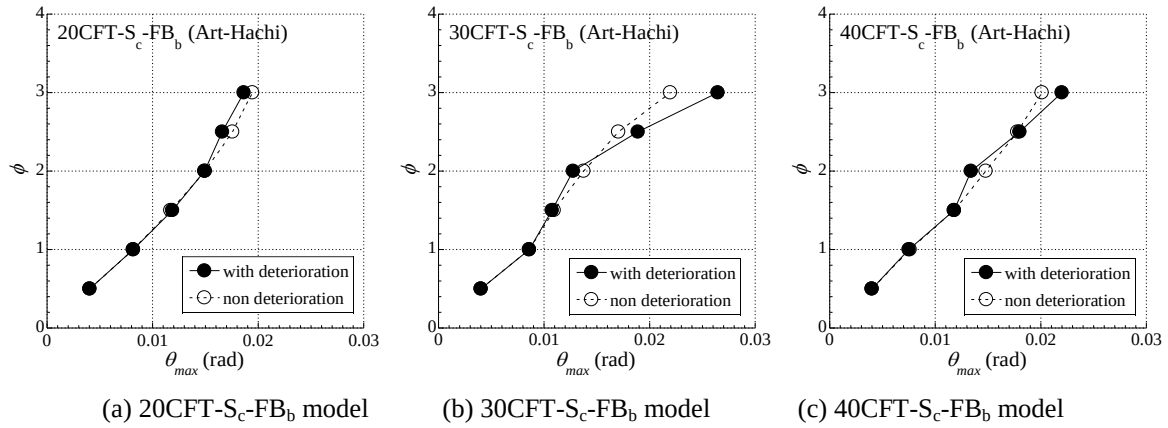


Figure 5.21 Dependence of story numbers on seismic response of various CFT building models

5.5 Plastic Deformation and Mechanism

With respect to the component damages of high-rise CFT buildings under seismic excitations, excursion and cumulative values of the deformation and energy absorption can be both treated as the indexes for evaluating the seismic structural damages of such building structures. The deformation capacity of members and connections in moment-resistant frames has been treated as an important factor for the acceptance criteria of the corresponding building structures in the Guidelines for the Seismic Rehabilitation of Buildings (1997) (FEMA 273). Mehanny and Deierlein (2001, ASCE) have proposed a seismic damage index of moment frames based on the cumulative inelastic deformations of composite beams. Besides, Park and Ang (1985) have defined the seismic structural damage as a linear combination of the maximum deformation (e.g., rotation angle for beam component) and the absorbed hysteretic energy. Therefore, in section 5.5, the maximum ductility ratio based on inelastic deformation and cumulative plastic deformation based on hysteretic energy absorption are respectively investigated to explore the effects of member deterioration on the seismic component damages of various high-rise CFT buildings.

5.5.1 Plastic deformation ratio

The plastic-hinge rotation at the ends of the beam and column members in high-rise steel/CFT buildings is an important index for evaluating the plastic-deformation capacity of local regions in high-rise buildings, as mentioned in the section 4.5.4 of Chapter 4. Considering the feature of the local-buckling of CFT column is different from that of hollow steel tubular column (in Chapter 2), the plastic deformation ratio μ of CFT column is also calculated to investigate the plastic-deformability of CFT columns after local-buckling by the section 4.5.4.

Figures 5.22~5.23 give the incremental relationship between maximum ductility ratios of beam and column components in various 40CFT building models, and the intensity of ground motions (i.e., PGV).

(a) It is obviously observed that the 40CFT-C_c-FA_b building model show stable behavior and the deterioration occurs at a high level of earthquake intensity, on the contrary, the 40CFT-S_c-FB_b building model has lower capacity not only to avoid the curve being softened but also resist the occurrence of member deterioration.

(b) There is one more significant relation that should be noted in Figure 5.22, is when the earthquake intensity is increased to initially generate the deterioration, the effect of member

deterioration would slightly decrease the plastic deformation ratio at the beam ends (μ_b) which simultaneously increases that of the column base (μ_c).

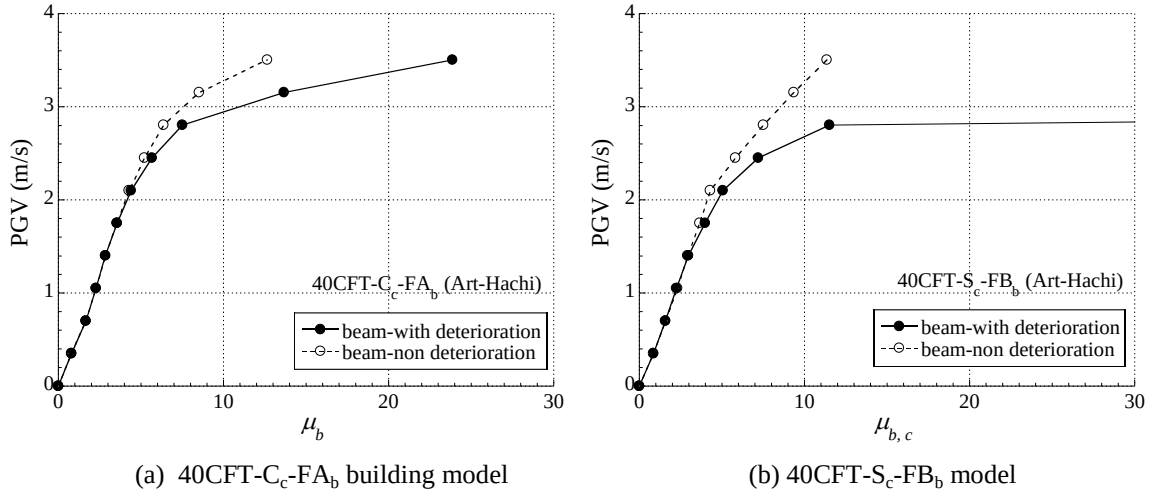


Figure 5.22 μ_b versus intensity (PGV) under incremental Art-Hachi

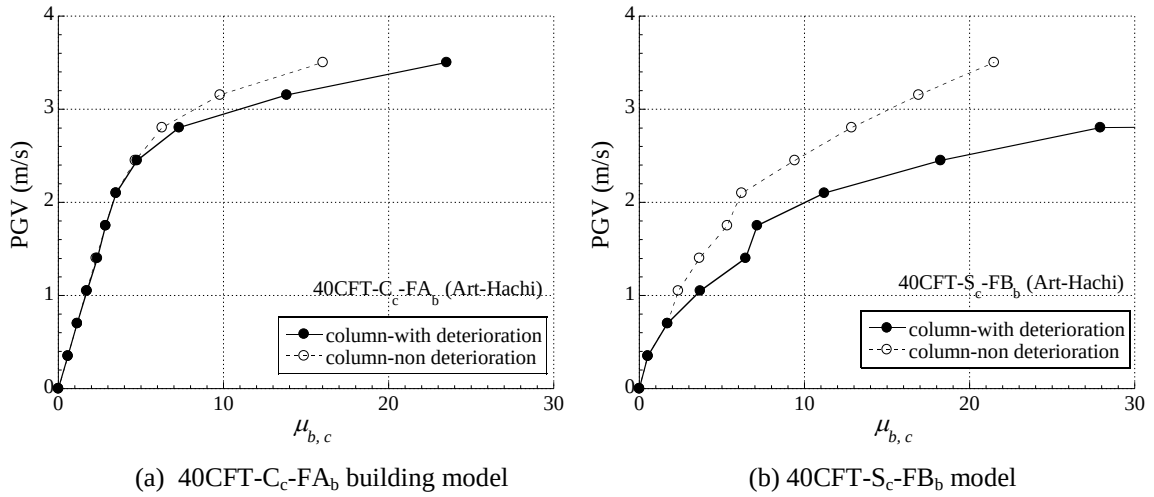
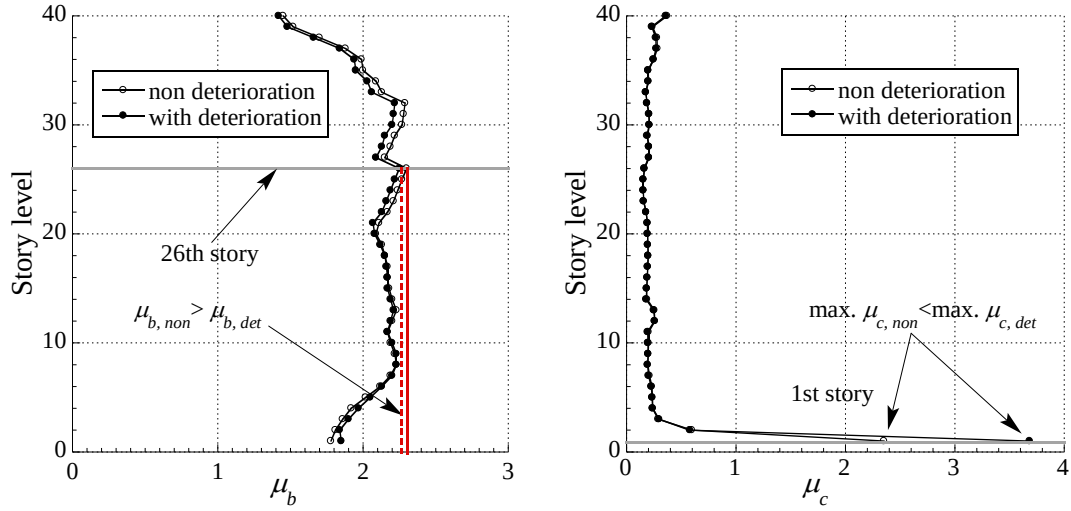


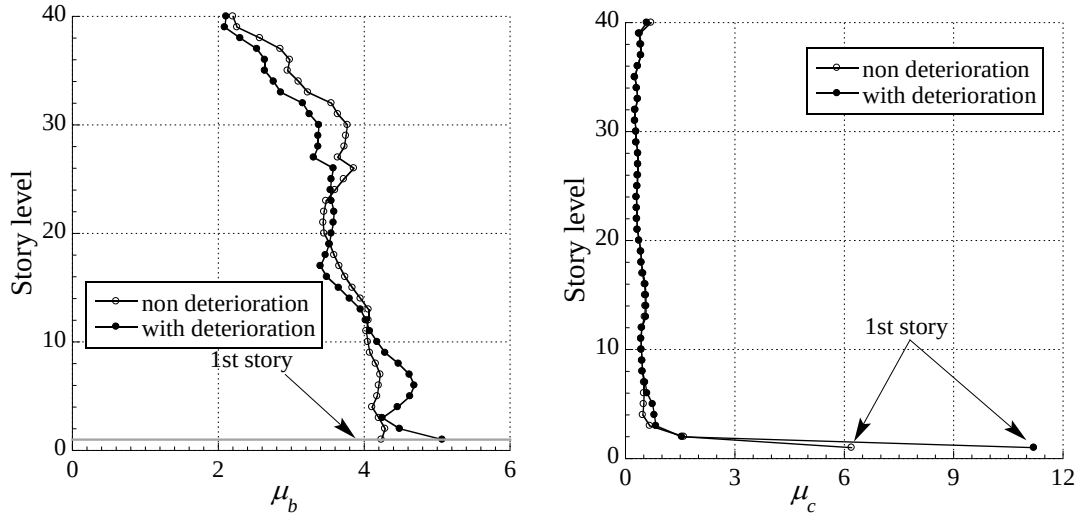
Figure 5.23 μ_c versus intensity (PGV) under incremental Art-Hachi

While, according to the comparison between Figure 5.24 (a) and Figure 5.24 (b), the plastic deformation ratios at the beam ends and column bases (μ_b , μ_c) at the lower stories incorporating member deterioration effects is larger than that ignoring member deterioration. Thus, it is noted that although the incipient deterioration did not amplify the maximum ductility ratio of beam, deterioration of beam develops at lower story faster than at upper story (as shown in Figure 5.25), and the plastic deformation ratio at the beam ends tend to be generated at lower story where member deterioration shows amplifying effect.



(a) Plastic deformation ratio at the beam end

(b) Plastic deformation ratio at the column base

Figure 5.24 Incipient deterioration of the members in 40CFT-S_c-FB_b model (Art-Hachi: PGV=1.05m/s)

(a) Plastic deformation ratio at the beam end

(b) Plastic deformation ratio at the column base

Figure 5.25 Deterioration developing of the members in 40CFT-S_c-FB_b model (Art-Hachi: PGV=2.10m/s)

5.5.2 Cumulative plastic deformation ratio

Likewise, the influences of member deterioration effect on the cumulative plastic deformation ratios at the ends of girder and column η_c , η_b of 40CFT-C_c-FA_b building model and 40CFT-S_c-FB_b building model are also assessed, to quantify the hysteretic energy dissipated inside building structures. The (η_c , η_b) of high-rise CFT buildings are calculated by

$$\begin{cases} \eta_b = \frac{W_h}{E_{u,g}}, & \text{for H-shaped steel girder} \\ \eta_c = \frac{W_h}{E_{u,c}}, & \text{for Square CFT column} \end{cases} \quad (5.6)$$

Where, W_h is the cumulative hysteretic energy dissipation at the ends of girder and/or column; $E_{e,g}$ and $E_{e,c}$ are the ultimate energy dissipating capacity of girder and column components which are calculated by

$$\begin{cases} E_{u,g} = {}_gM_u \times {}_g\theta_u, & \text{for H-shaped steel girder} \\ E_{u,c} = {}_cM_u \times {}_c\theta_u, & \text{for Square CFT column} \end{cases} \quad (5.6)$$

Where, the method to calculate the full-plastic moment (${}_gM_p$) of H-shaped steel girders in high-rise CFT building is identical to that of H-shaped steel girders in high-rise HST building. However, the ultimate moment (${}_cM_u$) of square CFT column, based on Recommendations for Design and Construction of Concrete Filled Steel Tubular Structures (AIJ 2008), is calculated by the sum of the ultimate moment of concrete (${}_cM_u$) and steel tube (${}_sM_u$) parts as

$$M_u = {}_sM_u + {}_cM_u \quad (5.7)$$

Where, ${}_cM_u$ and ${}_sM_u$ are respectively calculated by (AIJ CFT standard 2008)

$$\begin{cases} {}_sM_u = (1 - \frac{t}{B})B^2 \cdot t \cdot {}_s\sigma_y + 2(1 - x_{n1})x_{n1} \cdot {}_cB^2 \cdot t \cdot {}_s\sigma_y \\ {}_cM_u = \frac{1}{2}(1 - x_{n1})x_{n1} \cdot {}_cB^3 \cdot {}_c r_u \cdot F_c \end{cases} \quad (5.8)$$

In addition to plastic deformation ratio (μ), cumulative plastic deformation (η) is also capable of representing the member damages of various high-rise building based on hysteretic energy dissipation. Although the ultimate energy-dissipating capacity (E_u) of various ranks (FA, FB) of square hollow steel tubular and the corresponding CFT components is different to some extent, if the effect of member deterioration after peak-point show considerable influence on collapse-resistant capacity of such high-rise buildings is still not clearly. Based on the seismic responses of various elements in high-rise building models, the column-base elements at bottom story (see Figure 5.26) dissipate the largest amounts of hysteretic energy during earthquake.

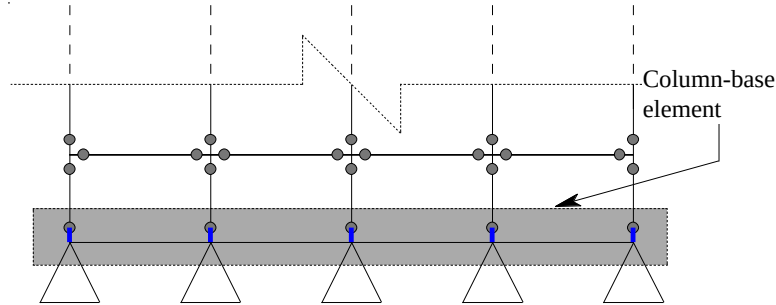


Figure 5.26 Captured column-base element at bottom story

Due to different collapse mechanisms of various high-rise steel building models is induced, as mentioned in Section 4.2, we correspondingly compare the time-history cumulative hysteretic energy absorptions of the elements where the largest η has been achieved, between deteriorating and non-deteriorating models, and between square HST and CFT columns as well. Comparisons of time-history hysteresis energy of the deteriorating and non-deteriorating steel and CFT models with high-deterioration and low-deterioration levels as above mentioned are demonstrated in Figures 5.27~5.30.

(a) According to the comparison between the steel and CFT building models with low level of deterioration level, as shown in Figures 5.27~5.28, although difference between the η accumulation of non-deteriorating steel and CFT building models are hardly observed, degrading of energy-dissipating capacity of the steel building model due to the member deterioration is gradually larger

than that of CFT building model. In addition, it is noted that extremely large η_c has been cumulated in both deteriorating and non-deteriorating models under severe earthquake (e.g., Art-Hachi, $\phi=5.0$).

(b) On the other hand, for the high-rise steel and CFT building models with high level of deterioration effects, as shown in Figures 5.29~5.30, ignoring the member deterioration is expected overestimate the energy-dissipating capacity of steel model compared with CFT building model, and non-ductile collapse of steel model is captured at the duration of 68.4sec of Art-Hachi wave ($\phi=2.0$).

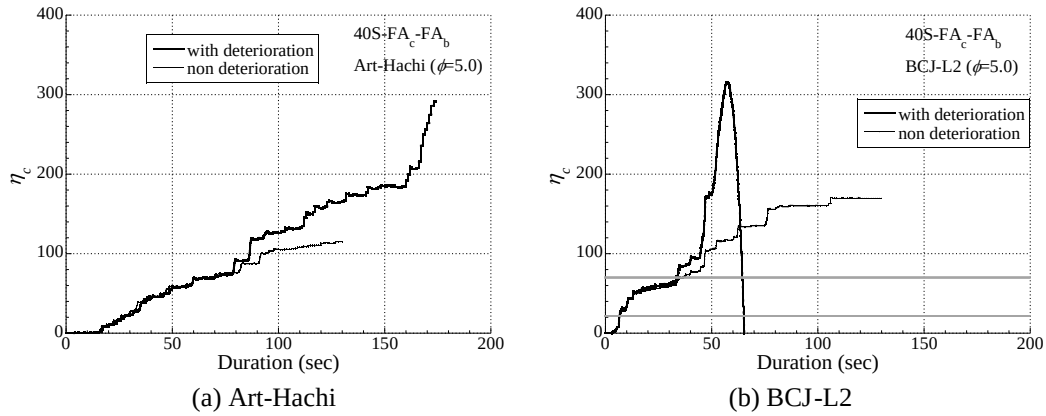


Figure 5.27 Cumulative hysteretic energy at the largest energy-dissipated element of 40S-FA_c-FA_b

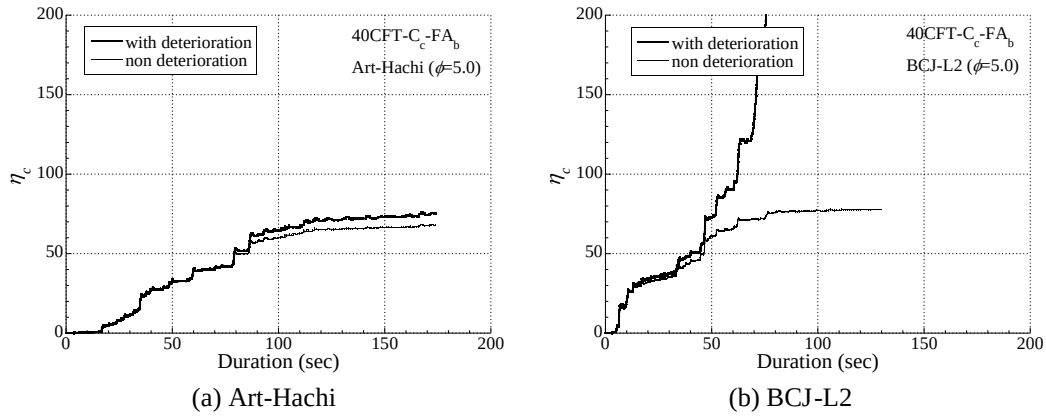


Figure 5.28 Cumulative hysteretic energy at the largest energy-dissipated element of 40CFT-C_c-FA_b

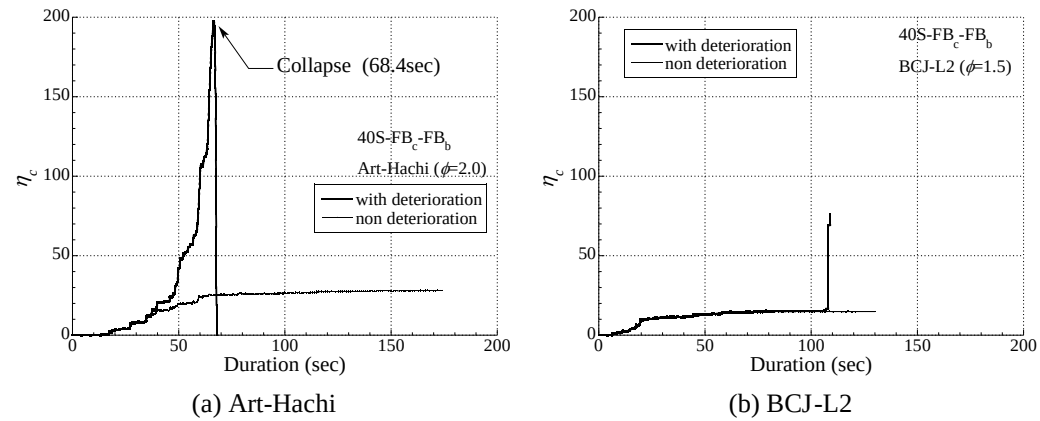


Figure 5.29 Cumulative hysteretic energy at the largest energy-dissipated element of 40S-FB_c-FB_b

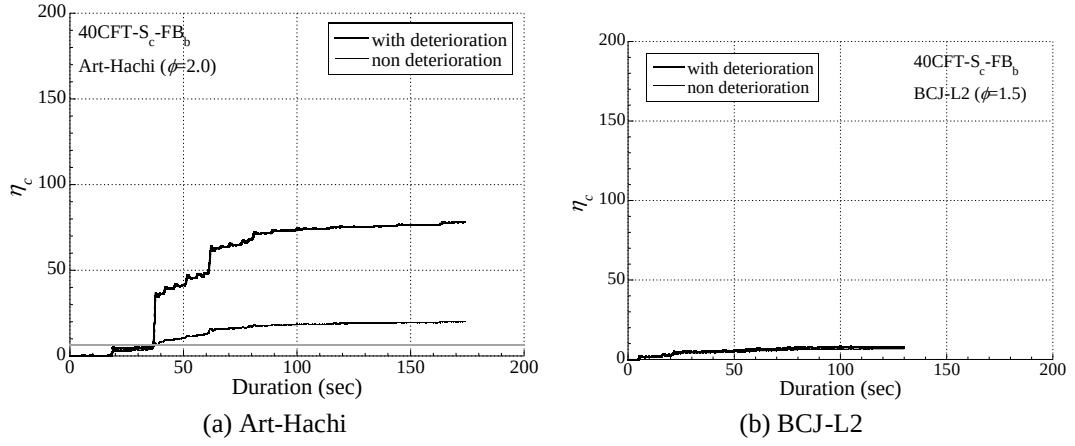


Figure 5.30 Cumulative hysteretic energy at the largest energy-dissipated element of 40CFT-S_c-FB_b

5.5.3 Plastic and collapse mechanism

Plastic hinges at the ends of girders and columns are formed and developed in high-rise building models when subjected to seismic excitation. We consider the local-buckling of steel members is the criteria of member's failure (collapse), and the collapse of frame is subsequently considered to be occurred by extensive local-failure of members at critical regions (e.g., endings of beam and column-based). Thus, member damages based on the plastic deformation ratios at the ends of girder and column are evaluated to indicate the plastic and collapse mechanism of corresponding high-rise buildings.

The plastic hinge distributions of high-rise steel and associated CFT buildings with low-det. levels (i.e., 40S-FA_c-FA_b and 40CFT-FA_c-FA_b model) subjected to Art-Hachi are respectively shown in Figures 5.31~5.32, in which, it is notable that asymmetric sway of 40S-FA_c-FA_b model is obtained in both deteriorating and non-deteriorating models, but collapse of deteriorating model mitigate the side-sway because collapsed columns has lost the capacity to resist gravity anymore. In contrast, although the 40CFT-FA_c-FA_b model also shows severe plastic damages, the seismic responses indicate that collapse of this model has not induced yet.

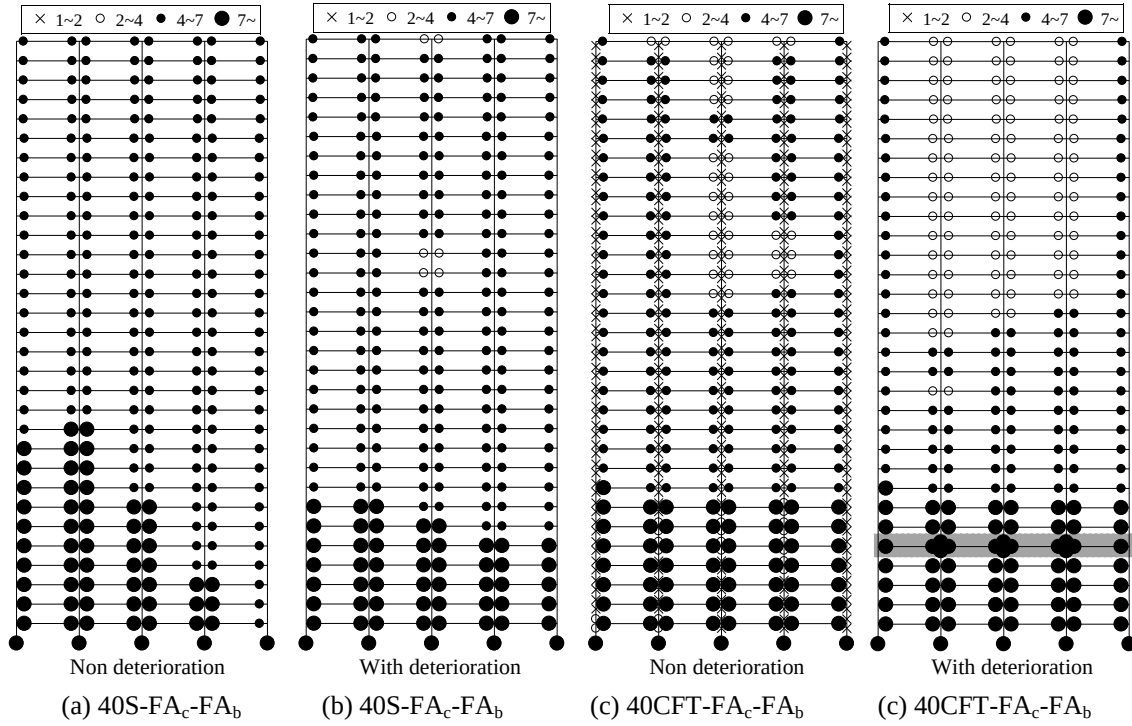


Figure 5.31 Collapse of low-det. high-rise steel and CFT buildings (Art-Hachi, $\phi=5.0$)

Contrarily, the corresponding high-det. high-rise building model (i.e., 40S-FB_c-FB_b building model and 40CFT-S_c-FB_b building model) demonstrates low capacity to resist collapse occurring (e.g., when subjected to Art-Hachi, incipient collapse of 40S-FB_c-FB_b building model begins to be induced if $\phi > 1.6$, and the incipient collapse of 40CFT-high-deterioration building model begins to be induced if $\phi > 4.5$), as shown in Figure 5.32. In particular, extremely large ductility factor μ_c of the square HST column-base caused by the column deterioration is the key reason to make the building structure collapse at bottom story. Moreover, according to the collapse mechanism of the 40CFT-S_c-FB_b building model under Art-Hachi with $\phi=5.0$ [see Figure 5.32 (d)], every girders has produced the ductility ratio larger than 7, and the columns from bottom story to the fourth story also produced high-level of plastic rotation. This notable differential indicates that CFT column effectively restrain the deformation of beam, and the finally collapse mode takes advantage of global earthquake resistant capacity of the building.

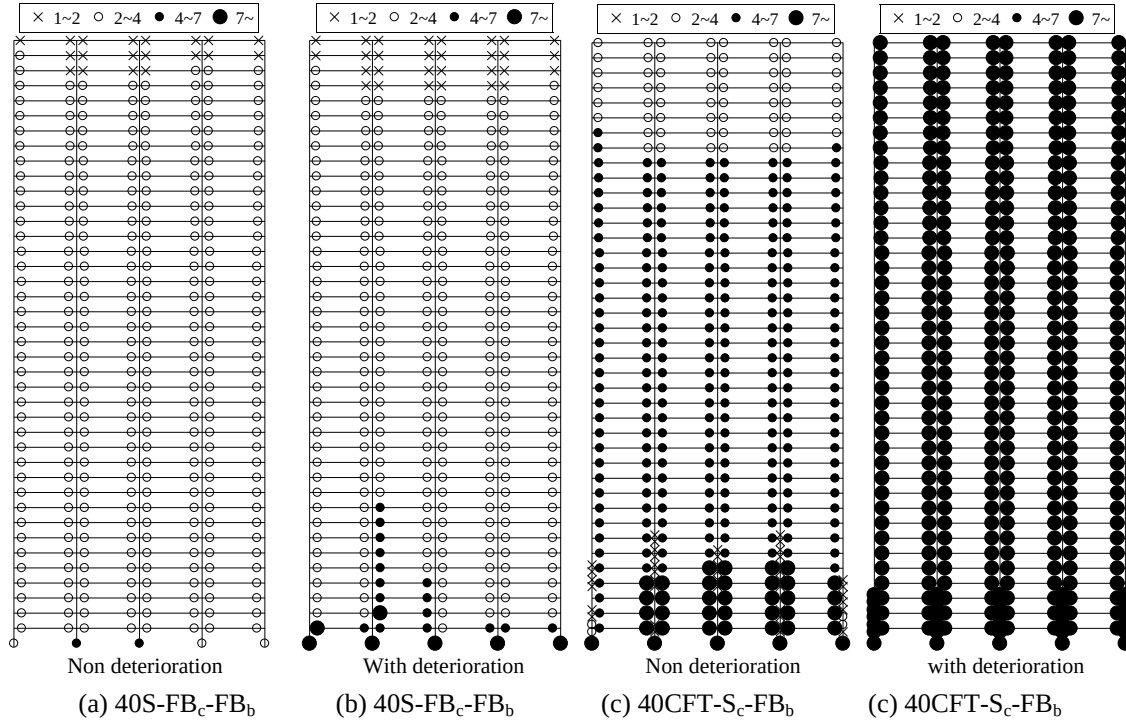


Figure 5.32 Collapse of high-det. high-rise steel and CFT building (Art-Hachi, $\phi=5.0$)

5.6 Collapse-Resistant Capacity

5.6.1 High-rise concrete filled steel tubular buildings

As discussed in section 5.4, the 40CFT-Sc-FB_b building model collapses after achieving $\theta_{max,det}=0.036\text{rad}$ and $\theta_{max,non}=0.026\text{rad}$ under incremental Art-Hachi waves. However, in order to evaluate the collapse resistant capacity of such high-rise CFT building incorporating member deterioration, subjected to incremental earthquake loadings, the intensity measure (PGV) of earthquake is needed to be included for judging the collapse-resistant capacity of the such building model. Due to the characteristics of earthquakes, seismic criteria are different from each other. 40CFT-FA_c-FA_b building model and 40CFT-Sc-FB_b building model have higher collapse resistant capacity. Residual story drift angle $\theta_{res}=0.03\text{rad}$ is regarded as a limit-state of collapse.

Thus, the relationships between the $\theta_{max}/\theta_{res}$ (rad) with PGV (m/s) of various high-rise CFT buildings are respectively demonstrated in Figures 5.33~5.34 to quantify the seismic-resistant capacity of each deteriorating high-rise CFT building models. Some points should be noted below.

(a) As for the 40CFT-C_c-FA_b building model (see Figure 5.33), only BCJ-L2 with very high intensity lead this building model collapse. JSCA-Kobe with the shortest duration induces larger maximum and residual story drift angle than other ground motions before collapse happening. Based on the limit-state of the collapse by residual story drift angle $\theta_{res}=0.03\text{rad}$, 40CFT-C_c-FA_b building model is capable of resisting collapse up to this criteria.

(b) 40CFT-Sc-FB_b building model demonstrates lower capacity to prevent deterioration and collapse happening (see Figure 5.34). When the residual story drift angle $\theta_{res}=0.03\text{rad}$, the collapse criteria can be briefly treated for this high-deteriorating CFT building. Hence, the collapse criteria should be respectively treated on the basis of the deterioration level of building structures.

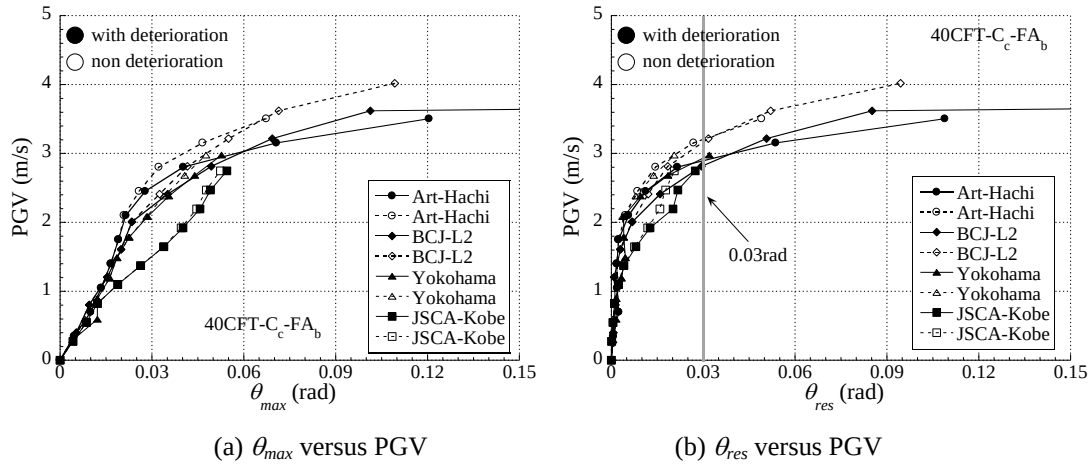


Figure 5.33 Maximum and residual story drift angle versus intensity of 40CFT-C_c-FA_b model

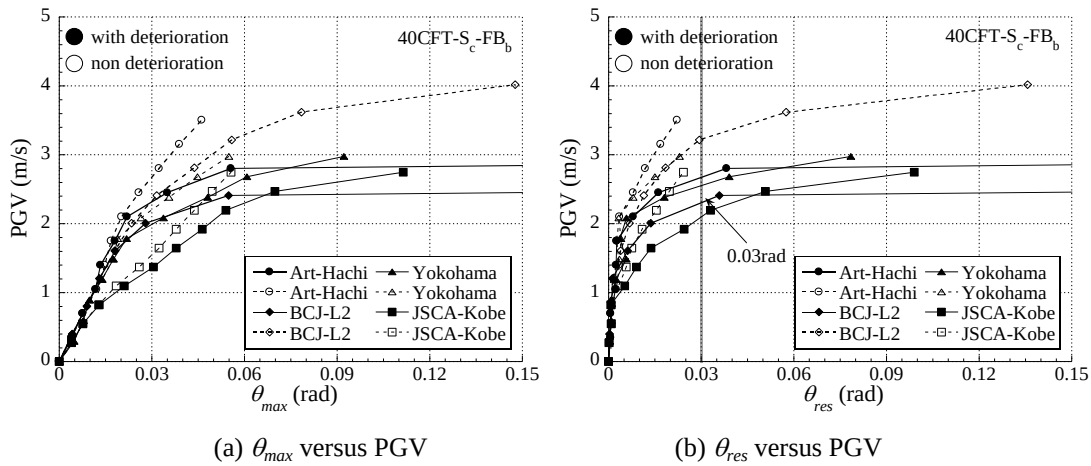


Figure 5.34 Maximum and residual story drift angle versus intensity of 40CFT-S_c-FB_b model

5.6.2 Comparative assessment of concrete filled steel tubular buildings with steel buildings

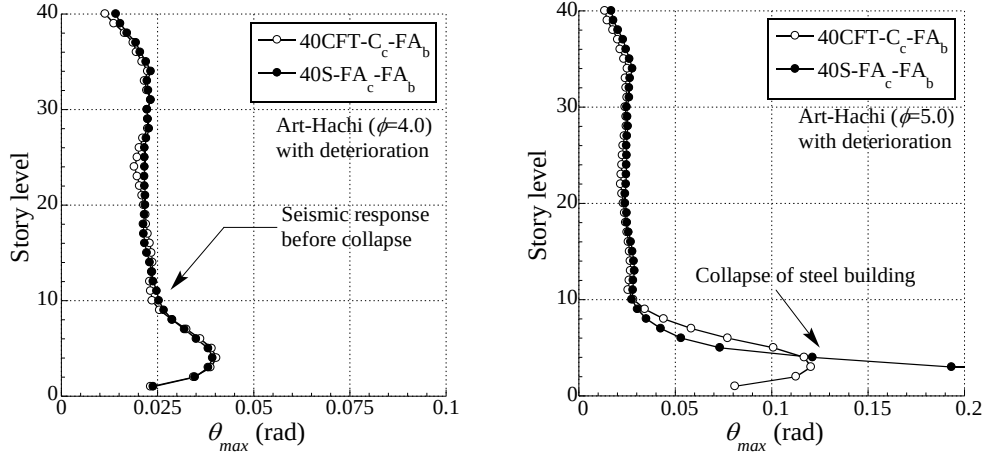
We compare the maximum story drift angle responses of various high-rise steel buildings with the corresponding high-rise CFT building models at the limit-states of the criteria before collapse and the incipient collapse, as shown in Figures 5.35~5.36.

(a) Even though both of the 40CFT-C_c-FA_b building model and 40S-FA_c-FA_b building model are capable of resisting high-intensity earthquakes (e.g., Art-Hachi: PGV=2.80m/s, $\phi=4.0$) without collapse is not because of the member deterioration, as shown in Figure 5.35(a). For the 40CFT-S_c-FB_b building model and 40S-FB_c-FB_b building model, the effect of member deterioration shows significantly sensitive on the seismic-resistant capacity of them in the range of the intensity of ground motion (e.g., Art-Hachi: PGV=0.70m/s, $\phi=1.0$), as shown in Figure 5.36(b).

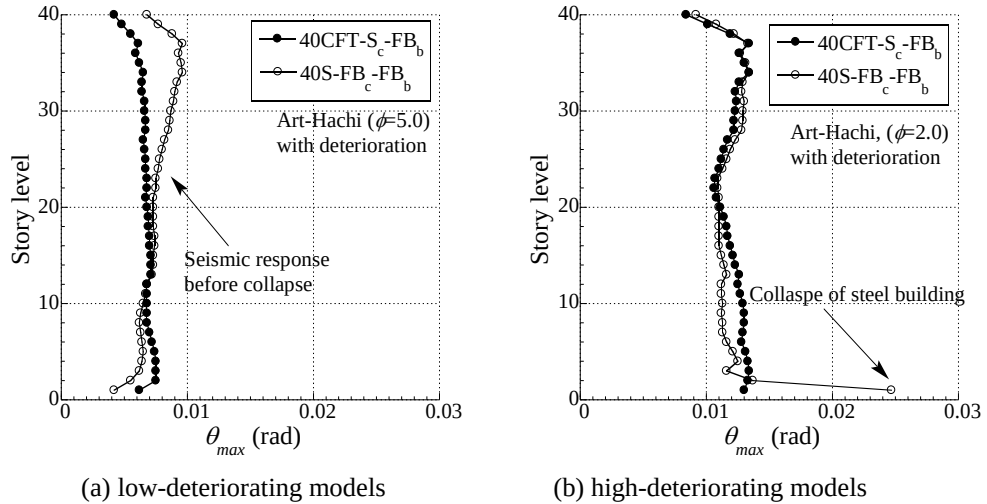
(b) Ductile global-sway collapse of low-det. S model is achieved in Figure 5.35 (b), while the corresponding CFT building model could effectively prevent collapse from being occurred. However, the case that collapse occurring of S building model is only induced under the ground motions with very large intensities that have very low probability of occurrence (e.g., Art-Hachi $\phi=5.0$). Furthermore, collapse only occurs when very large level θ_{max} (1/10) is achieved.

(c) According to Figure 5.36 (b), non-ductile story-sway collapse phenomenon of high-deteriorating S model is observed at a relatively small θ_{max} (0.01rad), subjected to incremental dynamic excitations with the intensity that has higher probability of occurrence than that for low-

deteriorating model (e.g., Art-hachi $\phi=2.0$). This implies that high-deterioration significantly influences the collapse resistant capacity of high-rise steel building models under high-level ground motions that have larger probability occurrence than that of the building model with low level of deterioration effect.



(a) Criteria before collapse (b) Incipient collapse
Figure 5.35 Comparison between low-det. high-rise steel and CFT building models

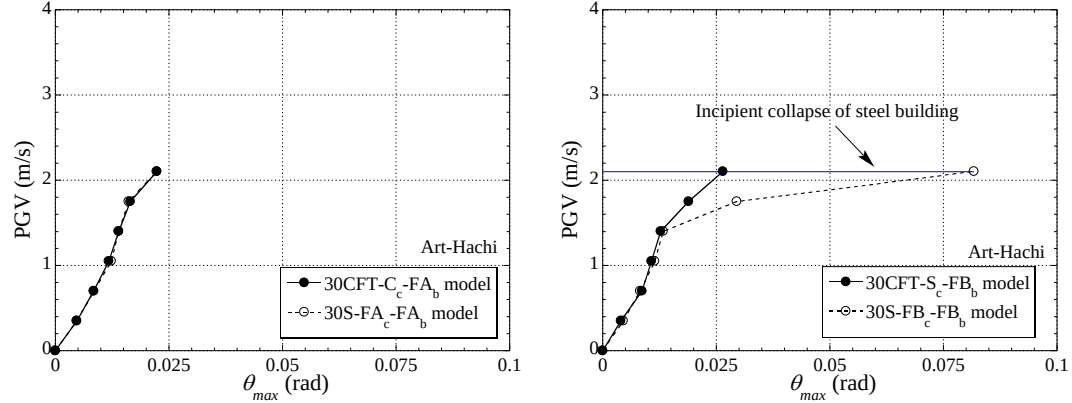


(a) low-deteriorating models (b) high-deteriorating models
Figure 5.36 Collapse capacities of steel and CFT building models

Therefore, for the seismic safety of various steel and CFT building models, steel building model faces large challenges of collapse than that of the corresponding CFT building models. Besides, steel building models with low level of deterioration effect and high-level of deterioration effect have significantly various seismic capacities to avoid collapse being occurred.

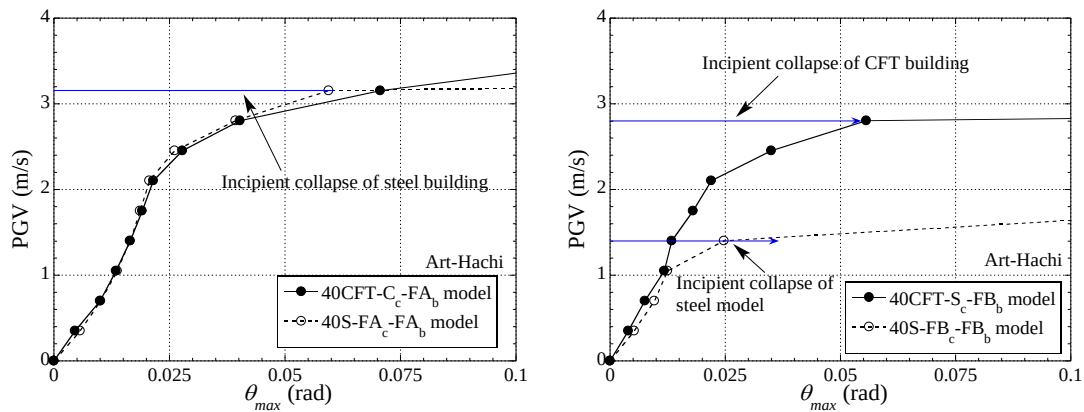
Based on the demonstration in Figures 5.37~5.38, the incipient collapse of steel and the associated CFT buildings has been induced with different intensity levels, which indicates that in the case of the equivalent steel and CFT building models with the identical lateral stiffness, various collapse-resistant capacity of them is captured. In particular, for the low-deterioration steel and CFT building models [as shown in Figure 5.37~5.38 (a)], only the steel building model (40S-FA_c-FA_b) collapses up to the range of intensity with incremental factor $\phi=5.0$. In contrast, with respect to the high-deterioration steel and CFT building models [e.g., 40S-FB_c-FB_b and 40CFT-S_c-FB_b model] in Figure 5.37~5.38 (b), even though collapses of such models have been captured in the

range of the intensity with incremental factor $\phi=5.0$, the incipient collapse of each model shows significantly different, e.g., 40S-FA_c-FA_b building model could avoid collapse until ϕ is up to around 1.5 times, while the incipient collapse of 40CFT-S_c-FB_b building model did not occurs until ϕ achieves around 4.0 times.



(a) Low-deteriorating steel and CFT buildings

(b) High-deteriorating steel and CFT buildings

Figure 5.37 Collapse-resistant capacities of 30-story high-rise steel and CFT buildings

(a) Low-deteriorating steel and CFT buildings

(b) High-deteriorating steel and CFT buildings

Figure 5.38 Collapse-resistant capacities of 40-story high-rise steel and CFT buildings

5.7 Safety Margin until Collapse and Deformability Margin until Collapse

The existent high-rise steel and CFT buildings show differential of collapse-resistant capacities, as discussed in section 5.3.3, due to influence parameters e.g., member deterioration, structural height, earthquake characteristics and structural system, etc. Collapse-resistant capacity is considered as the maximum capacity of the building structures to prevent collapse from being occurred. Even for the low-deteriorating high-rise steel buildings (e.g., 40S-FA_c-FA_b) and the corresponding high-rise CFT buildings (e.g., 40CFT-C_c-FA_b) that demonstrate ductile seismic-resistant capacity (e.g., deformability, ductility and energy-dissipating capacity), seismic response tend to become sensitive and collapse may occur suddenly after beyond the limit-state of their collapse-resistant capacities. Even for the high-deteriorating high-rise steel buildings (e.g., 40S-FB_c-FB_b) and the corresponding high-rise CFT buildings (e.g., 40CFT-S_c-FB_b), non-ductile collapse happens in the range of small increasing of PGV.

Thus, in order to derive a collapse safety coefficient which is capable of indicating the seismic safety of building to prevent deterioration and collapse from being induced, safety margin until collapse and deformation capacity until collapse of various high-rise buildings are expected to be quantified on the basis of considering the corresponding member deterioration by IDA method.

5.7.1 Safety margin until collapse

As discussed above in section 5.4~5.6, various building structures experience a process that from elasticity to elasto-plasticity until ultimate peak-point to performance deterioration (incorporating local-deterioration and global-deterioration), and finally collapse. The intensity of hazard (e.g., PGV for earthquake hazard) is considered to be associated with the corresponding performance demand in performance-based seismic design, e.g., so called ‘Level 2’ or ‘Level 1’ seismic criterion are treated as the intensities of input ground motions which respectively have various performance demands in seismic design. Hence the deterioration margin and the safety margin until collapse of high-rise buildings based on intensity measure can be defined by

$$\lambda_{det} = \frac{PGV_{det}}{PGV_{Level2}} \quad (5.4)$$

Where, λ_{det} denotes deterioration margin which is a fraction of PGV where incipient deterioration occurs to the PGV (0.5m/s) of Level 2 criteria.

$$\lambda_{col} = \frac{PGV_{col}}{PGV_{Level2}} \quad (5.5)$$

Where, λ_{col} denotes the safety margin until collapse which is a fraction of PGV where incipient collapse occurs to the PGV (0.5m/s) of Level 2 criteria.

Thus, in order to reveal the safety margin until collapse of various high-rise buildings under earthquake excitations based of the intensity of ground motions, the relations between intensity measure (e.g., PGV) and performance parameter (e.g., θ_{max} , θ_{res} , and component plastic deformation ratio μ) of various high-rise steel and CFT buildings are respectively demonstrated in Figures 5.39~5.40.

(a) The average collapse margin λ_{col} of 40CFT-FA_c-FA_b building model is larger than 6.62 (see Figure 5.39), because this building model is capable of sustaining the intensities of ground motions (Art-Hachi, BCJ-L2, Yokohama and JSCA-Kobe) up to the incremental factor $\phi=5$ without collapse occurring. In case of 40CFT-C_c-FA_b building model under Art-Hachi wave, both of the deterioration margin is approximately $\lambda_{det}=3.50$, as shown in Figure 5.39(a). No collapse is captured in the 40CFT-C_c-FA_b building models with and without deterioration effect.

(b) With respect to the collapse margin of 40CFT-S_c-FB_b building model (in Figure 5.40), relatively smaller collapse margin is captured comparing to that of 40CFT-C_c-FA_b building model. Correspondingly, the deterioration margin and safety margin until collapse of 40CFT-S_c-FB_b building model are $\lambda_{det}=2.10$ and $\lambda_{col}=5.61$ [see Figure 5.40(a)], respectively.

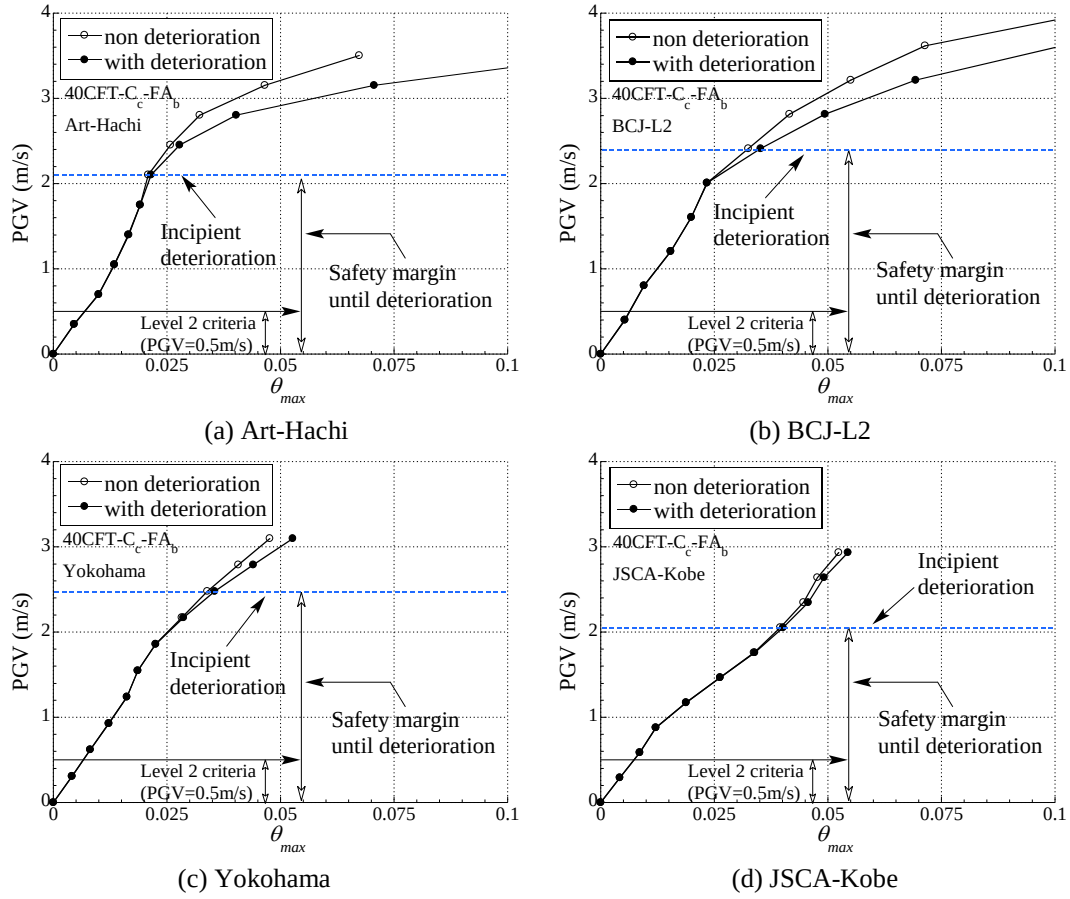


Figure 5.39 Safety margin until collapse of the 40CFT-C_c-FA_b building with low-deterioration

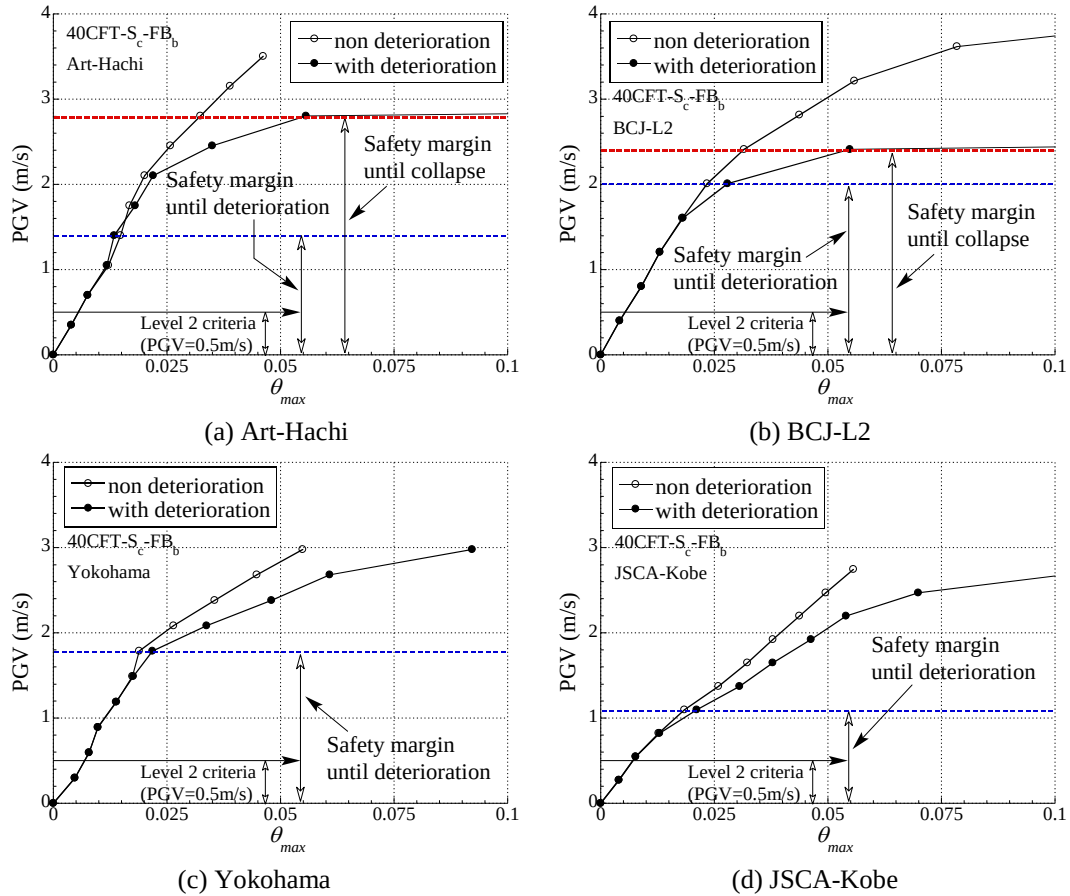


Figure 5.40 Safety margin until collapse of the 40CFT-S_c-FB_b building with high-deterioration

Hence, the safety margin until deterioration and collapse of the existent high-rise CFT buildings can be assessed on the basis of the proposed method. Furthermore, the member deterioration effect should be incorporated to evaluate the seismic vulnerability under extraordinary earthquakes.

5.7.2 Deformability margin until collapse

Corresponding to the collapse margin which is decided on the basis of the intensity of ground motions, the deterioration margin is proposed in the domain of the performance parameter of building structures.

The deformability margins until deterioration and collapse of high-rise buildings based on performance indicator (θ_{max}) and residual story drift angle (θ_{res}) can be defined by

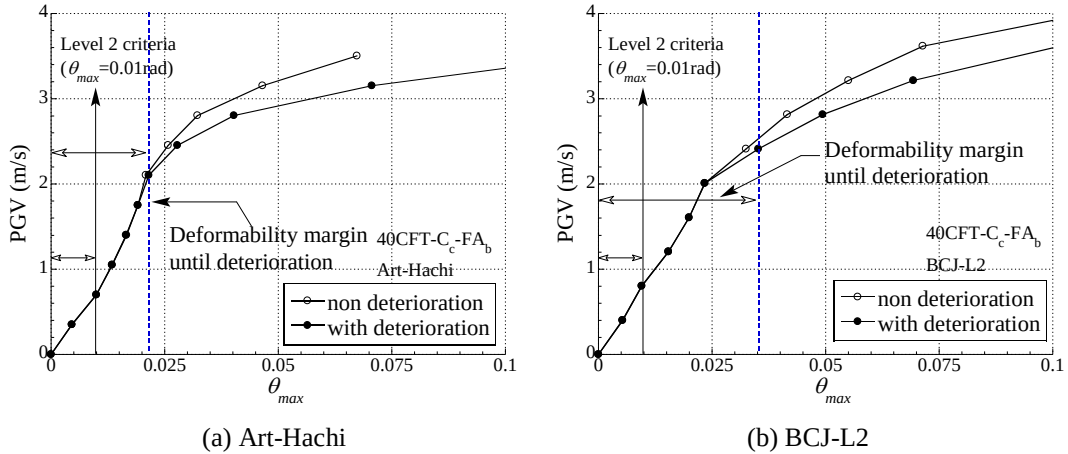
$$\xi_{det} = \frac{\theta_{max,det}}{\theta_{max,Level 2}} \quad (5.6)$$

Where, λ_{det} denotes deterioration margin which is a fraction of PGV where incipient deterioration occurs to the PGV (0.5m/s) of Level 2 criteria.

$$\xi_{col} = \frac{\theta_{max,col}}{\theta_{max,Level 2}} \quad (5.7)$$

Where, λ_{col} denotes the safety margin until collapse which is a fraction of PGV where incipient collapse occurs to the PGV (0.5m/s) of Level 2 criteria.

(1) *Global Deterioration*: Figures 5.41~5.42 show the global deterioration criterion of 40CFT-FA_c-FA_b building model and 40CFT-S_c-FB_b building model based on θ_{max} and θ_{res} . In case of 40CFT-FA_c-FA_b building model, effect of deterioration on θ_{max} is incipiently captured from yellow zone [Figure 5.41 (a)], and a small deterioration criteria of θ_{res} is derived as shown in Figure 5.41 (b).

**Figure 5.41** Deformability margin until collapse of the 40CFT-C_c-FA_b building with low-deterioration

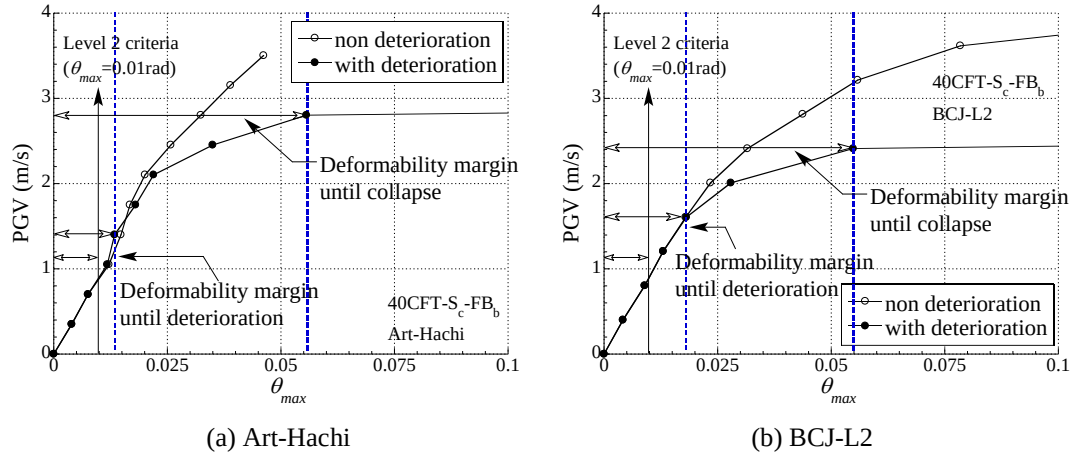


Figure 5.42 Deformability margin until collapse of the 40CFT-S_c-FB_b building with high-deterioration

(2) *Local Deterioration*: In order to evaluate the assess the amplified effect of member deterioration on the ductility-ratio response of these buildings simultaneously, the ductility ratio of beam plastic hinges by incorporating and non-incorporating the deterioration or not are compared with each other, as shown in Figure 5.43. Beside, for the weak-beam strong column CFT frame, the plastic hinge is also easily induced at the column-base of bottom story. Especially for the high-rise CFT building models subjected incremental earthquake excitations, the plastic rotation at the CFT column-base of bottom story is significantly affect the side-sway pattern of the corresponding building. Thus, as shown in Figure 5.44, the ductility ratio of column-base is also analogically compared between deteriorating and non-deteriorating models.

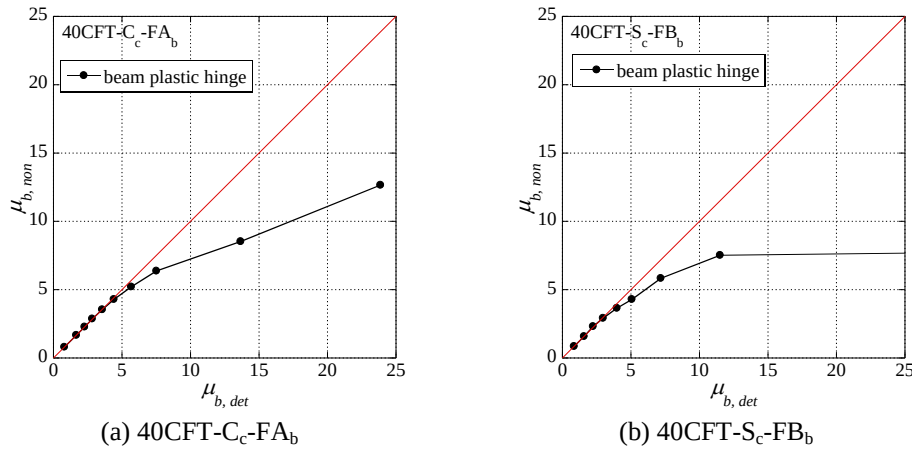


Figure 5.43 Effect of member deterioration on the beam ductility of 40CFT high-rise building models

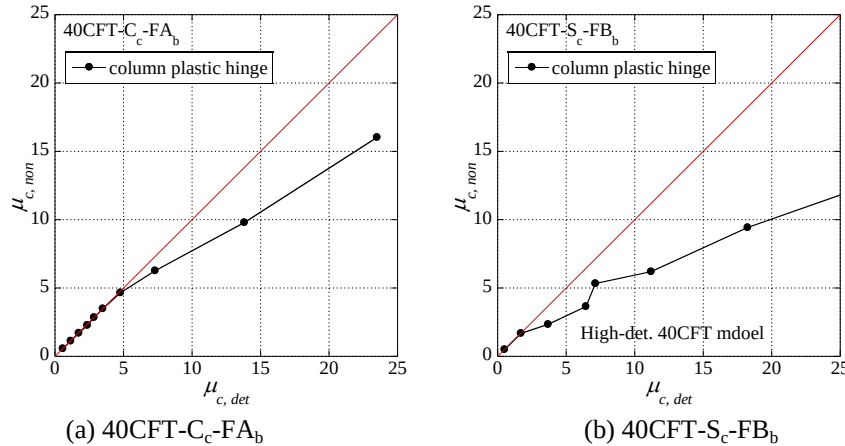


Figure 5.44 Effect of member deterioration on the column ductility of 40CFT high-rise building models

5.8 Conclusions

In this Chapter, the collapse-resistant capacity of various high-rise CFT buildings by substituting the square hollow steel tubular columns in high-rise steel buildings was correspondingly evaluated, based on the comparing to the associated high-rise steel buildings. In particular, the maximum and residual story drift angle, hysteretic energy absorption, the deterioration behavior and collapse criteria are specifically evaluated. Some concluding remarks can be obtained below.

(1) On the maximum and residual story drift angles of high-rise steel and CFT buildings, residual story drift angle initially stays at a low level comparing to the maximum story drift angle, and the difference value between maximum story drift angle and residual story drift angle gradually reduces when the intensity is increased after the incipient deterioration is induced (i.e., post branch point). However, the reducing ratios of the difference value of the deteriorating and non-deteriorating building models tend to show greatly different, e.g., from the branch points to the corresponding ending points with identical peak ground velocity. It indicates that the different value between maximum and residual drift angles of various building models (i.e., with-deterioration model and non-deterioration model, high deterioration building models and low-deterioration building models) is significantly dependent on the collapse-resistant capacity of them, e.g., the $\theta_{res}/\theta_{max}$ ratio increases after the branch point. The high level of member deterioration softens in θ_{res} versus θ_{max} relationship curve a little more than that of low-level of member deterioration. The 40-CFT high-deterioration building model will subsequently collapse earlier. On the other hand, the more θ_{res} versus θ_{max} relationship curve softens, the faster the level of IDR develops.

(2) According to the incremental member-level responses of plastic deformation ratios at the ends of beam and column base, the relationship between the seismic responses and the intensity of ground motions (i.e. PGV) was evaluated. When the earthquake intensity increases to initially generate the deterioration, the effect of member deterioration would slightly decrease the maximum plastic deformation ratio at the beam ends by increasing that at the column base of bottom story. Besides, although the incipient deterioration did not amplify the plastic deformation ratio at the beam ends, deterioration behavior of beam develops at lower story relatively earlier, and the plastic deformation ratio at the beam ends tend to be generated at lower story where member deterioration shows amplifying effect.

(3) Ductile global-sway collapse of low deterioration high-rise steel building models is achieved under severe seismic excitation, while the corresponding CFT building models could effectively prevent collapse from being occurred subjected to identical earthquake loading. However, in case that the collapse of high-rise steel building model is only induced under the ground motions with very large intensities that have very low probability of occurrence. Furthermore, the global-sway collapse of low deterioration high-rise steel building model under incremental seismic excitation (i.e. Art-Hachi, $\phi=5.0$) occurs when very extensive story drift angle (nearly 0.1rad) is achieved. In contrast, the story-sway collapse phenomenon of high-deterioration high-rise steel building model is observed at a relatively small story drift angle (nearly 0.01rad), subjected to the seismic excitation with the intensity that has higher probability of occurrence than that for low-deteriorating model (i.e. Art-Hachi, $\phi=2.0$). This implies that high-deterioration of the girder and columns in building models significantly influences the collapse resistant capacity and collapse criteria of high-rise steel building models under high intensity ground motions that have larger occurring probability than that of low-deteriorating model.

(4) Based on the safety margin until collapse and deformability margin until collapse of high-rise buildings, the deterioration behavior and collapse capacity is further indicated by comparing with the Level 2 seismic criteria. The effect of $P-\Delta$ effect and member deterioration effect are also incorporated in assessing the seismic margins of high-rise buildings based on the intensity of ground motion and deformability.

Therefore, based on the extensive comparative studies on the collapse-resistant capacity of high-rise steel and CFT building, CFT buildings tend to demonstrate significantly higher capacity to avoid collapse being occurred. Moreover, larger collapse margin of high-rise CFT buildings than that of high-rise steel building also indicates that such CFT buildings is more feasible for high-rise structural system. Safety margin until collapse and deformability margin until collapse are developed and capable of assessing the seismic-resistant capacity of high-rise steel and CFT buildings under extraordinary earthquakes.

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CHAPTER 6

SUMMARY AND CONCLUSIONS ---

6.1 Summary

In the preceding chapters, we have focused on the deterioration characteristics of various structural components, and explored the member deterioration and performance deterioration of the associated high-rise buildings subjected to severe ground motions. A database including extensive tests of H-shaped steel, hollow steel tubular member, concrete filled steel tubular (CFT) members was simulated to calibrate the feasibility and reliability of the unified constitutive model of stress-strain relations for concrete and steel taking the effect of deterioration behavior.

Moreover, the effects of member deteriorations on the seismic resistance of existent high-rise buildings of steel, CFT structures have been evaluated by the stress fiber model analysis with geometric nonlinearity. As a result, the seismic risk of such building structures is quantified on the basis of deteriorating stress-strain models. Thus, the seismic risk and collapse potentials of various existent high-rise buildings are expected to be determined by corresponding to the intensity of hazard, i.e., ground motions. Several conclusive remarks could be achieved via rigorous assessments on the seismic-resistant capacity of the existent high-rise buildings subjected to severe earthquakes, as follows.

6.2 Conclusive Remarks

The main part of this dissertation is including five chapters. Major findings derived in these chapters are respectively concluded as follows.

(1) Unified Constitutive Models of Various Structural Components

In Chapter two, the deterioration characteristics of the unified stress-strain models for the concrete and steel materials specifically in various associated structural members are developed. Confining effect on the concrete of CFT member was considered and the post-peak deterioration of concrete induced by crush and slip was also quantified based on a unified stress-strain model. Moreover, the strength and stiffness deteriorations of steel members with various cross sections are incorporated to evaluate the deterioration behavior of such members, e.g., local buckling of H-shaped steel and square steel tubular components may deteriorate the strength in a small interval of deformation. After the unified stress-strain models for concrete and steel elements being determined, the accuracy of these models to predict the occurrence of deterioration was calibrated on the basis of a database including extensive realistic experimental specimens. In particular, some calibration results can be concluded as follows:

(1) Unified Constitutive Models: A series of unified constitutive models for concrete and steel members have been developed by incorporating various deterioration effects of the unloading stiffness deterioration, Bauschinger effect, post-buckling strength deterioration and stiffness deterioration of negative slope. These constitutive models with deterioration effects are expected to accurately predict the deterioration behavior of various structural components

(2) H-shaped steel: High accuracy of the validation on the H-shaped steel under various loading procedures has been derived. The standard deviations of the test/analysis ratios of the strength and deformation at peak point are under 10%, and the standard deviation of the test/analysis ratios of the post-peak deterioration is around 10% which indicates that deterioration behavior is the most difficult part to be traced. The calibration accuracy of the unified constitutive models for H-shaped steel well satisfies the engineering application.

(3) Hollow steel tubes: As for the circular hollow steel tube specimens under various loading procedures, the standard deviations of the test/analysis ratios of the strength and deformation at peak point are both 1.3% and 3.3% and the standard deviation of the test/analysis of the post-peak deterioration is 8.7%. Besides, the circular hollow steel tubes under low and high axial load ratios can be stably predicted by the unified constitutive models. The analogous validation results are also obtained in case of the square hollow steel tubes.

(4) Concrete filled steel tubes: Circular concrete filled steel tubes under ultra-high axial load ratio ($n=0.8$) still shown stable hysteresis behavior, and the analytical results by using the unified constitutive models incorporating deteriorations of concrete and steel demonstrated instable behavior which is not march up to the test results. Hence, the confining effect tends to strengthen the concrete to prevent crush and slip and the deterioration behavior of the confined concrete in circular CFT should be ignored. In contrast, for the square CFT under cyclic loading, ignoring the steel deterioration would significantly overestimate the stiffness, and ignoring the concrete deterioration would overestimate the post-peak behavior. These characteristics of CFT were included in the constitutive model.

Overall, the unified constitutive models for the concrete and steel member of various structural components under various loading procedures, is capable to simulate the strength, deformation at peak point and also post-peak deterioration behavior on the premise of high accuracy. The deterioration behavior of concrete and steel members should be rigorously considered in the seismic design of these structural components.

(2) Simulation on Deterioration Behavior of Various Structural Frames

In Chapter three, a full-scale shaking table test of 4-story steel moment-resistant frames subjected to incremental earthquake loadings was used for the calibration of stress fiber model analysis employing the stress-strain models incorporating strength and stiffness deteriorations, in order to predict the collapse capacity of the steel frame under severe seismic excitation. Besides, due to the assessing on the collapse safety of existent high-rise buildings are urgent to be evaluated because the long-period ground motions tend to produce a lot more responses on such high-rise buildings, another E-Defense shaking table test of a high-rise steel building subjected to long-period ground motions is also used for the validation of the unified constitutive model. In addition, a CFT frame specimen subjected to cyclic horizontal loadings with constant axial loading is also simulated in this chapter.

(1) Overall, the analysis and analytical model is a plane frame takes the geometric nonlinearity into account, and calibration on the test specimen of 4-story steel frame by X-direction and Y-direction based on two dimensional modelling shown feasible to predict the collapse. The story

drift of bottom story of the test specimen is the most difficult part to be accurately predicted, because the local buckling of the columns at first story makes the seismic response calculation instable, especially subjected to 100% Takatori-JR loading. However, the collapse phenomenon of the test specimen was accurately predicted on both X and Y directions. Based on percentage of hysteresis energy absorption at each story, bottom story dissipates around 80% energy under 100% Takatori-JR loading.

(2) With respect to the shaking table test on the high-rise steel buildings subjected to long-period ground motions, story shear stiffness plays an important role on the predicting the seismic responses. According to the test phenomenon during loading, fractures occurred at the bottom flanges of girder at connections deteriorate the story shear stiffness of the test specimen. Subsequently, the story shear force and story drift angle relationship is significantly affected by the reduction of story shear stiffness subjected to San-NS/EW-3 loading. Due to the fracture is not incorporated in the analytical frame model and the two-axial buckling happened in the three dimensional test specimen could not be simulated in two-dimensional planar frame model. So reasonably errors are observed in the comparisons between experimental and analytical results.

(3) Besides, through simulating a cyclic test of CFT frame structures, the unified constitutive models with deterioration effect can accurately predict the deterioration behaviour of the CFT frame structure. Ignoring the deterioration effects of steel tube and confined concrete will significantly overestimate the seismic resistant capacity of CFT frame structures. Hence, deterioration effect should be rigorously considered in simulating the seismic performance of high-rise CFT building structures.

Although somewhat differences must be included in the modelling by two-dimensional plan frame model with respect to three dimensional frame test, the two dimensional frame models by adopting the unified constitutive models are reliably capable to predict the collapse and seismic responses of low-rise and high-rise steel and concrete filled steel tubular buildings.

(3) Evaluation of Seismic-Resistant Capacity of Existent High-Rise Steel Buildings

In Chapter four, the effect of member deterioration on the collapse capacity of existent high-rise steel buildings subjected to severe seismic excitations is conducted through a time-history analysis using stress fiber models consist of unified constitutive models. Herein, the characteristics and the intensity of ground motions and story levels are especially incorporated to evaluate the seismic risk of such buildings.

(1) According to the time-history analysis of various existent high-rise steel buildings with different story levels subjected to long-period ground motions, the effect of predominant period of such long-period ground motions have significant influences on the seismic responses of high-rise building models. For the existent high-rise steel buildings, the seismic safety is greatly dependent on the predominant period of ground and it should said that the corresponding synthetic ground motions should be incorporated in the assessment on the seismic safety of high-rise buildings. The seismic resistant capacity tends to be more affected by the deterioration of columns than that of girders. Due to the weak beam strong column system, the member deterioration proceeds from beam to column. Hence, even though the seismic design based on Level 1 PGV criteria and Level 2 PGV criteria is well satisfied for various existent high-rise buildings, long period ground motions induced unsatisfactory of those seismic criteria should be considered in seismic assessment and design and only PGV is not sufficient treated as the intensity criteria.

(2) Pulse-like ground motions with long-period component and short duration of vibrations excite larger seismic responses and deterioration behavior of high-rise building models. This

indicates that near-fault inland ground motions have large safety risks on those buildings and deterioration effect should be considered in the evaluation of existing high-rise steel buildings.

(3) With respect to the performance indicators and component indicators of the high-rise buildings based on incremental dynamic analysis, residual story drift angle is rarely induced when the intensity of ground motions stays at lower level. However, residual story drift angle increases and develops faster than maximum story drift angle when subjected to high-level intensity of ground motions, and eventually becomes very sensitive until collapse. Hence, how to judge the extent of residual story drift angle is important to evaluate the functionability and serviceability of the existent high-rise steel buildings after earthquakes.

(4) The local member deterioration causes to make the story shear stiffness of high-rise steel buildings softened. Even though the uncertainty of ground motion tends to affect the deterioration behavior of high-rise steel buildings, the incipient deterioration of the identical building model merges relatively close. As a matter of fact, from the deterioration behavior of such steel buildings, the effect of member deterioration starts to play a more and more important role on the seismic-resistance deterioration developing, e.g., the deterioration behavior of FB_c - FB_b building model greatly suddenly than that of

(4) P - Δ effect causes the high-rise building sway asymmetrically in one-side direction, and until instable state. In contrast, member deterioration effect further amplifies this asymmetric state and makes the building structures deteriorates and eventually collapse.

(5) After comprehensively evaluating the seismic-resistant capacity of the high-rise building models with different story levels and compactness steel members. Deterioration behavior of those building structures is significantly affected by the compactness ranks of steel members and the story level. The compact section demonstrated stable behavior after local buckling and higher story level (40S) shown most severe safety risks compared with others.

(6) The safety margin until collapse for existent high-rise steel buildings is proposed on the basis of the deterioration criteria and incipient collapse, in order to predict the degree of the collapse safety of various high-rise steel buildings subjected to incremental seismic excitations. The safety margin until collapse shown a lot more correlated to the level of member deterioration, e.g., low deterioration building models demonstrated collapse criterion around 10 times higher than that of the corresponding high deteriorating building models.

All in all, the P - Δ effect and member deterioration have been proved very significantly influence the seismic-resistant capacity and collapse behaviour of high-rise steel buildings. Those efficient factors should be rigorously considered in seismic assessment of high-rise steel buildings subjected extraordinary earthquakes.

(4) Evaluation of Seismic-Resistant Capacity of Existent High-Rise Concrete-Filled Steel Tubular Buildings: in Comparison with High-Rise Steel Buildings

In Chapter five, the collapse-resistant capacity of various high-rise CFT buildings by substituting the square hollow steel tubular columns in high-rise steel buildings was correspondingly evaluated, based on the comparing to the associated high-rise steel buildings. In particular, the maximum and residual story drift angle, hysteretic energy absorption, the deterioration behavior and collapse criteria are specifically evaluated. Some concluding remarks can be obtained below.

(1) On the maximum and residual story drift angles of high-rise steel and CFT buildings, residual story drift angle initially stays at a low level comparing to the maximum story drift angle, and the difference value between maximum story drift angle and residual story drift angle gradually

reduces when the intensity is increased after the incipient deterioration is induced (i.e., post branch point). However, the reducing ratios of the difference value of the deteriorating and non-deteriorating building models tend to show greatly different, e.g., from the branch points to the corresponding ending points with identical peak ground velocity. It indicates that the different value between maximum and residual drift angles of various building models (i.e., with-deterioration model and non-deterioration model, high deterioration building models and low-deterioration building models) is significantly dependent on the collapse-resistant capacity of them, e.g., the $\theta_{res}/\theta_{max}$ ratio increases after the branch point. The high level of member deterioration softens in θ_{res} versus θ_{max} relationship curve a little more than that of low-level of member deterioration. The 40-CFT high-deterioration building model will subsequently collapse earlier. On the other hand, the more θ_{res} versus θ_{max} relationship curve softens, the faster the level of IDR develops.

(2) According to the incremental member-level responses of plastic deformation ratios at the ends of beam and column base, the relationship between the seismic responses and the intensity of ground motions (i.e. PGV) was evaluated. When the earthquake intensity increases to initially generate the deterioration, the effect of member deterioration would slightly decrease the maximum plastic deformation ratio at the beam ends by increasing that at the column base of bottom story. Besides, although the incipient deterioration did not amplify the plastic deformation ratio at the beam ends, deterioration behavior of beam develops at lower story relatively earlier, and the plastic deformation ratio at the beam ends tend to be generated at lower story where member deterioration shows amplifying effect.

(3) Ductile global-sway collapse of low deterioration high-rise steel building models is achieved under severe seismic excitation, while the corresponding CFT building models could effectively prevent collapse from being occurred subjected to identical earthquake loading. However, in case that the collapse of high-rise steel building model is only induced under the ground motions with very large intensities that have very low probability of occurrence. Furthermore, the global-sway collapse of low deterioration high-rise steel building model under incremental seismic excitation (i.e. Art-Hachi, $\phi=5.0$) occurs when very extensive story drift angle (nearly 0.1rad) is achieved. In contrast, the story-sway collapse phenomenon of high-deterioration high-rise steel building model is observed at a relatively small story drift angle (nearly 0.01rad), subjected to the seismic excitation with the intensity that has higher probability of occurrence than that for low-deteriorating model (i.e. Art-Hachi, $\phi=2.0$). This implies that high-deterioration of the girder and columns in building models significantly influences the collapse resistant capacity and collapse criteria of high-rise steel building models under high intensity ground motions that have larger occurring probability than that of low-deteriorating model.

(4) Based on the safety margin until collapse and deformability margin until collapse of high-rise buildings, the deterioration behavior and collapse capacity is further indicated by comparing with the Level 2 seismic criteria. The effect of P- Δ effect and member deterioration effect are also incorporated in assessing the seismic margins of high-rise buildings based on the intensity of ground motion and deformability.

Therefore, based on the extensive comparative studies on the collapse-resistant capacity of high-rise steel and CFT building, CFT buildings tend to demonstrate significantly higher capacity to avoid collapse occurred. Moreover, larger safety margin until collapse of high-rise CFT buildings than that of high-rise steel building also indicates that such CFT buildings is more feasible for high-rise structural system. Safety margin until collapse and deformability margin until

collapse are developed and capable of assessing the seismic-resistant capacity of high-rise steel and CFT buildings under extraordinary earthquakes.

6.3 Overall Conclusions

Strength deterioration and stiffness deterioration in the negative slope after peak point of various steel components, and post-peak deterioration of the unconfined and confined concretes, have proved to be an essential factor for the seismic-resistant capacity of various structural components and system. By considering the influences of $P-\Delta$ effect and strength and stiffness member deterioration effect, the existent high-rise buildings have higher probability of collapse when under long-period ground motions and pulse-like ground motions, than under earthquake waves which are used in structural design so far. $P-\Delta$ effect tends to reduce the seismic resistant capacity of the building with higher story level based on Uetani theory. Strength and stiffness member deterioration make the high-rise building suddenly collapse and reduce the safety margin until collapse. Hence, the influences of $P-\Delta$ effect and member deterioration effect should be incorporated not only for the seismic evaluation on various existent building structures, but also for the performance-based seismic design of new constructing building structures in the future.

6.4 Future Work

(1) With respect to the unified stress-strain models of concrete incorporating post-peak strength deterioration, the uncertainty of strength deterioration still exists. The post-crushing behaviour of the concrete in various structural components should be further involved in, in order to improve the reliability of the deteriorating constitutive models to predict the deterioration behavior of concrete frame structures.

(2) The stress-strain relation model with deterioration behavior for steel element has been developed, in which, the deterioration induced by local-buckling, negative slope after local buckling, unloading stiffness deterioration and Baushinger effect have been incorporated. However, in the calibration of high-rise steel buildings under long-period ground motion (Chapter 3), the fractures at the flange of connections significantly deteriorate the story stiffness as the intensity or duration of ground motion increase, which has not been considered yet. Therefore, the deteriorating stress-strain model for steel is expected to incorporate the effect of fracture in the future.

(3) The seismic resistant-capacity of various existent high-rise buildings under severe earthquake excitations was addressed in Chapters 4~5, planar models of full moment-resistant frame was designed for each case building. However, due to braced frame has been frequently constructed since 1980s, whether the braced frame with various damper facilities is capable to effectively mitigate the seismic damages of the corresponding high-rise buildings still needs to be evaluated.

(4) For the seismic mitigation of new constructing high-rise buildings, in which various types of damper facility or seismic isolation would be feasible to be utilized in high-rise buildings to mitigate the performance deterioration and collapse risks.

(5) In order to apply for the performance-based seismic design, the deterioration criteria and the safety margin until collapse of high-rise buildings should be further developed to be more feasible and simplified in practical design.

APPENDIX A

DATABASES OF VARIOUS TEST SPECIMENS

Since extensive comparisons between experimental and the associated analytical results are conducted in Chapter 2, to evaluate the member deterioration on the seismic behavior of various structural components, e.g., reinforced concrete, circular and rectangular hollow steel tubular members and circular and rectangular concrete filled steel tubular members.

Therefore, the details of the deterioration parameters corresponding to such structural components are respectively demonstrated in Appendix A.

Table A.1 Parameters of calibrated H-shaped steel specimens

Specimen	D [mm]	B [mm]	T_w [mm]	T_f [mm]	σ_y [N/mm ²]	σ_u [N/mm ²]	n ($=N/N_y$)	ε_y	α_w	α_f	ε_{lb}	τ_{lb1}	r_d	τ_{lb2}	L [mm]
I-083C	100.40	99.90	3.27	6.30	304.99	445.22	0.30	0.0015	1.073	0.0935	0.0109	-0.0108	0.8656	-0.0030	
I-086C	99.50	100.20	3.28	6.27	304.99	445.22	0.60	0.0015	1.046	0.0950	0.0110	-0.0107	0.8648	-0.0030	
I-083D	135.30	100.30	3.13	6.62	300.08	403.05	0.30	0.0015	2.226	0.0840	0.0092	-0.0273	0.8710	-0.0030	
I-113C	99.60	99.50	3.24	4.51	320.19	457.48	0.30	0.0016	1.221	0.1901	0.0095	-0.0279	0.8116	-0.0030	
I-116C	100.70	100.30	3.27	4.51	320.19	457.48	0.50	0.0016	1.228	0.1931	0.0094	-0.0287	0.8098	-0.0030	
I-H-063B	136.50	76.10	6.12	6.12	456.01	538.39	0.30	0.0022	0.917	0.0860	0.0179	-0.0086	0.8698	-0.0030	
I-H-066B	137.10	76.00	6.15	6.12	456.01	538.39	0.60	0.0022	0.917	0.0858	0.0179	-0.0086	0.8700	-0.0030	
II-083C	100.20	99.30	3.27	6.21	304.99	445.22	0.30	0.0015	1.072	0.0951	0.0109	-0.0109	0.8647	-0.0030	
II-086C	100.20	100.10	3.29	6.28	304.99	445.22	0.60	0.0015	1.056	0.0945	0.0110	-0.0107	0.8651	-0.0030	
II-083D	135.60	100.20	3.13	6.61	300.08	403.05	0.30	0.0015	2.238	0.0841	0.0092	-0.0276	0.8709	-0.0030	
II-113C	99.80	99.70	3.27	4.51	320.19	457.48	0.30	0.0016	1.204	0.1908	0.0095	-0.0279	0.8111	-0.0030	
II-116C	100.30	99.30	3.29	4.51	320.19	457.48	0.60	0.0016	1.202	0.1893	0.0095	-0.0276	0.8120	-0.0030	
II-H-063B	137.20	75.70	6.43	6.43	456.01	538.39	0.30	0.0022	0.832	0.0771	0.0208	-0.0071	0.8748	-0.0030	
II-H-066B	136.80	75.50	6.41	6.41	456.01	538.39	0.60	0.0022	0.832	0.0772	0.0208	-0.0071	0.8748	-0.0030	
BH4-8-8-20	213.00	201.60	9.00	12.00	362.85	387.36	0.00	0.0018	0.781	0.1249	0.0138	-0.0122	0.8481	-0.0030	639
BH4-8-8-25	260.70	199.20	9.00	12.00	356.96	372.65	0.00	0.0017	1.204	0.1200	0.0116	-0.0154	0.8508	-0.0030	782
BH4-8-8-30	307.50	199.20	9.00	12.00	350.10	363.83	0.00	0.0017	1.695	0.1176	0.0104	-0.0216	0.8521	-0.0030	923
BH4-8-8-35	354.30	201.60	9.00	12.00	345.19	354.02	0.00	0.0017	2.268	0.1188	0.0095	-0.0322	0.8515	-0.0030	1063
BH4-8-10-20	213.00	249.60	9.00	12.00	357.94	372.65	0.00	0.0017	0.770	0.1889	0.0128	-0.0236	0.8122	-0.0030	639
BH4-8-10-25	260.70	249.60	9.00	12.00	358.92	375.59	0.00	0.0018	1.211	0.1894	0.0107	-0.0277	0.8120	-0.0030	782
BH4-8-10-30	307.50	252.00	9.00	12.00	354.02	364.81	0.00	0.0017	1.714	0.1904	0.0094	-0.0347	0.8114	-0.0030	923
BH4-8-10-35	354.30	252.00	9.00	12.00	344.21	349.12	0.00	0.0017	2.262	0.1851	0.0086	-0.0436	0.8143	-0.0030	1063
BH4-8-12-25	260.70	300.00	9.00	12.00	353.04	366.77	0.00	0.0017	1.191	0.2691	0.0101	-0.0483	0.7673	-0.0030	782

Appendix A. Databases of Various Test Specimens

BH4-8-12-30	307.50	302.40	9.00	12.00	348.14	356.96	0.00	0.0017	1.685	0.2696	0.0088	-0.0550	0.7670	-0.0030	823
BH4-8-12-35	353.40	302.40	9.00	12.00	342.25	348.14	0.00	0.0017	2.236	0.2651	0.0081	-0.0636	0.7696	-0.0030	1060
BH4-8-14-30	307.50	352.80	9.00	12.00	349.12	360.88	0.00	0.0017	1.690	0.3680	0.0086	-0.0908	0.7119	-0.0030	923
BH4-8-14-35	354.30	352.80	9.00	12.00	340.29	347.16	0.00	0.0017	2.236	0.3587	0.0077	-0.0968	0.7171	-0.0030	1063
BH5-7-7-15	157.20	168.00	9.00	12.00	361.87	475.62	0.00	0.0018	0.387	0.0865	0.0292	-0.0055	0.8696	-0.0030	472
BH5-7-7-20	203.10	168.00	9.00	12.00	358.92	445.22	0.00	0.0018	0.693	0.0858	0.0176	-0.0069	0.8700	-0.0030	609
BH5-7-7-25	246.30	168.00	9.00	12.00	357.94	397.17	0.00	0.0017	1.065	0.0856	0.0132	-0.0099	0.8701	-0.0030	739
BH5-7-7-30	291.30	168.00	9.00	12.00	363.83	376.58	0.00	0.0018	1.566	0.0870	0.0119	-0.0161	0.8693	-0.0030	874
BH5-7-9-15	157.20	204.00	9.00	12.00	355.98	428.55	0.00	0.0017	0.380	0.1255	0.0260	-0.0101	0.8477	-0.0030	472
BH5-7-9-20	202.20	204.00	9.00	12.00	357.94	420.71	0.00	0.0017	0.685	0.1262	0.0145	-0.0117	0.8474	-0.0030	607
BH5-7-9-25	246.30	204.00	9.00	12.00	360.88	388.34	0.00	0.0018	1.074	0.1272	0.0120	-0.0150	0.8468	-0.0030	739
BH5-7-9-30	291.30	204.00	9.00	12.00	360.88	378.54	0.00	0.0018	1.553	0.1272	0.0107	-0.0208	0.8468	-0.0030	874
BH5-7-10-20	202.20	240.00	9.00	12.00	359.90	384.42	0.00	0.0018	0.688	0.1756	0.0137	-0.0202	0.8197	-0.0030	607
BH5-7-10-25	246.30	240.00	9.00	12.00	361.87	382.46	0.00	0.0018	1.077	0.1765	0.0114	-0.0236	0.8191	-0.0030	739
BH5-7-10-30	292.20	240.00	9.00	12.00	364.81	377.56	0.00	0.0018	1.580	0.1780	0.0101	-0.0300	0.8183	-0.0030	877
BH5-7-12-25	246.30	276.00	9.00	12.00	360.88	372.65	0.00	0.0018	1.074	0.2328	0.0109	-0.0367	0.7876	-0.0030	739
BH5-7-12-30	291.30	276.00	9.00	12.00	360.88	367.75	0.00	0.0018	1.553	0.2328	0.0096	-0.0425	0.7876	-0.0030	874
BH5-8-7-15	160.80	172.80	9.00	12.00	425.61	514.85	0.00	0.0021	0.480	0.1076	0.0260	-0.0082	0.8577	-0.0030	482
BH5-8-7-20	206.70	172.80	9.00	12.00	424.63	469.74	0.00	0.0021	0.854	0.1074	0.0160	-0.0104	0.8579	-0.0030	620
BH5-8-7-25	252.60	170.40	9.00	12.00	424.63	441.30	0.00	0.0021	1.336	0.1044	0.0138	-0.0149	0.8595	-0.0030	758
BH5-8-7-30	297.60	170.40	9.00	12.00	424.63	435.42	0.00	0.0021	1.914	0.1044	0.0126	-0.0236	0.8595	-0.0030	893
BH5-8-9-15	160.80	208.80	9.00	12.00	422.67	462.87	0.00	0.0021	0.476	0.1561	0.0230	-0.0154	0.8306	-0.0030	482
BH5-8-9-20	206.70	208.80	9.00	12.00	424.63	452.09	0.00	0.0021	0.854	0.1568	0.0149	-0.0179	0.8302	-0.0030	620
BH5-8-9-25	252.60	206.40	9.00	12.00	426.59	443.26	0.00	0.0021	1.343	0.1539	0.0127	-0.0223	0.8318	-0.0030	758
BH5-8-9-30	297.60	206.40	9.00	12.00	425.61	436.40	0.00	0.0021	1.919	0.1536	0.0115	-0.0309	0.8320	-0.0030	893
BH5-8-10-20	206.70	244.80	9.00	12.00	429.53	446.20	0.00	0.0021	0.863	0.2180	0.0143	-0.0310	0.7959	-0.0030	620
BH5-8-10-25	251.70	244.80	9.00	12.00	424.63	438.36	0.00	0.0021	1.326	0.2155	0.0120	-0.0351	0.7973	-0.0030	755
BH5-8-10-30	297.60	244.80	9.00	12.00	423.65	434.43	0.00	0.0021	1.910	0.2150	0.0107	-0.0436	0.7976	-0.0030	893
BH5-8-12-25	251.70	280.80	9.00	12.00	428.55	436.40	0.00	0.0021	1.338	0.2862	0.0116	-0.0554	0.7577	-0.0030	755
BH5-8-12-30	297.60	280.80	9.00	12.00	429.53	442.28	0.00	0.0021	1.936	0.2868	0.0104	-0.0646	0.7574	-0.0030	893
BH5-9-7-15	163.50	177.60	9.00	12.00	486.41	496.22	0.00	0.0024	0.570	0.1299	0.0234	-0.0116	0.8452	-0.0030	491
BH5-9-7-20	213.90	177.60	9.00	12.00	491.31	503.08	0.00	0.0024	1.067	0.1312	0.0163	-0.0156	0.8445	-0.0030	642
BH5-9-7-25	259.80	177.60	9.00	12.00	491.31	496.22	0.00	0.0024	1.645	0.1312	0.0143	-0.0228	0.8445	-0.0030	779
BH5-9-9-15	164.40	213.60	9.00	12.00	484.45	494.26	0.00	0.0024	0.575	0.1872	0.0203	-0.0220	0.8132	-0.0030	493
BH5-9-9-20	213.90	213.60	9.00	12.00	488.37	498.18	0.00	0.0024	1.061	0.1887	0.0153	-0.0260	0.8123	-0.0030	642
BH5-9-9-25	259.80	213.60	9.00	12.00	493.27	499.16	0.00	0.0024	1.652	0.1906	0.0133	-0.0338	0.8113	-0.0030	779
BH5-9-10-20	213.00	254.40	9.00	12.00	487.39	495.24	0.00	0.0024	1.049	0.2671	0.0146	-0.0462	0.7684	-0.0030	639
BH5-9-10-25	257.10	254.40	9.00	12.00	491.31	494.26	0.00	0.0024	1.608	0.2693	0.0127	-0.0537	0.7672	-0.0030	771
BH5-9-12-25	257.10	290.40	9.00	12.00	485.43	490.33	0.00	0.0024	1.588	0.3467	0.0122	-0.0806	0.7239	-0.0030	771
BH6-8-6-14	153.60	151.20	9.00	12.00	478.56	624.68	0.00	0.0023	0.484	0.0927	0.0307	-0.0065	0.8661	-0.0030	461
BH6-8-6-18	191.40	151.20	9.00	12.00	483.47	605.07	0.00	0.0024	0.816	0.0936	0.0196	-0.0086	0.8656	-0.0030	574
BH6-8-6-22	227.40	151.20	9.00	12.00	490.33	541.33	0.00	0.0024	1.222	0.0949	0.0168	-0.0125	0.8648	-0.0030	682
BH6-8-6-26	264.30	151.20	9.00	12.00	484.45	524.66	0.00	0.0024	1.685	0.0938	0.0153	-0.0186	0.8655	-0.0030	793
BH6-8-8-14	153.60	187.20	9.00	12.00	479.55	573.69	0.00	0.0023	0.485	0.1423	0.0263	-0.0131	0.8383	-0.0030	461
BH6-8-8-18	190.50	189.60	9.00	12.00	482.49	580.55	0.00	0.0024	0.806	0.1469	0.0175	-0.0158	0.8357	-0.0030	572

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BH6-8-8-22	227.40	189.60	9.00	12.00	489.35	541.33	0.00	0.0024	1.219	0.1490	0.0151	-0.0200	0.8346	-0.0030	682
BH6-8-8-26	265.20	187.20	9.00	12.00	489.35	526.62	0.00	0.0024	1.715	0.1452	0.0137	-0.0260	0.8367	-0.0030	796
BH6-8-9-18	190.50	225.60	9.00	12.00	482.49	533.48	0.00	0.0024	0.806	0.2080	0.0167	-0.0281	0.8015	-0.0030	572
BH6-8-9-22	227.40	225.60	9.00	12.00	491.31	523.68	0.00	0.0024	1.224	0.2118	0.0143	-0.0330	0.7994	-0.0030	682
BH6-8-9-26	264.30	225.60	9.00	12.00	492.29	522.69	0.00	0.0024	1.712	0.2122	0.0129	-0.0396	0.7992	-0.0030	793
BH6-8-11-22	227.40	264.00	9.00	12.00	495.24	513.87	0.00	0.0024	1.234	0.2923	0.0138	-0.0562	0.7543	-0.0030	682
BH6-8-11-26	264.30	264.00	9.00	12.00	499.16	512.89	0.00	0.0024	1.736	0.2946	0.0124	-0.0638	0.7530	-0.0030	793
CT-4-4	150.00	150.00	6.00	9.00	315.77	429.53	0.00	0.0015	0.746	0.1070	0.0128	-0.0096	0.8581	-0.0030	450
CT-4-5	150.00	150.00	6.00	9.00	432.96	496.22	0.00	0.0021	1.022	0.1467	0.0143	-0.0176	0.8359	-0.0030	450
CT-5-4	150.00	150.00	6.00	9.00	400.11	519.75	0.00	0.0020	0.945	0.1355	0.0138	-0.0151	0.8421	-0.0030	450
CT-5-5	150.00	150.00	6.00	9.00	517.30	586.44	0.00	0.0025	1.221	0.1752	0.0155	-0.0249	0.8199	-0.0030	450
CF-4-4	150.00	150.00	6.00	9.00	291.26	457.97	0.00	0.0014	0.688	0.0987	0.0133	-0.0082	0.8627	-0.0030	450
CF-4-5	150.00	150.00	6.00	9.00	408.45	524.66	0.00	0.0020	0.964	0.1384	0.0139	-0.0157	0.8405	-0.0030	450
CF-5-4	150.00	150.00	6.00	9.00	408.45	524.66	0.00	0.0020	0.964	0.1384	0.0139	-0.0157	0.8405	-0.0030	450
CF-5-5	150.00	150.00	6.00	9.00	525.64	591.34	0.00	0.0026	1.241	0.1781	0.0157	-0.0257	0.8183	-0.0030	450
SCH062-C	138.00	108.00	6.00	9.00	299.59	442.77	0.00	0.0015	0.585	0.0526	0.0223	-0.0036	0.8885	-0.0030	414
SCH062-D	138.00	108.00	6.00	9.00	354.51	426.10	0.00	0.0017	0.692	0.0623	0.0212	-0.0049	0.8831	-0.0030	414
SCH082-C	138.00	144.00	6.00	9.00	299.59	442.77	0.00	0.0015	0.585	0.0935	0.0162	-0.0071	0.8656	-0.0030	432
SCH082-D	138.00	144.00	6.00	9.00	354.51	426.10	0.00	0.0017	0.692	0.1107	0.0151	-0.0097	0.8560	-0.0030	432
SCH102-C	138.00	180.00	6.00	9.00	299.59	442.77	0.00	0.0015	0.585	0.1461	0.0134	-0.0142	0.8362	-0.0030	540
SCH102-D	138.00	180.00	6.00	9.00	354.51	426.10	0.00	0.0017	0.692	0.1729	0.0135	-0.0197	0.8212	-0.0030	540
SCH122-C	138.00	216.00	6.00	9.00	299.59	442.77	0.00	0.0015	0.585	0.2104	0.0121	-0.0273	0.8002	-0.0030	648
SCH122-D	138.00	216.00	6.00	9.00	354.51	426.10	0.00	0.0017	0.692	0.2490	0.0129	-0.0380	0.7785	-0.0030	648
SCH083-C	198.00	144.00	6.00	9.00	299.59	442.77	0.00	0.0015	1.315	0.0935	0.0101	-0.0134	0.8656	-0.0030	594
SCH083-D	198.00	144.00	6.00	9.00	354.51	426.10	0.00	0.0017	1.556	0.1107	0.0109	-0.0186	0.8560	-0.0030	594
SCH103-C	198.00	180.00	6.00	9.00	299.59	442.77	0.00	0.0015	1.315	0.1461	0.0091	-0.0206	0.8362	-0.0030	594
SCH103-D	198.00	180.00	6.00	9.00	354.51	426.10	0.00	0.0017	1.556	0.1729	0.0099	-0.0287	0.8212	-0.0030	594
SCH123-C	198.00	216.00	6.00	9.00	299.59	442.77	0.00	0.0015	1.315	0.2104	0.0085	-0.0337	0.8002	-0.0030	648
SCH123-D	198.00	216.00	6.00	9.00	354.51	426.10	0.00	0.0017	1.556	0.2490	0.0093	-0.0470	0.7785	-0.0030	648
SCH143-C	198.00	252.00	6.00	9.00	299.59	442.77	0.00	0.0015	1.315	0.2864	0.0082	-0.0552	0.7576	-0.0030	756
SCH143-D	198.00	252.00	6.00	9.00	354.51	426.10	0.00	0.0017	1.556	0.3389	0.0090	-0.0771	0.7282	-0.0030	756
SCH163-C	198.00	288.00	6.00	9.00	299.59	442.77	0.00	0.0015	1.315	0.3741	0.0080	-0.0882	0.7085	-0.0030	864
SCH163-D	258.00	288.00	6.00	9.00	354.51	426.10	0.00	0.0017	2.767	0.4427	0.0075	-0.1474	0.6701	-0.0030	864
SCH104-C	258.00	180.00	6.00	9.00	299.59	442.77	0.00	0.0015	2.338	0.1461	0.0078	-0.0378	0.8362	-0.0030	774
SCH104-D	258.00	180.00	6.00	9.00	354.51	426.10	0.00	0.0017	2.767	0.1729	0.0086	-0.0528	0.8212	-0.0030	774
SCH124-C	258.00	216.00	6.00	9.00	299.59	442.77	0.00	0.0015	2.338	0.2104	0.0073	-0.0509	0.8002	-0.0030	774
SCH124-D	258.00	216.00	6.00	9.00	354.51	426.10	0.00	0.0017	2.767	0.2490	0.0081	-0.0711	0.7785	-0.0030	774
SCH144-C	258.00	252.00	6.00	9.00	299.59	442.77	0.00	0.0015	2.338	0.2864	0.0069	-0.0724	0.7576	-0.0030	774
SCH144-D	258.00	252.00	6.00	9.00	354.51	426.10	0.00	0.0017	2.767	0.3389	0.0077	-0.1012	0.7282	-0.0030	774
SCH164-C	258.00	288.00	6.00	9.00	299.59	442.77	0.00	0.0015	2.338	0.3741	0.0067	-0.1054	0.7085	-0.0030	864
SCH164-D	258.00	288.00	6.00	9.00	354.51	426.10	0.00	0.0017	2.767	0.4427	0.0075	-0.1474	0.6701	-0.0030	864
SCH184-C	258.00	324.00	6.00	9.00	299.59	442.77	0.00	0.0015	2.338	0.4735	0.0066	-0.1534	0.6528	-0.0030	972
SCH184-D	258.00	324.00	6.00	9.00	354.51	426.10	0.00	0.0017	2.767	0.5603	0.0074	-0.2147	0.6042	-0.0030	972
A65	318.00	288.00	6.00	9.00	362.36	423.16	0.00	0.0018	4.419	0.4525	0.0070	-0.2070	0.6646	-0.0030	2148
A45	318.00	252.00	6.00	9.00	362.36	423.16	0.00	0.0018	4.419	0.3464	0.0073	-0.1587	0.7240	-0.0030	1848

A25	318.00	216.00	6.00	9.00	362.36	423.16	0.00	0.0018	4.419	0.2545	0.0076	-0.1273	0.7755	-0.0030	1548
A05	318.00	180.00	6.00	9.00	362.36	423.16	0.00	0.0018	4.419	0.1768	0.0081	-0.1081	0.8190	-0.0030	1251
A85	318.00	144.00	6.00	9.00	362.36	423.16	0.00	0.0018	4.419	0.1131	0.0092	-0.0976	0.8546	-0.0030	960
B65	318.00	288.00	6.00	9.00	284.39	419.72	0.00	0.0014	3.468	0.3551	0.0059	-0.1277	0.7191	-0.0030	2148
B45	318.00	252.00	6.00	9.00	284.39	419.72	0.00	0.0014	3.468	0.2719	0.0061	-0.0980	0.7657	-0.0030	1848
B25	318.00	216.00	6.00	9.00	284.39	419.72	0.00	0.0014	3.468	0.1998	0.0065	-0.0786	0.8061	-0.0030	1548
B05	318.00	180.00	6.00	9.00	284.39	419.72	0.00	0.0014	3.468	0.1387	0.0070	-0.0668	0.8403	-0.0030	1251
B85	318.00	144.00	6.00	9.00	284.39	419.72	0.00	0.0014	3.468	0.0888	0.0080	-0.0603	0.8683	-0.0030	960
Y661	150.00	150.00	6.00	6.00	310.87	439.34	8.33	0.0015	0.802	0.2369	0.0106	-0.0355	0.7853	-0.0030	
Y663	150.00	150.00	6.00	6.00	310.87	439.34	24.99	0.0015	0.802	0.2369	0.0106	-0.0355	0.7853	-0.0030	
Y666	150.00	150.00	6.00	6.00	310.87	439.34	49.98	0.0015	0.802	0.2369	0.0106	-0.0355	0.7853	-0.0030	
Y691	150.00	150.00	6.00	9.00	291.75	437.87	9.71	0.0014	0.689	0.0988	0.0133	-0.0082	0.8627	-0.0030	
Y693	150.00	150.00	6.00	9.00	291.75	437.87	29.12	0.0014	0.689	0.0988	0.0133	-0.0082	0.8627	-0.0030	
Y696	150.00	150.00	6.00	9.00	291.75	437.87	58.25	0.0014	0.689	0.0988	0.0133	-0.0082	0.8627	-0.0030	
Y861	150.00	150.00	6.00	6.00	382.46	467.78	10.25	0.0019	0.987	0.2915	0.0117	-0.0534	0.7548	-0.0030	
Y863	150.00	150.00	6.00	6.00	382.46	467.78	30.75	0.0019	0.987	0.2915	0.0117	-0.0534	0.7548	-0.0030	
Y866	150.00	150.00	6.00	6.00	382.46	467.78	61.50	0.0019	0.987	0.2915	0.0117	-0.0534	0.7548	-0.0030	
Y891	150.00	150.00	6.00	9.00	377.56	463.36	13.27	0.0018	0.891	0.1279	0.0135	-0.0135	0.8464	-0.0030	
Y893	150.00	150.00	6.00	9.00	377.56	463.36	39.81	0.0018	0.891	0.1279	0.0135	-0.0135	0.8464	-0.0030	
Y896	150.00	150.00	6.00	9.00	377.56	463.36	79.62	0.0018	0.891	0.1279	0.0135	-0.0135	0.8464	-0.0030	

Table A.2 Parameters of calibrated circular hollow steel tubular members

Specimen	No.	Loading	D/T	D [mm]	T [mm]	σ_y	$n (=N/N_y)$
						[N/mm ²]	
Ochi et al.	ER6037	Type2	36.96	165.6	4.48	539.4	0
	ER6030	Type2	29.93	166.1	5.55	549.2	0
	SH6037	Type2	37.52	164.7	4.39	386.4	0
	SH6030	Type2	29.93	164.6	5.5	376.6	0
	201-c	Type3	43.58	165.6	3.8	473.7	0.4
	304-c	Type3	41.92	165.6	3.95	483.5	0
Tsuda et al.	I-401	Type2	39.94	139.8	3.5	414.8	0.07
	I-403	Type2	39.94	139.8	3.5	414.8	0.21
	I-405	Type2	39.94	139.8	3.5	414.8	0.35
	I-601	Type2	60.78	139.8	2.3	343.2328	0.07
	I-603	Type2	60.78	139.8	2.3	343.2328	0.21
	I-605	Type2	60.78	139.8	2.3	343.2328	0.35
	I-901	Type2	87.38	139.8	1.6	365.788	0.07
	I-903	Type2	87.38	139.8	1.6	365.788	0.21
	I-905	Type2	87.38	139.8	1.6	365.788	0.35
	II-401	Type3	39.94	139.8	3.5	414.8213	0.07
	II-403	Type3	39.94	139.8	3.5	414.8213	0.21
	II-601	Type3	60.78	139.8	2.3	343.2328	0.07
	II-603	Type3	60.78	139.8	2.3	343.2328	0.21
	II-605	Type3	60.78	139.8	2.3	343.2328	0.35
	II-901	Type3	87.38	139.8	1.6	365.788	0.07

Tsuji	PM4035	Type2	32.66	114.3	3.5	328.5228	0
	PM6035	Type2	32.66	114.3	3.5	328.5228	0
	PM8035	Type2	32.66	114.3	3.5	328.5228	0
	PM4235	Type2	32.66	114.3	3.5	328.5228	0.2
	PM6235	Type2	32.66	114.3	3.5	328.5228	0.2
	PM8235	Type2	32.66	114.3	3.5	328.5228	0.2
	PM6045	Type2	25.40	114.3	4.5	355.9814	0
	PM8045	Type2	25.40	114.3	4.5	355.9814	0
	PM4245	Type2	25.40	114.3	4.5	355.9814	0.2
	PM6245	Type2	25.40	114.3	4.5	355.9814	0.2
	PC4235	Type3	32.66	114.3	3.5	328.5228	0.2
	PC6235	Type3	32.66	114.3	3.5	328.5228	0.2
	PC8235	Type3	32.66	114.3	3.5	328.5228	0.2
	PC4245	Type3	25.40	114.3	4.5	355.9814	0.2
	PC6245	Type3	25.40	114.3	4.5	355.9814	0.2
	PC8245	Type3	25.40	114.3	4.5	355.9814	0.2
Yasui	D17-1	Type4	17.03	216.3	12.7	345	0.3
	D28-1	Type4	27.38	216.3	7.9	351	0.3
	E27-1	Type4	27.47	114.3	4.16	382	0.3
	E42-1	Type4	42.43	139.6	3.29	347	0.3
	D17-2	Type5	17.03	216.3	12.7	345	0.3
	D28-2	Type5	27.38	216.3	7.9	351	0
	D39-1	Type5	39.19	217.1	5.54	381	0.3
	E27-2	Type5	27.48	114.3	4.16	382	0.3
	E27-3	Type5	27.48	114.3	4.16	382	0.3
	E27-5	Type5	27.48	114.3	4.16	382	0.3
	E42-3	Type5	42.43	139.6	3.29	347	0.3

Table A.3 Parameters of calibrated circular CFT members

Test	Number	Geometrical properties			Material properties		Axial load level
		D/T	$T(\text{mm})$	$L(\text{mm})$	$f_c(\text{MPa})$	$\sigma_y(\text{MPa})$	n
Elremaily et al.	6	34/51	6.4/9.5	914	40/70/100	372	0.2~0.4
Xiao et al.	1	112	3	1500	39.1	303	0.47
Inai et al.	2	53	4.5	1440	40/90	276	0.4
Fujinaga et al.	21	33/39/77	3.01/2.55/1.25	500	30.9~33.5	406/391/410	0.2~0.8
	6						0.3, 0.5, 0.7
Summary	36	33~112	1.25~9.5	500~1500	30.9~100	276~449	0.2~0.8

Table A.4 Parameters of calibrated square hollow steel tubular members

Specimen	B [mm]	T [mm]	σ_y [N/mm ²]	σ_u [N/mm ²]	n (=N/N _y)	ε_y	α	ε_b	τ_{lb1}	r_{lb}	τ_{lb2}
221	100	4.45	385.4	421.7	0.10	0.0019	0.9494	0.0150	-0.0176	0.7350	-0.005
223	100	4.45	385.4	421.7	0.30	0.0019	0.9494	0.0150	-0.0176	0.7350	-0.005
225	100	4.45	385.4	421.7	0.50	0.0019	0.9494	0.0150	-0.0176	0.7350	-0.005

251	150	5.80	456.0	516.8	0.10	0.0022	1.4878	0.0103	-0.0360	0.6925	-0.005
253	150	5.80	456.0	516.8	0.30	0.0022	1.4878	0.0103	-0.0360	0.6925	-0.005
311	100	3.09	419.7	473.7	0.10	0.0020	2.1443	0.0058	-0.0694	0.6406	-0.005
313	100	3.09	419.7	473.7	0.30	0.0020	2.1443	0.0058	-0.0694	0.6406	-0.005
331	150	4.29	440.3	502.1	0.10	0.0021	2.6259	0.0045	-0.1015	0.6026	-0.005
333	150	4.29	440.3	502.1	0.30	0.0021	2.6259	0.0045	-0.1015	0.6026	-0.005
335	150	4.29	440.3	502.1	0.50	0.0021	2.6259	0.0045	-0.1015	0.6026	-0.005
471	151	3.17	324.6	464.8	0.10	0.0016	3.5928	0.0019	-0.1857	0.5262	-0.005
473	151	3.17	324.6	464.8	0.30	0.0016	3.5928	0.0019	-0.1857	0.5262	-0.005
473D	151	3.17	324.6	464.8	0.30	0.0016	3.5928	0.0019	-0.1857	0.5262	-0.005
471A	151	3.17	324.6	464.8	0.10	0.0016	3.5928	0.0019	-0.1857	0.5262	-0.005
473A	151	3.17	324.6	464.8	0.30	0.0016	3.5928	0.0019	-0.1857	0.5262	-0.005
471(27)	150	3.27	379.5	478.6	0.10	0.0019	3.8955	0.0019	-0.2175	0.5023	-0.005
473(28)	150	3.27	379.5	450.1	0.30	0.0019	3.8955	0.0019	-0.2175	0.5023	-0.005
601	150	2.51	386.4	450.1	0.10	0.0019	6.7313	0.0002	-0.6393	0.2782	-0.005
603	150	2.51	386.4	450.1	0.30	0.0019	6.7313	0.0002	-0.6393	0.2782	-0.005
751	150	2.00	355.0	433.5	0.10	0.0017	9.7409	-0.0005	-1.3334	0.0405	-0.005
753	150	2.00	355.0	433.5	0.30	0.0017	9.7409	-0.0005	-1.3334	0.0405	-0.005
941	150	1.58	356.0	445.2	0.10	0.0017	15.6510	-0.0011	-3.4343	-0.4264	-0.005
943	150	1.58	356.0	445.2	0.30	0.0017	15.6510	-0.0011	-3.4343	-0.4264	-0.005
253C	150	5.80	456.0	516.8	0.30	0.0022	1.4878	0.0103	-0.0360	0.6925	-0.005
331C	150	4.29	440.3	502.1	0.10	0.0021	2.6259	0.0045	-0.1015	0.6026	-0.005
333C	150	4.29	440.3	502.1	0.30	0.0021	2.6259	0.0045	-0.1015	0.6026	-0.005
335C	150	4.29	440.3	502.1	0.45	0.0021	2.6259	0.0045	-0.1015	0.6026	-0.005
471C	151	3.17	324.6	464.8	0.10	0.0016	3.5928	0.0019	-0.1857	0.5262	-0.005
473C	151	3.17	324.6	464.8	0.30	0.0016	3.5928	0.0019	-0.1857	0.5262	-0.005
NO1	150	6.00	318.7	455.3	0.00	0.0016	0.9717	0.0121	-0.0182	0.7332	-0.005
NO2	200	6.00	333.4	476.3	0.00	0.0016	1.8072	0.0059	-0.0507	0.6672	-0.005
NO3	200	6.00	333.4	476.3	0.00	0.0016	1.8072	0.0059	-0.0507	0.6672	-0.005
NO4	200	6.00	338.3	483.3	0.00	0.0017	1.8338	0.0058	-0.0521	0.6651	-0.005
NO5	250	6.00	318.7	455.3	0.00	0.0016	2.6992	0.0031	-0.1070	0.5968	-0.005

Table A.5 Parameters of calibrated square CFT members

Test	Specimen numbers	Geometrical properties			Material properties		Axial load level
		B/T	$T(\text{mm})$	$L(\text{mm})$	$f_c(\text{MPa})$	$\sigma_y(\text{MPa})$	
Ricles et al.	4	32/48	8.9/6.1	1500	110	600/660	0.1/0.2
Inai et al.	4	35/47	6.0/4.5	1260	35.5~42.4/ 84.5~94.5	295/276	0.4
Han	2	35/42	2.86	300/360	48.3	228	-
Matsui et al.	21	22/31/43	4.40/3.21/2.24	500	34.1~36.3	407/449/415	0.2~0.8
	6						0.3, 0.5
Summary	37	22~48	2.24~8.9	300~1500	34.1~94.5	228~660	0.1~0.8

APPENDIX B

DETAILS OF EXISTENT HIGH-RISE STEEL BUILDING MODELS

Appendix B.1 includes the seismic design for various existent high-rise steel buildings. Besides, the section, and deterioration parameters of the inside steel members (i.e., H-shaped steel girder, square hollow steel tubular column) are also incorporated.

Based on the B/t criterion (FA, FB) for steel members, details including the section, deterioration parameters of the H-shaped steel girders and exterior square hollow steel tubular columns are shown in Table B.1~3.

Table B.1 FA, FB H-shaped steel girders in existent high-rise steel buildings

Story level	Sectional parameter		α_f	α_w	Deterioration model parameter									
		$D \times B \times t_w \times t_f$ (mm)			E_s	σ_y	R_{ini}	σ_u	R_u	Ψ	ε_{lb}	τ_{lb}	γ_{spm}	τ_{re}
FA	37~RF	750×300×16×28	0.045	2.983	205000	325	10	490	0.9	0.8	0.014	-0.043	0.77	-0.003
	34~36	800×350×16×28	0.062	3.428	205000	325	10	490	0.9	0.8	0.009	-0.057	0.73	-0.003
	31~33	800×350×19×36	0.037	2.327	205000	325	10	490	0.9	0.8	0.019	-0.026	0.82	-0.003
	28~30	800×400×19×36	0.049	2.327	205000	325	10	490	0.9	0.8	0.014	-0.027	0.81	-0.003
	25~27	850×400×19×36	0.049	2.658	205000	325	10	490	0.9	0.8	0.013	-0.034	0.79	-0.003
	2~24	850×400×19×36	0.049	2.658	205000	325	10	490	0.9	0.8	0.013	-0.034	0.79	-0.003
FB	37~RF	800×300×14×28	0.045	4.477	205000	325	10	490	0.9	0.8	0.013	-0.094	0.68	-0.003
	34~36	800×400×14×28	0.081	4.477	205000	325	10	490	0.9	0.8	0.005	-0.096	0.66	-0.003
	31~33	900×350×16×32	0.047	4.328	205000	325	10	490	0.9	0.8	0.012	-0.088	0.69	-0.003
	28~30	900×400×16×32	0.062	4.328	205000	325	10	490	0.9	0.8	0.009	-0.089	0.68	-0.003
	25~27	1000×400×19×25	0.101	3.963	205000	325	10	490	0.9	0.8	0.004	-0.079	0.68	-0.003
	2~24	1000×400×19×25	0.101	3.963	205000	325	10	490	0.9	0.8	0.004	-0.079	0.68	-0.003

Table B.2 FA, FB exterior columns in existent high-rise steel buildings

Story level		Sectional parameter				Deterioration model parameter									
		$B \times t$ (mm)	B/t	ε_y		α	E_s	σ_y	R_{ini}	σ_u	R_u	Ψ	ε_{lb}	τ_{lb}	γ_{spm}
FA	37~40	□600×28	21.4	0.002	0.728	205000	325	10	490	0.9	0.8	0.017	-0.012	0.75	-0.005
	34~36	□600×32	18.8	0.002	0.557	205000	325	10	490	0.9	0.8	0.023	-0.009	0.77	-0.005
	31~33	□600×36	16.7	0.002	0.440	205000	325	10	490	0.9	0.8	0.029	-0.008	0.78	-0.005
	28~30	□650×40	16.3	0.002	0.419	205000	325	10	490	0.9	0.8	0.031	-0.007	0.78	-0.005
	25~27	□700×36	19.4	0.002	0.599	205000	325	10	490	0.9	0.8	0.021	-0.010	0.76	-0.005
	22~24	□700×36	19.4	0.002	0.599	205000	325	10	490	0.9	0.8	0.021	-0.010	0.76	-0.005
	19~21	□700×36	19.4	0.002	0.599	205000	325	10	490	0.9	0.8	0.021	-0.010	0.76	-0.005
	16~18	□700×36	19.4	0.002	0.599	205000	325	10	490	0.9	0.8	0.021	-0.010	0.76	-0.005
	13~15	□700×36	19.4	0.002	0.599	205000	325	10	490	0.9	0.8	0.021	-0.010	0.76	-0.005
	10~12	□700×36	19.4	0.002	0.599	205000	325	10	490	0.9	0.8	0.021	-0.010	0.76	-0.005
	7~9	□700×40	17.5	0.002	0.486	205000	325	10	490	0.9	0.8	0.027	-0.008	0.77	-0.005
	4~6	□700×40	17.5	0.002	0.486	205000	325	10	490	0.9	0.8	0.027	-0.008	0.77	-0.005
	1~3	□700×45	15.6	0.002	0.384	205000	325	10	490	0.9	0.8	0.034	-0.007	0.78	-0.005
FB	37~40	□650×22	29.5	0.002	1.384	205000	325	10	490	0.9	0.8	0.008	-0.032	0.70	-0.005
	34~36	□750×25	30.0	0.002	1.427	205000	325	10	490	0.9	0.8	0.008	-0.034	0.70	-0.005
	31~33	□800×25	32.0	0.002	1.623	205000	325	10	490	0.9	0.8	0.007	-0.042	0.68	-0.005
	28~30	□800×25	32.0	0.002	1.623	205000	325	10	490	0.9	0.8	0.007	-0.042	0.68	-0.005

25~27	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
22~24	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
19~21	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
16~18	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
13~15	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
10~12	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
7~9	□850×28	30.4	0.002	1.461	205000	325	10	490	0.9	0.8	0.008	-0.035	0.69	-0.005
4~6	□850×28	30.4	0.002	1.461	205000	325	10	490	0.9	0.8	0.008	-0.035	0.69	-0.005
1~3	□850×28	30.4	0.002	1.461	205000	325	10	490	0.9	0.8	0.008	-0.035	0.69	-0.005

Table B.3 FA, FB interior columns in existent high-rise steel buildings

Story level		Sectional parameter				Deterioration model parameter									
		B×t (mm)	B/t	ε _y	α	E _s	σ _y	R _{ini}	σ _u	R _u	Ψ	ε _{lb}	τ _{lb}	γ _{spm}	τ _{lb2}
FA	37~40	□600×28	21.4	0.002	0.728	205000	325	10	490	0.9	0.8	0.017	-0.012	0.75	-0.005
	34~36	□600×32	18.8	0.002	0.557	205000	325	10	490	0.9	0.8	0.023	-0.009	0.77	-0.005
	31~33	□600×40	15.0	0.002	0.357	205000	325	10	490	0.9	0.8	0.037	-0.007	0.78	-0.005
	28~30	□650×40	16.3	0.002	0.419	205000	325	10	490	0.9	0.8	0.031	-0.007	0.78	-0.005
	25~27	□650×45	14.4	0.002	0.331	205000	325	10	490	0.9	0.8	0.040	-0.007	0.78	-0.005
	22~24	□650×45	14.4	0.002	0.331	205000	325	10	490	0.9	0.8	0.040	-0.007	0.78	-0.005
	19~21	□650×45	14.4	0.002	0.331	205000	325	10	490	0.9	0.8	0.040	-0.007	0.78	-0.005
	16~18	□650×50	13.0	0.002	0.268	205000	325	10	490	0.9	0.8	0.050	-0.006	0.79	-0.005
	13~15	□650×50	13.0	0.002	0.268	205000	325	10	490	0.9	0.8	0.050	-0.006	0.79	-0.005
	10~12	□700×45	15.6	0.002	0.384	205000	325	10	490	0.9	0.8	0.034	-0.007	0.78	-0.005
	7~9	□700×50	14.0	0.002	0.311	205000	325	10	490	0.9	0.8	0.042	-0.006	0.79	-0.005
	4~6	□750×45	16.7	0.002	0.440	205000	325	10	490	0.9	0.8	0.029	-0.008	0.78	-0.005
1~3	□750×50	15.0	0.002	0.357	205000	325	10	490	0.9	0.8	0.037	-0.007	0.78	-0.005	
FB	37~40	□650×22	29.5	0.002	1.384	205000	325	10	490	0.9	0.8	0.008	-0.032	0.70	-0.005
	34~36	□750×25	30.0	0.002	1.427	205000	325	10	490	0.9	0.8	0.008	-0.034	0.70	-0.005
	31~33	□800×25	32.0	0.002	1.623	205000	325	10	490	0.9	0.8	0.007	-0.042	0.68	-0.005
	28~30	□800×25	32.0	0.002	1.623	205000	325	10	490	0.9	0.8	0.007	-0.042	0.68	-0.005
	25~27	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
	22~24	□800×28	28.6	0.002	1.294	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	-0.005
	19~21	□850×28	30.4	0.002	1.461	205000	325	10	490	0.9	0.8	0.008	-0.035	0.69	-0.005
	16~18	□850×28	30.4	0.002	1.461	205000	325	10	490	0.9	0.8	0.008	-0.035	0.69	-0.005
	13~15	850×28	30.4	0.002	1.461	205000	325	10	490	0.9	0.8	0.008	-0.035	0.69	-0.005
	10~12	900×32	28.1	0.002	1.254	205000	325	10	490	0.9	0.8	0.009	-0.027	0.71	-0.005
	7~9	900×32	28.1	0.002	1.254	205000	325	10	490	0.9	0.8	0.009	-0.027	0.71	-0.005
	4~6	900×32	28.1	0.002	1.254	205000	325	10	490	0.9	0.8	0.009	-0.027	0.71	-0.005
1~3	950×32	29.7	0.002	1.397	205000	325	10	490	0.9	0.8	0.008	-0.032	0.70	-0.005	

Width-thickness ratios (B/t for rectangular steel tubular columns, D/t_w and B/t_f for H-shaped steel girders) of the structural components of various 40S high-rise steel building models are demonstrated as an example.

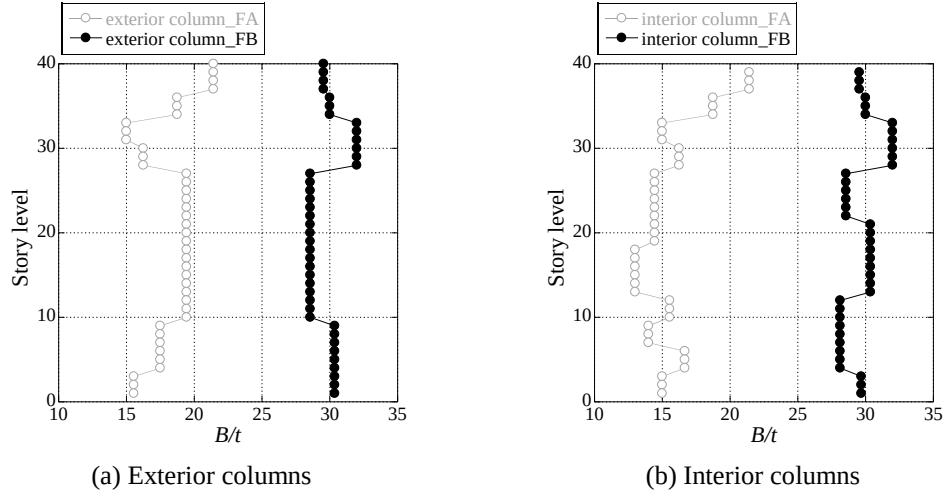


Figure B.1 Width-thickness ratios of exterior and interior columns

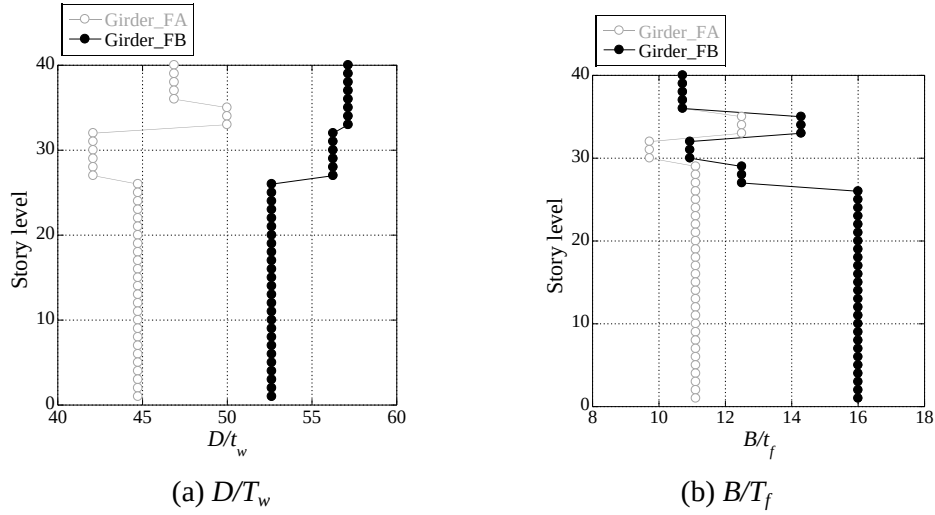


Figure B.2 Width-thickness ratios of girders

Axial load ratios of the interior and exterior columns (n) of 40S high-rise steel building models along with the vertical direction are demonstrated below.

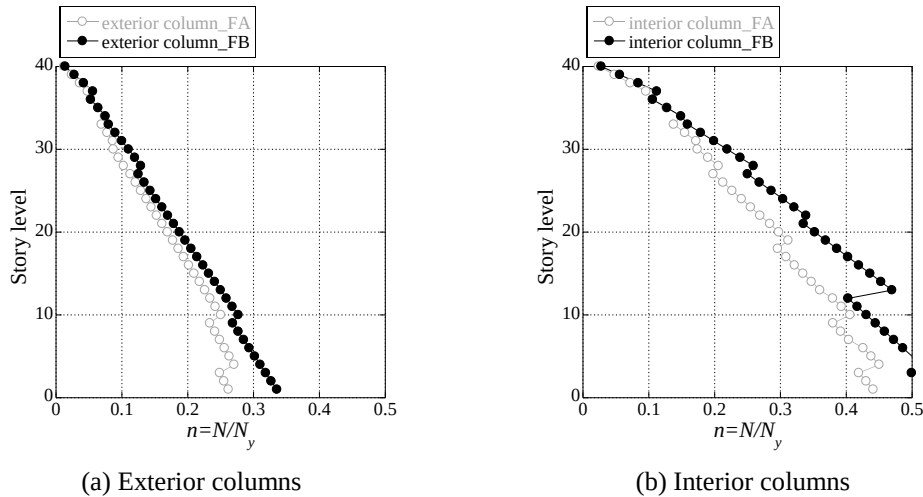


Figure B.3 Axial load ratios of exterior and interior columns

In order to calculate the column over-strength factor (COF) of for the exterior and interior beam-column connections, full plastic bending moment of the structural components of high-rise building models are calculated by

$$M = \frac{3EI}{l} \theta \quad (B.1)$$

For rectangular section:

$$M_p = Z_p \cdot \sigma_y = \frac{BD^2}{4} \cdot \sigma_y \quad (B.2)$$

$$M_y = Z \cdot \sigma_y = \frac{BD^2}{6} \cdot \sigma_y$$

The COF coefficients of the 40S high-rise building models are respectively shown below.

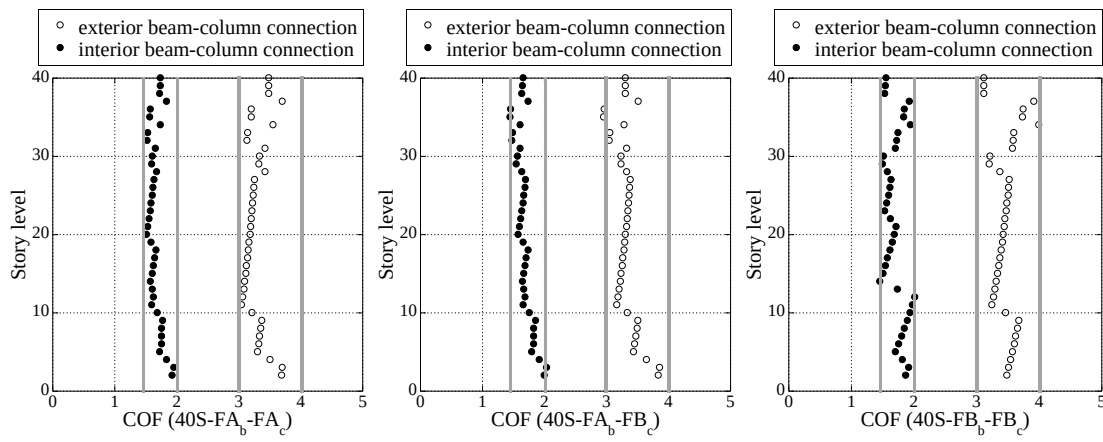


Figure B.4 COF of various high-rise steel buildings (40 floor model)

The Static designs of the high-rise steel building models are shown below.

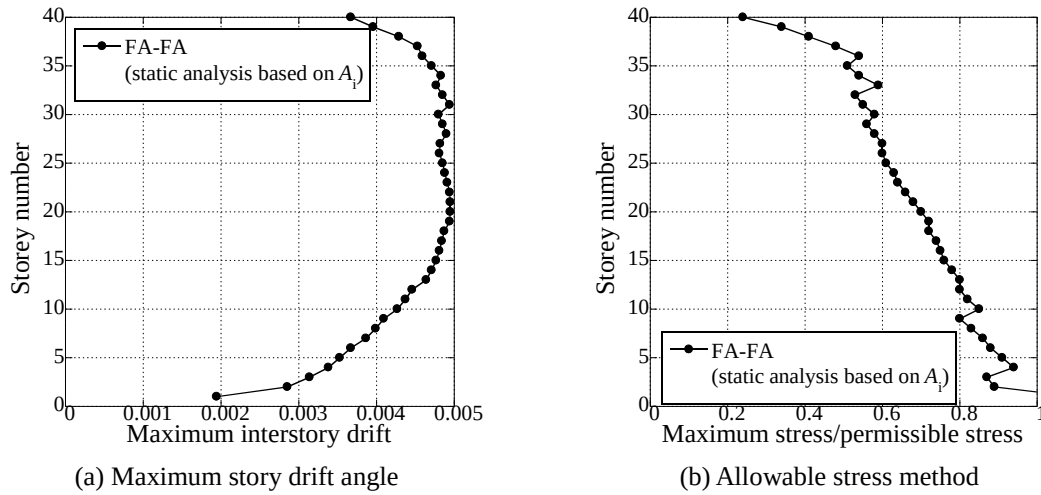


Figure B.5 Static elastic design of FA-FA rank frame

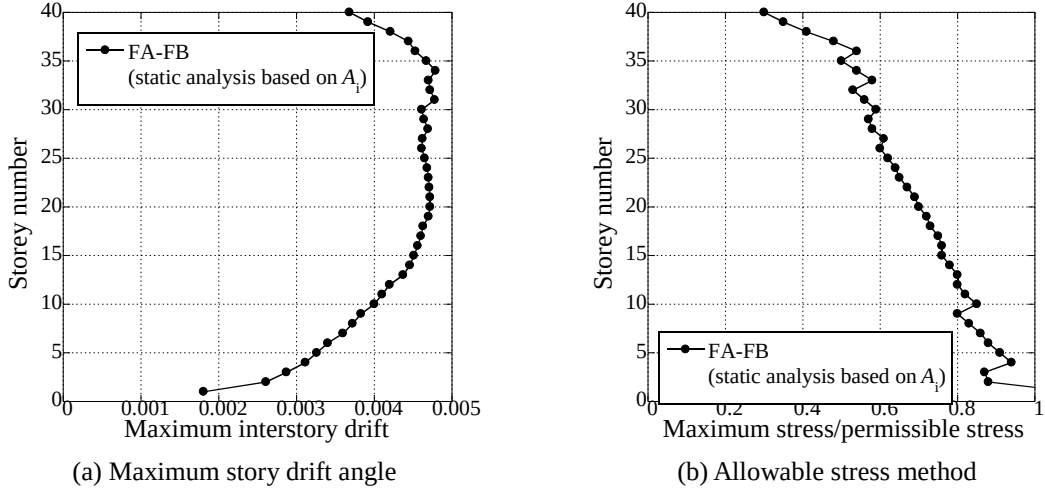


Figure B.6 Static elastic design of FA-FB rank frame

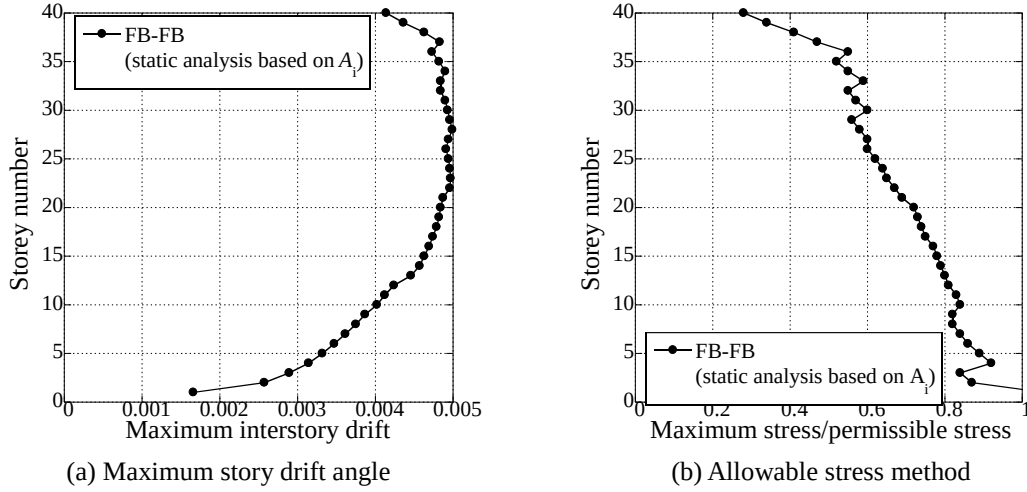


Figure B.7 Static elastic design of FA-FB rank frame

In addition, the dynamic designs (nonlinear time history analysis) of the 40S building models based on Level 1 and Level 2 are shown below.

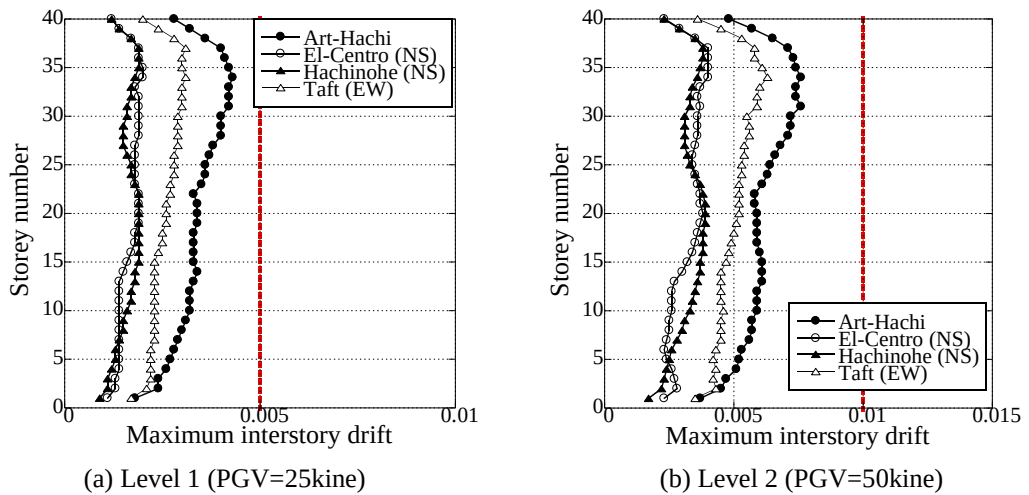
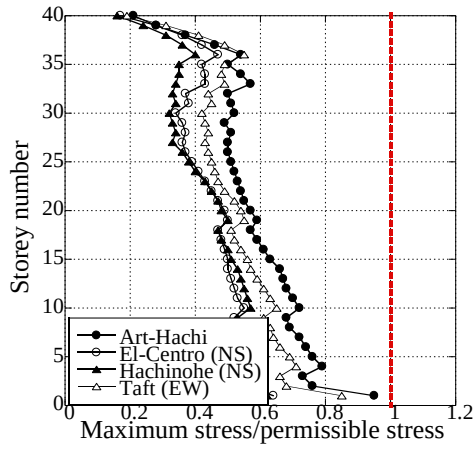
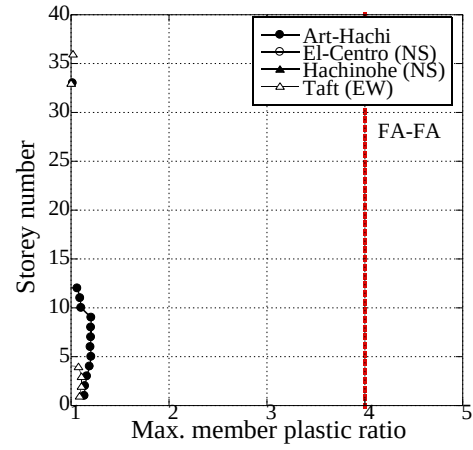


Figure B.8 Dynamic design of FA-FA rank frame building based on time-history analysis

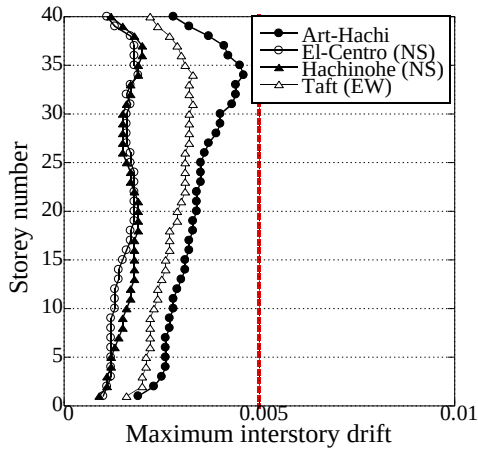


(a) Calibration of allowable stress ratio

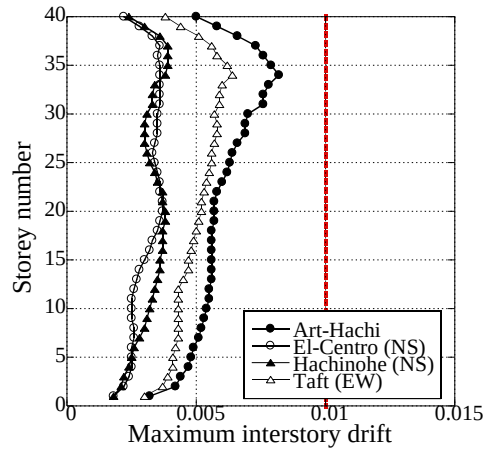


(b) Calibration of member plastic ratio

Figure B.9 Dynamic design of FA-FA rank frame building based on time-history analysis

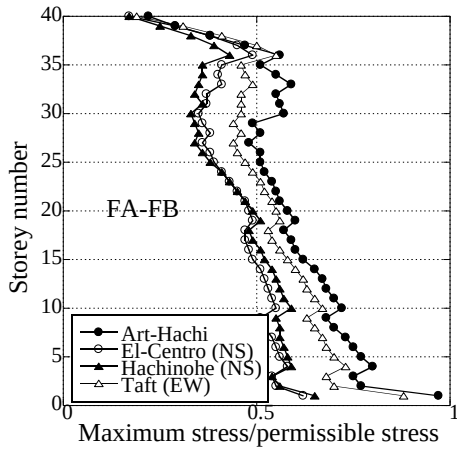


(a) Level 1 (PGV=25kine)

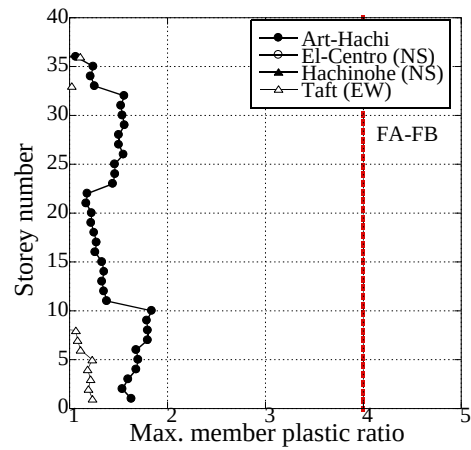


(b) Level 2 (PGV=50kine)

Figure B.10 Dynamic design of FA-FB rank frame building based on time-history analysis



(a) Calibration of allowable stress ratio



(b) Calibration of member plastic ratio

Figure B.11 Dynamic design of FA-FB rank frame building based on time-history analysis

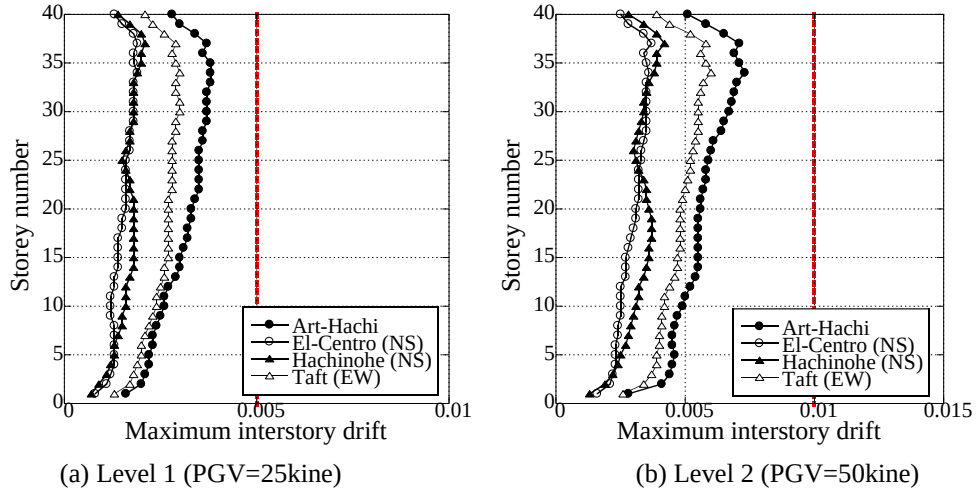


Figure B.12 Dynamic design of FA-FB rank frame building based on time-history analysis

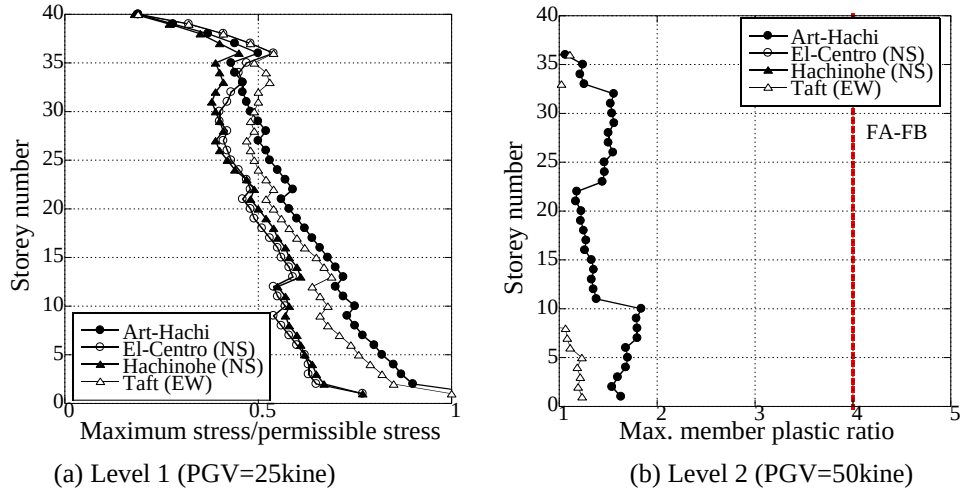


Figure B.13 Dynamic design of FA-FB rank frame building based on time-history analysis

Time history analysis

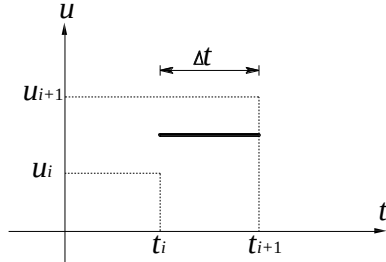
Newmark's method,

In 1959, N. M. Newmark developed a family of time-stepping methods based on the following equations:

$$\dot{u}_{i+1} = \dot{u}_i + [(1-\gamma)\Delta t]\ddot{u}_i + (\gamma\Delta t)\ddot{u}_{i+1}$$

$$u_{i+1} = u_i + (\Delta t)\dot{u}_i + [(0.5-\beta)(\Delta t)^2]\ddot{u}_i + [\beta(\Delta t)^2]\ddot{u}_{i+1}$$

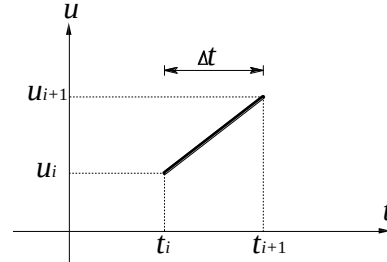
The parameter β and γ define the variation of acceleration over a time step and determine the stability and accuracy characteristics of the method. Typical selection for γ is $\frac{1}{2}$, and $\frac{1}{6} \leq \beta \leq \frac{1}{4}$ is satisfactory from all points of view, including that of accuracy.



Average acceleration

$$\gamma = \frac{1}{2}$$

$$\beta = \frac{1}{4}$$



Linear acceleration

$$\gamma = \frac{1}{2}$$

$$\beta = \frac{1}{6}$$

APPENDIX C

DETAILS OF EXISTENT HIGH-RISE CONCRETE FILLED STEEL BUILDINGS

Appendix C includes the details of existent high-rise CFT buildings. Based on the analogous flexural stiffness between square hollow steel tubular column and the associated CFT column, ductile (compact) and non-ductile (uncompact) square CFT columns are respectively assumed in terms of width-thickness ratio criteria for square CFT columns. For instant, [Tables C.1~C.2](#) respectively show the details of the assumed square CFT exterior and interior columns in existent 40-story high-rise CFT building models.

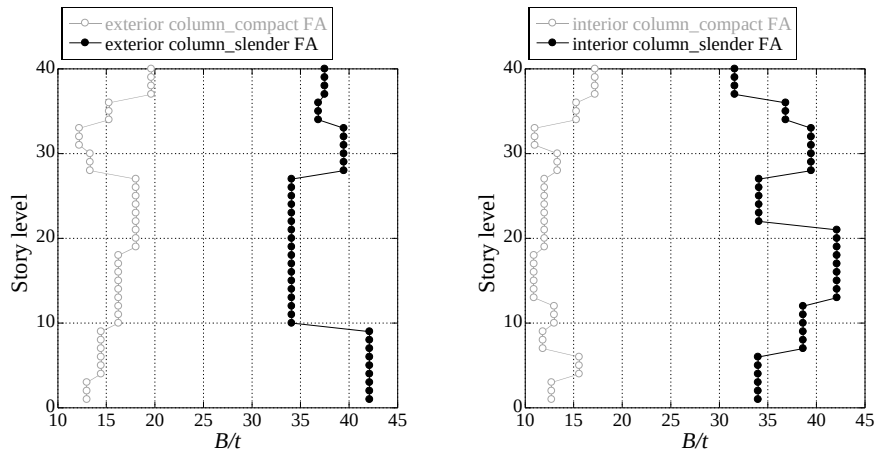
Table C.1 Compact and slender FA rank square CFT exterior columns in high-rise CFT buildings

Story level		Sectional parameter				Deterioration model parameter									
		$B \times t$ (mm)	B/t	ε_y	α	E_s	σ_y	R_{ini}	σ_u	R_u	Ψ	ε_{lb}	τ_{lb}	γ_{spm}	τ_{lb2}
Compact FA	37~40	□550×28	19.6	0.002	0.351	205000	325	10	490	0.9	0.8	0.044	-0.007	0.78	0
	34~36	□550×36	15.3	0.002	0.212	205000	325	10	490	0.9	0.8	0.088	-0.006	0.79	0
	31~33	□550×45	12.2	0.002	0.136	205000	325	10	490	0.9	0.8	0.162	-0.005	0.80	0
	28~30	□600×45	13.3	0.002	0.162	205000	325	10	490	0.9	0.8	0.128	-0.005	0.80	0
	25~27	□650×36	18.1	0.002	0.297	205000	325	10	490	0.9	0.8	0.056	-0.006	0.79	0
	22~24	□650×36	18.1	0.002	0.297	205000	325	10	490	0.9	0.8	0.056	-0.006	0.79	0
	19~21	□650×36	18.1	0.002	0.297	205000	325	10	490	0.9	0.8	0.056	-0.006	0.79	0
	16~18	□650×40	16.3	0.002	0.240	205000	325	10	490	0.9	0.8	0.074	-0.006	0.79	0
	13~15	□650×40	16.3	0.002	0.240	205000	325	10	490	0.9	0.8	0.074	-0.006	0.79	0
	10~12	□650×40	16.3	0.002	0.240	205000	325	10	490	0.9	0.8	0.074	-0.006	0.79	0
	7~9	□650×45	14.4	0.002	0.190	205000	325	10	490	0.9	0.8	0.102	-0.006	0.80	0
	4~6	□650×45	14.4	0.002	0.190	205000	325	10	490	0.9	0.8	0.102	-0.006	0.80	0
1~3	□650×50	13.0	0.002	0.154	205000	325	10	490	0.9	0.8	0.137	-0.005	0.80	0	
Slender FA	37~40	□600×16	37.5	0.002	1.280	205000	325	10	490	0.9	0.8	0.009	-0.028	0.71	0
	34~36	□700×19	36.8	0.002	1.235	205000	325	10	490	0.9	0.8	0.009	-0.026	0.71	0
	31~33	□750×19	39.5	0.002	1.418	205000	325	10	490	0.9	0.8	0.008	-0.033	0.70	0
	28~30	□750×19	39.5	0.002	1.418	205000	325	10	490	0.9	0.8	0.008	-0.033	0.70	0
	25~27	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	22~24	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	19~21	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	16~18	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	13~15	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	10~12	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	7~9	□800×19	42.1	0.002	1.613	205000	325	10	490	0.9	0.8	0.007	-0.041	0.68	0
	4~6	□800×19	42.1	0.002	1.613	205000	325	10	490	0.9	0.8	0.007	-0.041	0.68	0
1~3	□800×19	42.1	0.002	1.613	205000	325	10	490	0.9	0.8	0.007	-0.041	0.68	0	

Table C.2 Compact and slender FA rank square CFT interior columns in high-rise CFT buildings

Story level		Sectional parameter				Deterioration model parameter									
		$B \times t$ (mm)	B/t	ε_y	α	E_s	σ_y	R_{ini}	σ_u	R_u	Ψ	ε_{lb}	τ_{lb}	γ_{spm}	τ_{lb2}
Compact FA	37~40	□550×32	17.2	0.002	0.269	205000	325	10	490	0.9	0.8	0.049	-0.006	0.79	0
	34~36	□550×36	15.3	0.002	0.212	205000	325	10	490	0.9	0.8	0.063	-0.006	0.79	0
	31~33	□550×50	11.0	0.002	0.110	205000	325	10	490	0.9	0.8	0.123	-0.005	0.80	0
	28~30	□600×45	13.3	0.002	0.162	205000	325	10	490	0.9	0.8	0.083	-0.005	0.80	0
	25~27	□600×50	12.0	0.002	0.131	205000	325	10	490	0.9	0.8	0.103	-0.005	0.80	0
	22~24	□600×50	12.0	0.002	0.131	205000	325	10	490	0.9	0.8	0.103	-0.005	0.80	0
	19~21	□600×50	12.0	0.002	0.131	205000	325	10	490	0.9	0.8	0.103	-0.005	0.80	0
	16~18	□600×55	10.9	0.002	0.108	205000	325	10	490	0.9	0.8	0.125	-0.005	0.80	0
	13~15	□600×55	10.9	0.002	0.108	205000	325	10	490	0.9	0.8	0.125	-0.005	0.80	0
	10~12	□650×50	13.0	0.002	0.154	205000	325	10	490	0.9	0.8	0.088	-0.005	0.80	0
	7~9	□650×55	11.8	0.002	0.127	205000	325	10	490	0.9	0.8	0.107	-0.005	0.80	0
	4~6	□700×45	15.6	0.002	0.220	205000	325	10	490	0.9	0.8	0.061	-0.006	0.79	0
	1~3	□700×55	12.7	0.002	0.147	205000	325	10	490	0.9	0.8	0.092	-0.005	0.80	0
Slender FA	37~40	□600×19	31.6	0.002	0.907	205000	325	10	490	0.9	0.8	0.013	-0.017	0.74	0
	34~36	□700×19	36.8	0.002	1.235	205000	325	10	490	0.9	0.8	0.009	-0.026	0.71	0
	31~33	□750×19	39.5	0.002	1.418	205000	325	10	490	0.9	0.8	0.008	-0.033	0.70	0
	28~30	□750×19	39.5	0.002	1.418	205000	325	10	490	0.9	0.8	0.008	-0.033	0.70	0
	25~27	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	22~24	□750×22	34.1	0.002	1.057	205000	325	10	490	0.9	0.8	0.011	-0.021	0.73	0
	19~21	□800×19	42.1	0.002	1.613	205000	325	10	490	0.9	0.8	0.007	-0.041	0.68	0
	16~18	□800×19	42.1	0.002	1.613	205000	325	10	490	0.9	0.8	0.007	-0.041	0.68	0
	13~15	□800×19	42.1	0.002	1.613	205000	325	10	490	0.9	0.8	0.007	-0.041	0.68	0
	10~12	□850×22	38.6	0.002	1.358	205000	325	10	490	0.9	0.8	0.008	-0.031	0.70	0
	7~9	□850×22	38.6	0.002	1.358	205000	325	10	490	0.9	0.8	0.008	-0.031	0.70	0
	4~6	□850×25	34.0	0.002	1.052	205000	325	10	490	0.9	0.8	0.011	-0.020	0.73	0
	1~3	□850×25	34.0	0.002	1.052	205000	325	10	490	0.9	0.8	0.011	-0.020	0.73	0

Geometrical and material properties of existent high-rise CFT buildings are introduced in Appendix C, e.g., Low-deterioration (denoted as Low-det.) and high-deterioration (denoted as high-det.) high-rise CFT building 40-story models are show as follows.

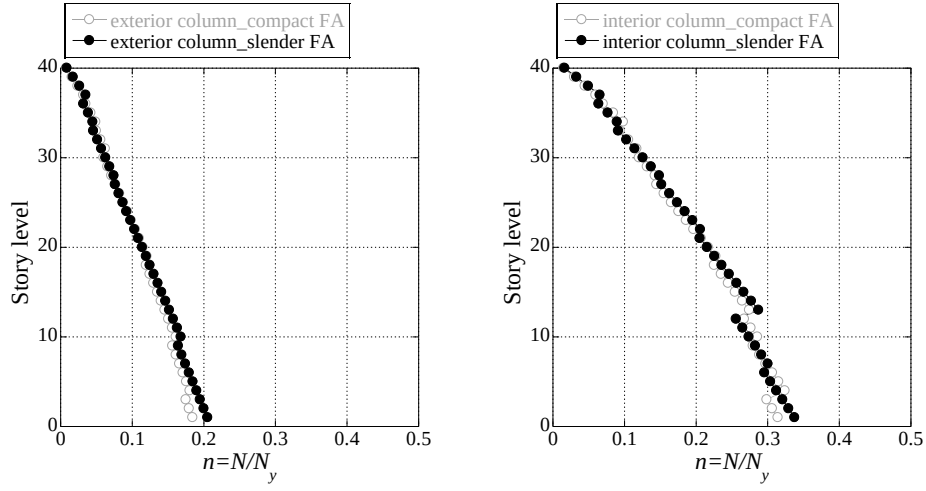


(a) Exterior columns

(b) Interior columns

Figure C.1 Width-thickness ratios for exterior and interior columns

The axial load ratios of the exterior and interior CFT columns in various CFT building models are shown in **Figure C.2**.

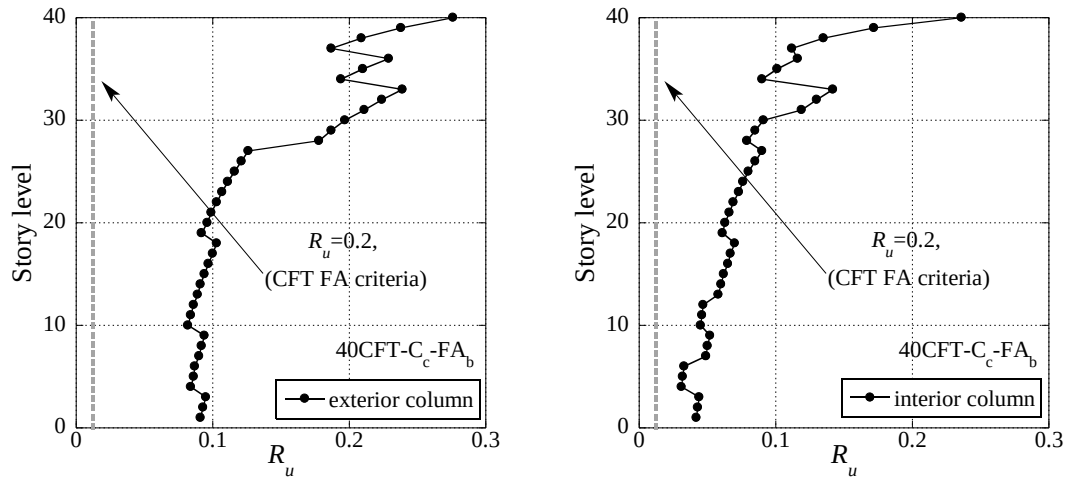


(a) Exterior columns

(b) Interior columns

Figure C.2 Axial load ratios for exterior and interior columns

The ultimate rotation angles R_u of 40CFT-C_c-FA_b building model are shown in **Figure C.3**.

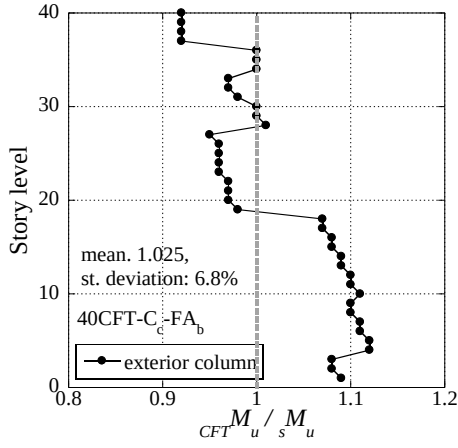


(a) Exterior columns

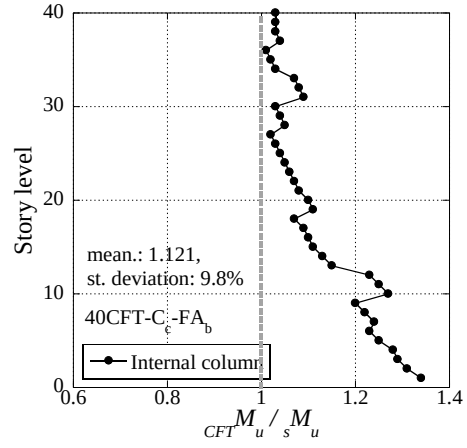
(b) Interior columns

Figure C.3 Ultimate component rotation of CFT columns (axial load incorporated)

Strength ratio (ultimate bending moment M_u) between concrete filled steel tubular columns and the corresponding hollow steel tubular columns in high-rise building models are shown in **Figure C.4**.



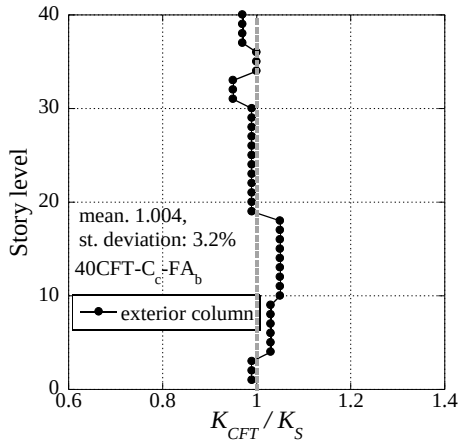
(a) Exterior columns



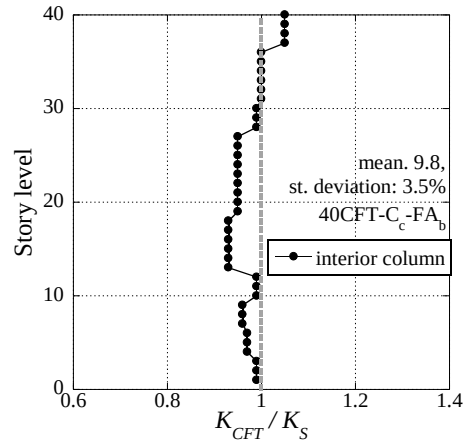
(b) Interior columns

Figure C.4 Full plastic moment (M_u) of concrete filled steel tubular columns in CFT building models

Lateral stiffness ratio between concrete filled steel tubular columns and the corresponding hollow steel tubular columns in high-rise building models are shown in Figure C.5.



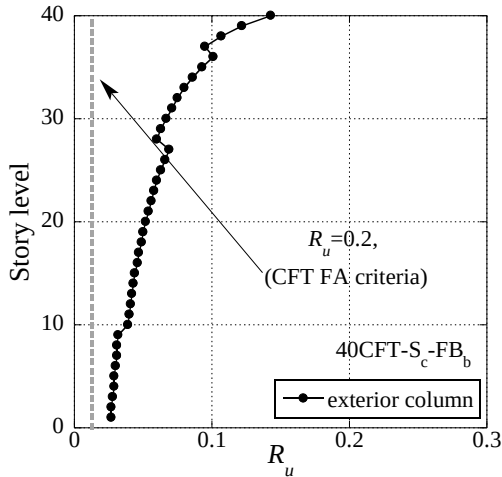
(a) Exterior columns



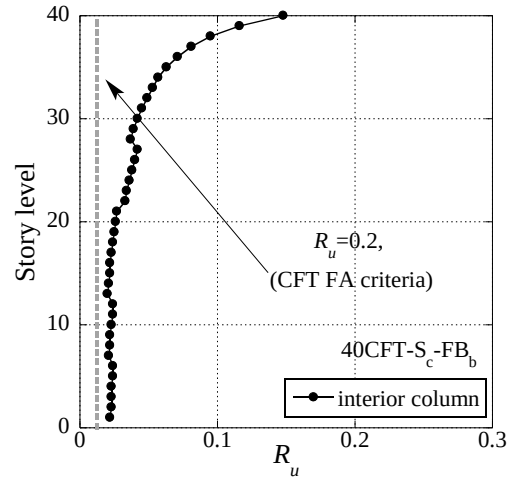
(b) Interior columns

Fig. C.5 Lateral stiffness ($K = (12EI)/l^3$) of concrete filled steel tubular columns in CFT building models

The ultimate rotation angles R_u of 40CFT-S_c-FB_b building model are shown in Figure C.6.



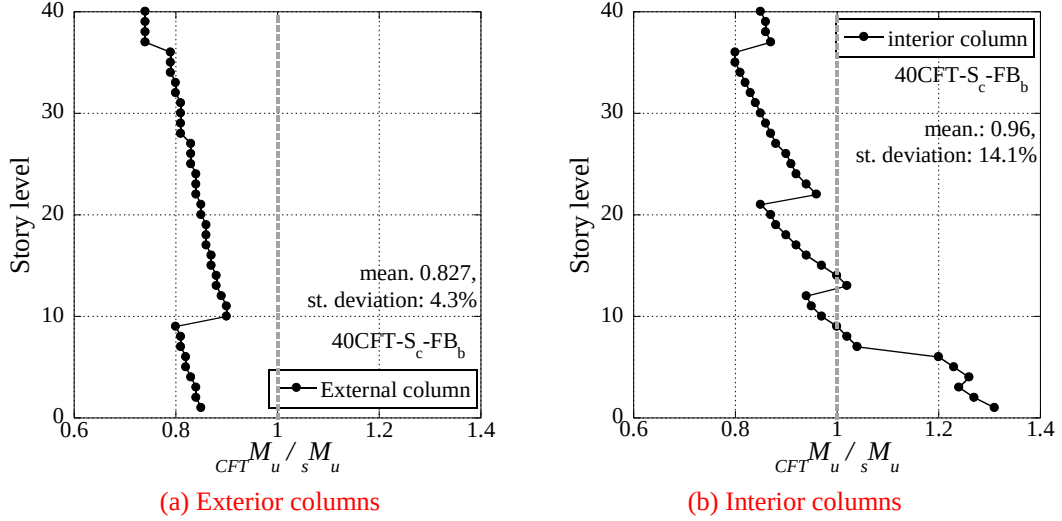
(a) Exterior columns



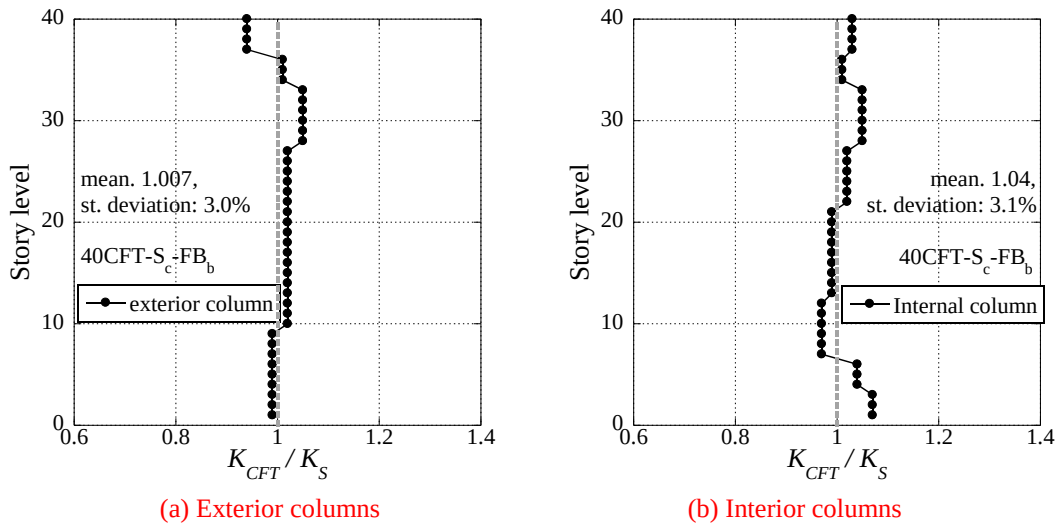
(b) Interior columns

Fig. C.6 Ultimate component rotation of CFT columns (axial load considered)

Strength ratio (ultimate bending moment M_u) between concrete filled steel tubular columns and the corresponding hollow steel tubular columns in high-rise building models are shown in Figure C.7.


Figure C.7 Full plastic moment (M_u) of concrete filled steel tubular columns in CFT-MFR models

Lateral stiffness ratio between concrete filled steel tubular columns and the corresponding hollow steel tubular columns in high-rise building models are shown in Figure C.8.


Figure C.8 Lateral stiffness ($K = (12EI)/l^3$) of concrete filled steel tubular columns in CFT-MFR models

Seismic dynamic design

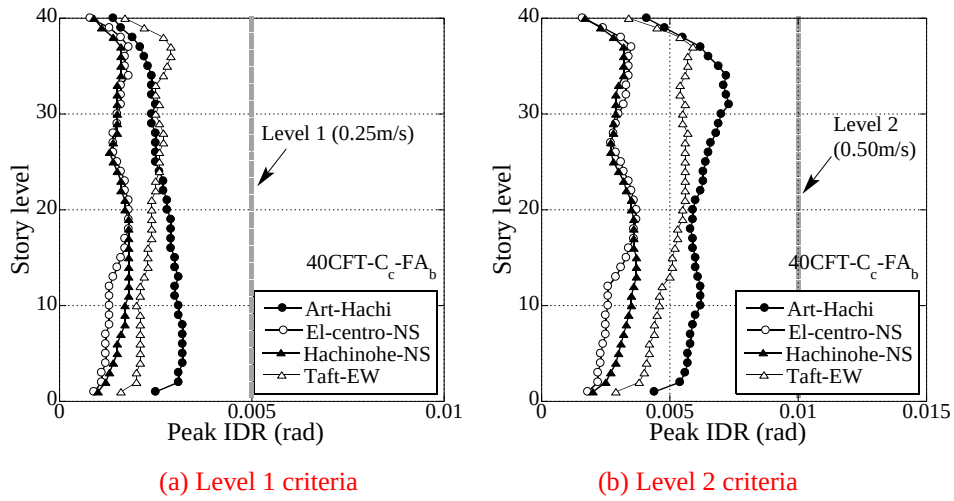


Figure C.9 Dynamic seismic design of low-det. CFT model (40 story)

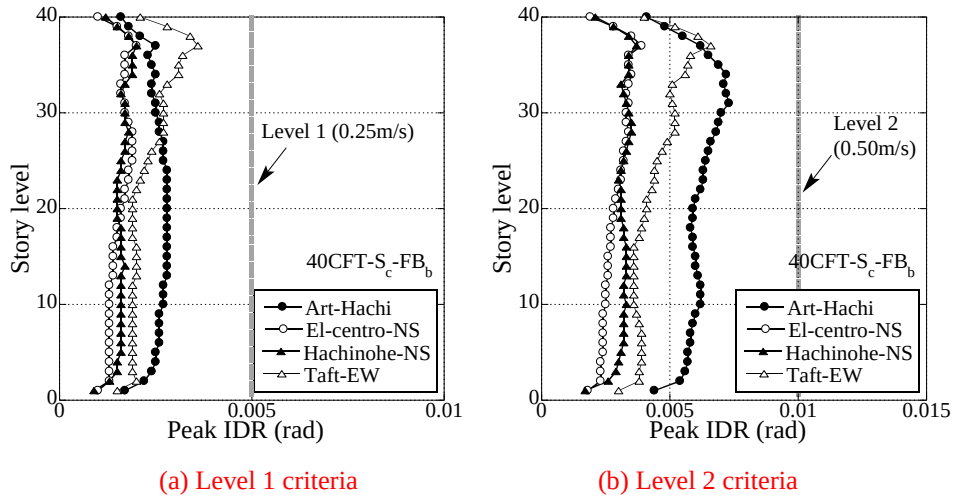


Figure C.10 Dynamic seismic design of high-det. CFT model (40 story)

For the sectional rigidity of CFT column, the ratio of rigidity between HST and CFT column should be designed to be near 1, in order to compare the high-rise steel building models and with the corresponding high-rise concrete filled steel building models. Hence, the sectional rigidity of square hollow steel tubular member and square concrete filled steel tubular member are calculated by

For square hollow steel tubular member:

$$M_{pX} = (1.5 - \frac{t}{D})D(D-t)t\sigma_y \approx \frac{3}{2}(1 - \frac{5t}{3D})D^2t\sigma_y \approx \frac{3}{2}(D-t)^2t\sigma_y$$

$$m_{pnX} = 1 - \frac{4}{3}n^2, (n \leq 0.5)$$

$$m_{pnX} = \frac{4}{3}(1-n), (n > 0.5)$$

$$r_w = 2$$

Where, r_w is the ratio of web area to one-side flange area,

$$r_w = \frac{A_w}{A_f} = 2(1 - \frac{2t}{D})$$

$$m_{pnX} = M_{pnX} / M_{pX}$$

$$M_{p-CFT} / M_{p-HST} \sim \kappa_{p-CFT} / \kappa_{p-HST}$$

For square CFT member:

$$M_u = {}_sM_u + {}_cM_u$$

$${}_sM_u = (1 - \frac{t}{D})D^2 \cdot t \cdot \sigma_y + 2(1 - x_{n1})x_{n1} \cdot {}_cD^2 \cdot t \cdot \sigma_y$$

$${}_cM_u = \frac{1}{2}(1 - x_{n1})x_{n1} \cdot {}_cD^3 \cdot {}_c r_u \cdot F_c$$

Where,

$${}_c r_u = 0.85$$

$$x_{n1} = x_n / {}_cD$$