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NUMERICAL STUDY ON COMPARISON BETWEEN DUCTILITY OF UNSTIFFENEE BOX SECTIONED STEEL STUB COLUMNS AND SIMPLY SUPPORTED STEEL PLATES

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Abstract. *This study is aimed at grasping differences between ductility formulations of unstiffened box sectioned steel stub column subjected to bending moment and simply supported steel plates subjected to axial force. For this purpose, numerical model about steel stub column and simply supported steel plate were pre-pared. At first, ductility formula of simply supported steel plates subjected to axial load were developed. Secondary, strength and ductility as seismic performance were calculated, through numerical analysis subjected to between compression load and bending moment. As a result, it is found that strength of compression flange plate in steel stub column is 1.2 times of simply supported steel plate. However, it is required to carry out more numerical study, for investigating ductility of these structures. Finally, deformation shape of these structures were discussed.*

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1 INTRODUCTION

Civil engineering structures were irreparably damaged by the Kobe earthquake which occurred in the southern part of Hyogo prefecture in 1995, besides buckling and deformation were confirmed on steel bridge piers which had not damaged by any earthquakes before[1]. Therefore, the study became more active to establish performance based design and seismic performance of steel bridge pier after Kobe earthquake.

For example, Fukumoto et al.[2] compiled databases on the results of compressive loading experiments for steel plates which carried out in Europe, Japan and US and analyzed statistically the relation between compressive strength and width-thickness ratio parameter. Usami[3),4] conducted the experiments which coupled the local buckling of box sectioned compressive element and whole buckling. They elucidated especially economical design of piers. Nara[5] estimated the design formulas of limited strength of steel plates subjected to compressive stress and bending by numerical analysis of Finite Element Method.

Recently, seismic design of civil engineering structures is significantly increasing and the structures will get a minor damage by even level 2 earthquake motions such as Hyogo earthquakes in 1995.

Seismic design guidelines[6] for steel bridges are summarized as researches on seismic design of steel bridge piers in recent years. Evaluation formulas for the strength and deformability of the steel bridge piers are included in the guidelines, but the formulas consider only standard initial irregular. That is, the formulas can not evaluate with using especially the amount of initial deflections because the formulas have evaluated the plate elements, which have half sin-wave of initial deflections and the maximum initial deflections of 1/150 of plate width, have configurated steel bridge piers. It is well known in buckling design, the buckling load changes depending on the amount of initial curvatures. However, the existing formulas take into account just standard initial curvatures about strength and deformation capacity of steel bridge piers. The formulas do not have problem with a newly established structure. nowadays it is difficult to compare strength and deformability between before earthquakes and after. Therefore, we did the parametric compressive analysis for simple steel plates and unstiffened box sectioned steel stub columns with amounts of initial curvature as basic examination. Obviously, the result about the study of unstiffened box sectioned steel stub columns is important though, we carried out the analysis of simple steel plates and compere these for evaluating how useful just evaluative formula of the plates is.

2 THE ANALYSIS MODEL OF PLATES

In this paper, assessing strength and deformability of unstiffened box-sectioned steel stub columns is considered with width-thickness ratio parameters. Therefore, the numerical analysis method is typical method. Figure 1 shows schematic diagram of the analysis model, (a) is the front view of the simple steel plate, (b) is side view of them, a is width of plates which is loading side and l_d is length of plates which is not loading side. Moreover, Table 1 shows boundary conditions, 1 indicates

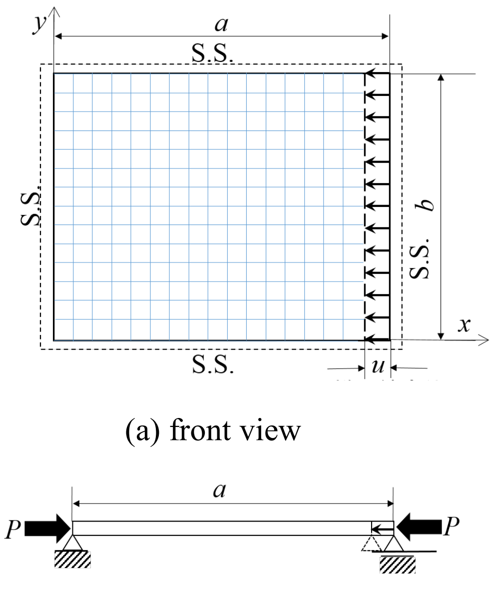


Figure 1 The analysis model

boundary and 0 indicates free. Shell elements are applied to the analyses. the number of divided elements of a and l_d are equal division of 100 in both case. The number of division is decided based on comparing a theoretical value by a buckling eigenvalue analysis. Elastic-plastic finite displacement analysis was conducted by ABAQUS[7)] and S4R (shell elements of 4 nodes) which is generally formulated thickness of plates is used because it is treated from thin plate to thick in this paper. In addition, steel material is SM490 because mostly this material is used in this kind of studies. The effect by differences of steel type should be considered. However, this study is analysis study and the difference of steel material would affect only thicker steel plates, that is the area will not occur local buckling.

2.1 The model of initial curvatures

Initial curvatures of half sin-wave are simulated in numerical analyses[8)]. The amount of maximum initial curvatures label w_{max} as shown in Figure 2. The amount of initial curvatures is calculated by formula (1).

$$w(x,y) = w_{\max} \cdot \sin\left(\frac{\pi x}{2a}\right) \cdot \sin\left(\frac{\pi y}{2l_d}\right) \tag{1}$$

2.2 The model of residual stresses

Occurring residual stresses by welding is supposed, so the form of residual stresses is triangular as shown in Figure 3[9)]. The maximum tensile residual stress σ_{rt} is yield stress σ_y and the maximum compressive residual stress σ_{rc} is 0.25 of σ_y.

2.3 The loading method

Displacement control analyses are adopted as the loading method to basically consider the study of simple steel plates. The displacement is uniform as shown in Figure 1-(a) and cyclic load is not considered in this study.

2.4 Constitutive law

Stress-strain curve has plastic region, yield plateau and strain hardening state as shown in Figure 4. Moreover, each of plastic region, yield plateau and strain hardening state is shown formula (2), (3), (4).

$$\sigma = E\varepsilon \quad (0 \le \varepsilon < \varepsilon_y) \tag{2}$$

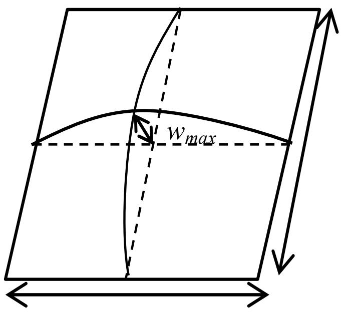


Figure 2 The model of initial curvatures

Table 3 The boundary condition of simple steel plates

	u	v	w	θ_x	θ_y	θ_z
x=0	1	1	1	1	0	1
x=l_d	0	0	1	0	1	1
y=0	0	0	1	0	1	1
y=a	0	0	1	0	1	1
Free=0, Fix=1 u,v,w=deformability of x,y,z $\theta x,\theta y,\theta z$=rotation angle of x,y,z axis						

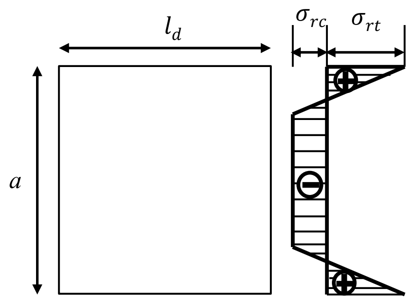


Figure 3 The model of residual stresses

$$\sigma = \sigma_y \quad (\varepsilon_y \le \varepsilon < \varepsilon_{st}) \tag{3}$$

$$\frac{\sigma}{\sigma_y} = \frac{1}{\xi}\frac{E_{st}}{E}\left[1 - \exp\left\{-\xi\left(\frac{\varepsilon}{\varepsilon_y} - \frac{\varepsilon_{st}}{\varepsilon_y}\right)\right\}\right] + 1 \quad (\varepsilon > \varepsilon_{st}) \tag{4}$$

E is young's rate, ε_y is yield stress, ξ is material parameter, E_{st} is strain hardening modulus, ε_{st} is the strain when strain starts to be hard. In SM490 case, $\xi = 0.06$, $E/Est = 30$, $\varepsilon_{st}/\varepsilon_y = 7$[9].

2.5 The structural parameter of buckling

The structural parameter of steel plates is shown in Table 2. R is width-thickness ratio parameter and calculated by formula (5).

$$R = \frac{b}{t}\sqrt{\frac{\sigma_y}{E}\cdot\frac{12(1-\nu^2)}{\pi^2 k}} \tag{5}$$

Moreover, ν is Poisson's ratio and k (=4) is buckling coefficient. We compared between 5 pattern of initial curvature amounts (1/50, 1/100, 1/150, 1/250, 1/500).

3 UNSTIFFENEE BOX SECTIONED STEEL STUB COLUMNS

As a basic study, unstiffened box sectioned steel stub columns which are part of the pier base section and simple steel plates which are part of the box were targeted as shown in Figure 5.

Elastic-plastic finite displacement analysis was conducted by ABAQUS and S4R which is shell elements of 4 nodes likewise simple steel

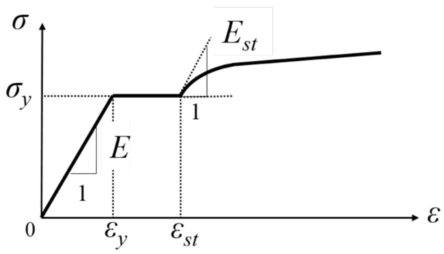
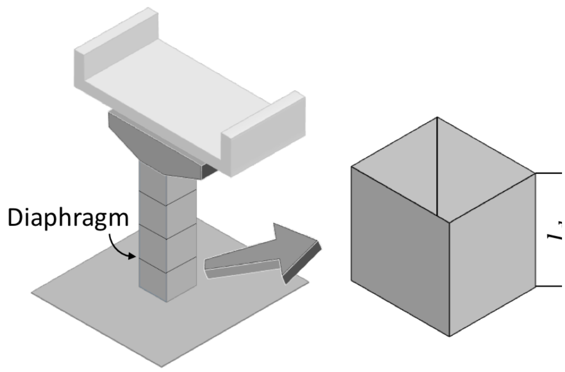


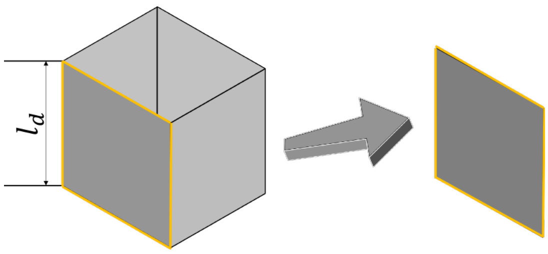
Figure 4 stress-strain curve

Table 2 The analysis parameter

R	0.26〜2.10
E (GPa)	200
σ_y (MPa)	315
ν	0.3
$\alpha = a / b$	1.0
σ_{rt} / σ_y	1.0
σ_{rc} / σ_y	0.25
t	32
w_{max} / b	1/50,1/100,1/150, 1/250,1/500,1/1000



(a) The target region of box sectioned steel stub columns



(b) The target region of simple steel plates

Figure 5 The target region

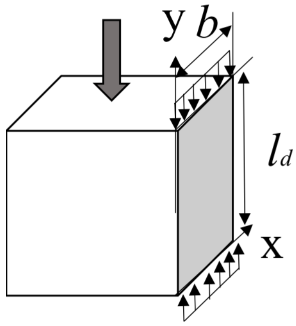


Figure 6 The model of box sectioned steel stub columns

plates. The constitutive law and the structural parameter are as same as simple steel plates.

Figure 6 shows schematic diagram of the analysis model. b is width of plates which is loading side and l_d is length between diaphragms which is not loading side. Shell elements are used to the analyses, in addition to the number of divided elements of b and l_d are equal division of 100 in both case.

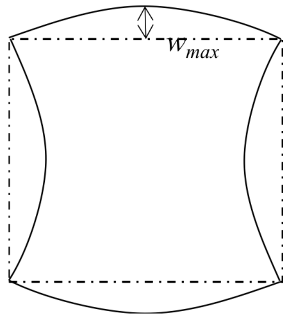


Figure 7 The illustration of initial curvatures

3.1 The model of initial curvatures

The center of plate is the maximum amount of initial curvatures w_{max} as illustrated in Figure 7 and the corners keep a right-angle. Moreover, flange face is a convex curvature and web face is a concave curvature.

Table 3 The boundary condition of box sectioned steel stub columns

	u	v	w	θ_x	θ_y	θ_z
$y = b$	1	1	1	1	1	1
$x = l_d$	0	0	0	1	1	1
Free=0, Fix=1 u,v,w=deformability of x,y,z $\theta x,\theta y,\theta z$=rotation angle of x,y,z axis						

3.2 The model of residual stresses

The residual stress of unstiffenee box sectioned steel stub columns is loaded as same as the residual stress of steel plate in Figure 3.

3.3 The boundary conditions

Table 3 shows the boundary condition of unstiffenee box sectioned steel stub columns. The boundary condition of pier base section is set up totally fixed point and analyzed in this time. However, the boundary conditions of real pier base sections are not totally fixed so that it should be elucidated about it in the furture.

3.4 The loading method

Displacement control analyses are conducted as the loading method to basically consider the study of unstiffenee box sectioned steel stub columns.

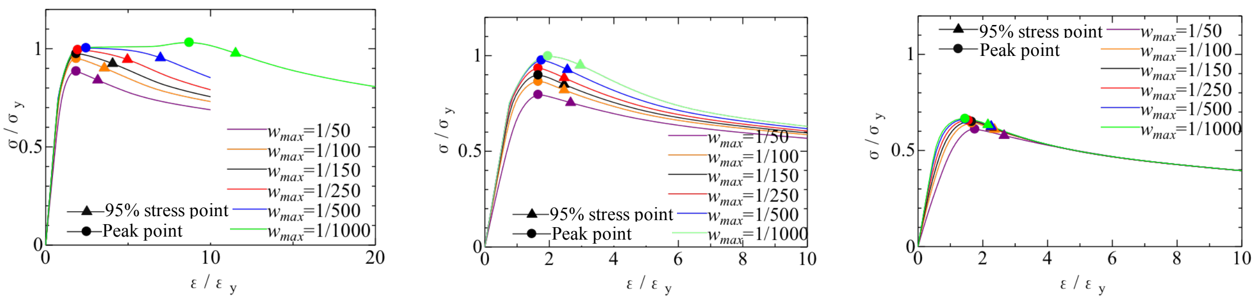
4 THE ANALYSIS RESULTS AND DISCUSSION

4.1 The definition of strength and deformability

Analysis results are assessed based on average stress ($\bar{\sigma}$) -average strain ($\bar{\varepsilon}$) curve. $\bar{\sigma}$ and $\bar{\varepsilon}$ is defined in Formula (6).

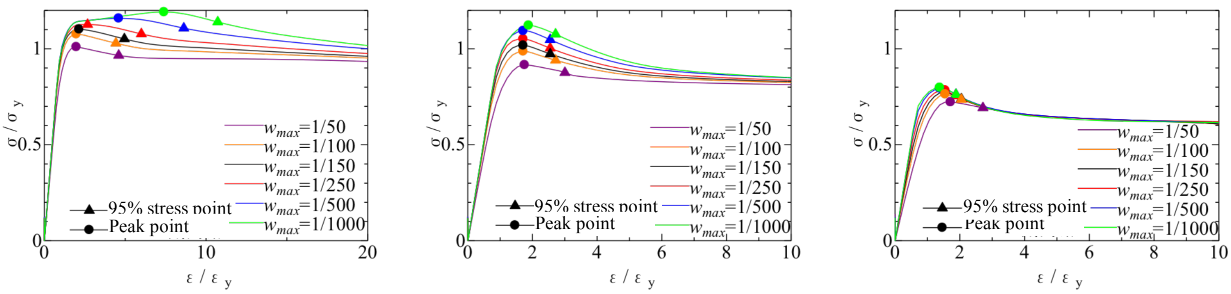
$$\bar{\sigma} = P / A \,, \quad \bar{\varepsilon} = \delta / a \tag{6}$$

Average stress-average strain curves of analysis results are shown in Figure 8. The point of the maximum average stress is called “Peak point” and the point where the stress decrease 5% from Peak point is called “95% stress point” in Figure 8. Moreover, only one plates of unstiffenee box sectioned steel stub columns made from four plates is consider to compare with simple steel plates.



(a)R=0.52 (b)R=0.72 (c) R=1.30

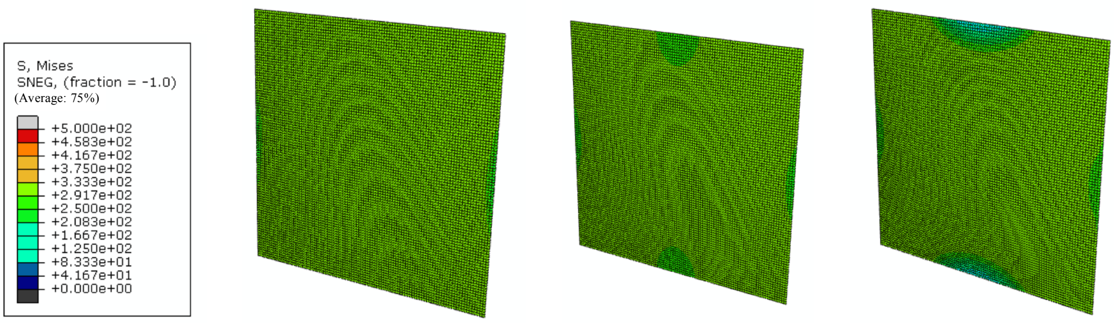
Simple steel plates



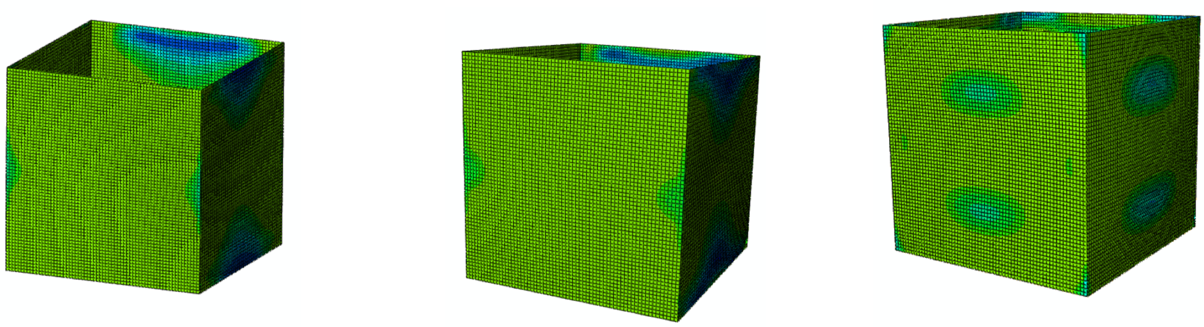
(d)R=0.52 (e)R=0.72 (f) R=1.30

unstiffenee box sectioned steel stub columns

Figure 8 Stress-strain curves



(a)R=0.52 (b)R=0.72 (c) R=1.30



(d)R=0.52 (e)R=0.72 (f) R=1.30

Figure 9 Deformation diagrams

4.2 Comparing average stress-average strain curves

Figure 8 (a) and (b), (c) show average stress-average strain curves of simple steel plates and Figure 8 (d) and (e), (f) show average stress-average strain curves of unstiffenee box sectioned steel stub columns. As shown in Figure 8 (a) and (d), local buckling is happened and strength decrease before strain hardening occurs when w_{max} is 1/50-1/500. Compare to them, there are yield plateaus and strength increases by strain hardening.

Furthermore, in unstiffenee box sectioned steel stub column's cases, strength slowly decrease and stop to decrease after strain reach 10. On the contrary, strength decrease rapidly in simple steel plate's cases.

Strength decreases before strain hardening happen when w_{max} is 1/1000 in Figure 8 (b), (c), (e) and (f) but there is no difference in the tendency of the graphs of (a) and (d) when w_{max} is 1/50-1/500.

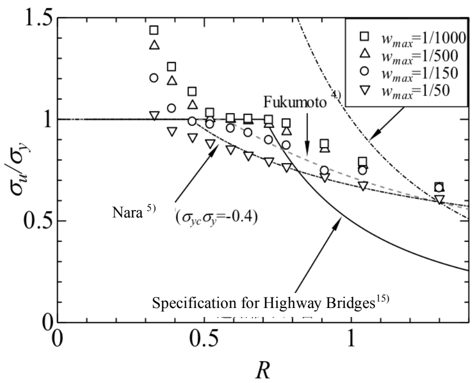
Figure 9 shows deformation diagrams when loading reaches maximum amount. (a)-(c) are diagrams of simple steel plates and (d)-(f) are diagrams of unstiffenee box sectioned steel stub column. Even though we compare plates and boxes from Figure 9, there is not much difference.

4.3 Strength

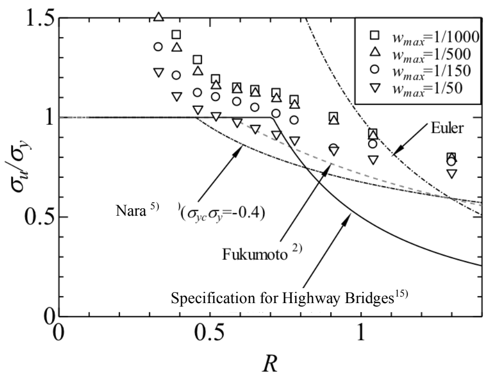
Figure 10 shows the relationship between strength and width thickness ratio parameter R when R=0.33-2.09 and initial curvatures w_{max}=1/50-1/1000.

Moreover, the maximum initial curvatures become larger the maximum strengths become smaller as shown in Figure 10. However, a relatively thin-walled area is very small difference between the maximum load by the maximum initial deflection amount and it becomes a big difference as it becomes thicker.

In addition, initial curvatures w_{max}>1/150 can handle enough by the formula of Fukumoto in plate's case as shown in Figure 10 (a). Even in the case of w_{max} <1/150, it is highly consistent with Nara's formula. As illustrated in Figure 10 (b), the strength exceeds 1 in the region of R <0.7 in plate's case but the strength



(a) Simple steel plates



(b) unstiffenee box sectioned steel stub columns

Figure 10 Comparing strengths

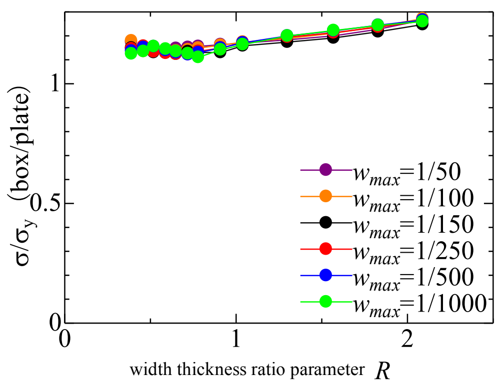


Figure 11 $\sigma/\sigma y$-R curves

exceeds 1 in the region of R<0.9 in box's case. As a result, it is too safe for plate's formula in case of a box.

Figure 11 compares the maximum strength of simple steel plates and the maximum strength of unstiffenee box sectioned steel stub columns. The vertical axis of Figure 11 is values of the maximum strength of simple steel plates divided by the maximum strength of unstiffenee box sectioned steel stub columns and the horizontal axis of Figure 11 is width-thickness ratio parameter.

There is much change of behavior by difference of the maximum initial curvatures in Figure 11. The ratio of plate's and box's maximum strength increases little by little and it keeps approximately 1.1 times in the region of R <0.8.

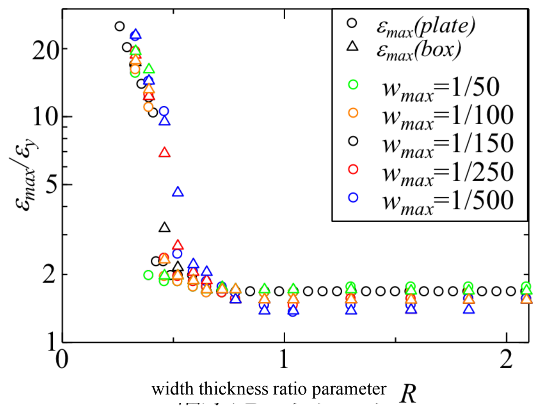
Then, the value of each maximum initial curvatures amount in Fig. 11 is averaged and we obtained a linear equation by approximation. The equation is shown in Figure 12 and Formula (7).

$$\sigma_{u,box}/\sigma_{u,plate}=0.0735\mathrm{R}+1.10 \tag{7}$$

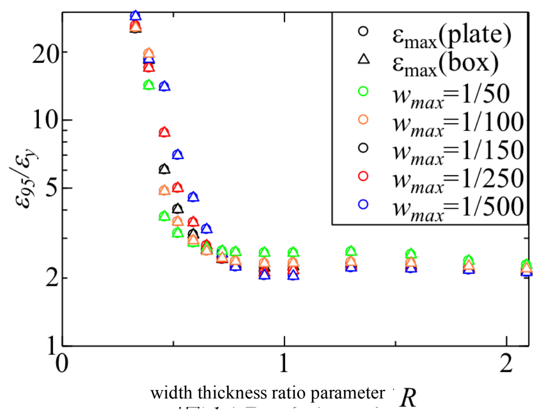
4.4 Deformability

Figure 13 shows the results of ultimate strains to compare simple steel plates and unstiffenee box sectioned steel stub columns. (a) is the ultimate strain when strength becomes maximum amount and (b) is the ultimate strain when strength decrease 5% after the maximum strength. In addition, the vertical axis of Figure 13 is logarithm.

As shown in Figure 13 (a), every parameter of the maximum initial curvatures are approximately the same behaviors when comparing simple steel plates and unstiffenee box sectioned steel stub columns. When comparing the amounts of initial curvatures, the maximum initial curvatures become smaller the ultimate strains become smaller in the region of R>0.7. there is not much difference of behaviors in the region of R<0.7. Accordingly, it is considered that it is easy to be



(a) The ultimate strain of the maximum strength



(b) The ultimate strain of the 95% strength

Figure 13 Comparing the ultimate strains

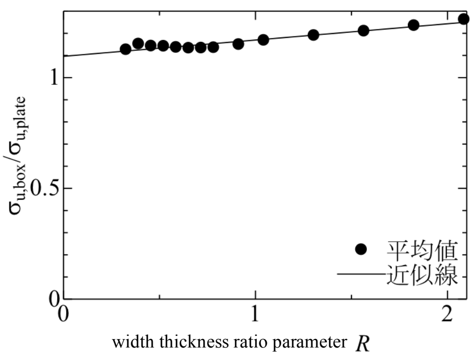


Figure 12 The liner of approximation

affected by the maximum initial curvatures when thickness of plates is thinner.

when comparing Figure 13 (a) and (b), the ultimate strains of (b) is consequently larger than (a) because the ultimate strains of (b) is the strengths reaches 95% of amounts from the maximum. However, as shown in Figure 13 (b), there is almost no difference between simple steel plates and unstiffenee box sectioned steel stub columns of behaviors like (a).

Therefore, there is not much difference like the results of strength but it is possible to apply the evaluation formula of deformability of steel plates to unstiffenee box sectioned steel stub columns.

5 CONCULUSION

In this study, we performed the parametric compressive analysis for simple steel plates and unstiffened box sectioned steel stub columns as basic examination and evaluated the stress-strain curve, σ_{max}/σ_y-initial curvatures curve, $\varepsilon_{max}/\varepsilon_y$-initial curvature and the ratio(boxes/plates) depth thickness ratio.

The results of this study were as follows.

- Strengths decrease as amounts of initial curvatures increase.
- Deformability is not effected by amounts of initial curvatures.
- The ratio of strengths increases approximately 20% when R is higher than 1.

the boundary conditions of the box are exaggeratedly strong in this analysis so we need to verify the analysis in a different boundary condition version from now on.

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