Study on Ultimate Strength and Ductility of Composite Column

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Summary

In response to the disaster of many steel bridge piers by the Hyogo-Ken Nanbu Earthquake which occurred in the Hanshin Districts(Osaka and Kobe), Japan in 1995, an experimental study was performed in order to investigate the ductility as well as ultimate strength of concrete filled steel box columns with hollow cross section subjected to such strong earthquakes as the Hyogo-Ken Nanbu Earthquake and to propose a method for improving the seismic performances of steel bridge piers. It is concluded that these composite columns have superior seismic performances.

1. Introduction

Since many steel bridge piers were locally buckled and damaged due to the Hyogo-Ken Nanbu Earthquake, a new seismic design method is required for the steel bridge piers not to suffer serious damages due to a strong earthquake. As one of them, a method for inserting the additional steel tube into the inside of steel bridge pier can be considered. In this method, the ductility of the bridge piers is significantly enhanced, if their cross section is designed in such a way that the axial compressive load caused by the dead load of the superstructure is mainly carried by the inner steel tube, and then the local buckling of the steel pier is prevented by filling the concrete between outer steel column and inside steel tube[1]. Hereafter, this kind of column is referred to as a composite column with hollow cross section.

2. Experimental Tests

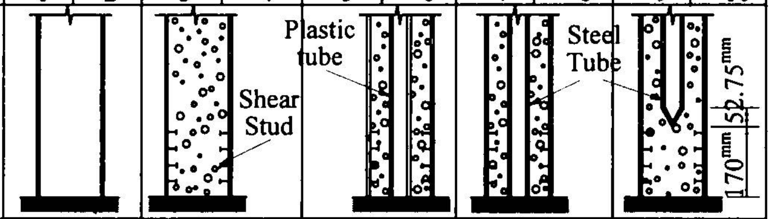
Ten cantilever column specimens, listed in *Table.1*, are adopted for the experimental tests. Eight of them are the composite column specimens and the other two are the steel column specimens. Eight composite column specimens consist of (1)two specimens with hollow cross section each having an additional inside steel tube, (2)two specimens with hollow cross section having an inside plastic tube, (3)two specimens with hollow cross section having an inside steel tube except for the lower part of them, and (4)the remaining two specimens of solid cross section filled with concrete.

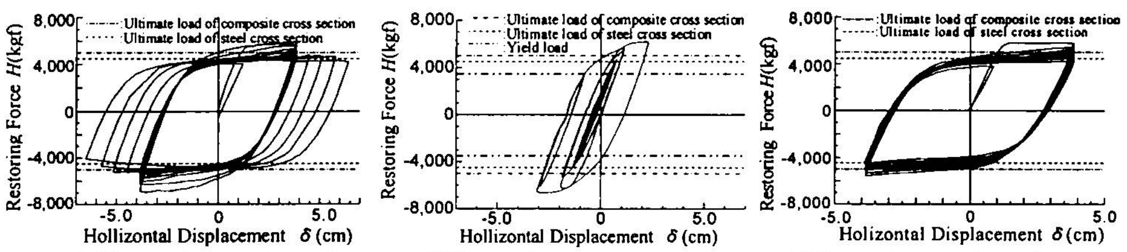
Firstly, five virgin specimens with the different types of cross section were tested under the condition of the horizontal cyclic load at the top of the specimens with the constant axial compressive force. Secondly, a large seismic load was applied to the remaining five specimens through a hybrid(pseudodynamic) testing equipment under the same axial compressive force by using one of the acceleration records of the Hyogo-Ken Nanbu Earthquake. Thereafter, the same cyclic test, as was conducted to the virgin specimens, was executed for these five specimens to investigate the ultimate strength and ductility of the specimens before and after applying the large seismic load.

3. Experimental Results

The Seismic hysteretic response curve and their cyclic curves of the composite column specimens with inside steel tubes are depicted in *Fig.1(a)-(c)*. It can be seen from these figures that these composite column specimens still remain the ultimate strength more than the fully plastic strength and less deterioration of strength due to the cyclic loading in the cases even after applying the strong seismic load.

Table.1 Characteristics of Specimens

Specimen No.	1	2	3	4	5	6	7	8	9	10
Side Elevation										
Cross Section										
Loading Method	C	C&S	C	C&S	C	C&S	C	C&S	C	C&S
C : Only cyclic loading	C&S : both of cyclic loading and seismic loading									



(a)Hysteretic curve by cyclic load test (Specimen No.7) *(b)Hysteretic response curve by hybrid test(Specimen No.8)* *(c)Hysteretic curve by cyclic load test (Specimen No.8)*

Fig.2 Experimental Results of composite column specimens with inside steel tube

4. Conclusion

(1)The composite column specimens with hollow cross section having inside steel tubes and solid cross section are of high ductility in comparison with the steel column specimens and the composite column specimens with hollow cross section having inside plastic tubes.

(2)The seismic performance of four types of the composite column specimens subjected to the large seismic load have almost similar tendency.

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Reference : 1)K. Nakanishi, T. Kitada and H. Nakai : Experimental Study on Deterioration of Ultimate Strength and Ductility of Damaged Concrete Filled Steel Box Columns, Proceedings of Association for International Cooperation and Research in Steel-Concrete Structures, pp.127-130, Kosice, SLOVAKIA, June 20-22, 1994.

Ultimate Strength and Ductility in Concrete-Filled Double Steel Tubular Columns

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Summary

This paper deals with the ultimate strength and ductility in concrete-filled double steel tubular columns. Two types of rectangular and circular cross-section were treated herein. First, the interaction curves concerning the ultimate strength of the cross section between axial force and bending moment are presented in comparison with normal composite columns. To evaluate the earthquake resistance of the column, the ductility is an important factor. Therefore, the ductility of these columns are, next, reported and compared with the reinforced concrete columns.

1. Assumptions for analysis

The following assumptions are used for the analysis of the column. a) The plane cross section of the column remains plane. b) The concrete strength in tensile area is ignored. c) Steel and concrete parts show full composite action. d) The state of full plastic stress is assumed for the analysis of *M-N* interaction.

2. *M-N* interaction curve

As one of the numerical examples for the circular section, Fig.1 shows *M-N* interaction curves of the cross section for concrete-filled double steel tubular column in comparison with the steel and the concrete-filled single steel tubular columns. The numerical data for the calculation is shown in Table 1. From the curves, it is found that the maximum load carrying capacity ratio (M_{max}/M_{pl}) is 1.90 for concrete-filled single steel tubular columns. In the case of concrete-filled double steel tubular column, however, its value is 1.35. The reason of this decreasing can be explained clearly that the cross sectional area of concrete in concrete-filled double steel tubular column is less than the concrete-filled single one. Therefore, it is recognized that the effect of cross sectional area of concrete for the maximum load carrying capacity of the composite column is one of the important factors for the design. The *M-N* interaction curves for the rectangular section have same properties with circular section.

Table.1 Numerical Condition

Yield strength of outside steel tube	σ_{y1}	235 N/mm^2
Diameter of outside steel tube	d_1	2000 mm
Thickness of outside steel tube	t_1	9 mm
Yield strength of inside steel tube	σ_{y2}	235 N/mm^2
Diameter of inside steel tube	d_2	1200 mm
Thickness of inside steel tube	t_2	9 mm
Specified concrete strength	σ_{ck}	23.5 N/mm^2

3. Ductility

To evaluate the ductility of the column, an actual bridge pier shown in Fig.2 was selected. The calculations were executed for three types of column, namely, reinforced concrete, concrete-filled single and double steel tubular column in which the column has same external dimensions. The yield and ultimate horizontal displacement at the top of column, δ_y and δ_u, are defined as the values when the strains of steel and concrete reach the yield and ultimate strains at most outer side of steel and concrete, respectively. Using these values the rate of ductility for column, μ, is calculated as follows:

$$\mu = \delta_u / \delta_y$$

Fig.3 shows the relationships between horizontal force and displacement of three columns. Concrete-filled steel tubular columns have large load carrying capacity and ductility compared with reinforced concrete column. Furthermore, the ultimate displacement in double steel tubular column is smaller than concrete-filled single one. However it is identified that the both composite column has almost same load carrying capacity.

4. Conclusion

A burden against the foundation can be reduced by employing the double steel tubular structure for column since the dead weight of bridge pier reduces. Furthermore, it can be mentioned that the concrete-filled double steel tubular column has almost same mechanical characteristics compared with the concrete-filled single one.

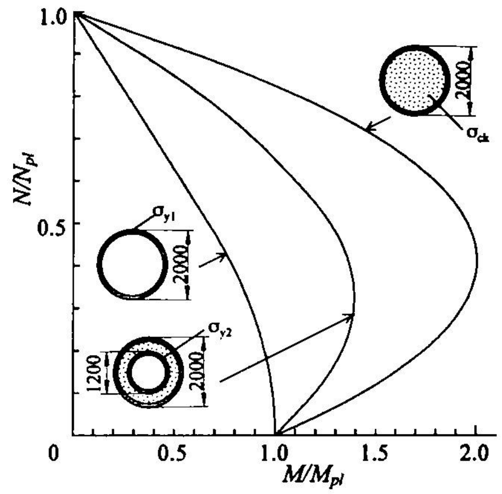

Fig.1 M-N interaction curves of three types of column section

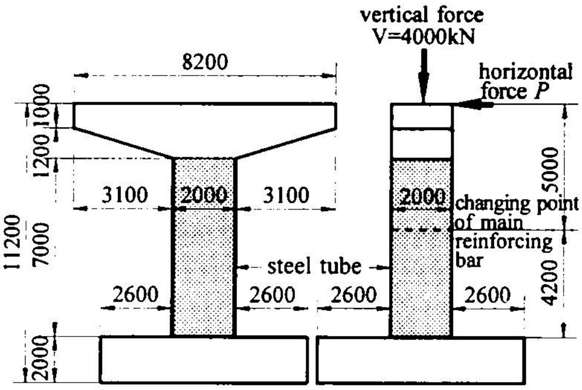

Fig.2 Analytical model

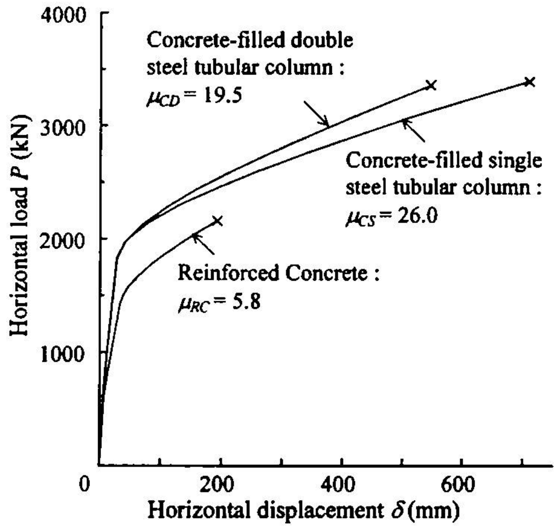

Fig.3 P-δ Relationship

Stable Ultimate Deformation of Confined Columns Subjected to Seismic Loads

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Summary

A method to evaluate the stable ultimate deformation of the confined concrete columns under seismic loading was proposed. In this method, stable ultimate deformation of a column was defined as the deformation reached when the axial strain at the center of the critical section was equal to half of the peak strain of the confined concrete. The validity of the proposed method was verified by comparing the theoretical ultimate deformation obtained using this method with the measured results of confined concrete columns under cyclic reversed bending moment.

1. Introduction

The ultimate deformation plays a fundamental role in evaluating seismic performance of the reinforced concrete columns. For easiness, the deformation in which the load is dropped to 80% of the maximum value has been conventionally defined as the ultimate deformation of the column under seismic load. However, the value of 80% in the conventional definition hasn't any physical meaning relating to the damage degree that can be tolerated by the column; besides, conventional definition might overestimate the deformability of the column under high axial load [1]. Therefore, it is necessary to develop a new criterion to define the ultimate deformation of the concrete column. The purpose of this paper is to propose a criterion for defining the ultimate curvature of the concrete column confined by transverse reinforcement.

2. Stable Ultimate Deformation

Fig.1 shows idealization of the strain distribution of confined section under cyclic moment. As shown in Fig.1, under high axial load, a compression zone exists in the section, this zone always subjects to compressive deformation while the moment is a reversed cyclic type. It is apparent that in order for the concrete section to provide stable resistance, the maximum compression strain (the strain in the center) at this zone should be limited below the peak strain ε_{co} ($\varepsilon_{co}/2$) of the concrete (see Fig.2), beyond which the strength deterioration in the concrete will become significant.

From the above theoretical consideration, the stable ultimate curvature of the confined column can be defined as the curvature in which the compressive strain ε_{cc} in the center of the section reached half of the peak strain ε_{co} of the concrete. Authors' experimental work [1] has indicated that the envelop curve of cyclic response coincided with monotonic curve until the ultimate curvature

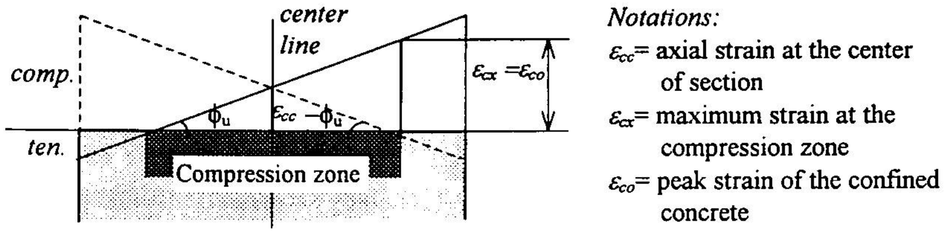

Fig. 1 Idealization of the strain distribution of column section under cyclic moment

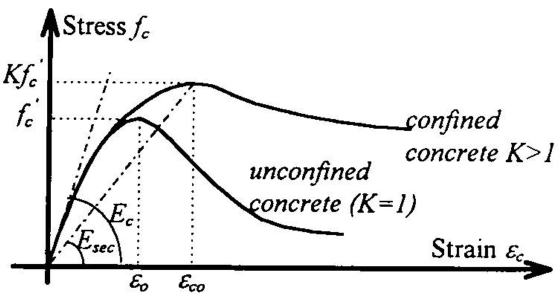

Fig. 2 Stress-strain curve for concrete

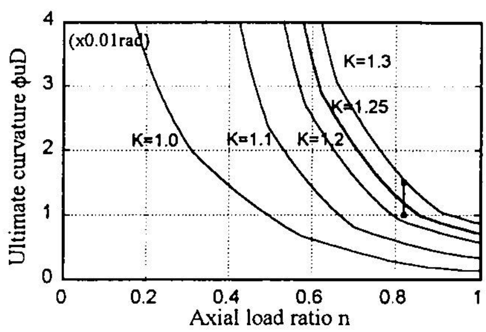

Fig.3 Stable ultimate curvatures

defined by the above-described criterion. Therefore the ultimate curvatures for the confined column can be obtained by only calculating the monotonic M-ϕ curves of the critical section of the column corresponding to various levels of axial load and confinement from transverse steels.

Fig.3 shows theoretical ultimate curvature (ϕ_uD)-axial load ratio (n) curves for the test columns described in Ref.1. The parameter K shown in Fig.3 is the ratio of the confined concrete strength to the concrete cylinder strength, an index denoting the confinement degree, and D is depth of the section. For calculating the M-ϕ curves, a stress-strain curve for the confined concrete proposed by the first two authors [2] has been used. The solid squares in Fig.3 denote the lower and upper limit (0.01-0.015rad) of the test results of specimens. It can be seen that the theoretical solid line with K=1.25, which represents the confinement degree of the specimen presented in Ref.1 and is obtained using authors' confinement model [1], predicted the experimental results very well. On the other hand, following the conventional definition, the ultimate curvature would be 0.02-0.025rad, and clearly overestimated the ultimate deformation capacity of the test column.

3. Conclusion

A new criterion was proposed to relate the ultimate curvature to the axial deformation of the confined columns. Theoretical predictions obtained based on this criterion and authors' stress-strain model for the confined concrete exhibited good agreement with the measured result.

References

[1] Sakino,K., et al., "Factors influencing the flexural behavior of confined concrete columns," The AIJ Annual Convention, Sept. 1996, Vol. C, pp.47-52. (in Japanese)

[2] Sun, Y., et al., "Flexural Behavior of High-Strength RC Columns Confined by Rectilinear Reinforcement," Journal of Struct. Constr. Eng. AIJ, No. 486, pp.95-106, Aug. 1996.

Evaluation of Seismic Resistivity of CFT Steel Pillar

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Summary

Aiming at establishment of a new seismic design method in which ductility is taken well into account, a series of alternating bending tests under constant axial compressive force with Concrete Filled Tubular, called CFT, steel pillar models were carried out. The study based on experimental results enabled the quantitative evaluation of seismic resistivity of CFT pillars. The fruits obtained through this study will be reflected in Model Code for Railway Hybrid Structure Design to be made public by Japan Ministry of Transport in 1997.

1. Introduction

Newmark's energy preservation rule is well known as a concept of the seismic design, where the ductility of pillars is taken into consideration. This idea is to check the bending yield point capacity to horizontal seismic load as corrected according to the ductility of pillars. In order to apply such a seismic design method to CFT pillars, it is necessary to establish the method of quantitatively evaluating the yield point load, yield point displacement and also relationship between the limit plastic displacement and the pillar components. Therefore, the study was done according to the following steps.

Step1 Alternating bending test with CFT pillar models
Step2 Analytical study on yield point load and yield point displacement
Step3 Statistical study on quantitative evaluation of ductility

2. Alternating bending test on CFT pillar models

The alternating bending test under axial compressive force with 1/3 models of CFT pillars was executed. The experimental parameters are diameter-thickness ratio, axial compressive force, concrete strength, and steel pipe strength.

3. Analytical study on yield point load and yield point displacement

The yield point load and yield point displacement were calculated with a fiber element model dominated by elasto-plastic stress-strain relationship of steel and concrete materials. *Figure 1* shows the comparison between the experimental values and the calculated ones.

4. Statistical study on quantitative evaluation of ductility

The expression of quantitatively evaluated ductility was induced from the recurrence analysis based on experimental results as well as on analytical ones. *Figure 2* shows the comparison between the calculated ductility and experimental one.

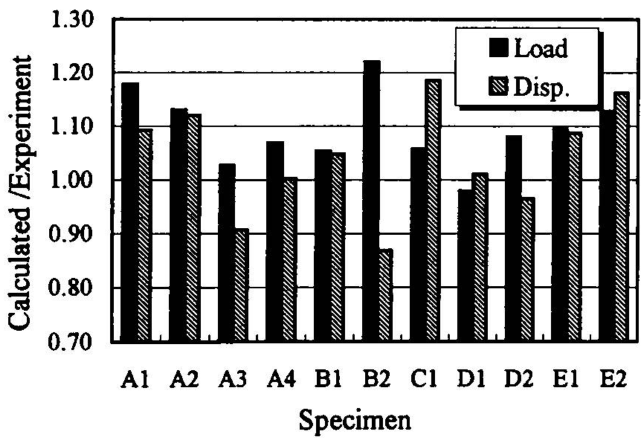

Fig 1 Yield Point Load & Displacement

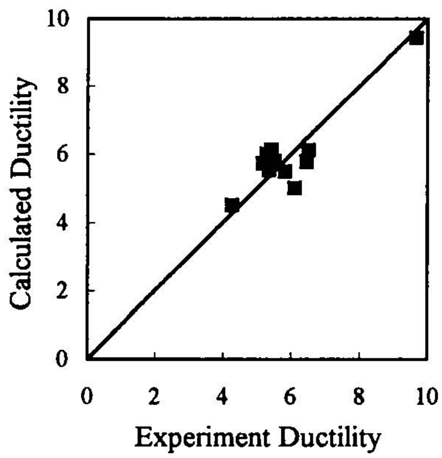

Fig 2 Ductility

5. Conclusions

The quantitative evaluation expression for seismic resistivity of CFT pillars was induced. As a result, a more rational and accurate evaluation of yield point load, yield point displacement, and ductility necessary for the seismic design of CFT pillars has been made possible.