Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 999 (1997)

Artikel: Plastic deformation capacity of steel-concrete composite member
Autor: Suzuki, Toshiro / Fukasawa, Takashi / Motoyui, Shojiro
DOI: https://doi.org/10.5169/seals-1064

Plastic Deformation Capacity of Steel-Concrete Composite Member

Toshiro SUZUKI
Prof. Emeritus, Dr. Eng.
Tokyo Institute of Technology
Tokyo, JAPAN

Shojiro MOTOYUI
Assoc. Prof., Dr. Eng.
Tokyo Institute of Technology
Yokohama, JAPAN

Takashi FUKASAWA
Manager, Dr. Eng.
Tomoe Corporation
Tokyo, JAPAN

Masahiko UCHIYAMA
Civil Engineer, Dr. Eng.
Tomoe Corporation
Tokyo, JAPAN

Summary

Various attempts have recently been undertaken to improve plastic deformation capacity of steel members. One of these is the approach of combining with concrete as the composite member. In respect to the composite beam-column which is made of H-shaped steel and concrete placed between upper and lower flange of the steel, the present paper describes quantitative evaluation of stiffening effect by concrete and also quantitative evaluation of plastic deformation capacity of the proposed structural configuration.

1. Introduction

Steel-concrete composite is conventionally considered as a steel encased by the reinforced concrete in Japan. The structural portion outside the steel in the composite is significant for strength of the member. However, such the composite is especially difficult to improve its plastic deformation until the load carrying capacity, because the load carrying capacity is dominated by crush of the concrete. Therefore, to investigate concrete stiffening effect to the steel, the objective in the present study is H-shaped section of the steel filled by concrete between the both flange plates, which is hereafter called SC. In this paper, firstly local buckling behavior and collapse mechanism of SC is discussed, secondary the specific criterion of the collapse is derived theoretically, and then method of evaluating the plastic deformation capacity is proposed.

2. Modelling of Local Buckling Behavior

2.1 Experimental Approach

Several kinds of tests[1),2),3),4)] have been carried out on the proposed structural configuration shown as cross-section in Fig. 1. Regarding effect of concrete, the most typical results have been obtained also by a test of cyclic flexural loading, noticed as Q in Fig. 1, under keeping a constant axial force, noticed as N. Two types of specimens, shown in Fig. 1, have been tested to clarify effect of concrete. One of these described as type SC is the objective in case that concrete is undergoing axial force. The other, type SC-s, is the objective in case that concrete is not affected by the applied axial force, which is realized by several urethane plates inserted in concrete as shown in Fig. 1. Used material combination is fixed to SM490A of steel, of which tensile yield stress is over 324N/mm^2 regulated by Japan Industrial code, and concrete required

34N/mm^2 of compressive strength.
Experimental results[4] of using a fixed size of steel, BC2 in Table 1, are shown in Fig. 2, comparing load versus displacement relation of the type of SC and SC-s with that of the only steel. Sufficient concrete stiffening effect to the steel in the composite is admitted by hysteresis characteristics of type SC or type SC-s. Specimen of the only steel is observed instability at the end of the first cycle according to forming local shear buckling of web plate, represented in Photo. 1(c). However, local buckling mode of the type of SC or SC-s is different from that of the steel, which is web plate stretched normal to the axial direction without out-of-plane deformation. Therefore, the concrete is considered to stiffen web plate. Some of the other experimental results are given in Table 1, which consists also results[3),4)] of monotonously flexural loading without axial force. The plastic deformation capacity of SC is determined to be much higher than that of the only steel.

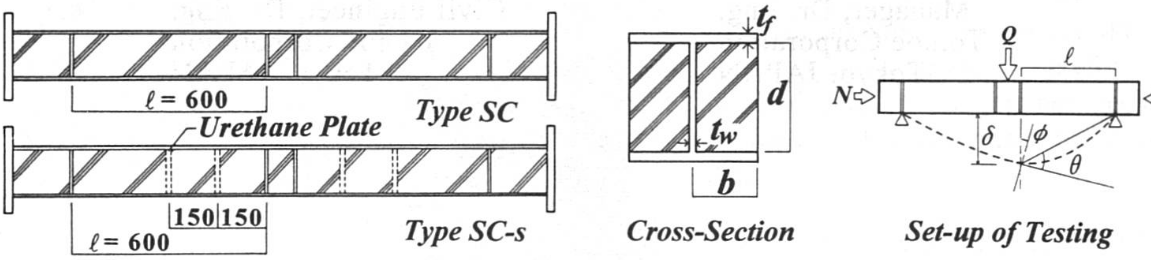
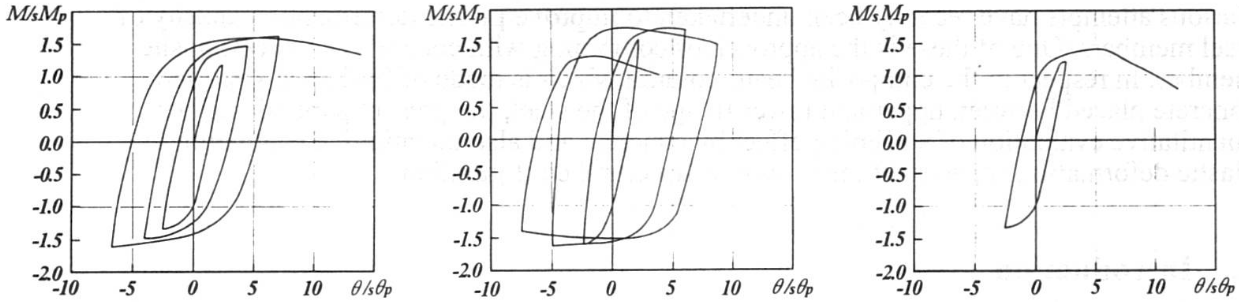
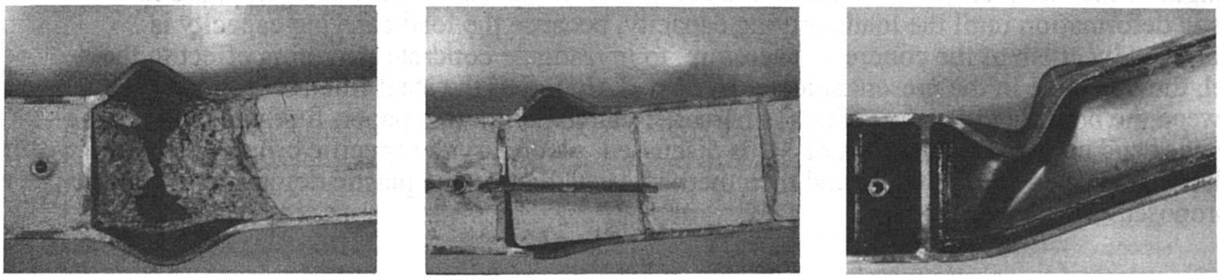


Fig. 1 Schematic of Specimens



(a) Type SC(BC2) (b) Type SC-s(BC2-s) (c) Steel

Fig. 2 Concrete Stiffening Effect to Steel Member



(a) Type SC(BC2) (b) Type SC-s(BC2-s) (c) Steel

Photo. 1 Close-up of Local Buckling after Failure

Table 1 Experimental Results

Cyclic flexural loading with axial force:

Specimen	b/t_f	d/t_w	N_u (kN)	n'	M_m(kN·m)	R
BC1	16.7	31.3	1450	0.3	6.7	3
BC2	8.33	29.3	1870	0.3	10.8	10
BC3	8.33	22.0	1930	0.3	10.7	20

Monotonously flexural loading without axial force:

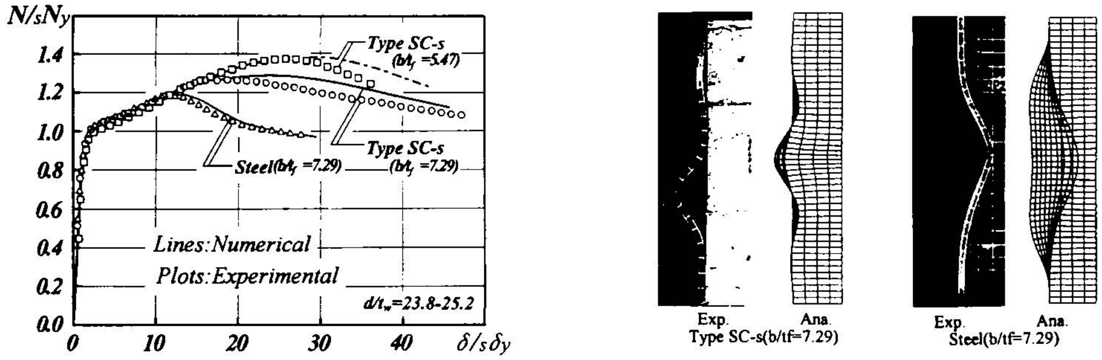
Specimen	b/t_f	d/t_w	M_m(kN·m)	R
B1	16.7	31.3	6.8	10
B2	12.5	30.7	8.8	14
B3	12.5	23.0	9.2	14

Note: N_u is the calculated compressive strength of the composite, but N_u of BC2-s(type SC-s) is for only steel., n' is ratio of the applied axial force to N_u., M_m is moment at the max. load., R: is plastic deformation capacity defined by plastic deformation at the load carrying capacity.

2.2 Numerical Approach

The particular local buckling mode, described in the former, is considered to be significant for evaluating plastic deformation capacity of SC. Therefore, compressive tests[2)] on stub columns have been carried out to investigate the buckling behavior. After failure web plate was observed to be stretched normal to the axial direction, but it was not deformed out-of-plane. Considering this phenomenon, the numerical analysis by finite element method has executed under the following boundary condition: out-of-plane displacement of web plate is constrained, and the other boundaries are governed by the same condition of the steel. In order to focus concrete stiffening effect, bearing load by concrete should be eliminated. Therefore, in this stub column test, specimens of SC are fixed to type SC-s, and the analytical model is subjected to a steel member without concrete. With respect to load-displacement characteristic and deformation after failure, the each analytical result is corresponding to the experimental result, which is shown in Fig. 3. Consequently, the present analytical model is verified by these results.

Several cases are analyzed numerically by the proposed model as a parameter of width-to-thickness ratio of web plate under that of flange plate fixed to 7.29. In these analytical results shown in Fig. 4, displacements at two points in the section of flange plate, noticed as w_1 and w_2, are discussed. If the buckling criterion of flange plate, noticed as ▽, is defined w_1 becomes 15% of flange plate thickness, the each point of this criterion in the load-displacement curve is determined to be shifted to that of infinite small width-to-thickness ratio of web plate in an asymptotic manner. Moreover, the each point of the load carrying capacity is determined to be located far from the point of the buckling criterion. Therefore, collapse mode should be affected by some other except buckling of flange plate. On the other hand, the provisional collapse criterion noticed as ▲ which is defined that displacement w_2 becomes 2% of width of flange plate. The each point of this collapse criterion is located after the point of the buckling criterion and nearer to the each point of the load carrying capacity. Therefore, collapse mode should be affected by stretching of web plate normal to the axial direction. Moreover, behavior of flange plate is mentioned to be similar to buckling of a lateral stiffened simple column shown in Fig.5.



(a) Load-displacement Characteristics (b) Deformation after Failure

Fig. 3 Verification of Analysis

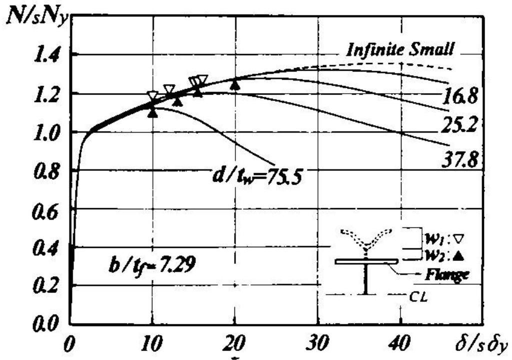


Fig. 4 Effect of Web to Plastic Deformation

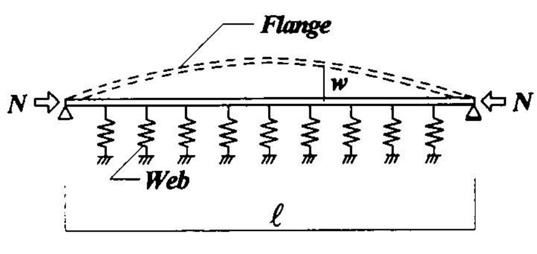


Fig. 5 Collapse Mechanical Model

2.3 Theoretical Treatment

From the above mentioned, which is significant on behavior of web plate stretching in the collapse mode, the collapse mechanism should be imagined by a model of simplified flange plate stiffened by many springs which is illustrated in Fig. 5. Collapse criterion of this model is able to be equivalent to buckling criterion of the lateral stiffened column. The bucking equation[5] of such the column is represented as eq. 1. In this equation, 'EI' is the flexural stiffness of the column which is provided with section properties of flange plate, and 'k' is the spring constant which is provided with section properties of web plate.

$$EI\frac{d^4w}{dx^4}+N\frac{d^2w}{dx^2}+kw=0 \quad \text{where,} \quad EI=\frac{Et_f^3b}{12(1-\nu^2)}, \quad k=\frac{Et_w}{(1-\nu^2)d} \quad \cdots\cdots\cdots\cdots \text{Eq. 1}$$

The above equation can be written in the non-dimensional form represented as eq. 2. Assumed a constant compressive force applying to the section, n(ξ)=1, a solution of eq. 2 is Asin(iπξ), where 'A' is a constant number, and 'i' is the wave number. Then, 'λ' is obtained as eq. 3 in case of infinite long column. Therefore, the buckling strength N_{cr} is given by eq. 4.

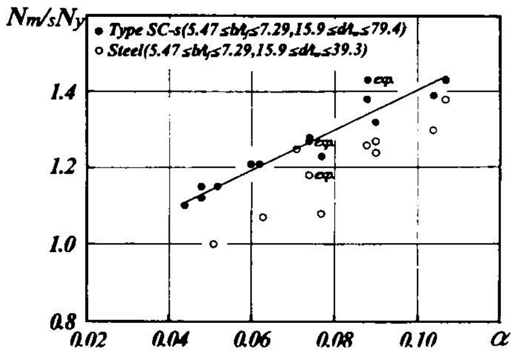
$$\frac{d^4w}{d\xi^4}+\pi^2\lambda n(\xi)\frac{d^2w}{d\xi^2}+(\beta\ell)^4w=0 \quad \cdots\cdots\cdots\cdots \text{Eq. 2}$$

$$\text{where,} \quad \xi=x/\ell,\ \beta=\left(\frac{k}{EI}\right)^{\frac{1}{4}},\ n(\xi)=N/N_0,\ \lambda=\frac{N_0}{N_E},\ N_E=\frac{\pi^2EI}{\ell^2}$$

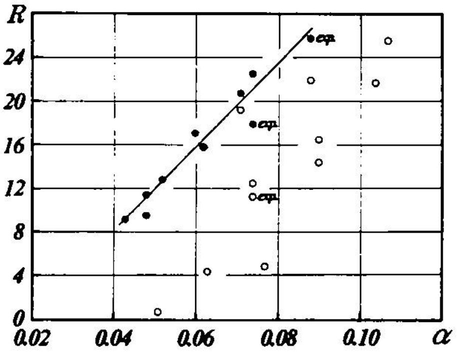
$$\lambda=2\frac{\ell^2}{\pi^2}\sqrt{\frac{k}{EI}} \quad \cdots\cdots\cdots\cdots \text{Eq. 3} \qquad N_{cr}=2\sqrt{\frac{E^2t_wt_f^3b}{12(1-\nu^2)^2d}} \quad \cdots\cdots\cdots\cdots \text{Eq. 4}$$

Finally, the buckling stress is given by eq. 5. Namely, this equation is related to the generalized slenderness, therefore; a geometrical term in eq. 5 is significant to examine behavior of the collapse. To evaluate efficiency of this term which is hereafter called α represented as eq. 6, the experimental results and the analytical results have been discussed on this α. A high correlation between the load carrying capacity or the plastic deformation capacity and α is recognized as shown in Fig. 6 (a), (b) respectively. Consequently, α is considered as a criterion of the collapse.

$$\left(\frac{\sigma_y}{\sigma_{cr}}\right)^{-1}=\frac{E}{\sqrt{3}(1-\nu^2)\sigma_y}\sqrt{\left(\frac{t_w}{d}\right)\left(\frac{t_f}{b}\right)} \quad \cdots\cdots \text{Eq. 5} \qquad \alpha=\sqrt{\left(\frac{t_w}{d}\right)\left(\frac{t_f}{b}\right)} \quad \cdots\cdots \text{Eq. 6}$$



(a) Strength



(b) Plastic Deformation Capacity

Fig. 6 Efficiency of Parameter α

3. Plastic Deformation Capacity

3.1 Method of Evaluating

Several assumptions and approximations are necessary to evaluate the plastic deformation capacity of the composite beam or beam-column. Firstly, it is assumed that effect of the concrete strength is ignored, because concrete in the present composite is crushed near after the yield of the steel, which is noticed as the point of 'b' in Fig. 7. Secondary, shape of the section is equivalent to the simplified model so called as the two-flange model[6], which consists only the upper and lower flange plates having the equivalent area of the original section and moment of inertia. Furthermore, according to the well known approximation for the steel, load-displacement curve of the composite is simplified to the linear model as shown in Fig. 7. On the other hand, plastic zone of the member is defined as the illustration in Fig. 8 which is the equivalent cantilever beam-column model. In this illustration, noticed 'n' means the ratio of the actually applying axial force to the steel in the composite. The actual value of this ratio can be approximately calculated according to the following assumption: The steel is assumed to be undergoing the all applied axial force between the point of 'b' and 'c' in Fig. 7. Additionally, in this model the plastic deformation capacity 'R' is defined by the equation: $R=\delta_{m2}/\delta_{m1}-1$.

Based on the model and the assumptions, distribution of the flexural moment is described as eq. 7. With respect to displacement after the point of the maximum strength of the steel noticed as 'b' in Fig. 7, relations of the moment to the curvature corresponding to the distinct loading or stress conditions are defined as eqs. 8,9,10; these equations are in case of no axial force applying, and in cases that the tensile flange plate, which is the lower flange plate, yields or keeps elastic under the applied constant axial force, respectively. The tensile flange plate tends to yield if the parameter α is relatively large; however, the present investigation does not examine the specific point of this yielding, which is considered as the future work. Finally, based on distribution of the curvature derived from these relations, the plastic deformation capacity of the each case, especially the flexural displacement at the free end in Fig. 8, is obtained by the similar process[6] for the steel. The plastic deformation capacity is consequently described as a function of the parameter α, the section properties, the material mechanical properties, and two ratios of the applied axial force to the calculated compressive strength of the steel before and after crush of the concrete.

$$M(x)=(1-n)\,{}_sM_{p0}+(s_2-1)\,{}_sM_{p0}\left(\frac{\tau_3\ell-x}{\tau_3\ell}\right) \quad \cdots\cdots \text{Eq. 7}$$

$$M=S_1\cdot{}_sM_{p0}+D(\phi-{}_s\phi_m)\text{, where}\quad D=\frac{(S_2-S_1)EI}{374\alpha'-6.93-(S_1-1)(E/E_{st})} \quad \cdots\cdots \text{Eq. 8}$$

$$M=S_1\cdot{}_sM_{p0}+D(\phi-{}_s\phi_m-\phi_u) \quad \cdots \text{Eq. 9}$$

where $\phi_u=(\varepsilon_u-\varepsilon_y)/h$

$$M=S_1\cdot{}_sM_{p0}+2D(\phi-{}_s\phi_m) \quad \cdots \text{Eq. 10}$$

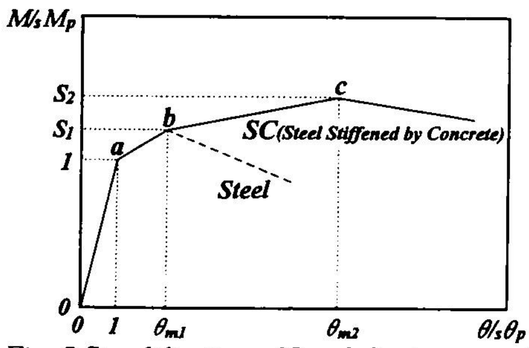


Fig. 7 Simplification of Load-displacement Curve

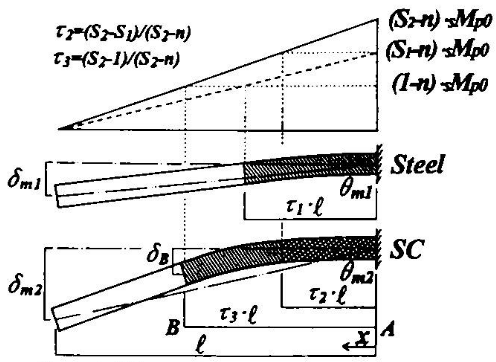


Fig. 8 Plastic Zone of Beam-column

3.2 Verification

Arranging the expression of evaluating the plastic deformation capacity 'R', almost linear correlation between 'R' and the parameter α is obtained, which is shown as lines in Fig. 9 corresponding to the distinct loading or stress condition. The proposed method of evaluating is verified by the experimental results, however in case of with axial force, two situations are considered whether the tensile flange yields or not. Transition point among these situations is not specified, but at least the equation for the tensile flange plate keeping elastic is adopted at the safety side. Furthermore, based on the relation of 'R' to α, limitations for width-to-thickness of flange and web plates are obtained as shown in Fig. 10. Consequently, these limitations of the steel in the composite is much lightened comparing with the steel only.

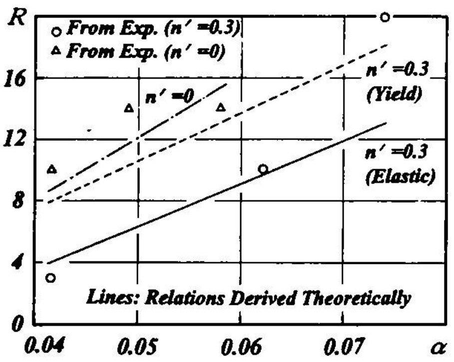


Fig. 9 Plastic Deformation Capacity

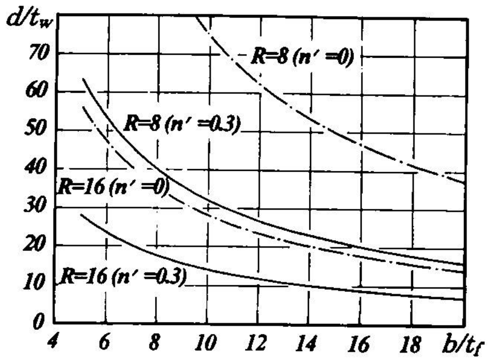


Fig. 10 Width-to-thickness Limitations

4. Conclusions and Future Works

The collapse mechanism of the proposed composite is clarified to be stretching of web plate normal to the axial direction. Theoretically derived from this mechanism, the geometrical parameter α is determined as criterion of the collapse. Furthermore, α is efficient to evaluate the plastic deformation capacity of the beam and the beam-column quantitatively. As future works, the proposed method of evaluating should be optimized to clarify the transition point of the tensile flange plate yielded. Additionally, effect of bearing ratio between the steel and the concrete should be evaluated in the proposed method.

References

1) Suzuki, T., Ogawa, T., Fukasawa, and Uchiyama, M., "An Experimental Study on Structural Properties of Steel-concrete Columns under Compressive Shear Stress"(in Japanese), J. Struct. Constr. Eng., AIJ, No. 483, pp. 165-172, May, 1996.
2) Suzuki, T., Motoyui, S., Uchiyama, M., "Study on Strength and Plastic Deformation Capacity of Composite Stub Column under Compressive Loading"(in Japanese), J. Struct. Constr. Eng., AIJ, No. 480, pp. 171-178, Feb., 1996.
3) Suzuki, T., Motoyui, S., Uchiyama, M., "Study on Plastic Deformation Capacity of Composite Member under Bending Shear Loading"(in Japanese), J. Struct. Constr. Eng., AIJ, No. 484, pp. 141-148, Jun., 1996.
4) Suzuki, T., Motoyui, S., Uchiyama, M., "Study on Plastic Deformation Capacity and Hysteresis Characteristics of Composite Member Loaded Cyclic Bending Moment under Constant Axial Loading"(in Japanese), J. Struct. Constr. Eng., AIJ, No. 490, pp. 213-220, Dec., 1996.
5) S. E. Svensson, "Lateral Buckling of Beams Analyzed as Elastically Supported Columns Subject to a Varying Axial Force", J. Constr. Steel Research, Vol.5, pp.179-193, 1985.
6) Kato, B., Nakao, M., "Strength and Deformation Capacity of H-shaped Steel Members Governed by Local Buckling"(in Japanese), J. Struct. Constr. Eng., AIJ, No. 458, pp. 127-136, Apr., 1994.