Future Use of Composite Steel-Concrete Columns in Highway Bridges

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INTRODUCTION

Modern seismic design practice of highway bridges tends to limit the inelastic activity during seismic events to the substructure of the bridge. This would limit the damage only in columns which had been designed and detailed to withstand significant inelastic activities. Inelasticity in the superstructure is not recommended in bridge engineering, because it is not feasible nor economical to allow damage to occur in that area. The recent seismic criteria for the design of reinforced concrete bridge columns as outlined in the first draft of *Revised Caltrans Bridge Design Specification*[8] by the *Applied Technology Council (ATC32)*, require a significant amount of steel to confine the concrete and to prevent buckling of longitudinal reinforcement. This amount of confinement steel tends to be excessive as was recently experienced during the design of the replacement structures in San Fernando Valley, California after the Northridge earthquake.[9] Confinement of concrete columns using full steel jackets becomes a feasible design solution since the steel jacket would confine the concrete and also provides strength for the column.

The use of Composite Steel and Concrete (CSC) columns in bridges have many advantages especially in high seismic zones. This type of column provide the structure with stiffness and ductility, which are required for the survival of any bridge during severe ground motion.[2,5] In simple terms, the CSC columns are concrete filled tubular steel columns without any longitudinal or transverse reinforcement. The idea of having the steel and the concrete act compositely has been used successfully in the past for new and retrofit construction.[4,5,6] The greatest advantage of this concept is that the two materials are put to their ultimate use. The steel jacket confines the concrete and provides the flexural strength for the system. The presence of the concrete delays steel problems in compression zones such as local and overall buckling and also provides stiffness for the column.

The main advantage of CSC columns is the nonusage of longitudinal and transverse reinforcement. The absence of the longitudinal bars will eliminate flexural shear cracks, caused by the bond between the bars and concrete. This should improve the compressive strength of concrete and free it from the traditional cracking that occurs in reinforced concrete members. CSC columns are therefore capable of providing both high ductility and large energy absorption capacity due to minimization of flexural cracking and delay of steel local buckling.

The other advantages of CSC columns are their easy and fast construction. The steel jacket will act as a slip form in which the concrete is poured during construction. This will not only speed up construction time, but also eliminates the cost of the forms.

Moreover, the relatively smaller size of CSC columns provide the same strength as standard sizes of reinforced concrete columns, which makes them very desirable in metropolitan areas where space is severely limited.

STUDY OBJECTIVE

The objective of this study is to determine the curvature ductility of CSC columns and to investigate whether or not they are adequate for use in seismic areas. This was achieved analytically by studying the moment curvature relationships (M-ϕ) and the curvature ductility of CSC columns.

COLUMN PROPERTIES

A typical bridge column with diameter of 5½ feet and a height of 30 feet was chosen for this study. The column is a part of a single column bent bridge with a 150 feet spacing between columns. The thickness of the steel jacket for CSC columns was varied between ⅜-in. to 1¼-in. as shown in Figure 1 to determine its effect on the ultimate capacity of the column. Table 1 lists the various steel jacket thicknesses used in this study, the *D/t* ratios for CSC columns, and the percentage of longitudinal reinforcement that the steel jacket represents.

The column height of 30 feet was selected to ensure a flexural response rather than a shear response. The aspect ratio of the column, height to diameter ratio, was 5.45. The variation of the steel jacket thicknesses provided a significant range of reinforcement ratios. The minimum reinforcement ratio was 1.8 percent using a steel jacket of ⅜-in. thickness with a maximum ratio of 6.2 percent using a steel jacket of 1¼-in. thickness. It is interesting to note here that using reinforced concrete bridge columns would limit the maxi-

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Table 1. Properties of Composite Columns Used in the Study			
Column Type	Thickness of Steel Jacket (in.)	*D/t* Ratio	% Longitudinal Reinforcement
C1	3/8	176	1.8
C2	1/2	132	2.4
C3	5/8	106	3.0
C4	3/4	88	3.6
C5	7/8	75	4.3
C6	1	66	4.9
C7	1 1/8	59	5.5
C8	1 1/4	53	6.2

mum amount of reinforcement ratios to 2.5 percent and 3 percent due to spacing requirements between the longitudinal reinforcement. Thus the use of a CSC column permits higher longitudinal reinforcement ratios than conventional construction.

BEHAVIOR OF CSC COLUMN

To ensure full composite action between the steel jacket and the concrete, shear studs were provided throughout the length of the column. The reason for the shear studs is to prevent any separation between the steel and concrete which is expected to occur in the elastic range.[2,5] This separation is due to the difference in Poisson's ratios of steel and concrete. The value of Poisson's ratio of steel is about 0.3, while, for concrete it ranges between 0.15 to 0.25 in the elastic range and 0.5 in the inelastic range. Therefore, the use of the shear studs is essential to achieve a full composite action especially in the elastic range.

Under combined service axial force and moment, the steel jacket will be subjected to compressive circumferential hoop stresses while the concrete will be under tensile stresses. At this stage the presence of the shear connector is important to force the steel jacket to be connected to the concrete to prevent any premature buckling of the steel jacket. However, as the load increases into the inelastic range, the concrete Poisson's ratio will increase more rapidly than the steel ratio, and will thus force the steel jacket to be subjected to tensile circumferential hoop stresses. At this point, the shear studs are not required to combine the two materials together. The steel tensile hoop stresses would confine the concrete and, hence, increase the ultimate flexural capacity of the column.

MATERIAL MODELS AND CURVATURE ANALYSIS

The model used for steel is shown in Figure 2. This model assumes full yielding of the steel with the effect of strain hardening. The model assumes that the steel will behave the same way in both the tension and the compression regions. It is also assumed that the elastic local buckling in the steel jacket is delayed until the section attains its ultimate capacity. The monotonic uniaxial stress strain relationship for the steel jacket is assumed to consist of three distinct regions,[7] as shown in Figure 2: the elastic region, the yield plateau and a strain hardening region. The following equations describe this relationship.

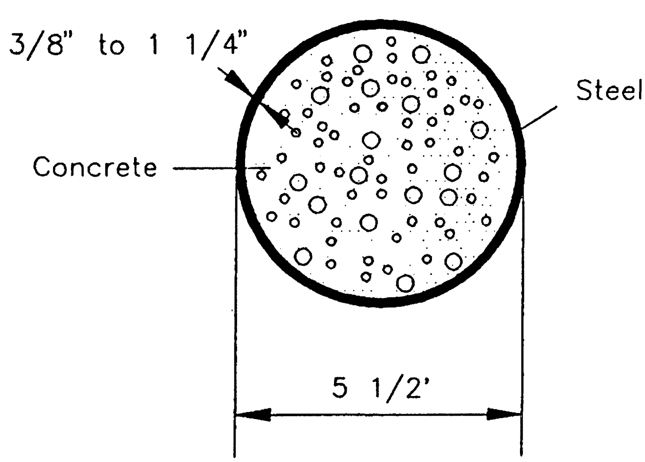

Fig. 1. Composite Steel-Concrete (CSC) column section.

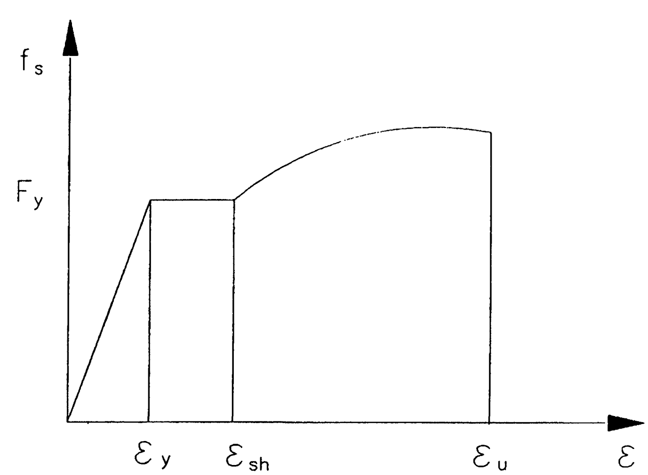

Fig. 2. Stress strain relationship for structural steel.

For $\varepsilon_s \leq \varepsilon_y$,

$$f_s = E_s \varepsilon_s$$

where

ε_s = axial strain
f_s = stress
ε_y = yield strain
ε_s = modulus of elasticity of steel

For $\varepsilon_{sh} < \varepsilon_s \leq \varepsilon_{su}$

$$f_s = f_y\left(\frac{m(\varepsilon_s - \varepsilon_{sh}) + 2}{60(\varepsilon_s - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_s - \varepsilon_{sh})(60 - m)}{2(30r + 1)^2}\right)$$

where

ε_{su} = ultimate strain in the steel
f_{su} = stress in the steel

and

$$m = \frac{(f_{su}/f_y)(30r+1)^2 - 60r - 1}{15r^2},\ r = \varepsilon_{su} - \varepsilon_{sh}$$

Assume that $f_{su} = 1.5 f_y$, $\varepsilon_{sh} = 14\varepsilon_y$, and $\varepsilon_{su} = 0.14 + \varepsilon_{sh}$ and that the tangent modulus at the strain hardening is given by

$$E_{sh} = f_y\left(\frac{2m - 120}{4} + \frac{60 - m}{2(30r + 1)^2}\right)$$

The concrete model is based on the model of Mander et al. as shown in Figure 3. The stress-strain relationship presented by Mander et al. has been proposed mainly for circular and rectangular concrete sections.[1,7] The longitudinal compressive concrete stress f_c is given by

$$f_c = \frac{f'_{cc}\, x r}{r - 1 + x}$$

where

$$f'_{cc} = f'_{co}\left(2.254\sqrt{1 + \frac{7.94 f'_1}{f'_{co}}} - \frac{2 f'_1}{f'_{co}} - 1.254\right)$$

$$f'_1 = \frac{1}{2} k_e \rho_s f_{yh},$$

$$x = \frac{\varepsilon_c}{\varepsilon'_{cc}},$$

$$\varepsilon'_{cc} = \left\{5\left(\frac{f'_{cc}}{f'_{co}} - 1\right) + 1\right\}\varepsilon'_{co},$$

$$r = \frac{E_c}{E_c - E_{sec}},$$

$$E_c = 60{,}200\sqrt{f'_{co}} \text{ in psi},$$

$$E_{sec} = \frac{f'_{cc}}{\varepsilon'_{cc}}$$

f'_{cc} = the compressive strength of the confined concrete
ε_c = the longitudinal compressive concrete strain
ε'_{co} = the longitudinal concrete strain at f'_{co}
f'_{co} = the compressive strain of unconfined concrete
E_c = the modulus of elasticity of concrete
ε'_{cc} = the strain of confined concrete

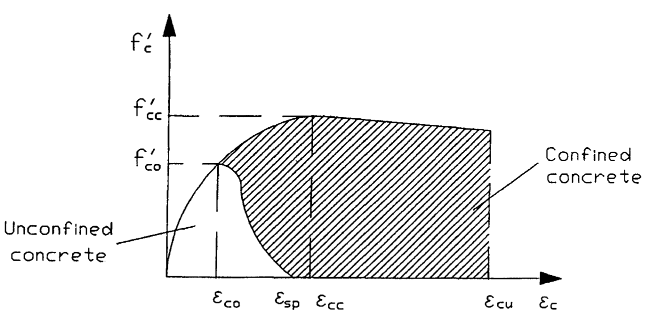

Fig. 3. Stress strain relationship for unconfined and confined concrete.

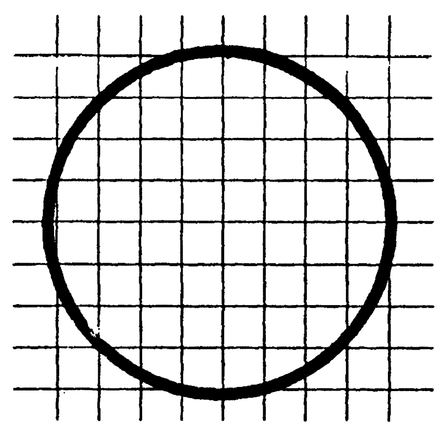

Fig. 4. Concrete fiber elements.

Table 2. Results of Moment-Curvature Analyses (Max. circumferential stress, Max. compressive strength of confined concrete, Max, strain of confined concrete)			
Column Type	f_1' (psi)	f'_{cc} ksi	ε'_{cc} %
C1	407	7.4	3
C2	541	8.0	3.6
C3	675	8.6	4.1
C4	809	9.2	4.5
C5	942	9.7	4.9
C6	1075	10.1	5
C7	1210	10.6	5
C8	1359	11.1	5

Table 3. Results of Moment-Curvature Analyses (Plastic moment, Ultimate curvature, and Curvature ductility)			
Column Type	M_p (k-ft)	ϕ_{ult} (rad/in.)	$\frac{\phi_{ult}}{\phi_y}$
C1	12,625	0.002063	35
C2	15,583	0.002339	40
C3	18,324	0.002372	39
C4	21,305	0.002325	38
C5	24,096	0.002234	36
C6	26,842	0.002243	36
C7	29,758	0.002440	38
C8	32,966	0.002384	37

In this model the effect of the confinement of the steel jacket is calculated from the equilibrium of forces acting on a dissected section as discussed in Reference 7.

The cross section of the composite column was divided into 100 fibers, composed of each of both concrete and the steel jacket as shown in Figures 4 and 5. These fibers represent steel and concrete longitudinal fiber each of which is assigned a uniaxial stress-strain relationship The effect of the confinement was calculated in terms of confining pressure or as circumferential stress f_1'. Thus, the steel jacket would be subjected to a biaxial state of stress as shown in Figure 6. Longitudinal stress, f_s, develops due to axial load and bending moment whereas a circumferential stress, f_1', develops due to concrete confinement. The two stresses define the yield criteria as outlined by Von Mises' yield criteria.

$$f_s^2 + f_1'^2 + f_1' f_s = F_y$$

where

F_y = the yield stress of steel jacket.

Each fiber in the cross section has an area, coordinates, and a stress strain relationship. The curvature analysis is performed based on the assumption that plane sections remain plane after

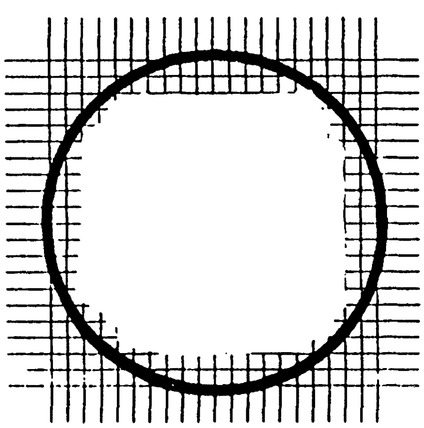

Fig. 5. Steel fiber elements.

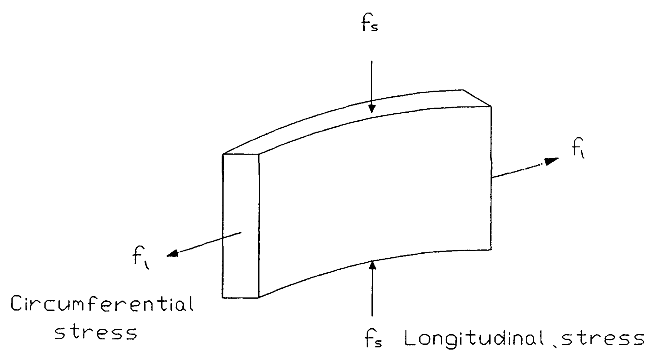

Fig. 6. Longitudinal and circumferential stresses in the steel jacket.

bending. The analysis was initiated by assuming a strain diagram that defines a critical section. Thus the stress diagram is computed based on the nonlinear constitutive relationship for steel and concrete. Based on this generated stress diagram, a static equilibrium state must be satisfied. Equilibrium is checked by equating the sum of forces from the various fibers to the applied forces. Thus, equilibrium is satisfied using iterative process in which the assumed strains are successively changed.

RESULTS OF CURVATURE ANALYSIS

The curvature analysis was performed on the strain compatibility of the steel and the concrete. A constant axial force of 2,000 kips, a typical load for a column of this size supporting a structural concrete box girder superstructure. A modified version of the *Colret* program, developed at University of California, San Diego,[7] was used to perform the curvature analysis. Figure 7 shows the results of the curvature analysis, M-ϕ, diagram. This figure shows that the composite columns manifest a high curvature demonstrating that these columns have excellent deformation capacity in the inelastic range. The curvature ductility used in this study is defined as the ratio of ultimate curvature, ϕ_{ult}, to the yield curvature, ϕ_y The yield curvature is calculated when the tension side of the steel jacket starts yielding. Tables 2 and 3 present the results of moment curvature analyses The curvature ductility ratio for all of the composite columns studied ranged between 35 to 40 as shown in Table 3. This is a good indication that these columns exhibits considerable energy dissipation capabilities necessary for adequate seismic performance. The plastic moment ranged from 12,625 k-ft for C1 column to 32,966 k-ft for C8 which indicated that the plastic moment increased on the order of 2.5 between C1 and C8. This significant increase in the moment capacity while retaining almost the same column dimensions makes composite columns highly suited to metropolitan areas.

The steel jacket thickness played a significant role in increasing both the flexural strength and the lateral confinement of the composite column. As shown in Table 2 the lateral pressure, f_l', varied between 407 psi for C1 column to 1,359 psi for C8 column. The lateral confinement increased the strength Of the unconfined concrete f_{co}' from 5,000 psi to 11,000 psi for C8 column. This significant increase in the concrete strength cannot be achieved with conventional hoops or spiral reinforcement in reinforced concrete columns. Due to this high confinement and large percentage of longitudinal reinforcement, the concrete cracking is minimized, indicating that the gross moment of inertia could be used to calculate the stiffnesses of these columns from the slope of the M-ϕ, diagram. It is customary for the reinforced concrete columns to have an effective moment of inertia within the range of 40 percent to 60 percent of the gross moment of inertia. CSC columns enhance effective stiffness since the total moment of inertia can be substantiated for the effective moment of inertia.

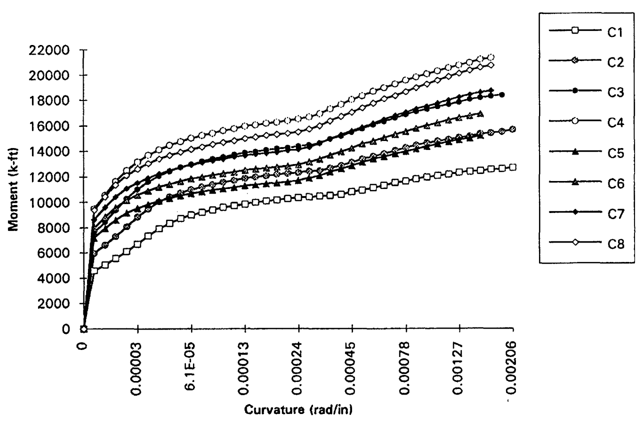

Fig. 7. Moment-Curvature, (M-f), diagram for various steel thicknesses.

CONCLUSIONS

The results of the analytical analysis showed that CSC columns possess significant curvature ductility that could be utilized in high seismic zones. The steel jacket proved to be sufficient to provide the strength and confinement for composite columns. Effective concrete confinement and high longitudinal reinforcement ratios were feasible with the use of steel jacket to a level that significantly minimized the concrete cracking.

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