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Concrete Filled Steel Tubes for Performance Improvement of Steel Truss Bridges

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Abstract: The use of concrete-filled steel tubes (CFST) in engineering structures has become popular because of their excellent seismic resistance structural properties such as high strength, high ductility and large energy absorption capacity. In CFSTs the surrounding steel tube provides effective confinement to the filled-in concrete and in turn the concrete helps to reduce the potential local buckling of the steel tube resulting improved seismic resistant performance. This study aimed at investigating the benefit of CFST members in railway steel truss bridges susceptible to earthquake loads. Since the end frames of truss bridges are mainly subjected to compressive loads CFST is a good alternative for end raker. The steel weight of the rib can be reduced with CFST and hence the method is economically sound. The seismic behaviour of steel truss bridges with steel and CFST end rakers is discussed based on the results of nonlinear time history analyses. Five truss bridges were designed with different types of end rakers namely existing HEB end raker, square hollow end raker, three square hollow CFST end raker bridges with varying concrete grades. Time history analyses were performed for transverse direction using selected past earthquakes and natural frequencies, maximum vertical and lateral deflections, residual vertical and lateral deflections and member stresses were checked. It was found that the use of CFST in steel truss bridges can be effectively utilized to improve the seismic resisting performance.

Keywords: Concrete-filled steel tube, truss bridges, material nonlinearity, seismic resistance capacity

1. Introduction

The use of concrete-filled steel tube (CFST) is a well-recognized technique for improving strength and ductility of steel structures. It is economical as CFTS members have much higher strength than the sum of individual strengths of equivalent steel and concrete members. The CFST is essentially a composite member where, usually, thin-walled steel tube is filled with concrete. The concrete in the tube improves the stability of the thin-walled steel tube in compression and delays its local buckling while steel tube confines the inside concrete resulting triaxial compression stress state in the concrete. Therefore, CFST has higher compression capacity and ductility [1, 2, 3, 4, 5, 6]. Moreover, in construction the tube acts as the formwork for the concrete. Hence, the technique is good for the application in bridges for members with compressive force and bending moments [7, 8, 9].

The ultimate strength of CFST members highly depends on the material properties such as yield strength of steel and compressive strength of the concrete and geometrical properties such as width-to-thickness ratio or diameter-to-thickness ratio of cross section [6]. The tubes can provide good confinement when circular hollow sections are used, especially, when the diameter to thickness ratio (*D/t* ratio) is small. Local buckling of steel tube is not likely to occur when *D/t* ratio is lower than 40 [10, 11, 12].

The CFST members have been innovatively applied in various structures as bridges, roof structures, sports stadiums, multi storey frame buildings with excellent seismic resistance. For bridges, CFST is adopted for arch ribs, members of truss girders and piers of which the members induced axial compression forces and bending moments [13, 14].

In this research seismic behaviour of a railway truss bridge is investigated based on the results of series of nonlinear dynamic analyses. An existing modified warren type 47.344 m span railway truss bridge at Kalutara on Southern Railway line in Sri

Lanka was modelled using finite element analysis method with and without CFST member for end raker. End raker is the end diagonal member of main truss which create the end frame of bridge to withstand lateral loads. With the use of time-history analyses performed in transverse direction for selected past earthquakes, maximum stresses and deflections were studied in view of identifying the effects of CFST members in improvement of seismic resisting capacity of steel truss bridges.

2. Design of Steel Bridges

The end raker of the existing single track railway bridge was checked for induced maximum design forces and verified the available member has an adequate strength to withstand the loads. For that, a linear static analysis was performed for relevant railway loads specified in BS 5400 part 2 [15] and Railway track and bridge manual [16]. The member was designed according to BS 5400 part 3 code [17]. Then another four bridges were designed for the purpose of comparison of performance of the bridges by replacing the end raker member with square hollow steel section and CFST members. The CFST member was designed according to the method in BS 5400-5 code [18].

The strength of steel was 350 N/mm^2 and concrete was varied from 30 N/mm^2 to 50 N/mm^2. Square hollow sections with B/t ratio lower than 40 were used for the end raker. The bridges are of modified warren type trusses with 47.344 m long span. First bridge, namely HEB, was with a European standard wide flange H steel beam (section HEB 300). Second bridge, namely SHS, was with square hollow section (section SHS 250x250x12.5mm) and the third bridge, namely CFST-C40, was with a square hollow section (section SHS 250x250x8mm) filled with grade 40 concrete.

3. Finite Element Model

Three types of truss bridges including existing railway bridge designed as explained in the previous chapter were modelled using finite element program OpenSees [19]. The software can be employed for linear and nonlinear structural and geotechnical modelling. In addition to the designed CFST end raker bridge with C40 filled in concrete, another two bridges with CFST end rakers with grades C30 and C50 concrete were modelled for the purpose of comparison. The model created with force based beam column elements for main truss girder members, stringers, cross girders and bracings as shown in Figure 1. The cross sections of H beam, hollow and composite members were divided into number of segments (fibres). The fibre arrangements of three basic types of end raker members are shown in Figure 2. The fibre sections of components were interpreted by assigning available uniaxial stress-strain relations for steel and concrete for discretised smaller regions.

The uniaxial bilinear material model assigned for steel consists of Young's modulus, E = 205 GPa, yield strength, Fy = 355 Mpa and hardening ratio, b = 0.015. For filled-in concrete in CFST members, a uniaxial concrete model with zero tensile strength was used. The parameters used for the model are; compressive strength, fpc = 40 Mpa, strain at fpc, epsc0 = 0.002, crushing strength, fpcu = 8 MPa and strain at fpcu, epsu = 0.0037. The unconfined concrete model explained in M. K. M. Reddiar [20] was adopted here.

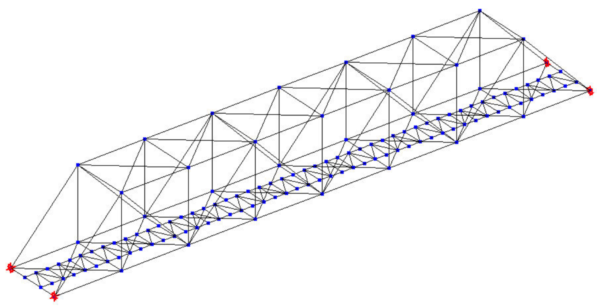

Figure 1: Finite Element Model of Bridge

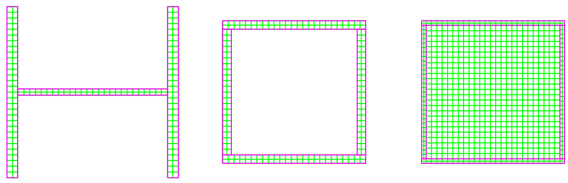

Figure 2: Fibre arrangements of end raker members

To evaluate the earthquake response characteristics, a time history analysis was performed considering the material nonlinearity. The time history analysis was carried out for each model for several past earthquakes recorded in different parts of the world. The relevant acceleration records were downloaded from PEER database [21]. The main parameter considered in selecting the earthquake records was the peak ground acceleration (PGA) even though the duration of the earthquake and local ground condition are also considered to be important in identifying behaviour of structures.

The names and PGA of the earthquake records used in this study are listed in Table 1. Each earthquake has two horizontal components. The analyses were carried out only in transverse direction of the bridge. Some selected components of these earthquake records are shown in Figure 3.

The selected earthquake records represent minor, moderate and severe earthquakes. Thus, the effects of use of CFSTs in truss bridges are investigated for different magnitudes of ground accelerations.

Table 1: Selected earthquake records and their peak ground accelerations (PGA)

Earthquake	Peak Ground Acceleration (PGA)/(g)
Cape Mendocino – 1992	0.178
Whittier Narrows – 1987	0.304
Coyote Lake – 1979	0.434
Erzincan – 1992	0.515
Victoria Mexico – 1980	0.621
Landers – 1992	0.721
Kobe – 1995	0.821

4. Results and Discussion

The results of the time history analysis of each bridge model under each earthquake which are recorded to estimate the influence of the earthquakes for the bridge are summarized here. The fundamental periods *(T)* of each bridge are listed in Table 2. It is clear from these results that fundamental periods of all three bridges do not differ much. This means that concrete-infilling does not have a great effect on fundamental periods. This will be useful in seismic design of CFST truss bridges.

The displacements of Node 1 and Node 2 which are located at the top of the end frame and mid span of the top chord consecutively are recorded for static and time history analysis. The maximum stresses induced in steel and concrete also presented separately for each type of analysis. Figure 4 shows Node 1, 2 and end raker member of which the results were obtained.

The maximum vertical deflections of the bridges from static analysis are given in Table 3. The displacements at Node 1 represents by $\delta max,z_1$ while displacement at Node 2 represent by $\delta max,z_2$. Maximum vertical displacements at two nodes have been slightly decreased when CFST member introduced and the bridge type CFST-C50 has the minimum vertical displacement among five. This is due to the fact that stiffness of the bridge increases with concrete infilling and the grade of the concrete increasing. On the other hand, the effect due to increase of self-weight would not be significant compared to the increase of stiffness. This fact was evident in fundamental periods of the bridges as well. The maximum steel and concrete stresses induced in extreme fibres of end raker member are given in Table 4. The stress in steel has been reduced when CFST end raker introduced. However, maximum concrete stress in CFST has been increased when the grade of the concrete increased.

The maximum lateral displacements of the Node 1 ($\delta max,y_1$) and Node 2 ($\delta max,y_2$) of the bridge subject to each earthquake are summarized in Table 5 and 6, respectively. It is noted that the maximum horizontal displacements at Node 1 and Node 2 have been slightly increased in most of the earthquakes when CFST members introduced. However, displacement at Node 1 in Victoria-1980 earthquake and displacement at Node 2 in Vicoria-1980 and Kobe-1995 earthquakes were slightly decreased in CFST bridges.

The lateral residual displacement of the Node 1 ($\delta res,y_1$) and Node 2 ($\delta res,y_2$) of the bridge are summarized in Table 7 and 8, respectively. The residual displacement in lateral direction at Node 1 and Node 2 have been slightly decreased in Cape Mendocino-1992, Coyote Lake-1979 and Kobe-1995 earthquakes while slightly increased in other earthquakes.

The maximum steel and concrete stresses induced in extreme fibres of end raker member are summarized in Table 9 and 10. The maximum steel stress induced in the end raker member was decreased when CFST member introduced. When the grade of the concrete increased, the stress has been further reduced. However, the maximum concrete stress in CFST has been increased when the grade of the concrete increased. These results imply that the response depends not only on PGA but also on features like period and duration of ground accelerations.

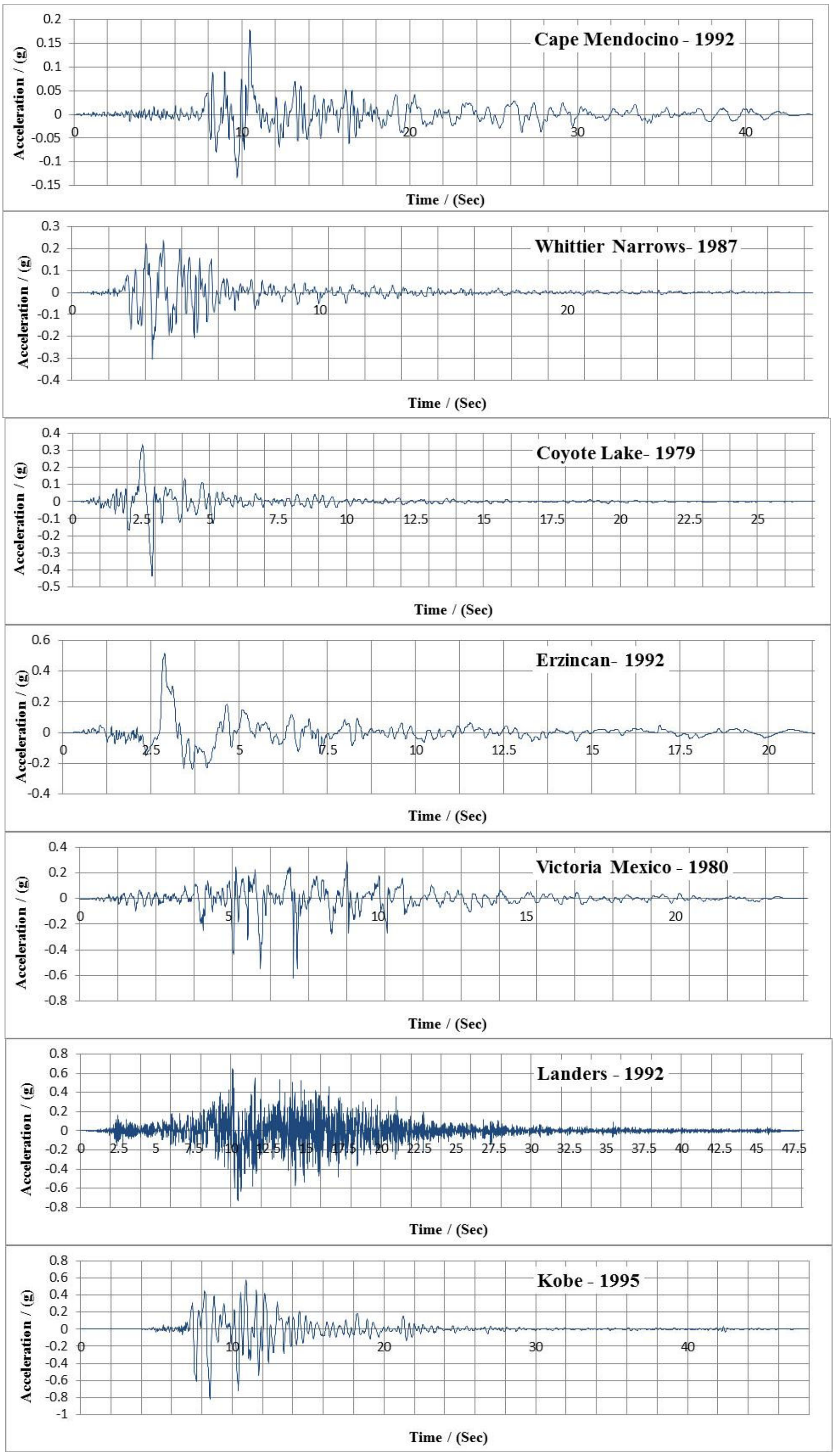

Figure 3: Selected components of earthquake records

Table 2: Fundamental periods

Bridge	T / (Sec)
HEB	1.107
SHS	1.096
CFST-C30	1.095
CFST-C40	1.096
CFST-C50	1.098

Table 3: Maximum vertical deflections (static analysis)

Bridge	$\delta max,z_1$ /(mm)	$\delta max,z_2$/(mm)
HEB	28	73
SHS	29	74
CFST-C30	28	73
CFST-C40	27	73
CFST-C50	26	72

Table 4: Maximum stresses (static analysis)

Bridge	Steel/(N/mm^2)	Concrete/(N/mm^2)
HEB	-185	-
SHS	-193	-
CFST-C30	-169	-19
CFST-C40	-153	-24
CFST-C50	-142	-28

Table 5: Maximum lateral displacement at Node 1

Earthquake	Maximum displacement / (mm)				
	HEB	SHS	CFST-C30	CFST-C40	CFST-C50
Cape	24	27	28	27	27
Whittier	50	54	55	54	54
Coyote Lake	97	109	111	110	109
Erzincan	218	254	267	263	260
Victoria	51	51	51	51	50
Landers	56	63	64	63	62
Kobe	164	173	175	174	174

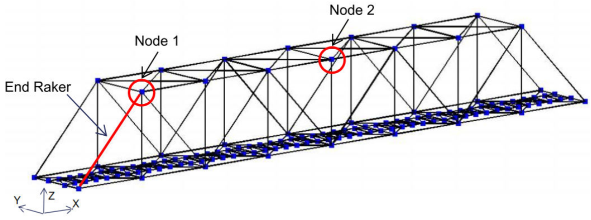

Figure 4: Node 1, Node 2 and end raker member

Table 6: Maximum lateral displacement at Node 2

Earthquake	Maximum displacement / (mm)				
	HEB	SHS	CFST-C30	CFST-C40	CFST-C50
Cape	29	31	31	31	31
Whittier	59	60	60	60	60
Coyote Lake	104	111	112	111	111
Erzincan	223	246	250	250	248
Victoria	57	54	54	54	54
Landers	61	65	66	66	65
Kobe	166	164	165	165	165

Table 7: Lateral residual displacement at Node 1

Earthquake	Maximum displacement / (mm)				
	HEB	SHS	CFST-C30	CFST-C40	CFST-C50
Cape	0	0	0	0	0
Whittier	1	2	2	2	2
Coyote Lake	5	7	6	6	6
Erzincan	15	38	54	51	45
Victoria	8	11	12	12	12
Landers	25	29	30	29	28
Kobe	8	8	7	6	6

Table 8: Lateral residual displacement at Node 2

Earthquake	Maximum displacement / (mm)				
	HEB	SHS	CFST-C30	CFST-C40	CFST-C50
Cape	1	0	0	0	0
Whittier	1	1	1	1	1
Coyote Lake	5	4	3	3	3
Erzincan	6	26	31	36	32
Victoria	9	11	11	11	11
Landers	22	24	25	24	24
Kobe	13	13	13	12	12

Table 9: Maximum stresses induced in steel

Earthquake	Maximum stress / (N/mm^2)				
	HEB	SHS	CFST-C30	CFST-C40	CFST-C50
Cape	-228	-239	-220	-204	-191
Whittier	-271	-277	-262	-246	-234
Coyote Lake	-337	-344	-337	-319	-305
Erzincan	-373	-375	-374	-373	-372
Victoria	-331	-350	-344	-325	-309
Landers	-340	-355	-355	-342	-324
Kobe	-357	-357	-357	-357	-356

Table 10: Maximum stresses induced in concrete

Earthquake	Maximum stress / (N/mm^2)				
	HEB	SHS	CFST-C30	CFST-C40	CFST-C50
Cape	-	-	-23	-29	-35
Whittier	-	-	-26	-33	-39
Coyote Lake	-	-	-29	-37	-46
Erzincan	-	-	-30	-40	-50
Victoria	-	-	-29	-38	-46
Landers	-	-	-29	-38	-47
Kobe	-	-	-30	-40	-50

5. Conclusions

The application of concrete-filled steel tubes (CFST) in truss bridges was investigated in view of identifying the seismic resisting behaviour. The effects of CFSTs on the maximum and residual displacement demands, and strength demands were checked for several past earthquake records. The main findings of the study are:

1. The fundamental natural frequencies of the bridge with H section or hollow section end raker and bridge with concrete-filled end raker do not differ much. However, this fact should be thoroughly investigated through a parametric study using different dimensions and material properties.
2. The maximum vertical displacement under static loads has significantly reduced when CFST is introduced. Therefore, CFST will be an effective technique for displacement control of railway truss bridges.
3. It is possible to reduce the thickness of steel tubes while maintaining the displacement under serviceability limit states with the use of CFSTs.
4. The behaviour of maximum and residual displacements of the bridge when subjected to a seismic event shows that the behaviour of bridges is not depending only on peak ground accelerations. Hence, further studies are needed considering other earthquake parameters such as acceleration profile, duration, frequency content and energy content for more accurate prediction of the seismic behaviour.
5. The maximum steel stresses induced in end raker member have decreased when CFSTs were introduced. Therefore, it is clear that CFST can be employed to improve the seismic performance of truss bridges.
6. Although the steel stresses were reduced, concrete stresses were increased when higher grade concrete used for composite member. Further studies are needed to find the minimum and maximum concrete grade that can be used for CFST bridge members in economical manner.

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