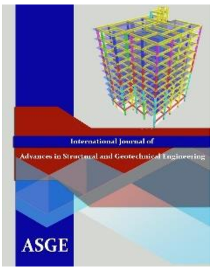

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EVALUATION OF SEISMIC RESPONSE OF CFST BRIDGE COLUMNS WITH DIFFERENT CONNECTIONS TO FOUNDATION

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ABSTRACT

This study presents a general evaluation of the seismic response of concrete filled steel tube (CFST) bridge columns. Column to footing connection is considered as a critical design parameter, so available experimental results for different connections are discussed. With regard to the structural response of CFST bridge columns integrally connected to reinforced concrete (RC) footing, the data collected are divided into emulative connections (socket or grouted embedded connection and exposed connection) and non-emulative connection (prefabricated structural units that are connected together using post-tensioned tendons). With additional external confinement of the critical flexural zone of the socket connection, this system can ensure high drift capacity with a stable hysteretic response that is appropriate to moderate earthquake regions. However, due to the elastic response of its main components, very limited residual inclination and quick and easy replacement of damaged parts, hybrid rocking connection with buckling restrained steel plate (BRS) can be used in high seismic zones.

Keywords: CFST; Seismic; Emulative connection; Prefabricated; Footing; Bridge columns.

INTRODUCTION

Bridges in earthquake areas must be able to withstand strong ground vibrations with little damage and minimal residual deformation so that they can be used immediately for emergency vehicles and minimize traffic interruptions after the earthquake. Therefore, the choice of bridge system is one of the important issues that have plagued researchers. As one of the important substructure bridges, in the past few decades, concrete-filled steel tube (CFST) columns have been used in bridges with fixed/rigid connections. This type has many characteristics under axial loads and in turn it has achieved great success in various countries. Compared with pure steel column, it reduces the amount of steel as well as increasing the bearing capacity of the column, as mentioned by [[1], [2], [3], and [4]], Although there are many criticisms in the use of this type in many countries, especially in United State, as mentioned in [1], and [3]. That's because: (1) few practical and economical connections are available, (2) the composite action of CFST is misunderstood and (3) design provisions in the AASHTO, AISC and ACI specifications are limited. Studying this connection under earthquake and gravity loads, the development of plastic hinge in the CFST column can cause permanent damage and inelastic deformation, resulting in irreparable seismic response including weld fracture, bolt fracture, overall damage, large residual displacement, and concrete crushing. Damage to these structures may cause the bridge to be temporarily closed to the public and need a high cost for repair, as is evident in [[1], [2], [5], [3],and [4]]. In order to improve the seismic performance of columns, recent studies have investigated the effectiveness of using self-centering (SC) system.

SC system is a new type of seismic system that has been studied through experimental and numerical calculations in the last decades. It has the advantage of gap opening strategy between the column and the foundation and existing post-tensioning elements at the connection help the column to prevent residual drift and return the structure to a plumb (upright position (self-center) after the earthquake has passed), as mentioned in [6]. Although this system has achieved remarkable success, it has a lack of damping capability. Therefore, adding replaceable energy dissipation (ED) devices/components to the structure is critical as mentioned in [[7], [8], [9], [10] , [11], and [12]].

This paper provides a general summary evaluation of the seismic response of several proposed connections of CFST columns to the foundation. In addition, comparison between emulative and non-emulative connections. The focus is on the general lateral response (load-drift response), energy dissipation capacity, and permanent deformations.

CFST BRIDGE COLUMNS-TO-FOUNDATION CONNECTIONS

There are three types of CFST connection: (1) Monolithic connection; (2) Emulative connection; (3) Non-emulative connection. As shown in Fig. 1.

Monolithic Connection

Monolithic connection directly embeds the tube and annular ring into the foundation concrete, as shown in Fig. 1(a), as mentioned in [[1], [3], and [13]].

Emulative connections

Emulative construction uses connections that are designed and detailed to make the performance (in terms of lateral strength, stiffness, and energy dissipation) of the structure comparable to that of a monolithic structure.

Emulative connections are further divided into two categories: ductile and strong. Structures with ductile connections are designed to undergo flexural yielding and form ductile plastic hinges in the connections across column-to foundation joints, whereas structures with strong connections are designed to experience flexural yielding within the columns at preselected and appropriately detailed locations adjacent to or away from the joints and the plastic hinges fully develop elsewhere in the structure. This study will show five types of ductile emulative connection: (1) Exposed connection; (2) Grouted connection; (3) Pocket connection; (4) Socket connection; (5) Coupled connection, as shown in Figs. 1(b, c, d, e, and f), respectively.

Grouted connection, a void is cast into the foundation with a diameter slightly larger than the outside diameter of the annular ring. The tube is placed into the void, then supported in the void using threaded rods which extend through the annular ring, and then the recess between the tube and corrugated pipe is filled with high strength grout, as shown in Fig. 1(c), as mentioned in [[1], [14] , [3], and [13]].

Pocket connections are originally designed to connect columns to foundations. They leave a large opening, or pocket, in a member of the bridge footing. The projecting reinforcement from column is inserted into the pocket. The pocket is then filled with cast-in-place (CIP) concrete, as shown in Fig. 1(d), as mentioned in [[13], and [15]]. This type has been used extensively in precast concrete column, and no research is found for CFST.

Socket connections involve embedding portion of prefabricated CFST column into a cavity within the precast footing and then filling the cavity with cast-in-place concrete or grout. It differs from a pocket connection in that reinforcing bars are completely encased in each member, and there is no bar reinforcement that crosses the interface between the two connecting members. Interface surfaces are often roughened to enhance the bond resistance, as shown in Fig. 1(e), as mentioned in [[13], [15], [14], and [16]].

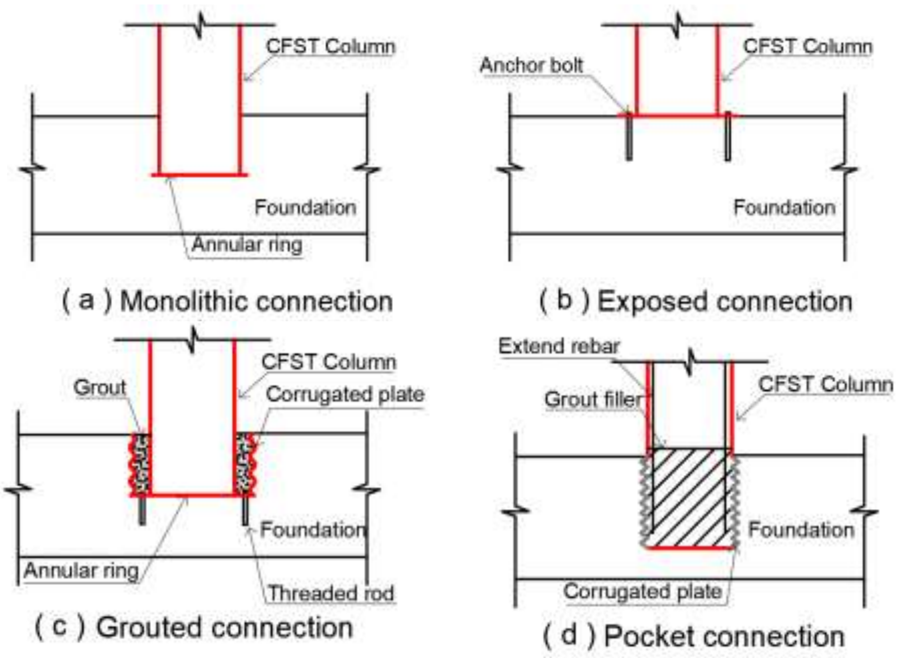
Coupler connections can also be used when smart materials, such as shape-memory alloy, are used in the plastic hinge regions connected with mild steel used elsewhere. The coupler connections may be located either in the column portion or footing portion, as shown in Fig. 1(f), as mentioned in [[14], and [17]]. This type has been used extensively in pure steel and precast concrete column, and no research is found for CFST.

Non-emulative connection

Non-emulative connection uses concepts that are distinctly different from emulative connections, the nonlinear rotations of the structure are deliberately concentrated at the ends of the column in the joint regions (through controlled rocking at the joint interface), without causing significant inelastic behavior or damage in the column. This unique behavior has been achieved by using unbonded post-tensioning (PT) tendon. There are two types of this connection: (1) simple rocking column; (2) hybrid rocking column.

Simple rocking columns are designed to rotate without any restraint from continuous rebar at the joints or any energy dissipation (ED) devices, as shown in Fig. 1(g) [[18], [14], and [13]]. This type is used to explain the concept, but it can't be used without energy dissipator in moderate-to-high seismic regions.

Hybrid rocking columns are designed to achieve a balance between emulative and simple rocking columns. The hybrid rocking column is characterized by both unbonded posttensioned (PT) tendon and longitudinal rebar connecting rocking components ED bar whether ED is internal or external, as shown in Fig. 1(h) [[18], [14], and [13]].



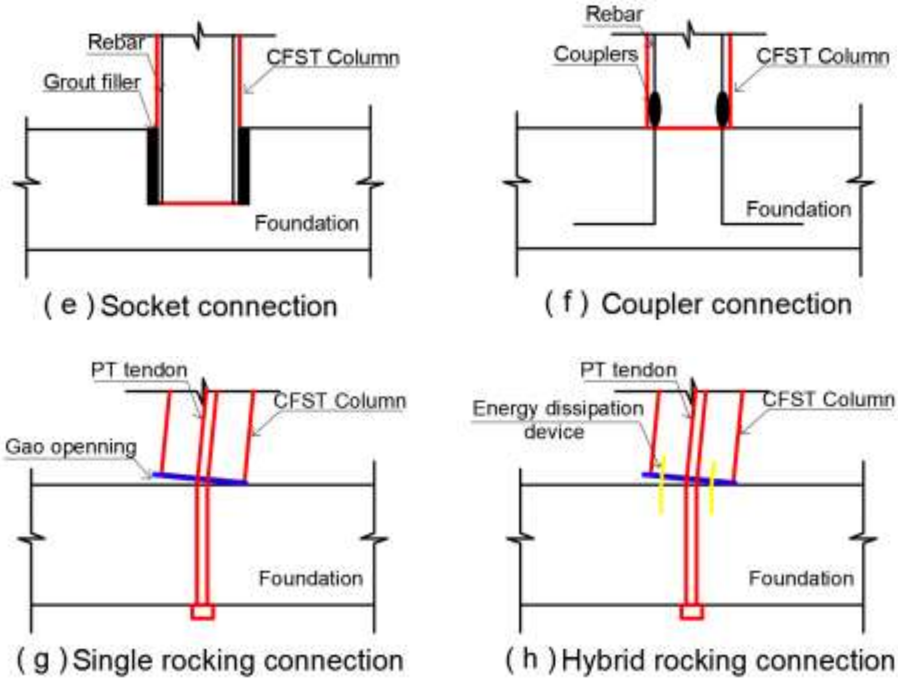


Fig. 1: Types of CFST connections

PREVIOUS WORK ON MONOLITHIC AND EMULATIVE CONNECTIONS

Exposed connection

Exposed connection, as shown in Fig. 1(b), was studied by [[19], and [20]], where the difference between them was in the shape and dimensions of the cross section. The cross section was square and circular in the first and second studies, respectively.

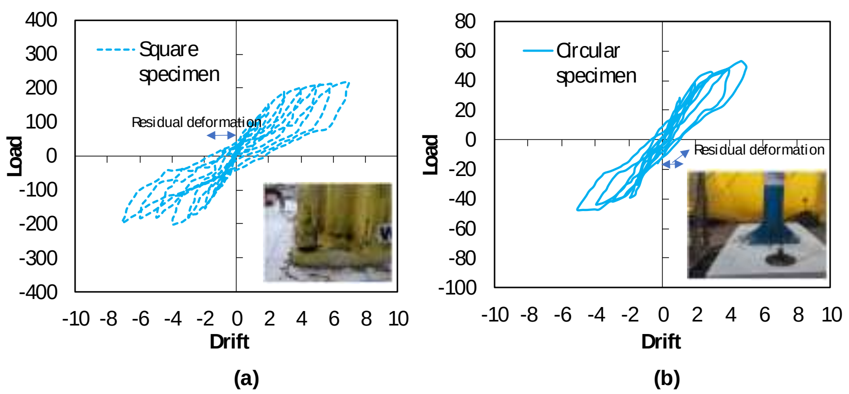


Fig. 2: Hysteretic response of specimens with failure mode; (a) [19]; **(b)** [20]

It can be observed from Fig. 2(a) that inelastic elongation of the anchor bolts was exhibited in the exposed base connection, poor hysteretic performance can be achieved with this specimen. On the other hand, there is small residual deformation. It can be observed from Fig. 2(b) that the bond between the interface of CFST column and the foundation decreased slowly and the anchor bolts in exposed column connection exhibited yielding. Severe damage occurred on the foundation concrete at 5% drift, as shown in Fig. 2(b). After 5% drift, the connection reached a peak load. It can be clearly seen from Fig 2. That there is a difference in performance between square and circular specimen with differences in dimensions, but in general, the circular specimen did not show degradation and had better performance than the square specimen.

Embedded connection

- **Monolithic connection**

Monolithic connection, as shown in Fig. 1(a), was studied in only one research [1]. The specimen was circular.

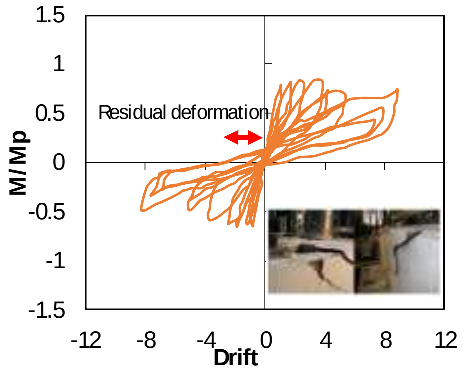


Fig. 3: Hysteretic response of specimens with failure mode in [1]

In Monolithic connection, at less than 0.3% drift, cracks initiated at the column-footing interface. A maximum moment was achieved during cycling at 2.4% drift. Increased degradation in resistance was noted at large drifts. At approximately 4% drift, the footing was severely damaged, and a large cone of concrete separated from the footing as shown in Fig. 3. Loading was terminated by the original authors at 8% drift due to widespread footing damage. In this study, all the monolithic connections failed in the form of cone pullout, which indicated that it was a mistake to choose a smaller embedding depth or footing dimensions in the design, but this failure mode was improved as the footing dimension increases to ductile tearing but there was no data for these specimens in the research, as mentioned in [1]. So, this type will come out of the comparison.

- **Grouted connection**

Grouted connection, as shown in Fig. 1(c), was studied by [[19], [1], and [20]]. The main difference between these studies are in the shape of the cross section, dimensions, and the height of the specimens. In [19], The embedded depth and stiffeners were design parameters. However, in [20], the embedded depth only was design parameter.

In [19], it was noted that when further comparing the responses of the 1-S-0.5D and 1-E-0.5D specimens, although the CFT columns in both specimens were embedded in the foundations at the same depth, the stiffened 1-S-0.5D successfully developed full CFT column strength, while the unstiffened 1-E-0.5D exhibited premature cracking in the base concrete before the CFT member reached its moment capacity, as shown in Fig. 4(a, and d). These phenomena revealed that the critical embedded lengths, i.e. lengths to prevent base concrete failures, can be significantly reduced when the strengthening scheme is adopted. The critical embedded length for the unstiffened specimen was approximately 1D. This value was significantly reduced to 0.5D in the stiffened specimen. This reduction indicated an improved efficiency and justified the effectiveness of the proposed strengthening scheme in the base connection design. Further benefits of adopting the stiffeners in the base connection designs can be validated by the enhancements in the rigidity of the base connections and the energy dissipation of the system, as shown in Fig. 4(a, b, c, d, and e).

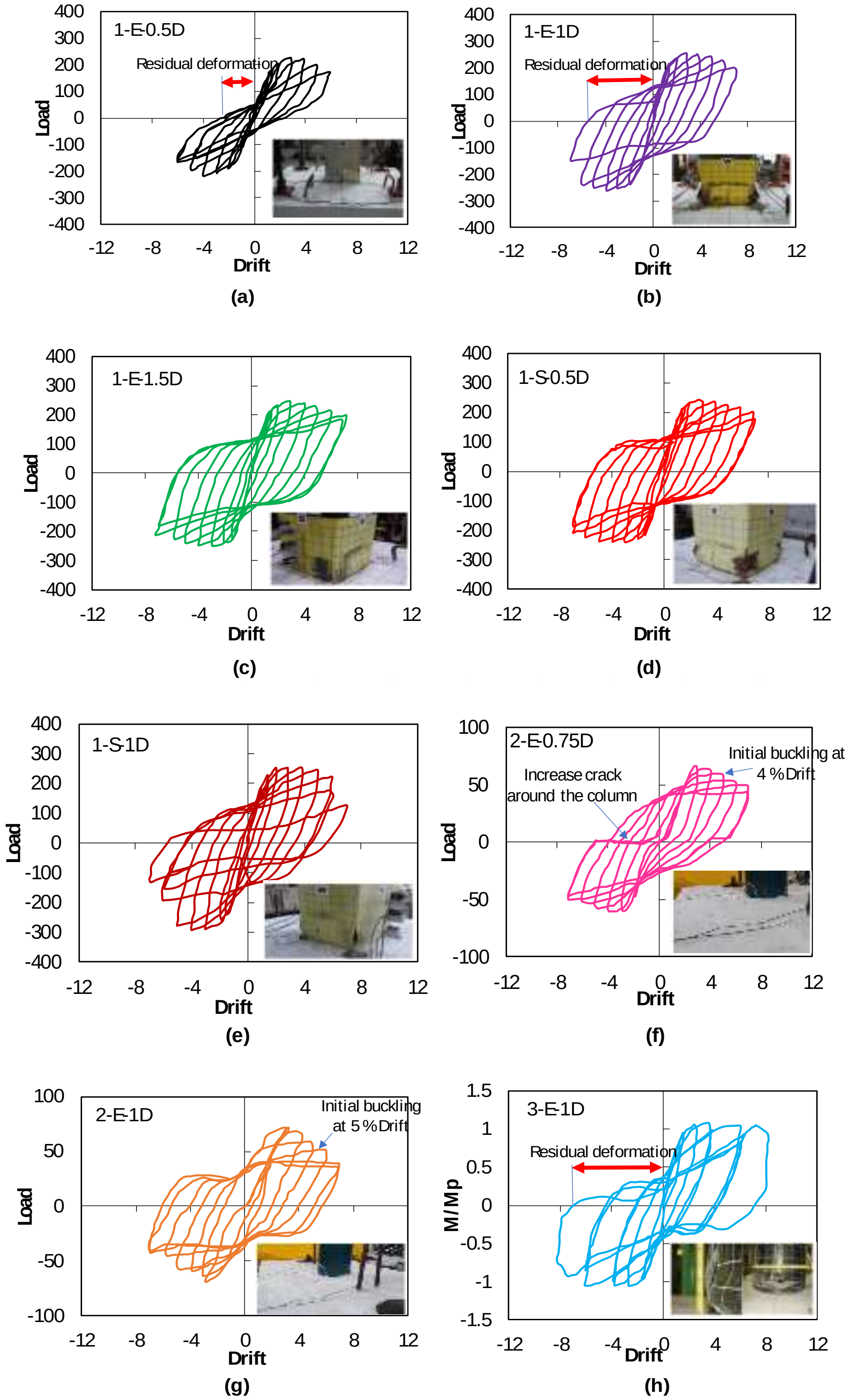


Fig. 4: Hysteretic response of specimens with failure mode for [19]**; [(a), (b), (c), (d), and (e)], for** [1]**; [(f), and (g)], and for** [20]**; [(h)]**

At the initial stages of loading for embedded connection with embedded length 2-E-0.75D in [1], no damage was observed until the drift level reached 1% drift. After 2% drift, there was deviation in the load- displacement curve, as shown in Fig. 4(f). During the 2nd cycle of push load at 3% drift, the crack was noticed between the interface of the CFST column and foundation connection. The maximum load reached at 3% drift. At 4% drift, first push and pull loading cycles, a detectable local buckling was identified on both sides of the column faces. At 6% drift level, the local buckling continued. After 6% Drift, inelastic buckling occurred in the CFST column, as shown in Fig. 4(f). The width of crack around the interface increased at 7% drift. At the beginning of the test for the specimen 2-E-1D in [1], the specimen displayed a linear behavior. At 3% drift, the specimen achieved a maximum yield load. The foundation concrete was intact, and no cracks were identified near the interface of column foundation connection. At 4% drift level, very small cracks were found in the foundation concrete. At 5% drift level, local buckling was found on the CFST column. When the load was released, local buckling of column got reduced. Finally, at 7% drift, the column reached the inelastic buckling with the strength degradation, as shown in Fig. 4(g). In [20], at low drift level, very small cracks formed in the footing, but these cracks remained small and local with increasing lateral displacements. Tube yielding was measured at approximately 1.3% drift. The maximum moment was achieved at 2.4% drift. Visible local tube buckling as illustrated in Fig. 4(h) was observed at approximately 4% drift. Local buckling did not influence the load carrying capacity of the CFST. Tearing initiated at the apex of the buckle at approximately 6% drift, and then tearing propagated around the base of the tube at 8% drift as shown in Fig. 4(h). In conclusion, it is clear from Fig. 4 that there is a difference in behavior between the circular column that contains stiffeners and the circular column without stiffeners [[20], and [1]], where the effect of the stiffeners positively affected the performance. In addition, it can be seen that the square column gives better performance than the circular column [[19], and [20]], which can be attributed to the expected higher contact area between the column embedded part and foundation, especially for columns with the same diameter and D/t ratio.

- **Socket connection**

Socket connection, as shown in Fig. 1(e), was studied in [16]**,** In which, five prefabricated CFDST columns with various column base details were tested under simulated seismic loads until failure, as summarized in Table 1.

Table 1: Details of test specimens

Specimen name	**Embedded depth**	**Column base details**
CFDST1	1D	Reference specimen
CFDST2	1D	Two steel rings confined the column base with a gap
CFDST3	1.5D	Two layers of Carbon fiber reinforced polymer (CFRP) wraps with a 2mm-wide gap confined the column base
CFDST4	1.5D	The column base was confined by two layers of CFRP wraps directly glued onto it
CFDST5	1.5D	two layers of CFRP wraps with a 2mm-wide gap confined the region started from 137mm below than the top surface of the footing to 200mm above the footing

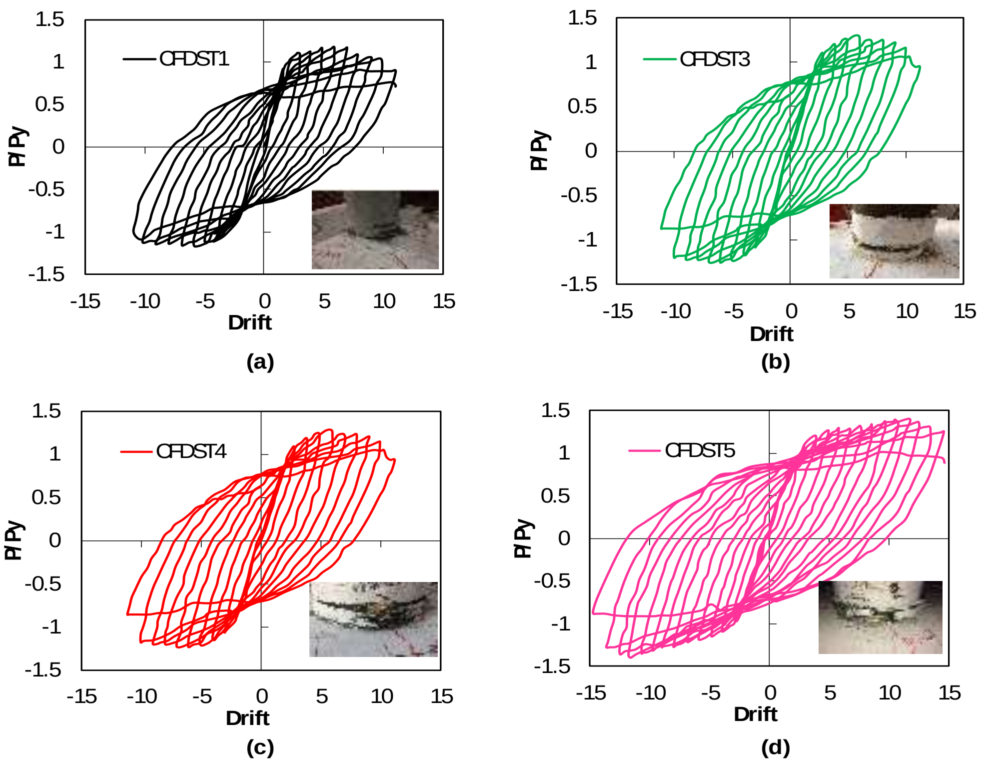


Fig. 5: Hysteretic curves and failure modes of tested specimens

For specimen CFDST1, cracks initiated at the column-footing interface at approximately 0.75% drift ratio and local within the corrugated pipe with increasing lateral deformation until 3% drift ratio. After that, the cracking around the corrugated pipe occurred at the third cycle of 4% drift ratio. The outer steel tube of the column started to buckle at the lateral drift ratio of 7% and was fractured at a drift ratio of 11%, as shown in Fig. 5(a).For specimen CFDST2, the buckling of the steel tube was successfully prohibited, and the outer steel tube was fractured at the column bottom due to too large inelastic deformation at the drift ratio of 13%. The grout in the corrugated pipe suffered more serious damage due to the slight increase of the loading capacity of the specimen, as shown in Fig. 5(b). For specimens CFDST3, CFDST4, and CFDST5, the cracking of the footing became slighter. The fibers of CFRP sheets were gradually fractured due to the serious buckling of the steel tubes at the large drift ratios larger than 5%. The use of sponge between the CFRP sheets and steel tubes slightly delayed the fracture of the CFRP fibers but had no significant effect on the deformation capacity of the specimens. Both the specimens CFDST3 and CFDST4 failed at a drift ratio of 11% due to the fracture of steel tubes. In specimen CFDST5, the un-bond region left in the footing significant delayed the fracture of both CFRP sheets and the outer steel tube. The initial yielding of the outer steel tube occurred in the un-bond region; however, with the increase of the applied displacements, the outer steel tube gradually buckled at the region above the footing and it was finally torn off during a drift ratio of 15%. The final failure modes of the tested specimens are presented in Fig. 5(c, d, and e).From the previous results, it can be concluded that the further increase of the embedment depth larger than 1.0D had no significant effect on the seismic behavior, the use of steel rings can effectively prevent the local elastic-plastic buckling of outer steel tubes, and thus can effectively improve the deformation capacity, ductility, and energy-dissipation capacity of the specimens, the use of CFRP wraps had no significant effects on the load-deformation responses of the tested specimens due to the gradual fracture of the fibers but resulted in fatter and more stable moment-curvature relationship curves at the plastic hinges, as shown in Fig. 5(d). It was also evident when using CFRP from below the footing to above the column that it had a great effect in improving behavior and access to high drift 15% (this drift is unreality demand).

PREVIOUS WORK ON NON-EMULATIVE CONNECTIONS

Hybrid rocking connection

Energy dissipation
PT bar
CFST
Steel slit damper
Steel
Concrete

(a) **(b)** **(c)**

Fig. 6: Tested specimen for; (a) [12]; **(b)** [9]; **(c) and** [21]

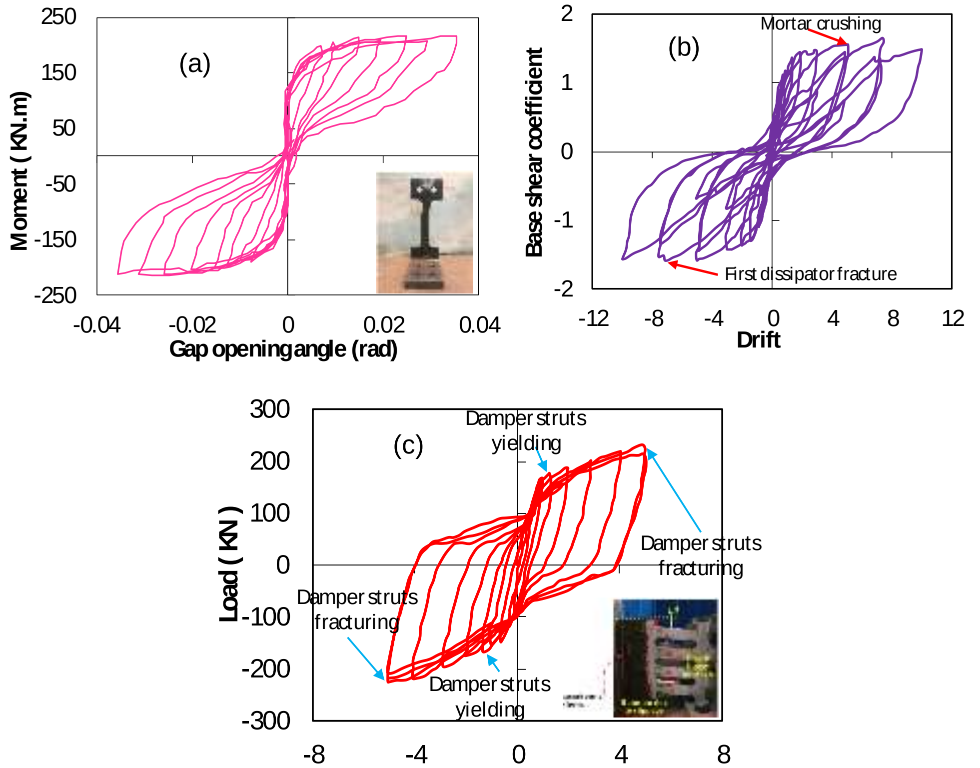


Fig. 7: Hysteretic curves and failure modes of tested specimen; (a) [12]; **(b)** [9]; **(c) and** [21]

In [12], this study presents new type of self-centering CFST column base connections constructed by CFST columns, PT strands to achieve self-centering behavior, and sandwiched energy dissipaters to absorb seismic energy, as shown in Fig. 6(a). For the tested specimen, when loading to 0.75% drift, the connection began to rotate at the bottom edge of the column, and a gap opening was observed. Under 4% drift, the buckling restrained steel (BRS) plates yielded while the column and strands remained elastic, as shown in Fig. 7(a).

Table 2: General discussion

Type connection	Connection		During earthquake evaluation: Stiffness	During earthquake evaluation: Drift capacity	During earthquake evaluation: Stability	Post-earthquake evaluation: Failure mode	Post-earthquake evaluation: Residual deformation	Post-earthquake evaluation: Repair-ability
Emulative	Exposed		Degradation in stiffness in case of square after 6% drift	up to 5%	Stable	(1) Inelastic elongation of the anchor bolts	Limited residual deformation	Easy to repair bolts
						(2) Cracks near the interface of base plate and foundation concrete		Difficult to repair or replace main components
			small energy dissipation					Take long time to repair
	Embedded	Grouted	Degradation in stiffeners after 6% and different between specimens with a difference of stiffeners and embedded length	Up to 8%	Stable	(1) Partial pullout from foundation pullout	Large residual deformation	Difficult to repair or replace
			Large energy dissipation in case of increase embedded length			(2) Ductile tearing		Take long time to repair
						(3) Ductile tearing with cracking		
	Embedded	Socket	Degradation in stiffness for original specimen and this degradation decreases in case of confined specimens	Up to 11% for original specimen	Stable	Cracking around the corrugated pipe	Large residual deformation	Difficult to repair or replace
				Up to 15% for the confined specimens		Steel tube fracture		Take long time to repair
			Large energy dissipation			Damage in grout in the corrugated pipe		
Non-Emulative	Hybrid rocking		No degradation in stiffness	Up to 8% and depend on the energy dissipation type	Stable	Fracture of damper	Very limited residual deformation	Easy to repair and replace
			Large energy dissipation			Mortar crushing		Fast repair

[9]describes an innovative bridge column technology for application in seismic regions. The proposed technology combines a precast post-tensioned composite steel-concrete hollow-core column, with supplemental energy dissipation, in a way to minimize post-earthquake residual lateral displacements, as shown in Fig. 6(b). The mortar bed started to crush during the 5% drift ratio cycles, with significant loss of stiffness and self-centering ability. This was caused by a significant loss of post-tensioning force upon mortar crushing, as shown in Fig. 7(b). External dissipators started bending between the buckling restrained central portion and the end connections during the 3% drift ratio cycles, due to the rotation imposed by the rocking body motion. The northwest dissipator fractured during the first negative cycle to 7.5% drift ratio, nearly at peak displacement. Two other dissipators fractured on the south side during subsequent cycles. Due to failure of three out of six dissipators, the test was interrupted after the first cycle to 10% drift ratio. In [21], Six large-scale experimental tests were conducted to investigate the behavior of an innovative resilient rocking (IRR) column which consists of a steel column that rocks and is connected at its base with replaceable steel slit dampers and subjected to quasi-static cyclic loading, as shown in Fig. 6(c). This system can achieve rapid restoration of structural functionality after earthquakes, which has recently emerged as an important issue facing the structural engineering profession. The loading process and behavior of specimen S16-5.5-0.1 (reference specimen) are described. The first yielding of damper struts occurred at 1.3% drift and the steel column was still in the elastic range. At 2.9% drift, the rocking could be observed at the column base. The rocking of the column base and its corresponding uplift on the tension side initiated the shear deformation of adjacent steel slit dampers, which caused subsequent plastic deformation in the damper struts. Several of the strain gauges on the damper struts detected steel yielding, indicating significant formation of plastic deformations in the steel slit dampers. This behavior is shown in Fig. 7(c). The damaged components (Steel slit dampers) can be replaced after an earthquake without difficulty while the other components of the IRR column remained elastic throughout the loading process.

Conclusion

This study presents a general evaluation of the seismic response of CFST bridge columns. To the best knowledge of the authors, the presented results included the available studies that examined different types of connection between the column and the footing. The studied connections are classified, with respect to the structural response of the traditional monolithic connection, to emulative connections and non-emulative connections. According to Table 2, the following conclusions can be drawn:

1- For the emulative connection, the socket connection with CFRP-confinement to both the embedded part of the column in the footing and the plastic hinge zone of the column (Fig. 5(d)) can ensure high drifty capacity with hardening performance in the inelastic stage and a fatty hysteric response; however, it is recommended to be used in bridges located in moderate earthquake regions because it cannot ensure the required recoverability at high drift demands.
2- For the non-emulative connection, among the studied cases, the hybrid rocking connection with BRS plates can be adopted as seismic resisting components in high earthquake zones due to the elastic response of its main components, very limited residual inclination, and fast and easy replacement of the damaged components of the ED system.

REFERENCES

[1] M. T. L. D. E. &. R. C. W. Stephens, "Design of CFST column-to-foundation/cap beam connections for moderate and high seismic regions," Engineering Structures, vol. 122, pp. 323-337, 2016.

[2] H. L. Y. &. Z. X. Zhu, "Rigidity estimation of embedded CFST column-to-beam connections," Engineering Structures, vol. 147, pp. 768-781, 2017.

[3] C. W. S. M. T. &. L. D. E. Roeder, "Concrete filled steel tubes for bridge pier and foundation construction," International Journal of Steel Structures, vol. 18(1), pp. 39-49, 2018.

[4] B. M. S. N. A. A. T. H. A. H. &. A.-S. Y. A. Khateeb, "Behavior of novel CFST circular column-to-foundation connections under cyclic loading," Engineering Structures, vol. 221, p. 111051, 2020.

[5] G. R. J. I. &. S. M. J. Guerrini, "Self-centering, low-damage, precast post-tensioned columns for accelerated bridge construction in seismic regions," Proceedings of the 16WCEE, 2017.

[6] D. S. R. R. J. M. &. G. N. Roke, " (2009). Design concepts for damage-free seismic-resistant self-centering steel concentrically braced frames," In Structures Congress 2009, pp. pp. 1-10, 2009.

[7] K. Kaptan, "Seismic base isolation and energy absorbing devices," 2013.

[8] J. H. M. C. H. a. L. S. Roh, "Development of a multiaction hybrid damper for passive energy dissipation," Shock and Vibration, 2018.

[9] G. R. J. I. M. M. &. V. A. Guerrini, "Seismic behavior of posttensioned self-centering precast concrete dual-shell steel columns," Journal of structural engineering, vol. 141(4), p. 04014115, 2015.

[10] F. D. C. A. &. K. T. L. Freddi, "Rocking damage-free steel column base with friction devices: design procedure and numerical evaluation," Earthquake Engineering & Structural Dynamics, vol. 46(14), pp. 2281-2300, 2017.

[11] V. C. K. T. L. &. V. G. Kamperidis, "Self-centering steel column base with metallic energy dissipation devices," Journal of Constructional Steel Research, vol. 149, pp. 14-30, 2018.

[12] X. T. X. C. D. L. L. H. &. L. J. Wang, "Seismic behavior of self-centering concrete-filled square steel tubular (CFST) Column Base," Journal of Constructional Steel Research, vol. 156, pp. 75-85, 2019.

[13] Y. C. S. S. F. R. B. R. J. I. H. R. S. C. N. M. .. &. B. P. Kurama, "Seismic-resistant precast concrete structures: state of the art," Journal of Structural Engineering, vol. 144(4), p. 03118001, 2018.

[14] S. &. P. A. White, "Quasi-static testing of posttensioned nonemulative column-footing connections for bridge piers," Journal of Bridge Engineering, vol. 21(6), p. 04016025, 2016.

[15] Z. L. T. Q. H. W. H. &. L. Y. Wang, "Seismic performance of precast bridge columns with socket and pocket connections based on quasi-static cyclic tests: Experimental and numerical study," Journal of Bridge Engineering, vol. 24(11), p. 04019105, 2019.

[16] X. M. X. H. D. B. D. Z. Y. W. &. Z. P. Li, "SEISMIC BEHAVIOR OF PRECAST CONCRETE FILLED DUAL STEEL TUBE COLUMNS IN SOCKET FOUNDATIONS," ADVANCED STEEL CONSTRUCTION, vol. 15(3), pp. 215-224, 2019.

[17] A. E. B. E. M. S. T. M. &. S. A. D. Ebrahimpour, "Seismic performance of columns with grouted couplers in Idaho accelerated bridge construction applications," Idaho. Transportation Dept, pp. 16-246, 2016.

[18] A. S. M. J. R. J. I. &. F. R. B. Belleri, "Dynamic behavior of rocking and hybrid cantilever walls in a precast concrete building," American Concrete Institute, 2014.

[19] H. L. &. L. H. W. Hsu, "Improving seismic performance of concrete-filled tube to base connections," Journal of Constructional Steel Research, vol. 62(12), pp. 1333-1340, 2006.

[20] B. &. U. C. Jothimani, "Experimental Investigation on Concrete Filled Steel Tubular Column to foundation Connections subjected to combined Axial and Lateral Cyclic Loading," Latin American Journal of Solids and Structures, vol. 16(6), 2019.

[21] Y. G. Z. L. X. C. R. &. S. B. Liu, "An innovative resilient rocking column with replaceable steel slit dampers: Experimental program on seismic performance," Engineering Structures, vol. 183, pp. 830-840, 2019.