Which capacity models for shear-critical reinforced concrete hollow piers?

Paolino Cassese[a], Antonio Bonati[b], Maria Teresa De Risi[c], Gerardo Mario Verderame[c] and Edoardo Cosenza[c]

[a] *Construction Technologies Institute, National Research Council, Via Claudio 21, Naples (80125), Italy*

[b] *Construction Technologies Institute, National Research Council, Viale Lombardia 49, San Giuliano Milanese (20098), Italy*

[c] *Department of Structures for Engineering and Architecture, University of Naples Federico II, Via Claudio 21, Naples (80125), Italy*

Abstract

The seismic assessment of existing bridges is nowadays a crucial aspect for resilient communities. Existing bridge piers are characterised by structural deficiencies often resulting in shear failures. Seismic assessment for shear-critical reinforced concrete hollow piers, widely used in bridges, is still an open issue. Currently, practitioners adopt formulations not validated for hollow piers notwithstanding their structural peculiarities. Herein, first, a review of existing shear capacity models from literature and codes is presented. Then, proper experimental data, related to both rectangular and circular hollow members, are collected and compared with the considered models to assess their reliability. Lastly, shear capacity models specific for the investigated structural typology are proposed. They showed very good agreement with the experimental data and can be simply used by practitioners for seismic assessment or design purposes.

1 Introduction and research significance

Among civil structures, bridges are crucial for economic and social reasons. The seismic performance assessment of existing bridges is a key issue nowadays in countries such as Italy, where a large part of viaducts has been realized before seismic design codes. Existing reinforced concrete (RC) bridges generally are low-standard structures and they often exhibited brittle failures due to earthquakes, such as shear failure of bridge piers, causing disastrous collapses [1]. During last decades, a great attention has been focused on the development of reliable analytical models for the assessment of the shear capacity of RC members under seismic action. Several studies from the literature addressed this issue for RC ordinary building members, defining reliable proposals for the assessment of the shear strength adopted in several seismic codes [2] – [7].

RC piers with hollow cross-section (HCSP hereinafter) are a widespread structural solution in bridge engineering due to several advantages if compared to solid cross-section members (reduction of seismic mass; low concrete cracking during hydration; lower loads to foundations and reinforcement ratios). Furthermore, circular HSCP are generally preferred due to the uniform response whatever the loading direction [8]–[9]. Despite their widespread use, current main codes do not address specific attention to the shear strength of HCSP [10], even though the shear-resisting mechanisms typical of these elements are more similar to those related to tube sections, mainly depending on the thickness of webs [11]. More in details, some of the specific issues concern, among others, the evaluation of the effective resisting shear concrete area, the effectiveness of the transverse reinforcement, the higher strength degradation due to the limited concrete thickness, the effectiveness of the inclined concrete compressive strut mechanism. Moreover, relatively few experimental and analytical studies exist in the literature about the seismic performance of HCSP, both rectangular and circular, and only very few of these investigated the assessment of shear failure [12]–[21].

1.1 Research significance

Within the framework described above, bridge engineers involved in seismic assessment of viaducts characterized by hollow core RC piers are somehow forced to adopt code-based shear-strength formulations calibrated on the experimental testing of members with solid cross-section, rectangular or circular [2]–[5], whose effectiveness for HCSP has to be, at least, proved and, if necessary, improved. This work aims providing a comprehensive perspective on the still open issue “which shear strength model for HCSP?”, trying to present a synthetic and useful guide, in which main references, discussion and conclusions are provided to researchers and practitioners. Therefore, the accuracy of existing shear strength formulations from codes and literature is assessed based on the collection and analysis of experimental data from past experimental campaigns on shear-critical HCSP. Then, shear capacity models specific for HCSP are proposed and briefly described, depending on the section shape (rectangular or circular). These proposals allow minimizing the prediction error and can be simply applied for seismic assessment or design purposes.

2 Shear capacity models

Some of the main shear-strength formulations from codes and literature are briefly described below. Such revision of the state-of-the-art is addressed separately for rectangular and circular HCSP, highlighting the main peculiarities and limits of the considered capacity models. Their prediction capability will be investigated in section 4 by applying them to the specimens collected in section 3.

2.1 Rectangular HCSP

According to the model by Kowalsky and Priestley [3], the shear strength can be assessed as the sum of the contributions due to concrete, transverse reinforcement and compressive strut resisting mechanism due to the axial load. This was validated on experimental results on ordinary columns with solid circular cross-section. The shear strength degradation is considered through a degradation factor (k) affecting concrete and transverse reinforcement contributions only, which decreases with increasing displacement ductility according to a linear function. Under cyclic actions, flexure-shear failure can be detected by calculating the ductility demand as the ratio between the displacement at peak load and at displacement at first yielding. Sezen and Moelhe [4] proposed an additive bi-component model, in which axial load contribution was accounted within the concrete term. This model was validated through a comparison with experimental results on ordinary building columns with rectangular cross-section. Unlike the previous model, in this case: (*i*) the shear-strength degradation factor affects all the terms; (*ii*) the ductility demand is assumed as the ratio between displacement corresponding to a drop in lateral load equal to 20% of the maximum strength and the displacement at first yielding. The shear strength can be determined as the sum of the contributions due to concrete, transverse reinforcement and axial load according to model by [5] too. This model was calibrated on the results deriving from a wide experimental database, in which also some rectangular HCSP were included. The degrade coefficient multiplies only concrete and transverse reinforcement contribution and the ductility demand is determined at a drop of 20% of the maximum strength. The model by [5] is provided by European code for the seismic assessment of existing buildings [23]. Moreover, it is recognised as the reference model for seismic assessment of bridges [22], [24].

The American guideline for seismic assessment and retrofitting of bridges FHWA-HRT-06-032 [25] suggests a formulation for the evaluation of the shear capacity based on the model by [26], in which the inclination of shear crack is not equal to 30° but determined as a function of aspect ratio and reinforcement ratios. No degradation law for the shear strength with ductility demand is provided, but only maximum (not-degraded) and minimum (degraded) values are suggested. Therefore, such a model cannot be adopted implemented for cyclic actions. The Italian technical code [27] adopts a slight modified version of the model by [26] in which a 45° angle truss model (instead of 30°) and a different strength degradation law are assumed.

2.1.1 Proposed model for rectangular HCSP

Cassese et al. [14] introduced a modification to the model by [3]. In order to apply the latter to HCSP a different definition of the effective shear area is necessary since the distribution of shear stress on tube cross-sections is substantially concentrated on the webs. Based on experimental evidence, authors proposed to assume as effective for shear strength the only confined portion of concrete webs. Therefore, the shear strength (V_R) of a rectangular HSCP can be determined as in equation (1).

$$V_R = \min\left(1.5; \max\left(1.0; 3-\frac{L_V}{H}\right)\right)\min\left(1.0; 0.5+20\rho_l\right)k\sqrt{f_c}(1.6t_w H) + \frac{A_w f_{yw}\left(d'-x\right)}{s\tan(30°)} + \frac{\left(H-x\right)}{2L_V}P \tag{1}$$

In equation (1), L_V is the shear span, H is the cross-section depth, ρ_l is the longitudinal reinforcement ratio, k is the degradation coefficient, f_c and f_y are concrete compressive strength and transverse steel yielding stress, A_w is the cross-section area of transverse reinforcement, d' is distance parallel to the applied shear between centres of peripheral hoops, s is the transverse reinforcement spacing, x is the neutral axis depth and P is the compressive axial load. The coefficient k is determined based on a ductility demand (μ) at the displacement corresponding to the maximum strength and it ranges between 0.29 and 0.05 for μ between 2 and 8. For more details about the model, see Cassese et al. [14].

2.2 Circular HCSP

Ranzo and Priestley [8] proposed the only shear strength model from literature specific for circular HCSP and accounting for cyclic degradation. The authors proposed just some modifications to the original model by [3], mainly about the effective shear area and dowel action contribution. Such modifications were calibrated based on only two cyclic tests carried out on circular HCSP. Jensen and Hoang [28] proposed a more complex procedure for the evaluation of shear strength of circular HCSP. The model was validated on the results of monotonic tests on high-strength concrete piles; therefore, it cannot be rigorously applied for seismic (cyclic) assessment. The shear strength is determined as the minimum between two distinct values depending on the level of the axial load. Turmo et al. [10] defined two factors to determine the efficiency of transverse reinforcement along longitudinal and transverse directions. Authors introduced these factors within the Eurocode 2 [29] formulation of shear strength contribution due to transverse reinforcement, which does not consider any degradation with ductility demand.

Regarding provisions from technical codes and guidelines, the formulations described in section 2.1 can be adopted also for circular HCSP, since ad hoc provisions are not available at all.

2.2.1 Proposed model for circular HCSP

A proper calibrated model for circular HCSP is proposed in [30]. Based on the capacity model by [8], the authors introduced the following modifications:

- Transverse reinforcement contribution: maximum effectiveness of transverse reinforcement is assumed both along transverse and longitudinal direction. This assumption is adopted because the shear stress flow has the same orientation of transverse reinforcement and shear load circular ties act along the same plane
- Concrete strength contribution: more coherent evaluation of effective shear area and higher concrete tensile strength are adopted, in agreement with provisions of ACI 314/14 [31]
- Axial load contribution: a proper formulation for the evaluation of the equivalent strut and an empirical efficiency factor, depending on the cross-section thickness, are proposed, since the efficiency of the strut mechanism for hollow circular members is less significant than in the case of solid core columns because it takes place along curved struts.
- Strength degradation: a specific law is derived from experimental data, assuming the degradation affects both concrete and transverse reinforcement terms.

As a result, the shear strength (V_R) of a circular HSCP can be determined as in equation (2).

$$V_R = k\left\{\min\left(1.5; \max\left(1.0; 3-\frac{L_V}{D}\right)\right)\min\left(1.0; 0.5+20\rho_l\right)0.3\sqrt{f_c}\lambda A_c + \frac{A_w}{s} f_{yw}\left(D-x-c\right)\cot\left(30°\right)\right\} + s_e \cdot \frac{(D-x^*_y)}{2L_V}P \tag{2}$$

In equation (2), λ is the effective shear-area factor, D is the external diameter, A_c is the concrete area, c is the concrete cover, s_e is the strut efficiency factor, x^*_y is the compression centre depth; all the remaining terms have been previously defined. For more details about the model, see Cassese et al. [30].

3 Collected experimental data

In order to estimate the accuracy of the prediction capacity of the shear strength models available in main codes and literature, two properly collected database of experimental test results are presented herein, representing the current state-of-the-art of existing shear-critical HCSP, rectangular and circular respectively, most widely adopted for bridges [32]. In Table 1, the main properties and experimental values of 25 experimental tests on rectangular HCSP are reported (for more details refer to [14]). 14 of the tests showed a failure in shear after flexural yielding and the remaining ones failed in shear without yielding (brittle shear failure). In particular, in addition to the reference to the corresponding experimental study, Table 1 shows for each test: the identification tag of the specimen (ID), the aspect ratio (defined as shear span to depth ratio, L_v/D), the void ratio A_v/A_s (void-to-solid cross-section area ratio), the mechanical transverse reinforcement ratio (ω_s), the maximum recorded shear force (V_{test}), the observed failure mode (FM_{exp}). About the definition of FM_{exp}, since all the collected tests exhibited a shear failure, it is assumed that: if shear failure occurred without flexural yielding, (S)-failure-mode is assumed; otherwise, flexure–shear (FS) failure mode is assumed.

Table 1 Collected experimental database: tests on shear-critical rectangular HSCP.

Reference	ID	L_v/D	A_v/A_s	ω_s (%)	V_{test} (kN)	FM_{exp}
[12]	S250	2.00	0.44	1.97	217	FS
	S500	2.00	0.44	2.92	247	FS
	S750	2.00	0.44	2.14	297	FS
	T250	3.00	0.44	4.56	217	FS
	T500A	3.00	0.44	4.65	209	FS
	T500B	3.00	0.44	4.23	226	FS
	T750	3.00	0.44	4.49	258	FS
[13]	PO1-N1	3.00	0.44	7.94	190	FS
	PO1-N2	3.11	0.44	2.96	130	FS
	PO1-N3	3.11	0.44	2.96	130	FS
	PO1-N4	3.11	0.44	2.93	170	FS
	PO1-N5	3.11	0.44	2.93	170	FS
	PO1-N6	3.11	0.44	5.87	210	FS
	PO2-N1	3.00	0.56	7.94	240	S
	PO2-N2	3.11	0.56	2.96	190	S
	PO2-N3	3.11	0.56	2.96	220	FS
	PO2-N4	3.11	0.56	2.93	190	S
	PO2-N5	3.11	0.56	2.93	200	S
	PO2-N6	3.11	0.56	5.87	250	FS
[14]	P3	1.50	0.33	4.54	278	FS
	P4	2.25	0.33	4.54	193	FS
[15]	MI1	3.60	0.36	4.04	2350	FS
	MI2	3.60	0.36	4.66	2610	FS
[16]	PI2	2.33	0.36	3.44	2650	FS
[17]	NI1-b	3.00	0.27	4.72	270	FS

Table 2 summarizes the collection of experimental tests on 13 circular HCSP with medium-low concrete strength, representative of typical existing bridge piers [32] with circular section, interested by shear failure. In particular, 8 tests showed a brittle failure (under monotonic loading) and the remaining 5 tests exhibited shear failure after flexural yielding (under cyclic loading). All terms of Table 2 have been previously defined (for more details refer to [21]). Note that, the aspect ratio (L_v/D) here is defined as the ratio between shear span and external cross-section diameter.

Table 2 Collected experimental database: tests on shear-critical circular HSCP.

Reference	ID	L_v/D	A_v/A_s	ω_s (%)	V_{test} (kN)	FM_{exp}
[18]	PA1	2.5	0.36	0.00	55	S
	PA2	2.5	0.36	0.00	91	S
	PB1	2.5	0.36	5.95	134	S
	PB2	2.5	0.36	5.71	163	S
[10]	Test 1	2.3	0.44	1.47	252	S
	Test 2	2.3	0.44	1.47	252	S
	Test 3	2.3	0.44	1.91	229	S
	Test 4	2.3	0.44	1.91	229	S
[8]	HS2	2.5	0.67	5.17	1470	FS
	HS3	2.5	0.67	5.91	1728	FS
[19]	PI2-C	2.3	0.36	3.47	2217	FS
[20]	PI2-C*	2.3	0.36	3.09	2284	FS
[21]	PC2	2.0	0.40	2.48	123	FS

4 Predicted-to-experimental shear strength comparison

The prediction capacity of the shear strength models mentioned in section 2 is evaluated in this section. To this aim, the formulations proposed by those models have been applied to all the collected specimens. Since the main target is the seismic assessment of existing bridge structures, some of the above-described models have not been considered. In fact, models that do not consider any degradation law of the shear strength with increasing ductility demand cannot be applied to specimens failed in shear after flexural yielding under cyclic loads. For this reason, for circular HCSP, the models from literature proposed in [28] and [10] have not been implemented; additionally, the technical-code model [25] has not been applied for both circular and rectangular HCSP. Fig. 1 summarizes the obtained results in terms of predicted-to-experimental shear strength ratio.In Fig. 1, for each of the considered shear capacity models, mean and Coefficient of Variation (CoV) are provided for rectangular (left) and circular (right) HCSP. Additionally, the ends of the black error bars represent the maximum and the minimum values of the predicted-to-experimental shear-strength ratio.

For rectangular HCSP (see Fig. 1, left), the models from literature [3] – [4] are characterized by the worst prediction capacity on average, probably because they were validated on experimental results related to ordinary (solid) RC building columns. The model by Kowalsky and Priestley [3] leads to a considerable dispersion in the results, with a maximum (not-conservative) error of +72% with respect to the mean value. Eurocode 8 [23] formulation gives predicted shear strengths quite far from the experimental values (mean predicted/experimental = 0.75). Such a result appears rather surprising since this formulation has been validated on a very large experimental database, in which also rectangular HCSP were introduced. The formulation suggested by the Italian technical guideline [27] for seismic assessment of bridges overestimates the shear strength of about 20% for rectangular HCSP, with a quite large CoV. The proposed model [14] shows good results in prediction shear strength, with a mean close to 1 and a quite limited dispersion (CoV = 16%).

By observing Fig. 1 (right), the only considered degrading shear-strength model from literature [8] underestimates on average the experimental capacity with a considerable dispersion (CoV = 19%), despite it was validated on the results of cyclic tests on circular HCSP. In contrast with the results on rectangular HCSP, in this case, Eurocode 8 [23] prediction is characterized by considerable overestimation (mean = 1.27) and very high dispersion (CoV = 40%), with a maximum error with respect to the mean equal to +146%. Rather surprisingly, the capacity model suggested in [27] is characterized by a very good prediction capacity on average even if the results are rather dispersed (CoV = 19%). Finally, the proposed model for circular HCSP [30], specifically calibrated for the considered structural typology, gives very promising results, with a mean error equal to 2% and a very limited CoV (8%).

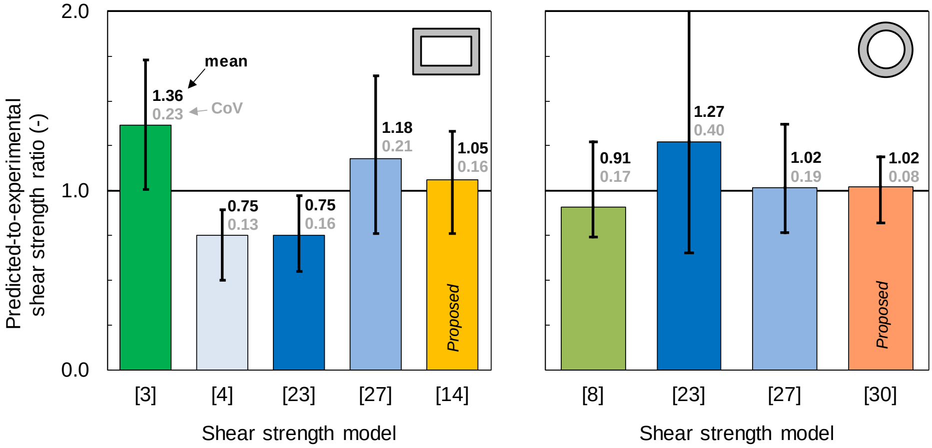

Fig. 1 Predicted-to-experimental shear strength ratio for the considered shear capacity models, in terms of mean, CoV, maximum and minimum values, for rectangular (left) and circular (right) HCSP.

5 Conclusions

The assessment of the seismic performance of existing RC bridges must not ignore an accurate prediction of piers shear strength, especially if they are in seismic prone areas [22]. Despite dramatic post-earthquake experiences and several experimental evidences [11] – [14], even nowadays no seismic code recognizes any peculiarity in the capacity assessment of HCSP, particularly in terms of shear strength, therefore no specific formulations are provided.

This research study provides to researchers and practitioners involved in seismic assessment of RC bridges characterized by HCSP: *(i)* some interesting indications about the effectiveness in prediction capacity of main models from both literature and codes, generally validated on experimental results of building columns with ordinary solid cross-sections, and (*ii*) ad-hoc proposed formulations, specifically calibrated on proper collected experimental results. To this aim, first, a brief revision of the main shear-capacity models available in literature has been presented, focusing in particular on their peculiarities and limits in the application to existing HCSP, and two specific formulations are proposed, respectively for rectangular and circular HCSP. Then, proper collected experimental databases are shown and the accuracy in prediction of the experimentally recorded shear-strength values is investigated, by applying the considered models to the collected tests.

From the predicted-versus-experimental comparison, some relevant remarks can be reported:

- For rectangular HCSP, the models from literature are characterized by the worst prediction capacity. The provisions of the European and Italian technical references for seismic assessment of bridges [23] – [27] are not accurate in the prediction., with errors of about 20-25% on average and large dispersion. Note that, the Italian document [27] provides not-conservative estimation.
- For circular HCSP, the model [8] underestimates, on average, the shear strength with a considerable dispersion. Eurocode 8 [23] shows an alarming overestimation of the real capacity and a very high dispersion (in same cases the expected shear strength is more than double the real value). The capacity model suggested in [27] is characterized by a very good prediction on average even if the results are considerably dispersed (CoV = 19%). The latter result, in author's opinion, appears quite surprising, since the formulation provided in [27] is a slight modified version of the original model [26], based on experimental results of ordinary columns.

- The proposed models [14] and [30], respectively for rectangular and circular HCSP, show the best results in the prediction of the shear strength on average, with mean values very close to the unity and a limited dispersion.

An analysis of the best formulations found in this study will be addressed in near future research efforts to make these models compatible with the probabilistic approaches generally adopted by codes, as suggested in [33]. Further studies will also address the proposal of displacement capacity predictions at shear failure for this structural typology, to be adopted in the framework of a displacement-based-design approach [34].

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