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CAPACITY MODELS FOR SHEAR-CRITICAL RC BRIDGE PIERS WITH HOLLOW CROSS-SECTION

Paolino Cassese[1], **Maria Teresa De Risi**[1], **and Gerardo Mario Verderame**[1]

[1] University of Naples Federico II
Department of Structures for Engineering and Architecture
via Claudio 21, 80125 Naples - Italy
e-mail: paolino.cassese@unina.it, mariateresa.derisi@unina.it, verderam@unina.it

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Abstract. *The assessment of seismic performance of existing bridge structures that have been constructed without appropriate details to support seismic loadings is a primary issue for seismic prone areas. Hollow section piers represent a very common structural solution for reinforced concrete (RC) bridge structures, due to the economical convenience and the higher structural efficiency with respect to solid sections. Nevertheless, a quite limited number of experimental tests are present in literature. Moreover, despite their widespread use, none of the current codes addresses specific attention to define proper capacity models for RC hollow core members, both for design and assessment. In addition, proposals from literature are generally calibrated on columns with solid cross-section or on a very small amount of experimental data.*

An experimental program of cyclic tests on four 1:4-scale models of typical RC existing bridge piers with hollow rectangular cross-section, poor structural detailing and small web thickness, has been carried out at the Department of Structures for Engineering and Architecture of the University of Naples Federico II. Different failure modes have been observed: flexure and flexure-shear failure, basically depending on the specimens' slenderness. On the basis of the major experimental results carried out by this experimental campaign, in this work, a database of similar tests from literature has been collected and presented. All tests exhibited a shear failure, with or without yielding of longitudinal bars.

Collected experimental results are compared with capacity models existing in literature or codes, in terms of strength and displacement. Their predictive capability is analyzed and commented, and alternative proposals to improve their reliability are carried out.

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1 INTRODUCTION

The design philosophy of modern seismic bridges is to pursue energy dissipation by ductile flexural hinges at the piers base [1]. Shear failure of bridge piers clearly has to be prevented to avoid disastrous collapse. Special attention has to be paid also to shear strength degradation with increasing flexural ductility demand, which may drastically reduce the energy dissipation capacity of the piers. On the other hand, past earthquakes around the world have highlighted the inadequate seismic performance of existing hollow rectangular piers, generally characterized by poor structural detailing and small web thickness [2]. The assessment of seismic performance of existing bridge structures that have been constructed without appropriate details to support seismic loadings is a primary issue for seismic prone areas.

Reinforced Concrete (RC) hollow section piers are a widespread structural solution for bridge structures, due to their economic and structural convenience. Despite their widespread use, none of the current codes addresses specialized attention to shear strength of RC hollow core members, both for design and assessment [2]. The main existing code-based shear strength models have been calibrated on experimental results of members with solid cross-section, rectangular or circular ([3]-[6]). Nevertheless, shear-resisting mechanisms typical of hollow RC piers can be very different from columns with solid cross sections, being more similar to those related to tube sections, depending mainly on webs aspect ratio. For example, in terms of degradation mechanisms, small thickness may limit the confined concrete core, crucial for shear strength.

Moreover, within the performance-based seismic assessment context, the definition of the drift value at shear failure is a paramount issue for shear critical elements. Ideally, a degrading shear strength model could be used to estimate the drift at shear failure. Actually, the main degrading shear strength models are not adequate to estimate the displacement at shear failure, because they have been calibrated in order to obtain the best result in terms of shear strength given a displacement ductility. If the inverse process was performed, a small variation in shear strength corresponds to a large change in estimated drift ratio at shear failure [7]. Different drift capacity models aiming predicting drift at shear failure have been proposed during last decades, but all these models were developed for columns with solid cross-section. The majority of these models has an empirical nature, based on experimental results of columns collected in different databases ([8]-[10]).

Therefore, a thorough investigation is necessary about the predictive capability of both force-based and displacement-based shear capacity models available in literature and codes for columns with hollow rectangular cross-section. To this aim, in the present study, a proper database of tests available in literature on shear critical specimens has been collected (including those tested at the Department of Structures for Engineering and Architecture of the University of Naples Federico II and presented in [11], [12]). Then, the attention will be focused both on the shear strength assessment and on the prediction of drift at shear failure for the columns included in the database. In both cases, a brief description of the main capacity models existing in literature will be presented and their applicability to hollow rectangular columns will be investigated. Finally, a modification of these prediction models will be proposed to improve shear strength assessment, and a new empirical model for drift at shear failure, calibrated on the database results, will be discussed.

2 EXPERIMENTAL DATABASE

Whereas for ordinary columns with solid cross section several capacity models are available in literature and codes, none specific model is suggested in literature or adopted by codes for hollow rectangular columns. The applicability of the existing models to this structural typology should be investigated in order to evaluate their effectiveness, and, eventually, some improvements or new proposals. To this aim, a proper database of tests existing in literature on shear-critical piers has been collected, including tests performed at the Department of Structures for Engineering and Architecture of the University of Naples Federico II and presented in ([11], [12]). This database includes 25 experimental tests, representing the experimental state-of-the-art about non-conforming RC columns with hollow rectangular cross section and exhibiting a shear failure (with or without flexural yielding). Note that, some other tests on hollow rectangular RC columns failing in shear are available in literature [13]. They are not included in the collected database because these specimens have high-strength concrete, characterized by different shear resistant mechanisms [14]. All the considered specimens were tested under unidirectional cyclic lateral load in single curvature. They presented a fixed-end at the bottom (cantilever scheme) and uniform longitudinal details throughout the height. Table 1 shows geometry and reinforcement details, material properties and axial load ratio of the specimens included in the database.

In order to perform a comparison between predictions by shear capacity models and experimental results, the experimental force-displacement responses are collected and analyzed. The backbone curve of the global response (namely the force-deformation curve that envelopes the experimental hysteretic loops) is defined for each test. Main characteristic values of lateral force (V) and drift ratio (DR) are identified, as explained in the following. For the specimens failing in shear after flexural yielding (FS-failure mode), the DR_y corresponding to flexural yielding is assumed equal to the experimental value, if declared. Otherwise, a section analysis is performed, assuming a stress-strain model by Mander et al. [15] for concrete and an elastic-plastic with strain hardening stress-strain relationship for steel. Starting from the yielding lateral load (V_y), the DR_y is obtained as the intersection with the experimental backbone curve (see Figure 1). The shear strength, V_{test}, is assumed as the maximum value of the lateral load reached during the test. The corresponding drift ratio is referred to as DR_{max} (Figure 1). Finally, the drift ratio at shear failure (DR_s) is defined as that corresponding to a drop in lateral load equal to 20% of V_{test}. This definition of the shear failure occurrence is consistent with that generally used in literature within the context of a displacement-based assessment ([7], [9],[16]). When the shear strength does not drop below the 80% of V_{test}, the DR_s is taken as that corresponding to the maximum-recorded displacement (as in [9]). Finally, the ductility capacity at shear failure (μ_s) is computed as the ratios between the DR demand at shear failure conditions and DR_y. Similarly, the ductility capacity at peak load (μ_{max}) is computed as the ratios between the DR demand at peak load and DR_y.

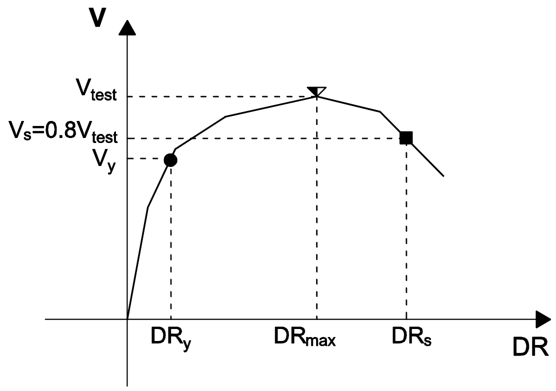

Figure 1: Definition of the characteristic points on the experimental envelope.

Experimental study	ID	Specimen	H (mm)	B (mm)	t_w (mm)	L_V (mm)	s (mm)	ρ' (%)	ρ″ (%)	f_c (MPa)	f_y (MPa)	f_{yw} (MPa)	P/A_cf_c (%)	V_{test} (kN)	FM (-)
Calvi et al. [17]	1	S250	450	450	75	900	75	1.07	0.13	35.0	550	550	0.06	217	FS
	2	S500	450	450	75	900	75	1.07	0.13	23.7	550	550	0.19	247	FS
	3	S750	450	450	75	900	75	1.07	0.13	32.3	550	550	0.21	297	FS
	4	T250	450	450	75	1350	75	1.79	0.25	30.3	550	550	0.07	217	FS
	5	T500A	450	450	75	1350	75	1.79	0.25	29.7	550	550	0.15	209	FS
	6	T500B	450	450	75	1350	75	1.79	0.25	32.7	550	550	0.14	226	FS
	7	T750	450	450	75	1350	75	1.79	0.25	30.8	550	550	0.22	258	FS
Delgado [18]	8	PO1-N1	450	450	75	1350	75	1.79	0.40	19.8	625	390	0.11	190	FS
	9	PO1-N2	450	450	75	1400	75	1.79	0.19	27.9	435	437	0.08	130	FS
	10	PO1-N3	450	450	75	1400	75	1.79	0.19	27.9	435	437	0.08	130	FS
	11	PO1-N4	450	450	75	1400	75	1.79	0.19	28.5	560	443	0.08	170	FS
	12	PO1-N5	450	450	75	1400	75	1.79	0.19	28.5	560	443	0.08	170	FS
	13	PO1-N6	450	450	75	1400	75	1.79	0.38	28.5	560	443	0.08	210	FS
	14	PO2-N1	900	450	75	1350	75	1.79	0.40	19.8	625	390	0.07	240	S
	15	PO2-N2	900	450	75	1400	75	1.79	0.19	27.9	435	437	0.05	190	S
	16	PO2-N3	900	450	75	1400	75	1.79	0.19	27.9	435	437	0.05	220	FS
	17	PO2-N4	900	450	75	1400	75	1.79	0.19	28.5	560	443	0.05	190	S
	18	PO2-N5	900	450	75	1400	75	1.79	0.19	28.5	560	443	0.05	200	S
	19	PO2-N6	900	450	75	1400	75	1.79	0.38	28.5	560	443	0.05	250	FS
Cassese et al. [11]	20	P3	400	600	100	900	120	0.88	0.12	17.0	540	655	0.05	278	FS
	21	P4	600	400	100	900	120	0.88	0.12	17.0	540	655	0.05	193	FS
Yeh, Mo and Yang [19]	22	MI1	1500	1500	300	5400	150	1.29	0.28	33.6	476	480	0.09	2350	FS
	23	MI2	1500	1500	300	5400	150	1.29	0.28	29.1	476	480	0.21	2610	FS
Yeh et al. [20]	24	PI2	1500	1500	300	3500	200	1.72	0.26	32.0	418	420	0.08	2650	FS
Mo et al. [21]	25	NI1-b	500	500	120	1500	50	1.88	0.24	20.2	476	405	0.14	270	FS

Table 1: Main properties of experimental tests of the collected database.

As above discussed, all the collected tests exhibited a shear failure, with or without flexural yielding. However, the failure mode (FM) definition (namely, S or FS mode) is not coherent among the various Authors from literature. Some of them defined the failure mode based on phenomenological aspects (analysis of the damage state evolution) and capacity model predictions; some others had adequate instrumentation (strain gauges on longitudinal bars located at the critical section) to check the effective achievement of yielding in longitudinal bars. In order to adopt a uniform failure mode definition, an unequivocal assumption is adopted in this study, based on the above-described values of lateral load (V_y) and drift ratio (DR_y) at first yielding. Starting from the evidence that all the collected tests exhibited a shear failure, when the peak load (V_{test}) is lower than load at yielding (V_y), a shear (S) failure mode is assumed to occur; otherwise, flexure-shear (FS) failure mode is assumed. Table 1 summarizes shear strength (V_{test}) and failure mode for all the tests of the database.

3 SHEAR CAPACITY MODELS

As explained in Section 1, the main shear strength models have been calibrated on experimental results of members with solid cross-section, nevertheless shear-resisting mechanisms of hollow RC columns may be significantly different, mainly depending on webs aspect ratio. A thorough investigation about the predictive capability of shear strength models available in literature is necessary when used for columns with hollow rectangular cross-section. In Section 3.1, first, a brief description of the considered capacity models is presented. Then, their effectiveness for hollow rectangular columns is investigated. Finally, a slight modification to predictive equations is proposed to improve shear strength assessment.

Several models have been also developed to represent the degradation of shear strength with increasing inelastic deformation demand. Although these models are useful for estimating the column capacity for conventional force-based design and assessment, they are not adequate to estimate the displacement at shear failure. Therefore, some displacement-based capacity models have been developed and proposed during the last decades. All these models have empirical basis, since they have been calibrated in order to reproduce the best fitting of experimental drift at shear failure of tests on building columns with solid cross-section. No similar formulations are available in literature for hollow rectangular columns. In Section 3.2, some of the existing models for displacement capacity at shear failure are first described and applied to the collected database of shear-critical piers to investigate their effectiveness for hollow rectangular columns. Finally, starting from the analysis of the database, a new empirical model for drift capacity at shear failure, properly calibrated for hollow rectangular sections, is carried out and presented.

Note that a comprehensive list of symbols adopted in the following and their meaning is reported at the end of this paper.

3.1 Shear strength

The main models available in literature for the shear strength assessment are considered herein, to carry out a comparison with the experimental values of shear strength reported in Table 1.

According to Aschheim and Moehle [3], the shear strength is the sum of strength contributions due to transverse reinforcement, V_w, and concrete, V_c, as reported in Eq. (1). The shear strength degradation influences only the concrete contribution, V_c, through a degradation factor decreasing with increasing displacement ductility (μ), varying between 1 and 0 for ductility demand between 1 and 4. The ductility demand at shear failure is assumed here as μ_{max} consistently with the model assumption.

$$V_R = 0.3\left(\frac{4-\mu}{3}+\frac{P}{13.8A_c}\right)\sqrt{f_c}\,(0.8A_c)+\frac{A''f_{yw}d}{s\tan(30°)} \tag{1}$$

Kowalsky and Priestley [4] define the shear strength as the sum of the contributions due to concrete, V_c, transverse reinforcement, V_w, and arch mechanism associated with axial load, V_p, as reported in Eq. (2). The shear strength degradation influences only the contribution due to concrete, through a degradation factor (k) that decreases with increasing displacement ductility, varying between 0.29 and 0.05 for ductility demand between 2 and 8. The ductility demand is assumed here as μ_{max} consistently with the model assumption.

$$\begin{aligned} V_R = {} & \min\left(1.5;\max\left(1.0;3-\frac{L_V}{H}\right)\right)\min\left(1.0;0.5+20\rho'\right)k\sqrt{f_c}\,(0.8A_c)+ \\ & +\frac{A''f_{yw}\left(d'-c\right)}{s\tan(30°)}+\frac{\left(H-c\right)}{2L_V}P \end{aligned} \tag{2}$$

Sezen and Moehle [5] define the shear strength as the sum of strength contributions due to transverse reinforcement, V_w, and concrete, V_c, as reported in Eq. (3). The degradation factor multiplies here contributions due to both transverse reinforcement and concrete, varying between 1 and 0.7 for μ ranging between 2 and 6. The ductility demand is assumed here as μ_s consistently with the model assumption.

$$V_R = k\cdot\left[\left(\frac{0.5\sqrt{f_c}}{L_V/d}\sqrt{1+\frac{P}{0.5\sqrt{f_c}A_c}}\right)\left(0.8A_c\right)+\frac{A''f_{yw}d}{s}\right] \tag{3}$$

Biskinis et al. [6] compute the shear strength according to a regression additive model accounting for three contributions: the classical 45-degrees truss model, V_w, the concrete contribution, V_c, and the axial load contribution, V_P, as expressed by Eq. (4). The degradation coefficient *k* multiplies both the concrete and the transverse steel contributions. It varies linearly between 1 and 0.75 for μ between 1 and 6. Consistently with the model assumptions, the ductility demand at shear failure is assumed as μ_s.

$$\begin{aligned} V_R = {} & k\left[0.16\max\left(0.5;100\rho'\right)\left(1-0.16\min\left(5;\frac{L_V}{H}\right)\right)\sqrt{f_c}b_wd+\frac{A''}{s}\left(d-d_0\right)f_{yw}\right]+ \\ & +\frac{H-c}{2L_V}\min\left(P;0.55A_cf_c\right) \end{aligned} \tag{4}$$

The selected shear strength models are applied to all the columns of the database, using the formulations described above. The results of the comparison between predicted and experimental shear strength values are summarized in Table 2, in terms of mean and coefficient of variation (COV) of the predicted-to-experimental shear strength ratio (V_{pred}/V_{test}), evaluated for the whole database and for each shear strength model.

V_{pred}/V_{test}	Aschheim and Moehle [3]	Kowalsky and Priestley [4]	Sezen and Moehle [5]	Biskinis et al. [6]
Mean	1.30	1.36	0.79	0.73
COV	0.27	0.15	0.11	0.17

Table 2: Comparison between experimental and predicted shear strength.

Model by Biskinis et al. [6] is a regression model calibrated on a very large experimental database, but only very few tests belonging to this database deal with hollow rectangular piers. It is adopted by the Eurocode 8 part 3 [22] for existing buildings, and it is suggested by some literature studies for the shear capacity assessment of existing bridge piers ([23], [24]). Nevertheless, this model considerably underestimates (-27%) shear strength for the collected database.

Model by Sezen and Moehle [5] (adopted by ASCE-SEI/41 [25]) is based on experimental data from a database including only RC columns with solid rectangular cross section, inadequate and poorly detailed transverse reinforcement, failing in shear during the inelastic response. Nevertheless, this model provides a significant underestimation (-21%) of the shear strength.

Model by Aschheim and Moehle [3] is calibrated on a database of 51 scale-models of bridge columns with solid square, rectangular and circular cross section, failing in shear after flexural yielding. Finally, model by Kowalsky and Priestley [4] was developed based on 18 solid circular columns that failed in shear after yielding and 20 solid circular columns failing in shear before yielding. It is adopted by the U.S. Federal Highway Administration [26] provisions document for seismic retrofitting of bridges. The latter two models are characterized by a significant overestimation, on average, of shear strength for the collected database.

Starting from the collected experimental tests on hollow rectangular RC columns, it seems reasonable to discuss about a simple improvement of existing shear strength models. In particular, the model by Kowalsky and Priestley [4] adopts a concrete contribution to shear strength characterized by an effective shear area of $0.8A_c$. In order to more suitably apply this model to hollow rectangular columns, it seems reasonable to modify this term. In fact, it is well known that shear stress distribution is substantially concentrated on the webs, whereas the flanges are essentially involved in the flexural response. Therefore, for hollow rectangular sections subjected to cyclic shear, it can be assumed that only the confined concrete of the webs gives a contribution to the shear strength of the column. In particular, the effective shear area is assumed herein as twice the product of the web thickness (t_w) and the effective depth (d), the latter equal to $0.8 \cdot H$ (see Figure 2).

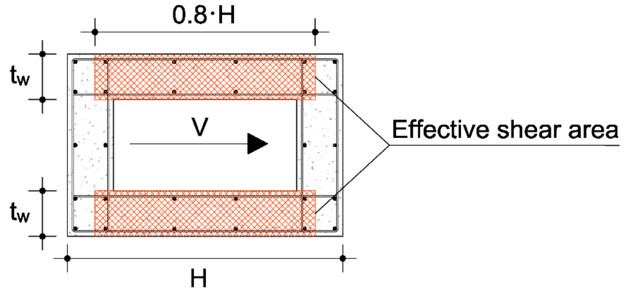

Figure 2: Proposed effective shear area for hollow rectangular columns.

By applying this modification to Eq. (2), the following equation is obtained:

$$V_R = \min\left(1.5; \max\left(1.0; 3-\frac{L_V}{H}\right)\right)\min\left(1.0; 0.5+20\rho''\right)k\sqrt{f_c}(1.6t_w H) + \frac{A'' f_{yw}\left(d'-c\right)}{s\tan(30°)} + \frac{\left(H-c\right)}{2L_V}P \quad (5)$$

Finally, Figure 3 shows the predicted-to-measured shear strength ratio (V_{pred}/V_{test}) for all the columns of the database, evaluated by applying the proposed modifications shown in Eq. (5). It can be noted that the proposed modifications lead to a substantial improvement in the predictive capability of the experimental shear strength: the mean value of V_{pred}/V_{test} is equal to 1.05, with a quite reduced COV, equal to 0.16.

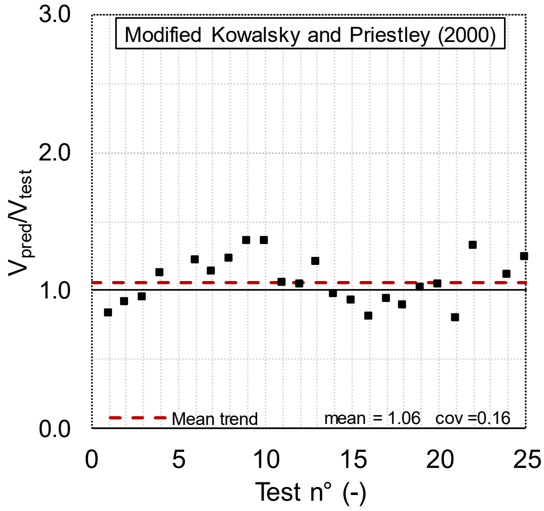

Figure 3: Predicted-to-measured shear strength ratio for the proposed modified version of the model by Kowalsky and Priestley [4].

3.2 Drift at shear failure

The main models available in literature for the assessment of drift at shear failure of existing RC columns with solid cross-section are considered herein, to evaluate their effectiveness when applied to hollow rectangular columns through a comparison with the experimental values of drift at shear failure reported in the collected database.

First, Aslani and Miranda [8], on the basis of 92 cyclic tests of non-ductile RC columns with solid cross-section, defined the drift ratio capacity at shear failure (DR_s) depending on the axial load ratio and the transverse reinforcement ratio, as reported in Eq. (6):

$$DR_s = \frac{1}{0.26\dfrac{P}{A_c f_c \rho''}} \geq \frac{1}{100} \tag{6}$$

Later, Elwood [9] introduced an empirical model based on the analysis of an experimental database of 50 tests collected by Sezen [7] and characterized by shear failure after flexural yielding. According to this model the drift at shear failure depends on axial load ratio, transverse reinforcement ratio and maximum nominal shear stress (v), the latter assumed as the ratio between the maximum experimental shear value and the cross-sectional area (taken equal to $B \cdot d$ for solid sections). The drift-capacity is expressed by Eq. (7):

$$DR_s = \frac{3}{100} + 4\rho'' - \frac{1}{40}\frac{v}{\sqrt{f_c}} - \frac{1}{40}\frac{P}{A_c f_c} \geq \frac{1}{100} \tag{7}$$

Zhu et al. [10] developed a drift-capacity model for columns representative of typical nonconforming RC buildings. This model can be applied only to columns classified as "shear-dominated" columns on the basis of a flow-chart procedure depending on three parameters, namely, aspect ratio, transverse reinforcement ratio, and plastic shear demand to shear strength ratio (the latter calculated according to the shear strength model proposed by Sezen and Moehle [5]). The drift-capacity at shear failure, if any, depends on the transverse reinforcement spacing, transverse reinforcement ratio, aspect ratio and axial load ratio, as shown in Eq. (8):

$$DR_s = 2.02\rho'' - 0.025\frac{s}{d} + 0.013\frac{L_V}{d} - 0.031\frac{P}{A_c f_c} \tag{8}$$

In the formulations reported above, transverse reinforcement ratio (ρ'') and maximum nominal shear stress (ν) always depend on the assumed cross-sectional width (b_w). Therefore, in order to investigate about the applicability of such drift capacity models to hollow rectangular columns, the width (b_w) is assumed herein as twice the thickness (t_w) of the webs (consistently with concepts reported in Section 3.1). The results of these comparisons in terms of drift at shear failure are summarized in Table 3, which shows the mean and the coefficient of variation of the predicted-to-experimental ratio ($DR_{s,pred}/ DR_{s,exp}$), evaluated on the basis of the overall database, for each of the considered models. Note that only the model by Aslani and Miranda [8] can be applied to all the collected tests. The remaining two models are characterized by some cases of inapplicability. In particular, the model by Elwood [9] should be applied only for columns failing in shear after flexural yielding; whereas the model by Zhu et al. [10] has been applied herein only for columns classified as "shear dominated", according to the flow-chart procedure mentioned before.

As a result, the model by Elwood [9] underestimates the experimental drift at shear failure on average (-17%). Moreover, in a rigorous approach, this model cannot be applied to four columns, since they are characterized by brittle shear failure (namely, occurring before yielding of longitudinal reinforcement). Vice-versa, the model by Zhu et al. [10] considerably overestimates the experimental drift at shear failure and it has been not applied to five specimens, classified as "flexural dominated" according to [10]. Finally, about the model by Aslani and Miranda [8], a considerable overestimation is observed (+28%) with a quite high dispersion in prediction (31%).

$DR_{s,pred}/ DR_{s,exp}$	Aslani and Miranda [8]	Elwood [9]	Zhu et al. [10]
Mean	1.28	0.83	2.02
COV	0.31	0.34	0.28

Table 3: Comparison between experimental and predicted drift at shear failure (DR_s).

The comparison shows that none of the considered models is able to predict the experimental values of drift at shear failure with adequate accuracy. Moreover, all the models show high dispersion in the prediction (equal to about 30%). Therefore, a proper drift capacity model to predict shear failure of hollow rectangular columns should be formulated. The main goal is to reduce the mean error of existing models, providing a simple relationship that identifies the critical parameters influencing the DR_s for shear-critical hollow rectangular RC columns. First of all, the relation between the experimental DR_s and main structural key parameters, potentially affecting the shear response of the database columns, is analyzed. In particular, the analyzed parameters are: geometrical transverse reinforcement ratio (ρ''); mechanical transverse reinforcement ratio ($\omega''= \rho'' f_{yw}/f_c$); geometrical longitudinal reinforcement ratio (ρ'); aspect ratio (L_V/H); axial load ratio ($P/A_c f_c$); maximum nominal shear stress ($\nu = V_{test}/2t_w d$). From this analysis, it has been observed that columns with higher transverse reinforcement ratios, ρ'', tend to reach larger drifts at shear failure compared with columns characterized by lower transverse reinforcement ratios, as expected. In contrast, there is no clear relationship between the mechanical transverse reinforcement ratio, ω″, and the drift at shear failure. The effect of the dowel action of longitudinal reinforcement does not influence the DR_s. On the other hand, such a result may be related to the limited variability of longitudinal reinforcement ratio, ρ′, for the columns in the database. Almost a clear relationship exists between the aspect ratio (L_V/H) and DR_s: the higher the aspect ratio, the lower the drift at shear failure. A very slight tendency of DR_s to reduce with increasing axial load ratio (P/A_cf_c) is also identified. A clear linear decreasing trend of DR_s exists with increasing maximum nominal shear stress (ν). Therefore, the key parameters clearly influencing the drift at shear failure are transverse reinforcement ratio (ρ″), aspect ratio (L_V/H) and maximum nominal shear stress (ν). Based on this observation, a simple empirical expression is proposed to estimate the drift ratio at shear failure, as shown in Eq. 9:

$$DR_s = \frac{1.6}{100} + 1.3\rho" + \frac{0.6}{100}\frac{L_V}{H} - \frac{1.9}{100}\frac{\nu}{\sqrt{f_c}} \tag{9}$$

Transverse reinforcement ratio (ρ″) and maximum nominal shear stress (ν) are evaluated according to Eq.s (10) and (11):

$$\rho" = \frac{A"}{2t_w s}\ (\text{-}) \tag{10}$$

$$\nu = \frac{V_{test}}{2t_w d}\ (\text{MPa}^{0.5}) \tag{11}$$

The coefficients in Eq. (9) were chosen based on a least-squares fit of the data. Figure 4 compares the drift at shear failure (DR_s) predicted by Eq. (9) with the corresponding experimental value from the experimental database.

The accuracy in the prediction is improved with respect to the models considered above in this Section. In fact, the mean value of the calculated-to-experimental ratio is equal to 1.02, and the coefficient of variation is 0.25. Note that the calculated drift ratio has been determined using the maximum experimental shear stress to calculated the parameter ν. In order to use Eq. (9) for the seismic assessment of existing RC columns with hollow rectangular section, it is necessary to define a value of shear strength for the calculation of the maximum nominal shear stress. To this aim, it seems reasonable, first, to classify the column by using the model defined in Eq. (5). Assumed as V_p the plastic shear capacity (namely, the flexural capacity M_p divided by the shear span L_V), if V_p is lower than the minimum degraded shear strength ($V_{R,min}$), no shear failure should be expected, and therefore there is no need to compute DR_s. Otherwise, the DR_s may be computed by assuming V_{test} equal the maximum non-degraded shear strength ($V_{R,max}$), if the latter is lower than Vy. In all other cases, it can be assumed Vtest=Vp

In conclusion, note that the application of the proposed empirical drift capacity model should be limited to hollow rectangular columns similar to those included in the database, namely, whose geometrical and mechanical properties are included into the variability ranges of these parameters related to tests belonging to the analyzed database.

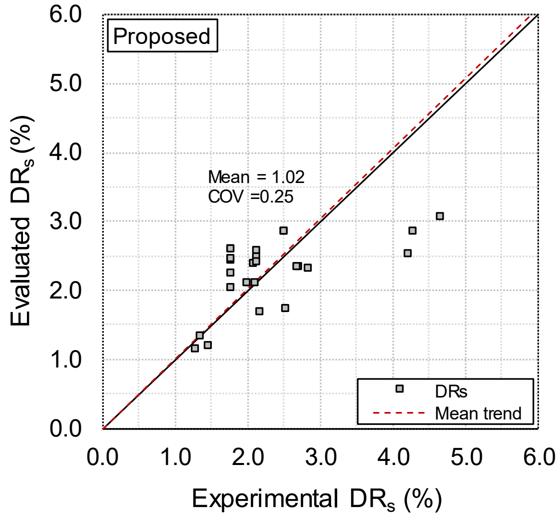

Figure 4: Comparison of predicted drift ratio at shear failure with related experimental results.

4 CONCLUSIONS AND FUTURE DEVELOPMENTS

Reinforced Concrete piers with hollow cross section are a widespread structural solution for bridge structures, due to their economic and structural convenience. Despite their widespread use also for RC bridges non-conforming to the more updated seismic codes, the main existing code-based shear strength models have been calibrated on experimental results of members with solid cross-section, whose shear resistant mechanism can be particularly different with respect to hollow sections. Therefore, in this work, a database of tests on shear critical piers with hollow rectangular cross-section from literature has been collected. All these tests exhibited a shear failure, with or without yielding of longitudinal bars. The collected experimental results were then compared with capacity models existing in literature or codes, in terms of strength and displacement.

About shear strength, it was observed that the main models from literature and codes provided an error that is around 30% (in under- or over-estimation) with respect to experimental results. Therefore, a slightly modification of the shear strength model by Kowalsky and Priestley [4] has been carried out, so leading to a mean error in prediction that is almost close to the unity. Furthermore, a new empirical formulation has been proposed to predict the displacement capacity at shear failure, specifically calibrated on experimental data related to shear critical piers with hollow rectangular cross section. Also in this case, the accuracy in the prediction is improved with respect to the existing models – which are generally calibrated for columns with solid cross section – leading to a mean error equal to 2%.

More efforts should be performed in the future to enlarge the collected database with new experimental tests on this pier typology. On the other end, further development of this work should be addressed to the definition of a comprehensive numerical model for shear-critical piers. Certainly, more data are required to analyze also the behaviour after shear failure including the eventuality of a subsequent axial load failure.

List of symbols

L_V	=	shear span (height from the base to the loading point)
B	=	width (external dimension of the cross-section orthogonal to loading direction)
H	=	depth (external dimension of the cross-section along loading direction)
t_w	=	thickness of cross-sectional webs
ρ'	=	longitudinal reinforcement ratio
ρ''	=	transverse reinforcement ratio
f_c	=	concrete compressive strength
f_y	=	yielding stress of longitudinal steel reinforcement
f_{yw}	=	yielding stress of transverse steel reinforcement
A_c	=	cross-section area (gross area without void)
P	=	axial load
A''	=	cross-section area of transverse reinforcement
d	=	effective cross-section depth (assumed as $0.8 \cdot H$)
d'	=	distance parallel to the applied shear between centers of peripheral hoops
d_0	=	depth of the compression reinforcement layer
b_w	=	width of cross-sectional webs
s	=	spacing of the transverse reinforcement
c	=	neutral axis depth

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