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EXPERIMENTAL RESEARCH OF SHEAR STRENGTH MODELS FOR REINFORCED CONCRETE MEMBERS

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Abstract. *The question of systematically calculating the shear strength of reinforced concrete (R.C.) elements in flexure-shear reversals with or without axial load has been central to the development of performance based design. The multitude of models in the international literature, some of which have been adopted by the Seismic Codes of Assessment or Design (such as ACI 318−2011, FEMA 273 and 356, ASCE/SEI 41–07, EC8−III (2005)) hints to the limited extent of the current level of understanding of this problem; the success of these models in reproducing strength values is very good when flexural failures dominate the response, but their performance, as measured by the amount of scatter between experiment and analysis, increasingly deteriorates when used to estimate shear capacity in shear driven modes of failure.*

The most important shear strength models [ACI 318−2011, FEMA 273 and 356, as modified by ASCE/SEI 41–07, EC8−III (2005)] are tested against two experimental databases comprising elements from the international literature. The first database consists of approximately 550 elements detailed according to the modern provisions whereas the second consists of 40 poorly detailed elements (inadequate transverse reinforcement with brittle details, inadequate lap splices in critical regions, low concrete and steel strengths etc.). Furthermore a parametric study was conducted with regards to the displacement ductility, the normalized axial load, the aspect ratio, the transverse reinforcement and other important design parameters.

The results are presented in the form of graphs, in order to highlight the deficiencies of each model whereas the scatter observed illustrates the need for additional investigation to improve the shear strength estimates.

1. INTRODUCTION

A primary objective of performance–based assessment and/or design is to dependably estimate the various strength indices that a R.C. member may sustain when subjected to seismic loads, namely the shear force required to develop the flexural capacity at the support, V_{flex}, the web shear strength, V_{shear}, the shear force that may be sustained when the anchorage of reinforcement or lap splice reaches its capacity, V_{lap}, and the shear force sustained when compression reinforcement buckles at the critical section, V_{buckl}. The most critical of these response mechanisms appears to be shear, as its occurrence prior to development of failure in other response mechanisms leads to brittle failure.

The issue of dependably estimating the shear strength of a R.C. element appears to be rather complicated as it presumes the full understanding of the several interacting behavior mechanisms under reversed cyclic loading whereas it is strongly affected by the imposed loading history, the dimensions of the element (e.g. the aspect ratio), the concrete strength, the longitudinal reinforcement ratio but mostly the ratio and the detailing of the transverse reinforcement. So far it has not been possible to theoretically describe the strength of the shear mechanism from first principles of mechanics without use of calibrated empirical constants. Therefore the shear strength estimates obtained from calibrated design expressions necessarily rely on the pool of experimental data used for correlation of the empirical expressions, but also on the preconceived notions of the individual researchers as to the role each variable has on the mechanics of shear.

In the present work the most widely used shear strength models, adopted by the modern Design and Assessment Seismic Codes (ACI 318−2011, FEMA 273 and FEMA 356, as modified by ASCE/SEI 41–07, Eurocode 8–2005) are compared. They are tested against two extensive experimental databases assembled from tests published in the literature international. The first database consists of approximately 550 elements detailed according to modern provisions whereas the second consists of 40 poorly detailed elements, referred to as ‘old-type’. Particular emphasis is placed on these ‘old type’ elements as the inventory of R.C. structures that qualify under this classification is vast throughout the seismic regions of the world. A major shift in international seismic design codes occurred in the early 1980’s introducing the modern form of detailing with closed stirrups and proper reinforcement anchorage as a required practice. Structures designed prior to that point conform to a variety of earlier standards, as these evolved through the years from the post world-war era to the 1980’s. These structures are at the core of the “seismic assessment codes” such as ASCE–SEI 41 (2007) and EC8–III (2005); this concern has been motivated by the observed poor performance of these structures in strong earthquakes throughout the world from the 1990s till today [7, 8, 9].

Furthermore, a parametric study of the models describing the shear strength mechanism was conducted in order to study the parametric sensitivity against the most important design parameters such as the displacement ductility, the normalized axial load, the aspect ratio, the cross sectional area, the volumetric ratio of the transverse reinforcement, the normalized stirrup spacing, the ratio of the longitudinal reinforcement, the yield strength of the longitudinal and the transverse reinforcement and the concrete strength. The scope of this study was to check whether those models interpret consistently with basic mechanics, the influence of the design parameters on the behaviour of reinforced concrete members.

2. DESCRIPTION OF THE EXPERIMENTAL DATABASES

The first database the shear strength models are tested against consists of approximately 550 tests assembled from the international literature [10]. The elements are designed according to modern provisions, hence this database will be referred to from here on as ‘Modern type Database’ or M.D. The tests it is assembled from are detailed with adequate transverse reinforcement, anchored inside the confined core with 135^0 hooks, lap splices are placed outside the critical region and the yield strength of the transverse and the longitudinal reinforcement is substantial. The Old type Database, referred to as O.D., consists of approximately 40 elements with poor and occasionally smooth longitudinal and transverse reinforcement, 90^0 stirrup hooks and lap splices with insufficient length located in the critical regions. Specimens of both databases were tested under cyclic lateral load reversals simulating earthquake effects with or without axial load. The most important database parameters are listed in Table 1.

Parameter	Modern type Database(M.D.)	Old type Database(O.D.)
$P/f_c'A_g$	0.0 to 0.85 (P=0 for 45% of the database)	0.0 to 0.60 (P=0 for 37% of the database)
L_s/h	0.8 to 10.0	2.0 to 6.0
r_c	0.5% to 6.0%	1.0 to 5.0% (lap-splices in the critical region: 60%)
$r_{s,\,vol}$	0.0% to 6.0% ($r_{s,\,vol}$= 0.0 for 10%)	0.2% to 1.5% (90^0 hooks for 55%)
s/D_{bl}	1.6 to 15.0	3.2 to 18.0
$k_{eff}\, r_s f_{yh}/f_c'$	0.0 to 0.7	0.002 to 0.150
$v_{fail}/\sqrt{f_c'}$	0.0 to 2.3 ($v_n/\sqrt{f_c'}$= 0.0 for 7%)	0.07 to 0.37
f_c' (MPa)	14.0 to 130.0	18.0 to 36.0
f_{sy} (MPa)	117.0 to 932.0	317.0 to 548.0
f_{yh} (MPa)	236.0 to 2050.0	210.0 to 481.0

$P/f_c'A_g$: normalized axial load, L_s/h: aspect ratio, r_c: ratio of the longitudinal Reinf., $r_{s,vol}$: volumetric ratio of transverse Reinf., s: stirrup spacing, D_{bl}: diameter of longitudinal Reinf., $k_{eff}r_sf_{yh}/f_c'$: effective normalized confining pressure provided by transverse Reinf. in the direction of lateral sway, $v_{fail}/\sqrt{f_c'}$: normalized applied shear stress, f_c': concrete strength, f_{sy}, f_{yh} : yield strength of the longitudinal and the transverse Reinf.

Table 1: Ranges of Database Parameters

3. SHEAR STRENGTH MODELS

In the following sections the shear strength models adopted by the Design Code ACI 318−2011 and the Assessment Codes FEMA 273 (1997) and FEMA 356 (2000), as modified by ASCE/SEI 41–07, and Eurocode 8 – III (2005) are considered. The analytical results from the implementation of each model on the two databases are graphically presented whereas an extra criterion was used concerning the ratio of the applied shear stress v_{fail} to the available shear capacity calculated according with the provisions of ACI 318 (2011), v_n^{ACI}. Based on recent tests it has become evident that shear strength of reinforced concrete degrades the faster with cyclic load, the higher the ratio of shear demand to shear supply. Wood and Sittipunt (1996) have proposed a limit of 60% as a cutoff point in identifying shear failures from other types of failures when processing the experimental literature on walls. Thus, a shear failure is likely to occur when $0.6v_n^{ACI}<v_{fail}$, even if the nominal check prescribed by the code holds, namely that $v_n^{ACI}<v_{fail}$. Furthermore the contribution of concrete V_c and of the axial load V_p to the total shear strength is graphically presented so as to highlight the influence of each mechanism.

3.1 Design Code ACI 318−2011

The shear strength model adopted by the American Design Code ACI 318−2011 is depicted by the following equations:

$$V_c = 0.17 \cdot \left(1 + \frac{P}{14A_g}\right) \cdot \sqrt{f_c'} \cdot b \cdot d \quad \text{and} \quad V_{s,ACI} = \frac{A_{tr} f_{yh} d}{s} \quad \text{(units in Nt,mm)} \tag{1}$$

where P is the axial load, A_g is the area of the cross section of the element, f_c' is the concrete strength, b is the width of the section and d is the effective cross section depth, A_{tr} is the area of the stirrups that may be crossed by a crack, f_{yh} is the yield strength of the stirrups and s is the stirrup spacing.

The analytical results of the model are presented in Fig. 1, graphs (a) and (b) for the M.D. and graphs (c) and (d) for the O.D. Note that the diagonal line drawn in each graph represents the equal value case (i.e., the case where test result equals the analytical prediction). The concrete contribution to shear strength V_c appears to be rather well estimated for elements failing in shear, whereas for the group where a flexural mode of failure is expected, values are overestimated. The results concerning the total shear strength for the shear-failing elements are in good agreement with the tests but significant scatter appears when the flexural elements are considered. Graphs (c) and (d) in Fig. 1 concerning the O.D. present similar trends, however it is noteworthy that the model adopted by ACI 318–2011 refers to design of R.C. elements and not to assessment and therefore does not take into consideration the imposed ductility level.

3.2 ASCE/SEI 41–2007

The above mentioned model is also used in FEMA Guidelines 273 (1997) and FEMA 356 (2000) and ASCE/SEI 41–07 under certain modifications that account for the concrete degradation under seismic loading using the reduction factor k:

$$V_c = k \cdot \left(\frac{0.5\sqrt{f_c'}}{L_s / d} \sqrt{1 + \frac{P}{0.5A_g\sqrt{f_c'}}} \right) \cdot 0.8A_g \quad \text{and} \quad V_s = k \cdot \frac{A_{tr} f_{yh} d}{s} \quad \text{(units in Nt,mm)} \tag{2}$$

where factor k equals to 1.0 for elements with ductility demand $m_D \leq 2.0$, $k = -0.1m_D + 1.2$ for elements with ductility demand $2.0 < m_D \leq 6.0$ and $k = 0.6$ for elements with higher ductility demands.

The analytical results are presented in Fig. 1, in graphs (e) and (f) for the M.D. and in graphs (g) and (h) for the O.D. There is sufficient accuracy in the concrete contribution to shear strength for both the group of the shear-controlled and the flexure-controlled elements. Yet the analytical results for the group of the flexure-controlled elements are overly optimistic when the total shear strength is under consideration. The analytical results for the O.D. seem to have very good correlation with the experimental data for both the flexure and the shear controlled elements.

3.3 Eurocode 8 – III (2005)

The shear strength model adopted by Eurocode 8 – III is presented in the following section. The cyclic shear resistance of a R.C. element V_R, decreases with the plastic part of ductility demand, expressed in terms of ductility factor of the transverse deflection of the shear span or of the chord rotation at member end: $m_{\Delta,pl} = m_\Delta - 1$. For this purpose $m_{\Delta,pl}$ may be calculated as the ratio of the plastic part of the chord rotation, q_{pl}, normalized to the chord rotation at yielding, q_y. The following expression is used for the shear strength, as controlled by the stirrups, accounting for the above reduction (with units Nt and mm):

$$V_R = \frac{1}{\gamma_{el}} \left\{ \begin{array}{c} \dfrac{h-c}{2L_s} min\left(P, 0.55 A_e f_c\right) + \left[1 - 0.05\, min\left(5; \mu_{\Delta}^{pl}\right)\right] \times \\ \times \left[0.16\, max\left(0.5; 100\rho_{tot}\right) \cdot \left(1 - 0.16\, min\left(5; \dfrac{L_s}{h}\right)\right) \sqrt{f_c}\, A_e + V_s \right] \end{array} \right\} \tag{3}$$

where g_{el} is a factor equal to 1.15 for primary seismic elements and 1.0 for secondary seismic elements (it is taken equal to 1.0 for the two databases), h is the depth of the cross-section (equal to the diameter D for circular sections), c is the compression zone depth, L_s is the shear span, P is the compressive axial load (positive, taken as being zero for tension), A_e is the cross-section area, taken as being equal to $b{\cdot}d$ for rectangular cross-sections (b is the width and d is the effective depth), or to $pD_c^2/4$ (where D_c is the diameter of the concrete core to the inside of the hoops) for circular sections.

The concrete compressive strength is f_c', r_{tot} is the total longitudinal reinforcement ratio and V_s is the contribution of transverse reinforcement to shear resistance, taken as being equal to:

$$V_s = \rho_{tr} b\, z\, f_{yh} \text{ for rectangular sections}, V_s = \frac{\pi}{2}\frac{A_{sp}}{s} f_{yh}\left(D - 2c_o\right) \text{ for circular sections} \tag{4}$$

where r_{tr} is the transverse reinforcement ratio, z is the length of the internal lever arm and f_{yh} is the yield stress of the transverse reinforcement, A_{sp} is the cross-sectional area of a circular stirrup, s is the centerline spacing of stirrups and c_o is the concrete cover.

If in a concrete column the shear span ratio L_s/h at the end sections where flexural moment is maximized is less or equal to 2.0, its shear strength, V_R, should not be taken greater than the value corresponding to failure by web crushing along the diagonal of the column after flexural yielding, $V_{R,max}$, which, under cyclic loading may be calculated from the expression (with units Nt and mm):

$$V_{n,max} = \frac{\frac{4}{7}\left[1 - 0.02\, min\left(5; \mu_{\Delta}^{\mathrm{pl}}\right)\right]}{\gamma_{el}} \left(1 + 1.35 \frac{P}{A_e f_c'}\right) \left[1 + 0.45\left(100\rho_{tot}\right)\right] \sqrt{min\left(40; f_c'\right)}\, b\, z\, sin 2\theta \tag{5}$$

where q is the angle between the diagonal and the axis of the column ($\tan q = h/(2L_s)$).

The analytical results are shown in Figure 1, graphs (i) and (j) for the M.D. and graphs (k) and (*l*) for the O.D. The concrete contribution to shear strength is adequately estimated as compared to the experimental results both for the group of the shear-controlled elements and for the flexure-controlled group. The results of the total shear strength also appear to have rather sufficient accuracy when the shear-controlled elements are considered, yet the analytical results for the group of the flexure-controlled elements are overly optimistic. The analytical results for the O.D. also seem to have very good correlation with the experimental data.

4. PARAMETRIC INVESTIGATION OF THE SHEAR STRENGTH MODELS

A parametric study of the shear strength models was conducted in order to highlight the deficiencies of each model over the most important design parameters such as the displacement ductility, the normalized axial load, the aspect ratio, the cross sectional area, the volumetric ratio of the transverse reinforcement, the normalized stirrup spacing, the ratio of the longitudinal reinforcement, the yield strength of the longitudinal and the transverse reinforcement and the concrete strength. The scope of this parametric investigation is to check whether the shear strength models interpret in a consistent manner the influence of the design parameters on the mechanical behaviour.

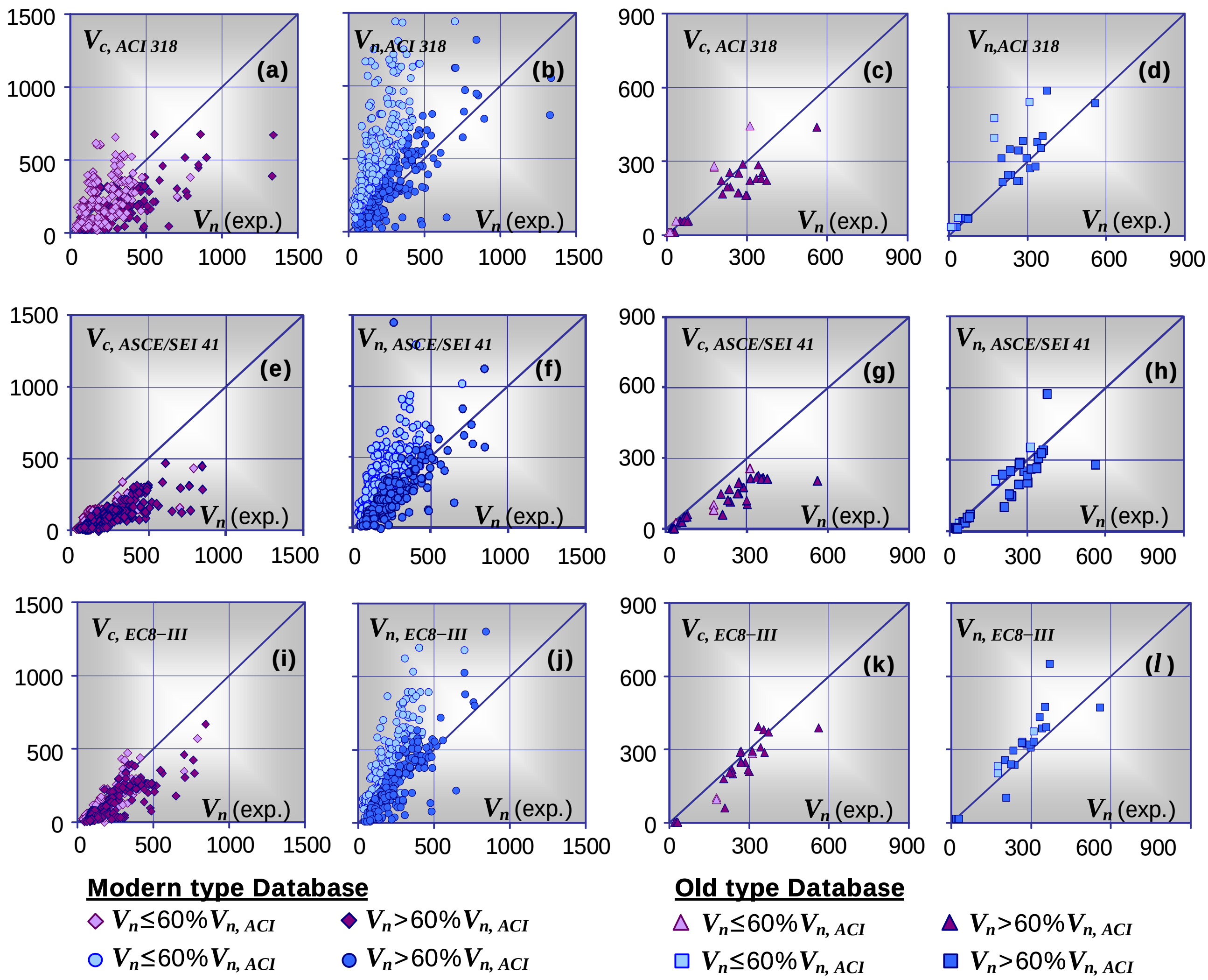

Figure 1 Implementation of the shear strength models upon the two experimental databases

As a reference point an element from Lynn et al. (1996) was used under the code name 3CLH18, with old type characteristics. The column had an aspect ratio of L_s/d=3.9 and a 457 mm square cross section and it was tested in double-curvature, by imposed lateral load reversals and a constant axial load equal to $P/(f_c'A_g)$=0.09, until loss of vertical load carrying capacity. The longitudinal reinforcement consisted of 8Ø32 bars evenly distributed around the perimeter of the cross–section, with yield strength f_{sy}=331 MPa and a clear cover of 50mm. The transverse reinforcement consisted of 9.5mm perimeter stirrups anchored with 90^0 bends and placed at a spacing of 457mm (≈d), having a yield strength f_{yh}=400MPa. The concrete strength was f_c'=25 MPa and the specimen failed in shear at a ductility level of m_D=1.6. The results of the parametric study are presented in Figures 2 and 3.

The estimated total shear strength V_n is plotted against the imposed ductility level m_D in the graph of Fig. 2a. Both the models adopted by ASCE/SEI 41–2007 and EC8–III (2005) show a mild descending curve when a case of moderate ductility demand is in question that is for m_D values greater than 3.0 or 4.0, whereas the ACI 318 model (2011) is insensitive to the ductility demand as would be expected by a Design Code. The influence of the axial load is presented in graph 2b and seems to enhance the shear strength both for ACI 318 and ASCE/SEI 41 regardless of the imposed magnitude whereas the EC8–III correctly suggests a cutoff point for exceeding axial load values. The effect of the aspect ratio is shown in graph 2c and sets a strong influence both for the EC8–III and the ASCE/SEI 41 model though ACI 318 code is insensitive to any variation in this parameter. The influence that the size of the cross section may have upon the shear strength is explored in graph 2d. All the models have proportional results and are enhanced with the increase of the cross section area A_g. The shear strength is plotted in graph 2e against the volumetric ratio of the transverse reinforcement and all the shear models are enhanced with the increasing values of this ratio. Note that the reference point is also depicted in each graph with better correlation to EC8–III.

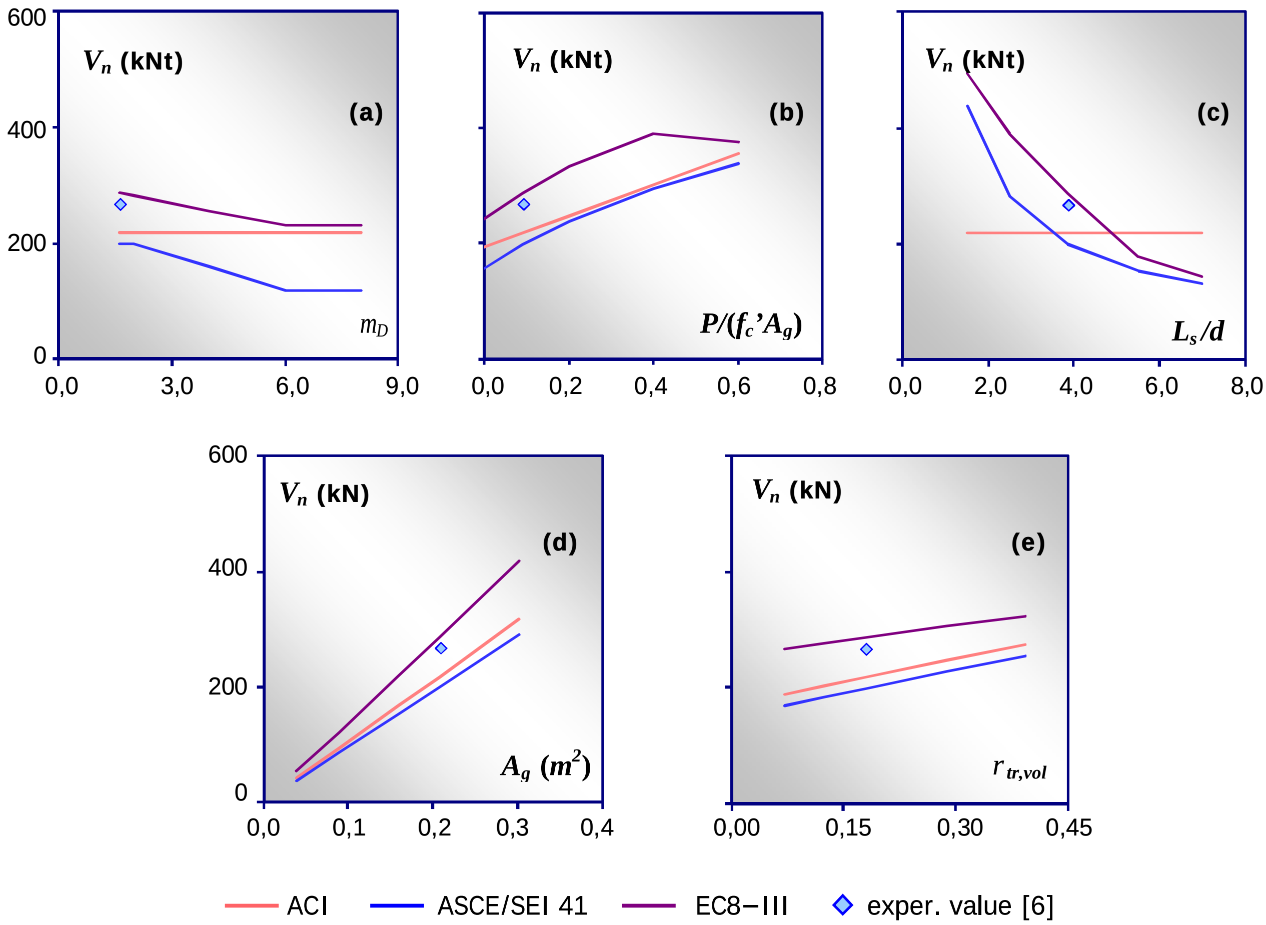

Figure 2 Parametric study of the shear strength models against the imposed ductility, the axial load, the aspect ratio, the cross section area and the transverse reinforcement

The next parameter tested is the ratio of the stirrup spacing normalized by the diameter of the longitudinal reinforcement, s/D_{bl}, a ratio that identifies the risk of reinforcement for compression buckling, as shown in graph 3a. The influence of this parameter is almost impalpable to all the shear strength values. The yield strength of the transverse reinforcement is under consideration in graph 3b showing the same ascending trend for all the shear strength models. Almost insensitive are the estimates provided by all the shear models when the yield strength of the longitudinal reinforcement is considered in graph 3c. The ratio of the total longitudinal reinforcement affects strongly the EC8–III shear model but almost does not affect any of the other models as plotted in graph 3d. Finally increasing the concrete strength affects a proportional increase in the shear strength estimates obtained by all the models as shown in graph 3e.

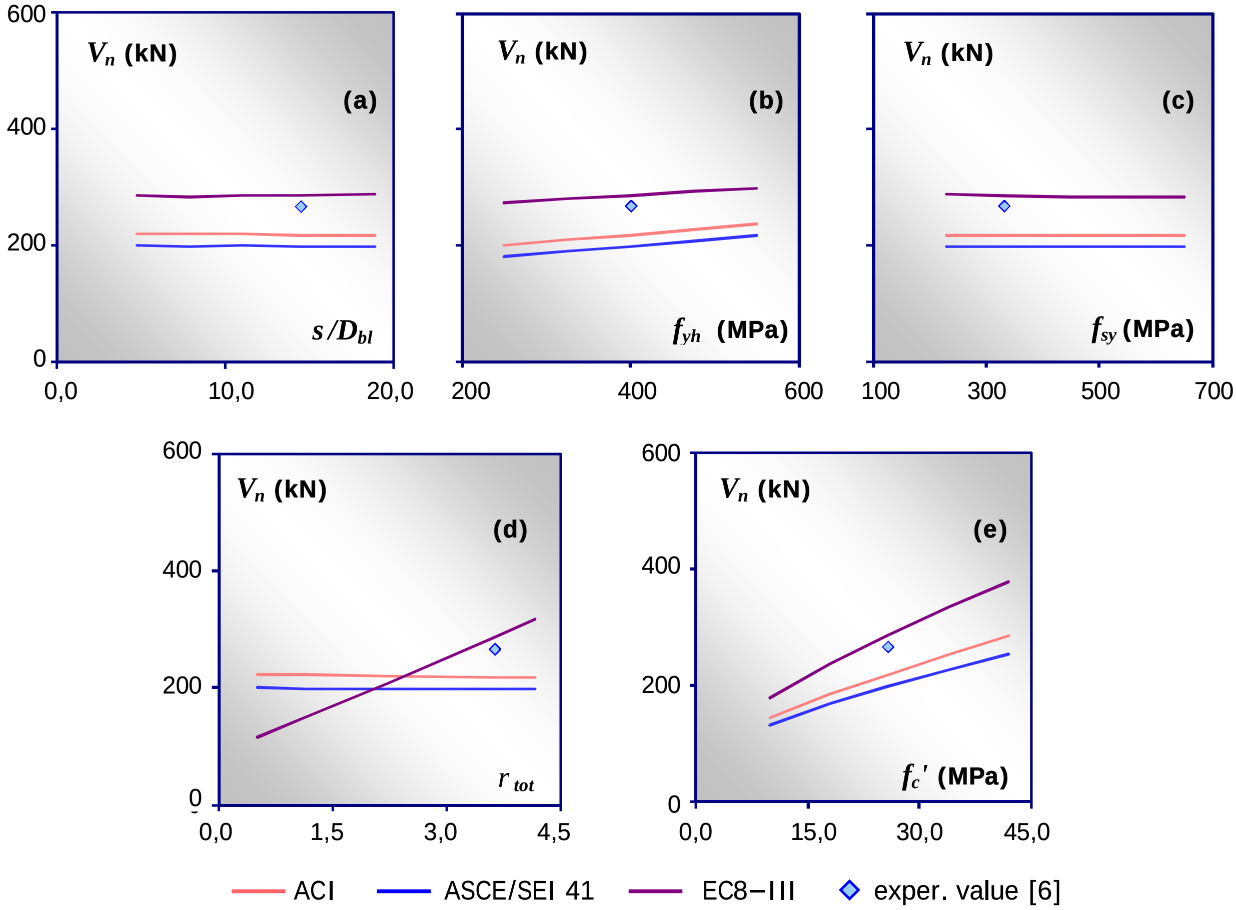

Figure 3 Parametric study of the shear strength models against the normalized stirrup spacing, the yield strength of the transverse and the longitudinal reinforcement, the ratio of the longitudinal reinforcement and the concrete strength.

5. CONCLUSIONS

The most important shear strength models adopted by current seismic Codes, namely the Design Code ACI 318−2011 and the Assessment Codes ASCE/SEI 41–07 and Eurocode 8 – III (2005), were presented and tested against two extensive experimental databases assembled with elements from the international literature. When the elements detailed according to modern provisions were considered, there was significant scatter and the correlation would deteriorate for the flexure-controlled tests. The results are similar for the old type elements (with insufficient longitudinal and transverse reinforcement, poor detailing, etc.) although the estimates according with EC8−III showed better correlation. Note that a criterion concerning the ratio of the applied shear stress to the available shear capacity was used in order to classify the available specimens in the two databases according to the anticipated type of failure, namely, shear-controlled and flexure-controlled failure.

A parametric study was also conducted with regards to various important design parameters such as the displacement ductility, the axial load, the transverse reinforcement, etc. and conclusions were drawn concerning the deficiencies of each model over those parameters; for this purpose the dimensions, the material properties and the details of an old-type specimen were used as a point of reference in the study. The scope of this study was to check whether the shear strength models reproduce consistently the influence of the design parameters on the mechanical behaviour of the reference element.

Although the EC8−III model produced adequate results when the old type database was considered, the small number of tests discourages generalization of conclusions. Furthermore the equations the model were calibrated to match a great number of tests from the international literature therefore posing a remarkable bias on the observed good correlation. Thus the primary finding regarding the models describing the shear strength is that intense scatter persists today, underscoring the great uncertainty in the relevant expressions of assessment procedures.

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