



Review Article

Shear Behavior of High-Strength Concrete Column: A Review of Mechanisms, Design Models, and Experimental Evidence

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Abstract

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High-strength concrete (HSC) lets columns carry more load in a smaller section, but its gain in compressive strength is not matched by a proportional gain in tensile capacity, and HSC columns therefore remain prone to sudden, low-ductility shear failure rather than the gradual, flexure-dominated failure that seismic design prefers. This paper reviews the mechanics, design-code treatment, and experimental evidence for the shear behaviour of HSC columns (nominal $f'_c \geq 40$ –50 MPa) under monotonic and cyclic loading. Section II summarizes the shear-transfer mechanisms specific to reinforced concrete columns and the scope and approach used for the review. Section III compares the governing shear-strength expressions of ACI 318, the UCSD/Priestley degrading-truss model, the Sezen–Moehle model for lightly reinforced columns, and recent data-driven formulations, and evaluates three representative code-type expressions for the concrete contribution, V_c , over a realistic range of axial load ratio. Section IV synthesizes the reported effects of concrete strength, axial load ratio, transverse reinforcement ratio and spacing, longitudinal and transverse steel grade, steel-fibre addition, and composite confinement on shear strength, failure mode, and ductility, drawing on studies published between 1994 and 2024. Section V presents a detailed case study: a cyclic-loading test programme on large-scale 70 MPa columns, in which shear-reinforcement ratio, longitudinal steel grade (CRB735 versus HRB400), stirrup spacing, and axial load ratio were varied, and the reported findings are compared against an 86-column literature database and against ACI 318's shear provisions. Across the reviewed evidence, higher axial load consistently raises shear strength while steepening the critical crack and reducing deformation capacity; closer or higher-grade transverse reinforcement delays, but does not eliminate, brittle shear failure; and the reporting of ductility and post-cracking deformation remains inconsistent across studies, which limits like-for-like comparison. Section VI compares these trends against code predictions, and Section VII proposes a small set of reporting items that would make future HSC column shear studies directly comparable, most of which are already measured in typical test programmes but not always published. The review concludes that shear design of HSC columns is best treated as a strength-ductility trade-off rather than a strength-maximization problem alone.

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I. INTRODUCTION

High-strength concrete (HSC) is normally defined as concrete with a specified compressive strength of more than 40–50 MPa, while the mixes with strength more than 100 MPa are currently in use in tall buildings, long-span bridges and offshore structures [1] and [2]. The strength is obtained using low water-cement ratio, well-graded aggregate and the use of supplementary cementitious materials like silica fume,

fly ash or slag which also give a denser, less permeable microstructure that provides resistance to chloride ingress, sulfate attack and freeze-thaw damage [2]. The simplicity of the attraction for the designer is that a smaller HSC column can support the same axial load as a larger column with the same normal strength, thus freeing up space on the floor, lowering self-weight, and potentially reducing the cost of materials and formwork. What makes it challenging is that compressive strength is not necessarily related to shear

behavior. The capacity of an HSC column to resist tensile and diagonal-tension is much lower than that to resist compression, and so an HSC column may fail in shear at a load close to that of a normal-strength column, with very little warning and very little post-peak deformation [1], [7]. Shear failure in a column effectively pre-empted vertical load path and is the failure mode most sought after by building codes to be avoided in members that are also resisting seismic drift. It is therefore not a secondary consideration in HSC column design to restore ductility to a strong brittle section – it is the primary consideration – it is this property that has drawn a huge and still expanding experimental literature on the shear behaviour of HSC columns. That literature is broken up in a particular manner. The studies compare concrete grade, column shape, longitudinal and transverse reinforcement ratio and grade, axial load ratio, shear span, and loading protocol independent of each other, and with no single study comparing to multiple others. Typically, a shear strength equation that has been validated for 30–40 MPa concrete is extrapolated to 70–100 MPa concrete without the necessary test data to support the extrapolation [4]–[6]. If the strength of the transverse reinforcement is high, the picture changes again: if the reinforcement is not yielding at diagonal cracks it will behave (Sections V and VI). The very short set of reporting recommendations in Section VII is inexpensive, and Section VIII concludes. Unlike reinforcement intended to yield, which was the case with current minimum-shear-reinforcement provisions, they were based primarily on normal-strength, normal-grade test data [10], [11].

There are three things this review does. First, it identifies the shear-transfer mechanisms applicable to columns and discusses the shear-strength models presented in the main design codes and the analyses used, along with the differences between them in their treatment of axial load (Section III). Second, it collates, by parameter, the information published in the literature, both experimental and analytical, obtained from 1994 to 2024 concerning shear behaviour of columns made of HSC in general (Section IV). Third, it is an extensive case study of a cyclic loading test programme conducted on large-scale 70 MPa columns, with the shear reinforcement ratio, grade of the longitudinal reinforcement, spacing of stirrups and axial load ratio all varied; it draws conclusions from this test programme and relates it to the wider literature and to the code predictions.

II. FUNDAMENTALS OF SHEAR BEHAVIOR AND REVIEW SCOPE

A. Shear-Transfer Mechanisms in RC Columns

The shear strength (V_n) of a reinforced concrete column is typically composed of a contribution from the concrete (V_c), and a contribution from the transverse reinforcement (V_s) (Figure 1). V_c summarizes several processes which are not

easily distinguishable in experiments: shear across the uncracked compression zone above the crack, aggregate interlock along the crack surfaces, dowel action of the longitudinal bars, and, in larger axial load, strut or arch action between the load point and the support [3], [4]. The V_s is supported, typically, by the stirrups or hoops that extend across the diagonal crack and is usually modelled using a 45° truss analogy, which means that V_s depends on the transverse-reinforcement ratio, the yield strength and the section depth. Both terms are affected by axial compression. Each shear model reviewed in Section III contains an axial-load term, as the addition of a moderate axial load increases V_c by inducing a delay in flexural cracking and a mobilization of arch action. However, the same axial load also increases the angle of the critical diagonal crack, decreases the curvature ductility available before failure occurs in flexure, and may cause a flexure-shear failure to be near-brittle shear failure after the column cracks (Figure 1). This duality - a strength gain at the price of ductility - will be repeated in the subsequent sections IV-VI and will be the basis of this review, which therefore reports strength gains in tandem with ductility losses.

B. Review Scope and Approach

This review focuses on experimental, analytical and numerical investigations on shear and shear-related seismic behaviour of RC columns of nominal concrete compressive strength higher or equal to around 40–50 MPa, published predominantly from the year 1994 (when the Priestley-Verma-Xiao shear model was published, which is still the foundation for several code and guideline procedures) to 2024. Academic search engines and publisher databases (ASCE Library, ScienceDirect, MDPI, Taylor & Francis, Springer, ACI and Semantic Scholar) were used to search for sources, with combinations of the terms “high-strength concrete,” “column,” “shear,” “axial load ratio,” “transverse reinforcement,” “high-strength steel,” and “seismic performance” used for the searches, along with citation tracing from papers identified. The shear behaviour of beam, wall and beam-column joints is referenced in companion literature only where it supports a model or a finding that is applied directly to the columns elsewhere in this review (for example, Sezen-Moehle and data-driven shear models); a review of these member types is not the scope of this review. This is a structured narrative and quantitative synthesis, as studies were selected for relevance, but not based on a pre-registered, reproducible query count, and for the specificity of the quantities reported (axial load ratio, transverse-reinforcement ratio, reported shear or lateral-load capacity, and, where available, a ductility or energy- dissipation measure). This restriction is explicitly restated here because it is relevant for the recommendations made in Section VII for making future HSC-column shear studies easier to compare and pool.

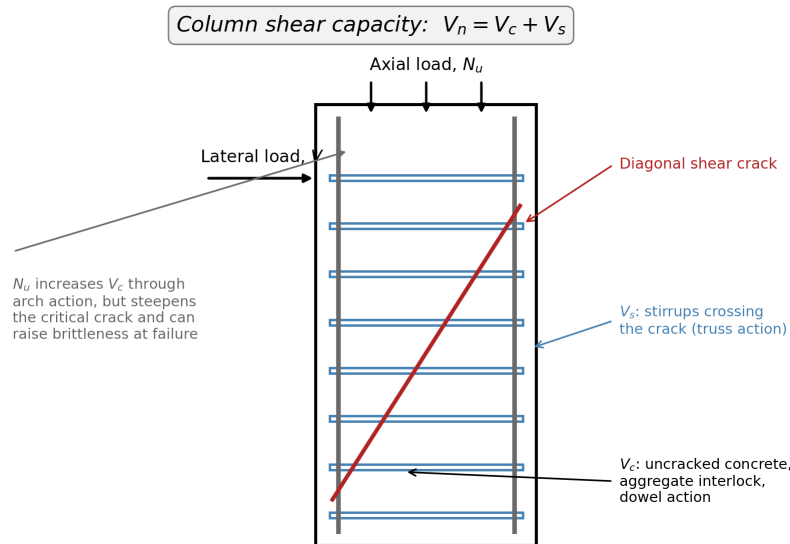


Fig.1 Idealized Shear-resisting Mechanisms in an RC column: the Concrete Contribution V_c (uncracked zone, aggregate interlock, dowel action) and the Truss Contribution V_s from Stirrups Crossing the Diagonal Crack. Axial Load Raises V_c through Arch Action but Steepens the Crack and can Reduce Ductility.

III. DESIGN-CODE AND ANALYTICAL SHEAR STRENGTH MODELS

Main expressions used to predict the concrete contribution to the shear strength of the column are compared in Table I. They all have the same form, which is based on a base term proportional to $\sqrt{f'_c}$ and the section area, with an upward adjustment for axial compression (some of them), but differ in the level of conservatism in the upward adjustment, as well as whether they reduce V_c due to displacement ductility, and whether they are calibrated to high-strength or lightly reinforced specimens.

A. ACI 318

ACI 318 nominal shear strength is determined as $V_n = V_c + V_s$, and there is a simplified portion of the formula for V_c that also includes the axial load $(1 + N_u / 14A_g)$ (SI units, where N_u is the axial load in newtons, and A_g is the gross area in mm^2), prior to multiplying $0.17\sqrt{f'_c} \cdot b_w \cdot d$ [1]. ACI 318 also allows a more specific calculation of V_c which is generally less stringent for high axial load ratios, based on the moment-shear ratio at the section under consideration. The minimum shear reinforcement requirements in ACI 318 are based primarily on the test results from normal-strength, normal-grade concrete and reinforcement; recent research has indicated that the minimum reinforcement-area requirements are less effective if either the concrete or the transverse reinforcement is high-strength, as a smaller amount of higher-grade steel can meet the area requirement in the code but behave differently after diagonal cracking [10] and [11]. The general material background for concrete with strength above 40–62 MPa, on which these provisions are based, is given in ACI 363R [2] and the historical development of the shear design provisions is detailed in ACI 445R [3].

B. UCSD/Priestley Model and the Sezen–Moehle Model

Priestley, Verma and Xiao presented a degrading shear-strength model where V_c is formulated using a concrete-strength term, an axial-load term and an aspect-ratio term multiplied by a factor that diminishes as the displacement ductility requirement increases [4]. The physical concept is that this physical contribution measured at first cracking is not present after a few cycles of inelastic displacement of the column, as both the aggregate interlock and the uncracked compression zone are degraded by cyclic damage. The model was later modified by Kowalsky and Priestley, specifically for small values of the aspect-ratio term, for circular columns in seismic regions [5]. A related but different model was proposed by Sezen and Moehle and calibrated specifically against lightly reinforced columns in which the contribution of the concrete also deteriorates with ductility but is scaled differently with axial load and aspect ratio and which was found to be more representative of observed column shear failures, including those that failed after flexural yielding, than the available ACI 318 expression at the time [6]. In a later work [9] Cao and Li put forward another modification specifically for lightly stirruted HSC columns, which are included in the group of columns with low transverse-reinforcement ratio.

C. Data-Driven and Machine-Learning Models

A recent related line of research is to fit the statistical or machine-learning model directly to the compiled column-shear databases without seeking a closed-form mechanical expression. Azadi Kakavand, Sezen, and Tacioglu developed data-driven models using rectangular and circular column shear-test databases and found that their models offer better predictive capabilities than the closed-form models mentioned above, especially when used for columns outside the range of parameters used to fit the closed-form models

[16]. With the same goal of extending the range of parameters where the column shear strength can be predicted, Ioannou et al. presented new symbolic and machine-learning-based expressions for column shear strength [17]. Nguyen *et al.* tested several of the shear-strength models mentioned above using a common test database and concluded that none of the models always performed best for the entire range of axial

load ratio and transverse-reinforcement ratio represented in the test database, as shown in the divergence of the models in the following Figure 2 [15]. Data-driven models are fit but lose the mechanical interpretability; most published databases are still relatively poor at modelling columns above 60–70 MPa (which is the range of interest of this review).

TABLE I COMPARISON OF CODE-BASED AND ANALYTICAL MODELS FOR THE CONCRETE SHEAR CONTRIBUTION, V_c , IN RC COLUMNS

Model / Reference	Basis for V_c	Ductility-dependent?	Notes
ACI 318 – simplified [1]	$0.17(1 + Nu / 14Ag)\sqrt{f'_c} b_w d$ (SI)	No	Widely used; conservative at low–moderate axial load ratio
ACI 318 – detailed [1]	Function of moment/shear ratio at section	No	Can be less conservative at high axial load
Priestley–Verma–Xiao (UCSD) [4]	$\sqrt{f'_c}$, axial load, and aspect-ratio terms	Yes – degrades with $\mu\Delta$	Basis of several seismic assessment guidelines
Kowalsky–Priestley [5]	Refinement of UCSD model for circular columns	Yes	Improves small-aspect-ratio behaviour
Sezen–Moeble [6]	$\sqrt{f'_c}$, axial load and aspect-ratio, calibrated on lightly reinforced columns	Yes – degrades with $\mu\Delta$	Tracks post-yield shear failures well
Cao–Li [9]	Refinement for low transverse-reinforcement ratio	Yes	Targeted at lightly stirrups columns
Data-driven / ML [15]–[17]	Statistical fit to compiled test databases	Model-dependent	Accuracy limited by training-database coverage above ~60–70 MPa

IV. PARAMETRIC INFLUENCES ON THE SHEAR BEHAVIOUR OF HSC COLUMNS

A. Concrete Compressive Strength

Xiao and Martirosyan subjected high-strength (nominal 60–90 MPa) concrete columns to constant axial load and cyclic lateral displacement, and concluded that, once shear-critical, greater concrete strength alone does not allow the column to regain ductile behaviour; several of the columns failed in shear immediately after, or even before, longitudinal-bar yielding, and there was significantly less hysteretic energy dissipation in these high-strength concrete columns, compared to similar normal-strength concrete columns [7]. Jin *et al.* conducted tests on shear-critical HSC columns, and also determined that when the transverse reinforcement was inadequate, relative to the higher shear demand that a stronger and stiffer column brings, brittle shear failure controlled the response [8]. In general, from this literature, the higher the f'_c , the greater the diagonal-cracking load and the concrete shear contribution (which are proportional to $\sqrt{f'_c}$); however, deformation capacity (ductility) is provided primarily by the transverse reinforcement and axial-load management (discussed below) and not by the concrete.

B. Axial Load Ratio

$Nu / (f'_c Ag)$ is the most commonly published parameter that increases shear strength and decreases ductility, namely axial load ratio (ALR). Both the UCSD and Sezen–Moeble models incorporate this trade-off directly in V_c [4]–[6]. However, Wang *et al.* conducted tests on steel-reinforced HSC columns and determined that the increase in the compression ratio (from 0.2 to 0.37) resulted in an increase in strength but a

decrease in the displacement ductility coefficient by approximately 29% and they identified the shear span ratio, steel content, and stirrup ratio as parameters that enhance seismic performance, while the compression ratio exhibited the opposite trend [13]. This is like the qualitative trade-off observed in the case study of Section V, in which an increase in the axial load ratio (ALR) from 0.3 to 0.5 decreased the ductility and the energy dissipated by a comparable amount to doubling the stirrup spacing (Section V.G, Figure 4).

C. Transverse-Reinforcement Ratio and Spacing

The literature reports that closer stirrup spacing and a higher number of stirrups per unit length of column (transverse-reinforcement ratio) always postpone diagonal cracking and at the same time increase the value of V_s and also increase the ductility after diagonal cracking. But it is evident that this benefit saturates and that of course the minimum-area requirement of a code does not guarantee that the transverse reinforcement will yield before the column loses its capacity. In testing columns of high-quality transverse reinforcement under high shear stress, Sokoli and Ghannoum observed that the high shear failure did not always occur at the yield of the transverse reinforcement, as assumed in the minimum-area provision, and that the reinforcement did not always yield at failure if it met the minimum area requirement at normal strength [10].

Ou, Alrasyid, and Nguyen studied the minimum shear reinforcement for a combined high-strength reinforcement and high-strength concrete and suggested some modifications to ensure that the shear reinforcement will yield before shear failure [11]. This is corroborated by the case-study result (Section V) that not all the columns tested

in this study exhibited stirrup yielding at peak load even for the highest tested transverse-reinforcement ratio.

D. Longitudinal and Transverse Steel Grade

Increasing the grade of reinforcing steel, not the amount of it, is an increasingly popular approach to HSC columns, as the same nominal capacity can be achieved with a less congested reinforcement and this can aid in the placement and consolidation of the concrete, which is of high strength, but often has a low workability. Wang *et al.* investigated the behaviour of steel-reinforced concrete (SRC) columns under axial compression using a Q690 high-strength structural steel and Q235 ordinary steel, and observed that the compression ratio and other parameters were kept constant, the displacement ductility coefficient of the former is about 38% higher than that of the latter, and the failure modes of both are different, that the former is the steep-diagonal "shear" failure, but the latter is the shear span ratio high bending-dominated failure [13]. This is like the CRB735 versus HRB400 in the Section V case study (Section V.F), which is the same exchange in strength for ductility that was observed throughout this section, except that it moves from the concrete part of this section to the steel part of the section.

E. Steel-Fibre Reinforcement

The most straightforward means of imparting some of the tensile ductility that HSC lacks to the concrete matrix is to add steel fibres which bridge microcracks and maintain some serviceability after the matrix has cracked. Perceka and Liao conducted cyclic lateral loading tests on high-strength steel-fibre-reinforced concrete columns and found their columns had a more ductile behaviour in the post-cracking state than fibre-free companion columns, with a higher shear strength, and that shear strength was reasonably predicted by their proposed design expression after accounting for the fibre content explicitly [12]. This effect - increased capacity and more evenly distributed cracking with no reduction in ductility as observed for increased axial load and only higher-grade reinforcement - makes fibre addition one of the few

strategies in this review that does not reduce ductility in exchange for increased capacity.

F. Composite Confinement and Boundary Elements

If HSC columns are used as the boundary elements of shear walls, they can be sheathed by a steel tube (concrete-filled steel tube, CFST) to regain much of the ductility which a bare HSC boundary column would not have. Shi *et al.* studied composite shear walls with SFRHC webs and square CFST boundary columns, and found that 0.5–1.5% volume addition of steel fibre increased the peak force by 16–46%, and the displacement capacity by 21%, but reduced the energy dissipation by 3% compared to the conventionally reinforced shear wall, while the ultimate bearing capacity of the composite shear wall was 26% higher than that of the conventionally reinforced shear wall [14]. The same study also showed that increasing the shear span ratio was associated with an increase in ductility and energy absorption, and with a decrease in peak force, and a slightly lower ductility and near constant energy dissipation - again the same qualitative trend as found for axial load (Section IV.B.). In summary, in this work it is shown that there is a system-level trade-off between confinement and fibre reinforcement, which can replace some of the ductility lost due to concrete strength alone.

G. Cross-Study Synthesis

Figure 2 displays a single percentage change reported by each of four independent sources mentioned above, relative to the control or reference specimen of each source. The 4 %'s are not the same. The figure is not a pooled effect size, but rather the response quantity - lateral yield/ultimate load for the CRB735 comparison, displacement ductility coefficient for the Q690 comparison and peak lateral force for the two SFRHC/CFST/fibre comparisons - and the figure shows that reported single-parameter gains across this literature fell in a fairly tight range between 15 and 40 percent, whether the parameter changed was reinforcement grade, structural-steel grade, boundary confinement, or fibre content.

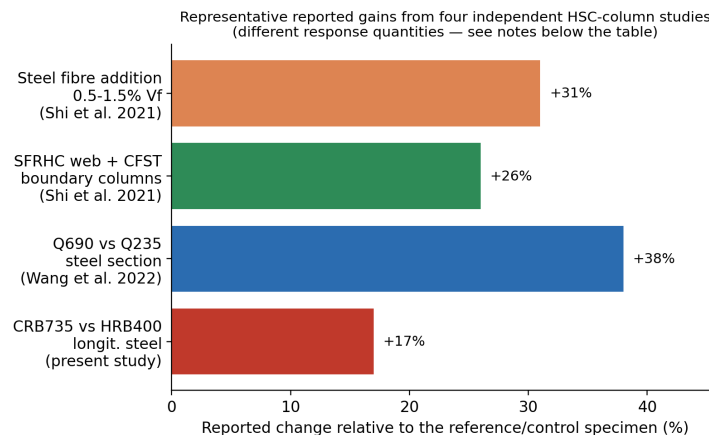


Fig.2 Representative Reported single-parameter Gains from four Independent HSC-column Studies, Normalized to each Study's Own Control Specimen. The Compared Quantities Differ (lateral load, ductility coefficient, or peak force - see Section IV) and the Figure Illustrates the range of Reported Effects Rather than a Pooled Statistic.

That clustering is itself instructive: it indicates that no one of these strategies is dramatically more efficient than the others, as measured at a single reported headline number, but that choosing between them in practice will depend on the ductility and constructability implications discussed in each of the above subsections.

V. CASE STUDY: LARGE-SCALE CYCLIC TEST ON 70 MPA COLUMNS

Several of the comparisons made in Sections IV and VI are based on a cyclic-loading experimental programme on large-scale high-strength concrete columns, which is reviewed in detail in this section. The main test variables of the programme were the shear-reinforcement ratio and the axial load ratio, while the shear behaviour, as well as the seismic performance of the HSC columns under double-curvature lateral cyclic loading with constant axial compression, was also investigated under a series of material and system-level variants such as longitudinal steel grade, stirrup spacing, etc. It is presented here as a case study in the review rather than a standalone document for several reasons: its presentation is in the same parametric terms as Section IV, and it is evaluated directly, in Section V.H, against other literature in the database.

A. Materials and Mix Design

The concrete used for the columns was cast with a nominal concrete grade of 70 MPa using a low water-to-cement ratio, with silica fume or slag to ensure a suitable concrete mix for the highly reinforced section [2]. The materials used for the longitudinal reinforcement comprised deformed high-strength steel bars with a reported yield strength of up to 600 MPa for the higher-grade type, and for the transverse reinforcement deformed bars were used to achieve the desired shear-reinforcement ratio, as described in Section V.E. Supplementary specimens in the wider test series

included fibre-reinforced polymer (FRP) sheets, alkali-resistant glass fibres and galvanized steel strapping for external strengthening and steel fibres for internal addition to the matrix material as described in Section IV.E.

B. Column Specimens and Test Setup

The cast column specimens were produced at two scales (small-scale (approx. 200 x 200 x 900 mm) and large-scale (approx. 400 x 400 x 1200 mm)), and square and circular cross-sections were both investigated throughout the programme. Figure 3 illustrates the sequence in which the large-scale specimens were fabricated: formwork and reinforcement for the foundation, foundation concrete placement, formwork and reinforcement of the columns (including strain-gauge instrumentation directly applied to the bars prior to casting) and the completed specimens ready for testing. The columns were tested in a heavy-duty reaction frame that could apply a constant axial compression by hydraulic jacks whilst a lateral incremental reversed cyclic displacement was applied at the column head by an actuator to simulate double curvature bending of a column framing into rigid beams above and below a storey.

C. Instrumentation

Strain gauges placed at critical sections on both longitudinal and transverse reinforcement to provide the strain history of the bars during cracking, yielding, and failure (Fig. 4), and linear variable differential transformers (LVDTs) measured axial and lateral deformations, with emphasis on the post-cracking stage, in which flexural and shear deformations start to separate. The axial and lateral forces applied were recorded during each test by load cells. It is a combination of instrumentation, strain gauges on the transverse steel, that give a test programme the ability to report whether the stirrups yielded at peak load or not, discussed further in Section V.E, and returned to later in Section VII.



Fig.3 Fabrication sequence for the large-scale Column Specimens: Foundation Formwork, Steel Reinforcement, Foundation Concrete Pour, Column Formwork, Column Concrete Pour, and Finished Specimens Prior to Testing.

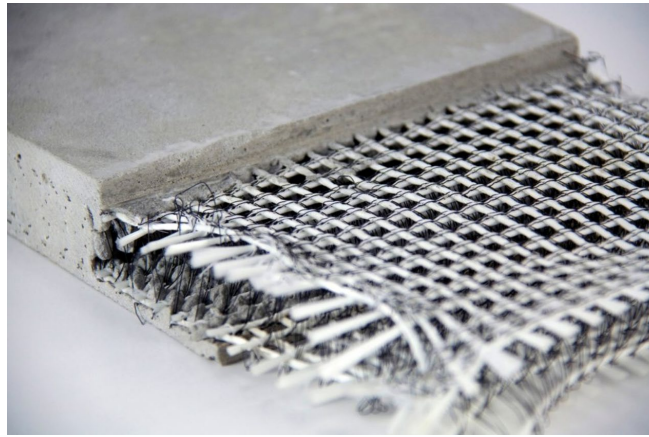


Fig.4 Representative Instrumentation Layout: Strain-Gauge Positions on Longitudinal Bars and Stirrups (Plan) and their as-Built Locations on a Reinforcement Cage Prior to Concrete Placement

D. Crack Patterns and Failure Mode

All the tested columns eventually reached their shear failure limit, and the failure sequence was generally observed to be flexural cracking near the ends of the columns at low drift, followed by the development and growth of diagonal cracking as the lateral load continued to increase and eventually a change in internal force distribution when diagonal cracking first occurred, thereby increasing column demand on the transverse reinforcement. The columns with the higher transverse-reinforcement ratio exhibited a delayed onset and increased width of the diagonal cracks and a higher drift at failure, as was also observed in Section IV. None of the configurations tested was able to transform the failure mode completely into a ductile, flexure-governed failure mode; shear failure occurred in all specimens, but was at a later drift and at a higher value in specimens with increasing amount of reinforcement.

E. Effect of Shear-Reinforcement Ratio

The shear strength increased and the point at which the transverse side reinforcement yielded was delayed with the increase in the ratio of transverse reinforcement. However, for the four columns that had the highest tested shear-reinforcement ratio (0.56%) three of the columns experienced the shear reinforcement yielding at the peak load; a reminder of the shear reinforcement for high-strength-reinforced columns in general as raised in Section IV.C [10], [11].

F. Effect of Longitudinal Steel Grade: CRB735 versus HRB400

The results of a direct comparison between longitudinally reinforced CRB735 high strength steel and companion columns reinforced with conventional HRB400 steel, otherwise identical in geometry, concrete grade and axial load ratio are presented.

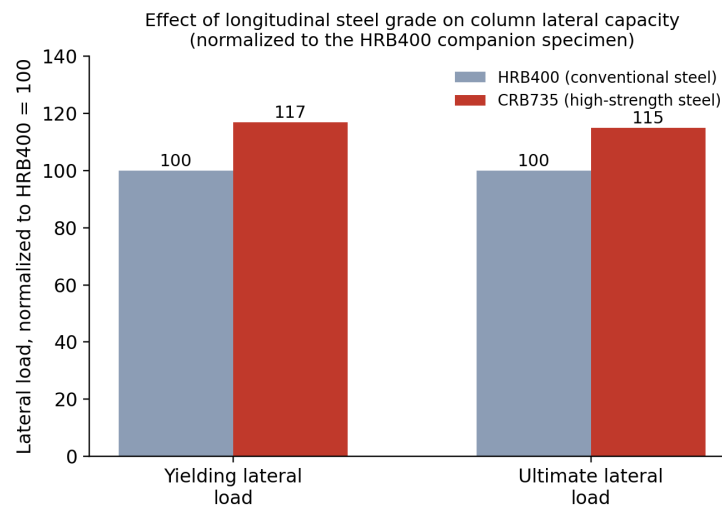


Fig.5 Effect of Longitudinal Reinforcement Grade on Column Lateral Capacity, Normalized to the HRB400 (conventional grade) Companion Specimen.

For the CRB735 columns, an increase of 17% in lateral yielding load and 15% in ultimate lateral load was observed when compared to HRB400 companions (Figure 5), whereas, for the HRB400 columns, a slight decrease in lateral yielding load when compared to higher grade columns was observed as was expected due to the decrease in steel yield strength. The reduction of ductility due to the increased strength of the CRB735 column was small, whilst the loss of stiffness with cycling was similar to the HRB400 column - therefore the improvement in strength due to the increased grade of longitudinal steel was greater than the loss of ductility compared to other increases in strength, such as an increase in the axial load ratio (Section V.G), which indicates that increasing the grade of the longitudinal steel is a relatively

efficient way of increasing the capacity without ductility being severely compromised [13].

G. Effect of Stirrup Spacing and Axial Load Ratio

The ductility and energy dissipation were lowered by about 5% as the stirrup spacing was increased from 50 mm to 100 mm. When the axial load ratio was increased from 0.3 to 0.5, the quantities were reduced by a similar amount (Figure 6) separately. This is a convenient practical rule of thumb: the loss of ductility to an increase in the ratio of the axial load was about the same as the loss of ductility with one stirrup spacing being doubled in this test programme; again, this is a rule of thumb and should not be extrapolated beyond the range of parameters tested.

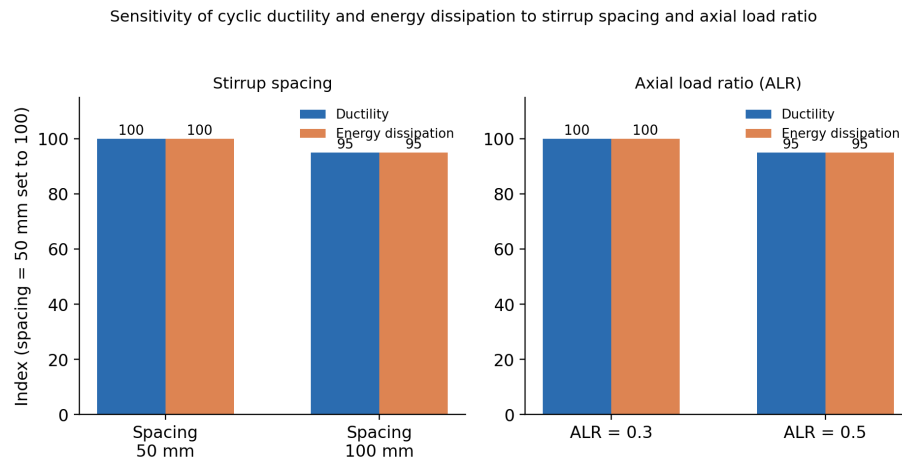


Fig.6 Sensitivity of Cyclic Ductility And Energy Dissipation to Stirrup Spacing (Left) and Axial Load Ratio (Right), Each Normalized to its Respective Lower-Demand Case.

H. Comparison with Code Equation and a Revised Expression

The shear capacities from this programme and those from previous literature on a total of 86 high-strength concrete columns were compared with three selected expressions for the concrete shear contribution V_c . More detailed expressions that include axial load through the maximum tensile-stress capacity of the section are given below in Equation (1) and Equation (2), respectively, and ACI 318 simplified expression in Equation (3) [1] below.

$$V_c = 0.29\sqrt{f'_c}b_w d \sqrt{1 + \frac{1.6N_u}{\sqrt{f'_c}b_w d}} \quad (\text{MPa}) \quad (1)$$

$$V_c = \frac{f_t(\text{max})}{F_2} b_w d \sqrt{1 + \frac{N_u}{f_t(\text{max})b_w d}} \quad (2)$$

$$V_c = 0.17 \left(1 + \frac{N_u}{13.8A_g} \right) \sqrt{f'_c} b_w d \quad (\text{MPa}) \quad (3)$$

where f'_c is the concrete cylinder compressive strength, b_w and d are the section width and effective depth, N_u is the axial compressive force, A_g is the gross cross-sectional area,

$f_t(\text{max})$ is a limiting concrete tensile stress and F_2 is a code-defined factor. To compare the responses of these three expressions to changes in axial load on a representative 70 MPa column section, Figure 7 shows this comparison for a realistic range of axial load ratio, not to duplicate the measured-versus-predicted comparison from the 86-column database. The test programme comparison showed that the ACI 318 minimum-shear-reinforcement requirement was not able to reliably predict shear failure across the 86-column set, and that the simplified ACI 318 expression was in general conservative as the more detailed expressions were non-conservative once the axial load was significant, which is contrary to the design logic due to the assumption that “more detailed” is “more accurate”. A modified expression including an explicit axial compression term was developed from this comparison and compared with the 86-column database, which gave a more consistent trend between predicted and measured shear strength across the range tested than did the code expressions above. The precise coefficients by which the expression needs to be revised in this test programme are not reproduced here, but the qualitative one that the axial load must be represented differently in the “simplified-but-conservative” expression, and in the “detailed-but-potentially-unconservative” expression, is the one that this review adopts for the discussion in Section VI.

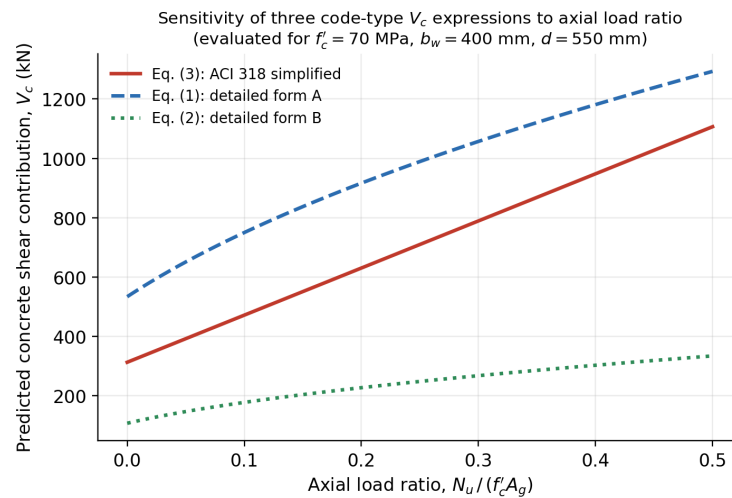


Fig.7 Sensitivity of the three V_c expressions of Eqs. (1)–(3) to axial load ratio, evaluated for a representative 70 MPa column section ($b_w = 400$ mm, $d = 550$ mm). The expressions diverge increasingly as axial load ratio rises, illustrating why “simplified” and “detailed” code expressions can rank differently once axial load becomes significant.

I. Related Findings from the Wider Literature Connected to this Programme

The broad test series of which this case study is a part also included some related systems evaluated separately in Section IV: steel-fibre-reinforced HSC columns and their proposed shear-strength equations [12]; composite shear walls with steel-fibre-reinforced HSC webs and CFST boundary columns, which exhibited improved seismic performance attributed to the confinement effects of the CFST boundary columns [14]; and the seismic performance of HSC columns using recycled aggregates, which recent literature has demonstrated can be very close to that of ordinary HSC columns at similar axial compression ratios [13], indicating the potential for this materially sustainable option to be used for earthquake-resistant HSC columns without a significant seismic-performance penalty. These threads are not presented here as further independent findings of the current case study, but are referred to as connections, and a reader who is interested in any of these should read the specific reference cited in Section IV.

VI. COMPARISON WITH CODE PREDICTION AND DISCUSSION

In looking at all three sections together (III–V), three points should be noted. First, all the models discussed (ACI 318's simplified expression, UCSD, Sezen–Moehle, and data-driven models) assume that axial load is beneficial for shear strength, and experimental data from Sections IV and V confirm this at the level of peak capacity. However, capacity is not the only quantity that is important to seismic performance; every one of the same studies that reported a ductility or an energy dissipation measure along with an axial load or reinforcement grade also reported a reduction in that measure as the axial load or reinforcement grade increased (Sections IV.B, IV.D, IV.F, V.F, V.G). Thus, a shear-strength check that passes at a higher axial load can still be a less safe

column seismically, because the column will have less ability to redistribute load or dissipate energy when it does crack.

Second, the case study result that a simplified code expression can be safe while a more detailed expression is not (Section V.H) is a valuable reminder that the safety of the model expression is not necessarily correlated with the sophistication of the expression. The reason is illustrated in the three-equation comparison in Figure 7, where the expressions that pass through similar values at zero axial load can differ by a factor of two or more when the axial-load ratio approaches 0.4–0.5, just due to the structure of their axial-load term. Therefore, none of the proposed models in Table I should be assumed accurate when extrapolating beyond the test data; they all should be checked against test data in the range of axial loads of interest.

Third, the minimum shear-reinforcement requirements based on normal-strength, normal-grade data are not necessarily applicable to high-strength concrete with high-strength transverse reinforcement as the actual quantity of steel that provides protection against brittle shear failure is not the area of steel, but rather whether that steel yields before the column loses capacity [10], [11]. It is apparent from the case study itself that even with a shear-reinforcement ratio as high as 0.56% (which is greater than most code minimums), the shear-reinforcement level may be extremely close to the limit, as illustrated by the fact that in three of four otherwise identical columns, stirrup yielding occurred at peak load, but not the fourth (Section V.E), suggesting a case for the reporting change proposed below.

VII. RESEARCH GAPS AND A MINIMUM REPORTING SET

In all the studies examined in Sections III–V, three gaps appear. Ductility or energy-dissipation measures are much less consistently reported than peak load, and thus safety-

relevant comparisons are easier to make than strength-relevant comparisons. Even in studies that explicitly measured peak load, such as the case study in Section V, it is not often reported whether the transverse reinforcement yielded at peak load – the one parameter of importance about the use of the V_s term in ACI 318. Comparisons to code or analytical shear models are typically only done to one model, most often ACI 318 and as per Section VI, this is insufficient to determine if a specific design would be safe with respect to all the models in Table I. No new test is required for any

of the three additions in Table II; they are all quantities already measured by instrumentation that is normal to a cyclic column test (strain gauges on the transverse reinforcement, LVDTs at the column head and a load cell). The barrier is not experimental – as it was for the analogous set of minimum reporting for flexural strengthening studies in related review work – but is editorial, and the use of even the first two items in Table II would enable future HSC-column shear studies to be compared and eventually pooled on both strength and ductility rather than on strength alone.

TABLE II PROPOSED MINIMUM REPORTING ITEMS FOR HSC COLUMN SHEAR/SEISMIC STUDIES

Reporting item	Why it matters	Typical current practice
Whether transverse reinforcement reached yield at peak load	Directly tests the validity of the truss (V_s) assumption behind every model in Table I	Rarely reported explicitly, even when strain-gauge data exist to answer it
A displacement-ductility or energy-dissipation measure, alongside peak load	Without it, a reported strength gain cannot be checked against the ductility cost documented throughout Section IV	Peak/ultimate load reported far more often than ductility or energy dissipation
Comparison against at least two independent shear models (e.g., one code-simplified and one detailed/ductility-dependent model)	A single-model comparison cannot reveal the kind of divergence shown in Fig. 7	Most studies compare against a single code provision, usually ACI 318

VIII. CONCLUSION

It is shearing behaviour and not compressive strength that is likely to be the most important factor in limiting the ductile behaviour of an HSC column, since high-strength concrete has greater compressive capacity than tensile or shear capacity. The trade-off is the same for all the design-code and analytical models summarized here (Section III), the experimental literature synthesized by parameter (Section IV) and the detailed case study of a 70 MPa cyclic-loading test programme (Section V): increases in the shear strength due to an increase in axial load, reinforcement grade and a reduction in the quantity of transverse reinforcement are all accompanied by reductions in ductility and energy dissipation, while no code expression reviewed here treats that trade-off explicitly, predicting shear strength only. Two methods reviewed here yielded enhanced capacity without the traditional ductility reduction: steel-fibre reinforcement and CFST-type composite confinement, and their use is recommended, particularly when the ductility aspect of earthquake response is a controlling parameter in design rather than just strength. The simplified code expressions used in the case study were found to be more conservative than the more detailed expressions at high axial load ratio (Section V.H, Figure 7), which is a warning against assuming that a more detailed expression is necessarily more conservative; the case study also found that a relatively high, code-satisfying transverse-reinforcement ratio did not guarantee yielding of the stirrups in every specimen (Section V.E), which is a warning against assuming that a more detailed expression of minimum reinforcement compliance is equivalent to ductile behavior. The relatively small changes to the reporting requirements suggested in Section VII – which include specifying if any transverse reinforcement has yielded and reporting a ductility or energy-dissipation measure in addition to the peak load – would allow comparison and, ultimately, pooling of future generations of

HSC-column shear studies on the basis that actually dictates the safety of the structure in a seismic event.

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REFERENCES

- [1] ACI Committee 318, Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary. Farmington Hills, MI, USA: American Concrete Institute, 2019.
- [2] ACI Committee 363, State-of-the-Art Report on High-Strength Concrete (ACI 363R-92). Farmington Hills, MI, USA: American Concrete Institute, 1992.
- [3] ACI-ASCE Committee 445, Recent Approaches to Shear Design of Structural Concrete (ACI 445R-99). Farmington Hills, MI, USA: American Concrete Institute, 1999 (reapproved 2009).
- [4] M. J. N. Priestley, R. Verma, and Y. Xiao, "Seismic shear strength of reinforced concrete columns," *Journal of Structural Engineering*, vol. 120, no. 8, pp. 2310–2329, 1994.
- [5] M. J. Kowalsky and M. J. N. Priestley, "Improved analytical model for shear strength of circular reinforced concrete columns in seismic regions," *ACI Structural Journal*, vol. 97, no. 3, pp. 388–396, 2000.
- [6] H. Sezen and J. P. Moehle, "Shear strength model for lightly reinforced concrete columns," *Journal of Structural Engineering*, vol. 130, no. 11, pp. 1692–1703, 2004.
- [7] Y. Xiao and A. Martirosyan, "Seismic performance of high-strength concrete columns," *Journal of Structural Engineering*, vol. 124, no. 3, pp. 241–251, 1998.
- [8] C. Jin, Z. Pan, S. Meng, and Z. Qiao, "Seismic behavior of shear-critical reinforced high-strength concrete columns," *Journal of Structural Engineering*, vol. 141, no. 8, art. 04014198, 2015.

- [9] C. T. N. Tran and B. Li, "Shear strength model for reinforced concrete columns with low transverse reinforcement ratios," *Advances in Structural Engineering*, vol. 17, no. 10, pp. 1373–1385, 2014.
- [10] D. Sokoli and W. M. Ghannoum, "High-strength reinforcement in columns under high shear stresses," *ACI Structural Journal*, vol. 113, no. 3, 2016.
- [11] Y.-C. Ou, H. Alrasyid, and N. V. B. Nguyen, "Minimum shear reinforcement for columns with high-strength reinforcement and concrete," *Journal of Structural Engineering*, vol. 147, no. 2, 2021.
- [12] W. Perceka and W.-C. Liao, "Experimental study of shear behavior of high-strength steel fibre-reinforced concrete columns," *Engineering Structures*, vol. 240, art. 112329, 2021.
- [13] J. Wang, X. Yi, Q. Liu, and X. Fang, "Seismic performance of steel-reinforced concrete columns with Q690 high-strength steel," *Materials*, vol. 15, no. 9, art. 2979, 2022.
- [14] K. Shi, M. Zhang, P. Li, R. Xue, P. You, T. Zhang, and B. Cui, "Seismic behavior of steel-fibre-reinforced high-strength concrete shear wall with CFST columns: Experimental investigation," *Fibers*, vol. 9, no. 11, art. 75, 2021.
- [15] V.-H. Nguyen, N.-M. Pham, T. Vo, and D.-D. Nguyen, "Assessment of shear strength models of reinforced concrete columns," *Engineering, Technology & Applied Science Research*, vol. 12, no. 6, 2022.
- [16] M. R. Azadi Kakavand, H. Sezen, and E. Taciroglu, "Data-driven models for predicting the shear strength of rectangular and circular reinforced concrete columns," *Journal of Structural Engineering*, vol. 147, no. 1, art. 04020301, 2021.
- [17] A. I. Ioannou, D. Galbraith, N. Bakas, G. Markou, and J. Bellos, "New predictive models for the computation of reinforced concrete column shear strength," *Computers*, vol. 14, no. 1, art. 2, 2024.