



Review Article

Seismic Behaviour of High-Strength Concrete Columns Reinforced with High-Strength Steel Bars: A Systematic Review of Experimental Evidence, Design Provisions and Research Gaps

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Abstract

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The combination of high-strength concrete (HSC) and high-strength steel bars (HSSB) is becoming more popular. Taller buildings with columns, rather than a flat roof, allow smaller sections, less material, and more usable floor area. Their seismic performance is based on an unresolved tension; however, the brittleness of HSC and the delayed yielding of HSSB adversely affect the ductility assumed by capacity design. This article reviews the experimental and numerical evidence on the cyclic behavior of such columns. Initially, 486 studies were identified, with 62 studies retained between 1995 and 2025. This led to 24 full-scale campaigns that reached parameter synthesis across six thematic lines. Three findings are repeated in separate programmes. High-strength stirrups usually fail to meet nominal yield at peak lateral load; crediting that yield strength overestimates shear capacity and ductility. The axial load ratio controls deformation. Strength exceeds the concrete grade: well-confined specimens easily surpass it. Displacement ductility decreases to 4 at a moderate axial load, then weakens further at a ratio greater than approximately 0.4; failure shifts to compression-controlled conditions. Confined-concrete models calibrated concentrically are not well suited for transfer to members in flexure if the strain gradient is not represented explicitly. The reported failure drifts are well above current code limits, which indicates that those limits are conservative for well-detailed members but provide little protection where member details are lacking and confinement is marginal. The review identifies a strain-compatibility formulation of effective stirrup stress, full-scale testing at high strength and high axial load, gradient-aware constitutive models, and the coupling of seismic with life-cycle objectives.

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I. INTRODUCTION

A. Background

Concretes with a cylinder compression strength greater than about 40 MPa are usually considered high-strength concrete (HSC) [1]. It has been widely used in structural engineering, and its use has been steadily increasing. The axial load on a stronger material is the same, which provides a simple economic logic for using a smaller section; this reduces self-weight and foundation demand, releases usable floor area in the lower storeys of tall buildings, and reduces the amount of

cement-bound material placed on site. Meanwhile, high-strength steel bars (HSSB) with nominal yield strengths of 600 MPa and above are now commercially available in various markets; hence, the moment that might occur at the corner of the building and the street, which might otherwise be caused at the corner of the building and the street, will be relieved by reducing the area to be reinforced in proportion. Heavily reinforced columns using the combination of the two materials are thus natural, and this combination is now common in the lower storeys of high-rise construction in East Asia. The geoseismic aspects of that combination are not as simple; HSC is more powerful, yet more brittle: it has a

steeper post-peak descending branch, lateral dilation under axial compression is smaller, and the pressure mobilized in transverse reinforcement at a specified axial strain is correspondingly lower [2]–[4]. Confinement, the mechanism on which the ductility of a seismic column ultimately depends, is less effective when it is most needed to deal with brittleness. HSSB provides a second, somewhat autonomous difficulty. A higher-grade bar reaches its yield strain at a greater strain; therefore, the transverse reinforcement is stronger. The transverse reinforcement is stronger because a higher-grade bar reaches its yield strain at a greater strain.

HSSB may still respond elastically even after the confined core has started to respond. However, it degrades, which means that, in design, the nominal yield strength is never attained [5], [6]. Both effects act in the same direction: strength is real and immediate, whereas the ductility penalty is subtle, dependent on the loading path, and observable only under reversed cyclic loading. Design codes have responded with guarded and mixed reactions. HRB600 is allowed in GB 50010 in China; it can be used as longitudinal reinforcement, but it cannot be used for seismic transverse reinforcement. Its application to seismic frames is currently considered to have insufficient evidence [6], [26].

ACI 318-19 allows Grade 550 longitudinal bars in special seismic systems and Grade 690 transverse bars, with additional conditions on the stress-strain properties of confinement bars [28], [33]. Both NZS 3101 and Eurocode 8 are even more conservative. The practical consequence is that a material combination that is already being built is governed by provisions that various jurisdictions support through various, sometimes mutually exclusive, interpretations of the same experimental literature.

B. Scope and Research Questions

The scope and research questions are covered in Section B. Although many years of cyclic test data have been built up for HSC columns, the data are fragmented. The number of parameters varied in individual campaigns is usually two or three, with five to nine specimens per campaign; loading techniques vary, including incremental loading and cycles per specimen amplitude; deformation capacity is reported using different definitions of yield/failure points; and these definitions are often not mutually consistent. As a result, it is not easy to compare studies, and readers of a particular paper cannot easily judge whether a reported trend is general or specific to that test series. What is missing is not so much more data as a formal narrative of what the existing data support; where codes thin out, it is needed most. This review fills this gap. It is structured to answer four questions:

1. What are the design parameters that influence the cyclic behavior of HSC columns containing HSSB, and what is the

consistency of each of these effects across independent test programmes?

2. If the nominal yield strength of the high-strength transverse reinforcement is mobilized, what is its value at peak lateral load, and what is the implication for shear and confinement design if it is not?

3. How well do confined-concrete constitutive models used in finite element practice perform when transferred from concentric calibration to flexural members that exhibit a significant strain gradient?

4. What untested parameter space exists, and what test, if any, should be conducted to fill it? What would be the most efficient programme to fill it?

This article's contribution is a synthesis; it is not an experiment. Three products are a parameter-coverage matrix that allows parameters to be varied so the model's parameter space is covered, a parameter-space exploration matrix, and the identification of combinations of concrete grade, reinforcement grade, axial load ratio, and detailing required to meet the specified requirements. The protocols have now been applied to full-scale testing, with a comparative review of code provisions against the test results. An experimental record supports them, and a structured map connects each gap to a specific, practical line of future work. Section II outlines the review methodology. Section III surveys the current state of the art thematically. Section IV synthesizes the evidence and discusses its consequences for design. The conclusions are given in Section V and the limitations in Section VI. The review is presented together with the recommended research programme.

II. METHODOLOGY

The review followed a structured protocol adapted from PRISMA guidance, modified for an engineering literature in which controlled trials and effect sizes have no direct analogue. The protocol has three stages: identification of candidate records, screening against explicit eligibility criteria, and structured extraction of design parameters and reported outcomes.

A. Search Strategy

Five databases were searched: Web of Science Core Collection, Scopus, ScienceDirect, the ASCE Library and CNKI. CNKI was included deliberately because a substantial fraction of full-scale testing on 600 MPa reinforcement has been conducted in China and reported first in Chinese-language journals; excluding it would have introduced a systematic bias in the region of the parameter domain this review is most concerned with. The search covered publications from 1995 to 2025. The search strings combined three concept blocks with Boolean operators, as set out in [Table I](#).

TABLE I SEARCH CONCEPT BLOCKS AND REPRESENTATIVE QUERY STRINGS

Concept	Search Category	Keywords/Search Terms
A	Material	"High-strength concrete"; "high strength concrete"; HSC; "high-performance concrete";
B	Reinforcement	"High-strength steel bar"; HSSB; HRB600; HRB500; "Grade 80 reinforcement"; "high-strength stirrup";
C	Response	seismic; cyclic; "quasi-static"; hysteretic; ductility; "drift capacity"; confinement

Reference lists of all retained articles were hand-searched for records the database queries had missed, and forward citation tracking was applied to the most frequently cited campaigns. This snowballing step added eleven records that the Boolean search had not returned, most of them older studies indexed under terminology that predates the current vocabulary.

B. Eligibility Criteria

Records were screened first on title and abstract, then on full text. Table II gives the criteria. The two most consequential exclusions were the strength floor and the scale requirement. Studies on concrete below 50 MPa cube strength were

excluded because they characterize a materially different post-peak response; studies reporting only heavily scaled specimens were excluded from the quantitative synthesis, though retained for qualitative discussion, because size effects on confinement efficiency and on bond are known to be significant and are not dependably removed by similitude scaling.

Figure 1 summarizes the flow of records through the review. Of 486 records identified, 341 remained after duplicate removal, 173 were assessed at full text, 62 were retained for qualitative synthesis, and 24 met the additional requirements for quantitative parameter synthesis.

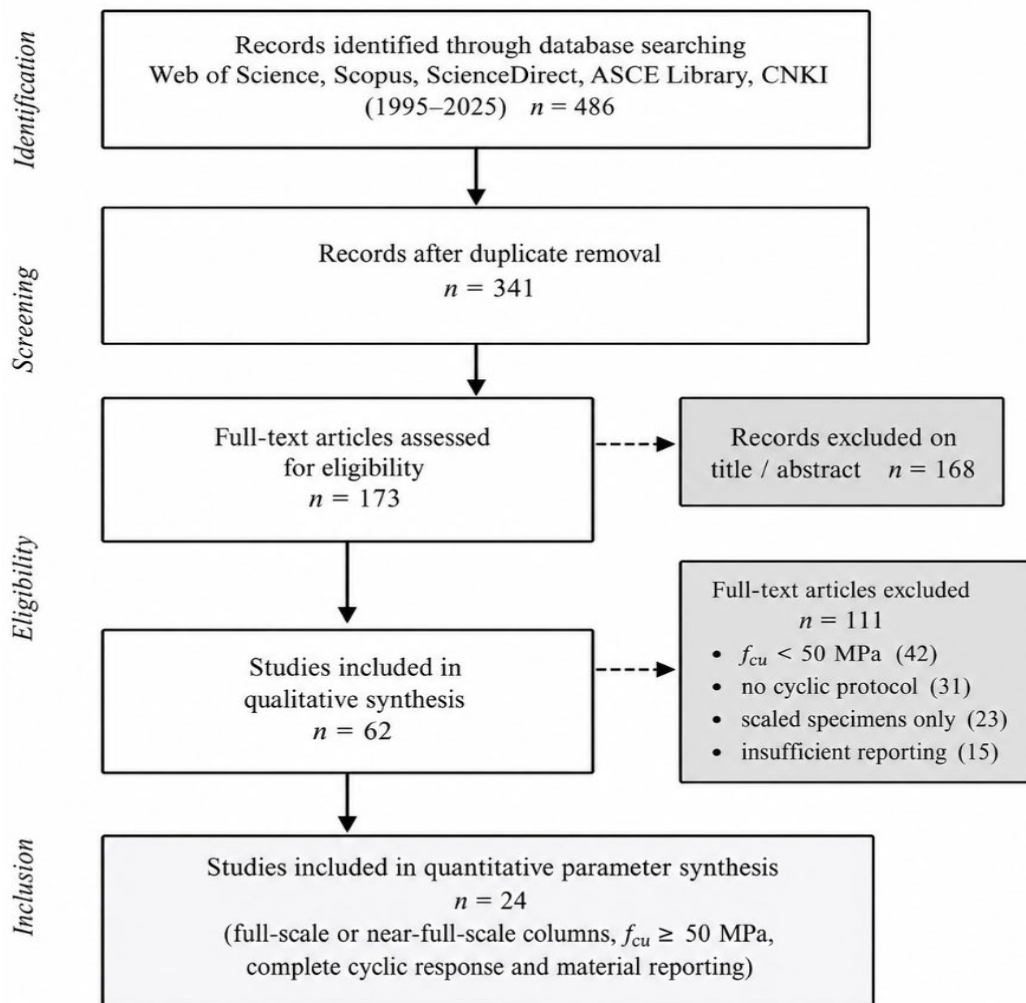


Fig.1 Flow of Records Through Identification, Screening, Eligibility Assessment and Inclusion

TABLE II ELIGIBILITY CRITERIA APPLIED AT FULL-TEXT SCREENING

Criterion	Inclusion Criteria	Exclusion Criteria
Concrete Strength	Cube compressive strength ≥ 50 MPa (or cylinder ≥ 40 MPa)	Normal-strength concrete only
Loading Protocol	Reversed cyclic or quasi-static cyclic lateral loading	Monotonic or purely axial loading only
Structural Member	Column or column-type member (including CFST, composite, semi-precast)	Beams, slabs, or joint sub-assemblies without column response reported
Specimen Scale	Full-scale or near-full-scale specimens (quantitative synthesis)	Scale factor below approximately 1:2 (quantitative synthesis only)
Data Availability	Material properties and loading protocol reported in sufficient detail	Incomplete reporting preventing parameter extraction
Publication Type	Peer-reviewed journal article, or code/standard document	Conference abstracts, theses superseded by journal papers, non-peer-reviewed reports

C. Data Extraction and Synthesis

For each retained study, the following were extracted where reported: specimen count and geometry; shear span-to-depth ratio; cube and cylinder compressive strength; longitudinal and transverse reinforcement grade, diameter, ratio and spacing; axial load ratio; loading protocol; observed failure mode; and the yield, peak and failure points of the lateral load-displacement envelope together with the definitions used to locate them. The definition of the yield point proved to be the single largest source of incomparability. Some authors use the energy-equivalent construction of Park [30], others a first-yield or a secant-stiffness definition, and the resulting displacement ductility values are not interchangeable. Where the definition was not stated, the study was excluded from quantitative synthesis. Because loading methods, yield definitions and instrumentation differ across the corpus, no attempt was made to pool numerical outcomes into a single meta-analytic estimate. Doing so would impose a false precision on data that were never generated under comparable conditions. The synthesis is

instead directional: a parameter effect is reported as established where it appears with the same sign in independent campaigns using different specimens, protocols and laboratories, and as unresolved where campaigns disagree or where too few studies address it.

III. STATE OF THE ART

Figure 2 plots the corpus against the six themes into which the data were organised for this section. Before the detailed discussion, two features should be noted. The first concerns HSSB work. The evidence base is largely limited to post-2019, and all work on longitudinal reinforcement occurs after 2019. The evidence supporting current code provisions on 600 MPa bars is still young. Second, composite, CFST, and semi-precast systems have been studied continually since approximately 2012, mainly as a separate literature. Although there was limited cross-referencing between these articles and the reinforced-concrete column work, a considerable amount of literature separately addresses overlapping mechanics.

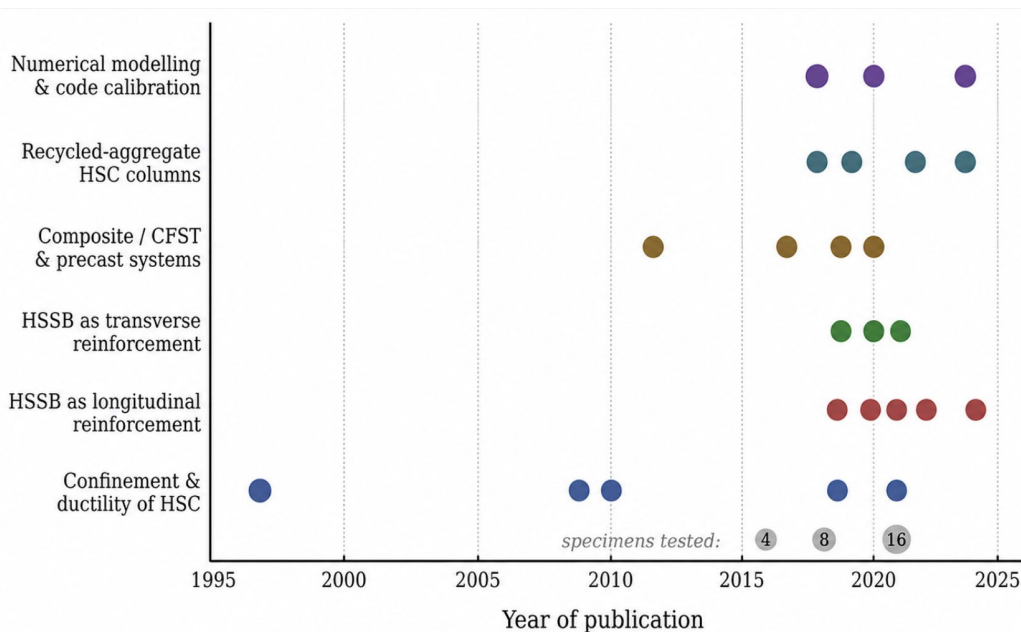


Fig.2 Chronological and Thematic Map of the Reviewed Corpus. Marker area is Proportional to the Number of Specimens Tested in Each Campaign

A. Mechanical Characteristics of HSC Relevant to Cyclic Response

The characteristics of HSC that make it different from normal-strength concrete under seismic loading are not only a matter of quantity but also of quality. The interfacial transition zone thickness decreases as compressive strength increases; the mortar or concrete strength is less than the aggregate strength, so cracks develop. When cracks pass through the coarse aggregate, the resulting fractured surface is smoother; shear transfer across an aggregate interlock between the cracks decreases, and the open flexural crack falls. Meanwhile, the descending part of the uniaxial compressive expansion is often observed and disappears. Lateral expansion is often observed with axial compression,

but it disappears as the response steepens; this lateral expansion is what actually mobilizes hoop pressure in the transverse reinforcement, and it diminishes [1], [4], [31], [32]. Figure 3(b) shows the consequence, with Mander-type confined responses [29] computed for representative normal- and high-strength cases. The closed HSC curve goes to a significantly higher peak. Nevertheless, it has a greater drop-off after peak degradation and a lower residual capacity at a given strain than the normal-strength confined case. The unconfined HSC response shown for reference has very little capacity after the peak because spalling on HSC columns is generally not progressive, and the reason for the abrupt transition is not explained. Transitions from cover to core resistance are an important step in the observed failure sequence.

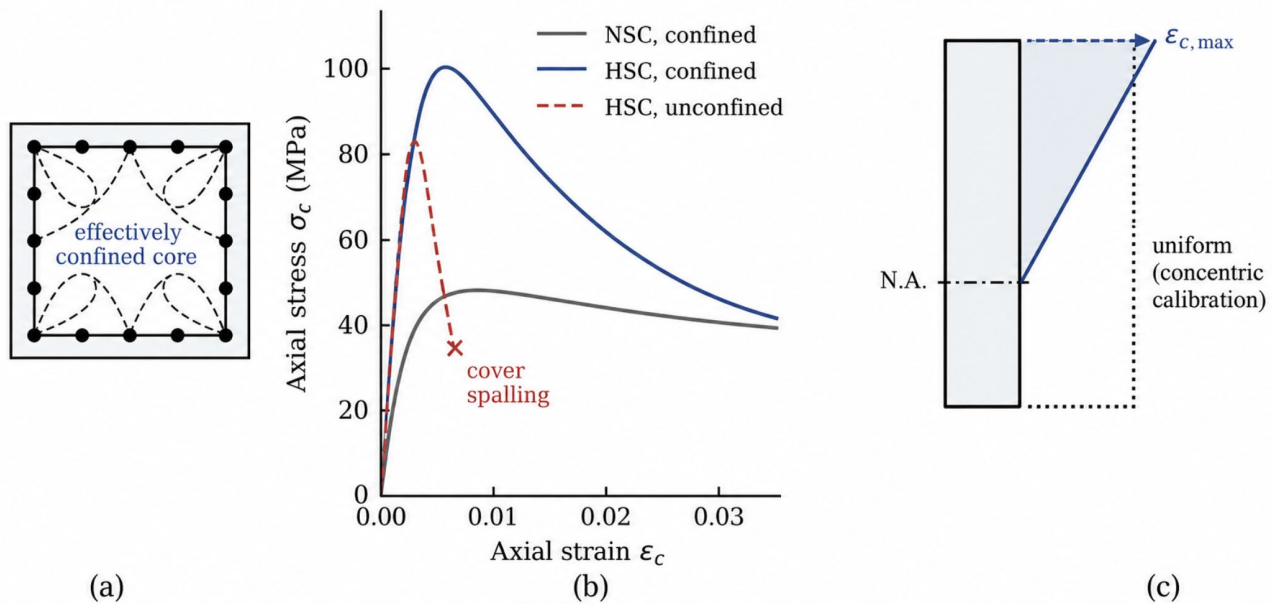


Fig.3 Mechanics Governing the Cyclic Response of HSC Columns: (a) Effectively Confined Core and the Arching Action Between Restrained Longitudinal Bars; (b) Mander-type Confined and Unconfined Uniaxial Responses for Representative Normal-Strength and High-strength Cases; (c) Flexural Strain Gradient Across the Section, Contrasted with the Uniform Strain Field Under Which Confinement Models are Usually Calibrated.

B. Confinement and Ductility

A column arches between lateral restraints. As illustrated schematically in Figure 3(a), longitudinal bars are used. The effectively contained core is undersized; this area is larger than the area of the perimeter tie, and this difference increases as the distance between ties increases and as the number of restrained longitudinal bars decreases. Bayrak and Sheikh [2] established early that the spacing, size, and shape of transverse reinforcement determine the deformation capacity of HSC columns more strongly than concrete grade does, and that provisions calibrated on normal-strength concrete were not conservative for transfer to HSC. It was further found by Paultre *et al.* [3] that ductility in circular HSC columns was recovered at volumetric transverse reinforcement ratios well above the standards for members with the same normal strength. Ho *et al.* [4] directly studied the efficiency of that recovery. Later literature shows substantial recurrence of this finding: adding confinement enhances the ductility of HSC,

but with diminishing returns, since less lateral dilation of the stronger material means less additional transverse steel is mobilized. That is, confinement by itself is a practical way to generate profit. While this ceiling is lower than the HSC values, it is reached at reinforcement ratios that again start to introduce congestion. Materials that could be used for performance were used to avoid. Both Xue *et al.* [14] and Yang *et al.* [5] report that close-packed spirals are effective in extreme abundance, while rectangular spirals and widely spaced spirals are effective in extreme scarcity. Detailed compound stirrup arrangements outperform conventional layouts at the same volumetric ratio.

C. High-strength Steel Bars as Longitudinal Reinforcement

Evidence for longitudinal reinforcement is fairly consistent and encouraging for HSSB. Under an equal-strength design basis, in which a smaller area of higher-grade steel is used for a larger area of HSSB, columns provide nominal flexural

capacity at a lower steel material consumption than conventional steel. They provide buildable lateral load capacity and deformation capacity that is generally as wide as traditionally reinforced counterparts [6], [7], [34]. Alaei *et al.* [7] tested mixed-grade longitudinal arrangements and found that drift capacity was maintained; Rautenberg *et al.* [34] obtained a similar result. They reached the same conclusion for 550 MPa reinforcement under cyclic loading. Koziński and Winnicki [8] extended the result to slender columns in biaxial bending and reported that less HSSB gave the same flexural strength and deformability. This consensus addresses crack control and residual deformation rather than strength. Since higher-grade bars will have a greater design stress at a higher strain, cracks are wider at a given load, and the remaining crack opening after the load is removed is larger. If durability or serviceability is the deciding factor, this can be the binding constraint even when seismic performance is satisfactory. Studies of ultra-high-strength and very high-strength steel are underway; weakly bonded reinforcement [12], [13] has explored deliberately decoupling bond to exploit the wider crack problem, providing a larger elastic strain range for self-centering. The downside of the centering mechanism is that it causes more damage to the concrete in the hinge region, but there it is.

D. High-strength Steel Bars as Transverse Reinforcement

This is where the literature is least comfortable and where the practical stakes are highest. Several independent programmes report that high-strength stirrups do not reach their nominal yield stress at the instant of peak lateral load [5], [6], [11]. The mechanism is simple: confinement is mobilized by the lateral dilation of the core; dilation in HSC is small, and a 600 MPa bar requires roughly fifty percent more strain to yield than a 400 MPa bar. The stirrup is therefore still in its elastic range when the core has reached the strain at which it begins to shed capacity. The design consequence is explicit and, in this review's view, insufficiently reflected in current practice. Shear design equations and confinement provisions alike credit the nominal yield strength of the transverse reinforcement. If the stress actually developed at peak load is materially below that value, both the calculated shear capacity and the calculated confinement effectiveness are overestimated, and the overestimate is unconservative in exactly the member type where brittle failure is the governing concern.

Zhang *et al.* [6] recommend, based on full-scale testing, that the stress assumed for HRB600 transverse reinforcement be held below its yield strength, and report that HRB400 stirrups achieved better composite action with C80 and C95 concrete than HRB600 stirrups did. Hou *et al.* [11] and Yang *et al.* [5] report consistent observations, with the latter proposing a confinement reinforcement design procedure that clearly accounts for the reduced mobilization. This is not a marginal correction. A stirrup that develops seventy percent of its nominal yield stress delivers seventy percent of the assumed enclosing pressure, and the ductility consequence of that

shortfall compounds with axial load ratio in the manner discussed in Section IV.

E. Composite, Concrete-filled Steel Tube and Semi-precast Systems

A parallel literature has pursued the same objective, adequate ductility in high-strength members, by changing the confinement method rather than the reinforcement grade. Concrete-filled steel tube members and steel tube-reinforced composite columns confine the core continuously rather than at discrete tie locations, removing the arching inefficiency illustrated in Figure 3(a) and substantially improving post-peak retention [9], [10], [23]. Zhang *et al.* [10] report that steel tube-reinforced HSC columns with HSSB achieved strength, stiffness and ductility superior to traditionally confined equivalents; Yin *et al.* [23] extended the approach to special-shaped multicell sections. Qiao *et al.* [9] examined the exposed column base condition, which is frequently the vital detail in such systems. Semi-precast columns, comprising a cast-in-situ core within a precast external shell, address constructability rather than confinement directly; still, the interface between shell and core can create a potential plane of weakness under reversed cycles. Xiao *et al.* [20] found seismic performance comparable to cast-in-situ columns, with slightly reduced ductility and initial stiffness, and reported that externally precast arrangements outperformed internally precast ones. Kim *et al.* [21] and Chuang *et al.* [22] reported equivalent or better performance for their respective fabrication methods. Zhang *et al.* [19] found that penetrating shear reinforcement across the interface, together with core diameter, had limited influence on seismic performance provided the interface was detailed to transfer shear, with all specimens exceeding a three percent ultimate drift ratio.

F. Recycled-aggregate High-strength Concrete Columns

Recycled coarse aggregate is a strong, sustainable aggregate. In 2019, demand for sand and gravel was estimated at around 40 billion tonnes per year. This is three times what it was 20 years ago and is projected to reach 60 billion tonnes by 2030 [15]. The volume of construction and demolition waste has followed the same trend, with crushed concrete being reused. Concrete is not only a supply constraint but also a disposal problem. From a mechanical point of view, recycled coarse aggregate is accompanied by old mortar attached to it and internal cracks. Crushing lowers the crushing index and stiffness compared with natural aggregate [16], [17]. The elastic modulus and tensile strength are both constant; as the replacement ratio increases, so too does the decline, as with acid-etching, mechanical treatment, thermal, and other processes.

Quenching or changing the mixture proportions can reduce the deficit but not eliminate it [17]. Under cyclic loads, Saribas *et al.* [18] found that full-scale columns with a high proportion of recycled aggregate performed comparably to natural-aggregate equivalents when axial load ratio and

confinement were matched, and Zhang *et al.* [19], [24] reached similar conclusions for high-strength recycled-aggregate columns with high-strength reinforcement. The literature on high-strength recycled-aggregate columns specifically remains thinner than that on low- and medium-strength members, and the interaction between replacement ratio and confinement demand at high axial load is not well characterized.

G. Numerical Modelling and Code Calibration

Finite element modelling in this field is used less for prediction than for parameter extension: a model is validated against a small test series and then exercised across a parameter range that testing cannot economically cover. The validity of that extension depends entirely on whether the constitutive model transfers beyond its calibration conditions. Figure 3(c) illustrates the recurring difficulty. Confined-concrete models in general use, including the widely adopted Mander formulation [29] and its high-strength refinements [31], [32], are calibrated predominantly on concentrically loaded specimens in which the axial strain field is essentially uniform. A column in flexure has a pronounced strain gradient across its section: the extreme compression fiber reaches high strain. At the same time, the material near the neutral axis remains lightly strained, and the resulting confinement demand and post-peak behaviour differ from the concentric case.

Zhang *et al.* [6] addressed this by developing a compressive stress model that incorporates the influence of the flexural strain gradient on the confined core, and report agreement with measured yield, peak, and failure points within approximately fifteen percent. Koziński and Winnicki [8] reached a similar conclusion for slender columns in biaxial bending, and Huang *et al.* [25] extended validated modelling to the post-fire condition. The wider point is that models that do not represent the strain gradient should be expected to misestimate deformation capacity, and that this error is systematic rather than random.

IV. SYNTHESIS AND DISCUSSION

A. What Has Actually Been Tested?

Figure 4 presents the parameter-coverage matrix for the 24 campaigns retained in quantitative synthesis. The matrix is deliberately coarse, distinguishing only whether each study addressed a given parameter fully, partially or not at all, but the pattern it exposes is insightful. Full-scale testing, cyclic protocols and axial load ratio variation are well represented. Coverage of high-strength transverse reinforcement is markedly sparser than coverage of high-strength longitudinal reinforcement, and explicit assessment against code provisions is inconsistent.

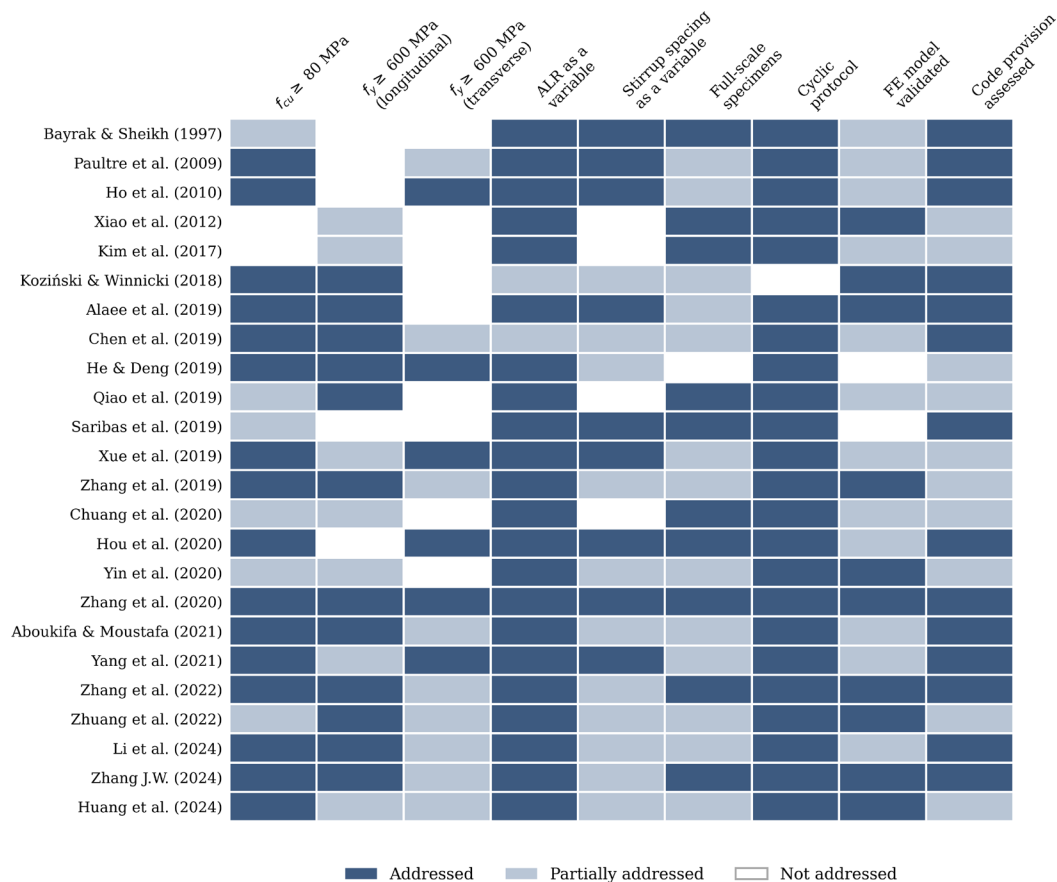


Fig.4 Parameter-Coverage Matrix for the Campaigns Retained in Quantitative Synthesis. A Parameter is Recorded as Addressed Where it was Varied Systematically and Reported, and as Partially Addressed Where it was Present but Held Constant or Incompletely Reported

Figure 5(a) shows the same information in the parameter domain that matters most for design. The envelopes indicate the approximate region of concrete strength and axial load ratio covered by each campaign family. Normal-strength benchmark studies extend to high axial load ratios; high-strength campaigns extend to high concrete strengths; the

region where both conditions hold simultaneously, cube strength above 80 MPa combined with an axial load ratio above 0.45, is very thinly populated. That region is not academic. It corresponds directly to the lower storeys of tall buildings, which is the application that motivates the use of these materials in the first place.

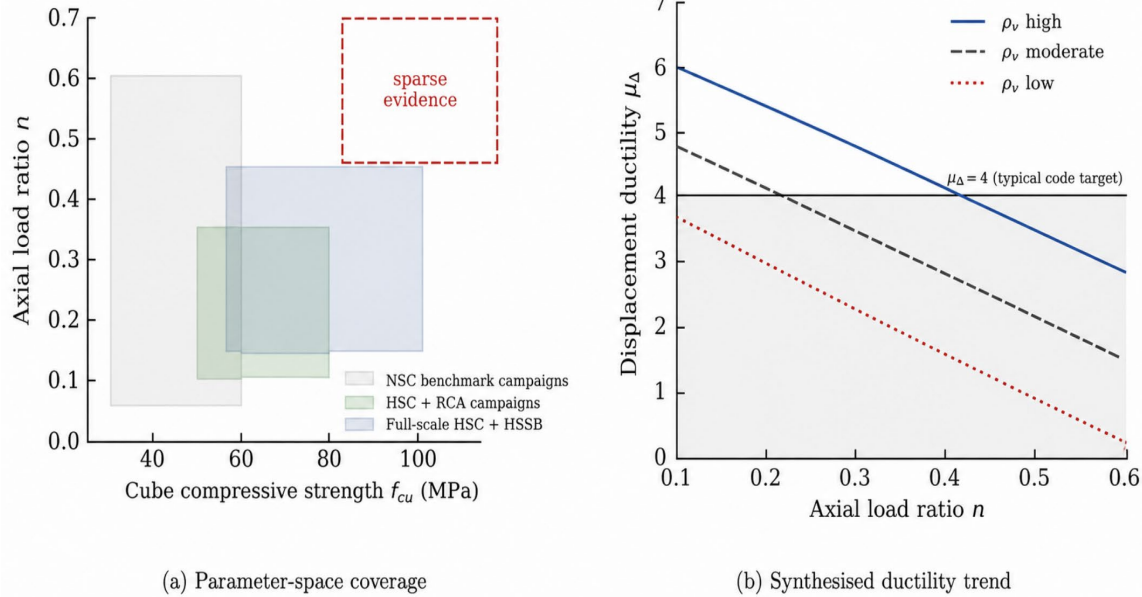


Fig. 5. (a) Approximate Parameter-space Coverage of the Reviewed Campaign Families, Showing the Sparsely Populated High-Strength, High-Axial-Load Region; (B) Directional Synthesis of the Relationship Between Axial Load Ratio, Volumetric Transverse Reinforcement Ratio and Displacement Ductility. Panel (B) Represents the Consistent Direction and Approximate Gradient of the Effect Reported across the Corpus and is not a Fit to Pooled Data

B. Governing Parameters

TABLE III DIRECTIONAL SYNTHESIS OF GOVERNING PARAMETER EFFECTS ACROSS THE REVIEWED CORPUS

Parameter	Effect on Strength/Capacity	Effect on Deformation Capacity	Overall Observation
Axial load ratio, n	Increases up to the balanced condition, then decreases	Decreases strongly and monotonically	Consistent across every campaign reviewed. The dominant control on deformation capacity; governs the transition from tension- to compression-controlled failure.
Concrete strength, f_{cu}	Increases	Decreases mildly at constant confinement	Consistent, but weaker than the axial load ratio effect. Can be offset by increased confinement up to a practical ceiling.
Stirrup spacing	Minor direct effect	Decreases sharply as spacing increases	Consistent. Governs the effectively confined core area and restrains longitudinal bar buckling.
Longitudinal bar grade	Increases at equal reinforcement area; neutral at equal strength	Broadly neutral under equal-strength substitution	Consistent. The principal caveat concerns crack width and residual deformation, not seismic capacity.
Transverse bar grade	Increases less than nominal strength implies	Increases less than nominal strength implies	Consistent where measured, but measured infrequently. The central unresolved issue of this review.
RCA replacement ratio	Slight decrease	Slight decrease at matched confinement	Reasonably consistent below moderate replacement ratios; poorly characterized at high strength and high axial load.

Table III summarizes the direction, consistency and design significance of each governing parameter as established across the corpus. Four points warrant emphasis. First, the axial load ratio dominates. Its effect on deformation capacity is larger, more consistent and more reliably reproduced than that of any other variable, including concrete grade. Figure 5(b) reflects this: displacement ductility falls approximately

linearly with axial load ratio, and the volumetric transverse reinforcement ratio shifts the relationship upward without substantially changing its slope. A well-confined member can hold a displacement ductility above four to an axial load ratio of roughly 0.4; a lightly confined member of the same materials cannot reach four even at low axial load. Beyond an axial load ratio of about 0.4, several campaigns report a

change in failure character, with vertical splitting cracks in the compression zone appearing at mid-range drifts and developing rapidly, showing a shift from tension-controlled to compression-controlled failure [6].

Second, concrete grade matters less than intuition suggests. Its effect on ductility at constant confinement is real but modest, and it is readily offset by increased transverse reinforcement up to the practical ceiling noted in Section 3.2. The corpus does not support the common framing of HSC as intrinsically unsuitable for seismic application; it does support that HSC requires detailing calibrated to its own dilation characteristics rather than borrowed from normal-strength practice.

Third, the substitution of high-grade longitudinal reinforcement on an equal-strength basis is well supported. This is the clearest positive finding in the review, and it is reproduced across laboratories, section geometries and reinforcement grades from 500 to 685 MPa.

Fourth, by contrast, the use of high-grade transverse reinforcement rests on a much narrower base, and the

evidence that exists points to a systematic overestimate in current design equations. This asymmetry, a well-supported case for longitudinal HSSB alongside an under-evidenced and probably unconservative treatment of transverse HSSB, is the single most important structural feature of the literature as it currently stands.

C. Code Provisions and Their Evidentiary Basis

Figure 6 compares the nominal reinforcement strength ceilings and drift limits adopted in five jurisdictions. Two observations follow. The dispersion in permitted transverse reinforcement strength is far wider than the dispersion in permitted longitudinal strength, ranging from 500 MPa in China and New Zealand to 1275 MPa in Japanese practice for shear reinforcement. That dispersion does not reflect a corresponding dispersion in the underlying material behavior; it shows differing regulatory postures toward the same incomplete evidence base. Given the mobilization problem discussed in Section III, the jurisdictions permitting the highest transverse strengths are relying most heavily on an assumption that the reviewed evidence does not fully support.

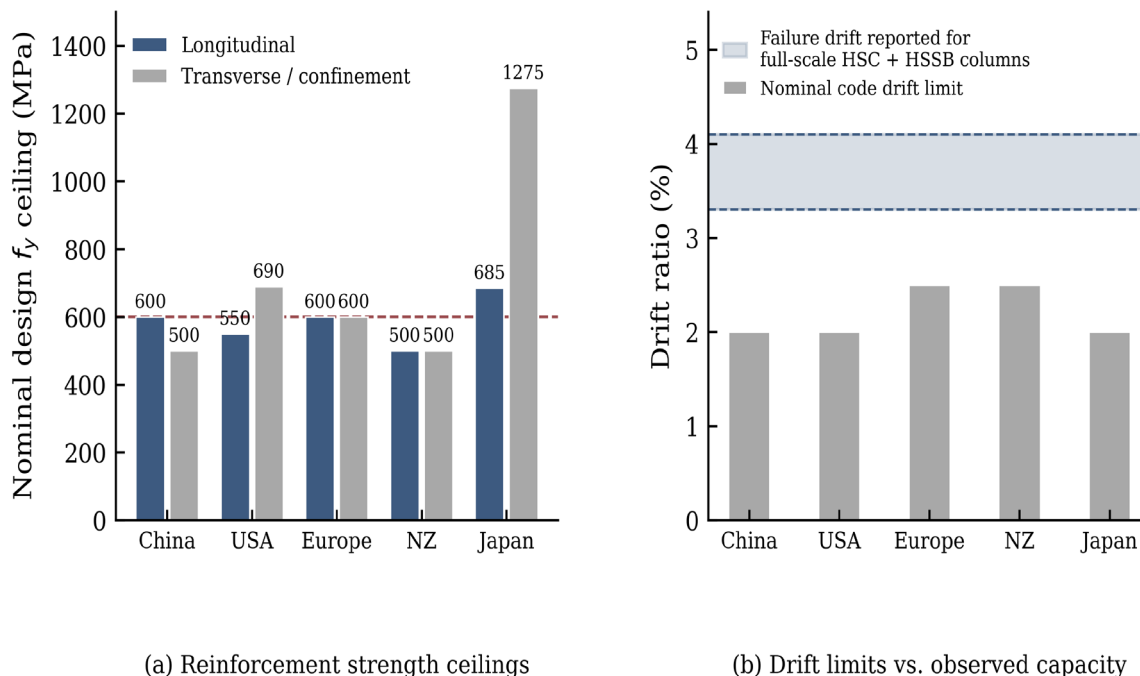


Fig.6 (a) Nominal Design Yield Strength Ceilings for Longitudinal and Transverse Reinforcement in GB 50010 (China), ACI 318 (USA), EN 1992/EN 1998 (Europe), NZS 3101 (New Zealand) and AIJ practice (Japan); (b) Nominal Elasto-Plastic Drift Limits Evaluated with the Range of Failure Drifts Reported for Full-scale HSC Columns with High-strength Reinforcement. Ceilings are Indicative and Subject to Conditions in the Governing Edition of each Document; Readers Should Verify Against the Current Edition

The second observation concerns drift. Full-scale HSC columns with high-strength reinforcement have been reported to fail at drift ratios in the range of approximately 3.3 to 4.1 percent [6], substantially above the nominal elasto-plastic drift limit of two percent specified for reinforced concrete frames in GB 50011-2010 [27] and above the comparable limits in other jurisdictions. Read positively, this indicates that a well-detailed HSC column with HSSB retains

reserve deformation capacity well beyond the code-anticipated demand. Read cautiously, it indicates that the drift limit is not the binding constraint on such members, and that the provisions which actually govern their safety, those for confinement and shear, are precisely the ones resting on the weakest evidence. A generous drift limit offers no protection against an overestimated restraining pressure.

TABLE IV COMPARISON OF RELEVANT PROVISIONS ACROSS JURISDICTIONS (INDICATIVE; VERIFY AGAINST THE GOVERNING EDITION)

Design Code / Standard	Maximum Longitudinal Reinforcement Strength	Maximum Transverse Reinforcement Strength	Key Provisions and Remarks
GB 50010 / GB 50011 (China)	600 MPa	500 MPa (seismic)	HRB600 permitted longitudinally; seismic application of 600 MPa transverse reinforcement restricted pending further evidence.
ACI 318-19 (USA)	550 MPa	690 MPa	Grade 550 permitted in special seismic systems; Grade 690 permitted for confinement, conditional on bar stress-strain characteristics.
EN 1992 / EN 1998 (Europe)	600 MPa	600 MPa	Ductility class requirements on reinforcement effectively constrain practice to lower grades in high-ductility systems.
NZS 3101 (New Zealand)	500 MPa	500 MPa	Conservative ceiling maintained; capacity design philosophy places strong emphasis on reliable yield of confining steel.
AIJ practice (Japan)	685 MPa	1275 MPa	Ultra-high-strength shear reinforcement long established in practice, supported by a distinct domestic testing tradition.

D. Effects On Design Practice

Three practical consequences follow from the synthesis. The first is that the stress assumed for high-strength transverse reinforcement should be limited by strain compatibility rather than by the nominal yield strength of the bar. Using an effective stress derived from the transverse strain developed at peak lateral load, rather than the nominal yield strength, would remove a known conservatism at modest cost in calculated capacity. Second, where the axial load ratio exceeds approximately 0.4 in a high-strength member, deformation capacity should not be assumed based on tests conducted at lower axial load, because the failure mechanism itself changes. Extrapolation across that boundary is extrapolation across a mechanism change, not across a continuous trend. The third is that finite element predictions of deformation capacity in flexural members should be treated with caution unless the constitutive model reflects the strain gradient. Where it does, agreement within roughly fifteen percent on yield, peak, and failure points appears achievable [6]; where it does not, the direction of the error is predictable, and the magnitude is not.

V. CONCLUSION

This review synthesized 62 studies on the seismic behaviour of high-strength concrete columns reinforced with high-strength steel bars, of which 24 full-scale or near-full-scale campaigns were carried through to quantitative parameter synthesis. The principal conclusions are as follows.

1. The axial load ratio is the dominant control on the deformation capacity of high-strength concrete columns, exceeding the influence of concrete grade, stirrup spacing and reinforcement grade. Its effect is consistent in sign and approximate magnitude across every campaign reviewed. Beyond an axial load ratio of approximately 0.4, the failure mode changes character from tension-controlled to compression-controlled, and deformation capacity should not be extrapolated across that boundary.

2. Substitution of high-strength longitudinal reinforcement on an equal-strength basis retains both lateral load capacity and deformation capacity. This finding is reproduced consistently across laboratories, geometries, and reinforcement grades from 500 to 685 MPa, and constitutes the best-supported result in the corpus. The principal qualification concerns wider crack widths and larger residual deformations at service load rather than seismic performance.
3. High-strength transverse reinforcement frequently does not attain its nominal yield stress at peak lateral load, because the reduced lateral dilation of high-strength concrete provides insufficient strain to yield a higher-grade bar. Design equations that credit the full nominal yield strength in shear and confinement therefore overestimate both capacity and ductility. The stress assumed for high-strength transverse reinforcement should be limited by strain compatibility.
4. Confinement improves the ductility of high-strength concrete columns, but with diminishing returns, because additional transverse steel is mobilized progressively less as concrete strength rises. A practical ceiling exists beyond which further transverse reinforcement reintroduces the congestion that high-performance materials were adopted to avoid. Configuration is as consequential as quantity at equal volumetric ratio.
5. Confined-concrete constitutive models calibrated on concentrically loaded specimens transfer poorly to flexural members unless the strain gradient across the section is explicitly represented. Where it is, prediction of yield, peak and failure points within approximately fifteen percent has been demonstrated; where it is not, the resulting error is systematic rather than random.
6. Reported failure drifts for well-detailed full-scale high-strength columns with high-strength reinforcement, in the range of approximately 3.3 to 4.1 percent, exceed the nominal elasto-plastic drift limits adopted in current codes. The drift limit is therefore not the binding constraint on such members; the confinement and shear provisions are, and those rest on the weakest evidence in the corpus.
7. The experimental parameter space is thinly populated where cube strength above 80 MPa coincides with an axial load ratio above approximately 0.45, which is precisely the

condition governing the lower storeys of the tall buildings that motivate these materials. International code provisions on transverse reinforcement strength differ by more than a factor of two across jurisdictions. This dispersion indicates differing regulatory postures toward a shared and incomplete evidence base rather than differing material behavior.

VI. LIMITATIONS AND FUTURE WORK

The gaps identified in the preceding sections resolve into four distinct lines of enquiry, each paired with the work that would settle it. The first concerns the utilization of transverse reinforcement: high-strength stirrups frequently have not yielded at the instant of peak lateral load, yet shear and confinement provisions credit their full nominal strength, and the remedy is to substitute a strain-compatibility-based effective stress capped by the stirrup strain actually measured at that instant. The second concerns coverage: almost no full-scale evidence exists where an axial load ratio above 0.4 coincides with a cube strength above 80 MPa, which is precisely the condition governing the lower storeys of tall buildings and the regime in which failure turns compression-controlled, so a dedicated test matrix spanning that region is required. The third concerns model transferability: confined-concrete models calibrated on concentrically loaded specimens omit the strain gradient characteristic of a member in flexure, and both eccentric calibration and an openly published benchmark set are needed to compare competing finite element formulations on common ground. The fourth concerns the mismatch between motivation and assessment in recycled-aggregate and semi-precast systems, which are adopted for partly environmental reasons yet evaluated almost exclusively against structural criteria; seismic performance and embodied carbon should be brought within a single objective framework, and the shell-to-core interface under reversed cyclic loading remains the aspect least well characterized and most consequential for reliability.

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