

Axial Performance of Reinforced Concrete Columns with Lap Splices and Flash-Butt Welded Splices under Concentric Compression

Mohammad Maghsoudi ^{a*} , Amirreza Toghroli Pour ^b

^a Department of Civil Engineering, University of Jiroft, Jiroft, Iran

^b Department of Civil Engineering, Faculty of Engineering, Shahid Bahonar University of Kerman, Kerman, Iran

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ABSTRACT

This study presents an extensive experimental investigation into the axial behavior of reinforced-concrete columns containing different splice types, transverse reinforcement configurations, and cross-sectional geometries. A total of six groups of circular and square columns were tested under concentric compression to evaluate the structural performance of lap splices and flash-butt (forging) splices in combination with various confinement systems, including individual ties, circular hoops, and spirals with different pitches. Mechanical and electrical strain gauges, displacement transducers, and load cells were employed to monitor stresses in longitudinal and transverse reinforcement as well as concrete strains at multiple heights, enabling a detailed assessment of stiffness degradation, crack initiation, ductility, and failure mechanisms. The results demonstrate that the measured peak axial loads ranged from 371.52 kN for the plain-concrete reference specimen to 958.48 kN for specimen FB. The pre-peak secant stiffness at 40% of peak load, $K_{0.4}$, ranged from 23.73 to 129.25 kN/mm, whereas the normalized energy-absorption ratio ranged from 0.45 to 1.10. Specimen FB, which contained a bottom flash-butt-forged splice and individual hoops, achieved the highest peak load, stiffness, and energy-absorption ratio. However, no consistent overall superiority of either lap splices or forged splices was observed; the response depended on the interaction among splice type, splice location, confinement, concrete strength, and specimen-specific construction quality. Specimens without transverse reinforcement in the splice region generally exhibited earlier cracking, wider cracks, and less stable deformation. The maximum crack width, 3.0 mm, was recorded for specimen LSM. Reducing the spiral pitch from 95 mm to 50 mm generally improved cracking resistance and deformation stability, although the magnitude of improvement varied among specimens. The results demonstrate that adequate transverse confinement and reliable construction quality are essential for stable axial performance of columns containing either lap splices or flash-butt-forged splices.

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1. Introduction

Reinforced concrete (RC) is the most widely used structural material in modern construction; concrete's limited tensile capacity is compensated by steel reinforcement, which frequently must be spliced because of bar-length limits and construction sequencing. Traditional lap splices overlapping bars for a specified length with transverse ties are simple, well codified and widely used, but they require additional length and can complicate detailing in congested regions [1–3]. Flash-butt forging (butt welding) produces a

* Corresponding author.

E-mail address: maghsoudi_mohammad@ujiroft.ac.ir (M. Maghsoudi).



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continuous metallic connection at bar ends and can reduce required splice length and ease congested detailing; however, metallurgical and geometric changes in the weld zone and the local stress-transfer mechanism raise questions about ductility, fatigue and post-peak behavior, particularly for members expected to form plastic hinges under seismic loading [4-11].

Parametric laboratory studies and code commentaries indicate that lap-splice capacity and post-peak ductility are governed primarily by lap length, bar diameter, concrete strength, concrete cover, and the level of transverse confinement, and that increasing lap length or improving confinement (tighter ties or spiral) consistently improves both strength and deformation capacity [1, 2, 12]. Controlled laboratory implementations and comparative evaluations show that properly executed forged splices can achieve static strength comparable to well-detailed laps in monotonic tests, but forged splices are more sensitive to welding procedure, heat-affected-zone (HAZ) effects, and fatigue performance; therefore, qualification, post-weld treatment, and non-destructive inspection are essential before their use in ductility-critical regions [3-7, 11].

A substantial body of experimental work demonstrates that continuous spiral confinement or closely spaced ties substantially increase compressive strain capacity and ductility of columns, delaying core crushing and stabilizing post-peak softening; enhanced confinement also reduces the performance gap between lap and welded splices, so that welded splices enclosed by strong confinement can approach the ductility of well-detailed lap splices provided the splice itself is metallurgically sound [2, 12-14]. Practical welding and inspection studies further emphasize the need for weld qualification: ultrasonic and other non-destructive testing methods have been proposed and applied to gas-pressure and flash-butt welded joints to detect defects in the HAZ and ensure reliable field acceptance [1, 7, 8, 11, 13, 15].

Country-level experience supports these laboratory findings. Japanese programs testing gas-pressure/flash-butt welded rebars reported that properly executed welds satisfied national tensile and bending requirements and that failures in many tests occurred away from the forged section, supporting the structural adequacy of qualified forging procedures; nevertheless, these programs also stress the importance of post-weld inspection and workmanship control, and recommend immediate post-weld finishing and stricter weld control as practiced after the 1995 Hanshin earthquake [7-9, 12]. Complementary experimental studies show that tighter spiral pitch or closer stirrup spacing increases ultimate load and ductility and can change failure modes (core crushing, spiral fracture, or longitudinal buckling), while full-scale cyclic and axial-cyclic tests confirm improved post-buckling and cyclic performance with enhanced confinement and with hybrid reinforcement schemes [10, 12, 16, 17].

Practical economy and detailing studies indicate welded splices can be economical for smaller bars when welding procedures and lengths are adjusted with bar diameter, but they also highlight the need for procedural qualification and inspection to manage HAZ and fatigue risks; review studies of weld fatigue and comparative welding methods reinforce the call for standardized qualification and inspection protocols [6, 7, 11, 18]. Research on strengthening and retrofit methods (FRP, external spirals, grooving/EBR techniques and hybrid steel-FRP reinforcement) further shows that confinement and external reinforcement substantially affect column capacity and ductility, underscoring that splice performance cannot be considered in isolation from confinement or retrofit measures [17, 19-21].

Recent experimental and analytical studies have confirmed that the structural behavior of reinforced-concrete members containing lap splices is governed by the interaction among lap-splice length, reinforcement diameter, concrete compressive strength, concrete cover, bond conditions, transverse reinforcement, and loading history [22-24]. Adequate transverse confinement is particularly important in lap-splice regions because it can improve bond transfer, delay splitting cracks, limit cover spalling, enhance post-peak response, and increase the deformation capacity of reinforced-concrete members [22-24]. Recent experimental research has also emphasized that lap-splice performance in reinforced-concrete columns is highly dependent on the confinement level and the detailing of the splice region, especially when the column is subjected to large inelastic deformation demands [23, 24].

The performance of flash-butt-forged or welded reinforcing bars is also dependent on factors beyond the nominal tensile capacity of the reinforcement. Recent investigations have shown that welding parameters, heat-affected-zone characteristics, local geometric irregularities, weld defects, and fatigue resistance can affect the mechanical reliability of welded reinforcing bars [3, 4, 11]. In particular, recent research on high-strength reinforcing bars joined by flash-butt welding has highlighted the importance of welding quality control, post-weld inspection, and evaluation of fatigue behavior before application in structural members subjected to cyclic or inelastic loading [3, 4]. Therefore, a welded or forged splice that satisfies a static tensile-strength requirement may still require careful assessment of its local stress transfer, ductility, and long-term performance.

Recent studies on reinforced-concrete columns further confirm that the transverse-reinforcement type, spacing, and configuration strongly influence the confinement effectiveness, concrete-core behavior, reinforcement stability, axial deformation capacity, and post-peak response of the members [14, 17]. Closer spiral pitch or denser transverse reinforcement generally improves confinement of the concrete core, delays crushing and spalling, and promotes a more gradual reduction in load after the peak response. Conversely, insufficient confinement can lead to premature deterioration of the concrete core, increased transverse reinforcement demand, reinforcement instability, and brittle failure [14, 17].

Although the effects of lap splices, welded reinforcement, and transverse confinement have been studied independently, relatively limited experimental information is available on the direct comparison of lap-spliced and flash-butt-forged reinforcing bars in short reinforced-concrete columns subjected to monotonic concentric compression. In particular, the combined effects of splice type, splice location, transverse-reinforcement configuration, confinement spacing, and cross-sectional shape on axial capacity, reinforcement stress, cracking response, post-peak behavior, and energy-based ductility require further investigation. Accordingly, the present study evaluates the axial performance of 22 short reinforced-concrete columns containing lap splices or

flash-butt-forged splices under monotonic concentric compression.

The experimental program consisted of 22 short RC columns arranged in six test groups, designed to isolate the effects of splice type (lap vs. flash-butt-forged), transverse confinement (stirrup spacing, spiral vs. ties), and section/reinforcement detailing. Specimens were instrumented to record longitudinal and transverse rebar stresses, concrete strain, crack widths, axial and lateral displacements, and peak load; failure modes and failure lengths were documented by visual inspection and photographic records. Test variables included bar diameter and arrangement, concrete strength, confinement configuration, and splice location; monotonic axial compression tests were performed to obtain peak and post-peak behavior.

The study aims to: (1) quantify the difference between theoretical capacity and measured peak load for lap and forged splices under axial compression; (2) compare ductility and post-peak softening as a function of splice quality and transverse confinement; and (3) evaluate the effectiveness of confinement detailing (spiral vs. ties, spacing) in restoring or improving ductility of welded splices.

2. Experimental program

2.1. Objectives and scope

The experimental program was designed to compare the axial compressive response, failure modes, and ductility of short RC columns containing either lap splices or flash-butt forged splices. The study varied cross-section geometry, transverse reinforcement type and spacing, and splice location to evaluate the influence of confinement and detailing on splice performance.

2.2. Specimen matrix and variables

A total of 22 short RC columns were cast and tested, arranged into six groups. The principal variables were:

- Splice type: lap splice (overlap) vs. flash-butt forging (butt weld/forging).
- Cross-section shape: square vs. circular.
- Transverse reinforcement: closed ties (stirrups) vs. continuous spiral confinement; spacing and diameter varied.
- Splice location: mid-height (central) vs. near the column end (top or bottom).

All reinforced-concrete columns were longitudinally reinforced with 16-mm deformed bars and transversely reinforced with 8-mm deformed ties. Tensile tests were conducted on representative samples of each bar diameter in accordance with ASTM A615 [25]. The measured mean yield strength, ultimate tensile strength, and modulus of elasticity are summarized in **Error! Reference source not found..** These measured properties were used to assess whether the stresses recorded by the strain-gauge instrumentation reached the yield level during the tests.

Table 1. Mechanical properties of the reinforcing steel used in the test specimens

Reinforcement type	Nominal diameter (mm)	Steel grade	Yield strength, f_y (MPa)	Ultimate tensile strength, f_u (MPa)	Elastic modulus, E_s (GPa)
Longitudinal reinforcement	16	Grade 60	420	600	200
Transverse reinforcement: ties/hoops/spirals	8	Grade 40	300	500	200

Longitudinal and transverse reinforcement spacing and detailing follow the provisions of ACI 318-19 [1], and the concrete cover for all specimens was maintained 20 mm. The nominal concrete cover was controlled during reinforcement-cage assembly by means of 20 mm spacers positioned around the reinforcement cage before casting. The same nominal cover-control procedure was adopted for all specimens to minimize variability in the effective core dimensions and reinforcement location. This constitutes a limitation of the experimental program and is acknowledged when interpreting specimen-to-specimen differences. Each group used the same concrete mix and curing regime for all specimens in that group; specimens differed only in splice type/location and transverse reinforcement spacing. Detailed descriptions for each group are provided below.

Group 1 comprised four circular reinforced-concrete columns, each reinforced with four longitudinal bars and separate circular ties (individual hoops) spaced at approximately 95 mm. Two specimens contained 300 mm lap splices, one located at mid-height and the other at the column base, while the remaining two specimens used flash-butt (forged) splices, likewise placed one at mid-height and one at the base. All specimens in this group shared identical concrete mix and environmental curing conditions; the geometric and reinforcement details are shown in Fig. 1.

Each column name encodes the splice type and splice location. FB: forging at Bottom (flash butt splice located at the column base), FM: forging at mid height (flash butt splice located at mid height). SB: splice (Lap) at bottom (300 mm lap splice located at the column base). SM: splice (Lap) at mid height (300 mm lap splice located at mid height).

Group 2 consists of four circular reinforced-concrete columns geometrically and materially identical to those in group 1 but with the transverse reinforcement intentionally omitted at the splice region (Fig. 2).

The specimens in this group are designated LFB (forging at bottom), LFM (forging at mid height), LSB (lap splice at bottom), and LSM (lap splice at mid height), where each code indicates the splice type (forging or lap) and its location along the column height.

Group 3 consists of four circular reinforced-concrete columns identical in geometry and materials to group 2, but with continuous spiral confinement used for the transverse reinforcement at a pitch of 95 mm (Fig. 3). The use of closely spaced spiral confinement in this group is intended to investigate the influence of enhanced transverse confinement on splice performance, ductility, and failure mode under the applied loading protocol.

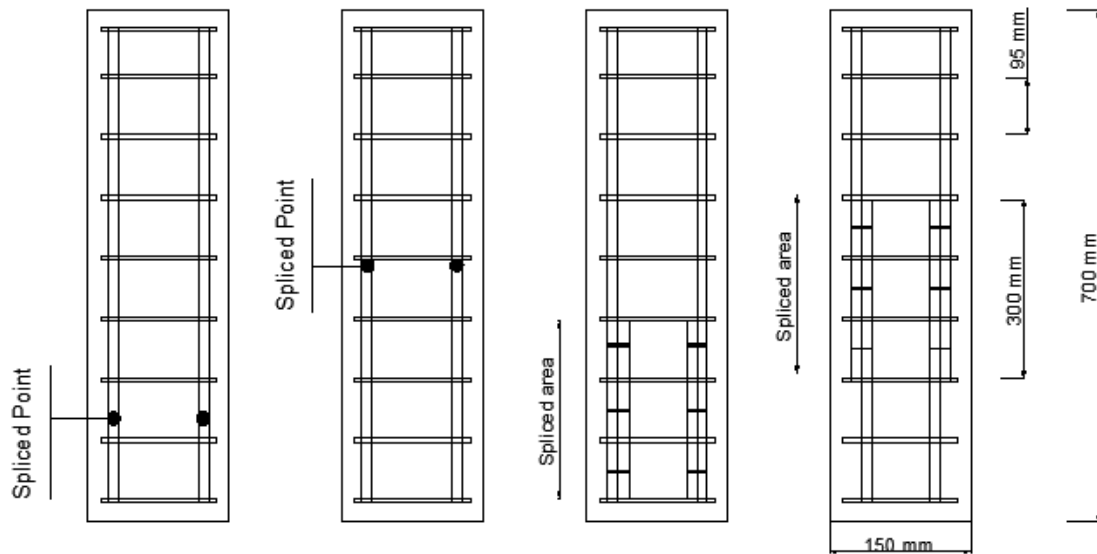


Fig. 1. Location, splice type, and tie details for group 1.

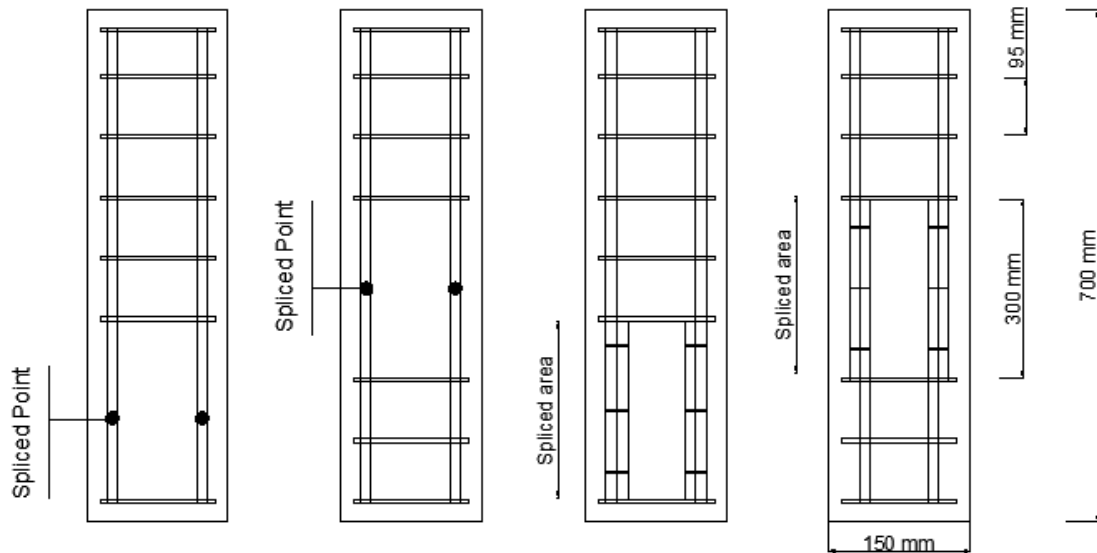


Fig. 2. Location, splice type, and tie details for group 2.

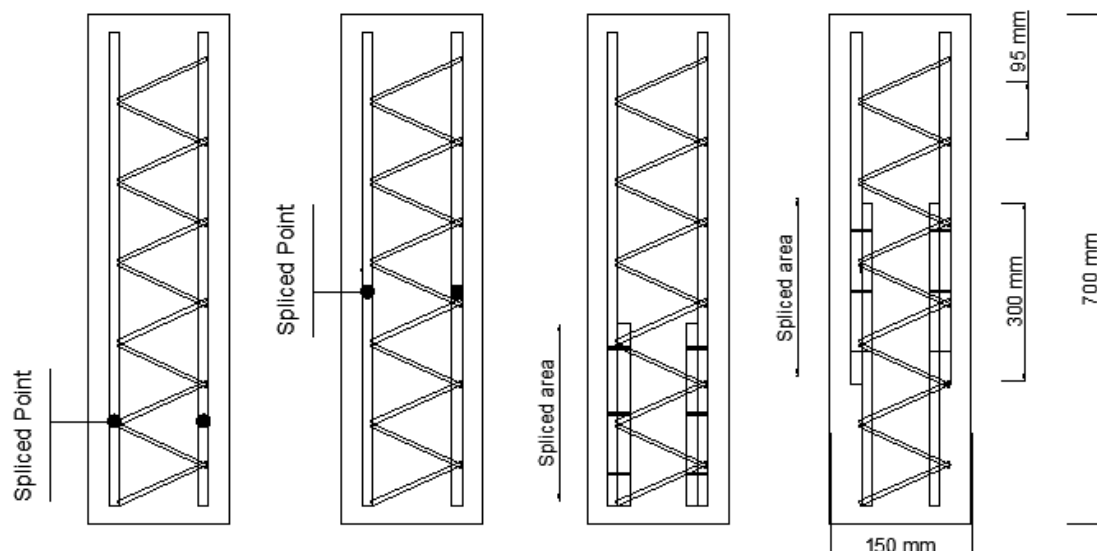


Fig. 3. Location, splice type, and the transverse spiral reinforcement details for group 3.

The specimens in group 3 are designated SFB (forging at bottom), SFM (forging at mid-height), SSB (lap splice at bottom), and SSM (lap splice at mid-height).

Group 4 comprises four circular columns similar to those in group 3 but with the spiral pitch reduced to 50 mm to provide higher transverse confinement; the specimens are designated HSFB (forging at bottom), HSBM (forging at mid-height), HSSB (lap splice at bottom), and HSSM (lap splice at mid-height). Group 5 contains four square columns whose tie arrangement and splice locations mirror those of group 1; these specimens are labeled RFB (forging at bottom), RFM (forging at mid-height), RSB (lap splice at bottom), and RSM (lap splice at mid-height). Group 6 includes two circular specimens used as control columns: an unreinforced concrete column denoted as PC, and a reinforced circular column with continuous (unspliced) longitudinal bars and individual hoops denoted as L.

2.3. Materials and fabrication

Normal-weight concrete with controlled mix proportions was used for all specimens. A crushed angular coarse aggregate with a nominal particle-size range of 6–12 mm and fine aggregate with a maximum particle size of 4 mm was also used in the concrete mixtures. The use of a maximum coarse-aggregate size of 12 mm was considered suitable for the relatively small cross-sectional dimensions of the test columns, the nominal concrete cover, and the locally congested reinforcement and splice regions, thereby facilitating concrete placement and consolidation around the reinforcement cage. The physical properties are summarized in Table 2. Coarse-aggregate and fine-aggregate properties were determined in accordance with ASTM C127 [26] and ASTM C128 [27], respectively.

Table 2. Physical properties of the fine and coarse aggregates used in the concrete mixture.

Aggregate type	Particle-size range (mm)	Specific gravity	Water absorption (%)	Bulk density (kg/m ³)
Fine aggregate	≤ 4	2.5	0.5	2.6
Coarse aggregate	6–12	2.7	1.5	2.6

Compressive strength f'_c was measured on companion cylinders at the time of testing; average values for each batch were recorded. Deformed steel bars of specified diameters were used. Lap lengths were chosen according to code provisions for lap splices; forging/welded splices were produced by flash-butt forging equipment with controlled heating and forging parameters. Either closed rectangular ties or continuous spiral reinforcement was provided. Spiral confinement was used in selected circular specimens to study confinement effects. Lap splices were prepared by overlapping bars for the design lap length and tying with binding wire. Flash-butt forging splices were produced by aligning bar ends, applying electric flash heating and axial pressure to forge the ends into a continuous section. Welding/forging parameters and visual inspection records were kept (Fig. 4).



Fig. 4. (a) Longitudinal clamped rebars prior to heating; (b) heating procedure; (c) Rebars spliced by the forging method; and (d) reinforcement cages of the specimens.

Circular specimens were cast using PVC tubes with an outer diameter of 160 mm, while square specimens were cast in a custom-designed, removable steel formwork (Fig. 5). 20 mm spacers were used to ensure the specified concrete cover and uniform cover thickness. To facilitate casting and vibration, all forms were secured in a laboratory holding device that had been fixed to the floor with gypsum. A single concrete mix design was used for all groups; however, companion specimens for compressive strength tests were taken at each casting stage. Although a common nominal concrete mix design and curing regime were adopted for the experimental program, separate casting stages were required for specimen fabrication. Therefore, companion concrete specimens were prepared and tested for each casting stage to determine the compressive strength corresponding to the relevant column specimens at the time of testing. The measured concrete compressive strength, associated with each specimen is reported in Table 3 and was used in calculating the theoretical axial capacity of that specimen. Consequently, the theoretical capacities reported in Table 4 reflect the measured concrete strength rather than a single assumed compressive-strength value for all columns. For each casting campaign, at least three 150 × 150 × 150 mm cubes and three 150 × 300 mm cylinders were cast and tested for capacity calculations. One day after casting, specimens were demolded and cured under identical moist conditions until the test age.



Fig. 5. Filled column formworks after placement and vibration.

Table 3. Numerical summary of all tested specimens.

Specimen	f'_c (MPa)	Service load $P_s = 0.4P_c$ (kN)	Cracking Load (kN)	Transverse reinforcement	Spacing (mm)
SM	29.2	206.4	562.0	Individual hoops	95
SB	33.8	238.9	630.0	Individual hoops	95
FM	30.6	216.3	297.6	Individual hoops	95
FB	33.8	238.9	724.0	Individual hoops	95
LSM	34.5	243.8	124.0	Hoops omitted at splice	95
LSB	23.1	163.3	230.0	Hoops omitted at splice	95
LFM	34.5	243.8	120.0	Hoops omitted at splice	95
LFB	27.5	194.4	297.0	Hoops omitted at splice	95
SSM	31.8	224.7	102.0	Spiral	95
SSB	22.5	159.0	207.0	Spiral	95
SFM	22.5	159.0	410.0	Spiral	95
SFB	31.8	224.7	310.0	Spiral	95
HSSM	31.5	222.6	700.0	Spiral	50
HSSB	31.5	222.6	282.4	Spiral	50
HSFM	32	226.2	358.0	Spiral	50
HSFB	26.4	186.6	351.7	Spiral	50
RSM	28	197.9	262.0	Closed ties	95
RSB	37.6	265.8	390.0	Closed ties	95
RFM	34.5	243.8	490.0	Closed ties	95
RFB	18.2	128.6	351.7	Closed ties	95
L	22.5	159.0	162.0	Individual hoops	95
PC	41	289.9	—	None	—

2.4. Instrumentation and test setup

Strain gauges were attached to longitudinal and transverse bars and to concrete surfaces to monitor axial strains. Concrete axial strain was monitored using electrical resistance strain gauges attached to the external concrete surface of the specimens. The gauges were oriented parallel to the column longitudinal axis and installed at mid-height and near ends. The concrete strain values reported in Table 4 correspond to the maximum measured surface strain. Because the gauges were attached to the external surface, the recorded values represent local axial strains in the cover concrete at the instrumented locations. They were used to monitor local deformation and cracking but do not directly represent the axial strain of the confined concrete core inside the transverse reinforcement.

The electrical strain recorded by each gauge was converted to steel stress using the adopted bilinear elastic-perfectly plastic stress-strain relationship. In the elastic range, the reinforcement stress was calculated as Eq. 1:

$$\sigma_s = \begin{cases} E_s \varepsilon_s & \text{if } |E_s \varepsilon_s| < f_y \\ f_y & \text{if } |E_s \varepsilon_s| \geq f_y \end{cases} \quad (1)$$

where σ_s is the local reinforcement stress, E_s is the elastic modulus of the reinforcing steel, and ε_s is the strain recorded by the strain gauge.

The modulus of elasticity was taken as $E_s = 200$ GPa. The yield strength of the reinforcement was $f_y = 420$ MPa.

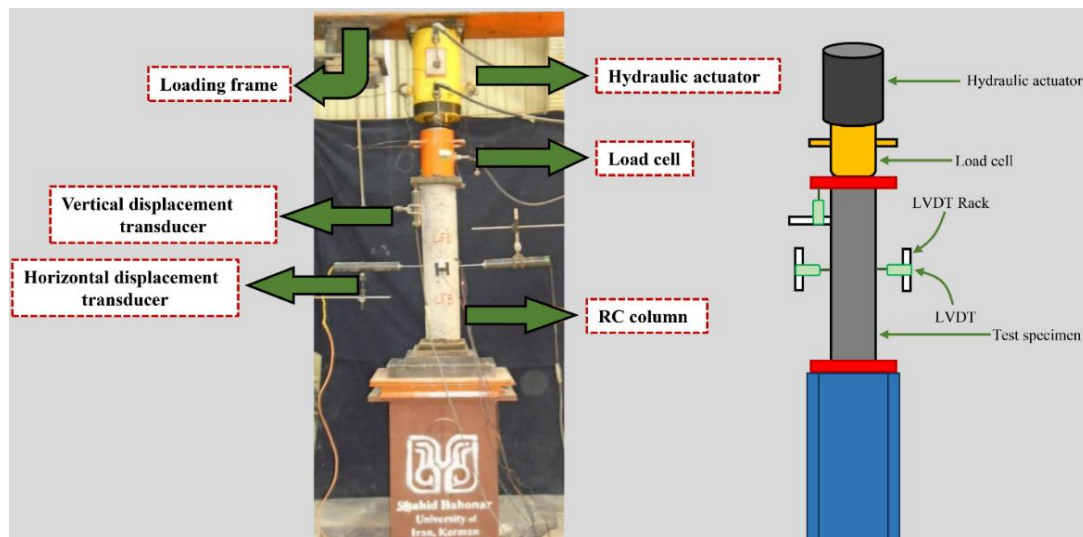
The stresses shown in Table 4 correspond to the local gauge-derived stress at the experimentally measured peak axial load (P_{exp}). When more than one gauge was installed on the same reinforcement category, the reported value represents the maximum gauge at the critical location. Gauge readings obtained after gauge debonding, wire damage, or data loss were excluded from the stress calculation.

Table 4. Comparative summary of theoretical capacity, stresses, strains, and deformations for tested columns.

Specimen	Theoretical capacity P_{th} (kN)	Experimental peak load P_{exp} (kN)	P_{th}/P_{exp}	Longitudinal rebar stress (MPa)	Transverse rebar stress (MPa)	Concrete strain	Failure Length (mm)	Crack Width (mm)	Axial Displacement (mm)	Lateral displacement (mm)
SB	713.0	817.3	0.87	434.0	102.0	0.0014	310	0.16	10.87	2.83
FB	713.0	977.4	0.73	312.0	226.2	0.0015	290	0.06	8.25	6.62
SM	660.2	825.0	0.80	297.8	124.0	0.0009	335	0.18	9.83	2.42
FM	676.9	739.2	0.92	308.6	353.8	0.0007	300	0.20	11.08	3.03
LSB	590.3	630.9	0.94	243.8	453.0	0.0004	250	1.80	10.47	3.07
LFB	640.2	547.1	1.17	283.2	283.2	0.0008	280	2.00	10.46	0.98
LSM	721.6	741.0	0.97	340.0	260.0	0.0005	210	3.00	11.48	—
LFM	721.6	615.7	1.17	281.6	398.2	0.0009	165	0.22	12.34	3.24
SSB	600.0	470.5	1.28	242.2	62.2	0.0013	370	0.20	6.87	2.02
SFB	713.6	610.0	1.17	356.2	387.6	0.0023	280	0.20	9.66	1.98
SSM	713.6	636.2	1.12	161.4	332.2	0.0012	310	1.80	12.43	4.70
SFM	600.0	541.0	1.11	272.4	69.8	0.0005	390	0.04	10.83	4.58
HSSB	598.5	620.0	0.97	210.4	158.2	0.0005	270	0.20	10.05	1.90
HSFB	647.3	687.0	0.94	286.4	290.8	0.0006	220	1.40	12.10	2.18
HSSM	709.4	795.3	0.89	242.8	174.4	0.0017	200	1.80	10.85	3.32
HSFM	551.0	574.2	0.96	250.6	188.6	0.0007	230	0.20	11.13	2.68
RSB	880.0	526.6	1.67	115.0	85.6	0.0008	460	0.24	10.30	2.33
RFB	592.5	764.9	0.77	312.0	296.0	0.0014	300	0.04	12.10	2.18
RSM	784.3	804.5	0.97	355.8	415.6	0.0012	350	0.40	9.34	2.60
RFM	834.4	732.6	1.14	296.0	425.4	0.0009	290	0.14	11.25	3.85
PC	492.6	396.6	1.24	—	—	0.0008	460	—	—	—
L	579.7	614.0	0.94	233.8	199.0	0.0009	265	0.14	11.01	2.82

Note: missing or unrecorded values are shown as —.

Displacement transducers measured axial shortening and lateral deformations. Crack monitoring and crack-width gauges recorded crack initiation and propagation. Data acquisition logged all channels continuously during monotonic concentric axial loading to failure. A hydraulic actuator applied concentric axial load under monotonic protocol until failure (Fig. 6).

**Fig. 6. Filled column formworks after placement and vibration.**

To ensure uniform load distribution and improve test reliability, all circular specimens were double-capped with sulfur caps, thereby ensuring that the applied axial load was distributed evenly over the entire specimen cross-section. Prior to each test, the specimen was manually centered on the lower loading platen so that its centroidal axis approximately coincided with the actuator loading axis. For circular specimens, sulfur capping was applied at both ends to provide level and approximately parallel bearing surfaces, thereby improving load distribution and reducing local stress concentrations at the specimen ends. A small seating load was applied before the main test to establish full contact between the specimen and the loading platens. The specimen position was visually checked and adjusted, where necessary, before the monotonic axial load was applied. Although the alignment procedure was carefully controlled, the initial loading eccentricity was not instrumentally measured for each specimen.

3. Results and discussions

3.1. General observations and failure modes

Failure location: almost all specimens failed in the upper region of the column (except one specimen), even when cracking initiated at the bottom (Fig. 7). This indicates that local conditions and stress redistribution influenced ultimate failure location. The observed concentration of failure in the upper region of many specimens may be related not only to local concrete quality, splice location, and end conditions, but also to small unavoidable deviations from ideal concentric loading. A residual loading eccentricity can introduce a secondary bending effect under nominally axial compression, resulting in nonuniform concrete stress, unequal stress development in the longitudinal reinforcement, and localized crack initiation or crushing. Because initial eccentricity was not measured instrumentally for each specimen, its individual contribution cannot be quantified. Nevertheless, the consistent loading setup, sulfur-capping procedure, and specimen-centering method applied to all specimens reduce the likelihood that alignment differences alone explain the overall comparative trends among the tested groups.



Fig. 7. Filled column formworks after placement and vibration.

Crack patterns: axial compression produced vertical splitting cracks; confinement detailing affected crack width and localization. Spiral-confined specimens showed more distributed cracking and delayed localization.

Concrete strain at failure: circular columns with spiral confinement reached concrete strains exceeding 0.003, surpassing the commonly referenced ACI nominal crushing strain for unconfined concrete.

Longitudinal steel behavior: in nearly all specimens, the longitudinal reinforcement did not reach yield at peak load, indicating that concrete crushing or confinement-controlled failure governed ultimate capacity.

The predominance of top-region failures suggests that local stress concentrations, slight eccentricities, or splice location relative to end conditions influenced ultimate behavior. The observation that longitudinal bars generally did not yield implies that concrete crushing or confinement-controlled failure governed the ultimate capacity in most specimens. This is consistent with confined concrete theory: when confinement is insufficient, concrete crushing can occur before steel yields.

3.2. Representative experimental summary

The representative numerical summary of all tested specimens shown in Table 3 provides a clear comparative view of how confinement type, splice detailing, and concrete quality influence the axial performance of reinforced-concrete columns. The measured concrete compressive strength was not uniform among all specimens; the recorded values ranged from 18.2 MPa to 41.0 MPa. Therefore, direct comparison of raw cracking load, peak load, and deformation capacity across all specimens would not isolate the effects of splice type or transverse-reinforcement configuration. To account for this variation, the theoretical axial capacity of each specimen was calculated using its corresponding measured concrete compressive strength, and the experimental response was interpreted together with the experimental-to-theoretical capacity ratio, P_{exp}/P_{th} . Comparisons between splice types and confinement details were also prioritized for specimens with comparable geometry, reinforcement arrangement, splice location, and concrete strength. Nevertheless, residual variation in concrete quality and strength remains a source of scatter in the experimental results.

As shown in Table 3, specimens in group 1, which were confined with individual hoops at 95 mm spacing, consistently exceeded their theoretical load capacities, with increases ranging from 9% to 32%, demonstrating the effectiveness of proper transverse reinforcement. In contrast, specimens in group 2, where hoops were intentionally omitted over the splice region, exhibited noticeable reductions in ultimate strength up to 15% below theoretical predictions and significantly larger crack widths, reaching 3 mm in LSM. This confirms that the absence of confinement in splice zones directly compromises structural integrity and accelerates failure.

Columns in group 3, reinforced with spirals at a relatively large pitch of 95 mm, also failed below their theoretical capacities (by 10–22%), largely due to insufficient confinement. Their failure lengths, often exceeding 30–39 cm, indicate extensive damage propagation. In contrast, group 4, with a reduced spiral pitch of 50 mm, demonstrated markedly improved cracking resistance such as the 700 kN cracking load in HSSM, highlighting the substantial benefits of dense spiral confinement. Square columns in group 5 showed competitive performance, with cracking loads up to 490 kN, reflecting the effectiveness of closed ties in rectangular sections. Finally, the reference specimens in group 6 illustrate baseline behavior: the plain concrete column (PC) exhibited high compressive strength but no ductility, while the continuously reinforced column (L) cracked at 1620 kN, emphasizing the role of splice detailing and confinement in enhancing column performance.

The table below consolidates the principal experimental outcomes for all tested columns across six groups. Each row corresponds to a single specimen and reports the theoretical axial capacity, measured peak load, central concrete strain (digital), and the experimental/theoretical capacity ratio. This condensed dataset is intended to support direct comparison of transverse reinforcement detailing, splice/connection type, and concrete quality on ultimate capacity and ductility. The paragraphs that follow interpret these results and draw engineering conclusions and recommendations.

The consolidated experimental dataset shows a clear and consistent pattern: most specimens failed by concrete crushing at the compression face, while longitudinal reinforcement generally remained below yield at peak load. Measured peak loads ranged widely (≈ 397 – 977 kN), and the experimental-to-theoretical capacity ratio varied from about 60% to 137%, indicating that detailing and material integrity produced both substantial capacity shortfalls and notable reserves relative to theoretical predictions.

Transverse confinement emerged as the primary control on both strength and ductility. Specimens with close spiral pitch or dense ties recorded higher central strains, smaller failure lengths, and more gradual post-peak softening, demonstrating that effective confinement raises the compressive strain capacity of the core and delays catastrophic core crushing. In these well-confined columns, the axial shortening at peak was larger, and the residual capacity after peak load was more pronounced.

Concrete quality and workmanship had an equally decisive effect. Columns exhibiting honeycombing or poor compaction (notably RSB and several group 3 specimens) showed the largest reductions in peak load – up to roughly 40% below theoretical – and developed broader, more diffuse failure zones. Surface repairs did not reliably restore core integrity, so poor placement and consolidation produced permanent losses in both capacity and deformation capacity.

Splice type alone did not determine ultimate performance; rather, splice behavior depended on the interaction of splice form, local detailing, confinement, and fabrication quality. At service levels, lap and mechanical/forged splices behaved similarly when confinement was comparable, but at ultimate loads lap splices combined with adequate confinement tended to produce more progressive, ductile failures. Mechanical splices achieved high peak loads in some cases but were more sensitive to fabrication defects and therefore require strict quality assurance to ensure reliable performance.

Observed stress patterns underline the demand placed on transverse reinforcement: several specimens showed transverse bar stresses approaching or exceeding yield while longitudinal bars remained elastic, indicating significant lateral expansion of the core under axial compression. Where ties or spirals were insufficient, tie rupture or opening precipitated rapid core disintegration; this highlights the need to design transverse reinforcement not only for confinement but also to resist the lateral pressures that develop prior to longitudinal yielding.

From a practical standpoint, the tests support three immediate recommendations: provide robust confinement around splice zones (close spiral pitch or closely spaced ties); enforce rigorous quality control for mechanical splices (factory certification, sample destructive tests and nondestructive inspection) and combine splices with confinement detailing; and ensure proper concrete placement and compaction, since workmanship defects cannot be reliably corrected by surface patching.

The study's limitations should be acknowledged: tests were monotonic axial compression on short columns, so behavior under combined axial-bending loading or cyclic seismic actions may differ. A statistical grouping of specimens (means and standard deviations by splice and confinement type) and targeted tests that vary splice location, splice length, and controlled fabrication defects would strengthen the conclusions and refine design guidance.

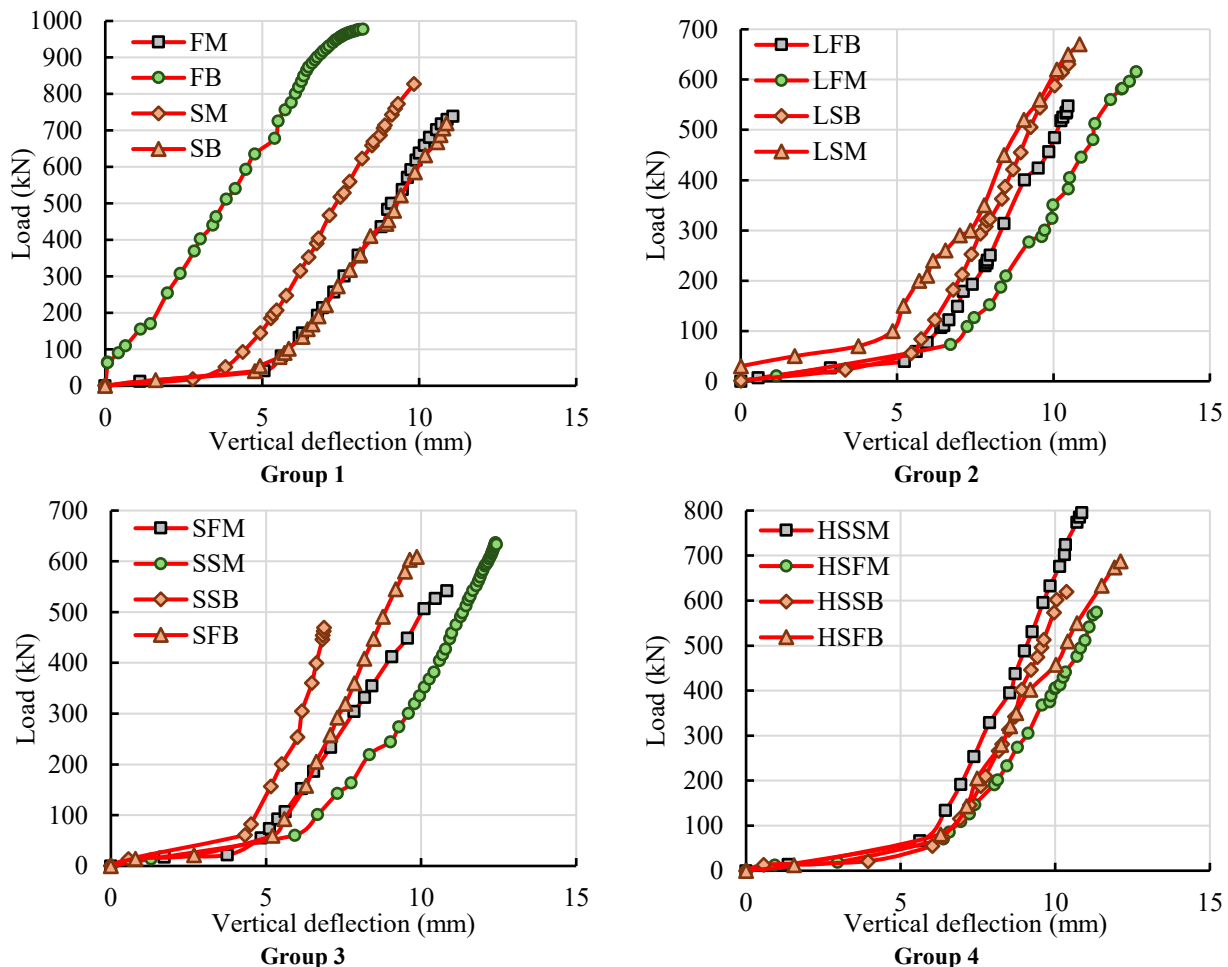
In summary, the experimental evidence is unequivocal: transverse confinement and concrete workmanship are the dominant determinants of column strength and ductility. Splice selection must be considered within that broader detailing and quality framework—adequate confinement and strict quality control are the decisive measures to ensure safe, ductile column performance.

Overall, the numerical summary reveals a consistent pattern: adequate confinement, especially with reduced spiral pitch, significantly increases cracking load, reduces damage length, and improves ultimate strength, whereas insufficient confinement or poor concrete quality can reduce capacity by more than 20% and lead to wider cracks and brittle failure. This comparative dataset provides a strong quantitative foundation for evaluating the structural reliability of different splice and confinement configurations.

3.3. Vertical load-deflection response

The vertical load-deflection diagrams for all column groups are presented in Fig. 8, illustrating the deformation behavior of the specimens under axial loading.

The vertical load-deflection curves derived from the experimental data provide a comprehensive view of the axial stiffness, nonlinear transition, and failure behavior of all tested column groups. To quantify the axial stiffness of the tested specimens, secant stiffness values were determined from the ascending branch of each experimental axial load-axial displacement curve.



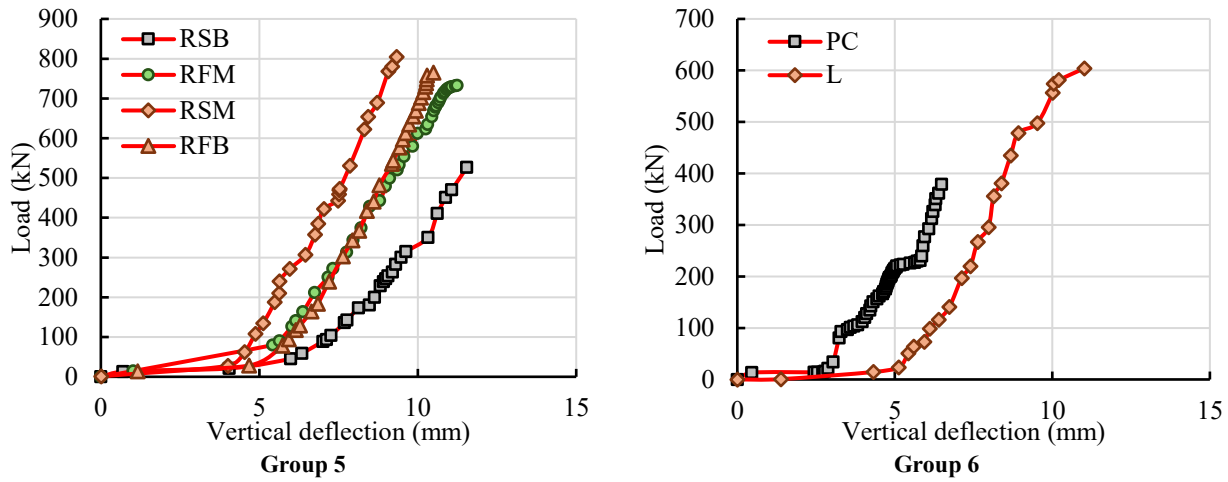


Fig 8. Axial load-deflection response of column groups.

The secant stiffness at 40% of the experimental peak load, $K_{0.4}$, was calculated as in Eq. 2:

$$K_{0.4} = \frac{0.4P_{max}}{\Delta_{0.4}} \quad (2)$$

where P_{max} is the experimental peak axial load and $\Delta_{0.4}$ is the axial displacement corresponding to $0.4P_{max}$. Similarly, the secant stiffness at 80% of the experimental peak load, $K_{0.8}$, was determined as Eq. 3:

$$K_{0.8} = \frac{0.8P_{max}}{\Delta_{0.8}} \quad (3)$$

where $\Delta_{0.8}$ is the axial displacement corresponding to $0.8P_{max}$. The values of $\Delta_{0.4}$ and $\Delta_{0.8}$ were obtained by linear interpolation of the measured ascending branch of the axial load-axial displacement curve. The stiffness values are reported in Table 5.

Table 5. Secant axial stiffness of the tested specimens.

Specimen	P_{max} (kN)	$\Delta_{0.4}$ (mm)	$K_{0.4}$ (kN/mm)	$\Delta_{0.8}$ (mm)	$K_{0.8}$ (kN/mm)
SM	811.37	6.33	51.31	8.51	76.30
SB	705.45	7.54	37.44	9.79	57.64
FM	724.91	7.57	38.30	9.74	59.56
FB	958.48	2.97	129.25	5.96	128.71
LSM	726.67	7.22	40.25	9.86	58.99
LSB	618.76	7.37	33.57	9.26	53.46
LFM	603.87	8.88	27.20	11.28	42.82
LFB	536.56	7.69	27.89	9.65	44.48
SSM	624.59	9.11	27.43	11.42	43.74
SSB	460.83	5.41	34.10	6.54	56.41
SFM	531.38	6.89	30.84	9.35	45.48
SFB	597.40	6.95	34.38	8.74	54.65
HSSM	779.92	7.80	39.97	9.86	63.29
HSSB	607.76	8.02	30.31	9.55	50.93
HSFM	563.10	8.40	26.81	10.52	42.81
HSFB	673.78	8.19	32.90	10.69	50.40
RSM	788.98	6.55	48.20	8.40	75.14
RSB	516.50	8.70	23.73	10.69	38.65
RFM	718.44	7.55	38.07	9.87	58.22
RFB	750.15	7.66	39.16	9.61	62.43
L	592.22	7.51	31.56	9.06	52.27
PC	371.52	4.33	34.35	6.12	48.54

The calculated $K_{0.4}$ values ranged from 23.73 kN/mm for RSB to 129.25 kN/mm for FB, while the $K_{0.8}$ values ranged from 38.65 kN/mm for RSB to 128.71 kN/mm for FB. Specimen FB exhibited the largest secant stiffness at both load levels, consistent with its high peak load and relatively small axial displacement at comparable load levels. In contrast, specimens LFM, LFB, SSM, HSFM, and RSB showed relatively low $K_{0.4}$ values, indicating greater axial deformation before reaching 40% of their respective peak loads.

The stiffness values varied among specimens with similar nominal detailing. Therefore, the observed stiffness response should not be attributed solely to splice type or transverse reinforcement. Differences in concrete compressive strength, local concrete quality, reinforcement location, splice location, and possible residual loading eccentricity may also have influenced the measured response. Because full post-peak descending branches were not consistently recorded, the present study quantifies pre-peak secant stiffness only; post-peak stiffness degradation is discussed qualitatively from the recorded load-displacement curves.

In group 1, the specimens exhibit a steep initial slope and relatively small vertical deflections up to loads of 600-700 kN, reflecting strong confinement and stable elastic behavior; the forged-splice specimen FB shows the highest stiffness, reaching nearly 980 kN at a vertical shortening of only 8 mm. In group 2, where transverse reinforcement was removed from the splice zone, the curves display noticeably larger deflections at comparable loads, with LFM and LFB entering the nonlinear range much earlier and reaching maximum deflections of 10-12 mm at significantly lower loads (500-600 kN). Group 3, incorporating spiral confinement with a 95 mm pitch, showed relatively low pre-peak secant stiffness in several specimens and larger axial displacements at comparable load levels. Because a complete common descending branch was unavailable for all specimens, post-peak stiffness degradation was not quantified. In particular, specimen SSM reached an axial displacement of approximately 12.4 mm, while specimen SFM reached approximately 10.8 mm, both at experimental peak loads below 600 kN. In contrast, specimen SSB reached an axial displacement of approximately 6.9 mm. The larger axial displacements observed in SSM and SFM indicate reduced deformation stability associated with the relatively wide spiral pitch and, potentially, specimen-specific differences in concrete strength and construction quality. In contrast, group 4 demonstrates the beneficial effect of dense spiral reinforcement; specimens like HSSM and HSFM maintain lower deflections throughout loading and reach higher load levels with improved pre-peak secant stiffness and more stable recorded load-displacement responses. However, the response also depended on specimen-specific concrete strength and construction quality; therefore, the effect of reduced spiral pitch should be interpreted as a general trend rather than an isolated variable effect. Square columns in group 5 show intermediate behavior, with RFM and RSM sustaining high loads (700-750 kN) while maintaining controlled deflections around 10-11 mm, reflecting the effectiveness of closed ties in rectangular sections. Finally, the reference columns in group 6 highlight baseline behavior: the plain concrete column PC exhibits a smooth, continuous increase in shortening without a clear peak load, while the continuously reinforced column L shows a more gradual stiffness reduction and reaches deflections above 11 mm at loads around 600 kN. Overall, the vertical load-deflection curves confirm that confinement quality, splice detailing, and concrete integrity strongly influence axial stiffness, deformation capacity, and failure mode across all specimen groups.

3.4. Energy-based ductility index

To quantify the observed differences in deformation capacity and post-peak behavior, an energy-based ductility index, μ_E , was calculated for each specimen from the axial load-axial displacement curves. The index is defined as Eqs. 4 to 6.

$$\mu_E = \frac{E_{total}}{E_{elastic}} \quad (4)$$

while:

$$E_{elastic} = \frac{1}{2} P_{max} \Delta_{max} \quad (5)$$

$$\mu_E = \frac{\int_0^{\Delta_{ref}} P(\Delta) d\Delta}{\frac{1}{2} P_{max} \Delta_{max}} \quad (6)$$

where P_{max} is the experimental peak load and Δ_{max} is the corresponding axial displacement. The reference displacement, Δ_{ref} was defined as the axial displacement at which the load decreased to $0.8P_{max}$ on the descending branch. For specimens that did not exhibit a clear descending branch, Δ_{ref} was taken as the last reliably recorded displacement point.

The calculated energy-based ductility indices are reported in Table 6. Specimens with denser spiral confinement and adequate transverse reinforcement generally exhibited higher μ_E values, reflecting greater energy absorption capacity and more gradual post-peak softening. However, the ductility index also reflects specimen-specific differences in concrete strength and construction quality.

The total absorbed energy ranged from approximately 671 kN·mm for the plain concrete specimen PC to approximately 4320 kN·mm for specimen FB. Specimen FB also exhibited the highest peak load, approximately 958 kN, and the largest normalized energy-absorption ratio, $\mu_E = 1.10$. In contrast, PC showed limited energy absorption because of its lower load capacity and lack of reinforcement. The reinforced control specimen L achieved an intermediate energy absorption of approximately 1925 kN·mm and $\mu_E = 0.59$.

The energy-absorption ratio varied considerably among specimens, ranging from 0.45 to 1.10. For example, LSM and SFM exhibited relatively high values of 0.71 and 0.65, respectively, whereas SSB and HSSB exhibited the lowest values of 0.45. These differences indicate that energy absorption was affected by the combined influence of peak load, axial deformation, splice configuration, transverse confinement, concrete strength, and construction quality. Although some specimens with denser spiral reinforcement showed improved deformation stability, the energy-absorption ratios did not show a uniform trend that could be attributed only to transverse-reinforcement spacing or splice type. Therefore, the energy results should be interpreted together with the peak load, normalized capacity ratio, reinforcement stress, crack width, and observed failure mode.

Table 6. Experimental peak load, axial displacement, absorbed energy, and normalized energy-absorption ratio of the tested reinforced-concrete column specimens.

Specimen	P_{max} (kN)	Δ_{max} (mm)	Δ_{ref} (mm)	E_{total} (kN·mm)	$E_{elastic}$ (kN·mm)	μ_E
SM	811.37	9.84	9.84	2532.99	3989.93	0.63
SB	705.45	10.88	10.88	2105.49	3835.89	0.55
FM	724.91	11.08	11.08	2305.93	4015.99	0.57
FB	958.48	8.21	8.21	4320.11	3936.97	1.10
LSM	726.67	11.40	11.40	2952.59	4142.03	0.71
LSB	618.76	10.48	10.48	1768.24	3240.76	0.55
LFM	603.87	12.64	12.64	2147.72	3817.99	0.56
LFB	536.56	10.46	10.46	1445.46	2807.56	0.51
SSM	624.59	12.40	12.43	2081.25	3873.99	0.54
SSB	460.83	6.88	6.88	709.92	1584.12	0.45
SFM	531.38	10.84	10.84	1872.31	2878.77	0.65
SFB	597.40	9.86	9.86	1632.59	2945.19	0.55
HSSM	779.92	10.86	10.86	2188.25	4233.03	0.52
HSSB	607.76	10.36	10.36	1410.51	3146.66	0.45
HSFM	563.10	11.33	11.33	1602.95	3189.95	0.50
HSFB	673.78	12.10	12.10	2425.30	4078.03	0.59
RSM	788.98	9.34	9.34	2000.21	3684.56	0.54
RSB	516.50	11.55	11.55	1441.66	2982.77	0.48
RFM	718.44	11.25	11.25	2499.18	4041.20	0.62
RFB	750.15	10.50	10.50	1935.69	3938.29	0.49
L	592.22	11.01	11.01	1925.11	3260.19	0.59
PC	371.52	6.48	6.48	670.57	1203.71	0.56

3.5. Comparison with ACI 318 and fib model code provisions

The experimental findings were compared qualitatively with the detailing principles provided in ACI 318-19 [1] and fib model code [28] for reinforced-concrete members containing lap splices. Both design documents recognize that splice performance depends on the bond-transfer capacity between the reinforcement and the surrounding concrete and that transverse reinforcement is important for controlling splitting cracks, restraining lateral dilation, and maintaining the integrity of the concrete surrounding the splice.

ACI 318-19 [1] permits compression lap splices for deformed bars and specifies requirements for lap-splice length, bar size, concrete strength, and transverse reinforcement in column splice regions. For tied columns, ACI 318-19 [1] permits a reduction in compression lap-splice length when ties are provided throughout the lap-splice length in accordance with the relevant code requirements. The results of the present study support the importance of this confinement requirement. The group 2 specimens, in which hoops were omitted over the splice region, generally showed earlier cracking, larger crack widths, and less stable behavior than specimens with transverse reinforcement provided through the splice region. These observations indicate that omission of transverse reinforcement can reduce the confinement effectiveness and increase the vulnerability of the splice region to cracking and localized concrete deterioration.

The fib model code [28] similarly recommends locating splices in regions of relatively low reinforcement stress where possible and emphasizes the role of transverse reinforcement in controlling splitting forces associated with bond transfer. For bars with diameters greater than 12 mm, the fib model code [28] recommends staggering lap splices, where possible, so that no more than half of the reinforcement force is transferred through splices at a single cross-section. It also specifies minimum transverse reinforcement requirements for certain lap-splice conditions and highlights the importance of placing confining reinforcement close to the lapped bars. The present results are consistent with this general design approach, as the specimens with improved spiral confinement or closely spaced transverse reinforcement generally exhibited more stable cracking patterns and improved deformation behavior compared with specimens having insufficient confinement in the splice region.

The results also demonstrate that peak axial load alone is not sufficient to evaluate splice performance. Some forged-splice specimens achieved high peak loads; however, their reinforcement stresses, crack development, and energy-absorption response varied depending on transverse-reinforcement configuration, concrete strength, and fabrication quality. Similarly, lap-splice specimens did not exhibit uniformly superior response under all conditions. Therefore, consistent with the philosophy of both ACI 318 [1] and fib model code [28], the experimental results indicate that splice performance should be evaluated together with concrete quality, transverse confinement, reinforcement stress demand, and the intended structural performance level.

It should be noted that the specimens were tested under monotonic concentric axial compression, whereas code provisions are

intended for a wider range of structural conditions, including members subjected to combined axial load and bending and, in seismic applications, cyclic inelastic deformation. Accordingly, the present findings should not be interpreted as a direct validation or modification of ACI 318-19 [1] or fib model code [28] design provisions. Instead, they provide experimental evidence that supports the general code requirement for adequate transverse reinforcement and careful detailing of splice regions in reinforced-concrete columns.

4. Conclusions

The experimental program conducted in this research involved a comprehensive series of axial compression tests on reinforced-concrete columns containing different splice types, transverse reinforcement configurations, and cross-sectional shapes. The specimens were designed to represent practical construction conditions, including columns with lap splices, flash-butt (forging) splices, individual ties, circular hoops, and spiral reinforcement with varying pitches. The response of the specimens was evaluated using experimental peak load, theoretical capacity, cracking load, axial and lateral displacement, crack width, failure length, local reinforcement stress, concrete surface strain, secant stiffness, and energy absorption. Based on the experimental results, the following conclusions are drawn:

1. Although the columns containing forging (flash-butt welded) splices and those with lap splices exhibited similar behavior in terms of visual and observable performance, the monitoring data obtained from various sensors installed on the longitudinal bars, transverse reinforcement, and concrete indicate meaningful differences. These differences highlight the importance of careful consideration when selecting between the two splice types.
2. Transverse confinement was a major factor influencing the axial response of the tested columns. The specimens with adequate hoops, closed ties, or spiral reinforcement generally exhibited more stable load-displacement behavior, better crack control, and more localized damage than specimens with insufficient confinement in the splice region. The experimental findings confirm that splice performance depends strongly on the quality and continuity of transverse reinforcement.
3. Omitting transverse reinforcement in the splice zone adversely affected the observed response. The group 2 specimens generally showed earlier cracking, wider cracks, and less stable deformation than the specimens with transverse reinforcement maintained through the splice length. The maximum crack width of 3.0 mm was recorded for specimen LSM, which had a lap splice at mid-height without hoops in the splice region. This result emphasizes the importance of transverse reinforcement for controlling splitting cracks and preserving the integrity of the concrete surrounding the splice.
4. Reducing the spiral pitch from 95 mm to 50 mm generally improved confinement effectiveness. The group 4 specimens with a 50 mm spiral pitch showed improved cracking resistance, more stable recorded axial deformation, and generally shorter or more localized failure zones than several group 3 specimens with a 95 mm spiral pitch. Nevertheless, the effect was not identical for every specimen because concrete strength, local concrete quality, splice location, and fabrication conditions also influenced the response.
5. At ultimate load, only one circular column with 95-mm hoop spacing and a bottom lap splice (specimen SB) reached stresses in the longitudinal bars exceeding the yield limit. In this specimen, the concrete strain at failure was approximately 0.0014, which is below the strain limits commonly assumed in design codes (0.003–0.0035). This suggests that columns with longitudinal reinforcement ratios near the upper code limit ($\rho \approx 8\%$) may require further investigation, as the expected ultimate concrete strain may not be achieved uniformly along the height. Additional testing is recommended.
6. The secant stiffness results quantified the pre-peak deformation response of the specimens. The stiffness at 40% of the experimental peak load, $K_{0.4}$, ranged from 23.73 kN/mm for RSB to 129.25 kN/mm for FB. The corresponding stiffness at 80% of peak load, $K_{0.8}$, ranged from 38.65 kN/mm for RSB to 128.71 kN/mm for FB. Specimen FB showed the highest stiffness at both load levels, whereas LFM, LFB, SSM, HFSM, and RSB showed comparatively low pre-peak stiffness. Since a complete and consistent post-peak descending branch was unavailable for all specimens, post-peak stiffness degradation was not quantified consistently.
7. The capacity results should be interpreted together with the measured concrete strength. The concrete compressive strength varied among the specimens; consequently, direct comparison based on peak load alone does not isolate the effect of splice type or confinement detail. The experimental-to-theoretical capacity ratio, together with stiffness, crack width, deformation, reinforcement stress, and energy absorption, provides a more reliable basis for comparing specimen performance.
8. Neither lap splices nor flash-butt-forged splices showed universally superior performance in all test configurations. Some forged-splice specimens, particularly FB, achieved high peak load, high secant stiffness, and high energy absorption. However, other forged-splice specimens showed lower capacity or less stable deformation. Likewise, lap-splice specimens did not consistently provide higher peak capacity than forged-splice specimens. The splice response was governed by the combined effects of splice type, splice location, transverse confinement, concrete quality, and fabrication quality.
9. The reduced longitudinal bar stress in specimen RFM compared to RSM, despite neither specimen reaching yield in the longitudinal bars, is explained in part by differences in transverse reinforcement behavior and concrete strength. The ties in both specimens exceeded yield, reaching 425 MPa in RFM and 415 MPa in RSM. Although RSM carried approximately

2.5% more than its theoretical ultimate load, RFM experienced a 12% reduction relative to its theoretical capacity. These differences highlight the sensitivity of forged-splice specimens to transverse-reinforcement demand and concrete quality. In this specific comparison, the lap-splice specimen exhibited a more favorable combination of capacity and reinforcement stress, but this trend should not be generalized to all forged- and lap-splice configurations without considering confinement, concrete strength, and fabrication quality.

10. At ultimate load, transverse reinforcement stresses in many forged-splice specimens were higher than in comparable lap-splice specimens, except in specimens SFM and LFB. The lower transverse steel stress in several lap-splice columns indicates a reduced demand on the confinement reinforcement under axial compression, which can be beneficial for ductility and post-peak stability. However, this trend was not uniform across all specimens and should be interpreted together with the effects of confinement configuration, spacing, and concrete strength.
11. Cracking in all columns (square and circular, with any transverse reinforcement type or spacing) containing either forging or lap splices occurred after the service load, except in group 2. Thus, both splice types can safely sustain service loads without cracking. The premature cracking in group 2 is attributed to the absence of transverse reinforcement over the 300-mm splice region.
12. The cracking observations show that both lap-splice and forged-splice columns generally remained uncracked under the stated service-load level when adequate transverse reinforcement was provided. In contrast, the specimens without transverse reinforcement in the splice region showed premature cracking. This result confirms that confinement detailing in the splice zone is essential for serviceability as well as ultimate-response performance.
13. The energy-based results indicate substantial variation in the deformation and load-carrying response among specimens. The normalized energy-absorption ratio, μ_E , ranged from 0.45 to 1.10. Specimen FB exhibited the largest value, whereas SSB and HSSB exhibited the lowest values. The results show that energy absorption was affected by the combined influence of load capacity, axial deformation, confinement arrangement, splice detailing, concrete strength, and construction quality. Therefore, the energy-based index should be considered together with the other measured response parameters.
14. The conclusions of this study are limited to short columns subjected to monotonic concentric axial compression. The behavior under combined axial load and bending, cyclic seismic loading, fatigue, repeated loading, and full-scale structural conditions was not investigated. In addition, variation in concrete strength, possible local construction defects, and the absence of direct measurement of initial loading eccentricity contributed to specimen-to-specimen scatter. Further tests with repeated specimens, controlled material properties, documented splice-quality testing, and cyclic axial-flexural loading are recommended.

Statements & declarations

Author contributions

Mohammad Maghsoudi: Conceptualization, Methodology, Validation, Visualization, Investigation, Writing – Original Draft & Editing.

Amirreza Toghrol Pour: Writing - Review & Editing, Visualization, Data curation, Investigation.

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Data availability

The experimental data supporting the findings of this study are available from the corresponding author upon reasonable request.

Declarations

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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