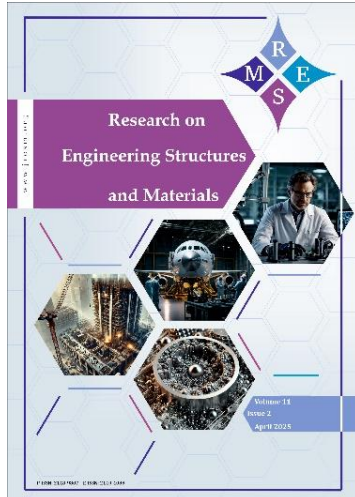




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Shear behavior and failure mechanisms of reinforced concrete beams with construction joints in the shear span

Falah J. Alkhafaji ^{1,a}, Mohammed A. Jennam ^{1,b}, Aws N. Yaseen ^{1,c}, Kumail R. Al-khafaji ^{*1,d}, Amer F. Izzet ^{2,e}

¹Civil Engineering Department, University of Shatrah, Dhi Qar, Iraq

²College of Engineering, University of Baghdad, Baghdad, Iraq

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Abstract

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This study investigates the influence of construction-joint geometry within the shear span on cracking, stiffness, deformation, energy absorption, and ultimate capacity of reinforced-concrete T-beams. Four beams were compared under three-point loading over a 1,000 mm net span (shear span, $a = 500$ mm; $a/d = 2.94$): a monolithic reference beam and beams containing vertical, inclined, and keyed construction joints in the shear span. All beams had identical reinforcement and a measured concrete cube compressive strength of 30 MPa. Relative to the monolithic reference beam, the jointed beams exhibited first-crack-load reductions ranging from 0.0% to 15.4% and ultimate-load reductions ranging from 0.9% to 4.5%. The maximum reductions in first-crack load and ultimate load were 15.4% and 4.5%, respectively, indicating that joint geometry affected cracking resistance more strongly than ultimate resistance. At 50 kN, the vertical, inclined, and keyed jointed beams exhibited midspan deflections that were 20.0%, 8.0%, and 3.0% greater than that of the reference beam, respectively. The keyed joint retained 97.1% of the reference secant stiffness at 50 kN and 99.1% of the reference ultimate load. Overall, the keyed configuration most closely reproduced monolithic behavior, plausibly because its geometry increased the effective contact area and mechanical interlock, whereas the vertical joint produced the largest stiffness and deformation penalty.

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1. Introduction

In the construction of concrete, it may be necessary to have a construction junction when casting is stopped at one point and resumed at a later period. It is important in these conditions that the deflections of the linked portions of the member are compatible, as failure of the joint can result in separation and the structural member can be effectively divided into two portions [1]. It is therefore necessary to pick the position of the construction joint and to carry it out in such a way that the structural integrity and functional performance of the member are not compromised [2]. In some circumstances the joint is produced at a site beyond the direct control of the contractor; therefore, the adequacy of the resulting structural member must be carefully reviewed. A construction joint is a potentially weak horizontal or vertical junction within an otherwise monolithic concrete element that may allow relative sliding between the adjoining surfaces [3]. The selection of an appropriate joint location is an important responsibility shared by the design engineer and the contractor [4,5].

Many reinforced-concrete structures are not cast in a single operation and so have construction joints, which are necessary because of construction sequencing, concrete supply limitations, or

*Corresponding author: kumailr@shu.edu.iq

^aorcid.org/0009-0008-1512-8759; ^borcid.org/0009-0004-1473-5064; ^corcid.org/0009-0001-5621-0440;

^dorcid.org/0000-0001-5536-7238; ^eorcid.org/0000-0002-1506-3471

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practical casting restrictions. Within the shear span, a joint cuts through the concrete matrix in an area of high shear and inclined main stress. Thus, the interface could intersect the key diagonal-crack path or disrupt the compression field between the loading point and support, resulting in an increase in deformation or premature interface-controlled damage. After the first bond fails, shear transfer across the joint is largely provided by friction, aggregate interlock, compression normal to the interface and the dowel and clamping actions of reinforcement spanning the joint. Their efficiency depends on joint position and orientation, surface preparation and interface geometry. Laitance removal, cleaning, moisture conditioning and roughening improve contact and resistance to sliding, while a keyed interface can add bearing and mechanical interlock.

The literature thus identifies four interacting control variables: the joint location, orientation, interface condition and reinforcement/detailing. Their effects are linked since each variable influences the stress state on the contact and the potential mechanisms for shear transfer [4,6–9]. After bond loss, reinforcement spanning or enclosing the joint inhibits opening, provides clamping action, and facilitates frictional transfer. Interface geometry has a role through bearing and mechanical interlock, notably in keyed joints. These mechanisms can influence serviceability and ultimate resistance differently. An interface can be sufficiently strong at failure yet still increase deformation or reduce stiffness during operating loads.

In the T-section, the web carries most of the shear while the flange carries compression and adds to the flexural rigidity. Under three-point loading, a joint in the shear span can cross the diagonal tension field and interrupt the load point-to-support force transmission path. These properties allow for the isolation of the effect of joint geometry, since the concrete strength, beam size, reinforcing and loading arrangement are kept constant. Monks and Sadgrove [10] observed that smooth construction joints did not perform well in high shear areas, whereas rough joints are more like continuous concrete. The existence of vertical joints was shown to diminish the modulus of rupture of the plain concrete, implying that the interface can be a preferential path for cracking under tension [11]. This set of results explains why the presence of a joint located inside the shear span may induce cracking or enable further sliding.

Joint location determines the stress state acting on the interface. A joint in a low-shear region may have little influence on overall behavior, while a joint between a concentrated load and the adjacent support can intersect the diagonal tensile field or the compression strut. Monks and Sadgrove [10] identified this location effect experimentally. Abdulmunaam and Farhan [8] likewise showed that construction-joint location affected the capacity, deformation, and failure mode of reinforced self-compacting concrete beams. Joint orientation controls the combination of shear and normal stress acting on the interface and its alignment with the probable diagonal-crack path. Akin and Güz [7] found that the angle of a cold joint in the shear zone affected crack development and beam capacity. Their findings also indicated that reinforcement provided near the interface reduced the adverse effect of the joint. Jabir et al. [6] reported that inclined construction joints exhibited strength losses that depended on joint inclination and the stress acting on the interface.

Reinforcement and interface geometry influence residual resistance following bond loss. Stirrups and other bars that span or confine the joint limit opening, create clamping action, and promote frictional transfer. According to Abdulmunaam and Farhan [8], increasing secondary reinforcement limited joint separation and made the response more ductile, while increasing longitudinal reinforcement without increasing interface and shear resistance could shift failure toward shear or joint separation. These findings suggest that joint performance cannot be judged in isolation of the surrounding reinforcement.

Keyed joints provide an additional load-transfer mechanism through bearing on the key surfaces and mechanical interlock at the interface. Abass [4] discovered that adequately specified keyed interfaces might approach monolithic beam behavior; however, vertical and inclined joints produced larger reductions under less favorable conditions. Horizontal construction joints showed higher deflection and different flexural response, suggesting that interface compliance might be more influential on serviceability than ultimate resistance Al-Rifaie et al., [12].

Further experimental research emphasizes the importance of the interface conditions and joint position. Aziz and Ajeel [13] examined reinforced-concrete T-beams with construction joints and

flange apertures that were simply supported. The specimen with a cold joint (defined as an interface with no specific treatment and material enhancement) showed about a 27% drop in shear strength, stressing the susceptibility of untreated joints in shear critical regions [13]. In another structural system, Bin Osman et al. [14] investigated six concrete slabs with different types of construction joints. The monolithic control had the highest overall load, stiffness, and energy response. The inclined-joint slab exhibited better load and energy capacity than the other jointed arrangements, while the Hy-Rib Mesh joint had the highest stiffness among the jointed slabs [14].

Other evidence has looked at vertical joints, specific interface materials and the positioning of joints. A study by [15] on singly reinforced beams with vertical construction joints produced charts to estimate the losses in flexural capacity in normal-strength concrete [15]. Jang et al. [16] showed that grooved UHPC interfaces with steel-fiber reinforcement may significantly improve joint shear resistance. They stressed the importance of interface geometry and material improvement [16]. Mathew and Nazeer [17] revealed that for the tested 20 and 40 MPa concrete mixes, the beams with joints in the middle third of the span had a somewhat higher load-carrying capability than beams with joints extending into the outer third [17]. These results collectively support that the structural implications of construction joints are contingent on the stress field at the joint, in addition to interface details and material properties.

Additional beam studies have addressed joint elevation, concrete type, and shear resistance. Abbas et al. [14] examined reinforced-concrete beams with longitudinal construction joints at different elevations and found that the ultimate load, first-crack load, and stiffness were affected by joint elevation. Ismael et al. [15] investigated the structural performance of self-compacting-concrete beams with construction joints; the influence on ultimate load was greater than that on first-crack load, and horizontal joints showed higher structural efficiency. Ibrahim et al. [16] investigated reinforced-concrete beams without shear reinforcement and reported that a vertical construction joint at mid-span reduced shear capacity by up to 10%, whereas an interface perpendicular to the main shear crack increased shear strength by more than 30%.

Recent studies have broadened the evidence base beyond conventional beam configurations. Abdul-Majeed et al. [18] considered the effect of the number of horizontal construction joints in reinforced-concrete beams, while Kadhum and Al-Zuhairi [19] examined construction joints in reinforced-concrete deep beams. Över et al. [20] studied the combined effects of cold-joint location and angle, and Feng et al. [21] evaluated the shear behavior of segmented cast UHPFRC beams with and without stirrups. Huang et al. [22] quantified the respective contributions of cohesion, friction, and dowel action at concrete interfaces. Together with the early shear tests of Clark and Gill [23] and the joint-detailing guidance in ACI 224.3R [17], these studies further show that interface response is governed by both joint configuration and the mechanisms available to transfer shear.

The literature review shows that the presence of construction joints can cause a reduction in the cracking resistance, stiffness, deformation capacity and ultimate capacity, but the magnitude of the reduction and the dominant failure mechanism, if any, depend on the location, orientation, surface condition, crossing reinforcement, shear-span geometry and loading scheme of the joint. It is also indicated that serviceability penalties may be observed prior to significant losses in ultimate resistance. However, direct comparison of vertical, inclined and keyed joints in the same shear span is still limited since the published studies generally vary many parameters at the same time or consider just one interface type. The present work aims to fill this gap by testing T-beams with same dimensions, material attributes, reinforcement and loading setup but with different joint configuration. The load-deflection response, secant stiffness, deformation, energy absorption and ultimate load are assessed to determine which design best mimics monolithic behavior.

2. Materials and Method

The beams were designed as reinforced-concrete T-sections with a flange width of 200 mm, a web width of 100 mm, an effective depth of 170 mm, and a flange thickness of approximately 50 mm, as shown in Figure 1. The overall beam length was 1,200 mm, with a 1,000 mm net span between supports. Under the central point load, the shear span was 500 mm and the shear-span-to-effective-depth ratio was $a/d = 2.94$. The measured concrete cube compressive strength was 30 MPa. Two

10 mm diameter longitudinal tension bars, with a total area of 157 mm^2 and a yield strength of 550 MPa tested according to [18], were provided. The shear reinforcement consisted of 8 mm diameter stirrups spaced at 100 mm center-to-center with a yield strength of 430 MPa tested according to [19].

The comparative test matrix comprised four beams: a monolithic reference beam and three beams containing vertical, inclined, and keyed construction joints in the shear span. All beams had the same geometry, reinforcement, material properties, and three-point loading arrangement. The assessment therefore isolates the effect of joint geometry on crack initiation, shear transfer, stiffness, deformation, energy absorption, and ultimate capacity. Only these four variants are included in the stated cracking, bending, and shear calculations, as well as the comparisons that follow. Concrete was produced with ordinary Portland cement. The recorded mix design, fresh-concrete slump, and $150 \times 150 \times 150 \text{ mm}$ cube-test results are summarized in Table 1. Detailed aggregate gradation was not retained in the available experimental record and is therefore not reported; the maximum aggregate size was 20 mm.

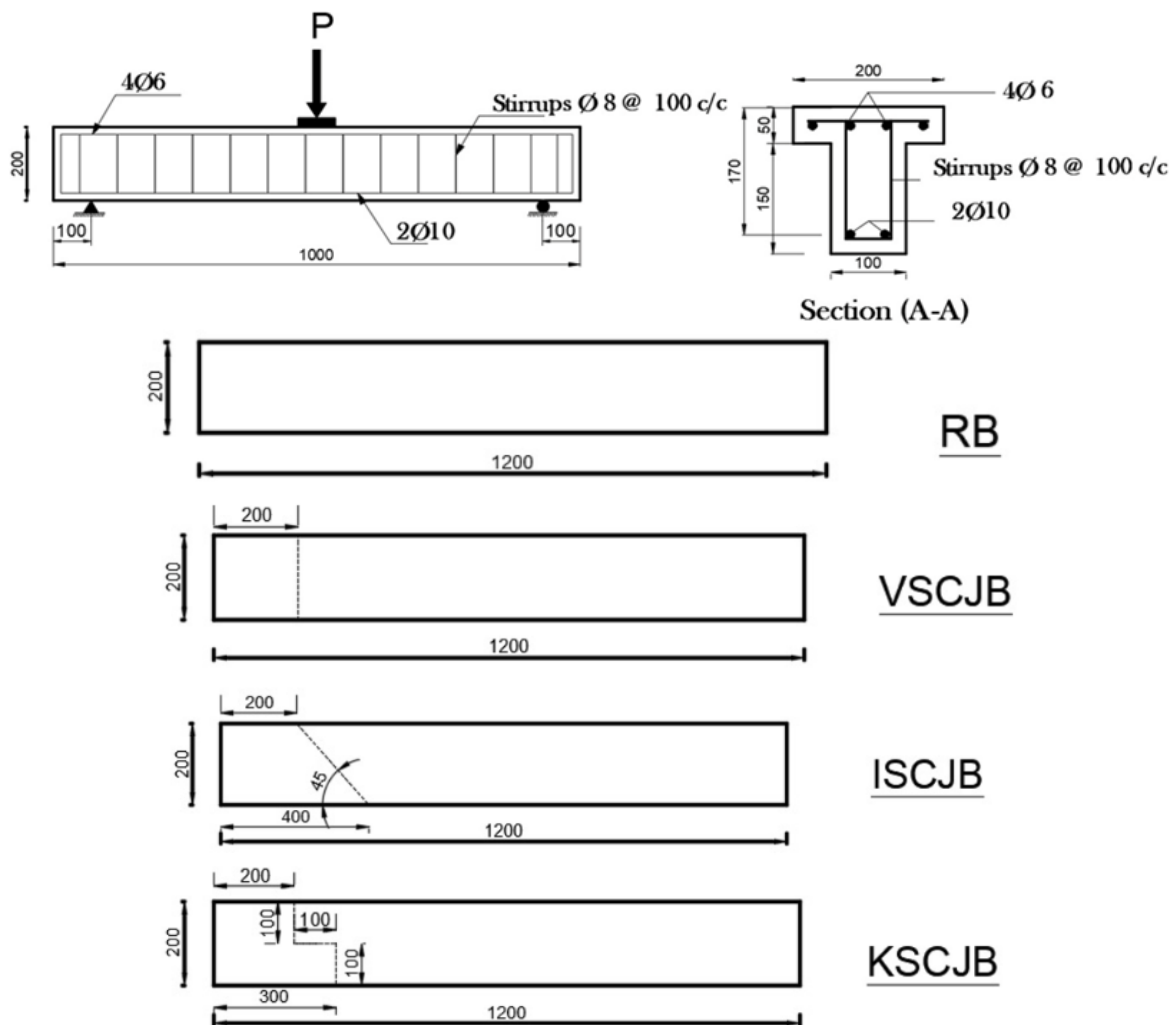
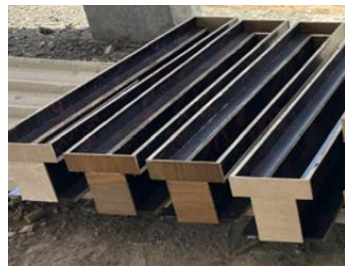


Fig. 1. Reinforced-concrete T-beam geometry, reinforcement, material parameters, joint layouts, and three-point loading; KSCJB key dimensions: height = 100 mm, width = 100 mm, recess = 100 mm



(a) Wooden formwork



(b) Reinforcement cages before concrete placement



(c) Initial concrete placement and companion cube molds



(d) Wet-burlap covering used for moist curing



(e) Specimens retained under wet burlap after casting



(f) Staged casting showing exposed reinforcement before subsequent placement

Fig. 2. Photographic record of reinforcement-cage assembly, first-stage concrete placement, wet-burlap curing, and staged casting (a-f)

Table 1. Recorded concrete mix proportions and cube-test results

Mix-proportion data (per 1 m ³)	Value	Concrete Cubes Results [20]						
Cement [21]	385 kg	Stage	Specimen	Mass (kg)	Unit wt. (kg/m ³)	Failure load (kN)	Strength (MPa)	Age (days)
Fine aggregate (sand)	820 kg	Stage 1	1	7.845	2324	465.75	20.7	7
Coarse aggregate (gravel)	965 kg	Stage 1	2	7.810	2314	474.75	21.1	7
Water	192.5 kg	Stage 1	3	7.800	2311	477.20	21.2	7
Admixture	0 L	Stage 1	4	7.820	2317	666.10	29.6	28
Total mix mass	2362.5 kg	Stage 1	5	7.790	2323	677.25	30.1	28
Water-cement ratio	0.50	Stage 1	6	7.770	2306	681.75	30.3	28
Maximum aggregate size	20 mm	Stage 2	1	7.84	2322	468.2	20.8	7
Fresh-concrete slump [22]	190 ± 20 mm	Stage 2	2	7.805	2316	472.40	21.0	7
		Stage 2	3	7.795	2312	479.10	21.3	7
		Stage 2	4	7.815	2318	664.80	29.5	28
		Stage 2	5	7.785	2321	675.60	30.0	28
		Stage 2	6	7.765	2308	684.20	30.4	28

To form the construction joints, the second concrete placement was made 48 h after the first. Before recasting, the hardened surface was cleaned with a steel wire brush to remove laitance/cement slurry and then mechanically roughened (chipped) and moistened. The specimens were wet-cured for 7 days. The inclined construction joint was formed at 45° to the beam longitudinal axis. The joint layouts are shown and documented in Figure 1 and 2.

For the comparative evaluation, secant stiffness on a given load was determined as $K = P/\delta$. The energy absorption was calculated by trapezoidal integration of the load-deflection response. The

average increase in deflection was calculated by utilizing the relative deflection increments at 15, 30, and 50 kN, employing the deformation-capacity index (δ_u/δ_{30}).

3. Results and Discussion

3.1 Load-Deflection Response

The load-deflection curves indicate that the four specimens exhibited generally comparable behavior during the initial loading phase, when differences in deformation were small. Nonlinearity began after first visible cracking, which occurred at 11.0–13.0 kN. The separation among the curves became more pronounced as the load increased beyond approximately 25–30 kN. VSCJB deflected the most, followed by ISCJB, whereas KSCJB remained close to the reference beam (Figure 3).

For the tested beams, the first visible crack was identified visually as a flexural crack in the midspan region, rather than as an interface or diagonal-shear crack at the construction joint. First-crack loads were identified by visual observation, and the corresponding midspan deflections were extracted from the original test records. Table 5 reports the principal performance metrics, including ultimate deflection, while Table 6 reports the first-visible-crack load and deflection. VSCJB reached first visible cracking at the lowest load (11.0 kN) and the greatest crack deflection (2.08 mm), whereas ISCJB reached 12.0 kN and 2.00 mm. KSCJB matched RB in first-crack load (13.0 kN) and had a crack deflection of 1.84 mm, close to the RB value of 1.75 mm. At failure, the ultimate deflections were 14.0, 15.2, 15.0, and 14.2 mm for RB, VSCJB, ISCJB, and KSCJB, respectively. Relative to RB, crack deflection increased by 18.9% for VSCJB, 14.3% for ISCJB, and 5.1% for KSCJB; the corresponding ultimate-deflection increases were 8.6%, 7.1%, and 1.4%. The larger crack and ultimate deflections of VSCJB and ISCJB indicate more compliant behavior; however, these deformation values must be interpreted with their lower first-crack and failure loads rather than as improved structural performance.

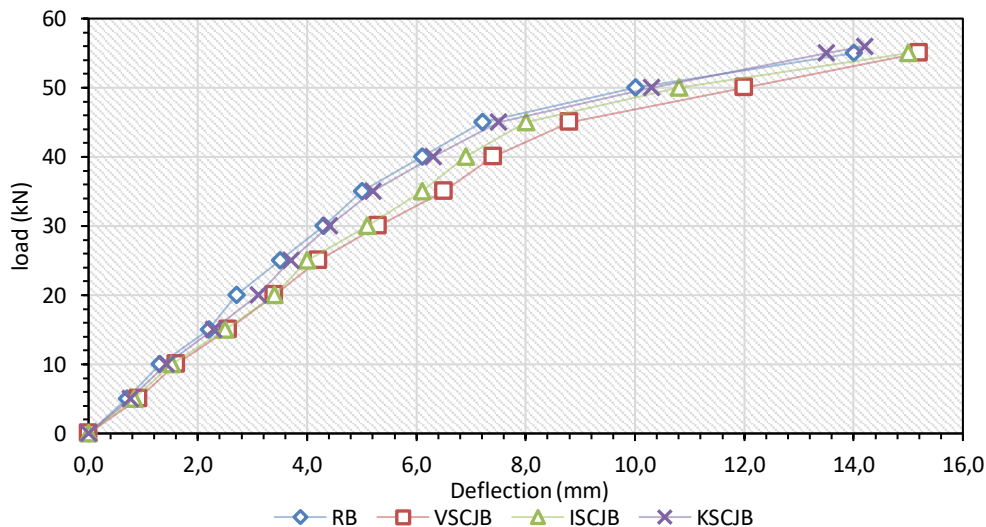


Fig. 3. Comparison of reference and construction-joint beam load-deflection responses

At 50 kN, the recorded deflections were 10.0, 12.0, 10.8, and 10.3 mm for RB, VSCJB, ISCJB, and KSCJB, respectively. The high-load stiffness penalty was greatest for VSCJB, which deflected 20.0% more than RB. VSCJB and ISCJB exhibited greater deformation, consistent with reduced composite action and increased sliding or opening along the construction-joint interface. KSCJB exhibited better stress transfer across the joint. All the curves became nonlinear after first evident cracking at 11.0–13.0 kN, and the response further steepened as the load reached 45 kN, which indicated rapid deformation of the specimen approaching failure. Corresponding ultimate deflections are described in Table 5 and first-crack deflections are provided in Table 6.

3.2 Failure-Load Comparison

The Failure load of RB was the highest (56.0 kN) as seen in the Figure 4. The failure loads of VSCJB, ISCJB and KSCJB were 53.5, 54.0 and 55.5 kN respectively, which are reduced by about 4.5%, 3.6%

and 0.9% respectively. VSCJB exhibited the largest capacity loss, indicating less effective stress transmission across the vertical interface. KSCJB maintained 99.1% of the reference capacity, showing that the keyed configuration preserved structural integrity and mechanical interlock.

The failure load of VSCJB reduced by 4.5% which is in agreement with 0-5% failure-load reduction reported by Abass [4] for vertical construction joints. The ISCJB reduction of 3.6% is slightly higher than the 1.25–2.5% range provided by [6] for favorable inclined-joint configurations, but lower than 8-20% reduction for some inclined joints with high interface requirements as suggested by [4]. These comparisons show that the joint effectiveness depends on the interaction of position, orientation, surface texture, reinforcement and crack path. Overall, the effect of construction joint geometry on failure capacity was restricted with the keyed joint performing the best.

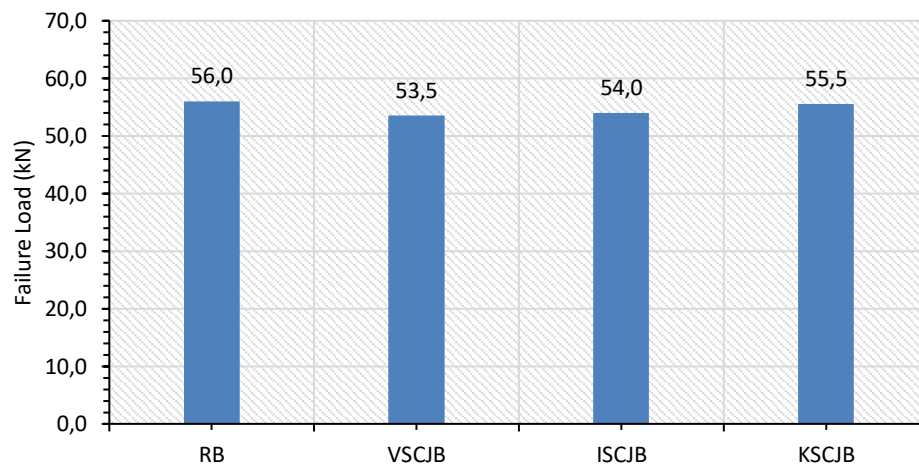


Fig. 4. Reference and construction-jointed beam failure loads

3.3 Cracking, Bending, And Shear Response

The first apparent crack in the monolithic reference beam was at 13.0 kN with an ultimate load of 56.0 kN (Table 2). The three jointed beams fractured between 11.0 and 13.0 kN, which is equivalent to first-crack-load reductions of 0.0-15.4%. Their ultimate loads were between 53.5 and 55.5 kN corresponding to lower reductions of 0.9-4.5%. These values are different indicating that for the shear-span designs evaluated, construction joints had a greater effect on crack initiation and serviceability than on ultimate load capacity.

The vertical shear-span jointed beam had the lowest first-crack load of 11.0 kN and the lowest ultimate load of 53.5 kN. Its reduction of 15.4% in first visible-crack load was greater than its reduction of 4.5% in ultimate load. Consequently, while the early apparent crack did not lead to a proportional loss of ultimate capacity, it is structurally significant as it shows earlier deterioration of serviceability. Because the crack-initiation location and specimen-to-specimen tensile-strength variability were not measured or replicated, the observed difference should not by itself be attributed solely to the interface.

The performance improved progressively from the vertical to the inclined and keyed joint types throughout the shear span. The vertical joint fractured at 11.0 kN and had a maximum failure load of 53.5 kN, whereas the inclined joint cracked at 12.0 kN and had a maximum failure load of 54.0 kN. The keyed joint was observed to crack at 13.0 kN, which was the same as the reference beam, and it failed at 55.5 kN. The near-monolithic response demonstrates that the keyed design provided an effective shear-transfer path across the interface and improved the mechanical interlock. The larger effective contact area of the keyed arrangement may possibly explain its higher capacity by providing a bigger surface for bearing, friction and interfacial shear transmission.

The moment and shear indices in Table 3 provide an equivalent interpretation. The reference and keyed shear-span beams exhibited a cracking moment of 3.25 kN.m and an average cracking web shear stress of 0.382 MPa. At failure, the keyed joint reached an ultimate moment of 13.875 kN.m

and an average web shear stress of 1.632 MPa, close to the reference values of 14.0 kN.m and 1.647 MPa. In contrast to code-based design capacity, these average shear-stress statistics are experimental indices based on the web width and effective depth.

Overall, the findings indicate that joint geometry determines the balance of fracture resistance, deformation control, and ultimate capacity. The limited ultimate-load reductions are consistent with continued shear transfer after cracking through friction, aggregate interlock, clamping by reinforcement crossing the interface, and dowel action. These mechanisms were not measured directly in the present tests; therefore, this interpretation is qualitative rather than a quantified mechanism attribution. Evaluation of construction-joint performance should compare the measured first-crack load, deflection, stiffness, and ultimate capacity rather than rely on a single derived indicator.

Table 2. Cracking and capacity metrics of the tested beams

Joint configuration	First-crack load (kN)	Ultimate load (kN)	Reduction in first-crack load (%)	Reduction in ultimate load (%)
Monolithic reference beam	13.0	56.0	0.0	0.0
Vertical joint in shear span	11.0	53.5	15.4	4.5
Inclined joint in shear span	12.0	54.0	7.7	3.6
Keyed joint in shear span	13.0	55.5	0.0	0.9

Table 3. Experimental bending and shear actions derived from the test loads

Joint configuration	Cracking moment (kN·m)	Ultimate moment (kN·m)	Cracking shear (kN)	Ultimate shear (kN)	Cracking shear stress (MPa)	Ultimate shear stress (MPa)
Monolithic reference beam	3.250	14.000	6.50	28.00	0.382	1.647
Vertical joint in shear span	2.750	13.375	5.50	26.75	0.324	1.574
Inclined joint in shear span	3.000	13.500	6.00	27.00	0.353	1.588
Keyed joint in shear span	3.250	13.875	6.50	27.75	0.382	1.632

3.4 First-Crack Response and Serviceability Implications

Table 4 isolates the first-crack response, which is the most direct experimental indicator of the service-stage effect of the construction joint. Relative to the monolithic reference, first-crack-load retention ranged from 84.6% to 100.0%. The associated cracking moment ranged from 2.750 to 3.250 kN.m, confirming that joint geometry changed the tensile continuity available before the formation of a visible crack.

The first-crack data reveal a clear ranking that is not fully visible from ultimate load alone. The keyed joint in the shear span fully retained the reference cracking load of 13.0 kN. The inclined joint retained 92.3% of the reference cracking load, whereas the vertical joint retained 84.6%, demonstrating the greatest reduction in crack resistance among the tested joint details. This pattern demonstrates that early cracking was influenced by the continuity and orientation of the interface within the shear-transfer path. A reduction from 13.0 to 11.0 kN represents a 15.4% observed decrease in first visible-crack load, even though a substantial proportion of the ultimate load remained available. First-crack response should therefore be treated as a primary serviceability criterion. The keyed shear-span configuration provided the most reliable first-crack performance because it reproduced the reference cracking load while also maintaining near-reference stiffness, deformation, and ultimate capacity.

Table 4. First-crack response and relative serviceability classification of the tested beams

Joint configuration	First-crack load (kN)	Retention vs. reference (%)	Decrease vs. reference (%)	Cracking moment (kN·m)	Relative serviceability response
Monolithic reference beam	13.0	100.0	0.0	3.250	Reference level
Vertical joint in shear span	11.0	84.6	15.4	2.750	Moderate
Inclined joint in shear span	12.0	92.3	7.7	3.000	High
Keyed joint in shear span	13.0	100.0	0.0	3.250	Reference level

3.5 Deflection, Stiffness, And Energy Assessment

Figure 5 illustrates the deflections of the examined beams under loads of 15, 30, and 50 kN. At a load of 15 kN, the deflections varied between roughly 2.2 and 2.5 mm, suggesting very minor discrepancies in the initial response.

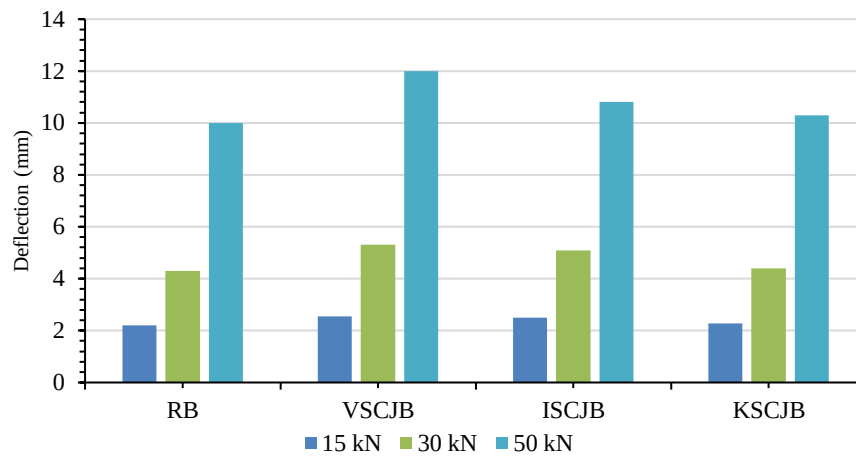


Fig. 5. Measurements of beam deflections under varying load intensities

At 30 kN, VSCJB and ISCJB recorded deflections of 5.3 and 5.1 mm, respectively, compared with 4.3 mm for RB, whereas KSCJB remained close to the reference value at 4.4 mm. Relative to RB, these values represent deflection increases of 23.3% for VSCJB, 18.6% for ISCJB, and 2.3% for KSCJB. Because 30 kN is below the ultimate-load range and closer to the service-load range examined here, these differences highlight the serviceability penalty associated with the vertical and inclined interfaces. At 50 kN, VSCJB exhibited the maximum deflection of 12.0 mm, exceeding RB by 20.0%. ISCJB and KSCJB reached 10.8 and 10.3 mm, representing increases of 8.0% and 3.0%, respectively. Consequently, the vertical joint produced the greatest serviceability penalty, whereas the keyed joint performed comparably to the monolithic beam.

Table 5. Key performance metrics, including ultimate deflection, for reference and construction-jointed beams

Specimen	δ at 30 kN (mm)	δ at 50 kN (mm)	Failure load (kN)	Ultimate deflection (mm)	K retention at 30 kN (%)	Energy to failure (J)
RB	4.30	10.00	56.0	14.0	100.0	522.0
VSCJB	5.30	12.00	53.5	15.2	81.1	532.3
ISCJB	5.10	10.80	54.0	15.0	84.3	539.7
KSCJB	4.40	10.30	55.5	14.2	97.7	522.6

The comparison results show that construction-joint geometry affected stiffness and deformation more clearly than total energy absorption. The keyed specimen (KSCJB) was closest to the

monolithic reference response. Its stiffness retention was 97.7% at 30 kN and 97.1% at 50 kN, and its average deflection increase across the selected load levels was 3.0%. Its ultimate secant stiffness was 3.91 kN/mm, compared with 4.00 kN/mm for RB.

Table 6. First-visible-crack load and deflection

Specimen	First-visible-crack load (kN)	Deflection at first visible crack (mm)
RB	13.0	1.75
VSCJB	11.0	2.08
ISCJB	12.0	2.00
KSCJB	13.0	1.84

Moreover, the deformation-capacity index of KSCJB was comparable to that of RB (3.23 vs. 3.26). The results revealed that the keyed interface yielded composite action with the relative slide limited by the mechanical interlock along the joint.

VSCJB presented the highest sanctions in terms of stiffness and serviceability. Its stiffness retention at 30 kN was 81.1% according to the deflections of 5.30 mm for VSCJB and 4.30 mm for RB. At 50 kN, the stiffness retention was 83.3% with deflections of 12.00 and 10.00mm respectively. Its ultimate deflection was 15.20 mm and its ultimate secant stiffness was 3.52 kN/mm as compared to 4.00 kN/mm for RB. Overall, these signs reveal that the vertical interface showed the worst continuity of the joint configurations considered, which allows a greater deformation of the interface after cracking and a lower diagonal transmission of shear.

ISCJB exhibited an intermediate response. The specimen retained 88.0%, 84.3%, and 92.6% of the reference stiffness at 15, 30, and 50 kN, respectively, with an average deflection increase of 13.4%. Its ultimate deflection and ultimate secant stiffness were 15.00 mm and 3.60 kN/mm, respectively. The relatively high K50/K15 ratio of 77.2% should be interpreted with caution, as it indicates that the relative stiffness reduction between the selected load levels was less pronounced after the initial joint-related penalty, but it does not mean that ISCJB was stiffer than RB or KSCJB in absolute terms.

The energy indices provide a supplementary measure but can be misleading when considered alone. The energy values for VSCJB, ISCJB, and KSCJB at 50 kN were 18.3%, 3.6%, and 1.9% greater than that of RB, respectively. The corresponding changes in energy to failure were 2.0%, 3.4%, and 0.1%. The higher energy value of VSCJB resulted from its greater displacement and the resulting larger area under the load-deflection curve, rather than from greater stiffness or shear capacity. This interpretation is supported by the lower deformation-capacity indices of VSCJB (2.87) and ISCJB (2.94), compared with 3.26 for RB. Energy absorption should therefore be evaluated together with stiffness retention, deflection, and failure load.

The trends shown here are consistent with the broader construction-joint literature. The ultimate-deflection ratios for VSCJB, ISCJB, and KSCJB were about 1.09, 1.07, and 1.01 with respect to RB, which are in or close to the range of 1.02–1.33 reported by Al-Rifaie et al. [12] for beams with horizontal construction joints. This comparison is relevant as the joints have different orientations and different failure capacity. Well-articulated keyed interfaces can appear to represent monolithic functionality, consistent with the near-reference behavior seen for KSCJB Abass [4]. In general, stiffness and serviceability are more sensitive than failure load, which is consistent with findings that joint orientation, surface condition, reinforcement crossing the interface, and the joint plane-governing crack path relationship control post-cracking deformation. Because published studies used diverse shear-span ratios, reinforcement design, joint locations, surface-preparation methods, and loading configurations, direct numerical comparison is risky.

3.6 Observed Crack Patterns

Figure 6 exhibits sample post-test pictures of the monolithic reference beam and the three construction-joint geometries in the shear span. To supplement the load, deflection and first crack results, the photographic record provides qualitative visual proof of the web cracking patterns seen during testing.

The reference beam produced a scattered field of flexural and inclined web cracks. In VSCJB, the post-test images reveal a significantly concentrated set of steep inclined web cracks in the jointed shear span, in agreement with the lower first-crack load, lower stiffness retention, and bigger deflection measured for this specimen. The ISCJB showed inclined and branching web cracks, spread over the shear-span area, which seemed to represent an intermediate behavior between VSCJB and the reference beam. KSCJB revealed several finer, branched web cracks with no single dominant crack visually following the keyed interface; this visual pattern is consistent with its near-reference first-crack load, stiffness, and ultimate capacity. The observations are limited to observable crack patterns. The photos alone do not prove interface slip, debonding or delamination, compression-strut disruption or key shearing/crushing; crack widths, interface opening or relative slip were not measured. Thus, the failure-mechanism explanation is limited to the observable qualitative cracking evidence and the measured global reaction.



(a) Monolithic reference beam (RB)



(b) vertical joint in shear span (VSCJB)



(c) inclined joint in shear span (ISCJB)



(d) Keyed joint in shear span (KSCJB)

Fig. 6. Representative post-test crack patterns (qualitative visual evidence): monolithic reference beam, vertical, inclined, and keyed shear-span joints

4. Conclusions

Across the four-beam comparative series, construction joints reduced first-crack load more strongly than ultimate load. First visible cracking occurred between 11.0 and 13.0 kN, corresponding to observed reductions of 0.0–15.4% relative to the monolithic reference beam, whereas ultimate-load reductions ranged from 0.9% to 4.5%. Construction-joint geometry reduced cracking resistance and affected serviceability while retaining a high proportion of the ultimate capacity.

Table 4 establishes a clear first-crack serviceability ranking for the three joint details in the shear span. The inclined and vertical joints retained 92.3% and 84.6% of the reference cracking load of 13.0 kN, respectively, whereas the keyed joint retained the full reference cracking load. The keyed arrangement also retained 99.1% of the reference ultimate capacity and showed the closest stiffness and deflection behavior to the monolithic beam, providing the most balanced response in terms of crack resistance and deformation control. This performance was plausibly aided by the larger effective contact area created by the key, although this mechanism was not measured directly.

The estimated bending moments, support shears and average web shear stresses also support the same ordering. The results are solely applicable for the geometry, material properties, reinforcement and loading arrangement used in the tests. A major drawback is that fracture widths, crack start locations, joint apertures, interface slip and reinforcing strains were not measured and replication specimens were not tested. Future study should include LVDTs and digital image correlation to quantify interface opening, slide and fracture development.

Where possible, avoid a vertical planar joint in a high-shear region for practical application in the tested configuration. If a construction joint is unavoidable in the shear span, the results favor a keyed, intentionally prepared interface with reinforcement crossing or confining the joint; however, these comparative findings do not replace code-based shear-friction design checks.

Notation and Abbreviations

The symbols and abbreviations used in this study are listed in the following table.

Symbol	Definition
a	Shear span
d	Effective depth of the beam
a/d	Shear-span-to-effective-depth ratio
A_s	Area of longitudinal tension reinforcement
b_f	Flange width
b_w	Web width
f_{cu}	Concrete cube compressive strength
f_y	Yield strength of longitudinal reinforcement
f_{yv}	Yield strength of transverse reinforcement
K	Secant stiffness at a specified load
L	Simply supported span
M_{max}	Maximum bending moment
P	Applied concentrated load
δ	Midspan deflection
δ_u	Ultimate midspan deflection
$KSCJB$	Keyed joint in shear span
RB	Monolithic reference beam without a joint
$VSCJB$	Vertical construction joint in the shear span
$ISCJB$	Inclined construction joint in the shear span
RC	Reinforced concrete
$UHPC$	Ultra-high-performance concrete
$LVDT$	Linear variable differential transformer
K_{50}/K_{15}	Ratio of secant stiffness at 50 kN to that at 15 kN

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