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Original Article

Comparative Study of Drop Panels, Stirrups, and Stud Rails for Enhancing Punching Shear Resistance under Seismic Actions

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***Abstract* -** *Problems with punching shear failure at slab-column joints have remained one of the major problems in RC flat slabs, especially where there is a combination of loadings from both gravity and lateral forces acting on the slab. This paper presents a numerical study of the use of drop panels and shear reinforcements in improving slab-column joints using nonlinear FEM with the CDP approach. Ten models (C25-C34) were considered while considering the two grades of concrete (M35 and M40), three types of reinforcement systems (unreinforced, with stirrups, and with stud rails) under static and Lateral loading conditions. The numerical results suggest that drop panels increase initial stiffness and Peak lateral load (for seismic analyses) capacity of slab–column joints. But, without geometric enhancement, the brittle punching behaviour under Lateral loading could not be prevented. The unstiffened configurations showed substantial loss of strength and higher deformations. Stirrups had been introduced to provide better confinement and post-peak stability. The systems with stud rail reinforcement had the best numerical response when measured in terms of load-carrying capacity, residual strength and ductility. The presence of a stirrup gives the added advantage of increased lateral load capacity of about 330% as well as good post-peak behaviour. The stud rail system exhibits the best overall behaviour, where the lateral load capacity can be increased by about 400%. The grade effect of concrete reveals that more the strength, the higher the stiffness and capacity and the lower the ductility in non-reinforced applications. The sensitivity analysis proves that the most important parameter that controls the performance is the shear reinforcement type. A comparison with codal provisions such as IS 456, ACI 318, and Eurocode 2 indicates that the existing formulations do not adequately capture the effect of reinforcement and the post-peak structural behaviour. The numerical results indicate that, out of all the examined configurations, the use of both drop panels and stud-rail reinforcement would yield the most favourable structural response. These results, however, are based on finite element simulation and require experimental validation in the future.*

***Keywords* -** *Punching shear, Flat slab–column joints, Drop panels, Shear reinforcement, Stud rails, Stirrups, Lateral loading, Nonlinear finite element analysis, Concrete Damage Plasticity (CDP), Residual strength, Structural ductility, Codal validation.*

1. Introduction

Reinforced Concrete (RC) flat slabs are common in modern buildings due to their flexibility, ease of formwork, low floor-to-floor height and efficient utilization of internal space.

The structural performance, however, is largely controlled by the slab–column connection, where concentrated shear forces and unbalanced moments may cause punching shear failure to occur. Punching shear is especially important since it is typically brittle, has limited warning, and can cause local disconnection of the slab from the supporting column, which can lead to progressive collapse [1, 2].

Slab-column connections are exposed to complex stress transfers from direct shear, flexure, torsion and unbalanced moments under combined gravity and lateral loading. Experimental studies have demonstrated that enhancing gravity shear demand decreases the lateral drift capacity of slab–column connections and drives them toward punching failure [3, 4]. As a result, these connections can then be expected to lose stiffness at very short load levels, have minimal deformation capacity, and become completely load-bearing incapable during earthquake-type loading [5, 6].

To enhance punching shear performance, several strategies have been proposed, such as the addition of drop panels, column capitals, slab thickening, material strength, and transverse shear reinforcement. Shear reinforcement, such as shear bands, shear reinforcement in the form of headed studs, shear reinforcement in the form of stirrups, or similar systems, is particularly effective, as it slows the growth of critical shear cracks and enhances the punching resistance and deformation capacity [7, 8]. Experimental studies show that if shear reinforcement is used correctly, then the response changes from a brittle punching mechanism to a more stable and deformable failure mode [8].

The effectiveness of shear reinforcement is not only determined by the quantity of reinforcement but also by its anchorage, spacing and layout, and by its intersection with the possible punching shear crack. Headed stud rails usually offer good anchorage and can be mounted in orthogonal or radial configuration around the column. Experimental research has shown that headed studs can significantly improve the strength and drift capacity of slab–column connections, but their performance is influenced by the layout of the studs, flexural reinforcement ratio, and gravity shear demand [9, 10]. Comparative studies of various punching-reinforcement systems have also revealed that the anchorage condition can significantly affect the contribution of the stirrups and headed studs to the punching resistance [11].

Drop panels are a standard geometrical approach to enhancing slab–column performance. A drop panel increases the effective critical perimeter, the connection stiffness and the nominal shear stress by increasing the slab depth at a local level around the column. Drop panels are thus known in the existing design provisions as a way of increasing the resistance to punching and unbalanced moment transfer [12-14]. However, a drop panel under combined gravity and lateral actions will be effective if it has the appropriate geometry and is provided with sufficient shear reinforcement. If a material is badly distorted in the lateral direction, simply improving the geometry may not be sufficient to provide adequate post-peak resistance or ductility.

A critical shear crack is usually the mechanism used to explain the mechanical behaviour of punching shear. The Critical Shear Crack Theory is based on punching resistance and offers a rational explanation for the effects of flexural reinforcement, concrete strength, deformation demand, and slab depth [1, 15]. Further research on this methodology has contributed to the punching shear provisions of the fib Model Code 2010 and has highlighted the constraints of empirical design equations [16]. The Model Code of fib [16] and the code of ACI 318 [12-14] and IS 456 offer practical procedures for estimating punching shear resistance. The critical perimeter is, however, located differently; concrete strength is treated differently; size effect is considered differently; the contribution of reinforcement is treated differently; and there are different maximum allowable punching resistance values. Some experimental comparisons indicate that the degree of conservatism can be influenced by slab geometry, reinforcement ratio, slab thickness, and detailing of shear reinforcement [7, 17]. Moreover, the traditional code equations are designed to estimate the design strength of the structure, and they do not capture stiffness degradation, damage localization, residual resistance or the whole post-peak behaviour.

Recent developments in the field of nonlinear finite element analysis have allowed the punching behaviour of slab-column connections to be studied beyond the prediction of punching capacities. The Concrete Damaged Plasticity model is a model that combines plasticity and tensile/crushing damage variable, and it is used to model cracking, crushing, irreversible deformation and stiffness degradation in concrete [18]. Three-dimensional finite element models, which include this formulation, can successfully mimic the load–displacement response, crack formation, stress redistribution, and failure patterns in RC slab–column connections when properly calibrated [19, 20]. However, numerical predictions are still dependent on tensile constitutive behaviour, mesh size, fracture-energy regularisation, the interaction between reinforcement, and the boundary and loading conditions [20].

Recent nonlinear modelling studies have extended these methods to flat slabs containing double-headed stud reinforcement. These studies show how the contribution of stud rails can be modelled and analyzed using NLFEA, and how certain reinforcement arrangements might not be explicitly modelled with simplified code equations [21]. These studies further emphasize the importance of model validation prior to using the numerical results for comparative or design-based purposes.

Although extensive research has been conducted on punching shear, much of the available literature considers conventional flat slabs, individual strengthening techniques, or predominantly concentric static loading. Fewer studies have been conducted that consider the interaction between the drop panel, shear reinforcement type, concrete strength, and lateral loading in the same numerical program. Furthermore, other parameters like residual load-carrying capacity, stiffness deterioration, deformation capacity, stress redistribution, and post-peak response have not been studied as much as punching strength. Therefore, the present study investigates the behaviour of RC drop-panel slab–column joints using nonlinear finite element modelling. The effects of two concrete grades (M35 and M40), two shear reinforcement configurations (unstiffened, stirrup-stiffened and headed stud-rail-stiffened), and two loading types (static gravity load and combined gravity and lateral load) are analyzed using ten numerical models. The investigation considers load–displacement behaviour, stress distribution, displacement and rotation, tensile and compressive damage development, peak resistance, and post-peak load retention. The aim is to understand the relative effectiveness of drop panels, stirrups and headed stud rails and to ascertain what is the most favourable structural response given the restrictions of the finite element model adopted.

2. Research Gap and Objectives

Even though studies on the punching shear of concrete flat plate structures have already been carried out extensively in the past, there remain some deficiencies. These previous studies were conducted on the slab-column interface or strengthening technique alone, without concentrating on the connection between the two.

The drop panels are often utilized in practice to obtain higher punching shear resistance and stiffness, but their behaviour when subjected to both gravity and lateral loads has been largely unknown. Stirrups and stud rails are also found to provide superior punching resistance, but few comparative studies have been conducted in drop-panel slab-column joints.

In addition, most of the previous studies are concerned mainly with Peak load capacity, with relatively little research dedicated to residual strength, deformability, ductility, and energy dissipation properties after the peak. There is a lack of understanding of how these performance indicators are affected by the combination of concrete strength, reinforcement detailing, and Lateral loading. The existing building codes, such as the Indian Code (IS 456), ACI 318, and Eurocode 2, are primarily dependent upon empirical equations that do not take into account the role played by the reinforcement detailing in the post-peak behaviour of structures. Therefore, a thorough investigation becomes essential to analyze the impact of the above-discussed factors on the structure.

The paper emphasizes a comprehensive numerical study of the RC slab-column joint incorporating drop panels subjected to static and lateral loads.

In particular, the paper shows:

- Evaluation of the structural behaviour of drop panel slab-column joints subjected to both static and lateral loads via nonlinear finite element analysis using the CDP method.
- Evaluation of the effect of concrete strength (M35 and M40) on the structural behaviour of drop panels.
- Comparative study of shear reinforcement (stirrups and stud rails) in improving shear resistance to punching, ductility, and post-peak performance of drop-panel configurations.
- A study on the stress distribution, load-Displacement response, and damage evolution to explain the failure mechanisms and structural response to combined loading.
- The identification of an optimal design configuration combining drop panels and shear reinforcement for improved seismic performance and enhanced punching shear resistance

2.1. Novelty of the Present Study

The novelty of this research can be attributed to its comprehensive approach towards assessing drop-panel slab–column joints under the effect of static and Lateral loads, using the method of nonlinear finite element analysis. While prior research has concentrated on the study of individual strengthening measures used in flat slab structures, or static punching shear, the current study takes a different approach, incorporating the effects of drop panels, concrete strength, stirrup reinforcement, and stud-rail reinforcement in one single analysis using the CDP model.

The investigation focuses on the aspects of punching shear capacity, stiffness deterioration, redistribution of stresses, development of damage, residual strength, ductility, and energy dissipation. These are further compared with the criteria set out in IS 456, ACI 318, and Eurocode 2 specifications. Through the detailed study, it becomes easier to understand the seismic behaviour of flat slabs and identify the appropriate measures for strengthening.

3. Methodology

3.1. Numerical Modelling Framework

The Finite Element model using the nonlinear analysis technique was created using the software Abaqus/standard. Tensile cracking, compressive crushing, and stiffness deterioration under nonlinear loadings are all included in the Concrete Damage Plasticity (CDP), which was used to simulate the behaviour of concrete material [12, 13]. Truss elements were embedded in the reinforcement to ensure interaction with concrete.

3.2. Geometry and Model Configuration

All models consisted of a square slab–column assembly with a centrally located drop panel. The geometric parameters were maintained constant across all cases to ensure comparability:

- Slab dimensions: 1800 mm × 1800 mm × 150 mm
- Drop panel: 900 mm × 900 mm, additional thickness of 100 mm
- Column size: 300 mm × 300 mm × 500 mm

The drop panel was integrated monolithically with the slab using Boolean merging, representing realistic construction practice. The geometry is shown in Figure 1.

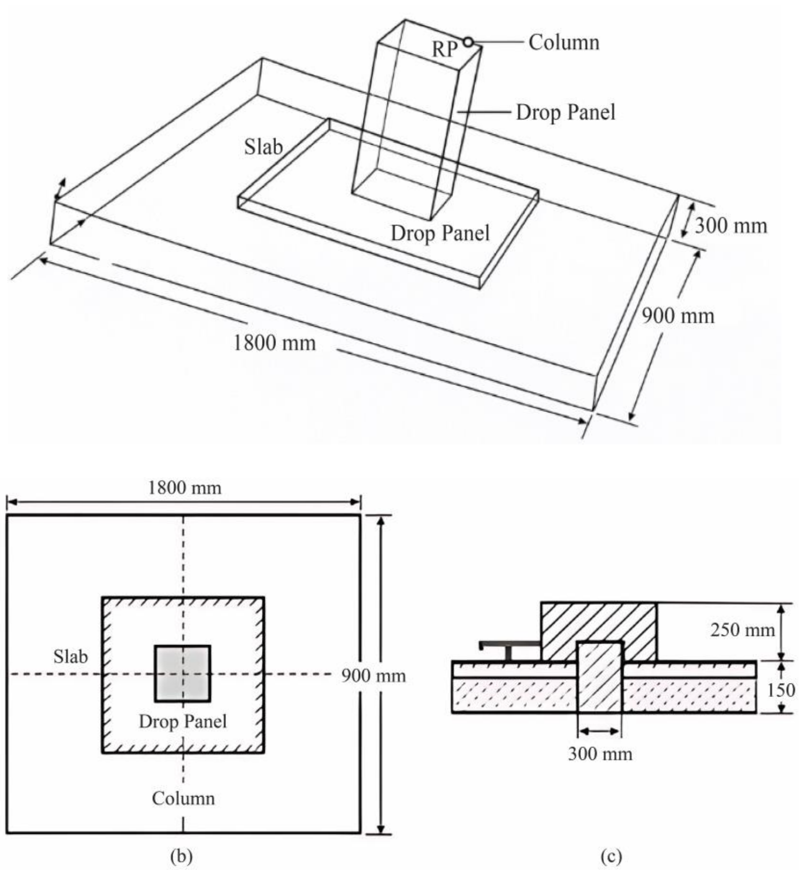


Fig. 1 Geometry of slab–column connection with drop panel: (a) finite element model, (b) Plan view showing dimensions, and (c) sectional view illustrating slab and drop panel thickness.

3.3. Case Matrix for the Study

For the numerical investigation, a systematic case matrix was chosen to isolate the effect of the main parameters. The following three variables were taken into account: concrete grade, shear reinforcement system, and loading condition. Two concrete grades, M35 and M40, were selected to study the effect of material strength. Three reinforcement configurations, namely unstiffened drop-panel slab, stirrup-reinforced slab and stud-rail-reinforced slab, were considered. Loading conditions were categorized into two types: static and Lateral. In this way, the effect of the dominant effect of gravity loading and the combined effect of gravity-lateral loading were evaluated directly on strength, stiffness, deformation capability, residual load carrying capacity and progressive damage development in all ten cases (C25-C34).

For the systematic study of the effect of concrete class, shear reinforcement and loading condition on punching failure behaviour, a suitable case study matrix was designed as depicted in Table 1 and 2.

Table 1. Case matrix considered for the study

Case	Concrete Grade	Reinforcement	Detailing	Load	Purpose
C25	M35	None	–	Static	Baseline drop-panel behaviour
C26	M35	None	–	Seismic	Ductility and seismic vulnerability

C27	M35	Stirrups	Ø10 mm @ 100 mm	Static	Shear strength improvement
C28	M35	Stirrups	Ø10 mm @ 100 mm	Seismic	Energy dissipation and crack control
C29	M35	Stud rails	2P, Ø10 @ 100 mm	Static	Capacity enhancement
C30	M35	Stud rails	2P, Ø10 @ 100 mm	Seismic	Seismic shear performance
C31	M40	None	–	Static	High-strength behaviour
C32	M40	None	–	Seismic	Brittle response under Lateral loading
C33	M40	Stud rails	2P, Ø10 @ 100 mm	Static	Ductility improvement in M40
C34	M40	Stud rails	2P, Ø10 @ 100 mm	Seismic	Optimal seismic configuration

Note: 2P = two perimeters of stud rails

Table 2. Concrete damage plasticity parameters used in ABAQUS

Parameter	Value
Dilation (ψ) angle	36°
Eccentricity (ε)	0.1
fb0/fc0	1.16
Kc	0.667
Viscosity	0.0005

3.4. Material Modelling

Two concrete grades, M35 and M40, were measured to calculate the effect of material strength. The CDP model parameters included:

- Elastic properties
- Compression hardening and damage
- Tension stiffening and cracking behaviour
- Viscosity parameter for numerical stability

The properties of materials were based on standard code considerations and proven literature [14, 15]. Steel reinforcements were considered as having elastic-plastic behaviour.

3.4.1. Calibration of the Concrete Damage Plasticity (CDP) Model

Abaqus software data have also been considered when formulating the Concrete Damage Plasticity (CDP) model, which depicts the nonlinear response of concrete, taking into consideration phenomena such as tensile cracking, crushing, reduction in stiffness and damage. Parameters of CDP have been chosen by considering suggestions made in the literature and validated numerical studies conducted on reinforced concrete slab-column joint. Below are the CDP parameters chosen for the current research: Dilation Angle = 36°, Eccentricity = 0.1 fb0/fc0 = 1.16, Kc = 0.667, Viscosity Parameter = 0.0005. The above-mentioned values of CDP parameters have been considered based on earlier research works where they were found to be yielding convergence and realistic modelling of concrete damage behaviour. For the compressive response, the nonlinear stress–strain relationships were used for concrete of the selected grades (M35, M40); for the tensile response, the tension stiffening relationships were adopted to consider crack initiation and propagation. Compression and tensile damage variables were introduced to account for stiffness reduction following crack and crush formation due to damage accumulation under compression and tension loading conditions. The accuracy of the CDP model calibration was validated using the load-displacement curve, stress pattern, and damage distribution, leading to realistic behaviour of the structure throughout the analysis.

3.5. Reinforcement Modelling

The stud-rail reinforcement system included two concentric perimeters of double-headed vertical studs around the column region. The spacing between the studs was 100 mm, and the placement of the studs was done symmetrically to the face of the column so that the studs will cross the line of possible punching shear failure plane. The type of reinforcement configuration used was selected depending on the standard practice for the design of the reinforcement required for punching shear resistance and provided improved performance in terms of load transfer, crack control, and confinement in the slab-column interface region.The reinforcement configuration used in numerical modelling is shown in Figure 2, which includes stud-rail reinforcement around the slab-column interface region and stirrup reinforcement. The stud rails were placed around the column symmetrically to improve punching shear resistance and load redistribution in the critical region. Two shear reinforcement strategies were considered:

- Stirrups: Closed vertical hoops (Ø 10 mm) arranged around the column at specified spacing
- Stud rails: Vertical headed studs (Ø 10 mm) arranged in multiple perimeters around the column.

The reinforcement depicted in Figure 2 was modelled using T3D2 truss elements and then incorporated into the concrete by the use of an Embedded Region constraint.

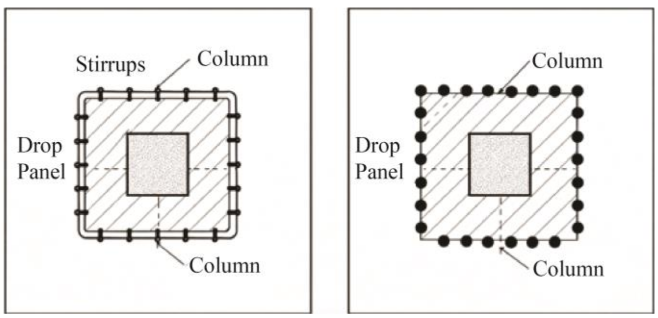


Fig. 2 Reinforcement configurations, (a) Stirrups, and (b) Stud rails.

3.6. Mesh and Interaction

Concrete elements C3D8R (eight-node reduced integration) were used for the discretization of the concrete domain, while reinforcement elements were chosen from truss elements. For stress concentration and crack behaviour, a small element size was used near the slab-column connection region.

All interactions were assumed to be perfectly bonded, with:

- Column-slab interface: Tie constraint
- Reinforcement-concrete: Embedded interaction

3.6.1. Mesh Sensitivity Assessment

To verify that the numerical results were not significantly affected by the size of elements, a mesh sensitivity study was performed. Three mesh densities were examined: coarse, medium, and fine, which represent the slab–column connection region discretized into coarse, medium, and fine meshes, respectively. The parameters analyzed were Peak lateral load (for seismic analyses), capacity, maximum displacement, and stress distribution in the critical punching shear area. The results showed that the difference in the predicted Peak lateral load (for seismic analyses) between the medium and the fine mesh was less than 5%, which was acceptable and shows good convergence. For this reason, the medium mesh option was selected for the subsequent analysis, which is an appropriate balance between computational efficiency and numerical accuracy.

3.7. Boundary Conditions and Loads

Boundary conditions were specified to represent the connection between a slab-column interior connection:

- Slab boundaries: Fixed in translation (Ux = Uy = Uz = 0)
- Column base: Encastre support
- Loads applied to a datum point at the column head

Two loading cases were considered:

3.7.1. Static Loading

- Gravity load + gravity pressure applied on the slab

3.7.2. Lateral loading

- Gravity load followed by lateral load applied incrementally at the column top

A two-step nonlinear analysis was performed:

1. Gravity loading step
2. Lateral (seismic) loading step

Gravity loads, with magnitudes ranging from 700 to 800 kN at the top of the column, are used for the load on the slab-column connection to reflect gravity effects. Additionally, a uniform gravity load is applied on the slab surface. In case of the seismic load, an increasing lateral load is applied at the column reference point as illustrated in Figure 3 after applying the gravity loads. The analysis involves two stages: (i) the application of the gravity loads; and (ii) the application of the increasing lateral loads.

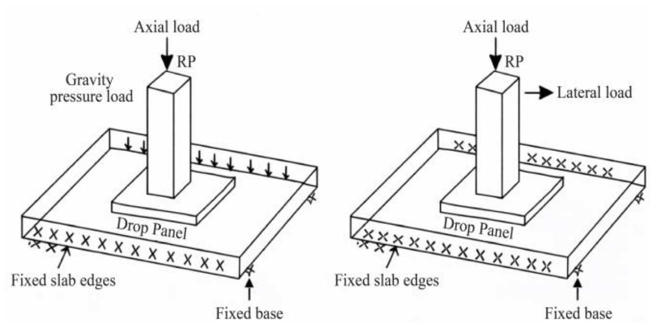


Fig. 3 Boundary conditions and loading scheme for static and seismic analysis

The term gravity load is used to indicate the load imposed vertically through the column, while lateral load is used to indicate the load applied horizontally as seismic action. Peak lateral load (for seismic analyses) capacity is the maximum load on the slab–column connection when the analysis is performed, and Residual load capacity is the load-carrying capacity that remains after peak response. These terms are applied to the results as consistently as possible throughout the presentation and discussion.

3.7.3. Modelling Assumptions

To make sure that the simulation is efficient and that all models remain consistent, specific modelling assumptions had to be made in the current study. It was presumed that the material used for modelling is homogeneous and isotropic, and for describing concrete nonlinear behaviour, the CDP model was used. The perfect bond (no slip) assumption was made and simulated through the use of truss elements. The drop panel and the slab were considered monolithic. To simulate the restraint of surrounding structural members, the slab edges were restrained against the translational movement (Ux = Uy = Uz = 0), and a consistent boundary condition was set for comparison. The column base was assumed as fully fixed (encastre) with no translation or rotation, thus simulating a rigid support condition frequently used in numerical examinations of slab–column joints. The loads were applied at a reference point at the top of the column, ensuring uniform load transfer and stable numerical convergence.

Lateral loading was incrementally applied following the gravity loading step for Lateral loading simulations. The analysis was performed in the quasi-static domain and not under the full dynamic earthquake loading. So, the results have a comparative implication on the effect of reinforcement detailing and drop panels on structural response to combined gravity and lateral loads. In the present model, imperfections in the material, construction tolerances, bond deterioration and cyclic degradation effects were not considered explicitly, which can be a source of uncertainty in practical use.

3.8. Case Matrix

In all, ten numerical models (C25-C34) have been formulated for the study of the combined impacts of:

- Concrete strength: M35 and M40
- Strengthening type: unstiffened, stirrups, and stud rails
- Load type: static and seismic

Such a matrix provides a scientific basis for studying the behaviour of drop panels.

3.9. Performance Parameters

The structural response was evaluated using the following parameters:

- Load–displacement behaviour
- Stress distribution (Von Mises and principal stresses)
- Damage indices (tensile and compressive damage)
- Deformation characteristics (displacement and rotation)
- Peak load capacity and post-peak response

These parameters were employed to evaluate punching shear behaviour, ductility and general seismic performance. The loading conditions used are, an Gravity load between 700 and 800 kN was applied at the column top to replicate the effect of gravity and seismic response was measured by subjecting the column to an incremental lateral load at the column reference point. Also, a steady-state pressure of gravity was imposed on the surface of the slab to simulate the conditions of service loading.

4. Results and Discussion

4.1. Unstiffened Drop-Panel Slab–Column Joints (C25–C26)

Joints in drop panel slab-column connection without shear reinforcement (C25-C26) are considered as the control model for evaluating slab-column joints with geometric capacity when under both static and lateral loading conditions. They can be considered as practical models, where only drop panels are used for shear reinforcement.

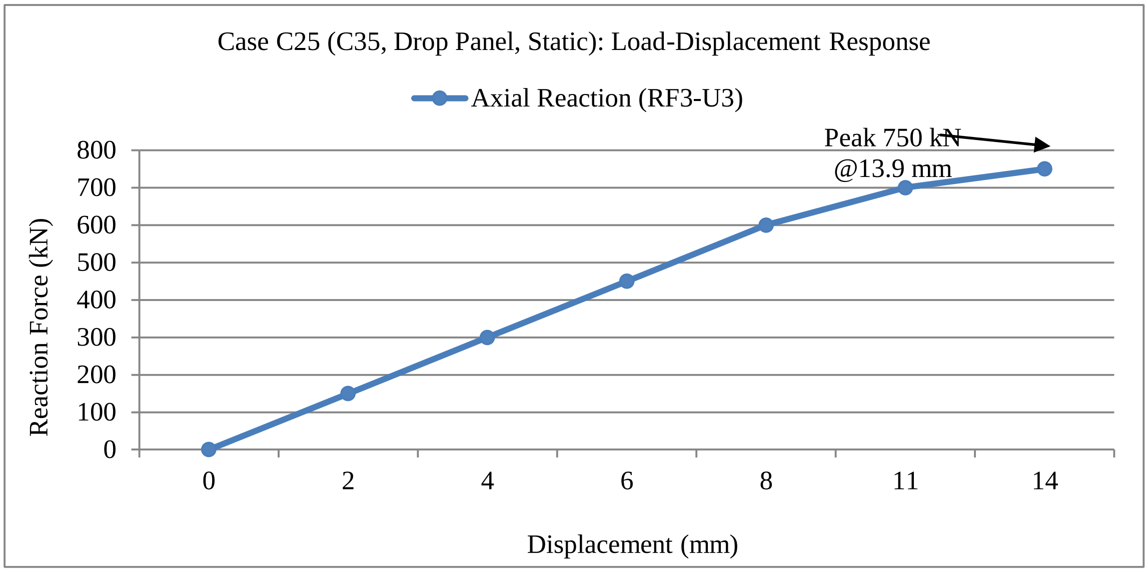
4.1.1. Load–Displacement Response of Unstiffened Drop-Panel Joints (C25-C26)

Figure 4(a)-(b) shows the load displacement responses of the unreinforced drop-column joints between the slab and columns. In the case of the static case (C25), the axial response (Figure 4(a)) exhibits a distinct three-stage behaviour. A linear elastic behaviour is noted until about 5 mm of displacement, where the reaction force is about 300 kN, which reveals the uncracked stiffness.

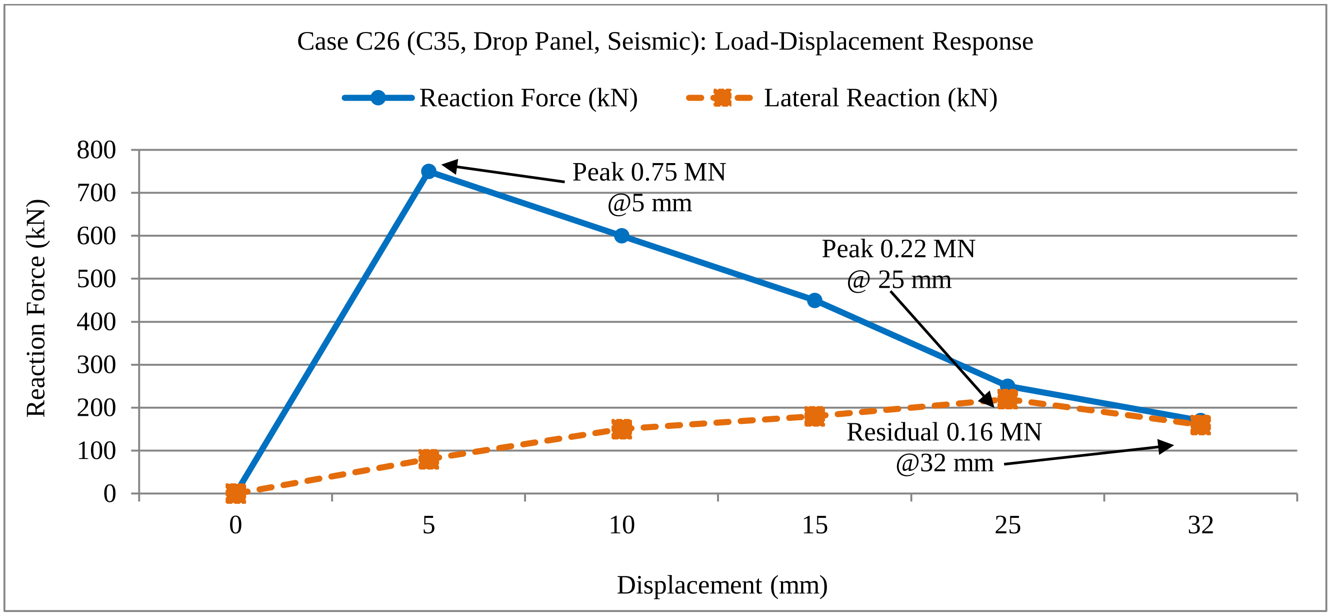
This is followed by a nonlinear phase (5-11 mm) in which stiffness decreases as the load increases between 300 kN to 600 kN because of crack initiation and redistribution of stress around the punching perimeter (principal tensile stress =33-34 MPa). The response is constant beyond 11 mm with a Peak lateral load (for seismic analyses) of approximately 750 kN at 13.9 mm without sudden softening, and proves that the drop panel increases the load capacity but is not a counter to localized punching behaviour.

In the case of seismic (C26), the response (Figure 4(b)) is coupled in both axial and lateral behaviour. The initial Gravity load is approximately 750 kN at about 5 mm, which is equal to C25, but later decays to about 170 kN at 32 mm as a result of progressive cracking under combined loading. The response to the lateral direction rises linearly to approximately 150 kN at approximately 10-12 mm and peaks at approximately 220 kN at approximately 25 mm, then slowly softens to about 160 kN at 32 mm, corresponding to the post-peak deformation stage. This shows that they have low ductility and stiffness deterioration in seismic action.

In general, drop panels enhance early stiffness and maximum strength, but the lack of shear reinforcement causes the deterioration of the axial strength and the low resistance to the lateral loading.



(a)



(b)

Fig. 4 Load–displacement response of unstiffened drop-panel slab–column joints: (a) Axial response under static loading for Case C25, showing nonlinear three-stage behaviour with Peak lateral load (for seismic analyses) of ~750 kN at 13.9 mm, and (b) Combined axial and lateral response under Lateral loading for Case C26, illustrating axial strength degradation and development of lateral resistance with peak ~220 kN and residual capacity under increasing drift.

4.1.2. Comparative Trends of Load-Displacement Response (C25-C26)

Comparative analysis of the load-displacement behaviour of C25 and C26 indicates that Lateral loading affects the structural behaviour of the unstiffened drop- panel connection.

The initial stiffness is similar in both cases, and the axial response is about 300 kN at up to 5 mm and about 750kN at up to 5 mm in C25 and C26, respectively, and this means that the drop panel controls the early elastic behaviour. But after this level, there is a great deviation. In C25, the Gravity load grows steadily up to approximately 750 kN at 13.9 mm, and the stiffness does not decrease as the strength declines. On the other hand, C26 exhibits a decrease in axial capacity between 750 kN and 170 kN with increasing displacement to 32 mm with progressive cracking under combined axial-lateral loading. The lateral loading of C26 creates another deformation or mechanism that leads to the formation of lateral resistance up to approximately 220 kN at approximately 25 mm, after which softening slowly takes place. This bifurcated response results in a higher demand of deformation and lower stiffness than that of a purely axial response in C25. In general, drop panels offer similar initial stiffness in both scenarios, but Lateral loading causes profound losses in the retention of axial strength, promotes deflection, and introduces the effects of lateral drift, which suggests low ductility and susceptibility to punching shear under combined loading conditions.

4.1.3. Damage Patterns of Unstiffened Drop-Panel Joints (C25-C26)

The stress and damage behaviour of both Cases C25 and C26 are plotted in Figure 5(a)-(d), which shows the development of the punching shear behaviour in both static and seismic cases.

The Von Mises stress distribution in the static (C25) case indicates a symmetric distribution of stress around the slab-column interface with maximum values of about 38-40 MPa. The major stress contours point to tensile stresses, which are greater than the concrete capacity (≈33-34 MPa) asymmetric distribution of stress.

Lateral loading causes the von Mises stresses to concentrate on one side of the column, and the other side has diagonal bands of stresses going out of the column region, which are the principal tensile stresses (approx. 27-29 MPa) and therefore radial cracks start forming along the critical punching perimeter.

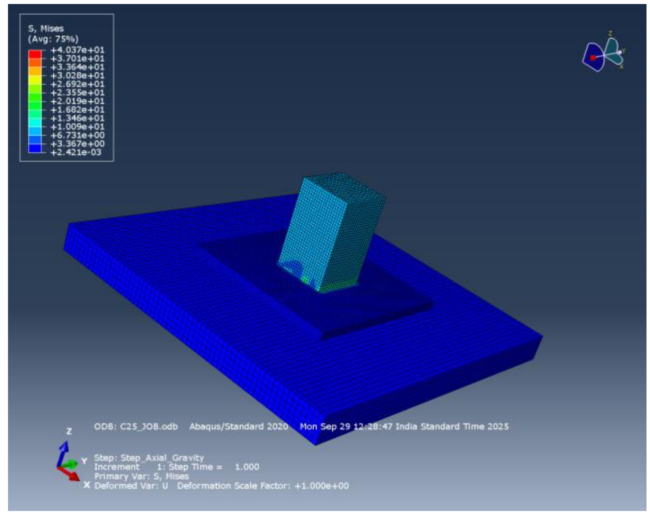
The stress field is also quite homogeneous, which proves that the drop panel is effective in spreading the load and postponing local failures. The seismic case (C26), on the other hand, has a very non-uniform and This implies cracking and rotation of the slab-column joint as a result of drift action.

The lateral and the combined Gravity loading cause an increased stress localization, faster propagation of cracks, lower confinement efficiency. On the whole, the damage patterns indicate that whereas drop panels enhance the distribution of stress under the condition of static loading, these materials cannot inhibit the asymmetric stress concentration and brittle punching behaviour under Lateral loading, thus underlining the importance of shear reinforcement.

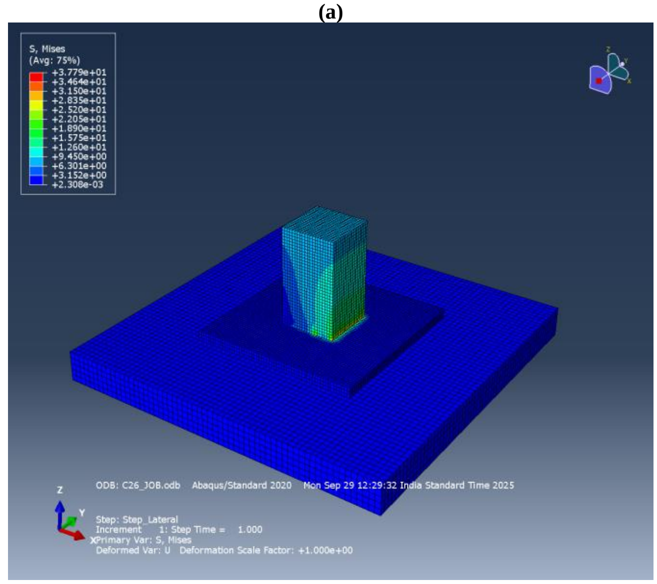
4.2. Stiffened Drop-Panel Slab-Column Joints (C27-C34)

Shear reinforcement through the use of stirrups and studs is highly significant in determining the behaviour of drop panel slab-column joints. Stiffened configurations (C27- C34) exhibit higher bearing capacity, stiffness, and improved post-peak behaviour when compared to their non-stiffened counterparts (C25-C26).

This is in relation to both static and lateral loading. The reinforcement offers the confinement of the concrete core, slows down the crack propagation and enhances the stress redistribution at the slab-column interface.



(a)



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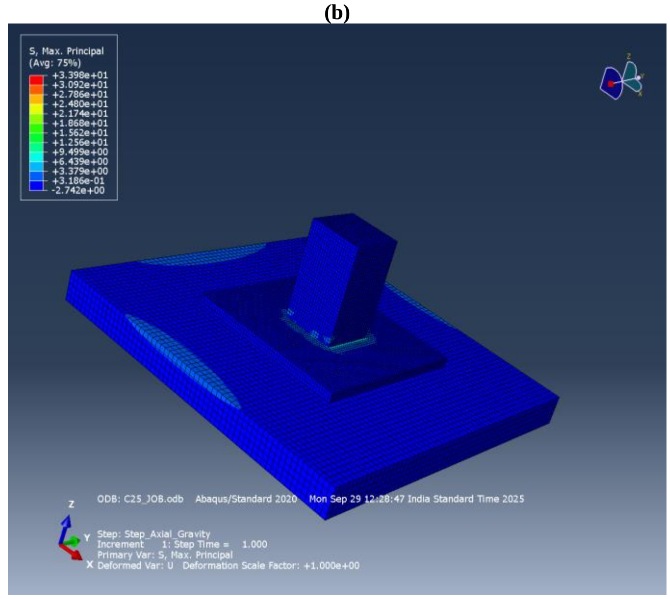

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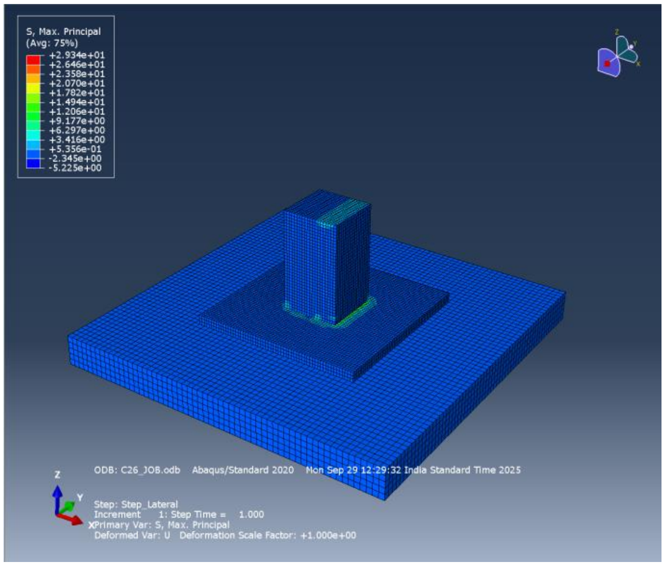


(d)

Fig. 5 Stress distribution characteristics of unstiffened drop-panel slab–column connections under static and seismic loading (a) Von Mises stress contour – C25 (Static Loading), (b) Von Mises stress contour – C26 (Seismic Loading), (c) Maximum principal stress contour – C25 (Static Loading), and (d) Maximum principal stress contour – C26 (Seismic Loading).

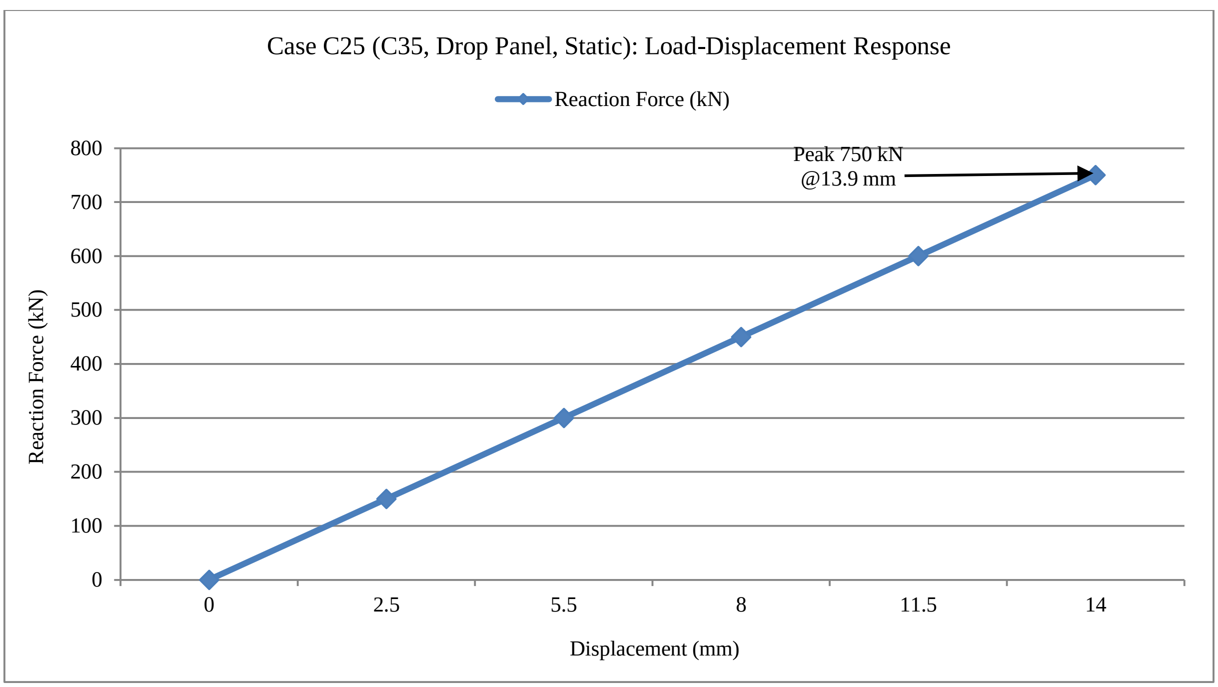
4.2.1. Load-Displacement Response of Stiffened Joints (C27-C34)

Load-displacement curves for the reinforced cases are depicted in Figure 6. For the stirrup-reinforced case loaded statically (C27), the stiffness in the axial case response is higher than that of C25, and the load attained is approximately 540 kN at a displacement of 6 mm. The response then proceeds to a Peak lateral load (for seismic analyses) of approximately 750 kN at a smaller displacement of approximately 12.5 mm, which means that the stiffness is better and the deformation is smaller. The lack of any sudden softening proves the effectiveness of stirrups to control crack propagation and increase load transfer. Increased Lateral loading (C28), lateral resistance and stability are enhanced with the introduction of stirrups. Lateral displacement is much less than that of C26, with the maximum displacement of approximately 0.446 mm, whereas the lateral load capacity rises with a smoother post-peak response. There is enhanced retention of the axial capacity, which implies enhanced confinement and retardation of degradation during combined loading. The most favourable behaviour of all configurations is the stud-rail systems (C29-30 and C33-34). These cases demonstrate a greater load-carrying capacity, better stiffness and a longer post-peak deformation. The load-displacement curves show plateau-type behaviour following Peak lateral load (for seismic analyses), which means that there is still resistance to loads despite crack formation. In Lateral loading, stud rails offer better lateral resistance and lesser strength loss than stirrups, which validates their excellent performance in strengthening ductility and energy dissipation.

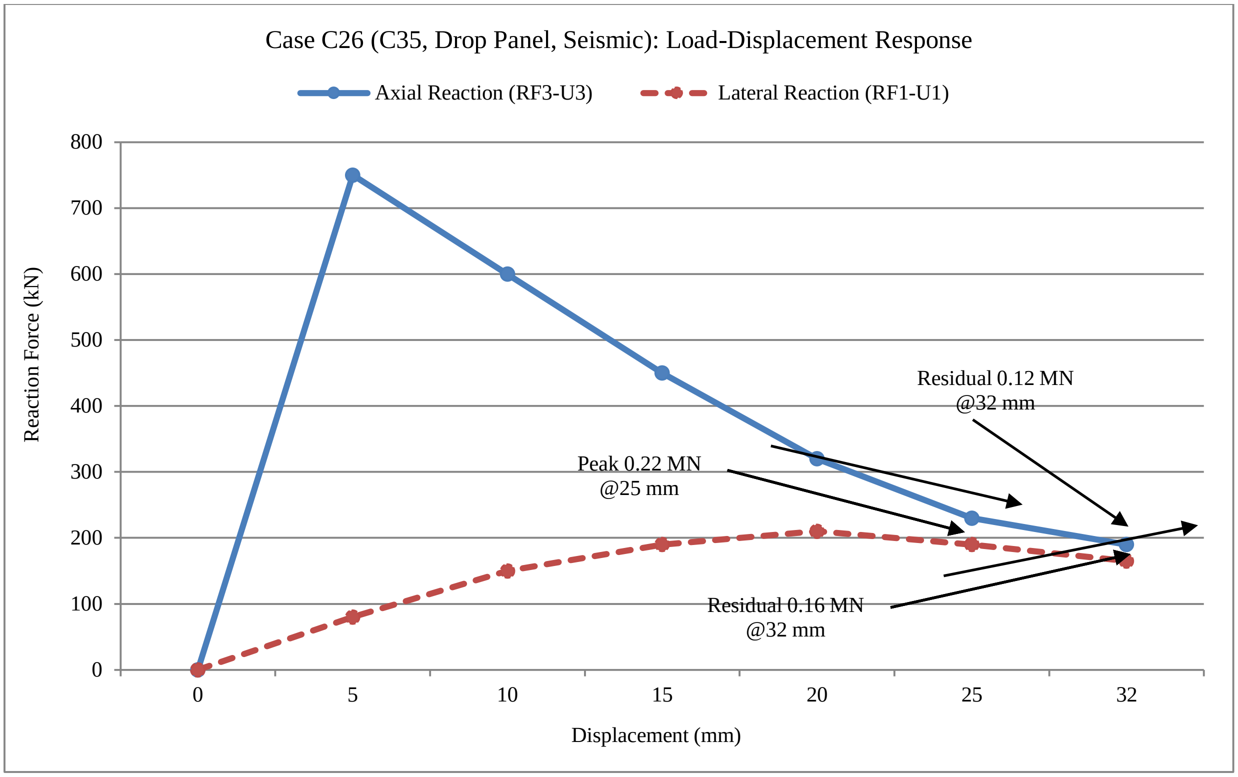
In general, the stiffened configurations are:

- Reduced displacement at Peak lateral load (for seismic analyses),
- Improved post-peak stability,
- Greater Lateral load lateral resistance,

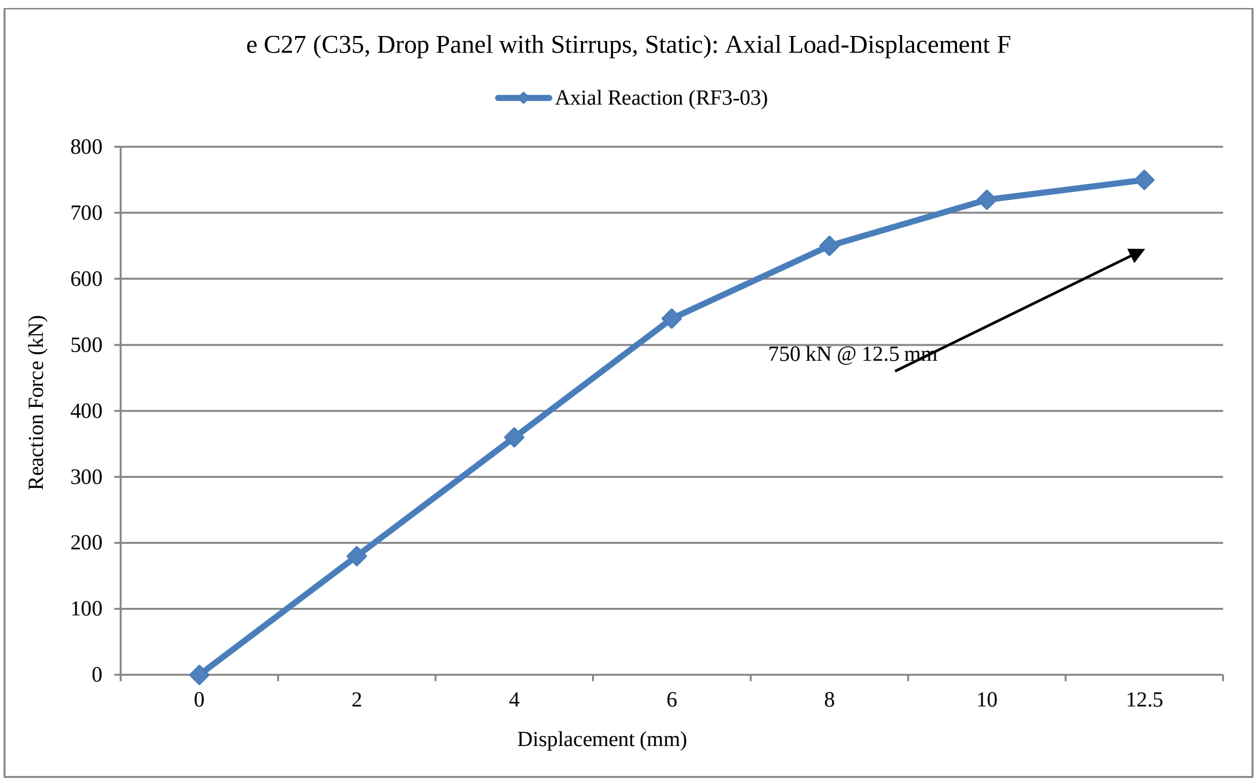
and showing a major enhancement in the structural performance over unstiffened drop-panel joints.



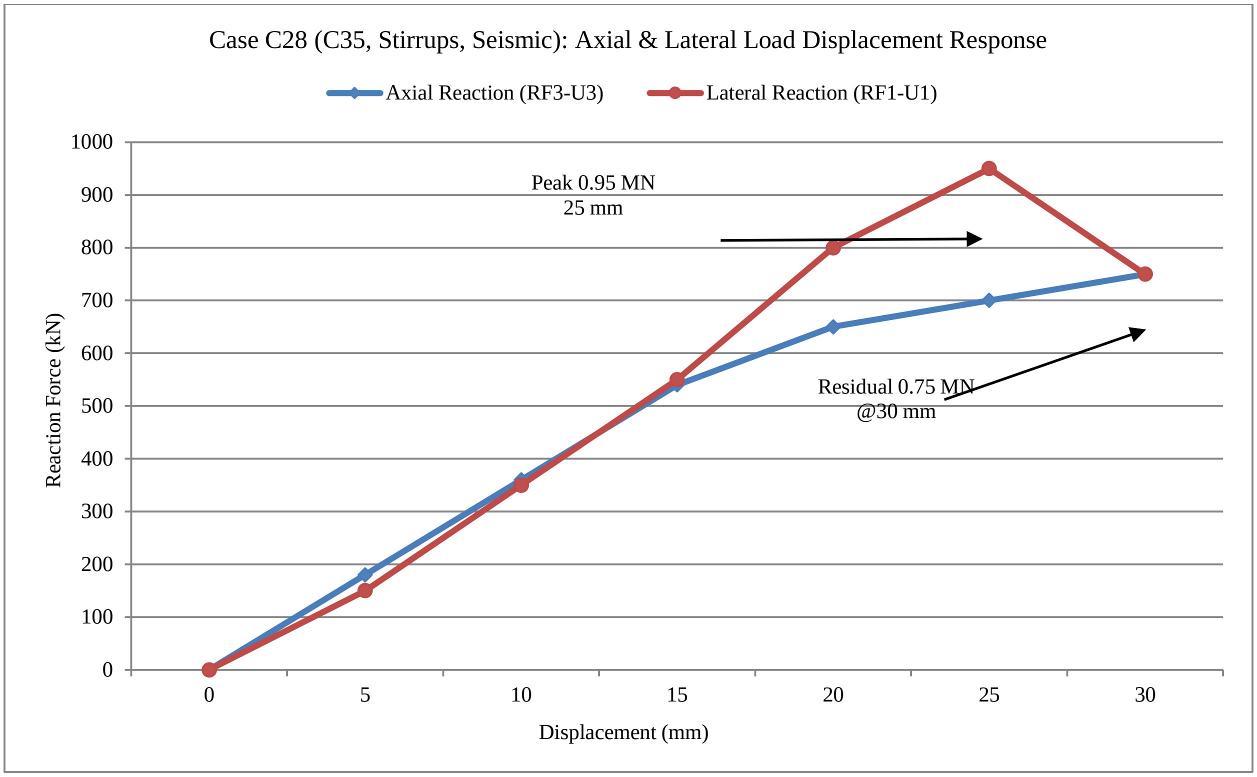
(a) C27 - Stirrups (Static)



(b) C28 - Stirrups (Seismic)



(c) C29 - Stud Rails (Static)



(d) C30 - Stud Rails (Seismic)

Fig. 6 Load–displacement response of stiffened drop-panel slab–column joints: (a) C27 (stirrups, static), (b) C28 (stirrups, seismic), (c) C29 (stud rails, static), and (d) C30 (stud rails, seismic), illustrating improved stiffness, load capacity, and enhanced post-peak behaviour compared to unstiffened cases.

4.2.2. Comparative Insights of Stiffened Joints (C27-C34)

Figure 7 gives the comparative performance of stiffened drop-column joints using slab-column joints. In terms of stiffness and confinement, the stirrup-reinforced configurations (C27-C28) demonstrated better performance than the unstiffened configurations. For a static load, the maximum gravity load capacity was raised in C25 to about 750 kN and was recorded at a lower deflection.

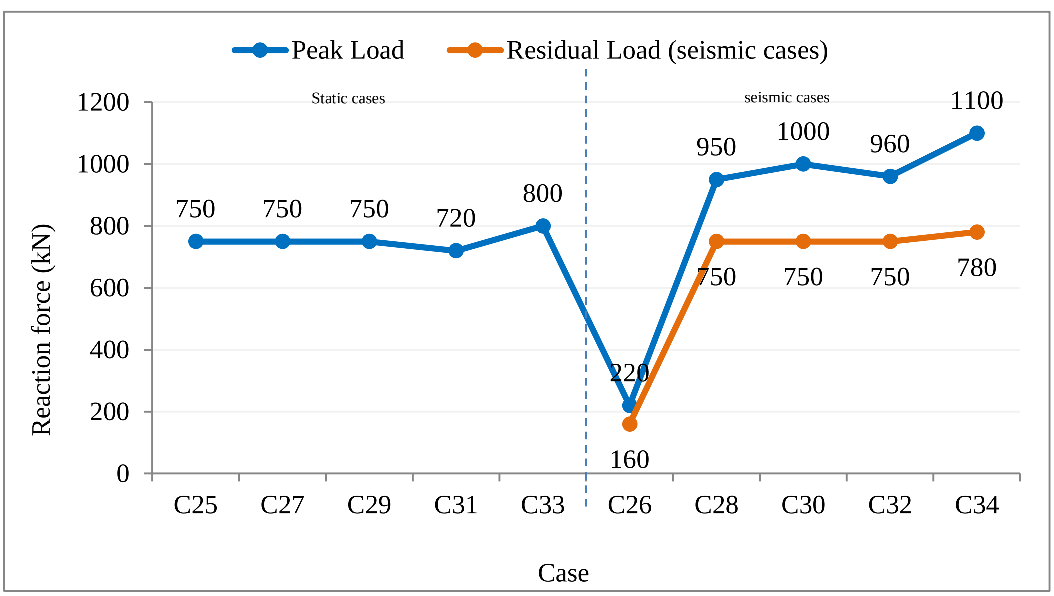
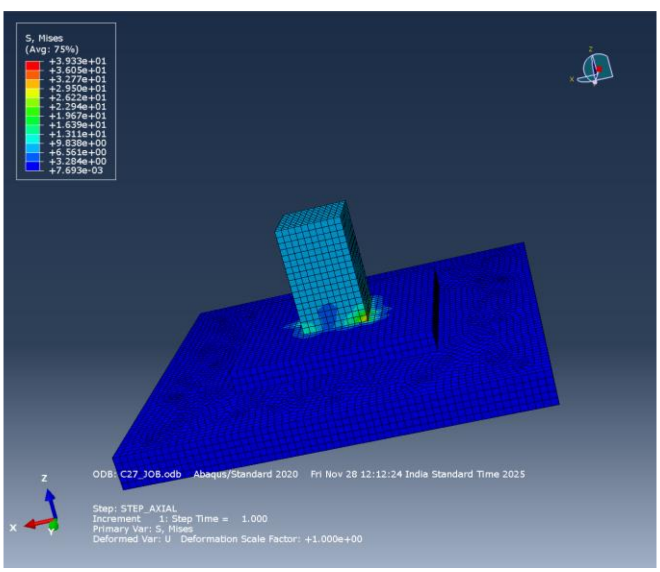


Fig. 7 Comparative peak and Residual load capacity response of drop-panel slab–column joints. Static cases are represented by peak Gravity load, while seismic cases are represented by peak lateral load and Residual load capacity-carrying capacity. The results show progressive improvement from unstiffened to stirrup-reinforced and stud-rail configurations, with the M40 stud-rail case (C34) exhibiting the highest seismic resistance.

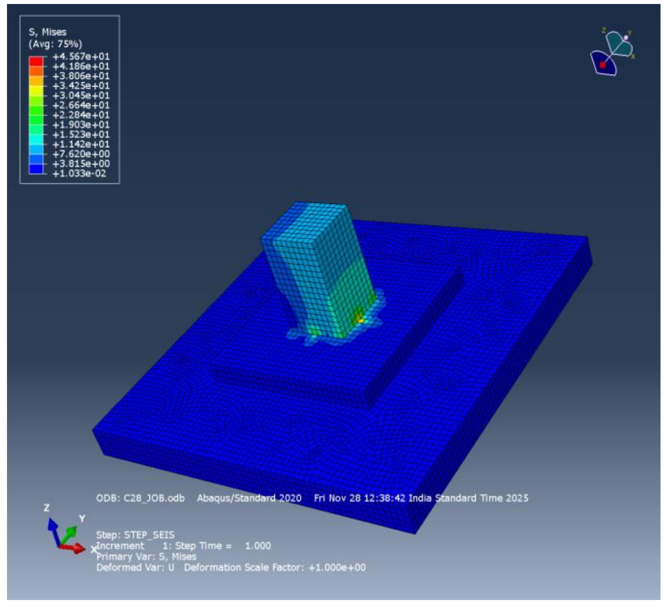
The increase in the peak lateral load capacity was from around 220 kN for C26 to 0.95 for C28 under lateral loading. The Residual load capacity was also substantially increased. The systems with stud-rains (C29-C30) also showed improvements in structural response. The residual capacity was maintained at about 750 kN even at small displacement, and the Peak lateral load (for seismic analyses) capacity was reached at a smaller displacement. The load-displacement response exhibited a more stable post-peak behaviour, indicating improved ductility and energy dissipation.Stiffness and maximum load capacity were further increased for the higher concrete grade (M40). But the unstiffened configurations had inferior deformation capacity and brittle behaviour. The C34 model of stud rail-reinforced M40 demonstrated the greatest numerical strength and Residual load capacity among all the models investigated. The results indicate that stud rails offer beneficial stress redistribution and post-peak performance under the constraints of the numerical model used.

4.2.3. Damage Patterns of Stiffened Drop-Panel Joints (C27-C34)

The patterns of damage of stiffened drop- panel slab-column connection are illustrated in Figure 8(a)-(h). Stirrups (C27-C28) give a more homogeneous stress distribution of the column, with the maximum stresses (~39 MPa) being localized, but with more gradual gradients and lower crack propagation under static loading. In seismic-loaded situations, despite the development of asymmetry, the presence of stirrups lowers the stress concentration and encourages controlled diagonal cracking (~45-46 MPa), which implies a higher ability to resist punching due to drifts. The stud-rail systems (C29-30 and C33-34) also promote behaviour, and the stress distribution is wider, the concentration of the column interface is reduced, and the cracking is more distributed, indicating more effective transfer of loads.



(a)



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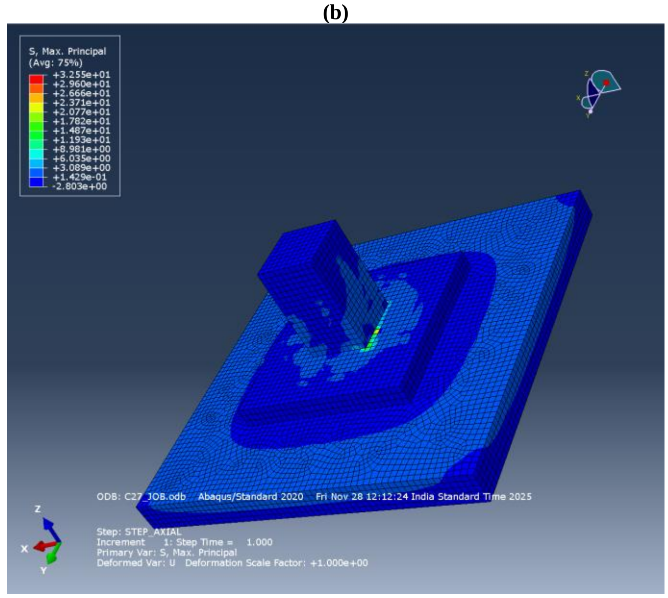

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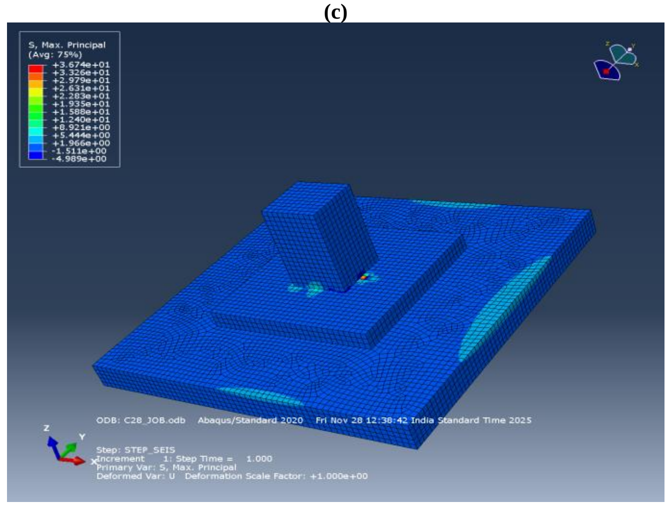

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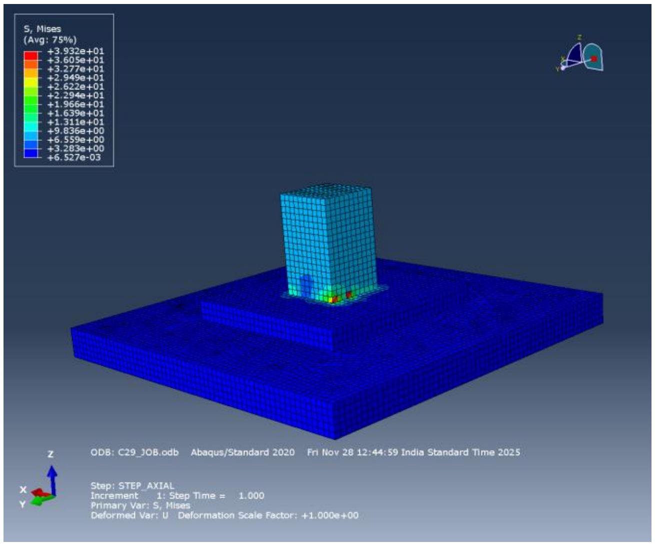

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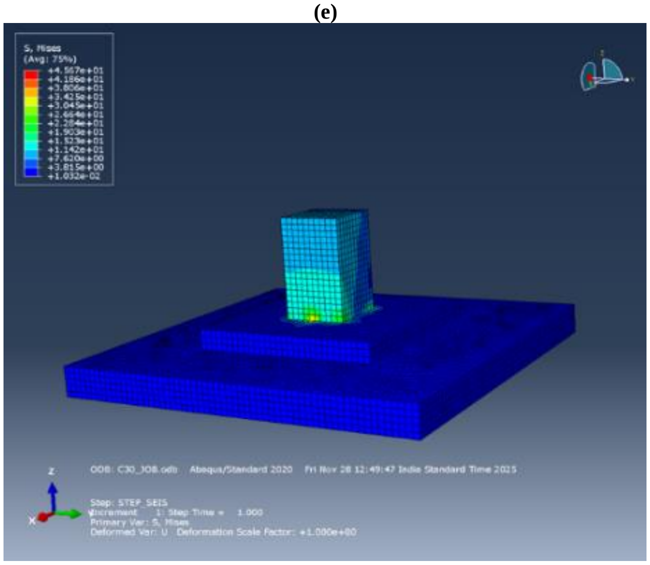
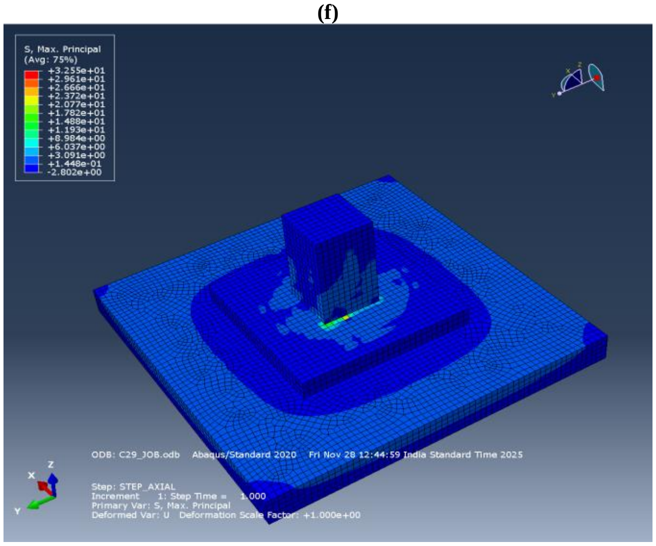
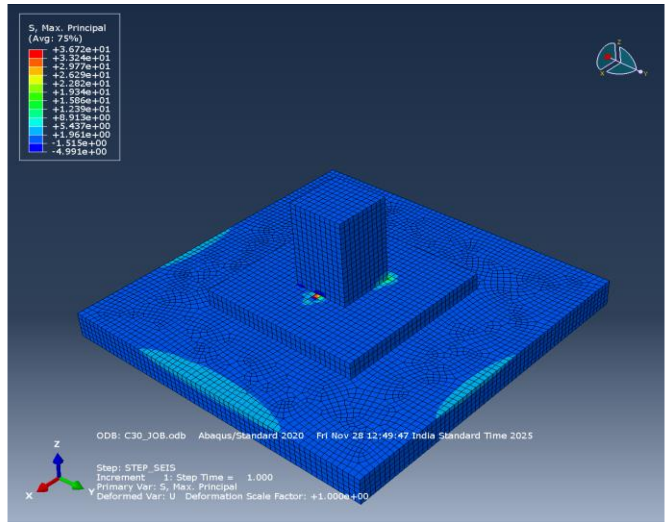


(f)



(g)



8(h)

Fig. 8 Stress distribution characteristics of stiffened drop-panel slab–column connections under static and seismic loading: (a) Von Mises stress contour – C27 (stirrups, static loading), (b) Von Mises stress contour – C28 (stirrups, seismic loading), (c) Maximum principal stress contour – C27 (stirrups, static loading), (d) Maximum principal stress contour – C28 (stirrups, seismic loading), (e) Von Mises stress contour – C29 (headed stud rails, static loading), (f) Von Mises stress contour – C30 (headed stud rails, seismic loading), (g) Maximum principal stress contour – C29 (headed stud rails, static loading), and (h) Maximum principal stress contour – C30 (headed stud rails, seismic loading).

In higher concrete grade (M40), there is a rise in the stress, and unstiffened cases (C31-C32) have localised and brittle behaviour, and the use of stud rails (C33-C34) is effective in reducing this by redistributing stress better. In general, stirrups enhance confinement and crack control, and stud rails offer better stress redistribution and localized punching converted into a more distributed mechanism of failure. Reinforcement is necessary to restrain brittle behaviour in concrete with higher strength.

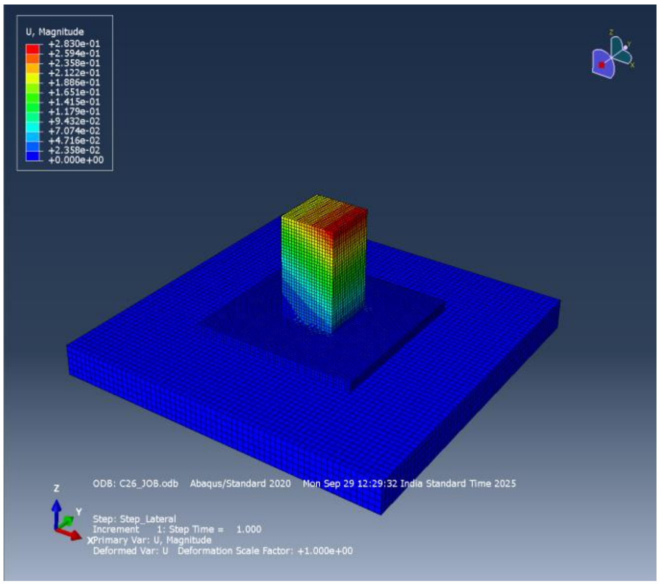
4.3. Deformation Behaviour and Load Transfer Mechanism

Slab column connection deformation behaviour indicates the efficiency of reinforcement to contain punching response. In those without reinforcement (C25-C26), the deformation is localized at the slab column junction.

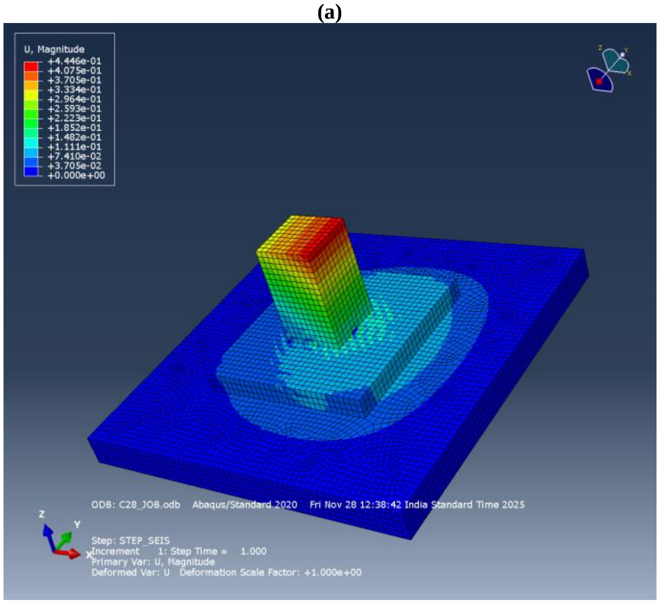
When loaded under a static (C25) condition, the displacement is smooth, symmetrical, which means that the load transfer is stable, and when loaded under a seismic condition (C26), the lateral displacement and joint rotation are high, which results in asymmetric deformation and deterioration of the stiffness. Using stirrups (C27-C28), the deformation becomes lower and more evenly distributed, with lower displacement gradients and better stiffness.

Lateral displacement and rotation are well contained under Lateral loading, which means that the confinement is enhanced. There is the most favourable response of stud-rail systems (C29-C30 and C33-C34) with more homogeneous displacement distribution and lower concentration around the column, and much lower rotation under Lateral loading, which is indicative of better load transfer and ductility. In the case of a higher concrete grade (M40), deformation is lowered because of the increased stiffness, but when not reinforced, localized and brittle behaviour occurs, and in such cases, the use of stud rails enhances the distribution and stability of deformation.

In general, unstiffened systems exhibit local deformation and increased rotation, enhancement of stiffness and deformation control by the use of stud rails, and the most consistent behaviour and minimum joint rotation by reinforcement, which is vital to avoid brittle behaviour in concrete with higher strength.



(a)



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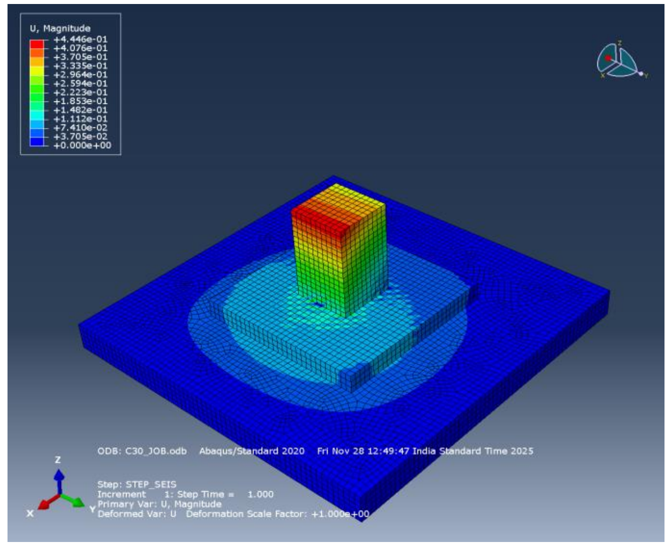

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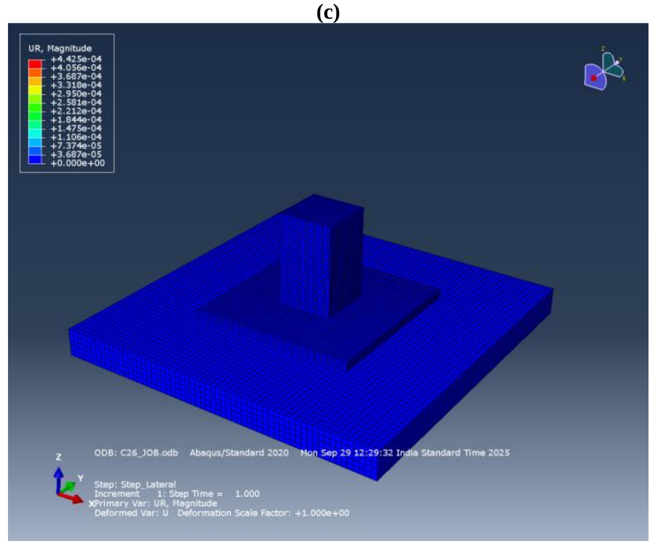

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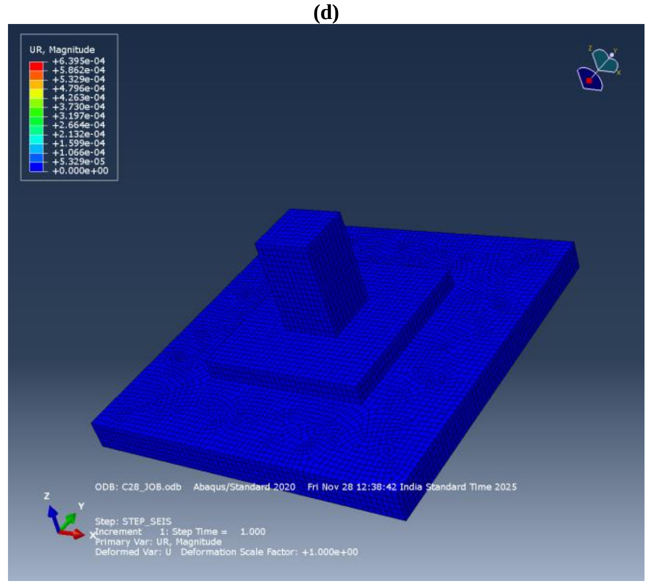

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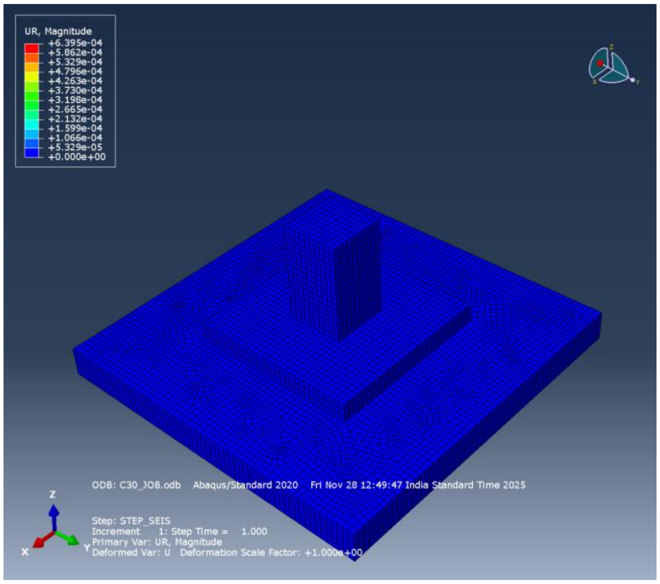


(f)

Fig. 9 Deformation behaviour of stiffened drop-panel slab–column connections under seismic loading: (a) total displacement contour (U, Magnitude) – C27 (stirrups, static loading), (b) total displacement contour (U, Magnitude) – C28 (stirrups, seismic loading), (c) total displacement contour (U, Magnitude) – C29 (headed stud rails, static loading), (d) rotational displacement contour (UR, Magnitude) – C27 (stirrups, static loading), (e) rotational displacement contour (UR, Magnitude) – C28 (stirrups, seismic loading), and (f) rotational displacement contour (UR, Magnitude) – C29 (headed stud rails, static loading).

4.4. Comparative Performance of Slab-Column Joints (C25-C34)

Figure 9 summarizes the comparative performance of all slab-column specimens (C25-C34) under static and Lateral loading and gives the values of the maximum Gravity load, the maximum lateral load and the remaining load-carrying capacity at high levels of deformation.

In the case of M35, the unstiffened specimen (C25) has the lowest peak axial capacity of about 690-750 kN. Shear reinforcement is introduced to give better results, both stirrups (C27) and stud rail (C29) reach a high of about 750 kN on the axial capacity. The deformation demand of Peak lateral load (for seismic analyses) is, however, lower in reinforced cases, which show better stiffness.

When the Lateral loading is applied, the maximum lateral capacity is found to rise to about 220 kN in C26 to about 950 kN in C28 and eventually to about 1000 kN in C30. Further, the Residual load capacity at large displacement (~30 mm) is constant at about 750 kN at reinforced cases, which means that it has better post-peak behaviour and energy dissipation.

The same trend is noted in the case of the M40 series. The unsupported statical (C31) case exhibits a maximum of 720 kN in the axial capacity, and this value rises to about 800 kN in the stud-rail case (C33). Both C32 and C34 have high peak lateral capacities (~1100 kN) under Lateral loading, but the reinforced case (C34) has a better residual capacity (~780 kN) and more stable behaviour after the peak than the unstiffened arrangement.

The comparison shows clearly that shear reinforcement has tremendous effects to strengthening and ductility. Although stirrups give better performance compared to unstiffened systems, the stud-rail reinforcement gives the best overall performance based on all performance indicators, such as peak strength, stiffness, and residual capacity.

Moreover, a high concrete grade (M40) enhances the peak strength, but it also causes more brittle behaviour in the case of no reinforcing. On the whole, drop panels combined with stud rails are the most favourable to use, especially when subjected to Lateral loading, when both peak and residual capacity are of paramount importance to structural safety.

Following the comparative assessment of structural performance presented in Figure 10, the numerical results are further validated against codal punching shear predictions.

4.5. Comparative Analysis of Numerical Punching Shear Capacity Prediction with Codal Prediction for Validation Purposes

For the purpose of verification and validation of the numerical model used, a comparative analysis was performed between FEA results obtained for the prediction of punching shear capacity with those from standard design codes such as IS 456:2000, ACI 318 and Eurocode 2. Comparison was carried out in select cases based on variation in concrete grades and reinforcement configurations.

4.5.1. Codal Formulations

The punching shear capacity was evaluated using standard expressions:

- IS 456:2000 (Clause 31.6: Punching Shear)

$$V_{u,IS} = \tau_c \cdot b_0 \cdot d \text{ where } \tau_c = 0.25\sqrt{f_{ck}}$$

- ACI 318 (Section 22.6: Two-Way Shear Strength)

$$V_{u,ACI} = 0.33\sqrt{f_c'} \cdot b_0 \cdot d$$

- Eurocode 2 (EN 1992-1-1:2004) (Clause 6.4: Punching Shear)

$$V_{u,EC2} = 0.18k(100\rho f_{ck})^{1/3} b_0 d$$

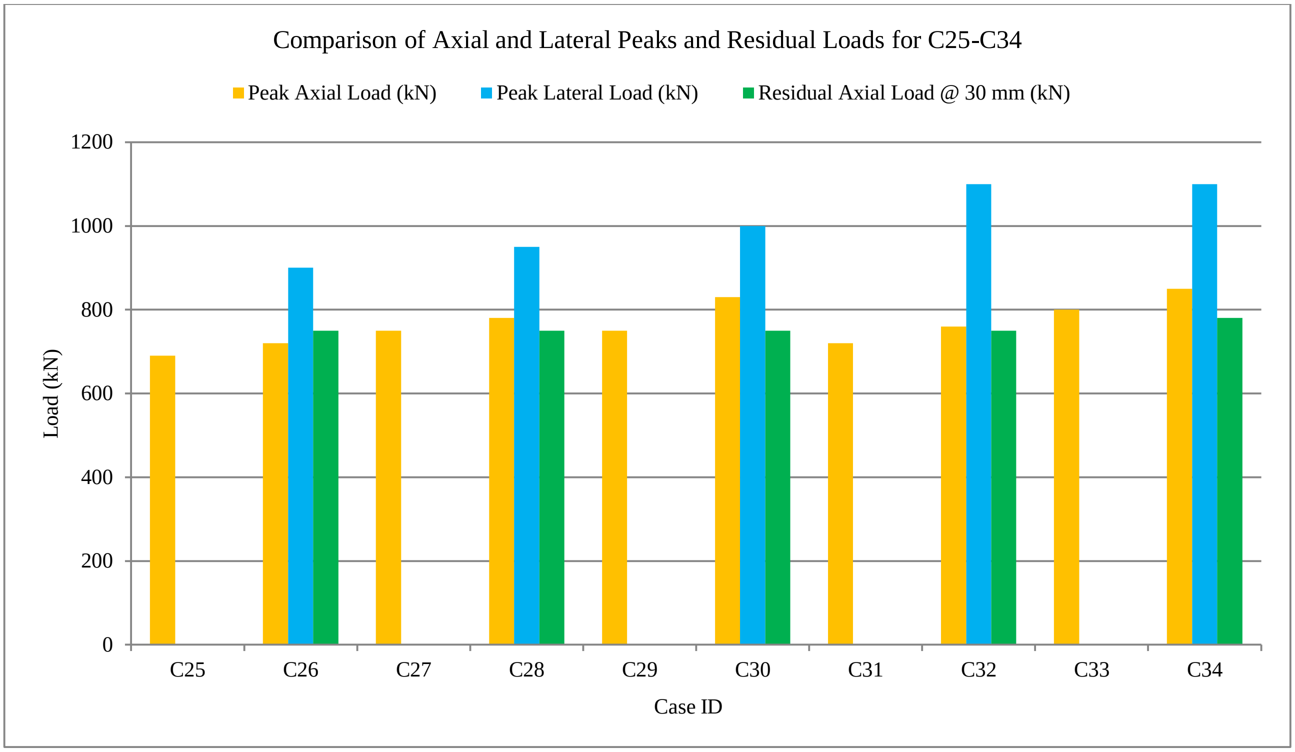


Fig. 10 Comparative performance of slab–column joints (C25-C34) showing peak Gravity load, peak lateral load, and Residual load capacity-carrying capacity at 30 mm displacement under static and Lateral loading, highlighting the effect of shear reinforcement and concrete grade.

4.5.2. Comparative Results

Table 3. Comparison of Punching Shear Capacity (FEA vs Codes)

Case	Concrete Grade	Reinforcement	FEA Capacity (KN)	IS 456 (kN)	ACI 318 (kN)	EC2 (kN)	FEA/IS	FEA/ACI	FEA/EC2
C25	M35	None	750	699	923	780	1.07	0.81	0.96
C27	M35	Stirrups	750	699	923	780	1.07	0.81	0.96
C29	M35	Stud rails	750	699	923	780	1.07	0.81	0.96
C31	M40	None	720	747	985	820	0.96	0.73	0.88
C33	M40	Stud rails	800	747	985	820	1.07	0.81	0.98

4.5.3. Discussion of Results

The differences between codal predictions and numerical results are more apparent in the cases reinforced using studs because most simplified punching shear provisions are geared toward estimating the nominal punching resistance of the concrete, which depends on the concrete strength, effective depth, column geometry, and critical shear perimeter. Shear reinforcement is not always explicitly included in the design provisions, and the equations do not fully capture the three-dimensional stress redistribution, anchorage effect of headed studs, crack-bridging mechanism and post-Peak lateral load (for seismic analyses)-transfer capacity seen in nonlinear finite element simulations. The stud-rail configurations in this present study resulted in better residual strength and stress transfer around the slab–column interface, while the codal formulations primarily predicted ultimate punching strength and failed to reflect ductility, damage evolution, or energy dissipation. Thus, it becomes necessary not only to view the disparity between the two as an error in strength prediction, but also as a shortcoming of codified formulae in modelling the behaviour of reinforced drop panels under gravity and lateral loads.

As can be seen from the comparison given in Table 3, the codified predictions tend to be conservative in comparison to the numerical values. The IS 456 forecasts have the least variation with FEA values, and the ratios are 0.96-1.07, which means that there is acceptable precision when it comes to practical design. Conversely, ACI 318 has a consistent overestimation in the punching shear capacity, and FEA/ACI ratios are within the range of 0.73-0.81, indicating that the code is not able to account for the stress redistribution and confinement effects that drop panels and reinforcement introduce to the structure. Eurocode 2 offers intermediate predictions which are more correlated but conservative in some instances. The effect of reinforcement is not clearly reflected in the fundamental codal expressions, especially in the stud-rail systems.

Although the results of FEA indicate a substantial increase in load-carrying capacity and post-peak behaviour caused by the addition of stud rails, concrete contribution is dominant in the codal equations and does not entirely represent an improvement in shear transfer mechanisms. The impact of concrete grade can also be seen in which concrete of a higher grade (M40) has a high capacity, but low ductility. This behaviour is partly described by codal equations, but which do not take into account the post-peak performance and deformation behaviour.

4.5.4. Key Findings

The numerical model was validated by comparison with the already available models like IS 456, ACI 318, and Eurocode 2. In the current study, however, no experiments were carried out to validate the models, and only nonlinear finite element analysis was conducted.

Hence, the obtained numerical results should be used as comparative performance indicators and further experimental investigations are suggested to validate the observed trends under realistic loading conditions.

- The IS 456 is the best match of the FEA results.
- ACI 318 is prone to exaggerating punching shear capacity.
- Eurocode 2 has moderate consensus.
- Codal equations fail to explain fully the effects of reinforcement.
- FEA better reflects the redistribution and confinement of stress.

4.6. Sensitivity Analysis

In order to quantify the effect of significant parameters, which are shear reinforcement type, concrete strength and loading condition on slab column joint performance, sensitivity analysis was carried out. The results have been summarized in Figure 11(a)-(c).

It should be noted that shear reinforcement contributes to the most effective performance among all three factors, as can be seen from Figure 11(a). For instance, the maximum lateral loading capacity without shear reinforcement reaches about 900 kN, and then it goes up to around 950 kN, and finally increases to about 1000 kN. In a higher concrete grade (M40), the maximum lateral load is about 1100 kN in C32 and C34, but in the reinforced case (C34), the stability is better and the sustained load carrying capacity. The numerical results suggest that stud rails provide a more effective punching shear resistance mechanism through improved load transfer and crack control when compared with the other investigated configurations.

The effect of concrete grade is shown in Figure 11(b). When the concrete grade is increased to M35 to M40, the peak of the load capacity of concrete is increased by an average of about 690 kN (C25) to about 720 kN (C31), and by an average of about 900 kN (C26) to about 1100 kN (C32), respectively, in the case of static and Lateral loading. The increase in strength is, however, coupled with a decrease in deformation capacity and brittle behaviour, especially in unstiffened forms. Figure 11 (c) shows the Residual load capacity bearing capacity at large displacement.

Reinforced configurations (C28, C30, and C34) exhibit a rather steady residual capacity of about 750-780 kN at 30 mm displacement, but the unstiffened systems are characterized by the rapid degradation of strength. Optimal residual capacity is noted in the stud-rail-reinforced M40 case (C34) with enhanced post-peak behaviour and energy dissipation.

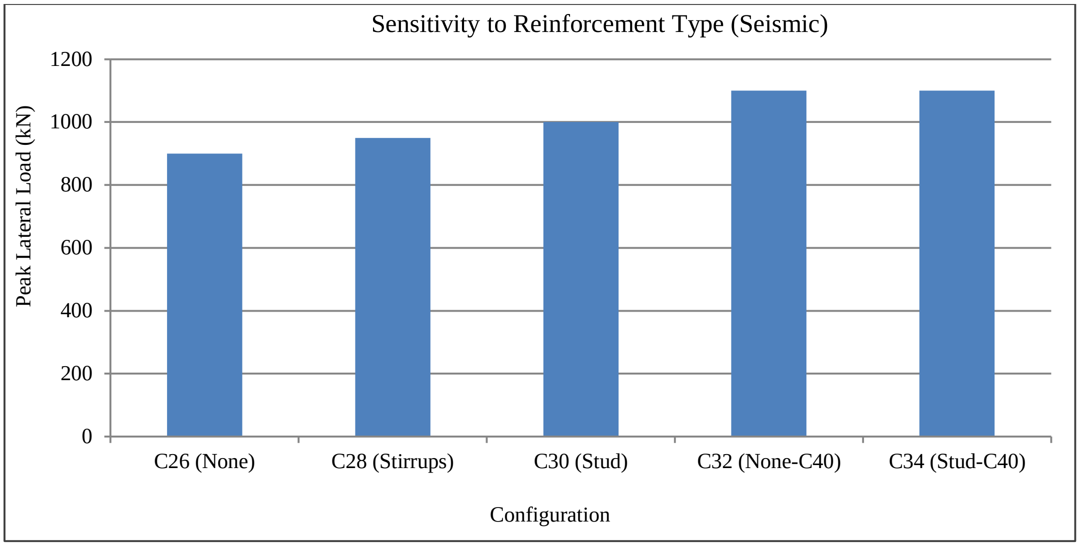
On the whole, from the results obtained through sensitivity analysis, it can be said that the most significant influencing factor on the structure is the nature of the shear reinforcement, followed by the nature of the loading and grade of concrete. Increased concrete strength helps in increasing it does not ensure improved ductility. Therefore, to achieve stable and reliable performance under combined loading conditions, the provision of shear reinforcement-particularly stud-rail systems-are essential.

4.7. Discussion of Modelling Uncertainty

Numerical predictions from the finite element analyses depend on various modelling assumptions and material parameters. All these values have been selected based on the literature validated, and the stable convergence of the results has been achieved; however, there is a degree of uncertainty associated with the actual material behaviour represented by the values adopted.

Also, the idealization of the bond between reinforcement and concrete, homogenous material properties, as well as simplified loading conditions, can result in discrepancies between numerical analysis and actual structural behaviour.

However, the relative performance of drop panels, stirrups and stud rails remained unchanged across the analyses, suggesting that the key findings on relative performance were not sensitive to the modelling uncertainties.



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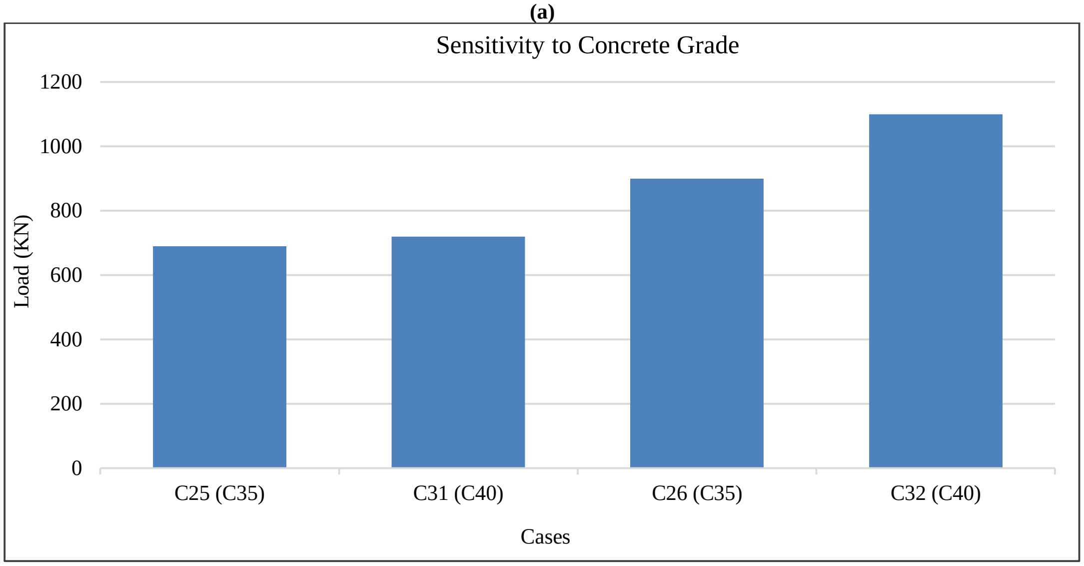

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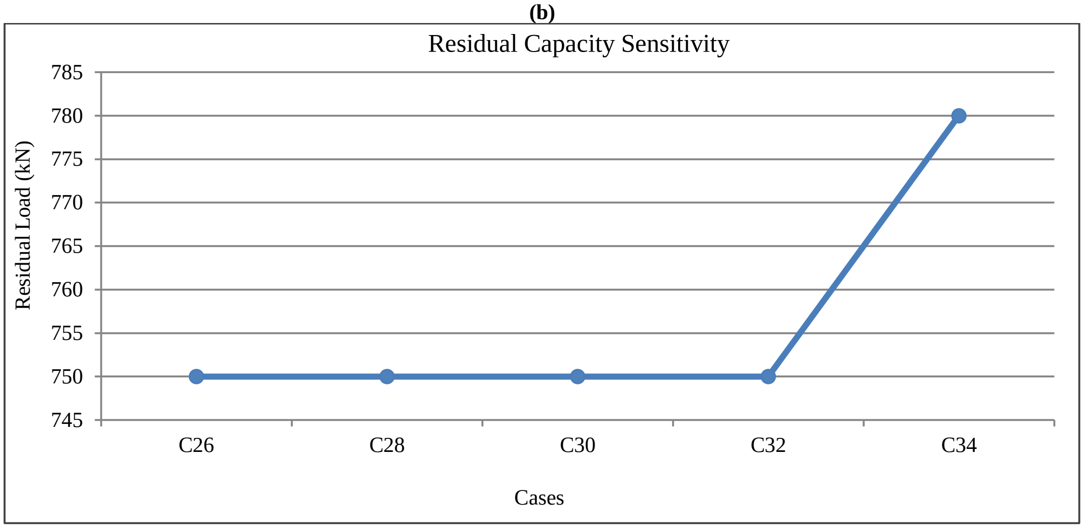

(c)

Fig. 11 Sensitivity analysis of slab–column joints: (a) Effect of shear reinforcement type on peak lateral load under Lateral loading, (b) Effect of concrete grade on Peak lateral load (for seismic analyses) capacity, and (c) Residual load capacity-carrying capacity at 30 mm displacement, highlighting the dominant role of reinforcement in controlling strength and post-peak behaviour.

4.7.1. Practical Implications of the Numerical Study

It is observed from the analysis that the inclusion of shear reinforcements in the design significantly increases the resistance to punching shear effects, provides ductility and energy dissipation properties under lateral loads. The best performing system in terms of load-bearing capacity and post-peak response was the stud rail-reinforced system. In terms of practical design, the findings indicate that the combination of drop panels and shear reinforcement might offer improved structural reliability in flat slab structures in earthquake-prone areas.

The numerical predictions, however, do not reflect all sources of variability that are observed in real construction, such as material variations, tolerances in construction, errors in the placement of reinforcement, and cyclic degradation effects. Thus, the results presented should be viewed as comparative design guidance and should be validated in experiments prior to direct application in engineering practice.

5. Conclusion

On the basis of the complete numerical and comparative study of drop-panel slab-column joints (C25-C34), the following conclusions are made:

Unstiffened Drop-Panel Joints Performance: It is observed that for the unstiffened drop-panel joints (C25-36 and C31-32), stable performance has been recorded under static loading, while degradation in stiffness and strength occurs under lateral loading. Despite the fact that drop panels enhance the initial stiffness and delay the onset of punching failure, the response is brittle with low ductility to combined axial-lateral loading, meaning that geometric enhancement is not enough to ensure reliable seismic performance.

Effect of Shear Reinforcement (Stirrups): Adding stirrups (C27-28) increases confinement, slows crack propagation and increases peak and residual strength. There is an increase in the peak lateral capacity to a range of 950 kN (C28) instead of 220 kN (C26), and better post-peak stability. Load-Displacement response is smoother, which means that ductility is increased, but moderate strength deterioration still exists at greater levels of deformation.

5.1. Comparative Performance of Stud-Rail Systems

The peak lateral loads were around 1000–1100 kN, and the residual capacities were in the range of 750–780 kN at large displacements. The improvement in performance is said to be due to better stress redistribution and increased punching shear resistance. The observations are, however, based on numerical simulation and are to be understood as relative indications of the structural performance. Experimental investigations are recommended to further validate the observed behaviour under actual loading conditions.

Effect of Concrete Grade: If there is an increase in the concrete grade (from M35 to M40), there will be an improvement in the stiffness and ultimate load-carrying capacity (in the range of 720 kN up to 800-850 kN statically and 1100 kN in dynamic cases).

Nevertheless, concrete of greater strength has lower deformation ability, and more brittle post-peak behaviour on the unstiffened forms. This limitation can be effectively overcome by the combination of a higher concrete grade with stud-rail reinforcement, which enhances the redistribution of stress and ductility.

Sensitivity of governing parameters: From the sensitivity analysis results, it can be seen that the shear reinforcement type is the most significant factor in affecting the behaviour of punching shear strength, followed by loading conditions and concrete grades. There is increased peak strength and stable residual capacity of reinforced systems, especially systems with stud rails, and the systems that are not reinforced are very sensitive to Lateral loading and rapid deterioration of strength.

Comparison with Codal Provisions: Comparison with codal predictions indicates that IS 456 gives fairly close results in estimating the punching shear capacity, whereas ACI 318 and Eurocode 2 are more or less conservative. But the codal formulations that exist do not explicitly explain reinforcement configuration or post-peak behaviour, which is well represented in the numerical analysis.

Design Implications: From the results of the study, it is evident that the drop panels in conjunction with the provision of the shear reinforcement constitute the essential requirement for the safety and reliability of the slab-column joint. In conclusion, from the results of the nonlinear finite element analysis, it has been found that the drop panel and stud-rail reinforcement system provided good performance in terms of strength, stiffness, remaining strength, and ductility amongst all the systems studied. Therefore, they cannot be considered for application in any future design until they are experimentally validated.

Additional research will be of great importance when it comes to assessing the relative effectiveness of stud-rail reinforcement from the numerical results, as well as assessing its practicality of implementation under real-world loading conditions.

Future research can be carried out to provide experimental evidence for the results obtained numerically, especially regarding the drop-panel slab-column joints subjected to Lateral loads. Future research can also be done regarding other systems including the use of FRC, polymer-reinforcement system, etc.

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