Composite slab effects on beam-column subassemblies: Further development

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ABSTRACT: Composite slab construction is gaining popularity in New Zealand. These slabs may influence beam column joint subassemblies as they are exposed to earthquake-induced shaking. However, several design issues with composite slabs need to be addressed so that they can be used to their full advantage in design. These relate to considering the effect of the slab on the beam design strength, the likely statistical variation of the beam and slab under strong seismic shocks that will affect the column joint demand, and the resistance of the panel zone.

In this paper, experimental test setups are described which consider slab isolation, beam overstrength, full depth slab around the column, low damage connection, and demand on the panel zone. A new concept of slab confinement using a shear key will be presented to form a force transfer mechanism to avoid failure of concrete either in crushing or spalling. Also the development of a non-prying sliding hinge joint low damage connection and its performance with composite slabs is discussed. The outcome of this will be useful to develop simple design recommendations for the New Zealand steel standard.

1 INTRODUCTION

After the recent earthquakes, there is wide acceptance of steel frame structures with composite deck slabs in the New Zealand construction industry. Here beam-slab composite action is achieved using steel studs welded onto the beams and cast into the concrete slab. These slabs can affect the seismic performance of beam-column subassemblies. However, their effect is not considered in the beam design, so beam sizes cannot be reduced due to this composite action. At the beam ends, the concrete slab may be connected to, or separated from, the column. If a gap is left between the slab and column, the design is easy since there is no need to consider the effect of the slab on the connection as well as on the panel zone. However, there is a greater possibility of column instability and local buckling as column restraint is reduced because of the separation. In addition, there may be an increase in beam axial force caused by slab inertial effects or “transfer” of forces across the floor diaphragms. If no gap is provided, force transfer between the slab and column face may occur through bearing under beam end sagging moments. However, in the case of hogging moments, the slab reinforcement is activated to transfer the forces in the slab around the column. Slab forces can increase the demand on the connection, panel zone and column, possibly resulting in an undesirable inelastic deformation mechanism. In no-gapping configurations, the participation of the slab to beam overstrength is considered only in the New Zealand code (NZS3404:1997) for column design. This overstrength factor is also affected by material characteristics. For economical design, it would be advantageous if the slab contribution to the beam strength could be considered both in the traditional bolted end plate connection as well as in newer low damage sliding hinge connections. Also for the overstrength design, realistic estimation of demand from the beam and slab are required for the column and panel zone. Design guidelines are needed to quantify the slab effect on the beam-column subassemblies. This paper aims to answer the following questions:

1) What are the effects of different slab details on panel zone demand and column demand in moment-resisting frames with traditional bolted end plate connections?
2) What is the effect of the slab on the performance of low-damage sliding hinge joint connections?

2 BACKGROUND

2.1 General

Research studies carried out in the past reveal that the ultimate strength of the test beams depends on the column face width, slab thickness, concrete strength and the steel yield strength (DuPlessis, 1972). Several researchers noted the following test specimen failure sequence: first the yielding of beam bottom flange, then column web panel yielding, and finally slab degradation due to crushing/spalling of the concrete near the column flange ((Lu and Lee,1989), (Leon et al., 1998), (Hobbs et al., 2013)). The slab effect at the column face reduces the top flange stresses and delays the beam local and lateral torsional buckling (Civjan et al., 2001). Slab performance can be improved by providing extra reinforcement near the connection or by providing a full depth slab around the beam-column joint (Leon et al. 2004).

2.2 Slab Confinement

A composite slab causing compression on a steel column is typically confined on three sides. This confinement is offered by a steel deck below the slab and by adjacent slabs on either sides of the effective slab being considered. There is generally no confinement on the top, so the stress and strain associated with the initiation of spalling can be conservatively considered to be the unconfined concrete crushing strength f'_c at a strain ε_c of approximately 0.002. The spalling can be determined by two possible ways; the first is spalling assessed from *strength considerations* and the second is *strain compatibility considerations*. The current NZS3404:1997 code specifies that the location of the first shear stud/connector should be at 1.5 times the depth of the beam from the column face. This is to avoid any stress concentration in the beam-yielding zone. The concrete compressive strain in this zone must be less than 0.002 for spalling to be avoided. However, limiting the strain may be difficult. Spalling failure has been observed by Hobbs during recent testing at the University of Canterbury. A possible means of increasing the concrete slab strain capacity in this zone (1.5 x beam depth) may be by applying confinement at the top of the slab.

2.3 A Strut and Tie Mechanism

Several research studies ((Salvatore el al., 2005), (Braconi et al., 2008)) have suggested that the strut–and–tie mechanism is necessary to resist the force applied by the column on the slab. Primarily, two force transfer mechanism are developed, known as “Mechanism 1” and “Mechanism 2” (Braconi et al., 2010) as shown in Figure 1.

When no transverse beam is present, the moment capacity of the joint may be calculated from the compressive force developed by the combination of Mechanism 1 (direct compression on the column flange) and Mechanism 2 (compressed concrete struts inclined at 45° to the column sides). These mechanism formations depend upon the width of column flange, enhancement in bearing by providing any additional plates, depth of slab available at the column face for the bearing, and provision of extra reinforcement around the column. The direction of the deck rib may also affect the formation of these force transfer mechanisms.

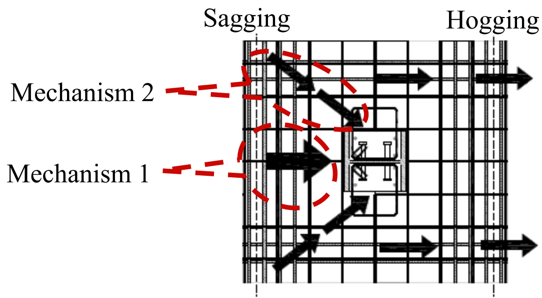
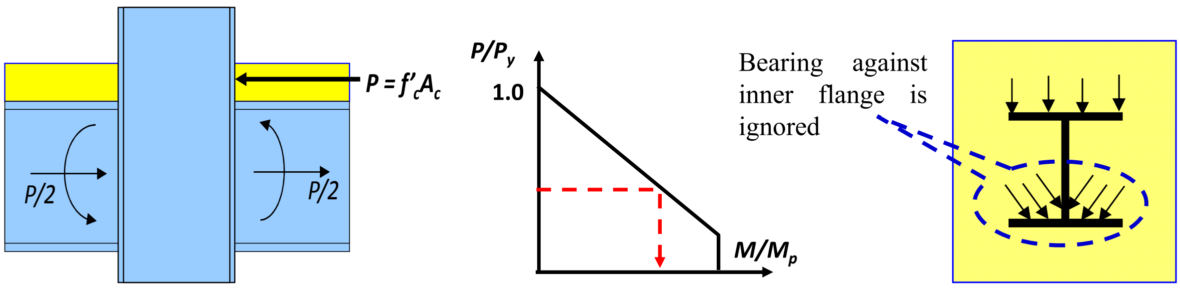


Figure 1: Slab Internal Force Interaction between Hogging and Sagging side (Braconi et al. 2010)

2.4 New Zealand Design Approach (NZS3404)

The New Zealand Steel code, NZS3404:1997, accounts for the effects of slab-column interaction in the calculation of overstrength moments at the column face. The overstrength moment capacity of each composite beam is calculated by applying an overstrength factor of 1.25 to the nominal moment capacity of the steel beam and then multiplying by a factor which represents the contribution of the composite slab. The overstrength moments at the column face are then calculated by taking the sum of the overstrength capacities of the composite beams framing into a joint and adding the moment caused by the axial load of the slab acting over a lever arm between the slab centroid and the beam centroid, as shown in Figure 2(a). The beams framing into the joint share a horizontal equilibrium. However this axial force reduces the moment capacity of the beams in accord with standard axial force-moment interaction as shown in Figure 2(b), where P = axial force developed in slab, f'_c = specified concrete cylinder compression strength @ 28 days, and A_c = effective area of concrete slab in the composite beam.



(a) Axial and Flexural Forces imposed by beams

(b) Axial Force- Moment interaction

(c) Assumed Load Distribution on Column

Figure 2. Effect of Slab Axial Force on Column Joint (MacRae et al. 2010)

The axial force of the slab is calculated based on its compressive capacity, with the condition that this must not exceed the axial capacity of the beam. As per C12.10.2.4, NZS3404:1997 code amendment, the overstrength due to the slab together with the beam, ϕ_{omss} should be calculated as, $\phi_{omss} = \phi_{oms}\,(1.0 + 1.08\ tef/db)$ giving the overstrength moment shown in Equation 1, where $M°$ = overstrength moment from the composite beams at the column face, tef = thickness of the concrete rib in direct contact with the column, db = depth of d steel beam, ϕ_{oms} = overstrength factor for the beam alone, and M_s = nominal beam section moment capacity.

$$M° = \phi_{omss} \text{ x } M_s \qquad (1)$$

The method of accounting slab participation in the joint overstrength moment is based on the assumption that the slab is infinitely rigid and strong axially and carries the force through the concrete bearing against the outer column flange; However, the effect of inner column flange is ignored (Figure 2(c)).

For composite beams expected to sustain large seismic demands, clause 13.4.11.3.3(b) of NSZ3404:1997 states that, *"the slab should be reinforced and confined so that the steel beam can reach a maximum tensile strain of 24 times the yield strain before developing the nominal compression capacity of the concrete and compression reinforcement. Here the maximum compressive concrete strain reached is not permitted to be any more than 0.004."* References regarding possible means of achieving this are given in the commentary to this clause, but it is not known if this option has ever been used in practice.

2.5 Recent Experimental Test Observations at the University of Canterbury

A testing of full scale beam-column-slab subassemblies under cyclic loading with different steel deck directions (transverse and longitudinal), slab isolation, full depth slab around column junction and sliding hinge joint were recently conducted at the University of Canterbury (Hobbs et al. 2013) . All specimens were tested for varying drift levels from 0.2% to 5.0% drift as per ACI testing protocol. The hysteretic behaviour (Figure 3) of various tests shows that the (partially) isolated specimen had around 40% less lateral load resistance than the specimen with the slab in contact with the column flange. In the isolated-slab test, a 25mm thick polystyrene block was used to separate the slab from the column, but it was still in contact with the column web and at the gusset plates of the bolted end the plate connection with a haunch, which resulted in partial isolation. The test results show that some interaction between the slab and column had occurred as plate bearing forces were developed (Hobbs et al., 2013).

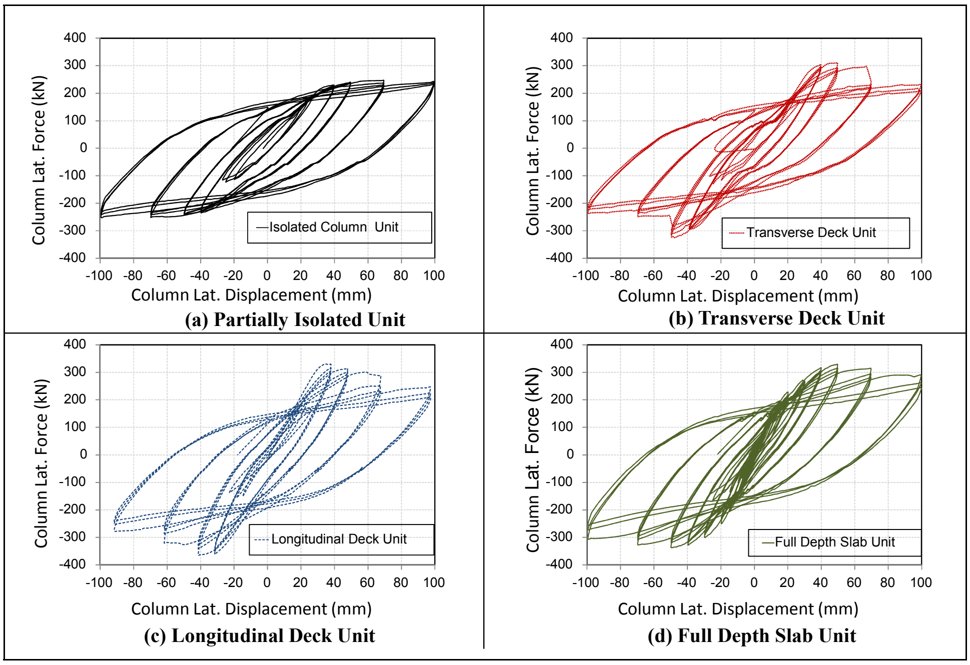
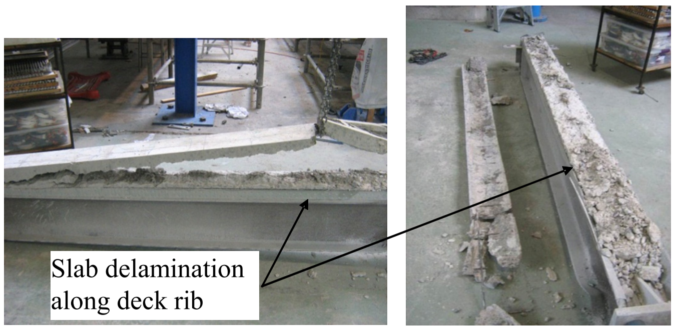


Figure 3. Subassembly Hysteresis Curves (Hobbs et al. 2013)

(a) Vertical Shear Failure



(b) Longitudinal Shear Failure

Figure 4. Different Shear Failure Modes (Hobbs et al. 2013)

At 5% drift all specimens had strength similar to that of the partially isolated specimen due to strength degradation. This degradation in the non-isolated specimens occurred at drifts from 2.5% to 3.5% because of shear failure of the concrete between the column flanges, shortly followed by spalling of the concrete. The area between the column flanges sheared away from the rest of the slab in all test specimens except in the full depth slab. While in the case of the deck running parallel to the primary beam, vertical shear failure as well as longitudinal shear separation, causing delamination of the composite deck slab, was observed, as shown in Figure 4.

2.6 Prying of Sliding Hinge Joint

A recent test on the sliding hinge joint (SHJ) points out two issues. The first one is the prying of top and bottom flange plates (Figure 5), and the second one is the bolt binding against the top and bottom side of the slotted hole, as shown in Figure 6 (Hobbs et al. 2013). This prying action can hamper the performance of the sliding hinge connection.

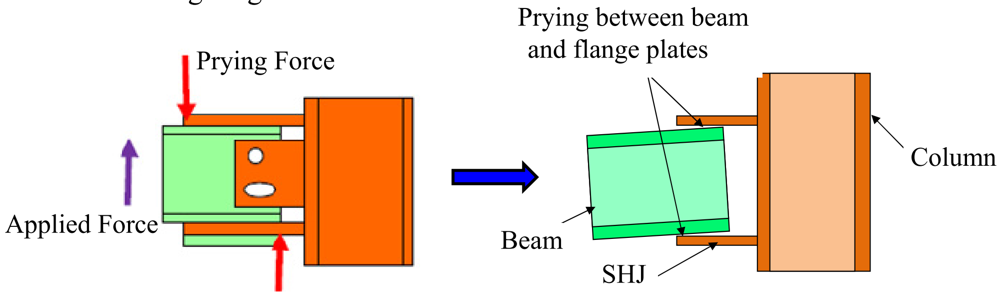


Figure 5. Prying of Sliding Hinge Joint (MacRae et al. 2013)

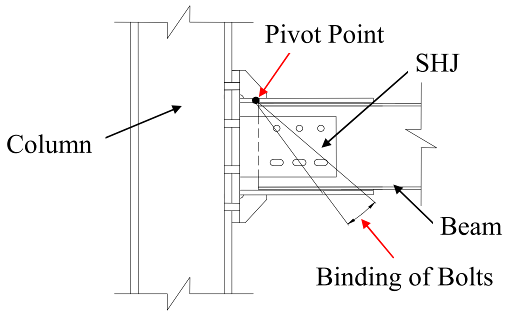


Figure 6. Illustration of Bolt Binding (Hobbs et al. 2013)

3 PROPOSED EXPERIMENTAL WORK

In order to quantify the slab effect on the beam-column-slab subassemblies, a series of full scale tests will be conducted at the University of Canterbury with different slab configurations, which are discussed in the subsequent sections.

3.1 Test Setup

The test specimens will be constructed and tested at the University of Canterbury structural laboratory. The column and beams will be pinned at the half-length of the span to represent the point of contra-flexure, as shown in Figure 7. The length of beam on each side of the column will be 3.0m and the column height will be 2.0m.The loading ram will be mounted at the column top. The slab width will be 3.0m to represent the tributary area of the interior beam.

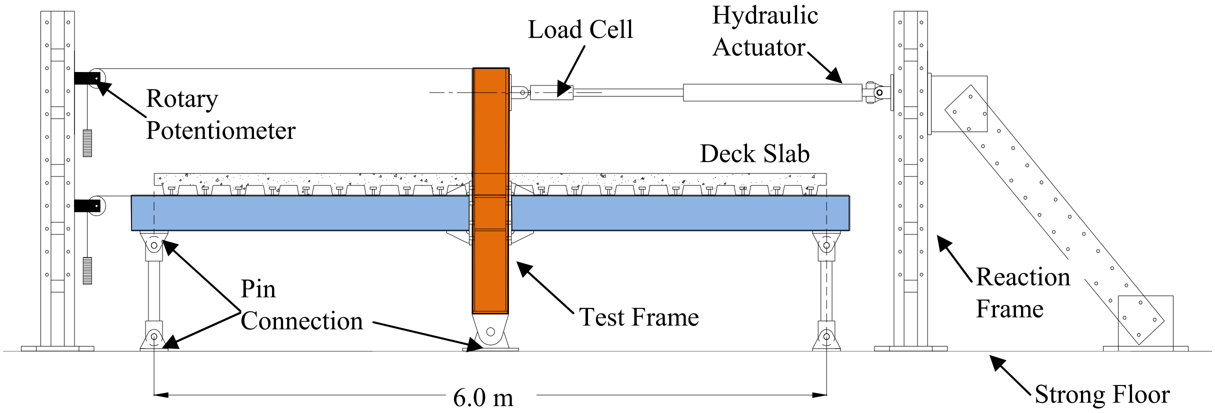


Figure 7. Proposed Test Set-up

3.2 Loading Protocol

The proposed experimental testing will be carried out using a displacement regime based on the testing protocol as per the ACI, 2001 as shown in Figure 8.

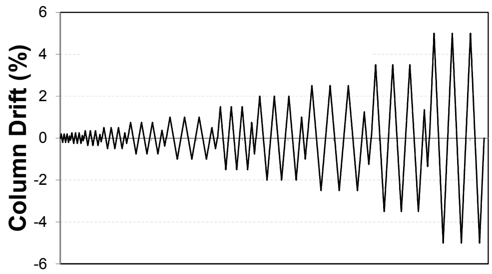


Figure 8. Test Regime (ACI 2001)

3.3 Different Test Configurations

3.3.1 Fully Isolated Slab

In this test configuration, the slab will be fully isolated from the column by using a polystyrene block, as shown in Figure 9. The aim of this test to study the effect of slab separation on the beam-column joint.

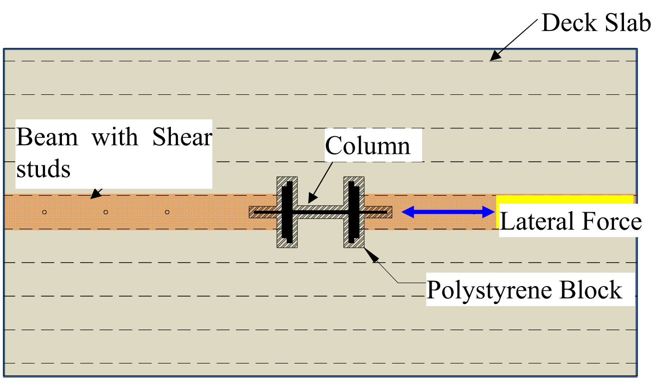


Figure 9. Fully Isolated Slab Assembly

3.3.2 Non Prying – Sliding Hinge Joint (NP-SHJ)

In the proposed non prying – sliding hinge joint connection, the issues related with the bolt binding and prying will be addressed. Wherein, the slotted holes will be provided in a radial direction, as shown in Figure 10. The top flange plate will be designed to remain in an elastic state, and at the bottom, the sliding plates will be oriented parallel to the beam web so that prying can be avoided.

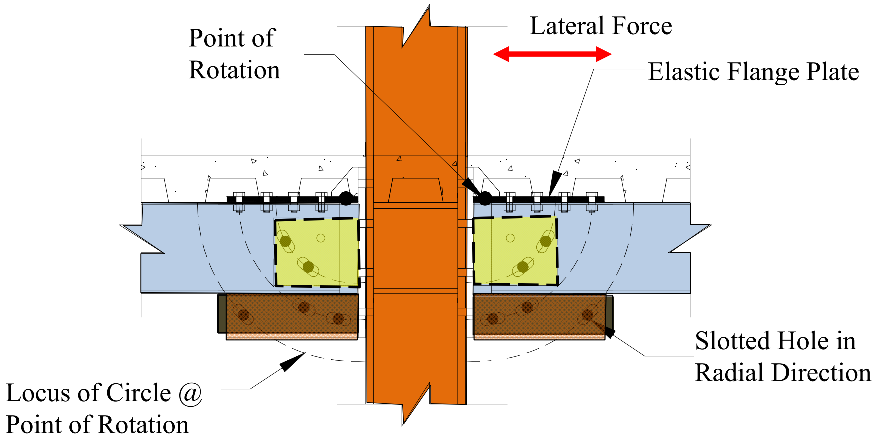


Figure 10. Proposed Non Prying Sliding Hinge Joint

3.3.3 Full Depth Slab with Confinement Reinforcement

In this proposed test configuration, the slab is confined at the top in order to enhance its strain capacity within the concrete and thereby to increase the deformation capacity of the subassembly as well as that of the slab. This will be achieved by placing a steel cage in the full depth slab region in front of the column, as shown in Figure 11. This has the advantage not only of confinement of the concrete, but also works as a part of the truss mechanism with longitudinal steel. This concept was also advanced in Section 13.3.5 of the "HERA : structural steel design guides, vol. 2"

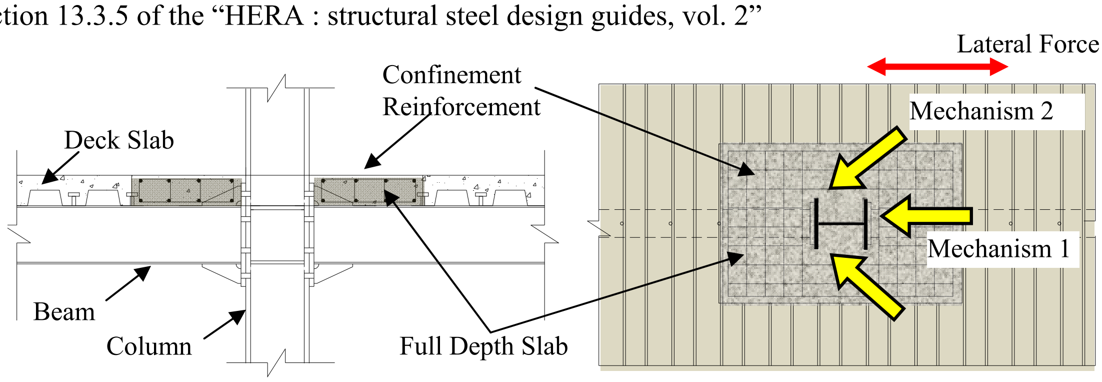


Figure 11. Proposed Arrangement of Confinement Reinforcement

Tests will be performed in two ways in order to study the effect of deck direction on strut and tie mechanism formation as well as on the shear failure mode. In the first test, the deck will be running perpendicular to the main beam; in the second test it will be running parallel to the main beam. In case of longitudinal deck assembly, the expected failure mode will be different than that observed in a recent test conducted at the University of Canterbury (Hobbs et al. 2013), due to the fact that there will be more deck ribs bearing against the full depth slab, which is wider than the column flange. The full depth slab with confinement reinforcement around the column will help to form a strut and tie mechanism as well as spreading of the in-plane force.

3.3.4 Provision of Shear Key between Column Flanges

The concept of shear key will be verified in this test configuration, wherein the shear key will contribute to arrest shear cracking when Mechanism 2 is activated (Figure 12). Such a detail allows reliable composite action at very little extra cost.

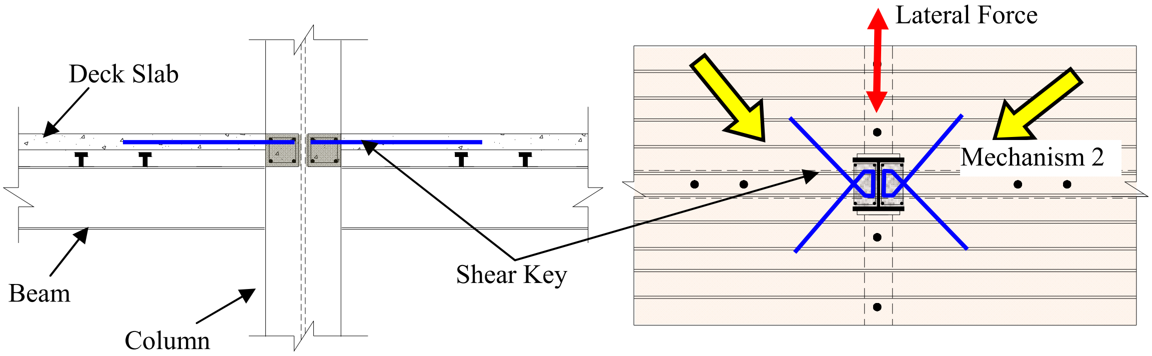


Figure 12. Proposed Arrangement of Shear Key

4 PERFORMANCE ASSESSMENT

The performance of the different test configurations will be assessed on the various parameters like: (a) Panel Zone Stiffness, (b) Beam Overstrength (and possible slab degradation), (c) Column Drift and (d) Beam Rotation. An analytical model will be developed using ABAQUS finite element software, wherein parametric study will be performed on the different slab configurations. Results obtained from the analytical study will be compared with the experimental test data in order to develop:

- Force (P) v/s Column Drift (δ) relationship considering the slab effect.
- Moment (M) v/s Beam Rotation (θ) relationship considering the slab influence.
- Overstrength Factor (*OSF*) incorporating the slab effect.

5 SUMMARY AND CONCLUSIONS

The current New Zealand Steel Standard (NZ3404:1997) considers the slab effect while sizing the column and designing the panel zone. However, this slab effect may benefit the beam design by reducing its sizes by incorporating the overstrength provided by the slab. The proposed research study will contribute to development of design recommendations, which will address the issues related to slab degradation, formation of strut and tie mechanism, and slab confinement. The concept of a novel non-prying sliding hinge joint connection will further enhance the adaptability of low damage connections into the construction industry. The outcome of this experimental work followed by the analytical study will result in the following design recommendations:

- Design provisions for confinement reinforcement to form strut and tie mechanisms.
- Overstrength factor for beam capacity design considering the slab effect.
- Design guidelines for non-prying sliding hinge joint connections.

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