Seismic performance of precast slab to beam connection: an overview

D. Vinutha, R. Vidjeapriya* and K. P. Jaya

Division of Structural Engineering, Department of Civil Engineering, College of Engineering Guindy, Anna University, Chennai 600 025, India

Precast construction is common across the world, but in India the level of acceptance is less, although it offers several benefits compared to the cast-in-place construction. Adopting a suitable connection for precast elements is very important in providing the overall robustness to the structure. Among all precast connections, the precast slab to beam connection is considered a vital one as the horizontal load is transferred to the vertical load resisting structural elements by the diaphragm action. This article mainly focuses on the seismic performance of the precast slab to beam connections with an experimental evaluation of the basic concepts of design and detailing of the connections. This overview will pave way to refine work on the slab to beam connections in future research.

Keywords: Connectors, hollow core slabs, precast slab to beam connection, seismic performance.

SINCE the early 20th century, use of precast concrete elements is being widely adopted particularly in the seismic prone countries such as New Zealand, Russia, China, Italy, Japan, Turkey, Indonesia, USA and Peru, due to their uniqueness and versatility. Precast construction has been increasingly replacing the traditional cast-*in-situ* construction because, it offers several advantages such as rapid speed of erection, better quality control, rapid construction on-site, lower manufacturing time and costs, reduced amount of scaffolding and formwork[1]. The precast structures can be dismantled, and they can be suitably used wherever required. The precast structure with large dimensions cannot be handled and transported as a single unit, hence they are manufactured as small parts that lead to creating the connections. These connections would allow proper transfer of loads from slab to the foundation and develop continuity like a monolithic structure without compensating the stability, capacity and durability of the structure.

For a structure to maintain its endurance and sturdiness, the connection between the structural components plays a vital role. Proper detailing, intactness and ductility of the connection ensure better performance of the structure. For a safe structure to maintain vertical support to the suspended flooring system, floor connection is the most basic requirement[2]. In the past, many cities have suffered severe damages due to the failure of the support system mainly due to the collapse of floors during the occurrence of various earthquakes, like Northridge (USA) Earthquake (1994); Turkey Earthquake (1998); Bhuj (India) Earthquake (2001); Canterbury (UK) Earthquake (2011); Kaikoura (New Zealand) Earthquake (2016). Previous studies on the earthquake observations have revealed that more than 90% of the damages can be attributed to inadequate detailing at joints and connections or errors made in the detailing, intactness of the structural elements, mistakes made in choosing the building configuration and poor construction quality caused by inadequate supervision[3,4]. The performance of precast slab to beam connections during past earthquakes is summarized in Table 1 (refs 5–12).

During strong earthquakes, the slab–beam connection undergoes severe cyclic loading. An earlier research has shown that the use of precast concrete floors with different support and diaphragm connections in certain cases was inadequate to accommodate the earthquake deformation. The floor system plays an important role in the lateral resistance of a structure, the slab transfers lateral load to the internal load resisting elements and the beam transfers vertical load horizontally into the structure. Therefore, detailing of the slab–beam connection is critical for the robust performance of the precast structure during an earthquake.

Extensive research has been done on precast structures, but only limited studies have been done on the slab–beam connections. This is a key concern, because, research on the slab–beam connections will provide insights on the load-carrying capacity, ductility and energy dissipation. In particular, studies have not been exhaustive on seismic resistance. This article reviews the studies conducted on the slab-beam connections in earthquake-prone regions and the various techniques, experimental investigations used for this connection.

Precast slabs and beams

The types of precast floor slabs can broadly be classified as fully precast slabs and partially precast slabs. The former is manufactured completely offsite, transported, and erected on site, whereas the latter has both cast-*in situ*

*For correspondence. (e-mail: vidjeapriya@annauniv.edu)

Table 1. Performance of slab to beam connections during past earthquakes

Earthquake	Structure	Connection	Failures observed
Armenia Earthquake[5,6] (1988)	Residential apartments	Hollow core (HC) slabs simply supported on the lateral force resisting system (LFRS).	Completely collapsed or extensively damaged because there were no metal anchors into LFRS and no inter-connections between the HC planks and lack of cast-*in situ* topping slab.
Northridge Earthquake[7,8] (1994)	Parking structure	Cast-in-place post-tensioned floor slab supported on precast beams.	Collapse of post-tensioned slab was triggered by the loss of support of the girders due to column flexure–shear failures in the interior of the structure, which pulled the exterior east frame inwards as it dropped.
	Parking structure	Deep double T-beams supported on bottom ledges of inverted T and spandrel beams.	1. A cast-in-place slab provided continuity over the double T beams and was connected to the spandrel beams around the perimeter of the structure by reinforcing bars. 2. Column failure occurred followed by subsequent loss of support and failure of the spandrel beams. 3. Another factor was the lack of continuity reinforcement across construction joints in the cast-in-place concrete slab that was intended to serve as a diaphragm.
	Parking structure	HC slabs supported on precast beams.	Damage to the exterior columns was also due to the presence of an access ramp, which introduced significant torsional eccentricity in the structure and interrupted the flow of forces in the horizontal diaphragm.
	Multi-storeyed apartment	HC slab supported on beam.	1. Loss of seat and collapse of the complete unit. 2. HC unit separating from topping slab, i.e. delamination. 3. Splitting of the HC unit webs leaving the topping slab and the top half of the HC unit intact while the bottom of the unit collapsed.
Adana-Ceyhan Earthquake[9] (1998)	Residential and commercial buildings	Connection between the masonry infills and floor slab.	1. Total destruction of the masonry infills by large differences between the horizontal displacements of top and bottom slabs. 2. Hinges observed in columns and not in beams or slabs.
Canterbury Earthquake[10] (2010/2011)	Multi-storeyed building	Floor diaphragm and the support of the precast double T units.	Wide cracks led to the fracture of non- ductile mesh used in the topping concrete.
		Connection between the precast flat slab units and beam.	Cracks were observed in the topping at almost all of the joints and noticeable sagging of the floor diaphragm was observed.
		Rib and timber infill floors and beam connection.	One commonly observed damage characteristic was the formation of positive flexural cracks.
	Single storey parking structure	HC slab and precast beam connection.	1. Corner cracks were observed in two HC units oriented parallel to the beam support and were possibly induced by torsional rotation of the HC unit. 2. Longitudinal splitting cracks in HC units and flexural cracks at the ends of the beams elongated the floor diaphragm. This behaviour was only observed in older buildings where no bearing strip was placed at the HC support.
Kaikoura Earthquake[11,12] (2016)	Buildings	HC slab and precast beam connection.	1. Transverse cracking at ends of HC floor units or diagonal cracking at the ends of ribs within 400 mm of the supporting beam and vertical dislocation. 2. Shear failure at the ends of precast floor elements. 3. Plastic hinge damage. 4. Longitudinal cracking of HC floor units. 5. Mesh fracture in floor toppings.

and precast parts. Among the various types, the most commonly used precast slabs are hollow core (HC) and solid slabs. The HC slabs are preferred as they have zones of zero stresses in hollow portions, reduced dead load, less concrete requirement, provide better insulation and utilized as large unsupported spans. Floors are required to behave as a diaphragm and transfer the earthquake induced inertia forces to the lateral load resisting system[13]. Thus, the connections should be capable of providing this diaphragm action.

Precast concrete beams supporting the slabs are designed to resist gravity and lateral loadings. Two categories of precast beams are – the internal and external beams. When the loading is symmetrical, internal beams are used, and when the loading is unsymmetrical, external beams are preferred[14]. The type of load applied will determine the connection type that is required at the end of the beam. Normally, inverted T-beams, L-beams and rectangular beams are frequently used in the precast structures. The shapes and sections of precast beams vary based on the spans.

Shear strength of hollow core slabs

In the case of design, several studies showed interest in the behaviour of HC slabs subjected to different loading and shear strength, as it is generally not possible to provide web reinforcement in HC slabs, which makes the slabs critical in shear. An analytical study was conducted on HC slabs of 203 mm thickness subjected to concentrated load at the edge of the slab. It displayed local punching shear failure and shear-torsion near the end with a recommendation of allowing edge service load in the range of 15.6 to 16.9 kN (ref. 15). In another study, shear strength of ten slab specimens was investigated analytically and experimentally in comparison with American Concrete Institute (ACI) and BS8110 code, out of which six failed in the shear tension mode. From the tests, it was concluded that the test values were slightly above those estimated by ACI code and reduction of 10% by BS8110. Based on this, an additional reduction factor of 0.75 was preferred to define the shear capacity as a modification in Fédération internationale de la précontrainte (FIP) recommendations which would lead to a better approximation of the test results[16,17].

A recent study for deep HC slab of thickness 406 and 508 mm reported that the support width was a crucial factor to its shear strength[18]. The actual behaviour of the structure can be reproduced by full-scale testing. Four full-scale tests on two-span floor system consisting of ten slabs on flexible supports and a reference test on a single slab on rigid supports were studied numerically using ABAQUS. Studies were done in Finland, Germany and their numerical investigations revealed that the shear strength of HC slabs was reduced significantly up to 60% due to transverse stresses with shear-tension failure of slabs when the slabs were bedded on the flexible supports. The German study provided better estimates of the shear strength of HC slabs on rigid supports and the Finnish on flexible supports[19,20]. Tests on web-shear strength of HC units performed by the US and European manufacturers[21] for depths greater than 320 mm concluded that the web-shear strength can be less than the strength computed using ACI 318–05 Eq. 11-12 (ref. 22), when coupled with a critical section. With such a reduction, web shear will control the design of deep HC sections more often. Using experimental results from three specimens, a simplified general design method was proposed for calculating shear for prestressed and non-prestressed members[23], and the same was compared and concluded that the proposed general method predicted shear failure more accurately than the equations in the provision of ACI 318 (refs 24, 25).

Recently, the ACI building code[24] incorporated strut and tie approach for prestressed and reinforced concrete (RC) members with arbitrary geometry and loading configurations. The strut and tie model was proposed to estimate the ultimate load carrying capacity, identifying the critical sections for transferring in-plane loads, studying failure modes and designing prestressed precast HC slabs. Use of this model led to a reasonable and safe estimation of reinforcement requirements in the topping slabs. This approach showed good correlation with the experimental results[26,27]. ACI building code recommends using this approach for the analysis and design of HC slabs.

Connections

A single connection may consist of several load transmitting joints. A 'joint' is the action of forces (e.g. tension, shear and compression) that takes place at the interface between two or more structural elements, whereas, a 'connection' is the action of forces (tension, shear, compression) and moments (bending, torsion) through an assembly comprising of one or more interfaces[28]. Therefore, the connection design depends on both the structural elements and the joints between them. In precast concrete construction, connections form the vital part, since their detailing and design depend on the detailing and design of the adjacent element.

Seismic and non-seismic detailing of connections

The detailing of the connections depends on whether the structure should resist only the gravity load (non-seismic detailing) or gravity load plus lateral force (seismic detailing). Some of these connections are simply bearing pads or grouted joints that carry gravity loads from spanning members to their supports. However, these connections are not suitable to transmit the lateral forces. Lateral forces are transferred through connections that are designed specifically for that purpose. Lateral forces are induced during the occurrence of an earthquake, where the seismic energy has to be dissipated from the structural system as elastic deformations, inelastic deformations and damping. The energy dissipated during elastic deformation is very less. Hence, it is important for the structure to possess ductility to achieve inelastic zone and energy dissipation. Generally, structures that do not have ductility will fail when they are subjected to ground motion that deforms them beyond their elastic limit[29,30]. Ductility enables the structures that do not have adequate strength to resist strong earthquake shaking elastically, to still survive such shaking through inelastic response. Therefore, ductile detailing is required in high seismic areas[31]. In the high seismic design categories, topping for HC slabs are provided to develop the diaphragm behaviour when the in-plane lateral forces are larger[32]. In addition, joints should be placed either far from the most stressed regions or made strong enough to not reach the failure first. In the low or moderate seismic design categories, mechanical connections with sufficient strength and ductility are used in double tee slabs, which do not require the addition of cast-in-place concrete or field-placed reinforcement, whereas, untopped HC slabs can be designed using chord and shear friction reinforcement in the joints at the ends of the units. The connection should be designed such that there is smooth force flow to the overall load resisting elements of the structure.

Energy dissipation also plays a major role in seismic resistance as it describes the ductile behaviour of structures[33]. According to BS EN 1998-1:2004 (ref. 30), precast connections are classified based on their position compared to the energy dissipation regions of the structure. There are three types of connections: (i) Connections placed outside the critical regions and not affecting the energy dissipation capacity of the structure; (ii) Connections placed inside the critical regions but overdesigned in order to remain elastic in the seismic design situation; and (iii) Connections placed inside the critical regions and detailed in order to develop substantial ductility and energy dissipation capacity in the seismic design situation.

Slab–beam connection

The slab and beam should be connected properly and detailed sufficiently so that the transfer of loads occur smoothly to ensure integrity and continuity in the structure. The quantitative design parameters that influence the performance of different slab to beam connections are shown in Figure 1.

Simple support

Initially, the precast slab was considered as simply supported on the beam, e.g. parking structures. Flexible bearing pads or mortar seating pads are provided to support the concentrated end reaction and allow for end rotation of simply supported slabs. Also, the continuity at the supports was enhanced by the addition of *in situ* topping concrete and reinforcement to make the connection strong by limiting its deflection and controlling floor vibration[34]. The continuity in the HC slab was related to the span capacity and the percentage of moment redistribution from the supports that was achieved by placing the rebar over the supports or in the openings made in the cores[35,36], allowing substantial plastic elongation before fracture. When this continuity was created, a negative moment formed at the critical region that resulted in the flexural and flexure-shear failure[37]. Based on this, failure modes were studied with two specimens of 300 mm depth HC unit[38]. Specimens with *in situ* topping with mesh (HCW1) failed in a brittle manner as soon as the HC unit cracked. However, the first yield moment was 60% less than predicted, whereas, those with 12 mm diameter deformed bar of grade E500 in the topping (HCW2), showed higher shear strength in the negative moment region (Figure 2).

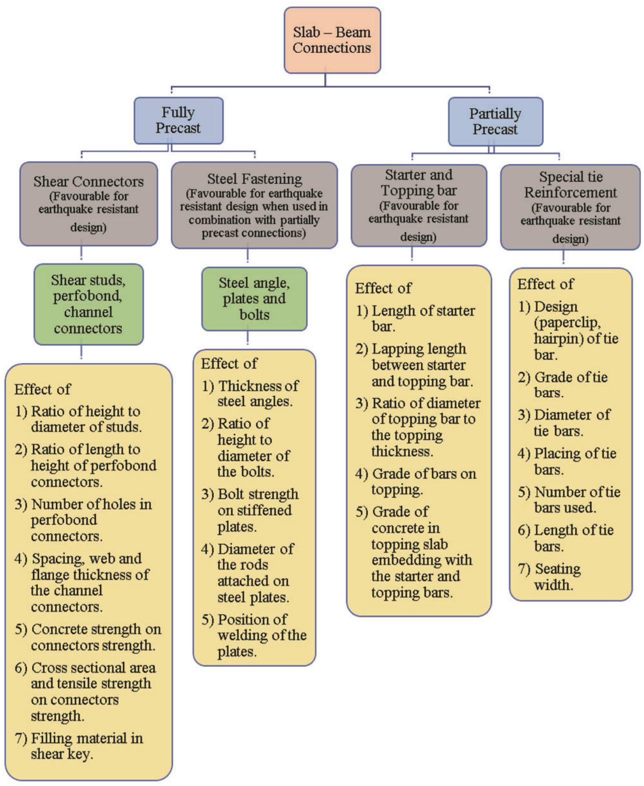

Figure 1. Quantitative design parameters influencing the performance of slab to beam connection.

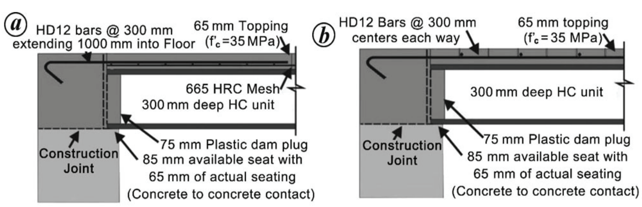

Figure 2. Connection details of test specimen[38]: ***a***, HCW1; ***b***, HCW2.

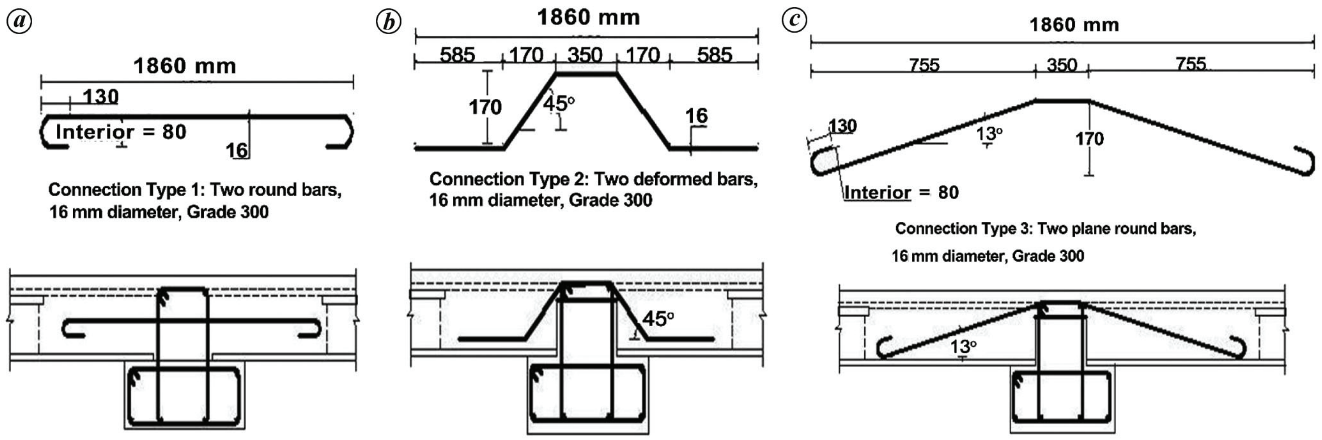

Figure 3. Special tie connection details of test specimen[40]: ***a***, Type 1, Hanger tie; ***b***, Type 2, Saddle tie with 45° bend; ***c***, Type 3, Saddle tie with 13° bend.

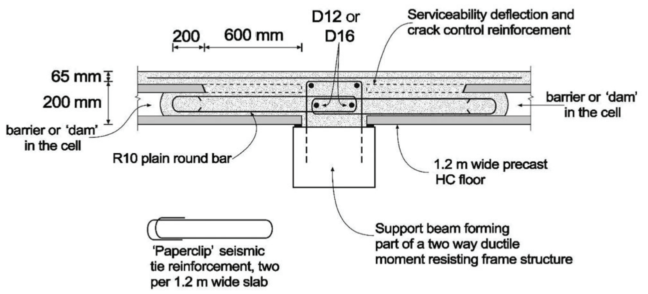

Figure 4. 'Paperclip' tie connection detail[42].

Special tie reinforcements

Special tie reinforcements are designed to provide integrity, robustness to the structure and to prevent the collapse of precast slab due to the failure of supporting beam. Tests were conducted to study the effect of seating conditions of HC units spanning between the supporting beams with 65 mm thick cast-in-place topping slab with welded steel mesh and saddle bars anchored into the infilled voids, both continuous over the support. It was concluded that the bearing lengths of 5 mm in the direction of the span was satisfactory for the support[39]. But very few tests were conducted for studying the effectiveness of special tie reinforcement in the form of hanger tie (Type 1), saddle tie with 45° bend (Type 2), and saddle tie with 13° bend (Type 3), subjected to large displacements (Figure 3)[40,41]. These connections were tested through vertical loading (Test A) and horizontal loading (Test B). The Type I tie performed better than the other types, since bond failure along the plain round bar allowed elongation and increased the energy absorption. The above mentioned study was continued[2] and concluded that the Type 2 failed as no plastic elongation was observed before fracture and therefore, it was not recommended for usage in the areas where large horizontal movements were expected.

With the use of special tie continuity reinforcement, an improved tie detail (paperclip) was provided to prevent the loss of support[42]. Paperclip was provided in the broken back voids of HC units to provide ductility and load carrying capacity. The paperclip tie reinforcement as shown in Figure 4 failed as did not had enough deformation capacity to sustain the required horizontal displacement. Herlihy[43] tested a starter bar detail and found that if the bearing was lost, the floor collapse could not be prevented. The starter bar detail was inadequate as it did not act as a ductile tie between the HC slab and beam.

Based on the special tie reinforcement, a preliminary investigation was carriedout on the effect of beam elongation (once plastic hinges form in a beam and the beam undergoes large inelastic rotations, the beam then significantly grows in length) on the required seating lengths for the HC units. A full-scale super assemblage test (Figure 5 *a*) was carried with 12 mm high ductility (HD) starter bar seated on mortar pad with 50 mm seating width[44,45], where the voids were closed using plastic plug to obtain the overall seismic performance of the building (Figure 5 *b*). It was loaded in three phases of reverse cyclic loading (Figure 6 *a*). The response of the specimens were recorded as load-displacement hysteresis loops were stable and dissipated a reasonable amount of energy (Figure 6 *b*). But, there was no evidence to indicate the overall poor performance of the system as a whole and particularly the very poor performance of the precast floor. It was observed that though the floor failed, the perimeter frames remained relatively undamaged (Figure 6 *c*) which indicated that extra attention might be required for the seating details so that the flooring system performed at a level that was not inferior to that of the structural frame. Hence, it was recommended to provide 75 mm seating width, to place the HC unit on low friction bearing strip and to replace the plastic plugs with compressible backing material.

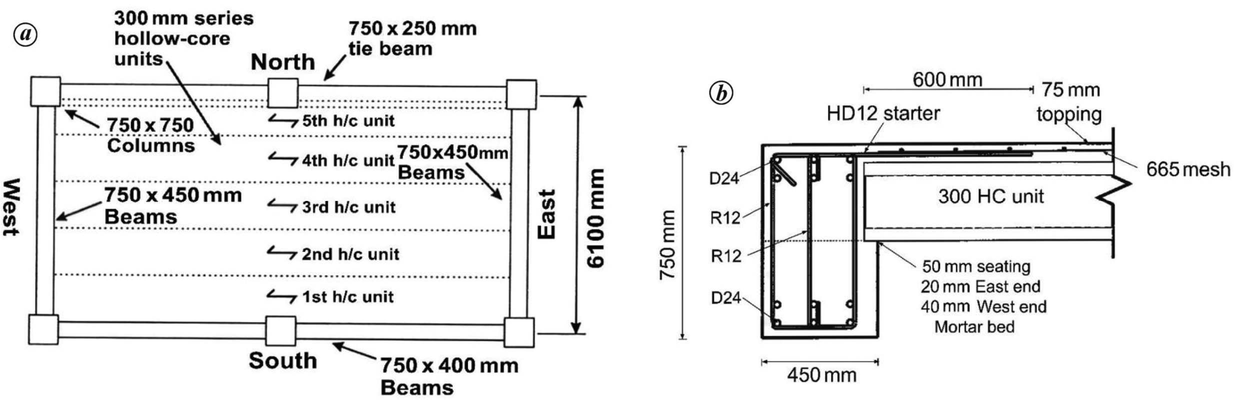

Figure 5. Details of super-assemblage[45]: ***a***, Plan; ***b***, Connection detail.

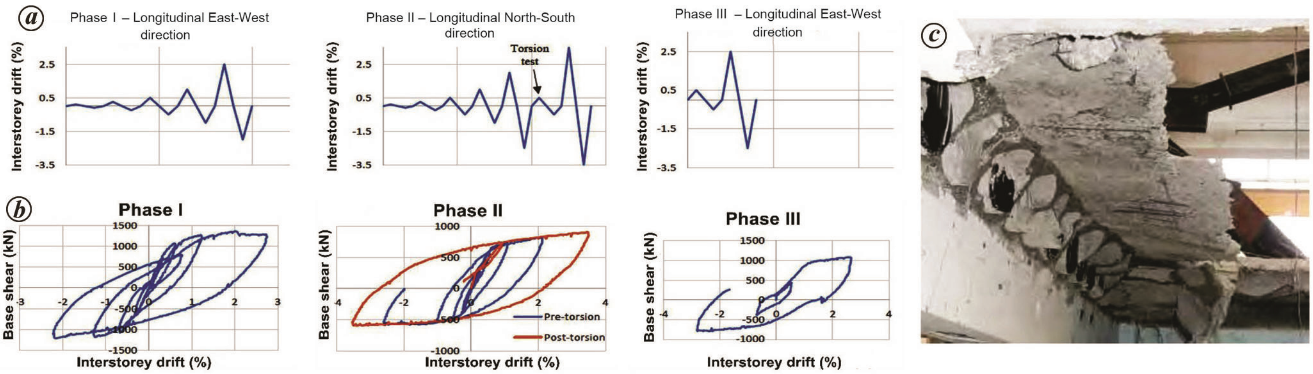

Figure 6. Performance of the super-assemblage test[45]: ***a***, Loading protocol; ***b***, Hysteresis loop; ***c***, Failure mode showing starters and end of the HC units remain attached to the east beam.

Table 2. Properties of HC specimens (HCS)

Specimen	Depth of HCS (mm)	Bearing material	Seating width (mm)	Continuity bar with spacing	Topping reinforcement
HC1	300	Mortar pad – 50 mm	50	HD12@300 mm	HRC665 mesh
HC2 (Compressible backing Board)	300	Low friction bearing strip – 50 mm	70		HD10
HC3 (Paper clip)	300	Low friction bearing strip – 50 mm	70		Hurricane ductile mesh
HC4	200	Mortar pad – 50 mm	50		HRC665 mesh

Due to this kind of poor performance, Technical Advisor Group (TAG) recommended a new HC connection detail using compressible material as backing board instead of plastic plug. This was incorporated and tested in a sub-assembly set-up by varying the details of connections between the HC unit, topping and support beam (Table 2). It was suggested that the combination of compressible backing board and the low friction bearing strip in the HC unit performed better than the combination of paper-clip and low friction bearing strip[46] (Figure 7).

Based on the findings from an earlier research[45], major shortcomings in the construction practice were addressed, that included seating HC units on a low-friction bearing strip with a compressible backing material replacing the plastic plug, and a ductile reinforcing mesh replacing the standard cold-drawn wire reinforcing mesh to increase the performance of the floor diaphragm[47,48]. It was concluded that the low friction bearing strip allowed the HC unit to slide on the supporting beam and the compressible material reduced the compression force applied to the beam under the negative moments and restricted the concrete from entering the cores of the unit. The conventional reinforcing in the form of HD10's bars at 300 mm centre to centre on each way should be used instead of

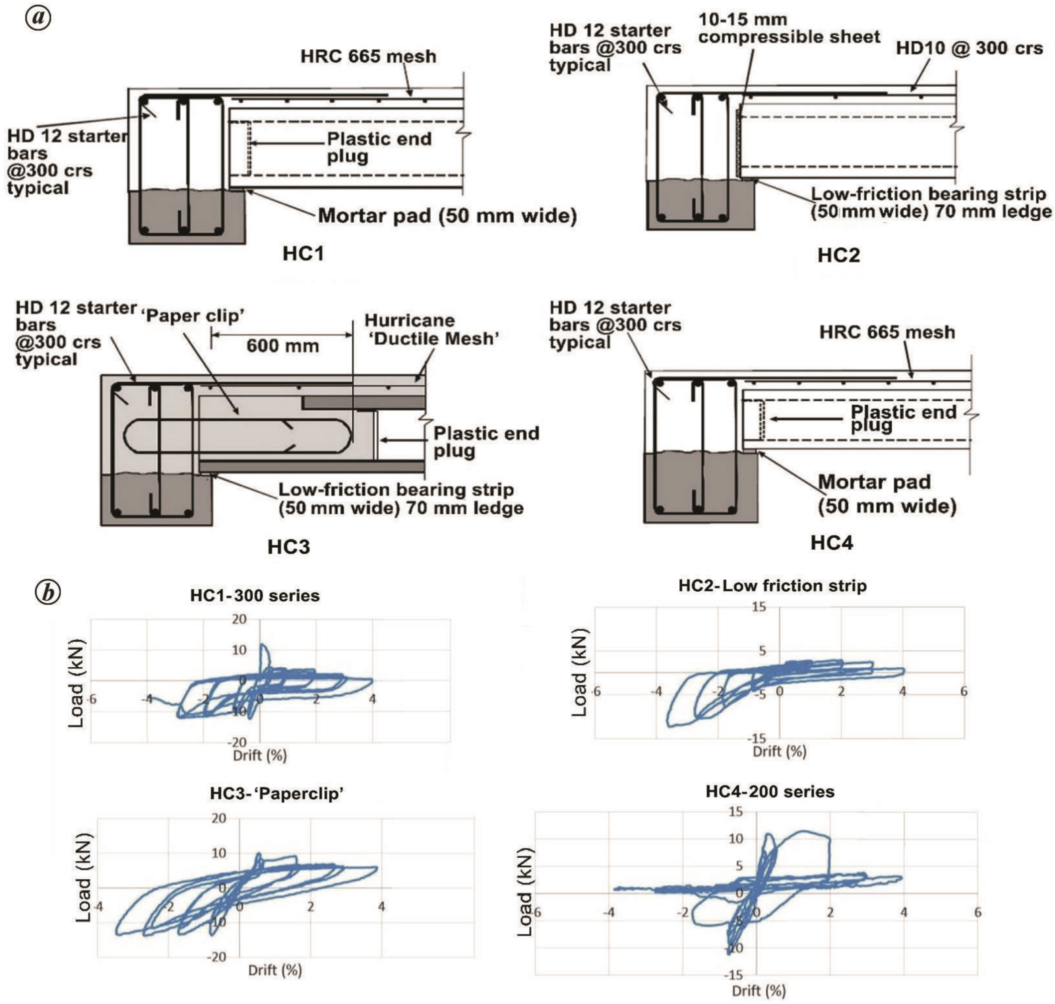

Figure 7. Details of sub-assemblage[46]: ***a***, Connection details; ***b***, Hysteresis performance of the tested connection.

the reinforcing mesh, ductile, or cold-drawn within the concrete topping slabs to increase the performance of the floor diaphragm.

The second stage of the present work investigated an improved detail using a 750 mm wide timber infill between the perimeter beam and HC slab on a low friction bearing strip and adopting a thin compressible material as a backing board for the supporting beam. During Phase I loading, the hysteresis loops showed minor necking arising from the opening and closing of the cracks in the plastic hinge zone. In the Phase II loading, the loops displayed no necking with insignificant strength deterioration, and in Phase III loading, it exhibited the same behaviour as in Phase I loading. It was concluded that: (i) HC units should not be seated in the potential plastic hinge zones of their supporting beams. Instead, the infill sections should be placed over the highly deformed zones with extra reinforcement placed within the beam, to force the hinge zone to occur beneath the infill section only and not under the brittle HC units. (ii) When an infill slab is placed adjacent to the perimeter beam, starter bars should be extended to cross the interface between the infill slab and terminate midway in the adjacent floor unit to increase interface's ductility, and prevent bar's fracture at the interface. (iii) The seat of the HC unit in the supporting beam should be reinforced to tie the seat back into the beam and prevent large sections of concrete peeling off. (iv) The compressible material placed across the end of the unit is only required when the neutral axis of the beam is above the soffit of the HC. Based on the above mentioned factors, revisions were made in the amendment to the New Zealand Concrete Standard (NZS)[49,50] as two 'acceptable solutions'. The first solution was as recommended from the previous findings of Lindsay[47], and the second solution[51] incorporated round hooked bars closer to the bottom of the unit provided at two of the four hollow cores in the slab (Figure 8).

Based on these factors, revisions were made in the amendment to the New Zealand Concrete Standard (NZS)[49,50], as two 'acceptable solutions'. The first solution was as recommended from the previous findings of Lindsay[47], and the second solution[51] incorporated round

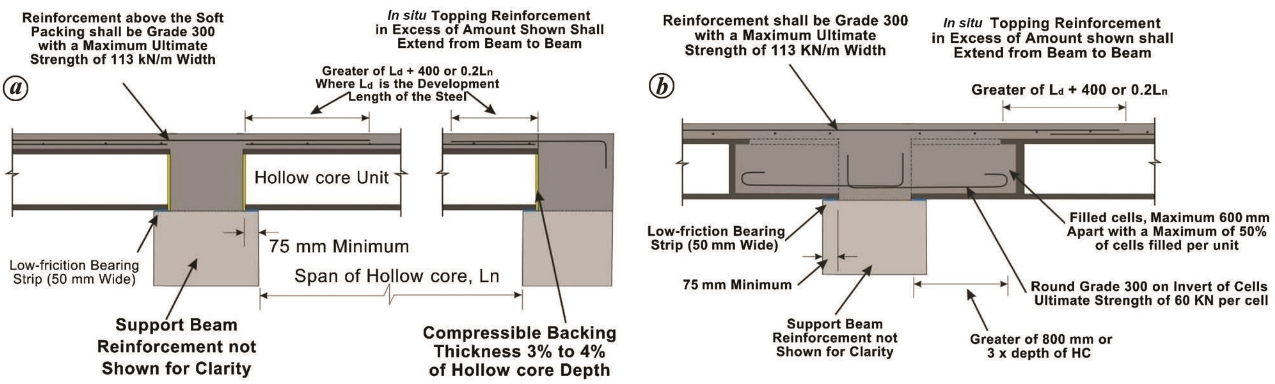

Figure 8. 'Acceptable solution'[51] proposed by New Zealand concrete standards: ***a***, First solution; ***b***, Second solution.

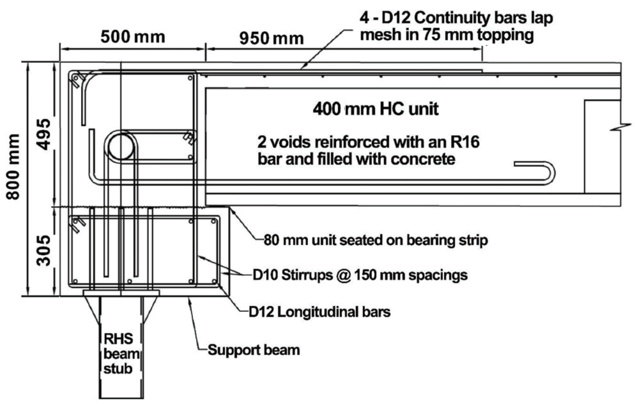

Figure 9. Recommended connection detail for 400 HC from NZS 3101:2006 (ref. 13).

hooked bars closer to the bottom of the unit provided at two of the four hollow cores in the slab (Figure 8).

The second 'acceptable solution' was tested in a full-scale super-assemblage set up using Lindsay's[47] loading protocol, where 50 mm seating length was provided in the east and 75 mm in the west direction. The connection performed well up to ±5% inter-storey drifts with minor damage in the floor unit[52]. No difference was observed in the behaviour between 50 and 75 mm seating ends, but considering the construction tolerances, 75 mm seating width was provided.

Most commonly, 200 mm, 300 mm HC units were being used and more recently 400 mm deep HC units have come into use. During the Canterbury (UK) earthquake in 2010, precast concrete floor units made with 400 HC units did not suffer any significant damage[10]. Hence, the seismic behaviour for 400 HC support connection as recommended by NZS 3101:2006 (ref. 50) was studied to ascertain the gravity and seismic load carrying capacity (Figure 9).

Five units – HC1, HC2, HC3, HC4 and HC5 were subjected to simulated seismic loading in two stages[13,53] (Table 3). In Stage 1, HC1 resulted in no collapse at a lateral drift of 4%, with minor spalling at the support ledge and only a single crack formation due to the high concrete strength topping, and no shear or flexural damage was noted. In Stage 2, the remaining units (HC2, HC3, HC4 and HC5) sustained a lateral drift in excess of ±4.5%, with elongation up to 60 mm and no significant impact on the performance of support connection due to different topping strength was observed. The typical buildings using 400 HC units ensured that deformation was confined to the interface between the HC unit to beam and the vertical support of the floor sustained through large lateral drifts. With the recommended seating detail in accordance of NZS 3101:2006 (span/180 ≥ 50 mm for solid slab and span/180 ≥ 75 mm for other members)[50], our study concluded that the ductile performance of the structure improved if the width of the seating was increased to about 125 mm.

Rib and timber infill floors are susceptible to undesirable failure modes when subjected to earthquake deformations. Damages consistent with these failure modes were observed during the Canterbury earthquakes. Structural Engineering Society of New Zealand (SESOC)[54] provided rib seating details that enabled the rib to slide and rotate without sustaining damage. An experimental programme[55] was undertaken to investigate the seismic performance of the recommended support connection details for rib and timber floors with three test specimens incorporating 175 mm deep ribs with a change in the stirrup detail (closed-loop, open leg and helix). The test concluded that the connection detail prevented the formation of positive moment cracks and enabled the rib to slide without becoming entrapped in the support connection to drifts in excess of ±4.5%. Due to the lack of rib damage, no significant influence of the rib stirrup configuration was observed during the tests.

In 400 HC units, rib and timber infill[56], and the connection between precast units provided by the *in situ* concrete topping gave a satisfactory performance for the recommended details. For *in situ* concrete topping, out of cold-drawn mesh (CM-1), ductile mesh (DM-1), and deformed reinforcement (D10-1), DM-1 was recommended as it sustained four displacement cycles before the

Table 3. Loading variation for 400 HC units

Specimen		Loading	Seating width (mm)	Topping strength (MPa)	Starter bar
HC1	(Stage 1)	Seismic to ±4%; no vertical load	80	43	D12
HC2	(Stage 2)	Seismic to ±4.5%; 3 tonne gravity load	80	25	D12
HC3	(Stage 2)	Seismic to ±4.5% followed by vertical load till failure	80	36	D12
HC4	(Stage 2)	Seismic to ±2% followed by vertical load till failure	150	20	D12
HC5	(Stage 2)	Seismic to ±1% followed by vertical load till failure	80	30	D16

reinforcement ruptured at a crack width of 18 mm, whereas D10-1 was able to sustain two displacement cycles before the reinforcement ruptured at a crack width of 13 mm.

To facilitate good practice in the design of structural connections between HC slab and beam, Fédération Internationale du Béton (FIB) 43 (ref. 57) guide to good practice recommended various continuous connections, like hairpin connection; loop connection; connection using continuous tie bars longitudinally and in transverse direction in cores opened by a slot at top; connection by anchorage in the longitudinal joint, and shear key connection.

Steel angles, plates and bolts

The connections in a structure using steel angles, plates, and bolts are designed to fulfil serviceability, level of safety, durability, and to withstand the stresses induced in them. When the new improved detail[47,51] was recommended for the revision of NZS 3101:1995 (ref. 58), some researchers retrofitted the seating support details using angle seat (150 × 150 × 12 mm) for the connection of the HC floor units to reinforced concrete (RC) frame supporting beams. This retrofit connection showed an improvement in the positive moment capacity but collapsed in the negative moment capacity of the connection. The angle seat restricted the rotation of the HC unit and thus the shear crack propagated to the end of the angle seat and resulted in the collapse of the HC unit. Hence, it was recommended to provide clearance between the soffit of the HC unit and seat angle[59]. This recommendation was incorporated to enhance the structural integrity between the HC seating and perimeter beam[60]. The steel connection detail tested earlier[45–47,52] was retrofitted with steel angle seat (R1), vertical saw cut (R2), and bolt connection (R3). A clearance was provided between the steel seat and unit soffit to avoid clamping effect in R1 retrofit. R1 primarily prevented the unit from snapping under seismic intensity levels and prevented splitting at the web of the HC unit. This R1 retrofit option provided the development of alternative connection detail for new construction in contrast to R2 and R3 connections, which had practical complications.

A conceptual retrofit strategy for HC connections with varying seating widths was proposed, providing an experimental validation through quasi-static cyclic testing. Though full-scale testing predicts the actual behaviour of the structures which is critical for qualifying connections in earthquake resistant structures, however, due to restrictions in full-scale testing facilities, half-scale testing was adopted. Model studies (scaled models) have their own pros such as reduced cost, easily manufacturable specimen with hassle-free handling, reduction in applied loadings and a corresponding reduction of test apparatus size. The success of a model study depends upon careful consideration of similitude requirements for material properties and loading conditions[61]. Hence, for the present study, the deformations were imposed on the half span of HC unit by hydraulic actuators and it aimed to represent *in situ* frame behaviour imposed on a floor system during a seismic event. Experimental tests[62] (HC1, HC2, HC3 and HC4) were carried out in which HC1, HC2, HC3 were taken as benchmark connection showing variations in HC seating width (35, 50 and 75 mm) as specified in NZS 3101:1995 (ref. 58). Under seismic loading, all the three connections (HC1, HC2 and HC3) showed damages in the form of rupture at the interface between the end of HC unit and seating beam, rupture of the starter bars at this cracked interface, and loss of vertical support of the HC unit. To rectify these failures, two retrofit measures were implemented on the HC4 specimen which, prior to retrofit, was identical to the HC3 specimen details. The first retrofit was a steel rectangular hollow section (100 × 50 × 6 mm RHS), which was provided at the soffit of HC unit for additional seating and to avoid spalling of the seating ledge. The second retrofit had holes drilled in perforation pattern behind the HC unit to weaken the interface thereby reducing the flexural strength of the interface and the forces imposed on the HC unit in order to avoid flexure-shear failure. Thus our study concluded that the first retrofit (RHS) significantly reduced the extent of spalling of the seating ledge and maintained 95 mm of elongation without the fracture of reinforcement and delamination of topping, which was higher than that demanded by a severe seismic event. The second retrofit (holes drilled) reduced 30% of peak positive flexural strength, which was less than expected. Therefore, the present study suggested that this reduction would have increased if the perforation penetrated to the full depth of the unit.

A double tee flanged roof element was first tested under monotonic and later under cyclic loading. It was supported on two transverse beams, which were fixed to the test floor. The four bearings were provided with low-friction teflon pads but only one was equipped with the connection devices subjected in the longitudinal direction to the loading. The other bearings were provided with sliding lines to prevent transverse movements. A couple of steel angles were fixed to the beam with fasteners of diameter 16/25 mm and the web of the double tee roof element with M16 dowel was tightened through the washers against the web[63]. A force of 660 kN in compression and 460 kN in tension was applied to the web of the specimen by means of hydraulic jack on the connection. The test results showed a non-symmetric behaviour. This was due to the testing arrangement which showed some uneven effects for large displacement. The eccentricity of the applied force with respect to the connection induced a longitudinal moment that pushed down one side and lifted up the other side of the supporting element. The uplifting force caused damages to one sliding line which led to a friction/interlock reaction. The connection sustained local damage and deformation on the steel devices. Therefore, it was recommended to modify the testing arrangement to avoid such failures.

FIB 43 (ref. 57), recommended connections using steel angles, steel plates, and steel corbels. These are made fire resistant, either by encasing them into concrete/mortar (containing an expansive agent) or by an adequate fire insulation. These connections are mainly used in confined and heavily reinforced areas where the joint length is to be minimized, and immediate structural stability is required. A small gap is provided around the welded plates, to avoid damage to the concrete due to the thermal expansion of the steel element during welding.

A steel plate was adopted for a new arch shaped ductile connection (ASDC), suitable for both new structure and as a retrofit measure, and was investigated using four different configurations such as (i) traditional hinge connection, (ii) out of plane with ASDC, (iii) in plane with ASDC, (iv) in plane with ASDC and traditional hinge connection for the seismic performance[64]. For retrofit connection, rectangular plates were attached using chemical (or) mechanical fasteners and for the new connection, rectangular plates were directly connected to the structural elements by means of anchor channels. Out of the four configurations, out-of-plane deformation had symmetric hysteretic behaviour and higher displacement capacity, both experimentally and numerically using ABAQUS (Figure 10). The failure of the connection was associated with fracture of the steel rods due to excessive bending deformations at the rod edges. A possible improvement of the device performance was obtained when the rods were placed inside holes in the plates and fillet welding was done from behind to avoid bending deformation. In-plane and out-of-plane ASDC experimentally showed good performance in terms of strength, ductility, and energy dissipation. Finally, in-plane ASDC with hinge connection provided in the roof element to dissipate energy, reduced the column top displacements.

Shear connectors

A composite construction was efficiently and effectively used because of the increase in both the moduli of rupture and stiffness at little cost of the special shear connectors. Shear connectors are used to distribute large horizontal inertial forces in the slab to the main lateral load resisting elements of the structure[65]. The advantages of the headed stud connectors are having good anchorage in the concrete, easy arrangement of reinforcement through the slab, simple production of the large-scale sized headed stud connectors, and practical utility in the steel deck slabs.

A full-scale push-out experimental test and finite element analysis was conducted on a shear stud connector to evaluate shear capacity and failure modes of the studs by varying the sizes, diameter, height, tensile strength of studs, concrete strength and spacing of studs. This showed that there were several parameters to influence the studs[66,67]. A comparison between the experimental and analytical results using DIANA commercial software showed a satisfactory correlation[68,69]. While evaluating the structural performance of the shear connection of the stud, its fatigue behaviour was also investigated by accounting for bed height and filling material properties

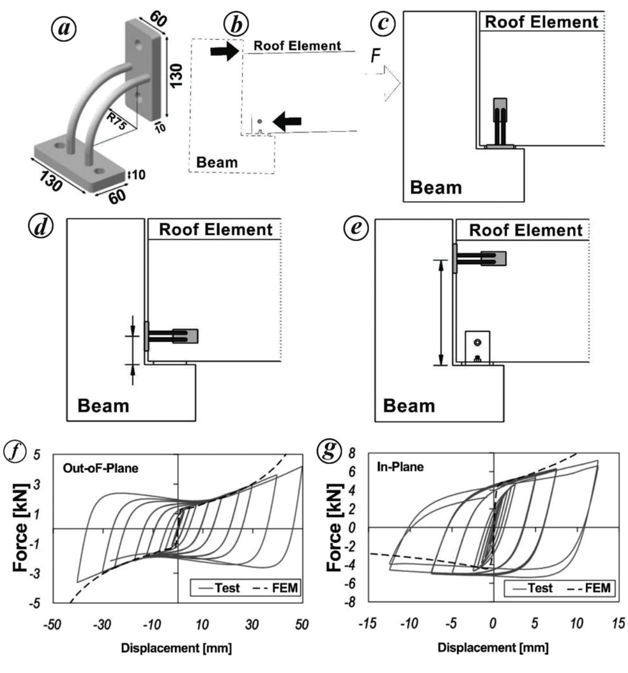

Figure 10. Ductile connection and cyclic test results in terms of load versus displacement (Experimental and Analytical)[64]: ***a***, ASDC details; ***b***, Traditional hinge connection; ***c***, Out-of-plane; ***d***, In-plane; ***e***, In-plane with ASDC and traditional hinge connection; ***f***, Out-of-plane cyclic test result; ***g***, In-plane cyclic test result.

which were conducted on a full-depth precast slab with a composite beam. It was concluded that due to lack of space on the top flange of the steel beam, studs with 25 mm diameter would create safety issues for the labourers[70–74]. A new type of semi-rigid composite connection with precast HC slabs was studied to calculate the moment-rotation capacity of the connection using eight full-scale tests of composite joints and 3D finite element model. Thus the full-scale study concluded that the elongation and ductility of the longitudinal bars played an important role in the moment-rotation capacity of the connections and the finite element model (FEM) study showed that the bottom flange of the steel beam should be thick enough to prevent the yielding of the bottom flange to achieve the high moment-rotation capacity of the connections[75,76]. The strength of double shear studs in favourable position (where the compressive zone in front of the stud in its load bearing direction is larger than the compressive zone behind the stud), and in staggered positions with various concrete strengths was investigated by comparing with the capacity of single shear stud[77]. It was concluded that the shear connector resistance of the double shear studs placed in the favourable position was 94% of the strength of single shear stud on an average, when the transverse spacing between the studs was 200 mm.

The studs were substituted with other shear connectors like perfobond ribs. A study[78] indicated that one meter length of a perfobond connector was comparable to twenty four 19 mm diameter studs disposed in three lines or eighteen 22 mm diameter studs disposed in two lines, because the concrete flow through the rib holes formed dowels that provided resistance in both the vertical and horizontal directions. Other tests results indicated that the shear capacity of the perfobond rib increased with the number of rib holes as long as the hole spacing was at least 2.25 times the diameter of the holes. The transverse reinforcement ratio in the range of 0.27% to 0.36% showed an increase of 16% in the shear capacity, and that of 0.11% to 0.20% showed an increase of 10% in the ultimate capacity[79,80]. There were other connectors like crestbond (CR) connector, where one CR connector of 560 mm length had a load capacity that was equivalent to four 19 mm studs, for concretes in the same range of strength. For an increase of 81% in concrete strength, the load-bearing capacity of the CR connector increases by 35% (ref. 81). Two series of push-out tests for a T-rib connector was performed to investigate the slip capacity, failure modes and stress distribution by using a concrete of class C25/30 in the first series and class C50/60 for the second series[82–84]. It was concluded that the resistance and stiffness of this type of connector were generally higher than that of the perfobond rib connectors.

Eight push-out tests were performed in accordance to Eurocode 4 with a variation in the number of holes in the perfobond plate connected side by side and they evaluated the characteristics that affected the connectors' shear resistance and ductility. It was concluded that at least four 19 mm stud connectors (with a resistance of 74 kN each according to Eurocode 4) can be replaced by a single one-hole perfobond connector[85,86]. The load-carrying capacity of an oscillating perfobond strip connector was compared to that of the headed studs and T-shape connector[87], and was found to be higher. However, in the case of ordinary strength and normal-weight concrete, the strip connector performance showed a fast drop of load capacity after the peak. This behaviour was not observed when light-weight concrete, concrete with fibres or a high strength concrete was used. In these cases, the oscillating perfobond strip connectors performed well.

A full-scale push-out test was carried out on four composite steel and concrete T-beams with channel connectors and the specimens were made of plain, reinforced and fibre reinforced concrete. It was observed that the shear strength showed 23% increase under monotonic loading when compared to reverse cyclic loading. Out of the three different concretes, fibre reinforced concrete slightly affected the load-displacement behaviour and shear strength of the specimen when polypropylene fibres were used[69]. A three series of push-out test[88,89] was performed for six specimens of solid concrete slabs and another six specimens of concrete slabs incorporating wide-rib profiled metal deck with ribs parallel to the beam. The results showed that the load-carrying capacity of the channel connector increased linearly with the increase in the channel length, and for a given length of channel, the concrete strength dictated the failure mode. Specimens with the solid slabs carried higher loads compared to those with the metal deck slabs and the increase was approximately 33% for specimens with 150 mm long channel connectors but only 12% for those with 50 mm long channel connectors.

Shear keys increase the shear strength of the interface between the precast slab and beam, because they provide an additional strength due to the shear strength of the concrete with the strength provided by transverse steel and friction in the contact surface[90]. Based on this, the slab and beam were connected using shear key[91] with various diameters of steel connector, like bars bent in a hoop, and concluded that the strength of slab–beam connection increased to 250%, when the shear key with the inclusion of fibres along with 10 mm diameter steel connector and concrete cast in the pocket with 50 MPa compressive strength was used. This contributed to the strength and dissipated energy until the failure of the connection.

Numerous research studies were performed on the use of high friction grip bolts (HFGB), and it was concluded that within the bolt holes, a sudden slip of the bolts occur once the shear load exceeds the friction resistance in the precast solid slab–steel beam interface[92–94]. To overcome the disadvantage of HFGB, a removable friction based shear connector (FBSC) and locking nut shear connector

(LNSC) were studied and both showed high shear resistance and stiffness of 161 kN and 104 kN/mm respectively, for FBSC, and 171 kN and 100 kN/mm respectively, for LNSC using M16 bolts[95,96]. Provision of locking steel and threaded bolt in the shear pocket was found beneficial in terms of strength, ductility and slip behaviour[97,98].

A summary of various failure modes and relative rating of connections is shown in (Table 4). Among the different slab to beam connections, usage of shear connectors would be suitable for earthquake resistant structures when fully precast slab–steel beam connection is considered. Similarly, special tie reinforcement, topping bar, starter bars with topping slab would be favourable for earthquake resistant structures when partially precast slab–beam connection is adopted. However, mechanical connectors like steel angle, plates and bolts are not recommended for high seismic design categories, but they can be used in combination with partially precast connections for earthquake resistant structures.

Conclusions

Precast members have been used for decades in the construction of bridges and flyovers. However, their poor performance under seismic loading limits their usage in commercial and high-rise residential buildings. This is due to the scanty knowledge on their design and force transfer mechanism of the connections. The monolithic performance in precast structures can be achieved by proper detailing of the connection between slab and beam element. The following points can be summarized from the past research studies on the seismic behaviour of slab beam joint.

(1) The floors should be properly tied to the beam and topping slab to ensure diaphragm action for lateral and gravity load transfer.
(2) Failures observed during induced earthquakes were: beam elongation in plastic hinges, relative rotation at the support connection, positive and negative moment failure, shear failure, incompatible displacement between frame and the floor, and torsional failure.
(3) The performance of the connections can be improved by using low friction bearing strip with a compressible backing board on the seat of HC slab to reduce concentrated end reaction and rotation of the simply supported slabs.
(4) A cast-*in situ* RC structural topping with conventional reinforcing bars should be provided to resist moving and concentrated load in partially precast construction, whereas, in fully precast construction, the untopped precast slab connected using steel ties with the grout, ensures structural integrity and diaphragm action.
(5) Minimum required support width to prevent the unit from being spalled off its ledge should be 75 mm or span/180. Also, the precast slabs should not be placed in plastic hinge zones of the supporting beam.
(6) Redundancy can be increased by introducing plain round bars at the base of each alternate void after breaking out the flange and filling it with concrete.
(7) In the solid precast slab, loop connection with transverse reinforcement enables the transfer of bending moment, shear force and tensile force which enhances the continuity and ductility of the system.
(8) The mechanical connectors, though they provide extended seating length, could not prevent complete collapse. Hence, they must be strongly anchored and embedded in concrete to obtain desired strength and stiffness. It is recommended to be used in combination with partially precast connections for earthquake resistant structures.
(9) In composite connections, connectors like perfobond ribs, T-rib, oscillating perfobond, T-connectors and channel connectors performed better than the shear studs in terms of ductility, stiffness and slip capacity.

Scope for future study

From the lessons learned from the past earthquakes and research done in the recent years, certain progress concerning the seismic performance of precast slab–beam connection has been achieved. Further research is still needed to understand the behaviour of connections between the slab and beam. Following recommendations for future studies are suggested for both the diaphragm design and support details for precast units.

(1) Elaborate experimental research is vital as the experiments cannot be repeated which make it difficult to understand the behaviour of the connections on an average.
(2) Analytical investigations and finite element modelling for HC slab–beam connections should be conducted to predict the failures before concerning the tests in sub-assemblage or super-assemblage set up.
(3) Mechanical connections should be studied more extensively to make them suitable for connections in the high seismic design categories.
(4) In Kaikoura earthquake, cracks were formed from the interface which propagated vertically and not towards seating. Such cracks need investigation, as this new failure mechanism was not observed previously.
(5) More experimental investigations are required to ascertain the response of the shear connectors and shear keys by varying the types of concrete used as infill, grades of concrete, spacing of connectors,

Table 4. Various failure modes and relative rating of connections

Type of connection	Details of connection	Type of loading and test	Failure modes	Cause of failure	Relative rating	Recommendation
HC slab – beam	Special tie reinforcement topping bar and starter bar.	Quasi static cyclic and relative rotation with pull off test.	Positive flexure	Tension at the soffit of the HC unit induces positive moment.	1. More favourable to design. 2. Care should be given to moment transfer of connection. 3. They are frequently used on load- bearing structures.	These details of connections are recommended for earthquake resistant structures.
			Negative flexure	Tension that occur on the topping induces negative moment. Shear failure can occur.		
			Incompatible displacements	The first HC unit undergoes curvature when cast adjacent to the beam. This leads to loss of the lower half of the first unit.		
			Over-reinforced cells of the units	Concrete filled cells that have Hairpins/Paper-clips tend to 'over-reinforce' that leads to failure and collapse starts at the infill ends.		
			Torsion	Cracking caused in the unreinforced section of the HC unit.		
			Loss of support and edge spalling	Inadequate seating width.		
Solid slab – beam and HC slab – beam	Steel seat angle, ASDC, steel plates and bolts	Quasi static cyclic and relative rotation with pull off test.	Clamping effect	Inadequate clearance between slab and beam.	1. Least favourable to design. 2. Problem in rectifying the tolerance limit.	These details of connections are not recommended for high seismic design categories. When these are used in combination with special tie reinforcements, they may be favourable for earthquake resistant structures.
			Snapping action	Over-reinforced in the connection region.		
			Rupture in the welded joint	No gaps are provided between welded plates.		
			Fracture of steel rods	Excessive bending deformation at the rod edges.		
Precast solid, flat, HC slab with steel beam	Shear connectors (shear studs, perfobond ribs, channel, shear key and bolts)	Push out test.	Interfacial slip	Improper friction forces in the composite interface.	1. Favourable to design. 2. Corrosion should be of concern and site welding should be avoided.	These details of connections are recommended for earthquake resistant structures when precast slab – steel beam connection is adopted.
			Crushing of concrete at the connector's root	Poor welding and less anchorage in concrete.		
			Rupture of the channel near the fillet with the web	Deformation of channel connector.		
			Concrete crushing-splitting type	High concentration of stresses within smaller area of connector.		
			Longitudinal splitting of slab	Low ductility and less stiffness of perfobond ribs.		
			Bearing failure	Excessive bearing stress under the nut face and bolt head.		
			Diagonal tension cracks	Small-dimensioned joints.		

length, thickness and materials of the connectors to validate the shear strength and to have a better understanding on the load-slip behaviour.

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