Exploring the feasibility of a floor system detached from seismic beams in moment resisting frame buildings

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ABSTRACT: In current practice, floor slab and beams in the perimeter seismic frames are monolithically constructed and rigidly connected to each other through starter bars. This rigid connection ensures that the shear friction between the floor and the seismic beam transfers the inertial force resulting from the response acceleration of the floor mass and any superimposed dead loads to the moment resisting seismic frame. But, this rigid connection between the floor and perimeter seismic beams leads to several complications such as: (i) possibility of stronger beam (than columns) because of the slab contribution on the negative moment capacity of seismic beams; (ii) possibility of (unidirectional) plastic hinge forming away from column face; and (iii) the floor-beam compatibility requirement leading to severe damage in the slab as the seismic beams deflect in double curvature and grow in length due to elongation of the plastic hinges at the same time. In a quest to avoid these complications, this paper investigates the feasibility of a floor slab that is detached completely from the perimeter seismic beams. In this system, the slab is rigidly connected to the intermediate beams, which are designed to transfer the inertial force to the columns through shear friction and/or torsional resistance. This idea is conceptually discussed and its validity is scrutinized in the paper.

1 INTRODUCTION

Floor slabs in buildings act as diaphragms that interconnect the components of seismic frames, gravity frames and shear walls at that level. In current practice, slabs are rigidly connected to and monolithically constructed with the lateral load resisting system in a building. The intention behind this is to transfer the inertial force from the seismic response of the mass on the floor (comprising the dead and live loads) to the lateral load resisting system through the shear friction between the floor slab and the seismic beam or shear wall along the direction of the earthquake shaking. In New Zealand, some problems related to the slab-frame connection in reinforced concrete buildings with precast floor have been identified lately (Matthews 2004). The first problem relates to the compatibility requirement between the vertical deflection profile of the ductile beams in the seismic frame parallel to the precast flooring units and the deflection of the monolithically constructed slab. The deformational incompatibility between the seismic beam and the slab results in severe damage to the slab, especially in the region close to the beams. In addition, the seismic beams elongate during cyclic inelastic response (Peng et al. 2007), thereby inducing tension on the slab. This results in significant diagonal cracks in the slab near the beams (Lindsay 2004; Macpherson 2005).

To reduce the impact of this compatibility requirement, the use of discrete shear links has also been explored in NZ. In this concept, the slab and seismic beams are connected through discrete shear links placed at some points through the span of the seismic beam. While efficiently transferring the inertial force, this would also free the slab from having to exactly follow the deformation pattern of the beam. Instead, the slab needs to deflect together with the beam at the points of connection (i.e. shear links) only. Even this requirement could be eased if shear links are provided at the locations of zero vertical deflection of the beam. Although there will definitely be at least one point in the beam deflected in double curvature where the vertical deflection is zero, this point varies from structure to structure and also within the same structure depending on the drift level. Hence, a general solution to completely avoid the compatibility requirement cannot be achieved through this approach.

One of the other problems related to the monolithic slab-beam construction is the increase in strength of the seismic beams due to their rigid connection with the slab. Capacity design intends to achieve a strong-column weak-beam strength hierarchy. To ensure this, the moment at the joint corresponding to the overstrength of the beams is required to be less than the joint moment corresponding to the design strength of the column. Any increase in beam strength from the strength assumed in the design may result in an alteration of the strength hierarchy, which can have dire consequences. Although the enhancement of beam strength due to slab participation has been investigated extensively (Durrani and Wight 1987; Cheung et al. 1991; Pantazopoulou and French 2001), the exact extent of increase in beam strength due to its rigid connection with the slab has not yet been quantitatively ascertained. The ACI building code (ACI 318:2005) considers the slab contribution by treating the seismic beam as a T beam including an equivalent slab width. For the T-beam, the total flange width is not allowed to exceed one-quarter of the beam span and the flange projection in each side of the beam is taken as the lesser of eight times the slab thickness and one-half the clear distance to the next beam web.

The NZ Concrete Structures Standard (NZS 3101:2006) provides different methods for incorporating slab contribution in calculating the nominal strength and overstrength of the seismic beams. For a beam monolithically constructed with the slab, the reinforcing bars inside an equivalent slab width in each side of the beam are taken into account while calculating the strengths. In order to calculate negative moment capacity of the beam, the equivalent slab width recommended in each side of the beam should be the lesser of (i) one-eighth of the beam span (applicable if the slab exists only on one side of the beam); (ii) span of the slab in the transverse direction to the beam multiplied by the ratio $h_{b1}/(h_{b1}+h_{b2})$ where h_{b1} and h_{b2} are respectively the depths of the beam under consideration and the adjacent beam supporting the other end of the slab; (iii) eight times slab thickness; and (iv) the beam depth. On the other hand, calculation of beam overstrength includes, in each side of the beam, an equivalent slab width which is lesser of: (i) three times the total beam depth; and (ii) slab span in the transverse direction to the beam multiplied by the ratio $h_{b1}/(h_{b1}+h_{b2})$.

As the actual extent of beam strength enhancement may depend on many parameters including the level of drift (Pantazopoulou and French 2001), calculating the beam strength using a constant flange width can be conservative for some cases and inadequate for some. In either case, the result is not-so-optimum design. If the slab participation is overestimated, it results in an unnecessarily too-strong column. On the other hand, if the slab width participating in the beam's overstrength capacity is underestimated, the designed beam may actually be stronger than the column and plastic hinge may form undesirably in columns. This no-win situation is due mainly to the designers' inability to predict exactly the extent of slab contribution to the beam strength, and would be completely avoided if the slab did not contribute to the beam strength at all. This can be achieved by detaching the slab from the perimeter seismic beams. However, even a slight mention of this idea invariably generates the following question in a designer's mind: how will the inertial force be transferred to the moment resisting system then? This paper tries to answer this question and explores an alternate approach to transfer inertial force from the floor slabs to the lateral moment resisting system.


2 **CURRENT DESIGN PRACTICE**

Currently, floors are designed to act as diaphragms that tie different lateral load resisting components and gravity resisting components. In order to achieve this, the floor is monolithically constructed with all beams and shear walls that are present in the building in that level. To ensure the rigidity of the connection, starter bars are provided from the beams and/or the shear walls and these starter bars are overlapped with the slab reinforcing bars for a length that is enough to develop full strength. Typical connection details between beams and floor are shown in Figure 1. The first detail (Figure 1a) shows a connection between a precast hollow-core floor slab and a beam at the end of the floor. This detail is similar to the one successfully tested by Macpherson (2005), which has been adopted in NZS 3101:2006. The second detail (Figure 1b) refers to a connection between a precast hollow-core slab and an intermediate beam inside the floor which has flooring units extending in both sides. The third detail (Figure 1c) shows a connection between a slab and a side beam. This applies to both cast-in-situ slab as well as precast floor with RC topping slab. For precast floors, a timber infill can also be used in the linking slab between the side beam and the first precast flooring unit (Lindsay 2004; Macpherson
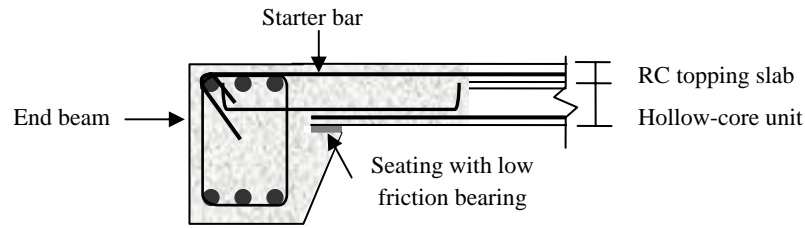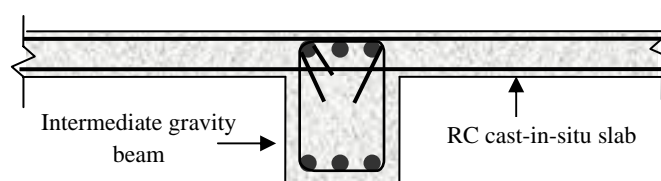
a. Connection between precast hollow-core floor and end beam
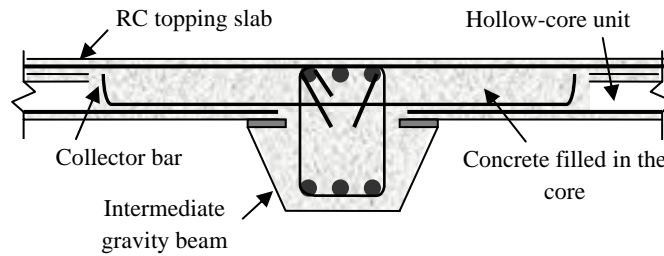
b. Connection between precast hollow-core floor and
intermediate gravity beam
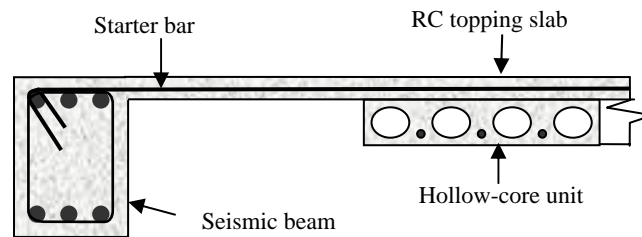
c. Connection between precast hollow-core floor and side
seismic beam

d. Connection between cast-in-situ floor and intermediate
gravity beams

Figure 1 Details of floor-beam connections in current practice

2005). The fourth detail (Figure 1d) shows a connection between an intermediate beam and a cast-in-situ slab. In all these connections, the common feature is the use of starter bars which mechanically connect the slab and the seismic beams; thereby forcing them to deform together at the connection.

Such a rigid connection between the floor and the beams serves two purposes. Firstly, the connection between the floor and the beam ensures that the dead and live loads on the floor are transferred to the columns through these beams. In doing so, the gravity load is distributed to all beams in proportion to the tributary floor area they are supporting. Secondly, the connection between the seismic beam and the floor slab ensures that the inertial force on the floor during an earthquake is transferred to the columns through shear friction between the seismic beams and the floor slab. This load path is schematically illustrated in Figure 2. This arrangement expects gravity beams to transfer vertical gravity load only whereas seismic beams are expected not only to carry a part of gravity load, but also to resist the lateral seismic force and transfer the inertial force to the columns. This design concept
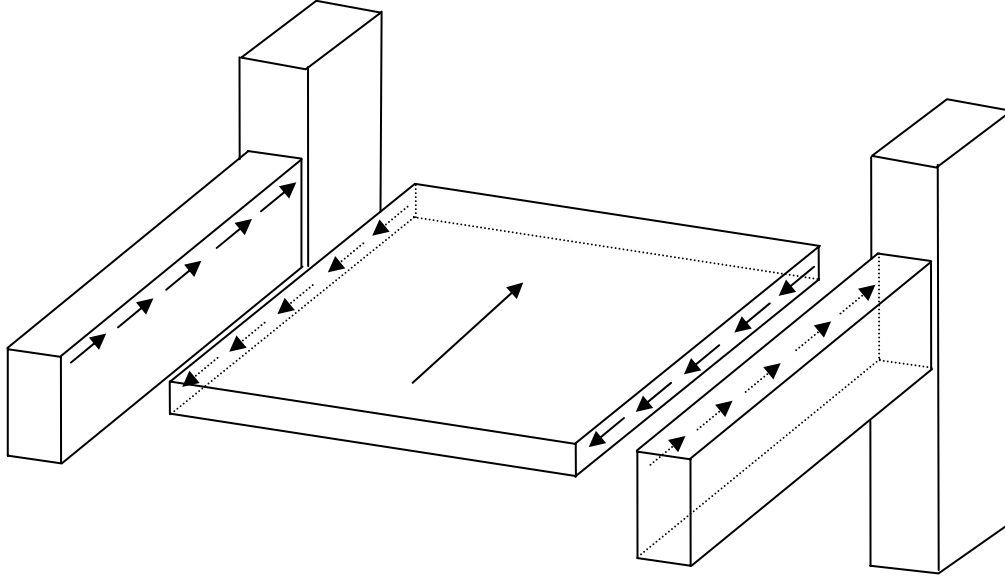
Figure 2 Load path assumed in current design practice: transfer of inertial force through shear friction between the seismic beam and the floor

ignores the ability of the transverse beams, to which the slab is rigidly connected, to transfer a part of the inertial force through torsion. This unaccounted ability of transverse beams to transfer inertial force is explored further in this paper. If this resistance is found to be enough to cater for the total inertial force demand, designers can completely detach the floor and the perimeter seismic beam, thereby avoiding the following undesired consequences.

2.1 **Compatibility requirement between the seismic beams and floor slab**

If the floor slab is rigidly connected to the seismic beam, any deformation of the seismic beam has to be followed closely by the floor slab as well. During cyclic excitations, the seismic beams are expected to deflect in double curvature during the elastic cycles. The additional stiffness provided by the slab will tend to reduce the flexural deformation of the beam, which is not taken into consideration in design because it is difficult to quantify. When the slabs are forced to deform in this mode as shown in Figure 3a, flexural cracks will be induced in the tension side of the slab. Note that the tension will be on the top of the slab near the column deflecting outwards and on the bottom of the slab near the other column (i.e. column drifting inwards). After plastic hinges are formed in the beam, the deformation pattern changes and the slab is also required to match the large local deformation in the region connected to the plastic hinge. Moreover, the plastic hinges elongate in length (Fenwick and Megget 1993), forcing the columns to move apart. This induces diagonal tensile stresses in the slab near the beam. These mechanisms cause the linking slab to crack extensively as shown in Figure 3b. Severe damages in the rigidly connected linking slab have also been observed during experiments (Matthew 2004; Lindsay 2004; Macpherson 2005).

2.2 **Possibility of plastic hinge forming away from the column**

Because of the rigid connection with the floor slab, the seismic beams will also be exposed to a part of the gravity load in addition to the lateral seismic force. When the moments due to gravity load and seismic force are combined, the maximum negative (hogging) moment occurs invariably at the beam-column interface. On the other hand, the maximum positive (sagging) moment can occur anywhere in the half span of the beam depending on the relative proportion of the gravity load and the seismic force acting on the beam. As long as the seismic force to gravity force ratio is large enough to ensure that the slope of the linear seismic moment diagram is larger than the maximum slope of the parabolic gravity moment diagram (i.e. at the column face), the resultant maximum positive moment also occurs at the column face, where a reversing plastic hinge will form. However, as the gravity load gets bigger the location of the maximum positive moment shifts away from the column face, and a unidirectional plastic hinge will form. The formation of unidirectional and reversing plastic hinges has been
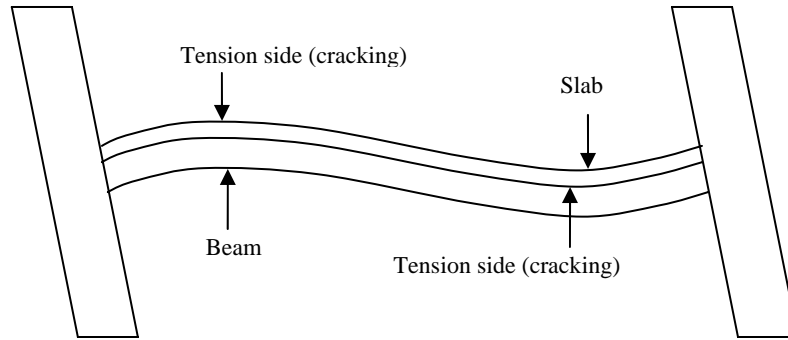
a. Deformation of slab and induced cracking
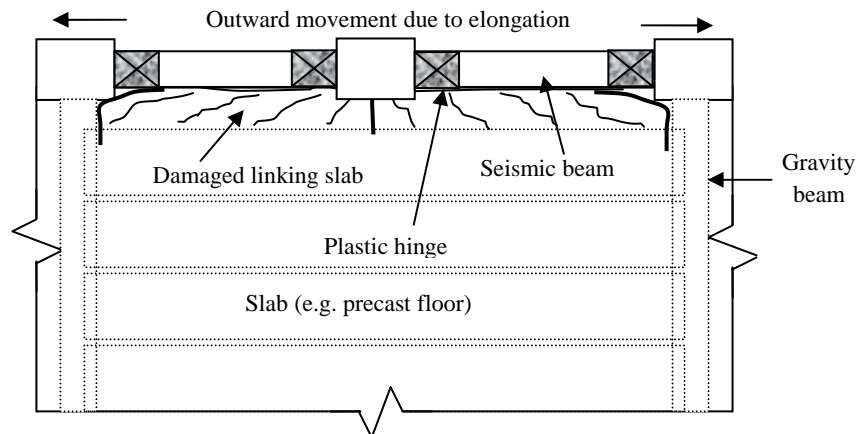
b. Damage in linking slab due to elongation of seismic beam

Figure 3 Damage in slab due to compatibility requirement

described in detail elsewhere (Fenwick and Megget 1993).

The possibility of formation of unidirectional and reversing plastic hinges due to different combinations of gravity and seismic moments is schematically illustrated in Figure 4. Unless a detailed calculation is done using the exact values of gravity load and lateral seismic forces acting on
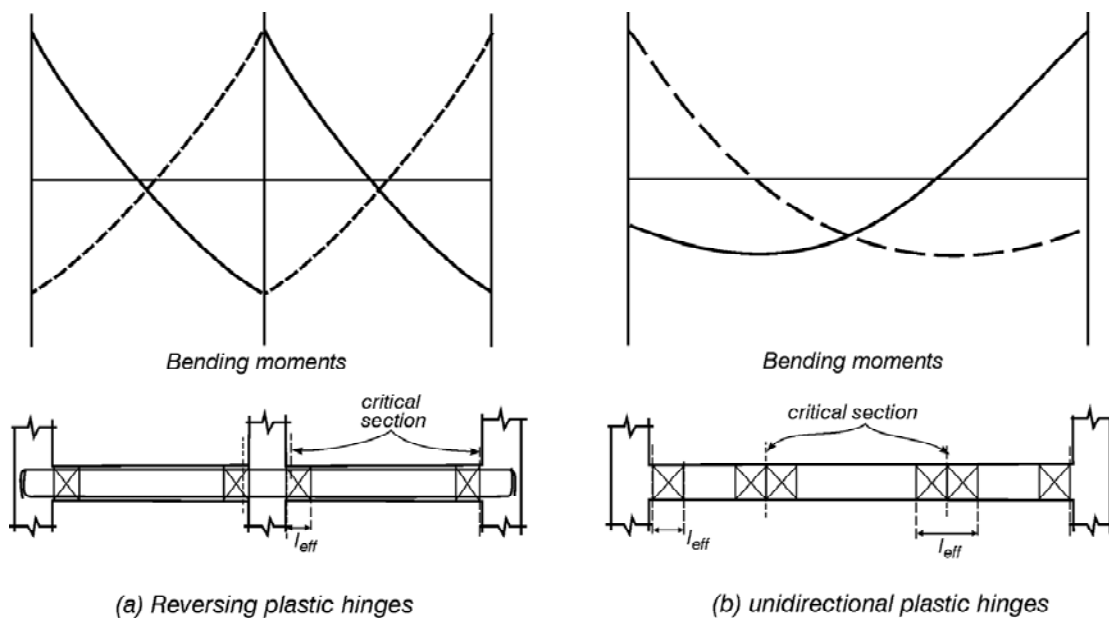

Figure 4 Formation of reversing and unidirectional plastic hinges

the beam (which are difficult to accurately predict), it is not possible to exactly locate the unidirectional plastic hinges. To cater for this uncertainty, the special detailing requirement for plastic hinges need to be provided throughout the beam length. Note that any chance of the unidirectional plastic hinges occurring away from the column face would not exist if the seismic beams were not required to cater for the gravity loads. This could be achieved by detaching the floor from the seismic beams.

2.3 **Effect of beam strength enhancement**

There are two possible reasons of the beam strength enhancement in monolithically constructed beam-slab sub-assemblies. Firstly, because the floor slab is rigidly connected to the top of the beam, some longitudinal reinforcing bars of the slab contribute to the negative (hogging) moment capacity of the beam. In the positive (sagging) moment direction though, the beam top will be in compression and hence this aspect of slab contribution will not matter as the capacity will be governed by the reinforcing bars at the bottom of the beam. The second reason for the increase of beam strength is the compressive force applied by the slabs in response to the elongation tendency of seismic beams. When the seismic beams undergo cyclic deformation, plastic hinges will form and the beam elongates in length. The slab will be pulled and in return, it applies axial compression (the maximum value of which is the yielding force of the slab longitudinal bars across the length of the tensile crack along the slab-transverse beam interface) to the seismic beam. This axial compression will increase the section capacity of the beam. The slab contribution to the beam strength due to the former mechanism is likely to be larger than that due to the latter.

Normally, any unforeseen gain in the strength of a structure would be perceived as favourable, but in capacity design an increase in the strength of the weakest component is a threat. As capacity design aims for weaker beams and stronger columns, any unaccounted increase in the beam strength can potentially alter the actual strength hierarchy. This increased strength of the beam affects the capacity design of the frame in two ways. Firstly, a larger flexural capacity of the beam requires higher shear strength to avoid shear failure. This increased shear demand will require either a more crowded transverse reinforcement or a larger beam cross-section. Next, a stronger beam will require a stronger (bigger) column to force plastic hinges to form in the beam. Both of these could potentially add to the cost of the building. Among the aforementioned two mechanisms leading to beam strength enhancement, the first (i.e. T beam action) can be avoided if the slab is detached from the seismic beam but the second (axial compression being induced in the seismic beams) cannot be. The only way to avoid this is by completely avoiding beam elongation; this could be achieved by using sliding connections (Butterworth and Clifton 2000) or slotted beam connections (Ohkubo and Hamamoto 2004) or non-tearing connections (Amaris et al. 2007) in the beam-column joints. In this paper, only the former mechanism is discussed as the latter is not unique to the proposed system.

Modern design codes (ACI 318:2005; NZS 3101:2006) account for this beam strength enhancement by considering a portion of the slab in both sides of the seismic beam, which is treated as a T beam. However, the effective flange width varies depending on the level of drift (Pantazopoulou and French 2001) and cannot be accurately predicted. Ongoing beam-column-slab subassembly test at University of Canterbury (Peng et al. 2008) has shown that the actual flexural strength and overstrength of seismic beams monolithically constructed with the slab can be as high as 1.4 times those calculated according to the current provisions of NZS 3101:2006. Hence, there is a possibility of the beam strength enhancement not being fully accounted for, especially as the response approaches towards large inelastic drift. In such cases, if the margin between the joint moments corresponding to the strengths of column and beam is small, the actual increased beam strength might lead to the undesirable strong-beam weak-column hierarchy. Even if this is avoided by providing a stronger than required column, the beam may still fail in brittle shear failure mode if the margin between the shear and flexural strengths of the beam is not big enough. Any possibility of these undesirable mechanisms could be avoided merely by detaching the floor from the seismic beam.

3 **PROPOSED DETACHED FLOOR SYSTEM**

3.1 **General concept**

In order to get rid of the aforementioned complications, a floor system that is monolithically connected
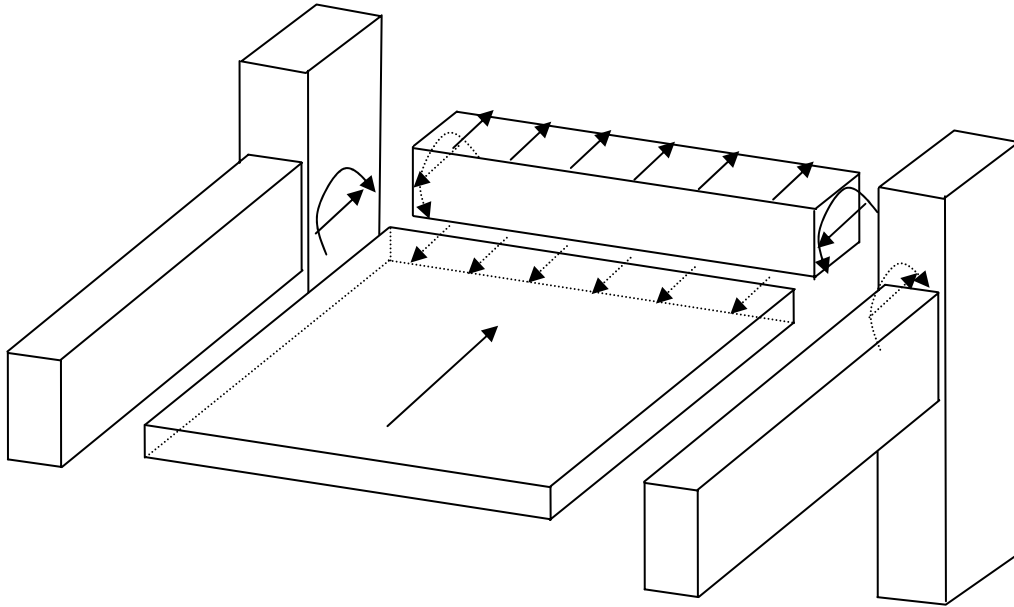
Figure 5 Load path in the proposed detached floor system

only to the intermediate beams but not to the beams in the perimeter seismic frames (referred as "detached floor" hereafter) is proposed herein. In a detached floor system, gravity loads are completely borne by the transverse beams. As the floor slab is not connected to the beams of the perimeter seismic frames, they are not required to carry the dead and live loads. Also, the strength of these seismic beams is not enhanced by the participation of slab reinforcing bars. Nevertheless, if the elongation of the perimeter seismic beams is not avoided, the slab will still induce an eccentric axial compression force in the beam, which will lead to increased beam strength. Nevertheless, as the beam strength can be easily and reliably predicted by section analysis, designers can have high confidence in the aimed strength hierarchy. In lateral seismic excitations, such beams freely deform without imposing any stresses in the slab. The slab can remain plane while the seismic beams deform in double curvature and form plastic hinges. The inertial force coming from the detached floor is transferred to the columns via the top of transverse beams. As shown in Figure 5, this way of transferring the inertial force subjects the transverse beams to a significant torsion in addition to bending in the weak axis. If the biaxial bending demand on transverse beams and the torsional demand on the transverse beam-to-column interface are catered for, there is no need to have a monolithic connection between the seismic beams and the floor slab.

A potential threat to this concept could be the formation of plastic hinges in the intermediate transverse beam, which could then lose any torsional capacity. Although designers expect the perimeter seismic frames (or shear walls, if any) to cater for the lateral seismic force, the intermediate gravity frames will also attract a portion of the total seismic force in proportion to their stiffness. If this lateral force component is large enough to create plastic hinges in the gravity beams, these beams will not have any torsional resistance, and inertial force can not be fully transferred by these beams in a subsequent orthogonal shaking. This problem could lead to a serious deficiency in the system when a large earthquake with strong bi-directional shaking (enough to create plastic hinges in the intermediate beams) occurs and if the intermediate beams exist only in one direction (e.g. in precast floor). Note that in two-way slabs with intermediate beams in both directions, the inertial force could still be transferred through shear friction between these intermediate beams and the slab. Although this potential problem is overlooked in this paper, its likelihood, consequences and remedy need to be investigated in more detail through advanced analytical and experimental studies.

3.2 Example: two-way cast-in-situ flooring system

Next, the proposed concept of detached floor is explained for a typical moment-resisting frame building with different types of floor. In Figure 6, a cast-in-situ two way slab in a moment resisting
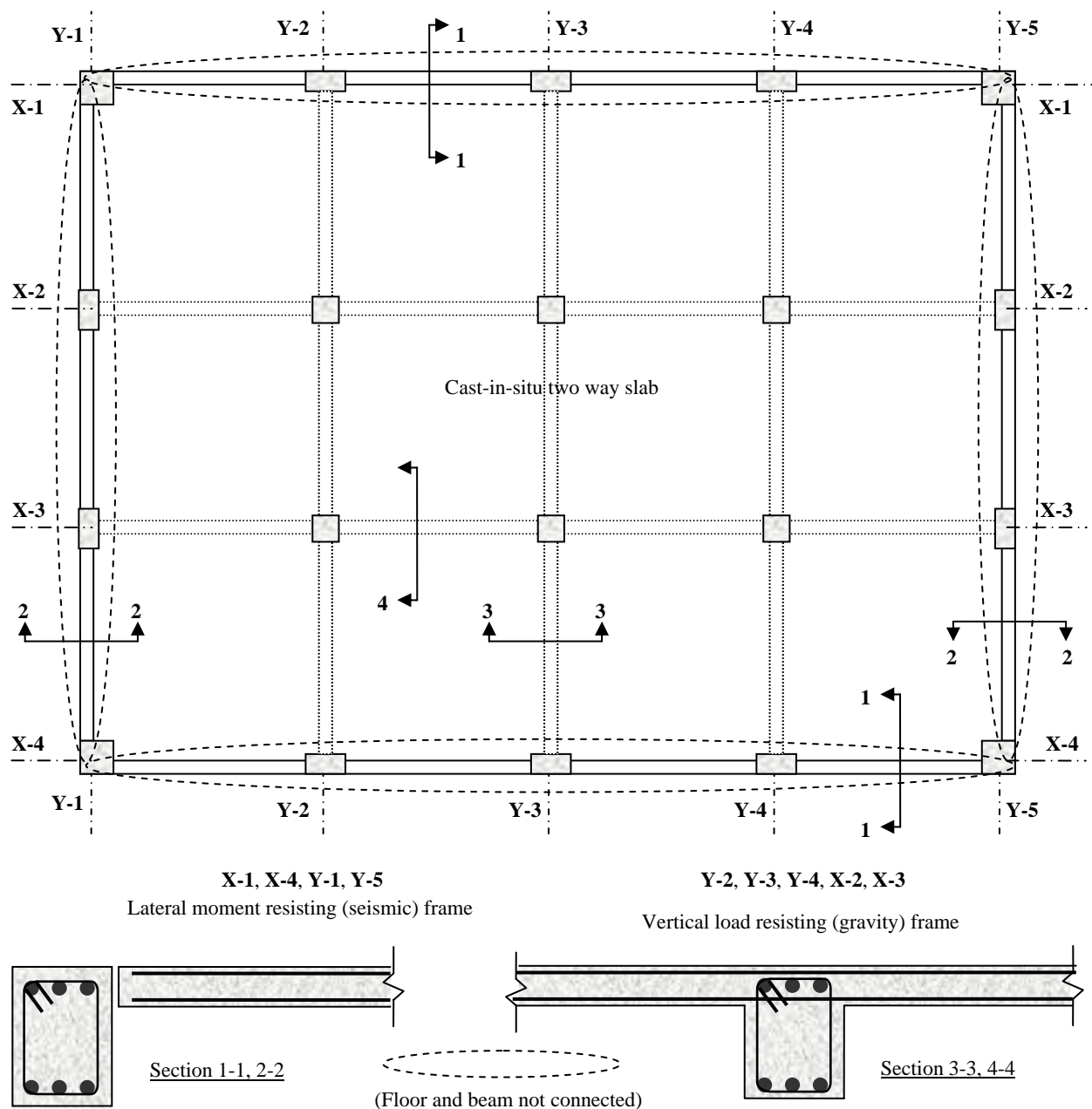
Figure 6 Conceptual illustration of the proposed approach for two way cast-in-situ floor

frame building is shown. Note that the proposed connection details shown in the figure are only for cast-in-situ two way slabs. As indicated in the figure, the perimeter frames are designed to resist lateral seismic forces in the two orthogonal directions and the internal frames are designed to resist the gravity load. The proposed concept of detached floor recommends rigid connection between the intermediate gravity beams and the floor slab. This can be ensured either by providing starter bars from the beams or by continuing the slab bars through the beam as shown in the figure.

In contrast to the current practice, in the proposed system the slab is completely detached from the seismic beams in the perimeter of the slab. By doing so, the slab will not constrain the flexural deformation of the beam nor will it contribute to the strength of the beam. In such an arrangement, the slab is expected to remain more or less undamaged as it does not need to follow the deformation profile of the seismic beams. The slab remains straight while the seismic beam adjacent to it will undergo flexural deformation in a double curvature mode. In this case, the deflection of the cantilever portion of the slab overhanging from the intermediate gravity beams need to be checked.

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As intermediate beams (to which the slabs are rigidly connected) exist in both directions, the majority of the inertial force will be transferred through the shear friction between the slab and the intermediate beams. Consequently, the probable loss of torsional resistance due to plastic hinge (if any) forming in the intermediate beams during a strong earthquake does not pose a potential problem in such two-way cast-in-situ flooring systems.

3.3 **Example: one-way precast flooring system**

When precast flooring units are used (very common in NZ), the transverse seismic beams at the end of the floor cannot be completely detached from the slab as precast units need to seat on these beams for

X-1, X-2, Y-1, Y-5
Lateral moment resisting (seismic) frame

Y-2, Y-3, Y-4
Vertical load resisting (gravity) frame

Section 1-1

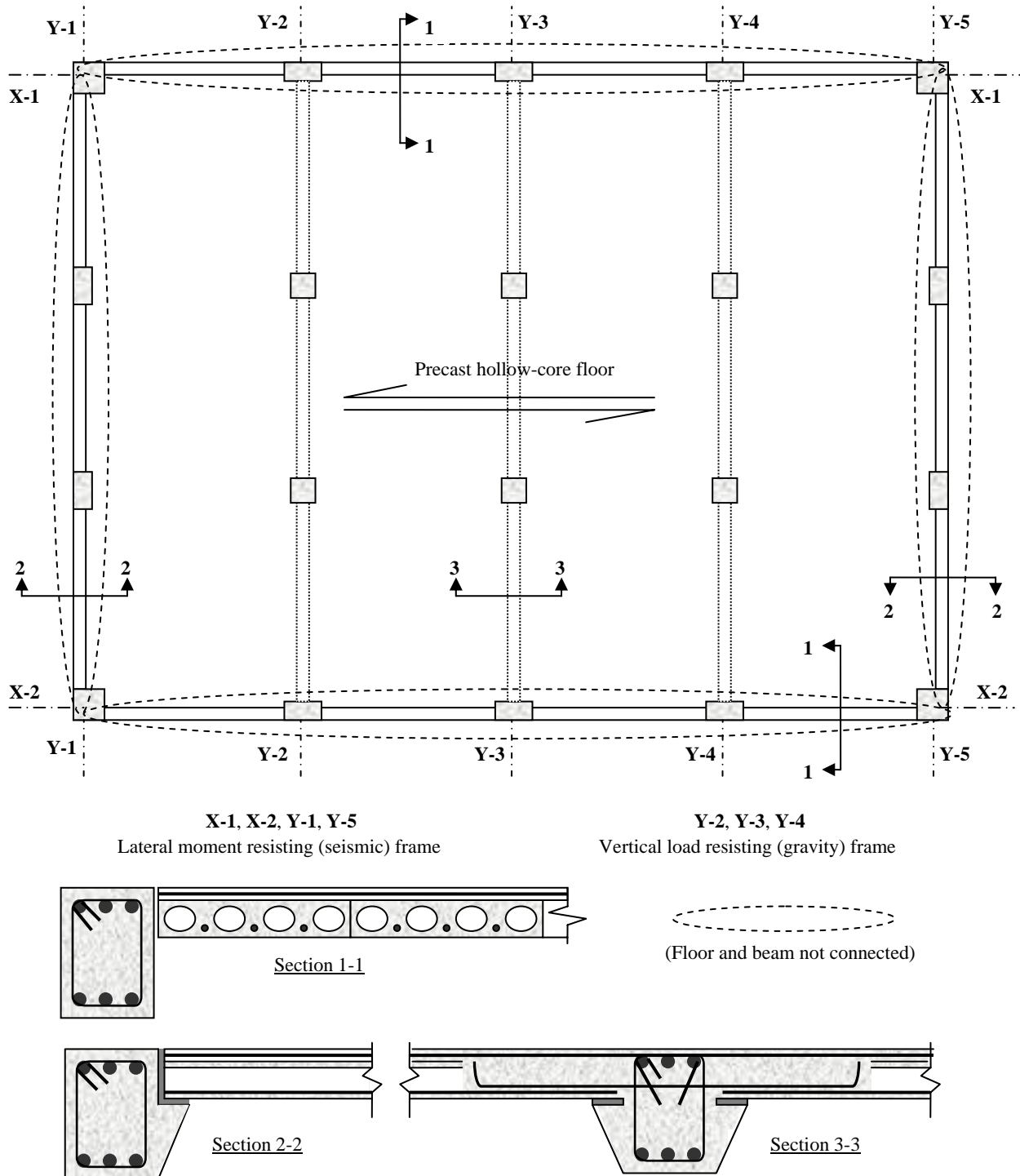(Floor and beam not connected)

Section 2-2

Section 3-3

Figure 7 Conceptual illustration of the proposed approach for one way precast floor

stability. A typical floor plan and proposed connection details for precast hollow-core floor system are shown in Figure 7. Similar to current practice in NZ, the precast units rest on the seating provided on the intermediate gravity beams and the transverse seismic beams at the end of the floor. Nevertheless, in the proposed system the topping slab is rigidly connected only to the intermediate gravity beams. As shown in the figure, if hollow-core flooring units are used the precast floor units can also be rigidly connected to the intermediate gravity beams by filling the cores and by providing an anchoring bar to connect the adjacent precast flooring units across an intermediate beam. This is similar to the connection between precast hollow-core floor slab and the end beam tested by Macpherson (2005). The only difference is that the proposed method allows this connection for the intermediate gravity beams only, and not for the end beams.

The transverse seismic beam at the end of the floor is not connected to the slab except for providing a dry connection in the form of seating for the precast flooring units. This requires that the end beams must be able to support the weight coming from the floor, but does not require any compatibility between the end beam and the floor. Also, as the beam and floor are not mechanically connected, the beam strength is not affected by the floor resting on the seating of the beam. In order to prevent damage by the impounding action of the precast flooring units on the end beams, a thin layer of compressible elastic material may be used between the faces of the beam and precast units. Note that the seismic beam in the side frames along the direction of the floor does not need to support the floor (see section 1-1 in Figure 7).

As can be noticed in the figure, the precast units span in one direction and the gravity beams are not required in the direction of the flooring units in such a floor. When an earthquake with predominant shaking in the orthogonal direction occurs, the inertial force has to be transferred by the shear friction between the floor slab and the intermediate gravity beams. If plastic hinges form in these intermediate
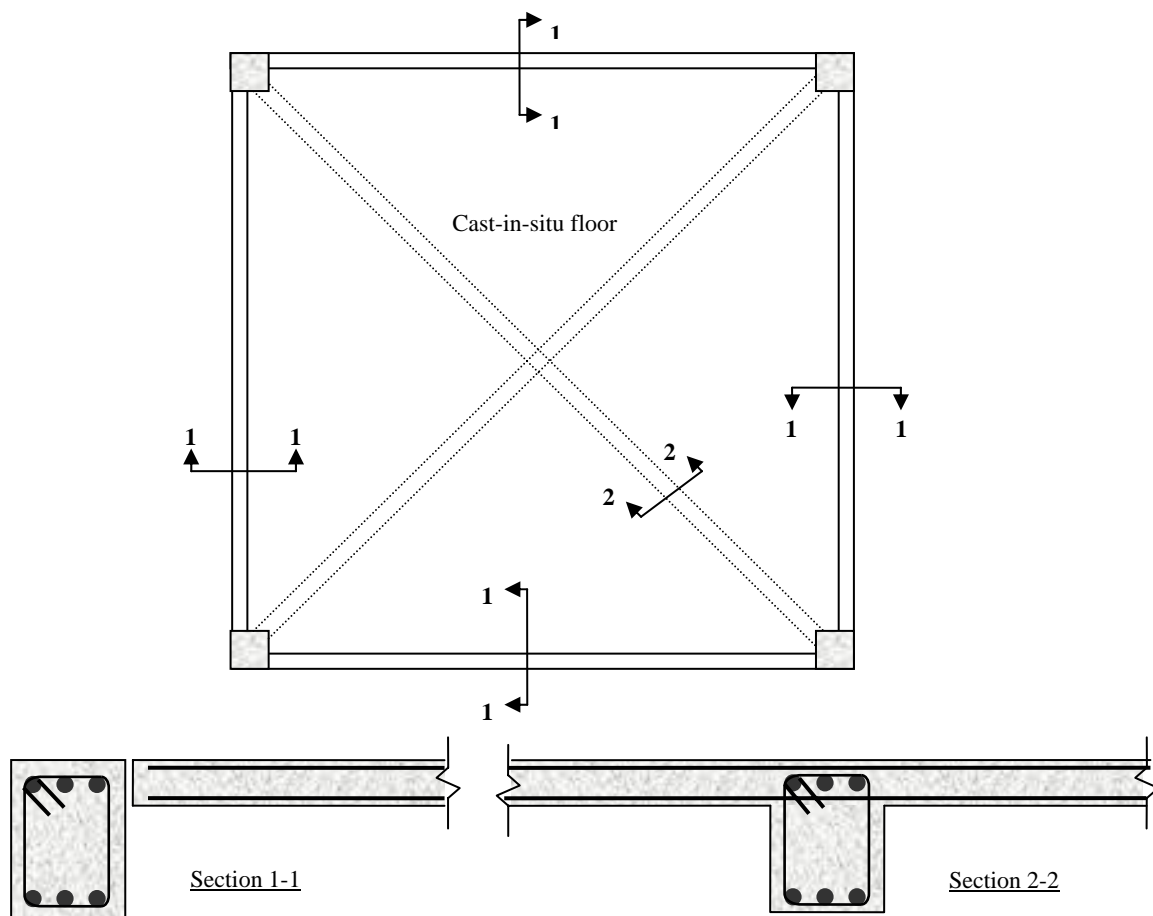

Figure 8 Diagonal beams and their connection with single bay floor slab

gravity beams resulting in loss of torsional resistance, inertial force cannot be transferred during subsequent shaking in the longitudinal direction. Hence, the likelihood, consequences and mitigation of plastic hinges forming in intermediate gravity beams in such systems need to be investigated further.

## 3.4 **Structures without intermediate beams**

The aforementioned examples do not cover buildings which have only one bay in both directions and have no gravity frames. When there are no intermediate gravity beams (i.e. when the slab spans across a single bay surrounded by moment resisting systems in all four sides), diagonal beams connected to the columns could be used to transfer the inertial force. Such a case with cast-in-situ floor is shown in Figure 8. Note that such buildings are rare and the recommendation is still to avoid monolithic connection between seismic beams and the slab by providing diagonal beams. In such cases, these diagonal beams are rigidly connected to the slab and they transfer the inertial force partly through strut mechanism, partly through torsional resistance and partly through shear friction. In cast-in-situ floor, providing additional diagonal beams is justified because the benefits of detaching the seismic beams and the floor outweigh the additional resources required. Nevertheless, the deflection of the cantilever portion of the slab near the perimeter beam should not exceed the allowable limit. Note that if the slab is made of precast flooring units, the diagonal beams will not be needed as the flooring units can rest on the seating provided in the two end beams. Note that in such single bay structure, these diagonal beams will be pulled apart to accommodate the elongation of perimeter seismic beams. This will induce compression in the perimeter beams, which will result in an enhancement of the beam strength. As mentioned earlier, this can be avoided if non-elongating connections (Butterworth and Clifton 2000; Ohkubo and Hamamoto 2004; Amaris et al. 2007) are used between the perimeter beams and columns.

## 4 **POSSIBLE FAILURE MODES IN THE DETACHED FLOOR SYSTEM**

In the proposed design concept, the slab is rigidly connected to the transverse intermediate beams but not monolithically connected to the perimeter seismic beams. The inertia force is intended to be transferred through the torsional resistance of transverse beams to column connections. For this to be possible, the intermediate gravity beams and/or the diagonal beams and their connections with the columns have to be designed to resist and transfer the maximum possible inertial force from the supported slab area. In such cases, five possible failure modes are possible.

Firstly, the inertial force applied at the top of the beam may induce large in-plane shear stress, thereby leading to a possibility of shear cut-off of the beam section immediately below the slab. This is specially a matter of concern in precast flooring system, in which the gravity beams are tapered to seat the flooring units in both sides and the shear area of the beam is small near the slab. Secondly, a large torsional moment is likely to develop at the end of the transverse beams, which might trigger twisting of the beam-column connection. Thirdly, the uniformly distributed inertial force will induce bending stresses and flexural deformation in the weak axis of the transverse beams. Next, the horizontal reaction due to the distributed inertial force combined with the torque is also likely to result in a significant shear stress at the interface between the transverse beam and the column. Lastly, due to the elongation of the unconnected seismic beam along the side of the slab, the transverse beams will be pushed apart, thereby creating a significant tensile stress in the slab. This may lead to the tearing of the transverse slab to beam connection and facilitate the unseating of precast flooring units. Note that the last mode related to beam elongation is not unique to the proposed system; this threat exists in the existing design approach, too. Apart from the beam elongation effects, expressions derived to estimate the demands corresponding to the other five possible failure modes are shown in Figure 9.

Through proper measures, all of these failure modes can be controlled. The shear cut-off of the beams can be avoided, if needed, either by increasing the beam width or by using shear keys inside the beam section to engage the increased width of the tapered beam below the slab seating. To avoid torsion and/or shear failure at the beam-column connection, torsion/shear keys can be provided in the column at two or four corners of the beam. The bending stress and deformation in the weak axis may not
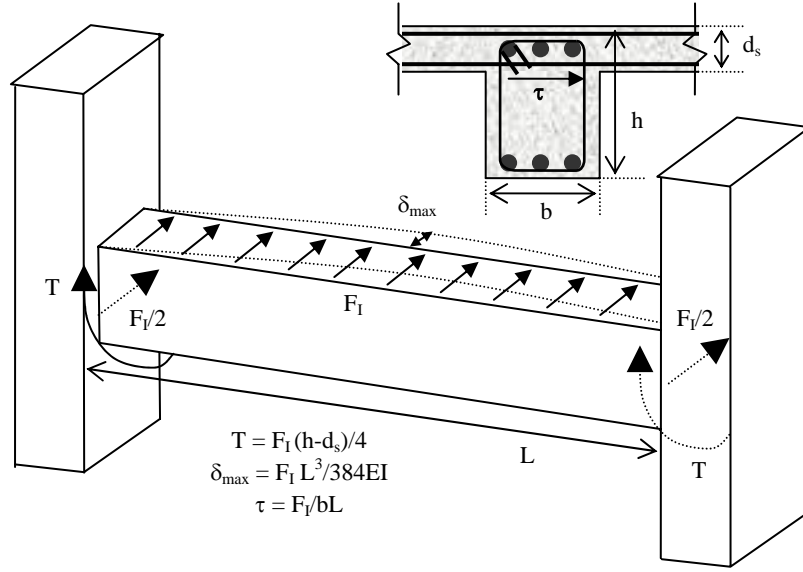
a. Transfer of inertial force from cast-in-situ floor

$$T = F_I(h-d_s)/4$$
$$\delta_{max} = F_I L^3/384EI$$
$$\tau = F_I/bL$$
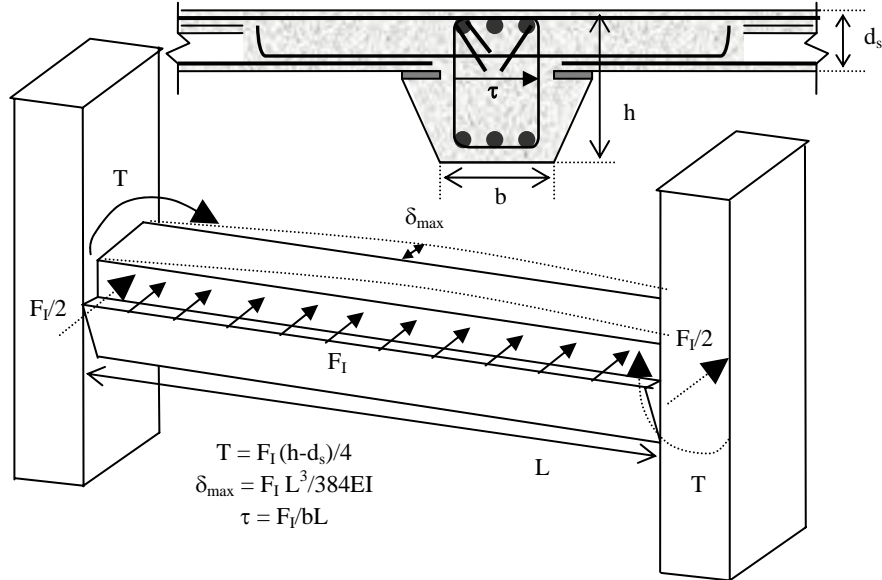
b. Transfer of inertial force from precast floor

Figure 9 Torsional moment and direct shear stress induced by inertial force

usually challenge the allowable limits; whenever they do, the beams can be modified to satisfy these limits. Finally, the potential threat of seismic-beam elongation can be nullified by providing sufficient seating length to accommodate the elongations of the beam plastic hinges in that bay. These issues are discussed in more detail below, where for regular floor systems, design calculations required to avoid the abovementioned damage/failure modes are presented.

5 QUANTITATIVE VERIFICATION OF THE PROPOSED SYSTEM

This section tries to quantitatively verify the concept of transferring the inertial force through the torsional resistance of gravity beams perpendicular to the direction of inertial force. Rather than designing new floor-frame-subassembly according to this concept, moment resisting frame buildings designed for transferring inertial force through shear friction are investigated by assuming no connection between the floor and the seismic beams. By doing so, it will also be clear if the sizes of

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beams decided according to the current design guidelines are enough to transfer the inertial force through torsion or if the proposed concept requires transverse beams unreasonably bigger than the existing ones.

Two different structures are used for this verification purpose; the first one is a half-scale 3-D beam-column-slab subassembly designed to satisfy the requirements of the current NZ Loadings Standard (NZS 1170:2004) and Concrete Structures Standard (NZS 3101:2006). The subassembly is currently being tested under cyclic loading at University of Canterbury (Peng et al 2008). The subassembly has two bays (3 m each) along the seismic frame direction and one bay of 6 m in the gravity frame direction. The floor is made up of precast Stahlton ribs (300 mm spacing) with a 50 mm thick cast-in-situ topping slab. The 400 mm deep gravity beams are 205 mm wide at the top and have 40 mm projection to provide seating for the precast Stahlton ribs, thereby rendering a bottom width of 245 mm for the intermediate beam and 285 mm for the end beams. The intermediate gravity beam is connected to the topping slab through starter bars and the top half of this beam is cast together with the floor slab to ensure a monolithic connection. The transverse beams at the two end support the precast flooring units through 40 mm seating but the topping slab is assumed to be detached from these beams. Although in the actual specimen the slab is rigidly connected to the seismic beam through starter bars, the calculation herein assumes no connection between the seismic beam and the floor slab.

The second structure used here for the verification of the proposed concept is the red book (Bull and Brunsden 2000) moment resisting frame building designed according to the 1995 version of NZ Concrete Structures Standard (NZS 3101:1995). This moment resisting frame building has a footprint of 29 m × 29 m. It has three bays in each direction and the external frames are designed for seismic forces whereas the internal gravity frames are provided only in one direction because the floor is designed to be made of precast hollow-core units with RC topping. A gravity beam with span of 8.7 m which has a tributary slab span of 9.2 m is chosen for verification here. The gravity beam has a depth of 750 mm and a minimum width of 530 mm with a tapered profile to provide seating for the hollow core floor units. Again, the original design used rigid joint between the precast slab and the seismic beams in the perimeter, but the calculation here is based on the assumption that the slab is rigidly connected to the intermediate gravity frames only and the beams along the precast floor direction do not have any interaction with the floor whereas the transverse seismic beams at the end of the flooring direction are provided with seating to support the precast units without any mechanical connection between them. The calculation to verify the proposed concept for these two structures is shown in Table 1.

As can be seen in the Table (most calculations are self-explanatory), both structures would be safe against all possible failure modes even if there was no connection between the floor and the seismic beams. Note that the inertial force is calculated using the design dead and live loads and conservatively calculated using a maximum response acceleration of 1 g. Due to the gravity load and the inertial force, the maximum vertical and horizontal deflections are found to be nominal. These calculations are based on uniform distribution of inertial force, and actual horizontal displacement will be smaller as the distributed inertial force is in fact larger in the sides than in the middle. Cracking would reduce the effective flexural rigidity (especially in the major axis direction), but still the maximum deflections calculated using reduced EI will not challenge the allowable deflection limit. The average in-plane shear stress across the beam width is 0.13 MPa and 0.2 MPa for the two cases, which are well within the shear capacity of concrete; thereby not requiring any extra measures.

The shear force and torsional moment at the beam-column interface combine to induce a total shear stress of 1.6 and 4.33 MPa in the two cases. As both of these are less than the allowable shear stress of 6 MPa (NZS 3101:2006), no extra measure is required to resist the induced shear force and torsional moment. As the elongation conservatively calculated assuming 4% of the beam depth is less than half of the seating length required by the current design code, this should not create any problem. Moreover, the seating length should not be an issue when the precast units are of hollow-core type, which are rigidly connected to the beam by filling the cores and providing an anchoring bar connecting the two units in both sides of the beam. The outward movement of the transverse beams (to accommodate the elongation of plastic hinges in the side seismic beams) will, however, be significant and will cause localised cracking and yielding of the longitudinal reinforcing bars in the slab near the

connections with intermediate beams. As in the proposed concept the slab will not be monolithically connected to the transverse seismic beams at the end of the floor, any precast units are free to slide on the seating of the end transverse beams and any cast-in-situ slab is free to move away from the end beam. Hence, the slab will not have any problem to negotiate the elongation of plastic hinges in the end bays and the aforementioned damage will be restricted to the intermediate bays only.

Table 1. Design calculations to check the validity of the proposed concept.

Parameters	Red book frame building	Half-scale lab sub-assembly	Unit	Reference
Concrete properties				
Compressive strength, f_c'	30	30	MPa	
Modulus of Elasticity, E_c	25084	25084	MPa	NZS 3101 5.2.3
Gravity beam details				
Length, L	8.70	6.00	m	
Height, h	0.75	0.40	m	
Width, b	0.53	0.25	m	
Cover to centre of stirrup, d'	0.030	0.025	m	
Area in shear flow path, A_0	0.324	0.068	m^2	
Perimeter of shear flow path, P_0	2.32	1.09	m	
Shear flow thickness, t_0	0.10	0.05	m	
Moment of inertia (minor), I_{gh}	0.009304813	0.000490204	m^4	
Moment of inertia (major), I_{gb}	0.018632813	0.001306667	m^4	
Slab/Floor thickness, d_s	0.37	0.15	m	
Tributary slab span	9.20	6.00	m	
Seismic mass for the beam				
Basic live load, Q_b	2.50	2.50	kPa	NZS 1170.5
$\psi_a = 0.4 + 2.7/\sqrt{A} \leq 1.0$	0.70	0.85		NZS 1170.5
Reduced live load, Q	1.75	2.13	kPa	NZS 1170.5
ψ_u	0.40	0.40		NZS 1170.5
Design live load, Q_u	0.70	0.85	kPa	NZS 1170.5
S.D.L., G	6.77	7.15	kPa	NZS 1170.5
Total seismic load, E_u	7.47	8.00	kPa	NZS 1170.5
Response acceleration	1.00	1.00	g	
Inertial force, F_I	598	288	kN	
Gross EI (horizontal)	233406	12296	$kN\text{-}m^2$	
Gross EI (vertical)	467393	32777	$kN\text{-}m^2$	
Maximum in-plane deflection, δ_{max}	4.4	13.2	mm	
Maximum vertical deflection, δ_{max}	2.2	4.9	mm	
Direct shear stress across beam, τ	0.13	0.20	MPa	
Interface torsion, T	57.6	18.4	kN-m	
Torsional shear stress, v_{tn}	0.85	2.86	MPa	NZS 3101 7.6.1.6
Sliding shear stress, v_n	0.75	1.47	MPa	
Total shear stress at interface, v_{total}	1.60	4.33	MPa	
Allowable shear stress, v_{max}	6.00	6.00	MPa	NZS 3101 7.5.10
Elongation of a plastic hinge	30	16	mm	Assuming 0.04h
Beam seating length provided	75	40	mm	

6 CONCLUSIONS

A novel concept to design floor slabs that are completely detached from the perimeter seismic beams

has been proposed. In such a system, the inertial force during an earthquake is transferred to the column through the shear friction between the slab and intermediate longitudinal gravity beams or through torsional resistance of the intermediate transverse gravity beams. If such a system can be achieved, it will offer at least the following advantages which are obvious at this point:

1. The slab does not need to be compatible with the perimeter seismic beams. Therefore, regardless of the deformation profile of the seismic beams, the slabs will not be damaged in the region close to the perimeter seismic beams.

2. In such a system, the slab reinforcing bars will not participate in the negative moment capacity of the perimeter seismic beam. Nevertheless, the elongation of perimeter beams will pull the slab between the transverse gravity beams, which will induce eccentric axial compression in the seismic beams and enhance the strength of these beams. To completely avoid this strength enhancement, non-tearing non-elongating beam-column connections may be used.

3. As the beam strength will not be increased by slab participation, it will be possible to estimate the beam strength with more certainty, which will avoid any threat to the strong-column weak-beam hierarchy. Moreover, shear demand to avoid shear failure will be smaller and smaller columns can satisfy the strong-column weak-beam requirement.

4. The construction will be easier as the seismic beams need not have starter bars to connect to the floor slab. Consequently, the beams in the moment resisting frames can be completely precast.

5. As the slab will not transfer the gravity load to the seismic beam, the plastic hinges will invariably be of reversing type and will form at the column face. This renders the detailing concise and easy.

The proposed detached floor system has been verified for two example structures; one full-scale moment resisting frame building with precast hollow-core floor with RC topping and the other a half-scale beam-column-slab subassembly with RC slab with precast ribs. Several possible failure modes (such as torsion, direct shear, biaxial bending and unseating of slabs) were checked assuming that the slabs were completely detached from the perimeter seismic beams. Simplified calculations showed that both structures would be able to avoid these failure modes and transfer an inertial force (resulting from 1g response acceleration) through the torsional resistance of the intermediate transverse beams without any alteration to the sizes of the components.

Nevertheless, this paper has presented only the preliminary state of verification of the proposed system and further work needs to be done before this can be implemented in practice. One potential problem needing further investigation is related to the possibility of plastic hinges forming in the intermediate gravity beam in the precast flooring system. If so, the hinged intermediate beams will lose torsional resistance; thereby making them unable to transfer inertial force in the longitudinal direction. As the slab will not be connected to any beam in the longitudinal direction (as no intermediate beams exist in the longitudinal direction), this creates a deficiency in such systems. Note that this will not pose a problem in two-way cast-in-situ flooring systems as the shear friction between the slab and the intermediate gravity beams will transfer the inertial force.

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