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TO CELEBRATE THE LIFETIME CONTRIBUTIONS OF
PROFESSORS EMERITI TOM PAULAY AND BOB PARK

UNDERSTANDING THE COMPLEXITIES OF DESIGNING DIAPHRAGMS IN BUILDINGS FOR EARTHQUAKES

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ABSTRACT

The variety of layouts of lateral force resisting elements in structures, subjected to inelastic behaviour, make the design of diaphragms significantly more complex than the traditional "simple beam" approach typically employed.

Traditionally held views that diaphragms are inherently robustness and hence do not require significant engineering input have been shown to be inappropriate by recent major earthquakes and recent laboratory studies.

The simple beam method, at times, fails to recognise that the traditional load paths assumed are compromised by localised damage in the floors (diaphragms) due to incompatibility of deformation between the floors and the supporting structures (walls, beam and columns). "Strut and tie" methods are suggested as a means of tying these diaphragms into the lateral force resisting structures and as a way of dealing irregular floor plates and penetrations (stairs, lifts, atriums) through the floors.

The focus of recent research in determining the seismic lateral forces into and through floor diaphragms has been on the magnitude of the floor inertias. However, it has been shown that primary structural elements interacting through the diaphragm, can cause stresses in the floors many more times than those of the inertia effects. These two sources of forces and stresses are interrelated. This relationship requires further study.

Survivability of a building rests, in part, on the diaphragms tying the structure together. Suggestions for maintaining the integrity of the diaphragms, particularly in precast concrete types, are discussed.

BIOGRAPHICAL NOTE

Des Bull completed his BE at the University of Canterbury in 1980 and an ME, under the supervision of Professor Bob Park, in 1983. His research topic was the performance of precast concrete "shell" beams with reinforced insitu concrete cores, subjected to earthquake attack. He is a Technical Director of Holmes Consulting Group Ltd. His duties involve marketing and development of structural engineering services for the company, with an emphasis on concrete structures (commercial and bridges) and the performance of concrete materials in a variety of environments and in-service conditions. He is also the Managing Director of Holmes Solutions Ltd, a company involved in R&D and prototype development for a range of emerging technologies.

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1 INTRODUCTION

Architectural trends today are presenting structural designers with a variety of shapes of floor plates and vertical arrangements. Further, because of diversity of building use, the regular layout for lateral force resisting systems that appeal to structural engineers do not often satisfy the requirements of the architecture. In short, today's building structures, particular the floor plates, are significantly more complicated to design than those buildings with simple rectangular floor plates.

In this context, roofs and floors, called "diaphragms", are relatively thin but stiff horizontal or nearly horizontal structural systems which transmit in-plane lateral forces to, or between, vertical lateral force resisting elements. In addition to the primary role of distributing lateral forces to vertical structural elements, the diaphragms must tie a whole structural system together.

Diaphragms exhibit two types of behaviour at the same time. The first type occurs in every floor, where the floor acts like a horizontal deep beam and transmits forces generated by wind, soil pressure and earthquakes to the various vertical lateral force-resisting components such as frames and structural walls. The second type of behaviour is required where large in-plane forces need to be transferred from one vertical lateral force resisting component to another. Traditional examples of this behaviour include diaphragms that connect a shear core to peripheral foundation walls or that connect moment resisting frames to structural walls (as in a "dual" or "hybrid" structural system). All diaphragms are "transfer" diaphragms as well as transmitting applied forces, such as inertia induced by earthquakes. The extent to which both types of behaviour occur, depends on the characteristics of the vertical lateral load resisting structures and diaphragms: the relative stiffness and strengths of these elements.

2 GENERAL PRINCIPLES AND REQUIREMENTS

2.1 Function of Diaphragms

Diaphragms are typically required to resist lateral wind, soil or earthquake forces in conjunction with supporting gravity loads. The analysis and design of slabs for gravity loadings is not covered in this paper. The subject is covered in depth in the technical literature (Park *et al* 2000) and design practices of numerous design standards.

In general, diaphragms can be modelled as simply supported or continuous deep beams. Figure 1 shows a schematic of the deep beam model often assumed in design. Design forces generated in diaphragms cannot be established with great accuracy; nor is this a concern as flexural and shear stresses are usually not significant in typical diaphragms when the floors are regular and of similar widths and lengths, i.e. squat or compact (Paulay *et al*, 1992). This may not be true for long floors or floors that have an irregular plan with a number of changes of direction or "wings" and re-entrant corners such as shown in Figure 2 (a).

A further assumption for squat diaphragms is one of infinite stiffness, implying rigid behaviour. Such an assumption leads to less effort in modelling the behaviour of vertical lateral force resisting components. The behaviour of an irregular floor diaphragm can make the determination of the actions on the vertical lateral load resisting structures more difficult and may lead to localised premature overload or damage because of concentrations of stresses such as at re-entrant or exterior corners (see Figure 2 (b)). If such floor plans are necessary for architectural or operational reasons, it is recommended that the floor plan be divided into smaller squat or compact plans that are, in effect, independent structures (Paulay *et al*, 1992) (see Fig. 2(c)). Gaps or separations between these independent structures must be sufficiently large to accommodate differential lateral deformations produced during a major seismic event. Examples of such requirements for these separations may be found in the New Zealand Loadings Standard, NZS 4203 (SANZ, 1992).

Diaphragms have openings or penetrations (required for stairs, lifts, elevators and mechanical services passing vertically within a building) that significantly reduce the ability of the diaphragm to resist in-plane flexure and shear, as seen in Figure 2 (d). Locating of penetrations needs to be undertaken by considering the load paths across the diaphragms, ensuring that equilibrium is maintained and localised stresses do not exceed acceptable levels (see Section 2.3 (b)). Preferred locations for openings are suggested in Figure 2 (e).

2.2 Limit Sates

2.2.1 *Serviceability Limit State*

Forces from restraint of strain inducing phenomena such as creep, shrinkage and temperature effects should be considered when investigating the "in service" behaviour of the diaphragms. The levels of forces or effects associated with Serviceability Limit State occur several times in the life of the structure. In such a limit state, crack width and deflection control needs to be maintained. Typically, elements remain generally in the elastic range with the possible exception of limited yielding of expansion joints and at supports of floors where continuity effects lead to yielding of reinforcement in the top of the floor slabs. Similarly during moderate earthquakes the diaphragms should remain essentially undamaged. "In service" performance of precast floor slabs is typically of more concern than for cast-in-place slabs, because of the relatively thin concrete topping cast on to the precast unit and because of the numerous joints between units. Examples of typical pretensioned, precast concrete floor units with cast-in-place concrete topping are shown in Figure 3. The case of untopped precast concrete floors, joined together by intermittent mechanical means (weld plates, shear keys), may be even more prone to undesirable performance. A detailed discussion of the serviceability aspects of precast concrete floors used in New Zealand is given in the "Guidelines for the Use of Structural Precast Concrete in Buildings" (CAE, 1999).

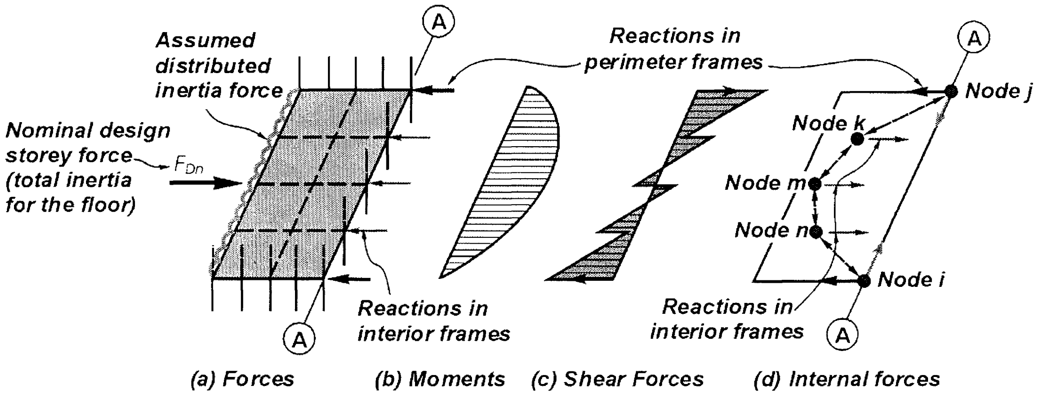


Figure 1. Beam Analogy for Diaphragm Design (NZCS,1994)

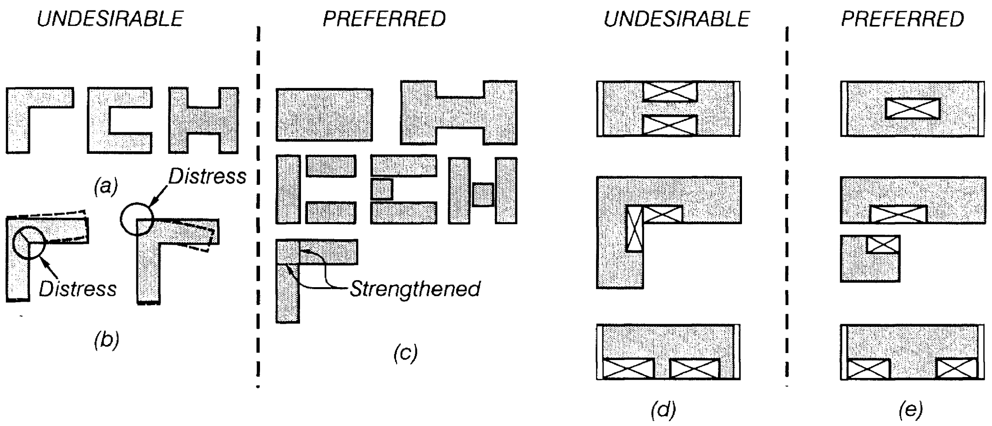


Figure 2. Plan Configurations in Buildings (Paulay et al, 1992)

2.2.2 *Ultimate Limit State*

Generally, diaphragms should be designed to remain elastic under the Ultimate Limit State (ULS) levels of loading. The ULS state occurs when the strength of the element is exceeded and some localised plastic deformation, "ductility", occurs. Whenever significant amounts of ductility or inelastic demands have been imposed on diaphragms, these have not performed well in terms of maintaining load paths for internal in-plane actions or for maintaining the gravity supporting functions. Poor performance of concrete buildings in the Armenian earthquake (Soviet Union) was in part due to catastrophic failure of diaphragms, principally at ineffective and brittle connections between floor diaphragms and vertical structures. Similarly, failure of a number of precast concrete garage structures in Los Angeles, during the 1994 Northridge Earthquake, highlighted the need to maintain paths for internal forces.

2.3 Structural Behaviour of Diaphragms

Provision and maintenance of load paths for internal forces within a diaphragm are principal concerns. The global performance of the vertical lateral force resisting structural system depends to a great degree on the continuous function of the floor diaphragms during extreme loading.

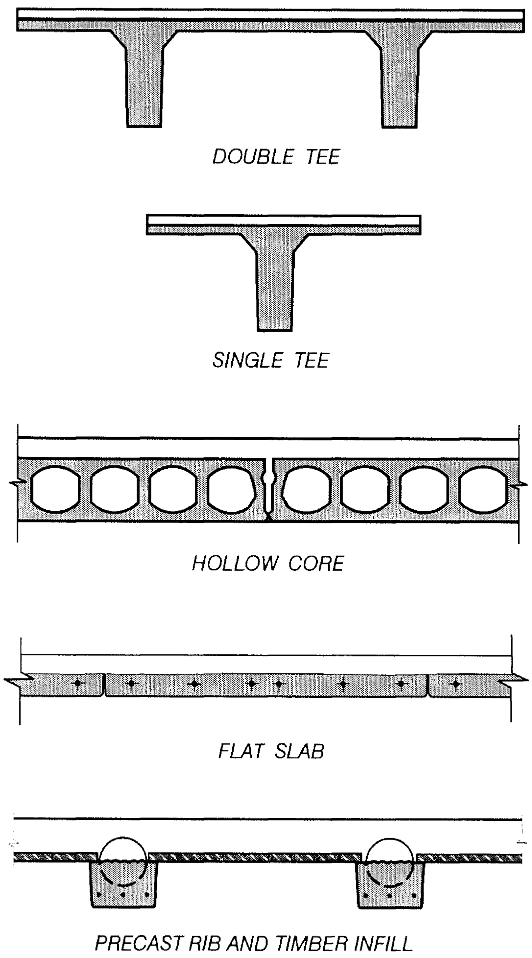


Figure 3. Typical pretensioned, precast concrete floor units with cast-in-place topping.

2.3.1 *Beam Action*

As discussed previously, for regular floor plans, the "beam analogy" is usually sufficient for design purposes (see Fig. 1). When the diaphragm is regular in plan the inertia forces of the diaphragm, resulting from earthquake attack may be modelled as distributed forces along the compression edge of the floor. Though not strictly a true representation of the distribution of inertia effects, this has been a traditional approach for visualising the flow of internal actions or load paths.

Figure 1(a) shows this equivalent applicable distributed inertia force. Figure 1(d) depicts the internal actions in the diaphragm, modelling the behaviour of a deep beam, with struts (compression stress fields in the concrete of the floor) "arching" to the supports (in this example, the primary lateral force-resisting structure). In Figure 1(d), the tension chord or tie along Grid A maintains the equilibrium between the strut-arch and the reactions at the supports, at the nodes i and j. The interior frames connecting to the struts in the diaphragm at nodes k, m and n happen to offer relatively less resistance to the inertia forces than the perimeter frames. The beams of the interior frames, along with reinforcing steel in the slab, act as tension elements which transfer the inertia effects of the parts of the slab that are "under the arch" (using an arch-suspension bridge analogy) up to the arching struts. The flow of internal in-plane compression and tension forces in the diaphragm is a simple example of a design method called the "strut and tie" method. This approach is discussed in more detail in the following section.

For beam action to occur, continuous reinforcement, acting as a tie must be provided along the tension edge of the "beam", Grid A. Where edge beams are present, they will generally be found to have sufficient tension capacity (summing up the contribution of beam longitudinal reinforcement, which is not required to resist gravity loads). It may be debated that this assumption may not hold when an earthquake attack occurs at an oblique angle to the main axis of the building when all the beams of the frames may be acting to resist the earthquake. Alternative locations for ties in the floors will be discussed later in the paper: that the ties should be placed within the floor and the ties should not involve the perimeter beams. Where edge beams are not present, such as along one-way slab systems, and tension effects need to be generated along that edge of a slab as part of the lateral force resisting system, additional continuous reinforcement will be required at diaphragm edges.

Floor diaphragm designed as a deep beam will need to have a shear design undertaken. The shear stresses should be checked against code requirements (SANZ, 1995). Figure 1(c) shows a typical shear force distribution for the simple beam action. The effective cross section for shear of a full depth cast-in-place slab shall be based on the overall thickness, at respective critical sections. For floors made up of precast floor units and cast-in-place topping the effective cross section will be that of the topping alone, at critical sections (SANZ, 1995). Shear stresses are seldom critical for floors with regular plans.

The deep beam analogy is a traditional method for designing diaphragms. However, there is concern over the inability of this method to clearly indicate all the critical locations of force paths, especially for floors with penetrations and irregular floor plans. Designers are recommended to consider the "strut and tie" method of analysis, discussed in the following section.

2.3.2 *Openings through diaphragms and the "Strut and Tie" Method of Analysis*

The presence of large openings in floor diaphragms may interfere with the simple beam action described above. The Concrete Structures Standard (SANZ, 1995) requires that the internal in-plane paths of forces be identified and a rational analysis be used to design and detail for these internal actions. This Standard recommends that a strut-and-tie approach be used to solve these design issues.

A comprehensive presentation of this approach may be found in the seminal paper by Schlaich, Schaefer and Jennewein, (Schlaich *et al*, 1987).

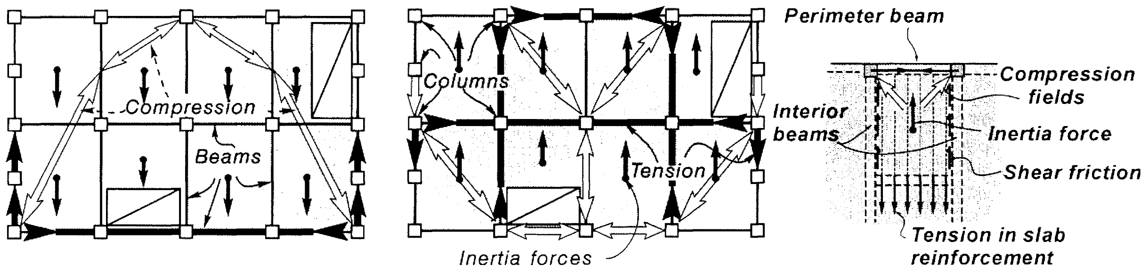
The use of strut and tie methods is not solely for determining force transfer around openings in floors. The method can be equally applied to irregular floor plans and as an alternative approach to simple beam action for the design of regular floor plans. When the strut and tie method is used, the traditional design features of flexural design and shear stress checking are not required. The designer is required to place reinforcement where there are tension fields in the diaphragm. Shear should not be a problem in the diaphragm provided that the diagonal compression introduced by the struts to nodes (see Figs. 1, 4 and 5) can be accommodated at the nodes. Whether or not such stresses can be withstood is a function of the assumed width of the compression fields and the configuration of struts and ties at a particular node. The strut and tie approach is a statically admissible method for determining possible load paths in the diaphragm.

With reference to Figure 4, the flow of internal forces is depicted as compression fields (struts) in the concrete diaphragm, in conjunction with tension chords or ties in perimeter and internal beams. Figure 4(a) is equivalent to the simple beam action shown in Figure 1, while Figure 4(b) is a more representative of a truss model. Both are equally admissible solutions for the different directions of applied inertia forces. In Figure 5, two alternative strut and tie solutions are proposed (Paulay, 1996). Here diagonal tension fields, provide the resistance in the diaphragm, with some beams acting as ties, while other beams act as struts. The tension field is generated by the reinforcement in the floor that must be present in both principal directions of the slab. However, it is suspected that for the diagonal tension field case, there may need to be redistribution of internal forces through yielding and extensive cracking of the diaphragm. This behaviour may be undesirable.

In Figures 4 and 5 the inertia forces of the floor are shown as a number of discrete forces within the floor, rather than as a set of applied forces, along one edge, as is traditionally shown. These forces where shown in this manner to highlight that parts of the slabs, including any supporting beams, columns or walls, are individual sections of floor with their own inertia forces. Local inertia forces can be estimated on a section-by-section basis. A "section" can be visualised as an area of floor bounded by beams (interior or perimeter). Each section generates its own inertia force which needs to be transferred by a viable load path in to the "strut and tie" or "truss" solution for a particular loading situation. Figure 4(c) indicates the possible paths available within a section of floor: compression fields (struts) out to the columns on the perimeter; tension reinforcement, tied back in to some part of the floor system; local struts or shear friction along the interfaces between the section of floor and the interior beams. In effect, each section of floor can be treated as a smaller strut and tie problem, which forms part of the overall strut and tie solution for the entire floor.

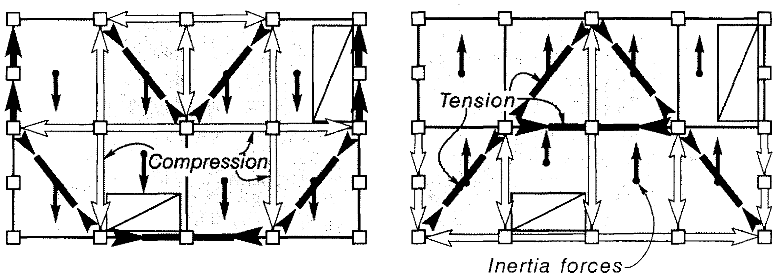
The discrete inertia forces accumulate across the diaphragm. These forces should be tied back in to the strut and tie truss mechanism either by beams or by additional bands of reinforcement within the slab ("drag bars" or "collectors" - see Figs. 6 and 7) or by the reinforcement in the slab or in the topping over precast units. Longitudinal prestressing steel in some types of precast concrete floor units can be considered to contribute. Typically for a regular diaphragm without significant penetrations, the designer can satisfy him or herself when using the "beam analogy" that shear stresses and force transfer are not critical for these panels.

The designer must ensure that these strut and tie mechanisms for transferring the accumulated inertia effects to the lateral force resisting systems are admissible. For sections of floor and supporting structures that are isolated from the main body of the diaphragm by being bounded on at least two sides by openings or for sections of floors that are effectively "wings", as mentioned in 2.1, maintaining equilibrium and load paths may require careful attention by the designer. If the diaphragm is to function as one element, tying the vertical structural components together, the integrity of the floor diaphragm must be maintained.



(a) (b) (c)

Figure 4. Load paths, using diagonal compression fields, for different direction of seismic forces (Paulay, 1996).



(a) (b)

Figure 5. Load paths, using diagonal tension fields, for different direction of seismic forces (Paulay, 1996).

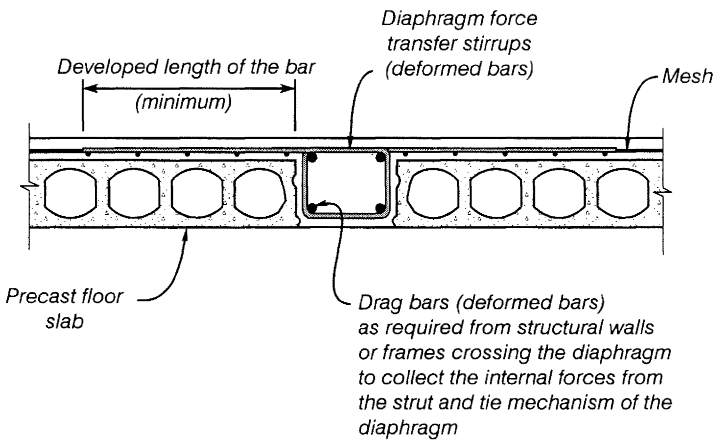
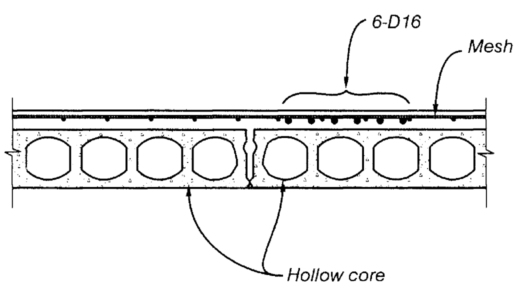


Figure 6. "Drag Bars" or "Collectors" (McSaveney, 1997).



SECTION

Figure 7. Additional "tie" reinforcement in the topping.

The versatility of the strut and tie method for dealing with irregular floor plans is demonstrated in Figures 8 and 9. The tie of the "tied arch" of Figure 8 can not be formed along the interrupted length of the perimeter frame. Therefore additional reinforcement is required along the diaphragm to provide the required tension. A typical detail of the banded reinforcement is shown in Figure 7. In Figure 9 the building has two "I"-shaped structural walls surrounded by penetrations. A strut and tie solution has been shown for transferring the inertia forces of the parts of the floor across the concrete diaphragm. Because the penetrations either side of the webs of the walls preclude the transfer of diaphragm forces directly in to the webs, "drag bars" (dark elements of Figure 9) are needed as part of the strut and tie mechanism. These drag bars receive the accumulating forces from across the floor and transfer them in to the webs of the walls. The drag bars must be anchored well in the webs. Part of the strut and tie solution permits some of the inertia forces to strut directly in to the ends of the webs (lower third of the plan shown in Figure 9).

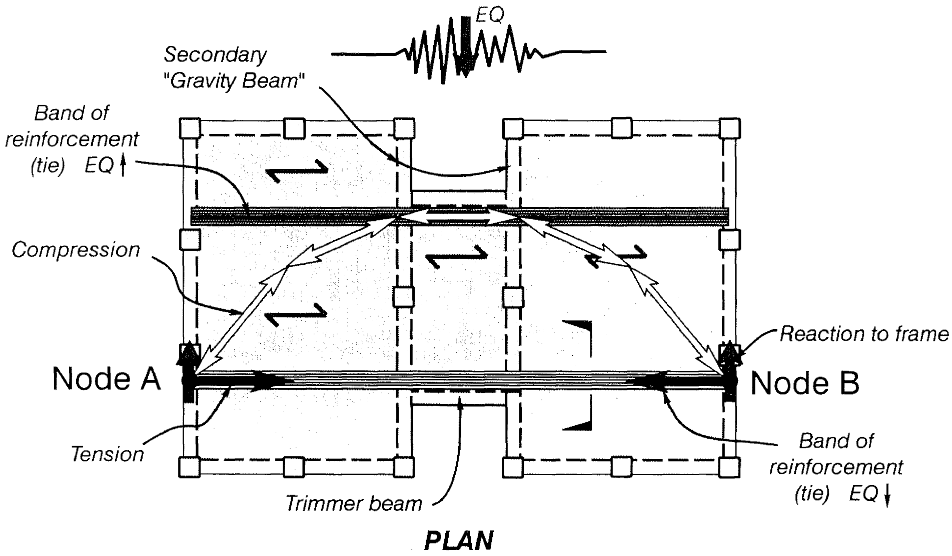


Figure 8. Provision of "tie" reinforcement in the diaphragm.

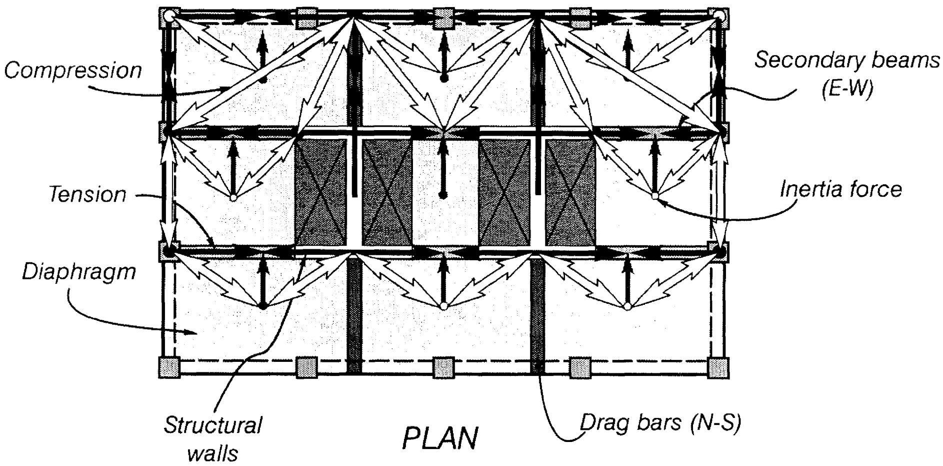


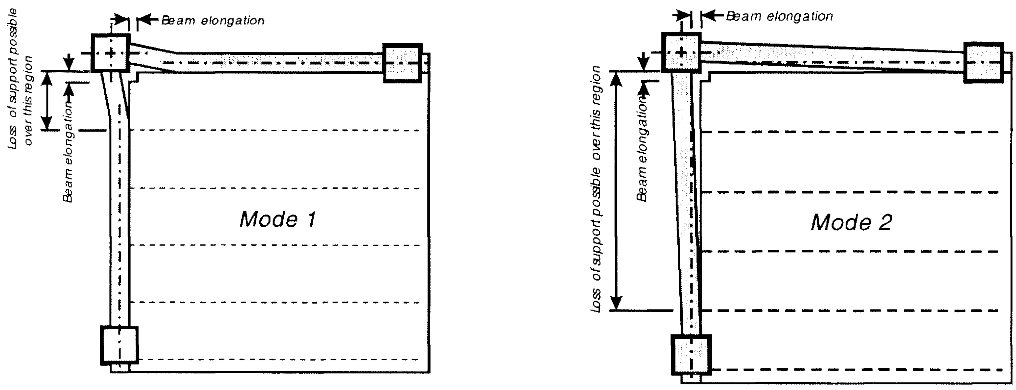
Figure 9. Strut and tie solution in combination with drag bars for a structural wall core building.

2.3.3 *Issues for diaphragms when resisting earthquakes*

During large earthquakes, ductile moment resisting frames may under go a phenomenon called "beam elongation". This occurs on the formation of plastic hinges in yielding beams. These beams lengthen at plastic hinge zones and push the adjacent columns and beams (supporting the ends of the floors) out and away from the floor units. This could result in the loss of support of gravity loads at one or more regions of floor (CAE, 1999, Fenwick et al, 1993).

Figure 10(a) describes the possible pushing out of corner columns as a result of "beam elongation": Mode 1. Figure 10(b) shows the alternative of both the column and supporting beam being pushed out by this elongation: Mode 2.

A contributing factor to the degradation of cast-in-place concrete and precast concrete floors is the deformations imposed on the floors adjacent to the beams framing or passing across the floor. Typically the floor systems are designed to support gravity. However, the double curvatures of beams may not have been envisaged in the original designs of the floors. In some cases, these compatibility deformations will lead to catastrophic failure of part or the whole of the floor (Matthews *et al*, 2003a, 2003b). Similarly, the vertical elongation of walls or the rocking of walls will impart deformations that can overstress the adjacent floors.



(a) Beam plastic hinge rotates to allow for beam elongation

(b) Entire beam rotates to allow for beam elongation

Figure 10. Particular deformation modes to deal with beam elongation.

It is felt that precast floors will be more susceptible to the loss of gravity carrying capacity than full-depth cast-in-place floors. When large beam elongations occur (10-50 mm or more), in combination with additional stresses from deforming adjoining structures, the diaphragm can be severely damaged.

A further concern in the localised damaged zones of the floor is the maintenance of composite action between the cast-in-place topping and the precast concrete units. Figure 11 shows a plan with progressive delamination damage (indicated by expanding zones of delamination associated of increasing lateral displacement of the frame) (Matthews *et al*, 2003a). Associated with this delamination is the failure of the cold drawn wire mesh reinforcement present in the cast-in-place topping. Experimental programmes have demonstrated that the wire mesh had no significant ductility and fractured at the ends of the laps with conventional reinforcement starter bars from the perimeter beams or continuity bars across interior beams lines (Herlihy, *et al*, 1996, Oliver *et al*, 1998, Matthews *et al*, 2003b).

Hollowcore support details that have been used in New Zealand are shown in Figure 12. Figure 13 shows the situation of a precast concrete unit losing support after the supporting beam has rotated relative to the floor.

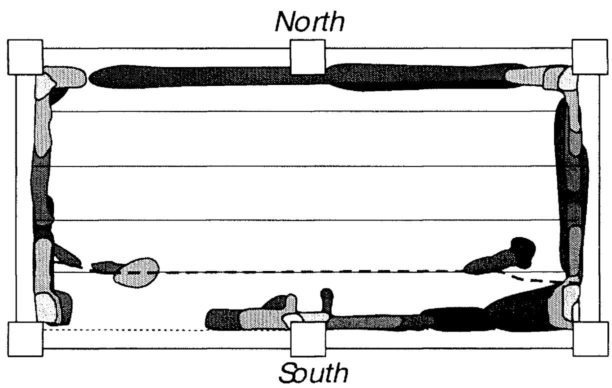


Figure 11. Plan showing delamination zones in the concrete topping (Matthews et al, 2003a).

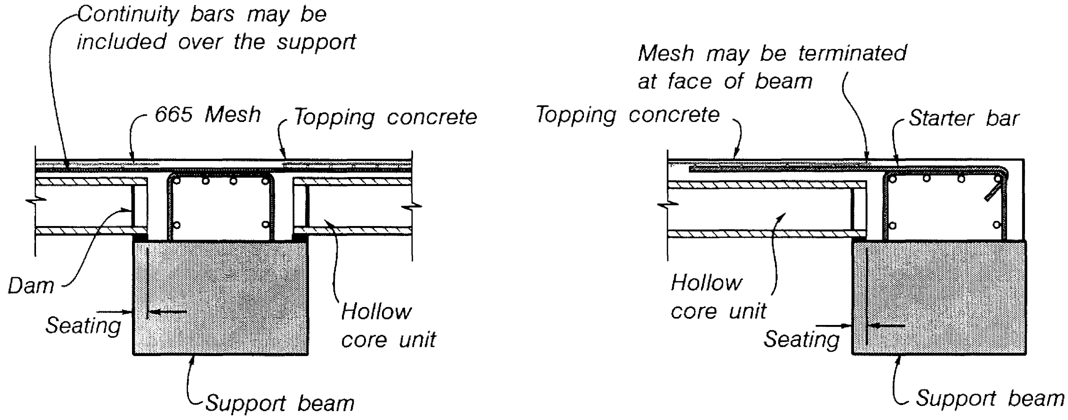


Figure 12. Typical seating details that have been used in New Zealand.

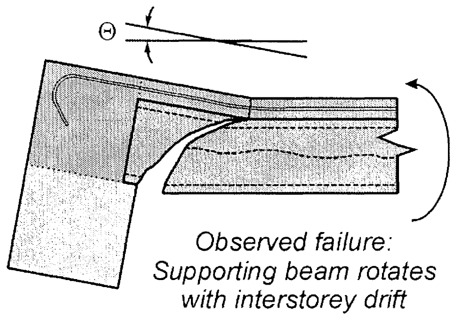


Figure 13. Loss of seating for a hollowcore unit (Matthews et al, 2003a).

The topping above the precast concrete units was susceptible to delamination through buckling of the topping. This occurred in the zone of floor that has under gone extensive yielding in tension. The same zone is subjected to compression on the next reversing cycle of the earthquake. Due to a most likely combination of significant yielding of bars (in tension) and local bond failure between the topping and precast unit, this zone buckles upwards (see Fig. 14).

This buckling destroys the ability of the support point (node) to receive the force generated by the local compression field. In effect this node becomes only capable of receiving tension, provided that the reinforcement in the topping remains intact. This sort of failure was seen in the car parking garage after the Northridge Earthquake, 1994 (Iverson *et al*, 1994).

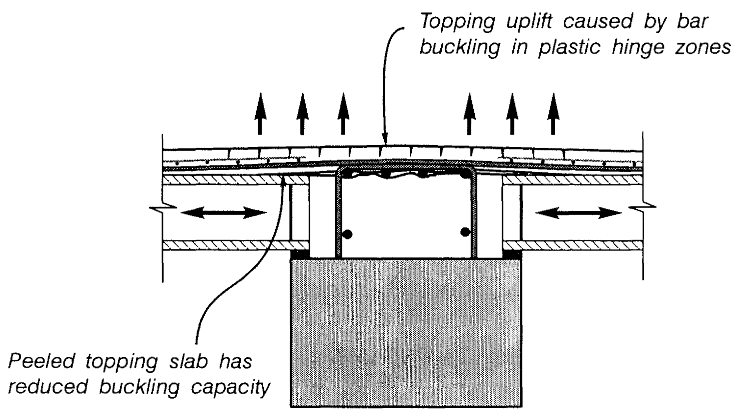


Figure 14. Buckling of the topping concrete (Herlihy et al, 1996).

One of the common assumptions of strut and tie solutions for floors is shown in Figure 15(a). This shows the corner column acting as a node of the truss would restrain a compression field. However it is unlikely that compression fields or struts can cross wide cracks or damage zones as described above. Reinforcement in the cast-in-place slab or in the cast-in-place topping over precast units has a greater likelihood of transmitting forces across such cracks by diagonal tension fields.

Figure 15(b) indicates a possible detail for generating a tension field across a cracked floor slab at a node. Reinforcement in placed through the column and anchored into the slab or topping sufficiently far to develop adequate tension. There has been some debated as to whether or not the tie bars (out of the column node) would have sufficient ductility to withstand crack widths in the order of 15-35 mm. Further, there is concern that the diagonal ties will have a component of tension parallel to the hinging beam that will add significantly to the negative moment capacity of the beam. This could lead to detrimental increases in moment, shear and axial forces imposed on the column connected to the beam (Matthews *et al*, 2003b).

It is recommended that an alternative to the detail of Figure 15(b) be used. The node that anchors the strut and tie of the floor into the supporting frame should be relocated along the supporting beam into a relatively undamaged region away from the plastic hinges of the beam. Figure 16(a) and (b) show the possibilities. In this situation the compression field in the concrete floor can still be utilised. Figure 16(a) shows the tie being formed in the mesh reinforcement lapping with starter bars. Figure 16(b) shows additional reinforcement being placed in the wide band (also see Figure 7) to add tension capacity when the mesh was insignificant to carry the tension forces alone.

A significant justification for using the relocated node, especially in the case of Figure 16(b) when additional steel is added to the floor, is that the width of slab acting as a flange can be up to half the clear distance to the next parallel beam (SANZ, 1995). If the additional reinforcement is activated as flexural steel under negative moment demand, the flexural capacity will be enhanced, in some cases, significantly (Lau, 2001, Matthews *et al*, 2003b). This can lead overloading of adjacent columns and excessive shear demand on the hinging beam. By placing the band of reinforcement midway between the parallel beams, the likelihood of appreciable contribution to the negative moment capacity is low.

Detailing recommendations are made here for maintaining the integrity of the floor diaphragms. Typically the discussion centres on the details to deal with delamination and severe cracking that occurs around columns and along beams that are part of buildings that exhibit **ductile** performance.

However, failure of floor diaphragms will not be limited to **ductile** structures as the relative rotations of supporting beams to that of the floor can induce local flexure-shear failure in the floor of a flexible, but elastically responding structure, should the relative rotations be sufficiently large (Matthews *et al* 2003a).

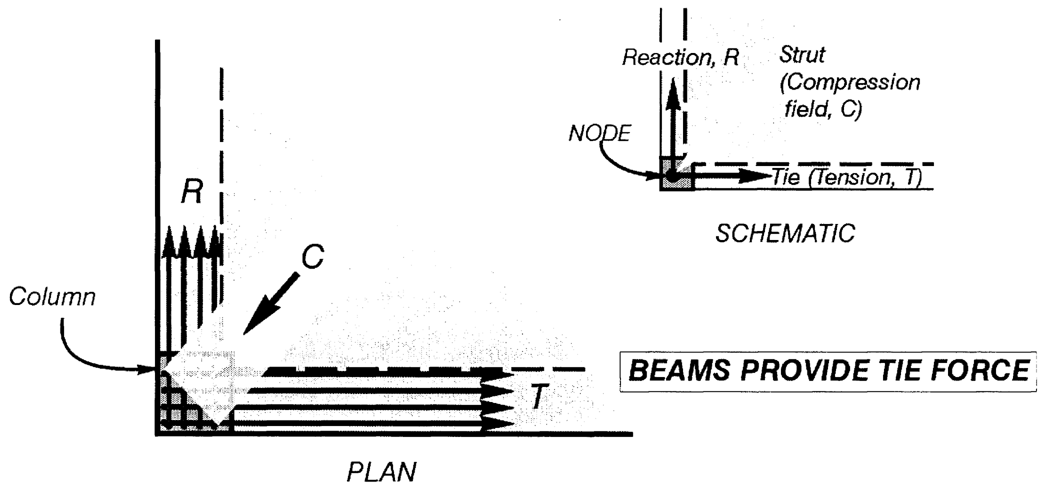


Figure 15 (a). Node restraining a strut.

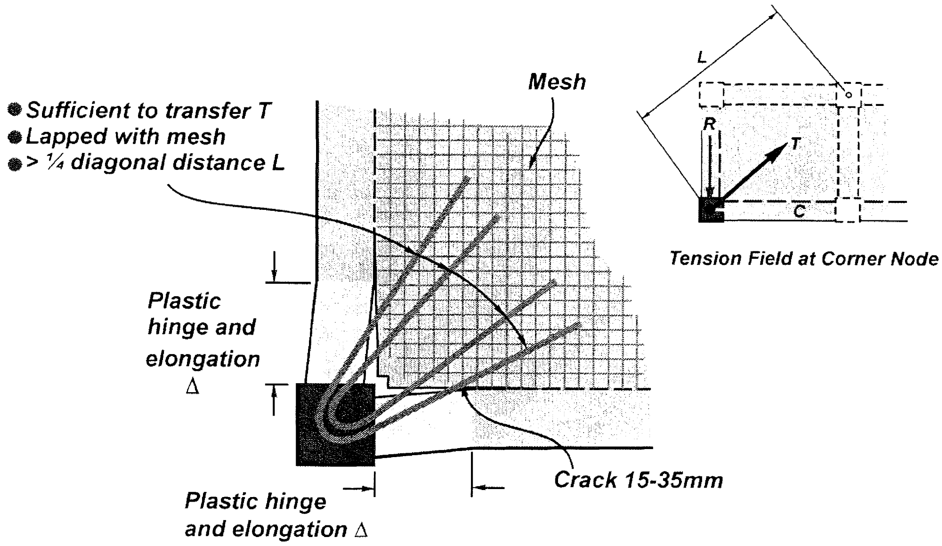


Figure 15 (b). Node restraining a tie across a cracked slab.

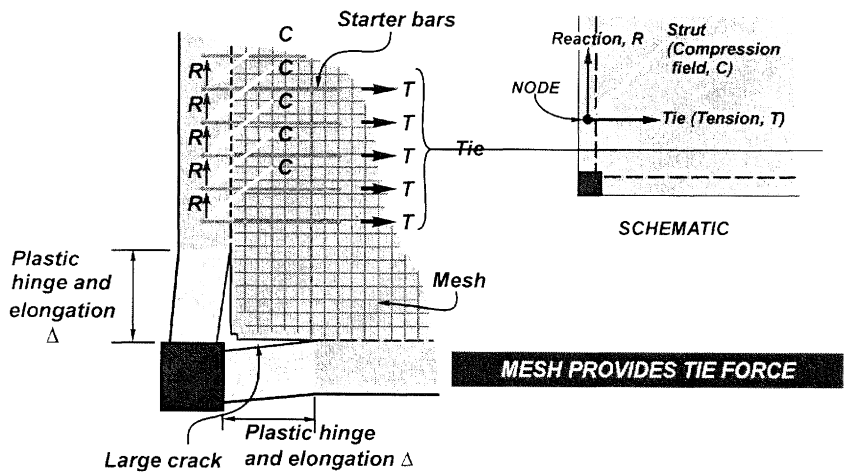


Figure 16 (a). Relocated node – mesh only providing the tie (tension).

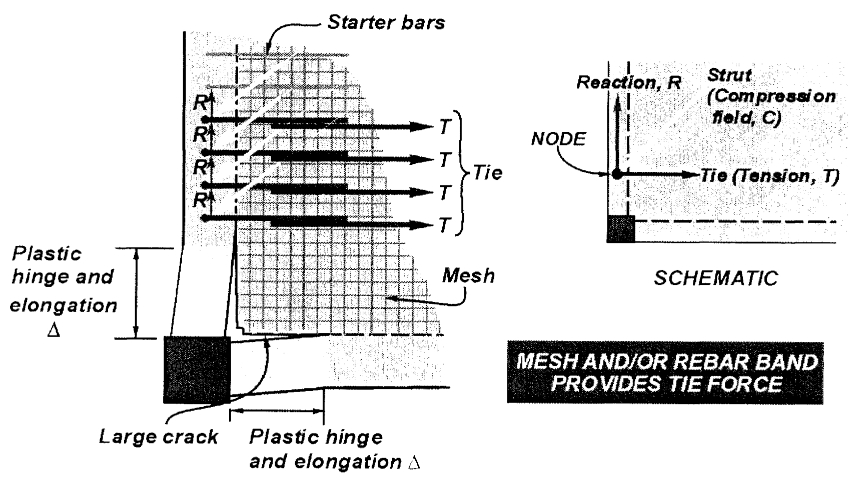


Figure 16 (b). Relocated node – mesh plus additional reinforcement providing the tie (tension).

2.3.4 *Integrity of the Concrete Structure*

There is the need to have diaphragms largely intact, with only localised damage, during a major seismic event. Such damage is typically found next to beam plastic hinge zones where the top of slabs may have been spalled away or been severely cracked. The integrity of a diaphragm must be maintained for the structural primary lateral force resisting systems to perform essentially as the designer had planned. The connections between the diaphragms and the vertical structures often prove more critical than the body of the diaphragm for satisfying design objectives. The reinforcement of such connections must be adequately anchored both in the beams, columns or walls and into the slab sections adjacent.

Typically this sort of reinforcement is lapped with the reinforcement running across the diaphragm.

The Concrete Structures Standard (SANZ, 1995) uses the term "structural integrity" when describing the avoidance of premature failure of a connection or member. These requirements ensure that certain structures have a minimum amount of adequately detailed reinforcement to maintain integrity under the most adverse conditions. This is a similar philosophy to the "robustness requirements" of the British Code, BS 8110 (British Standards Institute, 1985).

In the section on design of diaphragms, the Concrete Structures Standard (SANZ, 1995) provides details for maintaining the connection to node points of tension fields in the diaphragms, namely columns, because compressive fields cannot be relied upon in frame systems subjected to large ductility demands leading to severely cracked slabs (see 2.3.3). Figure 17 shows typical bars, placed at approximately the mid-depth of the topping slab, of exterior and interior columns respectively. To ensure adequate anchorage, these bars should extend beyond the centre of a column by at least a length equal to 1/4 of the diagonal distance between adjacent columns or the intersection of orthogonal beams around the edges of a slab panel. The amount of corner reinforcement so provided should be sufficient to generate the total tension force required at the node point (established from strut and tie analysis) and should be detailed to deal with the local ductility demand expected in that zone of cracking, as discussed earlier (e.g. breaking the bond on some length of the ties by sleeving).

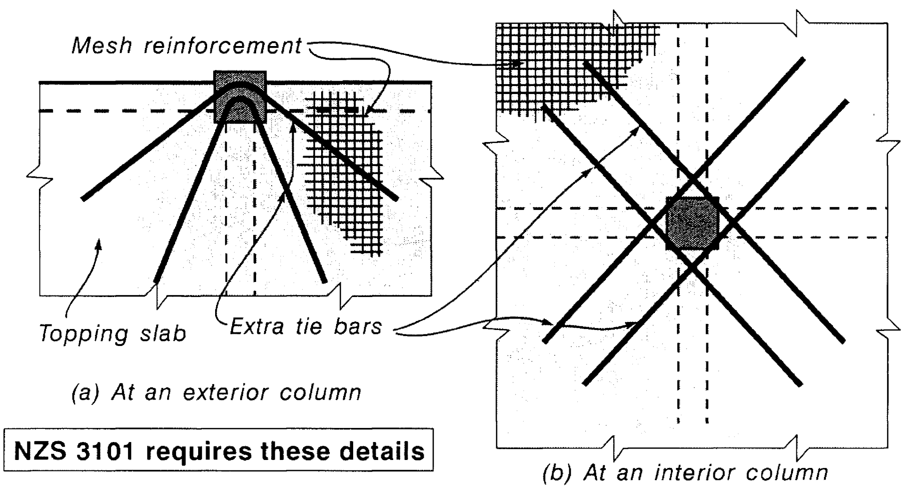


Figure 17. Reinforcement to improve performance of topping slabs in buildings with ductile frames (SANZ, 1995).

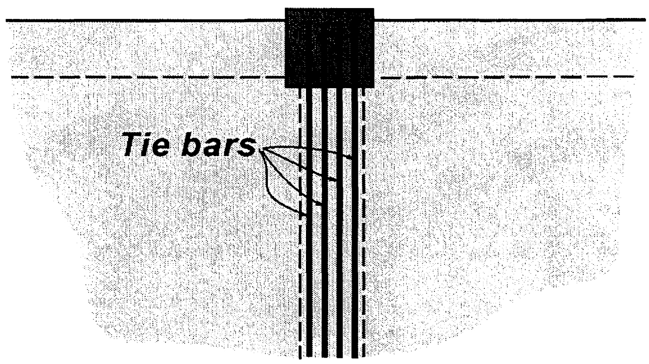


Figure 18. Alternate layout of reinforcement tie the column in to the topping slabs (Matthews, 2003b).

As discussed in 2.3.3, research by Herlihy (Herlihy et al, 1996) has highlighted the potential for toppings to delaminate from hollow core-type precast units when the units were pulled away from their supports under beam elongation. The maintenance of the composite action between topping and units and the continued support of gravity loads are intrinsically expected in the performance of the structure. Herlihy's findings suggest a need to confirm the effectiveness of the details in Figure 17. To avoid the component tension of the diagonal ties that enhances the negative moment capacity of the edge beam (see 2.2.3), the tie bar grouping at 90° to the edge beam, in Figure 18, is preferred (Matthews, 2003b).

McSaveney (McSaveney, 1997) recommended an enhanced version of the details of Figure 17 for hollow-core units. A version of this detail is shown in Figure 19. McSaveney also suggests equal applicability of this concept to double tees, rib and timber infill floors.

Figure 20, from the paper by McSaveney (McSaveney, 1997), shows details to prevent loss of support for a range of precast floor units.

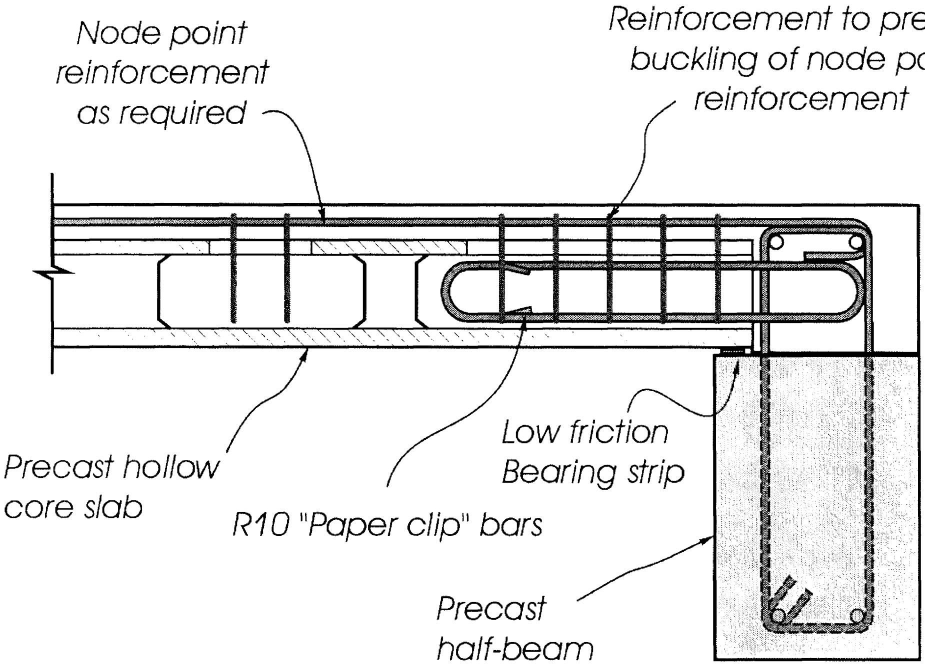


Figure 19. Strut and Tie Node Point for Hollow Core Precast Units, with Topping (after McSaveney, 1997).

2.4 Design Forces for Ultimate Limit State Earthquake Resistance

There continues to be ongoing and expanding discussion on methods for designing diaphragms for earthquakes. It appears that, globally, it is desirable to avoid plastic deformation in diaphragms. The debate centres on estimation of design actions and detailing of the load paths through the diaphragms.

National design codes tend not to deal with estimation of design forces in a clear fashion. Some are prescriptive while others do not deal with the issues at all (SANZ, 1992).

It has become clear that using of lateral forces from an Equivalent Static Analysis (ESA) for diaphragm design will grossly under-estimate the forces in the diaphragms:

- The ESA forces are a first mode approximation and under-estimate the acceleration of floors, particularly in the lower levels of the building (Bull, 1997, Nakaki, 2000, Fleischman *et al*, 2001, Rodriguez *et al*, 2002).

Figure 21 describes how the maxima of floor acceleration (determined from non-linear time history analysis) depart significantly from the ESA distribution.

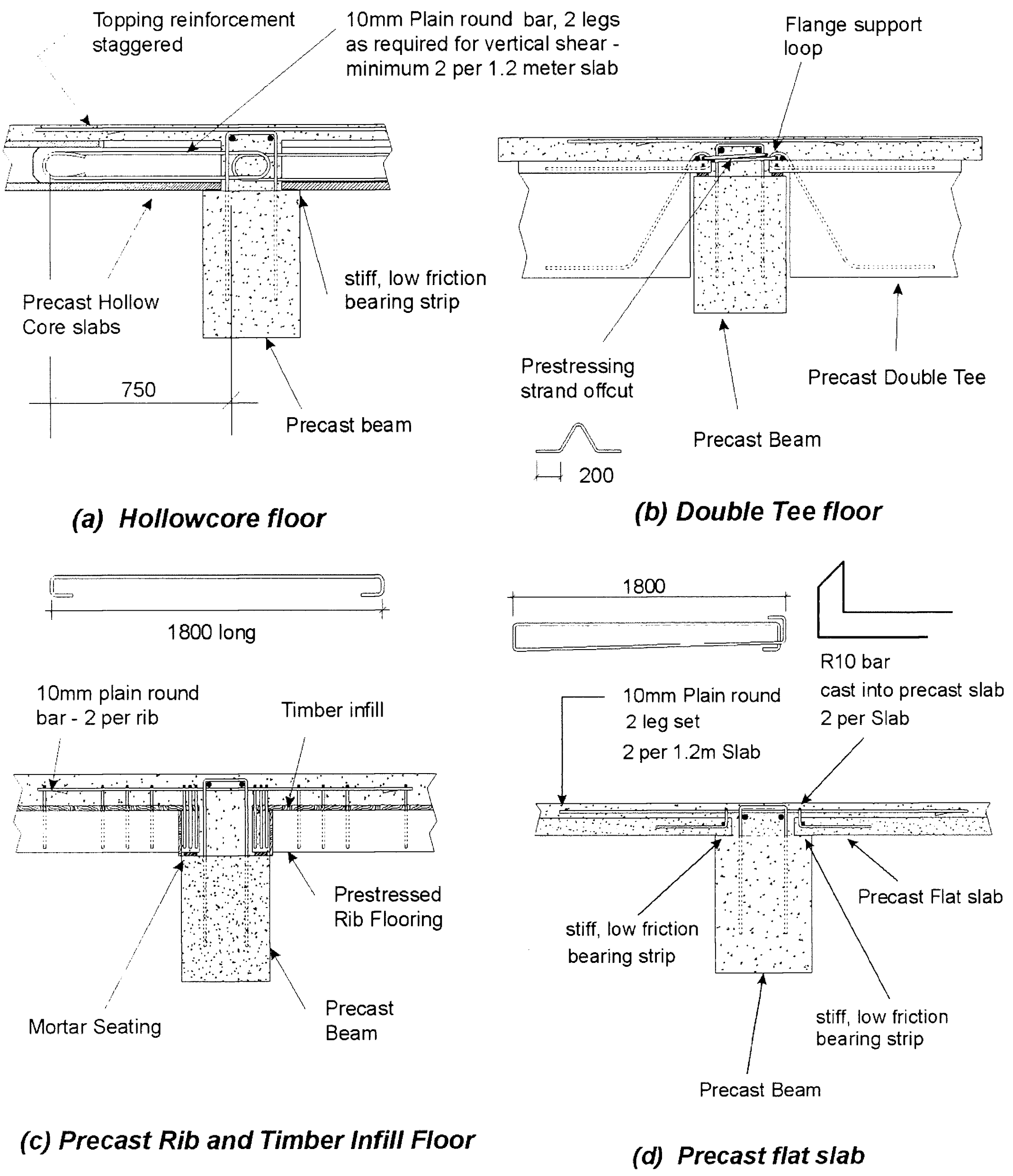


Figure 20. Loss of Support Details (McSaveney, 1997).

(a) Equivalent Static Analysis ESA) **(b) Maxima Envelope of Floor Accelerations**

Figure 21. Lateral forces on floors.

The use of ESA forces is attractive as equilibrium can be maintained and load paths visualised accordingly. The use of maximum values of floor accelerations do not permit rational following of forces across diaphragms and into the vertical lateral force resisting structures. For this reason as well, **elastic modal analysis** offers no improvement on the understanding of diaphragm-vertical structure interaction.

- The need to amplify lateral forces in diaphragms is understood. These actions arise from higher mode effects and the overstrength of a structure that is inherent and developed during inelastic response. A number of researchers and texts (NZCS, 1994, Paulay, 1996, Bull, 1997, Nakaki, 2000, Fleischman *et al*, 2001, Rodriguez *et al*, 2002) have suggested allowance for this overstrength feature.

- Interaction of "transfer" effects and inertia forces:

International research has focused largely on estimating the inertia developed at particular floors. "Transfer" diaphragms are discussed from a traditional perspective above. Often the two types of behaviour (inertia distribution and "transfer" forces) are treated as separate issues, where, in fact, the two sources of force generation are coupled and inter-related.

In resisting the inertia forces the building must deform (diaphragms and vertical lateral force resisting structures). The magnitudes of the inertias are, in part, a function of the stiffness and strengths of the structural elements. The stiffness and strengths dictate the deformed shapes that the structural elements (walls, frames, floors) will take up. This behaviour affects the response of the building, hence the inertia distribution.

Further, as the structural elements must deform, under the varying inertia distributions, the incompatibility of the deformed shapes of the various structures within the building lead to very large "transfer" forces as each structural element constrains the others in terms of the overall deflection profile. Figure 22 shows a wall-frame structure modelled as separate elements and then as a "dual" structure. The deformation patterns of the wall and frames are not compatible. Each structure attempts to constrain the other. The wall and frames are antagonistic via the diaphragms and as a result the diaphragm forces generated are potentially many times larger than the inertia forces on the diaphragms alone (Goodsir, 1985, Paulay *et al*, 1992, Stewart, 1995, Bull, 1997).

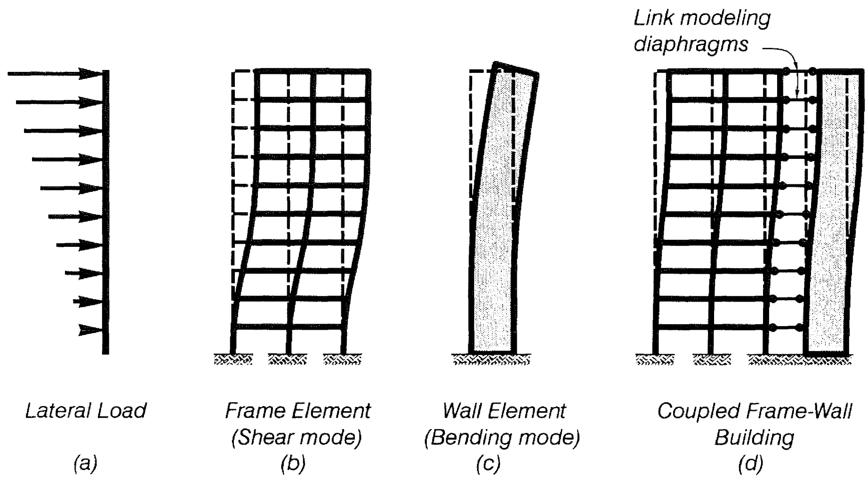


Figure 22. Deformation Patterns (Paulay et al, 1992).

At this stage, the challenge for researchers is develop a desk-top methodology to allow designers to assess the magnitudes of inertia forces and "transfer" forces, along with visualisation of load paths through diaphragms and out through the primary lateral force resisting structures.

Non-linear time histories with the diaphragms modelled appropriately is one course of action. However, such studies are not normal practise, limited to specialists in this field. Such an approach requires an estimation of the structural layout (slab thicknesses, beam layout and dimensions, connections between the diaphragms and lateral force resisting structures, reinforcement location and quantities) and provide member strength/deformation characteristics. The time history analysis of the estimated structure then permits the performance of this particular structure to be reviewed. Should the performance of the diaphragms or other elements be unsatisfactory then another iteration is required and so on. The effort and detail of modelling 3-D structures, including diaphragms, can be prohibitive. However, there are advantages in modelling in this manner as redistribution of internal actions by yielding is accounted for.

The above method is a verification method of as envisaged solution. It is an upper bound solution which does not necessarily result in the most economic solution.

Another time history method finite element method of analysis of the 3-D structures, involves generation of elements with given material characteristics (concrete and reinforcing steel). Information may be retrieved as maxima stress vectors and a truss or strut and tie solution based on these maximum stress vectors may be appropriate. However, the matching of tension stress concentrations with reinforcing steel may not necessarily capturing all the possible intricacies of the load paths that develop in real time. An alternative is to investigate the performance of the structure in real time steps and overlaying each solution for the time steps. This too is prohibitive as the data manipulation required to do this may out strip current computing capacity on typical hardware and software.

The above solution can lead to design actions. However, it has a possible disadvantage as it probably needs to be undertaken as an elastic analysis and is, in effect, another upper bound solution.

To complete the strut and tie solution for a particular floor, the inertia on the floor are required as well as the forces in the primary lateral force resisting elements, bounding and distributed across the floor. These walls, columns and beams act as boundary elements on the "disturbed" zone which is the floor plate. The distribution of the floor inertia must be in equilibrium with the forces at boundary elements, at any point in time.

A current further issue is torsion about the vertical axis of the building and the effect on the distribution of forces in the diaphragms.

Consider Figure 23(a), based on a possible equivalent static analysis (ESA). The forces in the walls (1,500 kN and 500 kN) are generated to resist the lateral forces of that floor (2,000 kN) applied through the centre of mass of that floor. The six columns of the frames produce 250 kN each as part of the force couple to resist the induced torsion as the location of the centre of resistance does not coincide with the centre of mass. These column forces are normally not traditionally accounted for in the diaphragm design.

The floor is partitioned in to 12 zones, each having inertia. 75% of the inertia is distributed to the left wall and 25% to the right wall as indicated by the rectangular regions (see Fig. 23(a)). In this simplified example, this distribution matches the forces applied to the walls (being seen as steps in the shear force diagrams for the vertical elements). It is quite probable that because of transfer effects that force couples between the frames and between the walls, respectively, will form in response to the vertical structural systems constraining independent lateral deformations. Overlain on this state will be the inertia force distribution which will, in turn, influence the forces in the vertical structure. However, be these additive, these two phases (transfer and inertia) can not be analysed independently as each phase affects the other, as alluded to above.

Figure 23(b), shows the completed solution for this direction of attack. A compression field of varying intensity develops from the "top" of the floor to the lower end of the left wall. Smaller compression fields form on the right side of the floor. The "top" parts of these struts are balanced, in part, by the forces in the columns. The components of the struts at 90 degrees to the frames are balanced by ties in the floor, as part of a truss mechanism over the whole floor.

The struts reaching the walls are balanced parallel to the walls by the forces in the walls (which increase from one end to the other, as indicated by the stepped chart next to the wall). The components of the struts at 90 degree to the wall are balanced by ties in the floor, linking in with the rest of the truss.

The diagonal struts and ties in a grid, along and across floor, are akin to the "truss model" visualised for beam-column junctions (Paulay *et al*, 1992, SANZ, 1995).

If the beam along the frame at the "bottom" of the floor is used as a tie for the skewed arch (Fig. 23(b)), as indicated by the chart parallel to the beam, the tension drops away in the tie as the column forces contribute to balancing that tension force.

The floor plate needs to be in equilibrium with forces at the boundary elements and the internal flow of forces (both "transfer" and inertia).

Other features of this solution that need to be noted are:

- The shaded zone at the "bottom" of the floor has to be tied back in to the compression fields of the floor (Fig. 23(b)) in a "suspension bridge" manner.
- The varying intensity of the diagonal struts arriving at the left wall demonstrates that firstly, "drag" or tie reinforcement is required in the wall to introduce the forces from the floor in to the internal truss mechanism of the wall (Fig. 23(b)). This tie reinforcement must be in the region of concrete common to both the floor and the wall, that is, the nodal zone formed by the junction of the floor and the wall. Designers have put this "drag" steel in the slab next to the wall, trying to engage the "starter bars" (distributed shear friction reinforcement along the wall-to-floor junction) that typically project out of the wall. This option will be ineffective as the distortions of the floor that would be required are inadmissible in terms of deforming the concrete slab. Secondly, on the point of starter bars, for many years designers have detailed **uniformly** distributed reinforcement along the junctions of the walls and floors and believed that that was sufficient. As the forces in the floor maximise at the end of the wall, the use of evenly distributed bars may be inadequate.

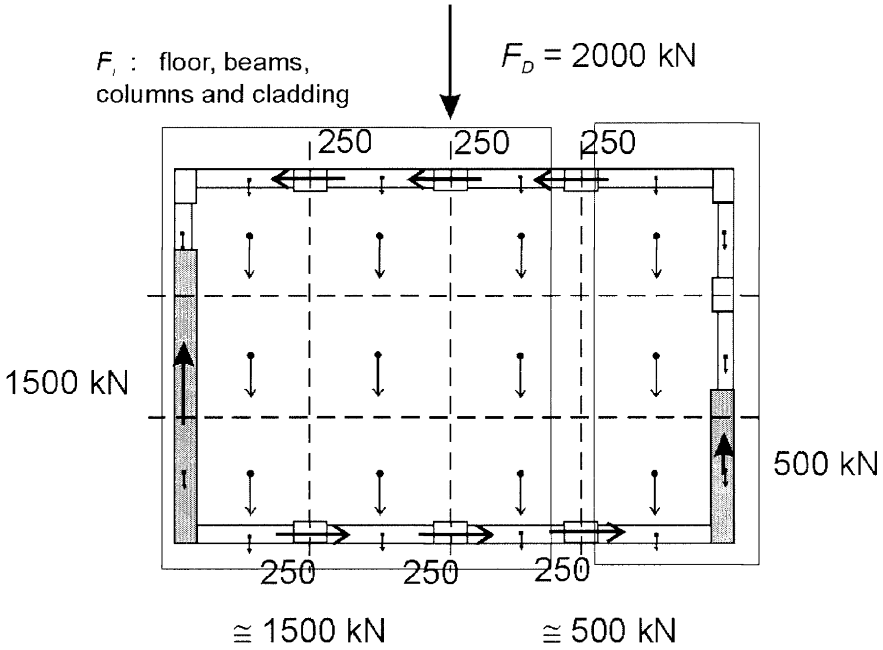


Figure 23(a). Assumed inertia, wall forces and column forces on a floor.

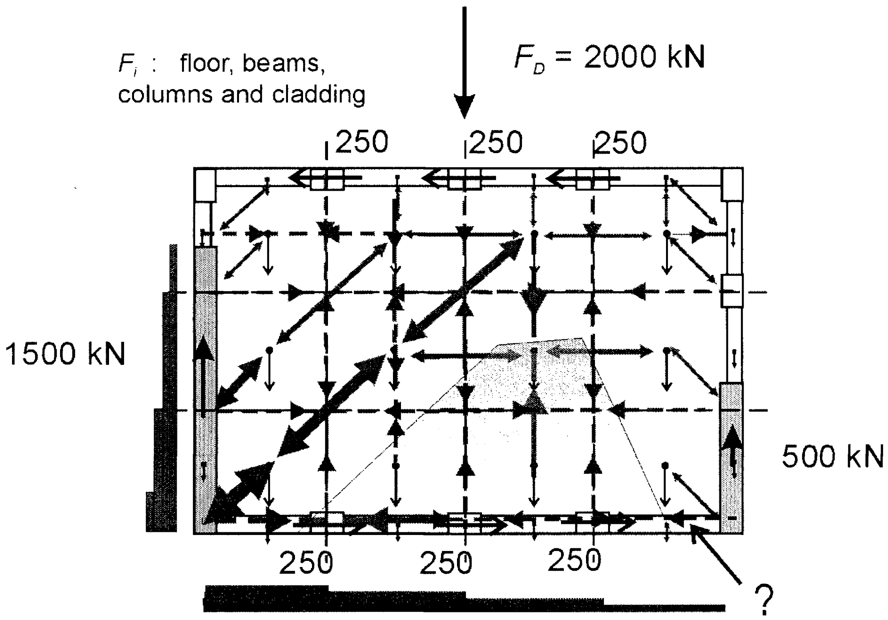


Figure 23(b). Assumed inertia, wall forces and column forces on a floor.

3 CONCLUSIONS

The deep beam analogy is a traditional method for designing diaphragms. However, there is concern over the inability of this method to clearly indicate all the critical locations of forces, especially for floors with penetrations and irregular floor plans or floors that receive localised damage during major seismic events. Designers are recommended to consider the use the "strut and tie" method of design when confronted with these difficult issues.

Detailing recommendations are made for maintaining the integrity of the floor diaphragms. Particular emphasis was placed on the use of precast concrete floor systems. Designers need to ensure that the load paths across and out of the floors are detailed to deal with delamination of toppings on precast concrete units and the severe cracking that occurs around columns and along beams that are part of buildings that exhibit ductile performance.

However, failure of floor diaphragms will not be limited to ductile structures as the relative rotations of supporting beams to that of the floor can induce local flexure-shear failure in the floor of flexible, but elastically responding primary lateral force resisting structures.

Much of recent research on the estimation of the forces to be applied to diaphragms has focused on the inertia forces. The effects of deformation incompatibility between primary lateral force resisting structures causing large forces/stresses in the diaphragms are recognised. However, separate review of inertia and transfer effects, as visualised in the past, is inappropriate as these two sources of forces in diaphragms are related. This interdependency of the inertia forces and the "transfer" forces has not been addressed in the literature to any great extent and requires further investigation. All diaphragms exhibit inertia and transfer effects, to varying degrees.

Equivalent Static Analysis under predicts the magnitudes of forces in diaphragms as does the modal response spectrum analyses. Allowances for higher mode effects and inherent overstrength on the structural systems must be made. Non-linear time history studies can give representative outputs. However, the diaphragms must be modelled and this requires considerable effort. Non-linear time histories involving modelling of structural elements may require a number of iterations to produce an effective structural layout. If a finite element analysis is used then either real time steps need to be investigated or a degree of caution needs to be employed if maxima of stresses are used.

A focus of future research should be towards a desk-top study using a pseudo-Equivalent Static Analysis that produces reasonable estimates of the "transfer" and inertia forces. This approach has the advantage of being transparently in equilibrium, permitting load paths to be visualised and detailed through and out of the diaphragms and in to the load paths of the vertical lateral force resisting structural systems.

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