Floor diaphragms – Seismic bulwark or Achilles' heel

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ABSTRACT: Floor diaphragms form a critical component of seismic resistant buildings, but unfortunately, in the main their analysis and design in New Zealand leaves much to be desired. No worse example exists than the CTV Building in Christchurch. Despite the critical importance of diaphragms, there is a paucity of code provisions and design guidance relating to them.

Using generic examples, the author describes a number of common diaphragm design deficiencies. These include diaphragms where valid load paths do not exist; diaphragms where the floors are not properly connected to the lateral load resisting elements, diaphragms that lack adequate flexural capacity and where re-entrant corners are not properly accounted for, and transfer diaphragms into which the reactions from the walls above cannot be properly introduced or transmitted.

Three main types of diaphragm action are discussed – 'inertial,' 'transfer' and 'compatibility.' These are, respectively, the direct inertial load on a floor that must be carried back to the lateral load resisting elements, the transfer forces that occur when major changes in floor area and lateral load resisting structure occur between storeys, and the compatibility forces that must exist to force compatible displacements between incompatible elements, such as shear walls or braced frames and moment frames, or as a result of redistribution.

The author presents a simple Truss Method that allows complex diaphragms to be analysed for multiple load cases, providing accurate force distributions without the multiple models that rigorous Strut and Tie methods would require.

1 INTRODUCTION

With regard to the seismic resistant design of buildings in New Zealand, a great deal of attention is given to the *vertical* lateral load resisting elements (moment frames, shear walls and bracing) at university, in the code compliance documents and standards, in texts, guides and seminars, and in design office and Building Consent Authority practice.

Unfortunately, the equally critical *horizontal* lateral load resisting elements (diaphragms) are largely ignored in NZ, and the design effort that does go into them is usually woefully deficient. This has to change, immediately.

The floor diaphragms hold the building together, and prevent all of the vertical elements from buckling. They are the repository of most of the weight in a building, and unless the diaphragms can sustain the seismic forces they are subjected to, and transfer those forces properly to the vertical lateral load resisting elements, those vertical lateral load resisting elements may as well not be there.

Several factors seem to conspire in NZ to prevent diaphragms being treated with the respect they deserve, and being given the significant design effort they require. They receive little attention in the lectures and notes at university, and they appear to receive little attention from academics, perhaps because of their large size and hence the difficulty in testing them. They receive little coverage, if any, in engineering texts, they receive only slight and partial coverage in the structures standards, the usually valid assumption of being 'infinitely rigid' may lead to the assumption of 'infinite strength' as well, they are very difficult and time consuming to analyse and design properly, except for the most trivial of configurations, and the fees and time required for that analysis and design are simply not there, at present, at least.

This paper aims to change that.

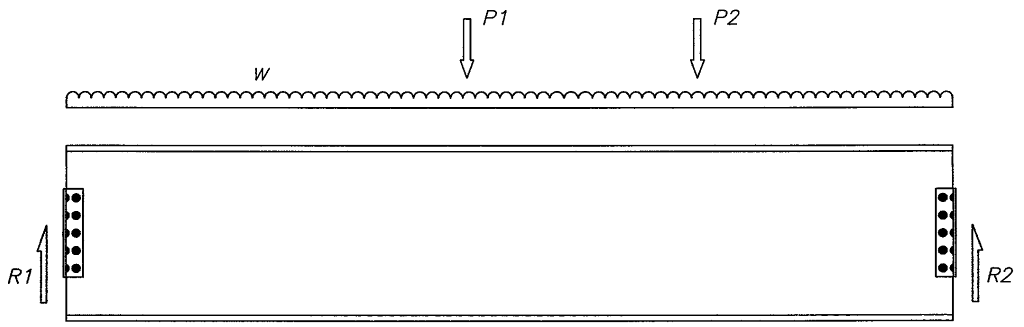

Figure 1

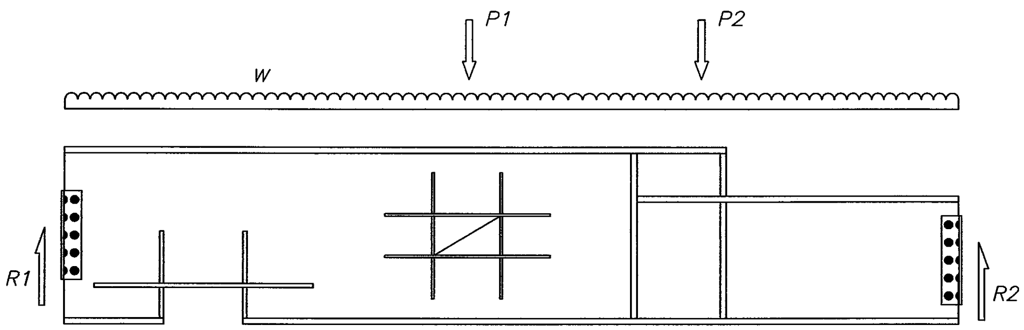

Figure 2

Figure 1 shows a simply supported steel beam subject to gravity loading. In simplistic terms, the web resists the shear forces, the flanges resist the bending actions that must exist for equilibrium, and robust connections ensure the loads are transferred to the supports.

Figure 2 is a more complex beam, with a reduction in section, a notch and a web opening. However, all of these items appear to have been accounted for through appropriate strengthening following detailed rational analysis and design.

One likes to think that standard design office practice, supported by available design guides, would ensure that both these steel beams would be rigorously analysed, designed and detailed to produce safe, robust structures.

Now imagine that these beams are turned on their side, increased in dimension by a factor of ten or more, and converted to reinforced concrete. In other words, they become floor diaphragms. All too often, the design paradigm changes completely. A rudimentary check of the overall shear strength *may* be made, but most of the other actions will be ignored. In many instances, there will not even be robust connection of the floor diaphragm to the very lateral load resisting elements that are put there to support the diaphragm in the first case.

This paper aims to draw the profession's attention to the need for the proper analysis and design of diaphragms. Through examples, it shows how the viability of a diaphragm can often be assessed very quickly, simply by inspection, and describes a simple but effective truss analysis method that can handle what is often an extremely complex analytical problem with relative ease.

The emphasis is on cast in-situ reinforced concrete slabs, pre-cast concrete floor units with in-situ toppings, and composite steel metal decks. The reader should refer to standards such as NZS 3101 Concrete Structures Standard and NZS 3404 Steel Structures Standard, and relevant papers, such as those produced by Bull (Bull, 2004), Cowie *et al* (Cowie *et al*, 2013), (Cowie *et al*, 2014) and NIST (Moehle *et al*, 2010), (Sabelli *et al*, 2011), for guidance on the detailed design of the reinforced concrete and structural steel components that form a typical diaphragm.

A word of caution, however. As with all engineering standards and design guides, do not accept the standards and guides relating to diaphragms as 'gospel.' Check the content and the details proposed against sound engineering principles and (un)common sense. Just because "it's in the standard" (the fallacious *argumentum ab auctoritate*) or "everyone else does it this way" (the fallacious *argumentum ad populum*) does not make it right. The only thing that matters is the actual performance of the diaphragm, especially under extreme seismic loading.

Untopped diaphragms with discrete, usually welded, connections, such as site welded 'spider plates,' are not considered in this paper. They are, in the opinion of the author, a concept that should not be used to resist seismic loads, or even significant shrinkage or thermal effects.

This paper draws the attention of the reader to the types of loads that various types of diaphragm are subjected to, with appropriate references, but does not cover the derivation of these loads in detail, because space does not allow the subject to be covered properly.

Space also prevents the inclusion of as many diagrams as would be desirable. Please pay particular attention to the details of the text – there are many important points which may not be highlighted through the inclusion of an accompanying diagram.

The author is indebted to Chris Thom, PhD, for reinforcing an already developing appreciation of the importance of good diaphragm design, back in the mid-1980's. Chris carried out the preliminary design for a proposed multi-storey hotel in Wellington. Reinforced concrete shear walls provided the lateral load resisting system to the tower, with a four storey podium structure surrounding the bottom of the tower. The tower floors were to be precast concrete units with in-situ topping, but the podium slabs had to be cast in-situ, because of the magnitude of the transfer diaphragm forces.

The central shear core had two C shaped shear walls, with the floor slabs attached to the full length of one side of the web of each shear wall. The force transfer from these walls at the top floor of the podium was so great that the in-situ floor slab in the vicinity of the shear walls had to be 600mm thick, with numerous drag bars projecting from both ends of each shear wall web, to complement the force transfer through the side of the web.

The thickness of the in-situ slab in the top floor of the podium reduced to 200mm at the surrounding podium walls, and the variation in thickness across the slab was based on an assessment of both strength and the propensity of the slab to buckle.

All of this was done without specific code provisions or papers such as this. It simply resulted from a competent engineer applying the principles of equilibrium, a load path and resistance to buckling.

2 EXAMPLES OF POOR DIAPHRAGM DESIGN

Poor diaphragm design featured significantly in my Open Letter (Scarry, 2002), particularly in the First Version, and several examples were shown. At the time, I had not developed the Truss Method, and so the proper assessment of the internal actions in these example diaphragms was difficult, but by inspection, the diaphragm designs simply could not be right. Large openings surrounding shear walls or irrational geometries showed that viable load paths and dependable performance could not be achieved.

Over 60% of the fatalities in the Christchurch earthquake on 22 February 2011 were due to the collapse of one modern building – the CTV Building. A major contributory factor to the collapse was the woefully deficient floor diaphragms. The building was almost completely dependent on the shear walls of the North Lift Core for its strength and stability, but there was no effective connection of the floors to that core.

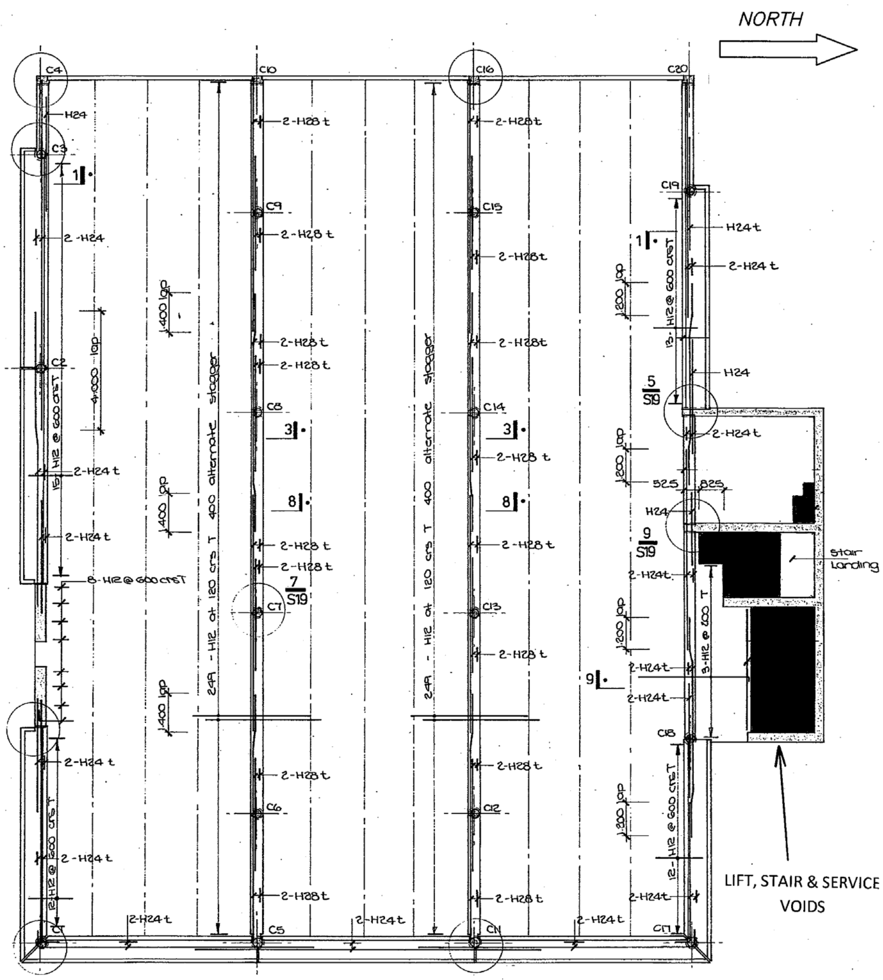

Figure 3. Typical Floor Plan of the CTV Building, Christchurch

Figure 3 shows a typical floor plan. In the north-south direction, there were originally no drag beams or even drag bars to connect the floors directly to the walls. Only the western bay of the core had direct connection to the north-south direction walls, and the 'drag angles' that were retrofitted to the eastern bay did not extend into the main body of the floor. The situation in the east-west direction was no better. The building was completely dependent on the connection of the floors to the rear north wall of the core. But large openings for the lifts, the stair and services (shaded in black) meant that the length of floor connected to this wall was very limited, and even then, the floor system was not capable of resisting the resultant 'cantilever' bending actions in the diaphragm.

The main load carrying structure of the CTV Building was described as a 'shear wall protected gravity load system.' This was a common form for many mid-rise Christchurch commercial buildings designed in the 1970's and 1980's, although these other buildings at least had the shear core within the footprint of the building.

Unfortunately, often little attention was paid to the diaphragm design and invariably, low ductility welded wire mesh in the floor topping was used as the main diaphragm reinforcing. After 22 February 2011, some of these buildings had to have large angles bolted in place to connect the floors to the shear core, simply to make the buildings safe enough to demolish.

Yet despite weeks of streaming evidence from the Canterbury Earthquakes Royal Commission (CERC) regarding the CTV Building, and the written findings, at least one multi-level structure of the re-build was designed and constructed with this same fundamental defect – inadequate connection of the floors to the lateral load resisting elements. In the building the author has dubbed 'Son of CTV,' in one direction the lateral load resisting elements are located on the outside of external stair wells, without any effective load path from the main floors to the lateral load resisting elements. The deficient design was reviewed and consented, and this serious deficiency was only discovered by good luck during construction through some observant engineers not involved with the project. Hopefully, the retrofit that followed is more effective than that applied to the CTV Building.

In the following examples, the gravity load resisting structure has generally been omitted for clarity, and the indicated seismic loading is diagrammatic only, and would obviously vary to account for the +/- 0.1b eccentricities required by most loadings standards.

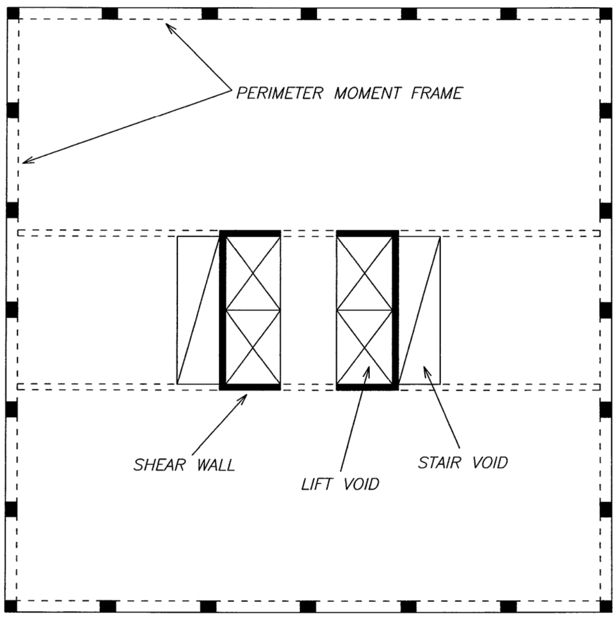

Figure 4

Figure 4 shows the all too familiar case of large openings around the shear walls. Unfortunately, in this case, the openings are very large and on both sides of the shear wall webs, so that the shear walls have little effective connection to the building in one direction. Another problem is the fact that the webs have no connection to each floor, and web slenderness is a serious issue given the large length of the web relative to its thickness.

Concrete structures standards continue to assume that wall elements are restrained at each floor level, and this is clearly often not the case.

Figure 5 was a concept for an extension to a large building, driven by the architect stating he did not want bracing at the far end of the extension. It was proposed that each floor would cantilever horizontally off the strut and the connection of the floor topping back into the main building. If this was a vertical shear wall, the author believes it would have been dismissed outright, and such forms appear in all sorts of texts and guides as what *not* to do in a seismic zone, but because it was a diaphragm, in this instance it was considered viable. The engineer tasked with implementing this design had to fight to correct this, and at least ensured that this extension got adequate bracing at the far end.

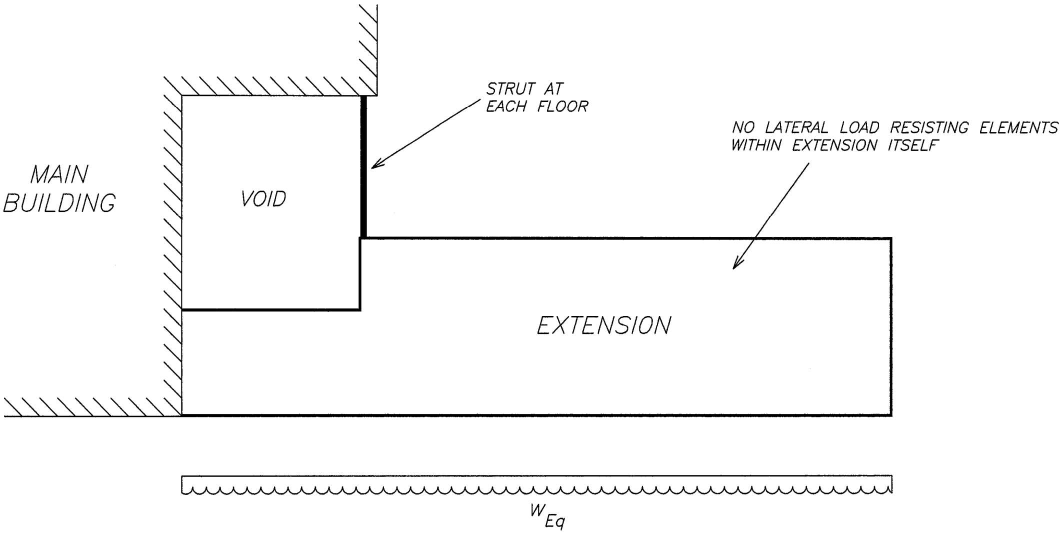

Figure 5

Ideally, buildings in seismic zones should have a simple, stocky rectangular floor plan, with a symmetrical arrangement of vertical lateral load resisting elements. Figure 6 shows a not uncommon but undesirable L shaped floor plan. L shaped diaphragms are very prone to 'tearing' at the re-entrant corner, even if our conventional treatment of seismic load and our analysis methods cannot always identify the actions that would cause this tearing. Such re-entrant corners should at least be strengthened with 'stiffeners' in *both* directions, in the form of beams extending well into the building.

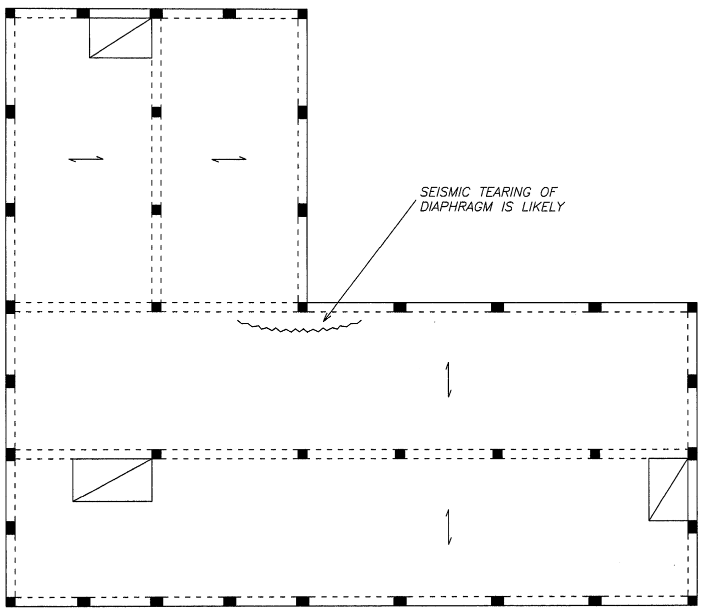

Figure 6

Figure 7 is not such a bad overall concept in itself, but when submitted for a Building Consent, a simple check showed that the diaphragm-wall interface (simply the topping thickness over the length of each wall) was overstressed.

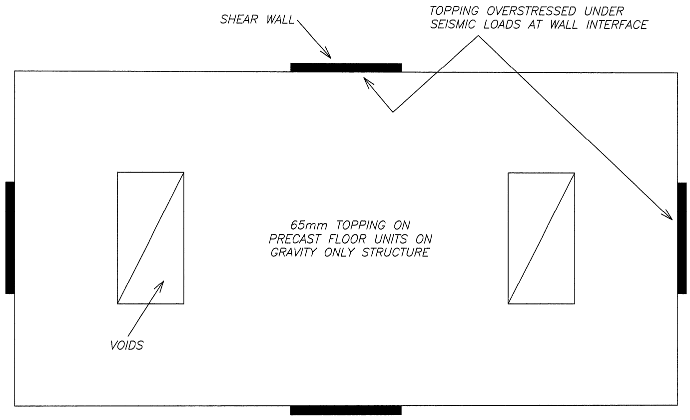

Figure 7

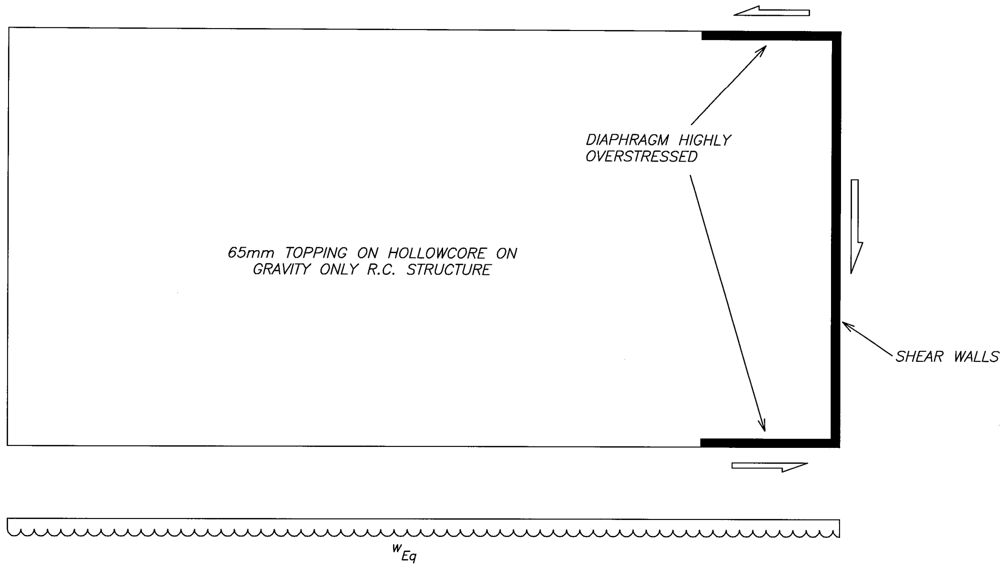

Figure 8

Figure 8 shows a very large suspended concrete floor, which had a lightweight steel structure over the top. The gravity system included precast concrete floor units with 65mm topping and low ductility 665M mesh, supported on a gravity only reinforced concrete frame structure. Shear walls at one end were intended to resist all of the seismic loads. The seismic actions on the walls were determined through an ETABS analysis. Clearly, the floor must 'cantilever' off the three shear walls, but an engineer 'designed' the diaphragm as 'simply supported,' for a bending moment of $wL^2/8$, and a shear force of $wL/2$! The reviewing engineer's quick check showed the shear stresses in the diaphragms adjacent to the 'flange' walls were almost twice the maximum allowed. After much wasted effort in trying to get the design firm to solve the problem in a rational manner, the reviewing engineer and the author were engaged to determine how many shear walls needed to be added to the opposite end of the building to make the diaphragm work. The author developed the first version of the Truss Method for this purpose, and it allowed alternative strengthening arrangements to be quickly checked.

The Truss Method was described by a colleague to another consulting engineer, to assist that engineer in designing a large L shaped floor diaphragm. This consulting engineer duly analysed the diaphragm, and the results showed very large axial force components at the re-entrant corner of the L. The consulting engineer then wished to ignore these large forces, because he considered them 'errant.' They were not – they were demanded by simple equilibrium.

3 TYPES OF ULTIMATE LIMIT STATE LOAD DIAPHRAGMS MUST RESIST

3.1 Inertial Loads

All suspended floors will be subject to direct lateral acceleration and hence lateral inertia forces during an earthquake, including diaphragms also subject to transfer and compatibility forces.

These accelerations and forces can be derived through:

1. The Parts and Portions provisions of the appropriate loadings standard,
2. Parts and Portions as modified in Seismic Design of Composite Metal Deck and Concrete-Filled Diaphragms – A Discussion Paper (Cowie *et al*, 2013), (Cowie *et al*, 2014), and the papers quoted therein,
3. A non-linear time history analysis.

The height limitations and warnings contained within the papers by Cowie *et al* (Cowie *et al*, 2013), (Cowie *et al*, 2014), and the papers quoted therein, must be taken heed of.

The derivation of accurate inertial loads on the various floors of a multi-storey building is a difficult process. Higher order modes and peak ground accelerations mean that for the lower storeys, the floor inertial forces will be greater than those indicated by, for example, the Equivalent Static Method.

Great care must be taken to ensure that the inertial forces for each floor are adequate, but not overly conservative.

Papers by Bull (Bull, 2004) and Gardiner *et al* (Gardiner *et al*, 2008) provide additional discussion on the difficulty in determining the inertial forces acting on floor diaphragms.

These inertial diaphragm forces cannot simply be assumed to act through the centre of mass of the floor. Loadings standards typically mandate that +/- 0.1b eccentricities must also be allowed for. This will involve having to vary the loading along the floor for each loading direction and eccentricity, unless rational analysis shows that an appropriate scale factor can adequately cover these effects.

Diaphragms will, by their very nature, be 'elastic' ($\mu = 1$) or 'nominally ductile' ($\mu = 1.25$) structural elements. Therefore, in accordance with AS/NZS 1170.5:2004 Section 5.3.1.2, the action set must comprise 100% of the earthquake actions in one direction combined with 30% of the earthquake actions in the orthogonal direction. A review of the member actions for each direction of loading alone may show that a suitable scaling factor (typically a 10% increase) would allow each principal direction to be considered separately, and still comply with the requirements of Section 5.3.1.2.

3.2 Transfer Forces

Transfer forces occur at floors where there are changes in the layout of the lateral load resisting elements. This is usually accompanied by a change in the shape and size of the floor plates.

The most well-known type of example where significant transfer forces develop can be seen in Figure 9, in which a tower is surrounded by and connected to a larger podium structure that is significantly stiffer than the tower. Instead of cantilevering off its foundations, the tower receives most of its stability against lateral loads from the top floor(s) of the podium, with a 'kick back' at the base. Very large floor forces can develop, and these must be transferred from the tower into the top floor diaphragm(s) of the podium, and carried out to the stiffer lateral load resisting elements (usually shear walls) of the podium.

For ductile or limited ductile tower structures, the transfer forces must be based on overstrength values.

For diaphragms where shears are required to be re-distributed between vertical lateral load resisting elements, a transfer of forces through the diaphragm must also occur.

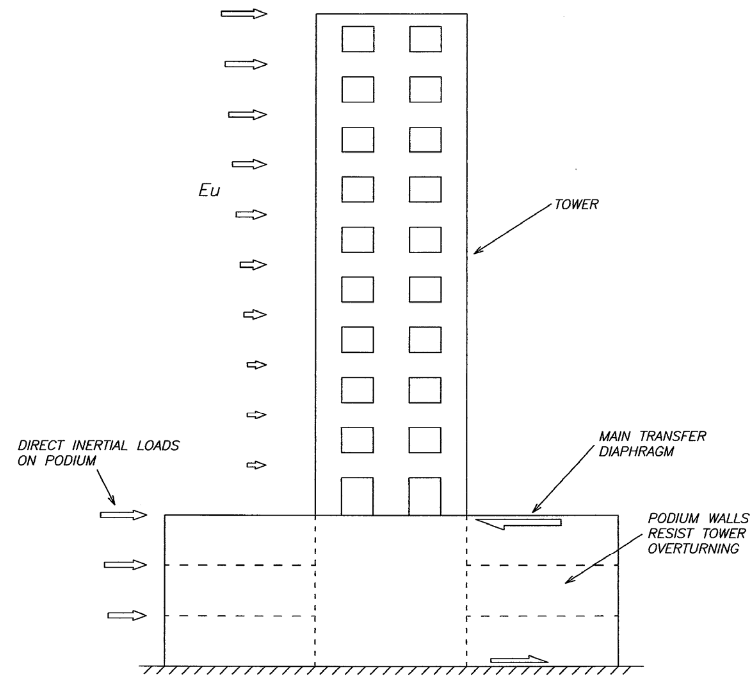

Figure 9

3.3 **Compatibility Forces**

Instead of relying solely on one type of lateral load resisting element, some buildings incorporate mixed shear walls and moment resistant frames or mixed bracing and moment resistant frames. This is often done to increase the strength and improve performance at the ultimate limit state, for example by reducing the risk of a storey sway mechanism developing.

In other instances, mixed systems are used deliberately to reduce the lateral deflections, for the reasons that follow.

Figure 10 shows the deflected shapes of a moment resistant frame and a cantilever shear wall under lateral loading. Inter-storey deflections tend to be greater in the lower parts of the frame, and greater in the upper parts of the shear wall. When the two structures are combined in a building and tied together with floor diaphragms, they 'fight' each other. The deflected shape of both structures is changed, and overall, deflections are reduced as a result.

One consequence, however, is that very large forces develop in the floor diaphragms as compatible deflections are imposed.

Unfortunately, whereas the rigid diaphragms usually assumed in the analysis of multi-storey buildings impose compatible displacements, they provide no information as to the magnitude of the force components in the slab required to enforce compatibility.

Determining these compatibility forces is not a simple task. Modelling a floor with triangular or rectangular finite elements will include the compatibility forces in the form of element stresses, but extracting those forces in a useful form will be difficult if not impractical.

Alternatively, the floor diaphragms at each level of the building could be included in a three dimensional model of the building using truss elements, either the non-linear lattice elements described in Section 5.3, or the simple truss elements described in the Truss Method.

Inertial loads could be distributed to all of the nodes in a floor to represent the seismic loading on the building, and the truss forces will be the resultant of the inertial and compatibility effects.

These compatibility effects will result from the elastic deformation profile. However, the problem becomes much more complex when the inelastic deflection profiles described in AS/NZS 1170.5:2004 Section 6.5 and Section 7 have to be considered. Specialist advice should be sought.

Compatibility effects will exist to some extent in all realistic diaphragms because the lateral load resisting elements will not be perfectly uniform and symmetric.

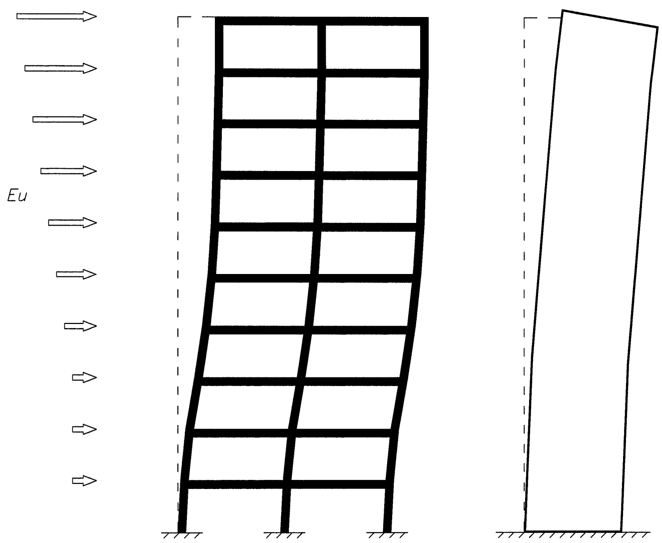

Figure 10

Tearing forces due to plastic hinge elongation in reinforced concrete moment frames with precast floors are another type of localised compatibility effect. In Christchurch, severe damage was caused to some floor diaphragms by such plastic hinge elongation.

For a discussion of this form of damage, and other related problems, refer Understanding the Complexities of Designing Diaphragms in Buildings for Earthquakes (Bull, 2004).

3.4 **Lateral Restraint Forces**

All diaphragms must provide lateral restraint to all beams, columns, walls and braces that require it.

It must not be assumed that all vertical elements are automatically restrained by a floor, especially with regard to walls. All details must ensure that the required restraint is provided, especially with respect to elements near slab openings and free edges.

3.5 **Static Forces**

Suspended floor diaphragms are of course subject to wind forces above ground level.

Often, the lower part of a building will be required to retain soil, and sometimes ground water. ‘At

Rest' soil conditions usually apply, because of the inherent stiffness of the building system.

Usually, these static retaining loads will have to be carried by diaphragms, either to transverse shear walls or other lateral load resisting elements, or across the building to oppose retained soil (and ground water) acting in the opposite direction. In other cases, lateral loads from the building will be reacted against retained soil, with these lateral loads being transmitted through the floor diaphragms.

Under earthquake conditions, seismic forces from the soil, in addition to the static retaining forces, are generated.

Sloping and offset columns can also generate very large forces in diaphragms.

3.6 **Sloping and Offset Columns**

Sloping and offset columns, such as those shown in Figures 11(a) and 11(b), can generate very large diaphragm forces, especially in high rise buildings.

Unfortunately, the large diaphragm forces required to resist the horizontal components from the inclined column force will not show up in most three dimensional computer analyses, because of the rigid diaphragms usually used to model the in-plane stiffness of the floors. Therefore, these horizontal force components must usually be determined by additional analysis.

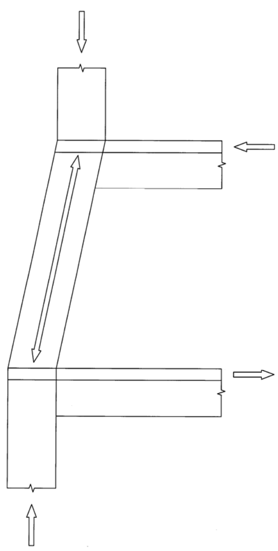

Figure 11(a)

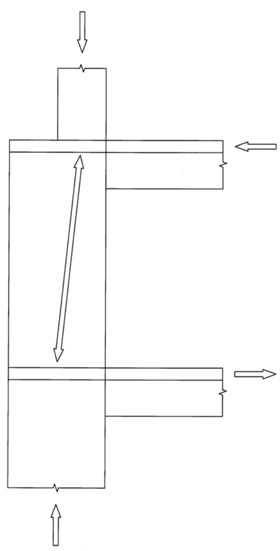

Figure 11(b)

3.7 **A Special Type of Slab on Grade**

In low rise shear wall buildings in particular, a common problem relates to the foundations of the shear walls. Often these walls carry little of the total weight of the building, hence their foundations have limited resistance to sliding, or the walls bear on piles with negligible lateral load resistance, yet the shear walls must resist the seismic loads from the whole building. As a consequence, what would otherwise be essentially a non-structural slab on grade must act as a diaphragm, tying in the shear walls and transferring the seismic shears to gravity foundations or embedded foundation beams, where the shears can be resisted by friction or horizontal bearing against the soil.

4 PRACTICAL CONSIDERATIONS

4.1 Layout

It is essential that at the earliest stages of developing the form of a building, the diaphragms are given as much consideration as any other architectural or structural element, if not more so.

At present, diaphragms are invariably ‘designed’ late in the design phase, if at all. For many of the bad examples shown in Figures 4 to 8, major changes in the layout of the building or at least parts of it would be required if sound seismic resistant diaphragms are to be the result. This cannot occur late in the design process without causing major problems.

Before a preliminary layout is finalised, the structural engineer should sketch the compression and tension load paths he/she envisages in the diaphragms for each load case. Positive connection of the lateral load resisting elements to the diaphragm must be demonstrated, and that all steps, openings and re-entrant corners allow positive load paths to develop. Specific attention to the actual restraint conditions and real slenderness of walls must be given, and the layout of stairs, lift shafts, and other openings near walls should allow for reasonable increases in wall thickness should final design require this.

In general, diaphragms should have a good ‘backbone’ in the form of edge beams to resist the flexure in the diaphragm, and ‘stiffeners’ in the form of beams trimming any opening, which extend sufficiently back into the slab to dissipate any large components of force.

The reader should refer to Figure 1.11 and Figure 1.12 of Seismic Design of Reinforced Concrete and Masonry Buildings (Paulay & Priestley, 1992) for examples of undesirable diaphragm geometries, and preferred alternatives.

The most economic and best performing diaphragms will be the simplest – the K.I.S.S. principle applies especially to diaphragms – *keep it simple, stupid.*

4.2 Diaphragm Size, and Serviceability Limit State Effects

The first consideration in the layout of a diaphragm is its size. Any floor incorporating concrete will be subject to the serviceability limit state actions of creep, shrinkage and temperature. For larger floors, these effects can be significant, and can be exacerbated by layouts of lateral load resisting elements that restrain free movement. The thin toppings on precast floors can be particularly susceptible to severe cracking due to these effects.

Cracks of 10mm width and more have been observed in shopping mall slabs incorporating precast floor units. The three questions one has to ask are, how much of the reinforcing crossing the joint remains unfractured, how can compression and shear be transferred as a result of this wide cracking, and what is the strength of the diaphragm as a result of this wide cracking?

The issue of floor size and restraint is covered in many texts on reinforced concrete and prestressed concrete. Guides such those issued by The Institution of Structural Engineers and The Institution of Civil Engineers in Britain recommend that any large reinforced concrete building be ‘broken up’ with complete movement joints at maximum 50m centres. Each part of the building between these joints must act as a completely independent, stable structure.

A very careful assessment of creep, shrinkage and thermal effects should be made if it is intended to have a floor plate longer than 50m, especially if a thin topping on precast floor units is involved.

It is essential to ensure that when all of the creep, shrinkage and temperature effects have occurred, and all of the cracks have formed, the expected ultimate limit state strength of the diaphragm and the building as a whole can still be developed.

4.3 **Drag and Collector Beams**

In many instances, the length of diaphragm in direct contact with a shear wall or braced bay will be insufficient to transmit the diaphragm forces that the wall or brace must resist. This is particularly true in transfer diaphragms, although in this instance, the most intense force transfer is usually in the opposite direction - from the wall or brace to the diaphragm.

The most practical solution to this problem is to ensure that beams are aligned with and connected to the shear wall or braced bay, and that these 'drag beams' or 'collector beams' extend far enough into the diaphragm to allow proper load transfer.

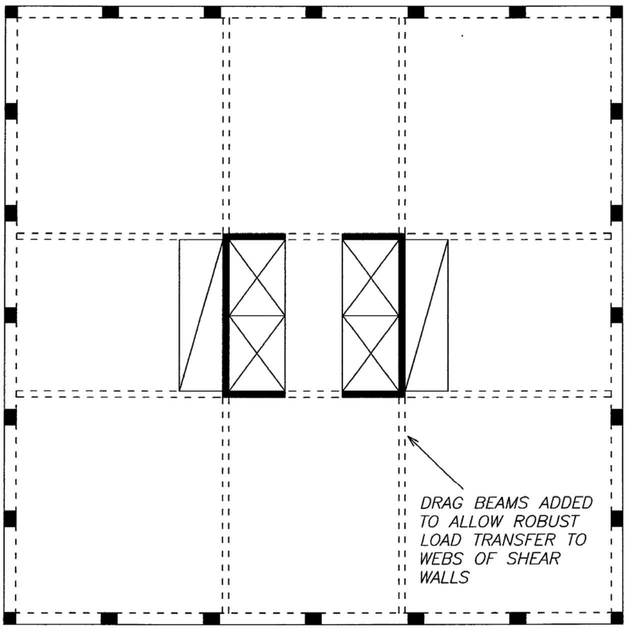

Figure 12

Figure 12 shows how the diaphragm shown in Figure 4 can be improved through the addition of drag beams, provided the shear wall webs are thick enough to accommodate the drag reinforcing, and not buckle due to the lack of floor restraint.

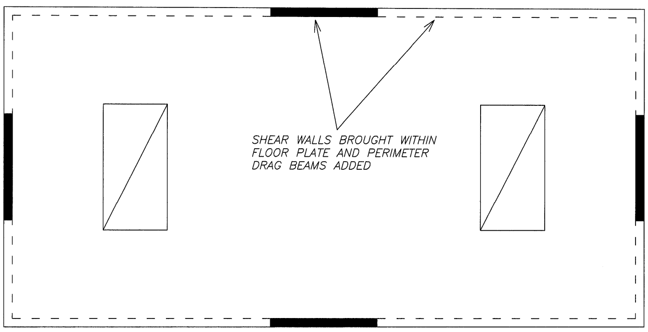

Figure 13

Figure 13 shows how the diaphragm shown in Figure 7 can be improved by bringing the walls within the floor plate, and adding drag beams.

For light loads, ‘drag bar’ reinforcing extending into the slab may be sufficient.

Invariably, there is an eccentricity between the diaphragm slab and the centre of the concrete or steel beam that is required to act as a drag beam (or diaphragm chord or stiffener), and the effect of this eccentricity must be considered by the design engineer. Usually, the length over which the force transfer from the diaphragm slab to the beam occurs is sufficient for the resulting bending moments and shears to be small, but there may be instances where the force transfer occurs over short lengths of beam, and very large bending moments and shears will result. These effects must not be ignored.

Steel collector beams attached to steel braced bays must have sufficient composite slab concrete on both sides or properly detailed confining steel to ensure the shear studs can transmit the loads from the slab into the beam. For all but the lightest loads, end moment connections should be used for steel collector beams in steel frames, so that the large axial strength of the two flanges can be utilised.

Web side plate (‘WP,’ or shear cleat) end connections to beams have limited axial strength in tension, and the eccentric nature of the connection, with the web side plate offset from the beam web, must not be ignored when designing a WP to transmit axial load in compression.

4.4 **Shear Wall Thickness**

In good seismic resistant reinforced concrete design, the size of members must be determined not just by the strength and stiffness requirements for each individual member, but also by the requirement to ensure that the members, and in particular, all of the reinforcing, will fit together properly at the joints, without the mindless cranking of reinforcing in regions of high stress that so often occurs.

In many instances, openings in the floor adjacent to the web or flange of a shear wall mean that lateral restraint from the floor slab, as usually assumed by various concrete design standards, does not exist, and the engineer must ensure that the web or flange will be stable through other means, and that the real slenderness effects in the web or flange are accounted for.

Singly reinforced shear walls may be up to 200mm thick. The author’s opinion is that singly reinforced seismic resistant shear walls should be limited to low rise buildings, and if the walls are critical to the stability of the building, they should be stocky, not slender.

Shear walls that are 250mm thick are impractical – by the time cover and the finite size of the bars are considered, there is little gap between the two layers of reinforcing.

The American Concrete Reinforcing Steel Institute (Wyllie *et al*, 2003) insists that for diagonally coupled shear walls, the minimum thickness of wall should be at least 16 inches (406mm), with an absolute minimum of 14 inches (356mm), in order to accommodate realistic pairs of diagonal coupling reinforcement that can fit past each other, and two lots of cover concrete, and two vertical and two horizontal layers of reinforcing in the walls.

The author agrees with this advice, and suggests that similar minimum wall thicknesses are required if the reinforcing from properly configured drag beams is to be accommodated in uncoupled shear walls. Furthermore, walls with two layers of reinforcing should have cross ties at close centres along their length, not just for confinement and improved axial load performance, but to prevent splitting of the wall that has been observed in some earthquakes.

4.5 **Coupling of Walls and Braced Bays by the Slab**

One aspect of slab behaviour that is often overlooked is the tendency of in-situ and precast slabs in particular to attempt to ‘couple’ closely spaced shear walls or braced bays. As a wall or braced bay deflects and rotates, one end goes up and one end goes down at each floor level, distorting the slab. If two shear walls or braced bays are close together, these distortions can generate significant coupling moments and shears in the slab, causing significant damage.

4.6 **Ductile Reinforcing and the Dynamic Fracturing of Reinforcing**

The traditional low ductility welded wire mesh used in the topping of floors with precast concrete units or in composite metal deck floors performed poorly in the Christchurch earthquakes. Fracture of the mesh was noted in many buildings and was sufficiently serious to warrant the demolition of a number of buildings.

In the Christchurch earthquakes, unexpected fracturing of the ductile main longitudinal reinforcing was observed in numerous beams, columns and shear walls. Concrete strengths much higher than specified, combined with additional dynamic strength enhancement due to the high rates of strain loading, caused cracking patterns significantly different to those that usually occur in standard university type tests. In these tests, numerous smaller cracks form in the hinge zones, and there is significant bond breakdown over long bar lengths, allowing the reinforcing to stretch without fracture. However, what was observed in Christchurch was a small number of wide cracks, with very limited bond breakdown. As a consequence, the strain in the reinforcing became excessive, leading to bar fracture.

In the author's opinion, all diaphragm slab reinforcing should be ductile deformed reinforcing, detailed and handled on site to ensure that re-bending on site does not occur. The same applies for all starter bars from walls and frames into the diaphragm slab.

The diaphragm slab reinforcing must be firmly supported in position to the correct height, seated on approved concrete spacers, not plastic, so that the reinforcing can support the weight of the workmen and other loads during concrete placement and compaction and remain at the correct height. The use of deformed bars instead of mesh often allows a 300mm spacing of bars, which allows workmen to place their feet between the bars. 'Lifting' of unsupported reinforcing as the concrete is placed is not acceptable. For cast in-situ slabs with two layers of reinforcing, ensure that steel support chairs to support the top layer in position are shown on the drawings, and are supplied and installed.

In areas of the diaphragm such as drag beams where significant strains may develop but the load path must be maintained, care should be taken to ensure that dynamic fracturing of the reinforcing does not occur. Expert advice should be sought, and localised de-bonding may be necessary.

4.7 **What Part of a Floor Forms the Diaphragm?**

With an in-situ slab, the full slab thickness can usually take part in diaphragm action. Especially when there are two layers of reinforcing, the layout and curtailment of reinforcing must suit the diaphragm requirements as much as the bending requirements due to gravity loading. Ideally, two uniform layers top and bottom would provide the diaphragm tensile strength, with supplementary reinforcing placed to resist any additional gravity load demands.

For precast floors incorporating rib, double tee and flat slab precast components, only the topping concrete will act as a diaphragm, unless special details are used to tie the double tee flanges or flat slabs together.

For hollowcore units, with concrete filling the keys between the units, the effective thickness of the diaphragm slab can be considered to be the topping thickness plus the thickness of hollowcore concrete above the voids.

In general, for concrete filled metal decking composite floor slabs, the section of concrete above the metal decking units should be taken to be the effective thickness. Any attempt to utilise the strength of the decking, either in tension or in shear, must be done on a rational basis bearing in mind that the decking sheets are finite in size, and end connections to the decking sheets can develop only a small fraction of the decking strength.

Axial force components in the diaphragm can also be resisted and transmitted by properly designed and detailed reinforced concrete beams and steel beams acting compositely with the diaphragm slab. Full account must be taken of the eccentricity between the diaphragm slab and the beam, *by rational calculation*, not *a priori* assumptions. As indicated above, a simple check may show that the length of beam over which the force transfer occurs is sufficient to keep the resulting bending moments and shears small. However, there may be instances where large forces are transferred over short lengths, and significant bending moments and shears will result.

Particular attention must be paid to areas of diaphragms where the thickness is at a minimum. In one transfer diaphragm in an apartment building damaged in a recent earthquake in NZ, the effective thickness of the diaphragm reduced significantly adjacent to the perimeter, to satisfy architectural requirements. This reduced section cracked significantly.

4.8 **'Code' Provisions for Diaphragms**

Explicit code provisions for diaphragms in New Zealand are largely limited to Section 13 of NZS 3101:2006. The provisions are very general in nature, with the only specific requirements relating to tying the topping of precast floor units in Section 13.4, and only then if the diaphragm is considered to be acting in a 'ductile' or 'limited ductile' fashion.

Clearly, whilst diaphragms must be 'tough,' they should not be assumed to be forming plastic hinges. Diaphragms must only be designed to resist elastic (μ=1), nominally ductile (μ=1.25) or overstrength actions. However, the provisions of Section 13.4 should be applied to all diaphragms incorporating precast floor units where the diaphragm stresses are anything but minor.

Figure 14 of Bull's paper (Bull, 2004) and the related text shows one of the reasons for tying the topping to the precast floor units where the stresses are high. Even with nominally ductile response, the concrete will crack and the diaphragm reinforcing will yield due to tension in one cycle, and then go into compression on the next.

4.9 **Should Diaphragm Component Forces Be Combined With Forces From Other Types of Action?**

Contrary to what is often expressed in other papers on diaphragms, the author considers that diaphragm force components should not be considered in isolation, and must be combined with actions from other loads that must be acting simultaneously, for example, gravity loading and soil retaining loads.

The simple fact is that these actions occur at the same time, and if a slab requires a certain strength to support its self-weight and imposed gravity loads, and in some instances, axial loads from retained soil loads, it must require additional strength to resist additional imposed seismic forces.

4.10 **Buckling of Diaphragms**

The potential buckling of diaphragms is often a complex problem, but should not be overlooked, especially when transfer or compatibility effects lead to very large force components.

Under non-seismic conditions, a slab in a buried podium structure subject to soil retaining forces may be subject to uniform axial load, in one or two directions. Gravity loads on the slab will cause vertical deflections, which will increase due to creep, and these P-delta effects will reduce the buckling resistance of the slab.

Most diaphragms above ground in NZ will consist of the topping on a precast floor slab, or the solid concrete above a profiled metal decking. In both instances, the relatively thin diaphragm will be completely dependent upon composite action with the precast floor units or the concrete filled metal decking to prevent buckling. This is another reason to ensure toppings are tied to precast floor units when the diaphragm actions are anything other than nominal.

It should not be assumed that the in-situ slab or composite precast or metal deck floor will not buckle – it should be checked by rational analysis. This is a complex problem, because the buckling will often be caused by diagonal compression force components, not overall uniform axial load, and P-delta effects due to the vertical deflection of the slab must be considered.

The potential buckling of beams near floor openings that are unrestrained by the floor but required to carry axial load must not be ignored.

4.11 **Detailing of Reinforcing**

The general principles of developing and anchoring reinforcing apply to diaphragms. A conservative approach should be taken when terminating reinforcing.

Around the perimeter of a diaphragm, the outwards thrust of the inclined compression struts must be 'tied' back by the diaphragm reinforcing. Standard Strut and Tie practice requires the ends of the reinforcing to be hooked or be fitted with an end bearing plate to allow this force transfer to occur at the end of the bar.

The 90^o or 180^o bar hook must lie towards the inclined compression strut, for effect force transfer.

Under reversed seismic loading, however, the inclined compression struts will come from opposite sides of the bar.

Generally, 10mm or 12mm diaphragm bars will be anchored around the perimeter of a diaphragm by being lapped onto starter bars from walls or beams. These starter bars should project as close as possible to the outside face of the wall or beam, and be hooked down into the wall or beam.

(Ideally, these starter bars would be installed as straight bars, which are bent down on site after the flooring has been placed, so as to avoid re-bending on site, with all its attendant problems).

Where 16mm bars and larger are required to tie compression struts around the perimeter of a diaphragm, explicit checks of the end anchorage capability of the details under reversing seismic loading should be carried out.

Bull's paper (Bull, 2004) covers some of these details in depth.

Sometimes, one sees very large diameter bars, even 32mm, specified in 65mm thick toppings, to resist diaphragm 'chord' forces. This is highly suspect – there is a very real tendency for the large bar to buckle in compression, particularly after a couple of cycles of tension loading. A far better approach is to include an actual beam to act as the chord, complete with stirrups to prevent bar buckling.

4.12 **Proper Compaction of Concrete in Diaphragms**

Far too often in NZ, the proper compaction of concrete on site is neglected, especially concrete that will be required to form a diaphragm. It is as if many of the people placing the concrete and supervising its placement view an immersion vibrator as a levelling or concrete movement device and nothing more, and they have no comprehension of the need for sound dense concrete to not only develop the required concrete strength, but also to provide durability and resistance to moisture transfer.

The proper use of an immersion vibrator on site should be one of the simplest tasks to be achieved, but it isn't.

For in-situ work, all too often large 'boney' areas of uncompacted concrete are exposed upon removal of the formwork. Worse is the in-situ component on top of precast floor units or composite metal decking, where the screeded top surface and the stay in place underside prevent exposure of any uncompacted concrete.

In many of the photos showing the placing of topping on precast floor units and even composite metal decking one sees in industry journals, there are workmen, concrete pump hoses, rakes and screeds all present, but no one compacting the concrete with an immersion vibrator. And 'self-compacting' concrete is not being used.

Even when one can get the concrete in precast and composite slabs to be vibrated, there is a recurring problem with regard to screeding and the working face. As concrete trucks change over at the pump, screeding proceeds right to the edge of the placed concrete, even though the last several hundred millimetres width of concrete could not have been vibrated. But when one asks the workmen to compact that unvibrated concrete once the new concrete starts to be placed, they refuse, because it has already been screeded.

These practices have to stop. Unless self-compacting concrete is being used, all concrete that is placed on site must be fully compacted with immersion vibrators, and the placement, compaction and screeding must be done in a sensible staged manner so that there are no uncompacted zones, even if that means some re-screeding.

And, of course, all diaphragm concrete must be properly cured, especially given that diaphragm concrete is relatively thin, with a very large surface area to volume ratio.

4.13 Steps in Diaphragms

Steps in diaphragms occasionally occur. Often, steps occur in the top floor of podium structures, due to the slope of the surrounding land, and architectural requirements. Such floors are often subject to very large transfer diaphragm forces, and the effect of steps must be properly accounted for.

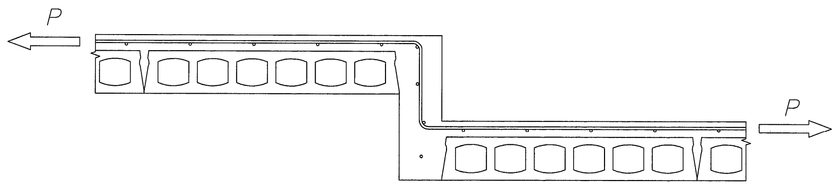

Figure 14(a)

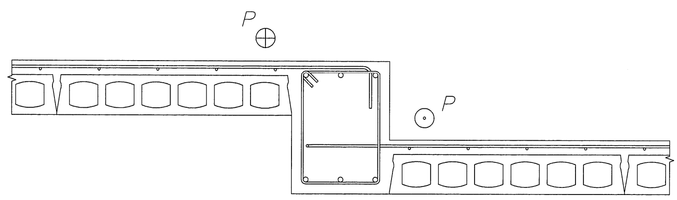

Figure 14(b)

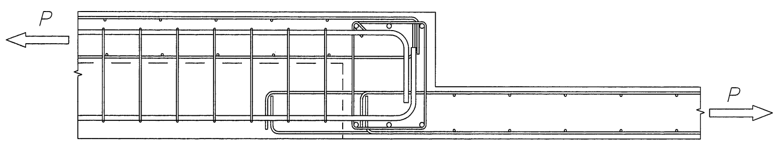

Figure 14(c)

Figure 14(a) shows the type of diaphragm step detail the author has seen proposed on more than one occasion. It is nonsensical, but 'convenient.' There is no way that any appreciable force, particularly a tension force, can be transmitted transverse to such a step, unless the diaphragm was extremely thick, and the corners of the step were able to be detailed as reliable opening/closing moment joints.

Figure 14(b) shows a detail which can be used to transfer forces parallel to the step, provided the beam forming the step is properly designed and adequately supported by columns or walls.

Figure 14(c) shows the likely detailing required where forces transverse to a step must be transferred across it. Either side of the step, the slab is solid in-situ, and closely spaced beams are installed on the left hand side, to transmit the forces from the high slab to the low slab. Slab reinforcing from the lower slab is extended well into these beams, to allow for proper tension load transfer. Again, the beam forming the step should be properly supported by columns or walls.

4.14 **Sawcuts in Diaphragms**

In shopping malls and car park buildings in particular, diaphragms, particularly those consisting of in-situ toppings on precast floor units, are often subject to all sorts of saw cuts in an attempt to ‘control’ cracking for non-structural purposes.

Why cut through a perfectly good structural element that has cost the client a lot of money to have had installed?

If diaphragm toppings and slabs must be saw cut in such a manner, then their strength should be assessed on the thickness that is left *under* the sawcuts, *not* the initial gross thickness.

As with all structural concrete, the concrete used in diaphragms should have the minimum water and cement contents consistent with other material performance criteria, to ensure shrinkage is minimized. Diaphragm concrete must be fully compacted, and well cured. Use superplasticizers to achieve workability, not water.

4.15 **Giving Diaphragms the Attention They Deserve**

To reiterate, floor diaphragms are a critical component of all buildings subjected to seismic loading. Often, the derivation of the loads they are subject to is difficult, and their proper analysis and design can be extremely complex and difficult.

Diaphragms must have design time and resources allocated to them sufficient to deal with the real complexities they present, and diaphragms must be treated with the same respect that we demand is paid to the main frames, walls and bracing of a building.

The author has seen one large complex diaphragm, which contained numerous steps, large openings and complex load paths ‘designed’ by another engineer in less than two hours. The ‘design’ consisted of a small increase in topping thickness and mesh size. In reality, to have properly and rigorously analysed and detailed that diaphragm would have taken weeks, and may well have required significant changes to the structure.

Fees and programmes will have to expand to allow for the proper analysis and design of all diaphragms in a building.

If clients, architects and project managers don’t want this expenditure of time and money, then they will have to conceive of extremely simple buildings, with repetitive floor diaphragms throughout the height.

5 **ANALYTICAL METHODS**

5.1 **Simple Hand Methods**

Simple methods treating a floor diaphragm as a simply supported beam or cantilever are only applicable for the most simple of diaphragms (rectangles) and support conditions, with very small openings placed in locations where they have very little effect on the strength of the diaphragm.

5.2 **Conventional Strut & Tie Analysis**

The conventional, rigorous Strut & Tie (S&T) method can be applied to analyse complex diaphragms subject to inertial and transfer forces. However, the S&T method cannot identify the compatibility forces that may exist in a slab.

Whereas the S&T method can be used to arrive at a safe design for even very complex diaphragms, it usually requires a great deal of effort, and *every load case* for one diaphragm requires *its own unique S&T model.*

The more complex a diaphragm becomes, the more difficult it is to arrive at these unique models, and in many instances, an initial linear finite element analysis should be done using a suitable program to identify an appropriate layout of struts and ties based on the principal stress components. In addition, if the support conditions change to accommodate changes to the location of walls or braces, or simply to make the diaphragm 'work' as part of the design process, completely new models must be developed and analysed, one for each load case, whereas the Truss Method proposed in this paper requires only one model that can be quickly modified and re-run, for all load cases.

By its very nature, in areas of solid, continuous slab, the S&T method results in assumptions of highly concentrated load transfer to reinforcing, whereas in reality, the load transfer is much more distributed.

5.3 **Non-linear Finite Element Methods**

Normal finite element analyses using linear elastic triangular and rectangular finite elements can be used to model the geometry and stiffness of a diaphragm. However, deriving useful design information on the internal forces acting within the diaphragm from such analyses is problematic, at best.

Advanced non-linear finite element programs may allow some useful information on the internal forces to be derived, but these programs are very expensive, difficult to use for all but experienced operators, and require significant processing resources and time for each analysis, along with considerable post-processing time.

One of the most useful suites of non-linear programs for the analysis and design of floor diaphragms appears to be Ruaumoko, which was developed by Atholl Carr at the University of Canterbury (Carr 1981-2009).

For her PhD studies into seismic forces acting on diaphragms, Debra Gardiner used lattice elements in Ruaumoko as one analytical tool. She used concrete and steel lattice elements in parallel to model the non-linear behaviour of reinforced concrete diaphragms. Refer Design Recommendations and Methods for Reinforced Concrete Floor Diaphragms Subjected to Seismic Forces, PhD Thesis (Gardiner, 2011).

5.4 **The Truss Method**

The Truss Method is essentially a linear finite element method of analysis that uses simple truss elements instead of triangular or rectangular elements, and is at the same time a type of strut and tie analysis. It can be implemented on any basic structural analysis package such as Microstran or SpaceGass which incorporates compression-only elements.

The method allows simple analyses similar to the more complex analyses that can be done on Ruaumoko using lattice elements.

Just as, with care, a two way slab can be analysed under gravity loads as an equivalent grillage of beams, each representing a tributary width of slab, the Truss Method allows a diaphragm to be analysed for in-plane loads by treating it as an orthogonal grillage of truss elements, each representing a tributary width of slab, with compression-only pairs of diagonals throughout the grillage to allow for diagonal strut action. It is assumed that the reinforcing in the diaphragm is placed orthogonally.

The refinement of the truss is a matter of judgement – fine enough to allow the diaphragm geometry and behaviour to be effectively modelled, but not so fine as to produce an overwhelming amount of output data.

Supports are modelled as being rigid or as springs, so that the reactions onto them match the forces being transferred from the diaphragms to the lateral load resisting elements in the structure as a whole.

Multiple load cases can be handled by the one model, unlike the S&T method. Provided changes to geometry or support conditions do not require a redefinition of the main grillage, modifications to the model can be made and quickly re-analysed, unlike the S&T method.

For each load case, the location of 'struts' and 'ties' is automatically determined by the analysis, based on the requirements for equilibrium. Drag beam requirements are clearly indicated, and forces around openings and at re-entrant corners are automatically produced, and the analysis indicates how far those actions must be developed past the opening or corner.

Because most floor diaphragms are flat, the truss model will be 'flat,' and two dimensional. However, complex diaphragms that incorporate steps, and are hence three dimensional, can be modelled through the addition of beam elements and rigid links to model the steps.

Openings and re-entrant corners are automatically accounted for in the geometry. If parts of the diaphragm are likely to be ineffectual because of damage caused by plastic hinge growth in reinforced concrete beams or other precast floor damage, truss elements in those areas can be removed.

The Truss Method handles highly concentrated force transfer around openings and at re-entrant corners like the S&T method, but where there are large areas of slab, the Truss Method allows for much more distributed force transfer, approximating continuous shear flow, providing the grillage is fine enough.

With the Truss Method, the design procedure for the diaphragm is similar to the S&T method. The shear strength, v_c, is taken to be zero, and all actions, including shears, are resisted by strut and tie action.

This method is not, of course, merely confined to diaphragms, and can be used, with caution, to analysis complex beams and walls that are more often analysed using S&T methods. At the very least, it can be used to determine the location of the struts and ties, before carrying out a more rigorous S&T analysis.

The Truss Method is described in detail in the following section.

6 THE TRUSS METHOD FOR THE ANALYSIS AND DESIGN OF DIAPHRAGMS

6.1 The Truss Method

The Truss Method is best described by way of example.

Consider a fixed base, cantilever reinforced concrete shear wall. This simple wall is considered in the first instance, because it allows a check of the accuracy of the method with regard to deflections, and internal force distribution.

The wall height is 9.25m, the length 3m and the thickness 300mm. Load cases consist of one 1000 kN point load pushing against the left hand side of the wall, at a height of 9m (Unit Load A), and one 1000 kN point load pulling from the right hand side of the wall, at a height of 9m (Unit Load B). Refer Figure 15.

To create the truss model in Microstran, the wall was 'cut' into a series of 500mm strips horizontally and vertically. Truss elements were placed between node points at the intersections of the centrelines of these strips. Pairs of compression-only diagonals were then added to every bay of the grillage. All base nodes were restrained with rigid supports in the horizontal and vertical directions.

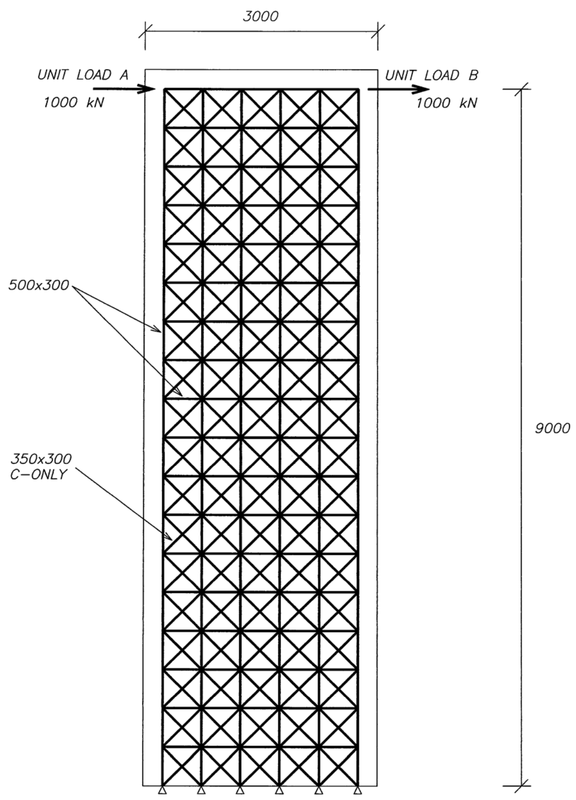

Figure 15

For this exercise, gross sections were considered, with Young's Modulus $E_c = 2.7 \times 10^7$ kN/m^2, and Poisson's Ratio $\nu = 0.2$. The horizontal and vertical truss element properties were those for a 500mm wide by 300mm thick rectangle. The diagonal dimension of each bay in the grillage is $500 \sqrt{2} = 707$mm. Rounding this off to 700mm, half this dimension was assigned as tributary to each diagonal, leading to the properties of each diagonal element being those for a 350mm wide by 300mm thick rectangle.

[For the Truss Method, the size of the grillage is a matter of judgement. Fine enough to allow the geometry to be accurately modelled, and realistic internal structural actions to occur. The way the member properties have been assigned, with the full tributary width of each strip assigned to the horizontal and vertical truss members and half the diagonal bay width assigned to the diagonal truss members is somewhat arbitrary, but this approach has always produced realistic results. When using the Truss Method for the first few times, the engineer may wish to test the sensitivity of each model to slightly varying assignments of section properties.]

The theoretical horizontal deflection of this cantilever wall at the height of the loads, taking into account flexural and shear deformations, is 14mm. The deflection for the truss model was 16mm, which was in good agreement.

Figures 16 and 17 show the truss member forces at the top of the wall for Unit Loads A & B respectively. Tension forces are positive, compression forces are negative. The 'struts' and 'ties' automatically adjust for each load case. Figure 18 shows the truss member forces at the base of the wall for Unit Load A, and the corresponding forces were almost identical for Unit Load B.

The distribution of forces is consistent with conventional reinforced concrete beam theory and design, assuming the shear strength of the concrete, v_c, equals zero.

The quantity of reinforcing required for each orthogonal tie is simply the tensile axial force divided by the yield stress of the reinforcing reduced by the capacity reduction factor, and this reinforcing can be distributed across the depth of the truss member. Simple checks as to the rate at which the tension force is increasing along a strip, and whether major loads are introduced at discrete points, can readily be done to determine if the bond stress is likely to be exceeded, and if so, more detailed nodal checks as per the S&T method can be carried out.

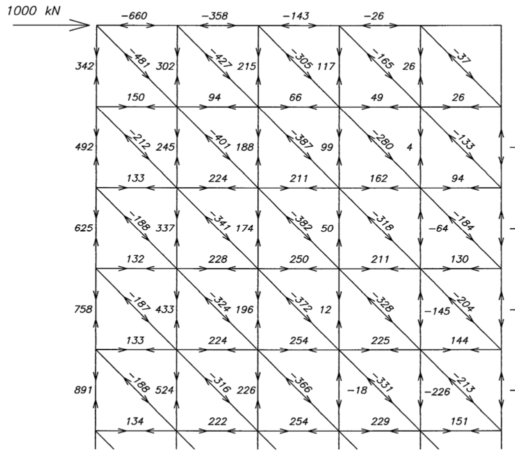

Figure 16

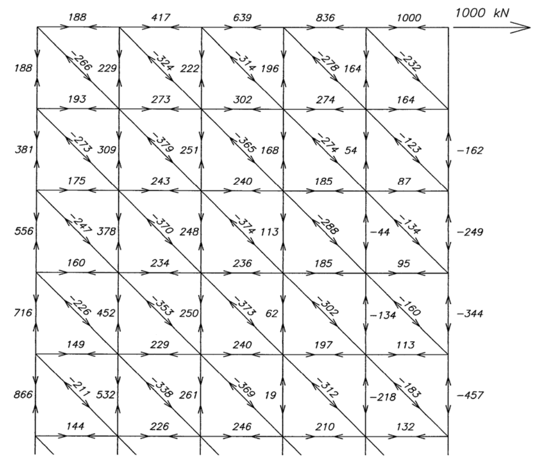

Figure 17

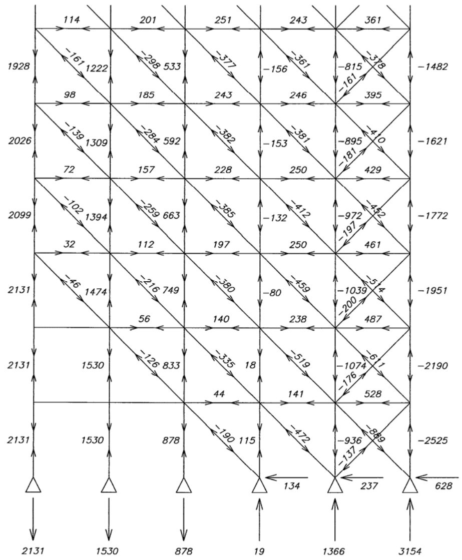

Figure 18

The concrete stresses for both the orthogonal and diagonal struts can be easily derived, and checked against the allowable, including allowance for slenderness and buckling if required. If the stresses are high, the interaction of the components of orthogonal and diagonal compression stress may need to be checked.

For this particular example, the tie forces at the base and the magnitude of the concrete stresses indicate that the wall thickness should be increased, or column boundary elements introduced at each end.

6.2 **Comparison With Strut and Tie Examples**

The Truss Method was tested against several S&T models in Examples for the Design of Structural Concrete with Strut and Tie Models (Editor: Karl-Heinz Reineck, 2002).

For a cantilever shear wall with two openings, subjected to five load cases involving various combinations of vertical and horizontal point loads, reasonable agreement in the strut and tie forces for each load case was achieved. Whereas the S&T method required five complex models to be developed, the Truss Method required only one model. A major difference in the results between the Truss Method and the S&T method was that clear of openings, the Truss Method showed a realistic, near uniform shear flow in the outer vertical members, instead of highly discrete zones of force transfer.

For a deep beam with one large opening and a reduced section at one end, and subject to a single vertical point load over the opening, the reactions were, of course, the same for both methods, but the distribution of internal forces was significantly different. The author believes the Truss Method produced the more accurate distribution. The S&T method assumed the point load over the opening was resisted by two opposing diagonal struts, tied just above the opening, whereas the Truss Method showed diagonal struts, plus significant negative bending moments as the concrete beside and above the opening was able to act like a deep portal frame.

6.3 **Diaphragm Example**

Figure 19 shows the plan view of a suspended reinforced concrete First Floor in a theoretical two storey retail building. A steel structure forms the upper walls and roof, and the weight of this was assumed lumped at the First Floor level as an additional superimposed dead load (SDL) for the analysis and design of the diaphragm.

The location is Auckland, with Site Subsoil Class C, and μ=1.25 for the main structure and the diaphragm. The first period, T_1, was assumed to be 0.1 seconds.

The floor consists of a cast in-situ reinforced concrete ribbed slab, spanning one way between beams. This would be constructed efficiently using modern formwork systems and a skilled workforce. Four reinforced concrete shear walls are assumed to provide all the lateral load resistance.

At the re-entrant corner, a ‘stiffener’ beam running parallel with the floor system has been introduced, to ensure the corner is strengthened in both directions.

There are three large floor penetrations. One of these is adjacent to the left hand shear wall. This affects not only the load transfer from the diaphragm to the wall, but also compromises the stability of the wall. As a consequence, two 1200x400 return flanges have been introduced to stiffen the ends of the wall out of plane. Slenderness effects in the wall will be checked in the horizontal direction. These return flanges were ignored in the analysis, however, because it was assumed that the squat nature of the walls, and the effect of shear deformations and foundation flexibility on the stiffness of the walls meant that the actual impact of the flanges on overall force distribution would be negligible.

The floor penetration adjacent to one of the perimeter beams mean that the slenderness of that beam would have to be checked, especially when subjected to compressive diaphragm ‘chord’ actions.’

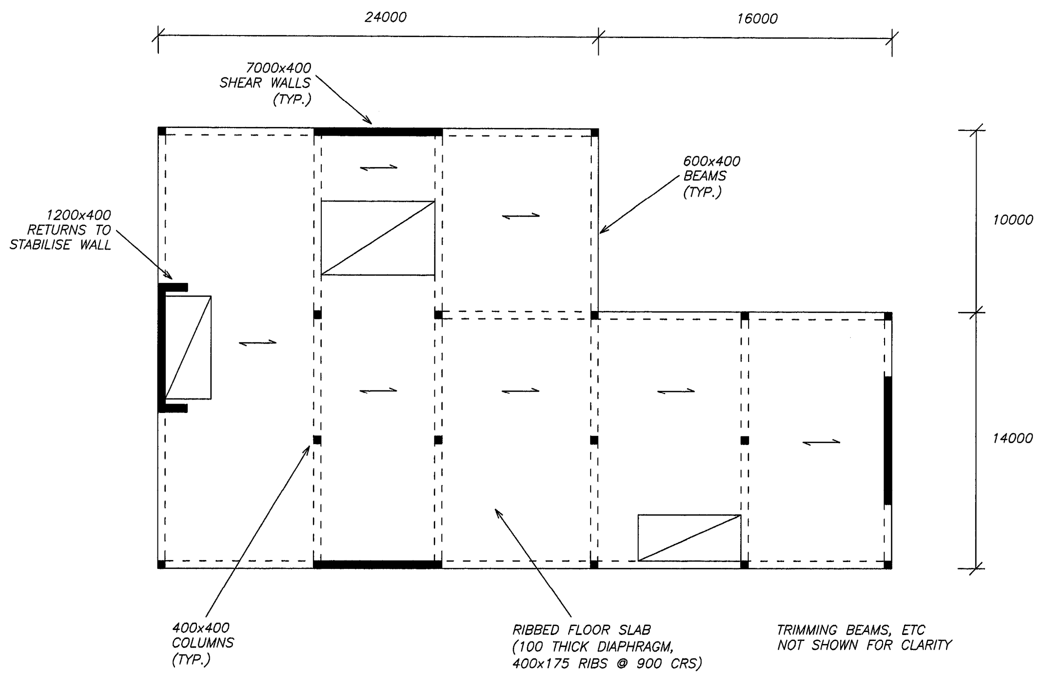

Figure 19

(In reality, the 400x400 columns shown are probably too small for a sound, seismic resistant design. Consistent with good Californian design, a structure such as this, where shear walls are used to resist 100% of the code specified seismic loads, should still be able to develop sufficient frame action so that if the shear walls were removed, the frames can resist 25% of the code specified seismic loads).

Figure 20 shows the proposed truss model laid over the actual geometry. The ribs of the floor system have been ignored in the assignment of member properties, and for the purposes of this exercise, section sizes have been rationalised and edge members have generally been assigned the same properties as internal members.

Figure 21 shows the 2D truss layout alone. Single springs are used to represent each shear wall, the spring stiffness having been easily derived for a cantilever shear wall, adjusted to allow for foundation flexibility.

(For a floor in a multi-storey building, the spring stiffness of each lateral load resisting element should be chosen to ensure that the distribution of reactions to each lateral load resisting element matches that which is occurring in the structure as a whole).

The total seismic weight tributary to the First Floor was calculated, based on the weight of the main reinforced concrete structure, an SDL of 0.75 kPa to represent the steel structure above, an SDL of 0.75 kPa for the First Floor, and a live load contribution of 0.6 kPa (being derived from a design live load of 4.0 kPa).

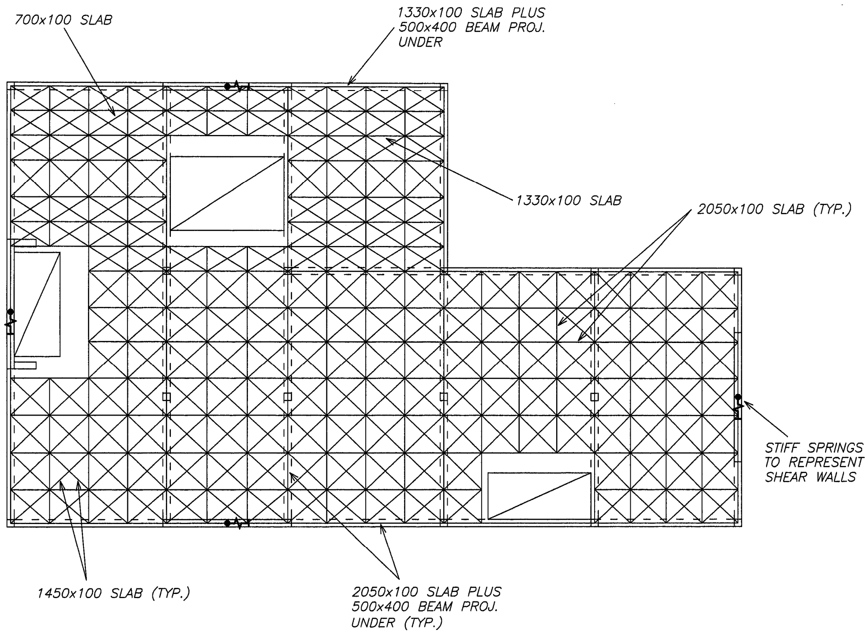

Figure 20

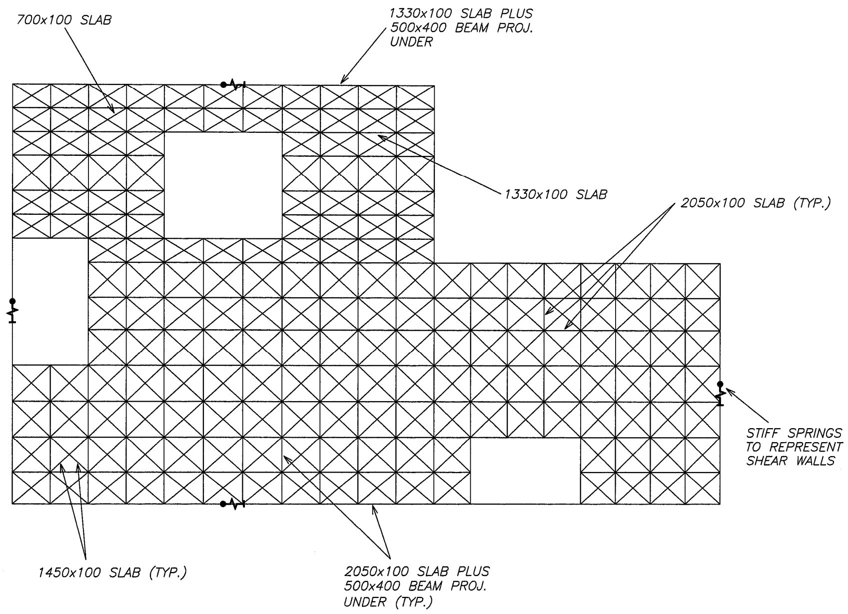

Figure 21

The horizontal design action (base shear) coefficient for this 'one mass level building' ($C_d(T_1)$) equals 0.27, whereas C_{dia} derived by the Cowie *et al* method was 0.25. The 0.27 value was used as the seismic coefficient for this analysis.

Ordinarily, a minimum of eight load cases would be required, to account for earthquakes acting in both plus and minus principal directions, with +/- 0.1b eccentricities in each case. Numerous additional load cases would be required to account for the requirement of 100% actions for one principal direction combined with 30% actions for the other principal direction, because of the nominal ductile response.

To reduce these to four load cases with the resultant acting through the centre of mass (E+X, E-X, E+Y, E-Y), the seismic forces were scaled up to account for the eccentricities and 100% + 30% requirement.

Because there are two shear walls in each direction, and these walls are located at the extremities, a 20% increase in base shear should cover the +/- 0.1b eccentricity requirement, and from experience, a 10% increase should cover the 100% + 30% requirement.

(These should be checked in more detail for an actual analysis for a real building. The +/- 0.1b eccentricity requirement can lead to more than a 20% increase in force on a lateral load resisting element. The 100% + 30% effect can be assessed readily from the actions onto each lateral load resisting element for each load case).

The seismic weight is 6770 kN. The scaling factors and seismic design coefficient combine to produce a total seismic load on the diaphragm for each direction of $E_u = 1.20 \times 1.10 \times 0.27 \times 6770 = 2413$ kN.

This was distributed to each node point of the truss, based on approximate tributary areas to each node.

An interesting problem developed when the model was run, however. Previously, models derived using the Truss Method have always run and converged, and produced sensible results.

For this particular example, the iterative 'non-linear' analysis required because of the presence of the compression-only diagonals would not converge.

By trial and error, it appears that for this particular model, the combination of the largest opening and the spring supports caused some sort of instability, perhaps due to too many compression-only members going into tension and being eliminated from the analysis. Increasing the support stiffness significantly, or changing them to rigid supports, solved this problem, and the iterative analysis converged.

This prompted the development of more models to check for this potential instability, and the results of this check, and the implications for the Truss Method, are discussed in Section 6.4. Suffice to say that in the rare instances when the model becomes unstable, usually due to *large* central openings in the diaphragm combined with spring supports, the method still works if rigid supports are used instead, provided the reactions onto the supports are similar to what would be expected if spring supports were used.

With rigid supports substituted to represent the shear walls in this example, the iterative analysis converged.

Figure 22 shows the truss forces for the load case E+Y in the right hand part of the diaphragm, with the seismic load in the positive Y direction. Figure 23 is for the load case E-Y, with the seismic load in the negative Y direction.

These forces can readily be used to design the required diaphragm reinforcing, and check the compression stresses in the diaphragm. Major 'chord' forces can be seen, and it can clearly be seen that the loads onto the right hand shear wall are 'collected' by the beam projecting from both ends of the wall, as well as being introduced directly into the wall itself.

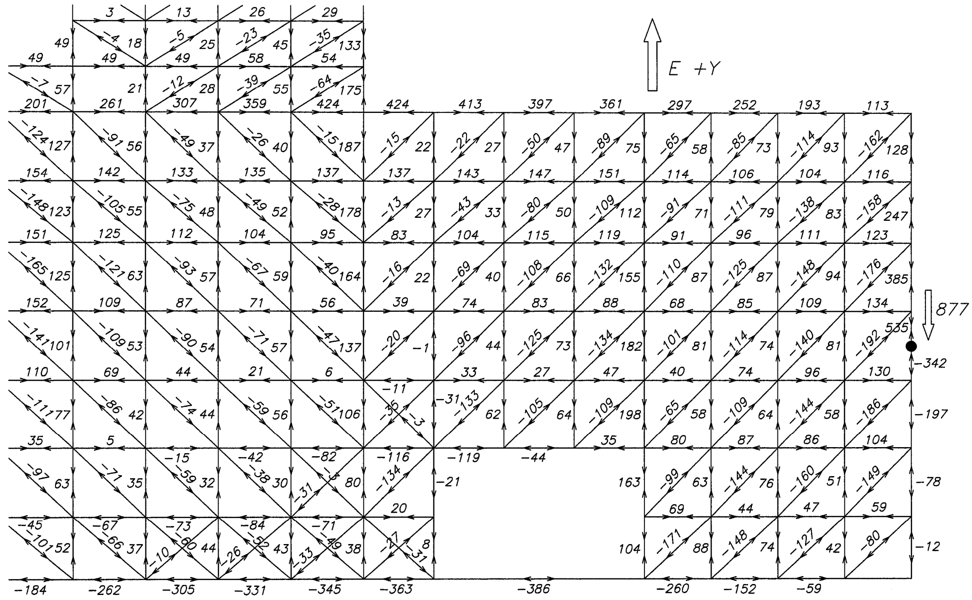

Figure 22

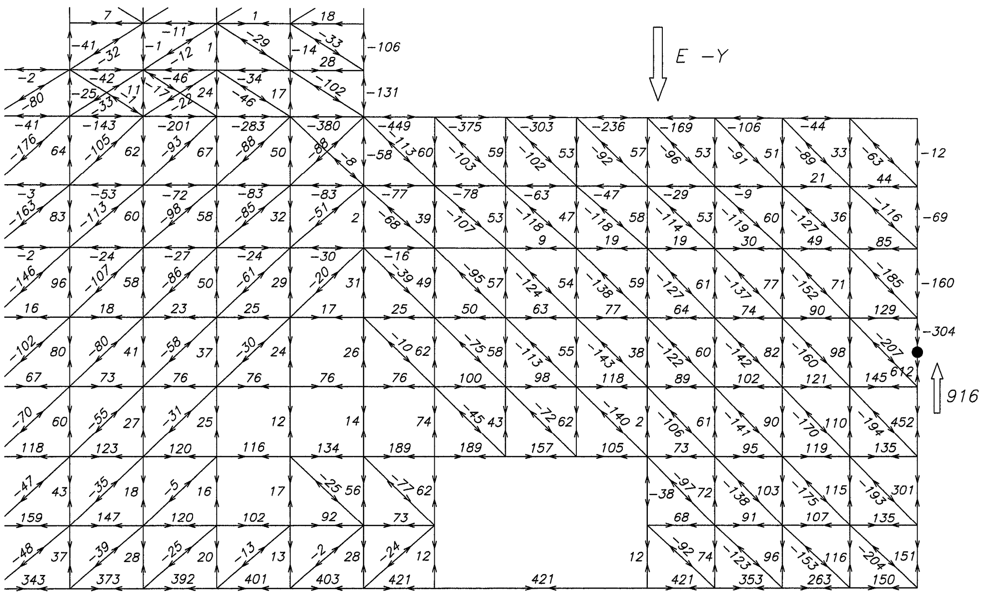

Figure 23

6.4 Check of the Dependability of the Truss Method

The Truss Method has been used to analyse numerous diaphragms and walls, and in all instances the iterative solution required by the compression-only diagonals has always converged without problems. However, as luck would have it, the particular diaphragm example developed for this paper failed to converge initially, apparently because of the presence of one large opening and the spring supports, but it did converge when rigid supports where used instead.

To investigate the sensitivity of the Truss Method to openings, notches, re-entrant corners and spring supports, the set of models shown in Figures 24(a) to 24(h) was developed and analysed.

Figure 24(a) shows the truss model of a simple rectangular diaphragm, with springs to model shear walls to the left and right edges, and moment frames along the top and bottom (the 'base' model).

Two load cases represented earthquakes in the +X and +Y directions. This simple model converged during analysis.

Figure 24(b) is the base model, with one large central void added. This did not converge, but with rigid supports instead of spring supports, the model did solve (Figure 24(c)).

However, Figures 24(d) to 24(h) had spring supports, and a wide range of re-entrant corners, notches and openings, and they all converged.

Therefore, in general, the Truss Method incorporating spring supports can reliably analyse a wide range of complex diaphragm geometries, and in the few cases where the analyses will not converge, rigid supports can be used to achieve convergence, provided the resulting reactions are sufficiently accurate. It may be that a more refined model would converge with spring supports.

If these rigid reactions are not sufficiently accurate, and another scale factor cannot be used to account for these errors, then the design engineer can always fall back on the standard S&T method, provided the reactions used *are* more accurate.

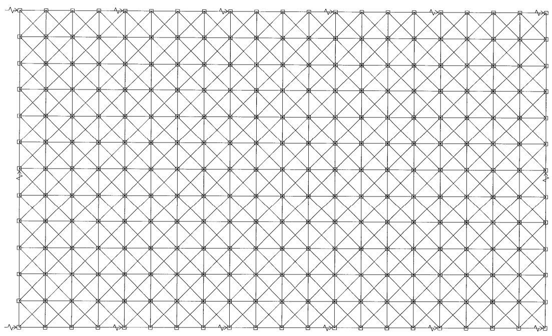

Figure 24(a) - Spring Supports – Solved

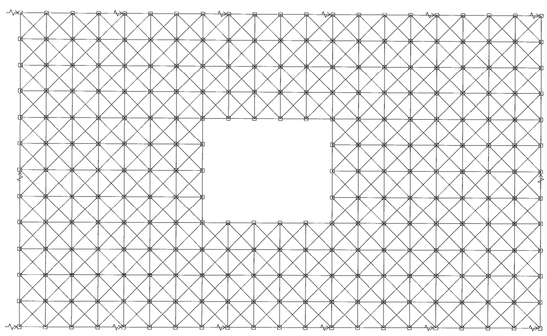

Figure 24(b) - Spring Supports – Did Not Solve

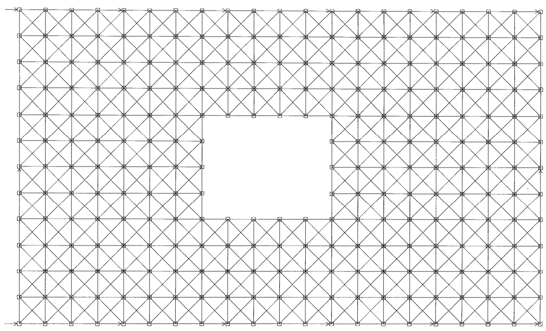

Figure 24(c) - Rigid Supports – Solved

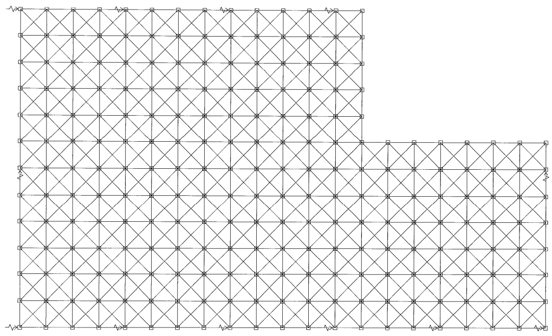

Figure 24(d) - Spring Supports – Solved

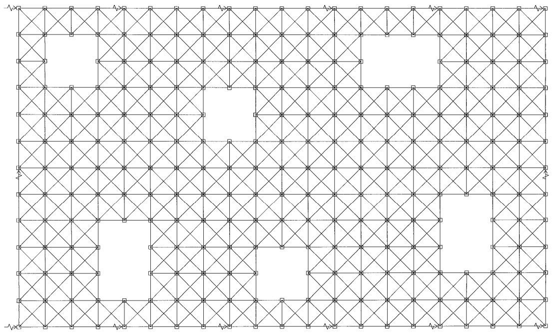

Figure 24(e) - Spring Supports – Solved

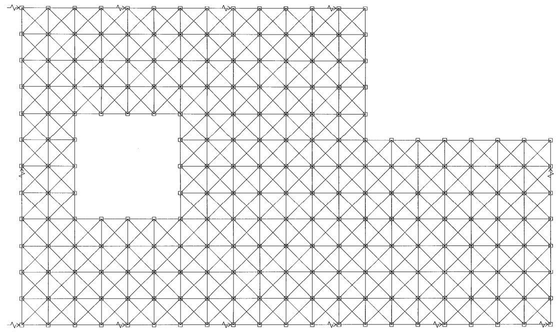

Figure 24(f) - Spring Supports – Solved

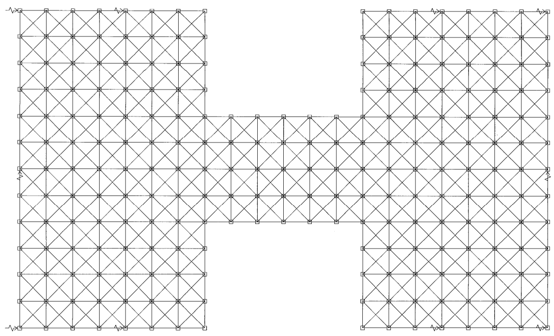

Figure 24(g) - Spring Supports – Solved

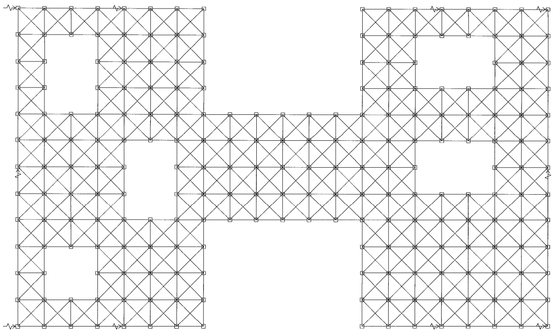

Figure 24(h) - Spring Supports – Solved

One way to 'force' rigid supports to have reactions similar to those expected for spring supports is to copy the model, so that each model deals with one load case only. For each model and load case, all but two of the rigid supports are then replaced by the desired reactions applied as forces, and the requirements of equilibrium should 'do the rest.'

Therefore, the Truss Method is not perfect, just like every other method available to analyse and design diaphragms. But the Truss Method offers a very efficient, lower bound analytical method that can be used for a very wide range of complex diaphragms in a normal office environment, and the results produced are readily able to be used for design purposes.

6.5 **Practical Considerations in Modelling a Diaphragm as a Truss**

Just as with the S&T method, the diagonal members must not be 'too flat' or 'too steep.' If so, dependable strut action cannot be relied upon. Consistent with the NZS 3101:2006 provisions for the S&T method, the diagonals must not be closer than 25^o to the horizontals or verticals, and the closer to 45^o the better.

When modelling a 'flat diaphragm,' the truss model should be purely 2D, and all of the truss members should, in general, be fully pinned truss elements, subject to no bending moments or shears, and with no moment transfer at the joints. Therefore, only the cross-sectional area of the truss members needs to be modelled correctly.

The reason for noting this relates to potential instability problems as the iterative solution is performed, due to the 'thin' nature of most diaphragms.

The programs most likely to be used to analyse a diaphragm truss model in NZ at present are Microstran and Spacegass. For a model incorporating compression-only members, an iterative non-linear analysis will have to be performed. These usually incorporate 'P-Delta' and 'P-delta' effects. The P-Delta effect accounts for the additional actions due to displacement of the nodes of the structure. The P-delta effect accounts for the second order effects due to lateral displacement of the members between the nodes, with bending moments increasing in the presence of axial compression, and reducing with the presence of axial tension. If members are too slender with respect to the applied loads, they will become unstable within the analysis itself.

For the diaphragms being considered in this paper, the P-Delta and P-delta effects should be turned off. However, if the axial compression in a truss member exceeds the Euler buckling load, the member will still become unstable during the analysis.

In a real precast concrete or composite metal deck diaphragm, the concrete thickness acting as the diaphragm cannot buckle in plane because of its continuity, and, subject to proper design, will not buckle out of plane because of the flexural stiffness of the precast floor units or the composite steel slab.

For a 2D truss model, once the diaphragm is 'cut up' into discrete truss members, buckling of the truss members in the non-linear iterative analysis should not be a problem, provided the properties of the truss members are correctly oriented, with their 'wide' dimension in plane, and their 'thin' dimension out of plane.

Alternatively, the second moments of area of the various truss members could be arbitrarily increased to ensure buckling cannot occur.

For a 3D truss model used to model a stepped diaphragm, with the vertical support from walls and columns accurately included, the truss members will, in general, have to have their second moment of area for the out of plane direction increased to prevent buckling.

In general, the truss model will incorporate gross section properties, or if the engineer so desires, some uniform reduction in gross properties to allow for some cracking. This 'uniform' approach to cracking and section properties is consistent with most reinforced concrete analysis, and not dissimilar to the S&T method.

However, some engineers may wish to allow for greater cracking of those truss members subject to the greatest tension forces. For a typical model with multiple load cases, trying to iterate by hand is impractical. Therefore, consideration should be given to using a non-linear finite element analysis program such as SAP 2000 Advanced or Ultimate, which can allow for a more complex, non-linear definition of material and section properties, including cracking under tensile load.

7 CONCLUSIONS

As an alternative to the Strut and Tie method for the analysis and design of diaphragms, the author has developed a Truss Method which allows complex diaphragms to be modelled and analysed in an efficient manner consistent with common design office methods. The diaphragm is modelled as a single truss, typically with multiple bays in both principal directions, and this single model can handle multiple load cases. This method effectively constitutes a type of linear finite element analysis using truss elements, and is also a type of strut and tie analysis. The analysis automatically determines the struts and ties for each load case. The results of the analysis are in a form that can readily be used to design the diaphragm elements.

Floor diaphragms form a critical component of seismic resistant buildings, as evidenced by the collapse of the CTV Building, which caused over 60% of the fatalities in Christchurch on 22 February 2011.

Unfortunately, in New Zealand many diaphragms have received little, if any, proper analysis, design and detailing, and this deficiency continues despite the CTV Building collapse.

In many instances, the very concept of the proposed diaphragm, and the layout of the vertical lateral load resisting elements, does not form a rational seismic resistant system.

It is essential that rational diaphragms and layouts of lateral load resisting elements are determined at the earliest stages in the architectural planning of a building, and that the fundamental principles of engineering such as equilibrium, positive load paths, resistance to buckling and proper detailing of reinforced concrete connections are applied to diaphragms in all instances.

Despite the apparent simplicity of diaphragms, accurately determining the loads and the resulting distribution of internal forces acting on them can be difficult and time consuming.

There are three main types of diaphragm action – 'inertial,' 'transfer' and 'compatibility.' Inertial loading is the direct horizontal seismic acceleration onto the floor. Research on the accelerations on individual floors is limited, and guidance in the form of 'parts and portions' and other provisions in loading standards can be too conservative or unconservative, depending on the position of the floor. Care must be exercised when determining which guidance to follow. Transfer forces acting where there are significant changes in floor layout and the stiffness of lateral load resisting elements between floors can be relatively straightforward to determine, but very large in size and difficult to design for. Compatibility forces within diaphragms caused as incompatible lateral load resisting elements such as shear walls and moment frames 'fight' each other can be very difficult to determine, and can be of very large magnitude.

The inevitable presence of lift, stair and service penetrations in floor slabs has meant that even apparently simple diaphragms should be analysed using relatively time consuming Strut and Tie (S&T) methods, until now. The Strut and Tie method requires the creation of a separate model for each load case. Therefore, in general, multiple models must be developed and solved for each diaphragm, and often additional preliminary analyses are required to determine an appropriate S&T model for each load case.

The Truss Method allows the use of a single model to analyse and design a diaphragm for all lateral load cases.

REFERENCES

Bull, D.K. 2004. Understanding the Complexities of Designing Diaphragms in Buildings for Earthquakes, *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol. 37, No. 2

Cowie, K., S. Hicks, G. MacRae, G.C. Clifton and A. Fussell 2013. Seismic Design of Composite Metal Deck and Concrete Filled Diaphragms – A Discussion Paper, *Steel Innovations Conference, 2013*

Cowie, K.A., A.J. Fussell, G.C. Clifton, G.A. MacRae and S. J. Hicks 2014. Seismic Design of Composite Metal Deck and Concrete-Filled Diaphragms – A Discussion Paper, *NZSEE Conference, 2014*

Gardiner, D.R., D.K. Bull and A.J. Carr 2008. Internal forces of concrete floor diaphragms in multi-storey buildings. *NZSEE Conference, 2008*

Gardiner, D.R., 2011. Design Recommendations and Methods for Reinforced Concrete Floor Diaphragms Subjected to Seismic Forces, PhD Thesis, *The University of Canterbury*

Moehle, J., J. Hooper, D. Kelly and T. Meyer 2010. NEHRP Seismic Design Technical Brief No. 3 – Seismic Design of Cast-in-Place Concrete Diaphragms, Chords and Collectors. *National Institute of Standards and Technology*

Paulay, T. and M.J.N. Priestley 1992. *Seismic Design of Reinforced Concrete and Masonry Buildings.* John Wiley & Sons

Reineck, K.H. (Editor) 2002. *SP-208 Examples for the Design of Structural Concrete with Strut and Tie Models.* American Concrete Institute

Sabelli, R., T. Sabol and W. Easterling 2011. NEHRP Seismic Design Technical Brief No. 5 – Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms. *National Institute of Standards and Technology*

Scarry, J.M. 2002a. *An Open Letter to IPENZ on the Parlous State of the Structural Engineering Profession and the Construction Industry in New Zealand,* First Version

Scarry, J.M. 2002b. *An Open Letter to IPENZ on the Parlous State of the Structural Engineering Profession and the Construction Industry in New Zealand,* Second Version

Wyllie, L.A. and R.W. LaPlante 2003. *The Designer's Responsibility for Rebar Design.* Concrete Reinforcing Steel Institute

AS/NZS 1170.5:2004 – Structural Design Actions, Part 5: Earthquake Actions – New Zealand. *Standards New Zealand*

NZS 3101:Part 1:2006 – Concrete Structures Standard. *Standards New Zealand*

NZS 3404:Part 1:1997 – Steel Structures Standard. *Standards New Zealand*

Microstran Version 9 – A general structural analysis program for frame and truss structures. *Engineering Systems Pty Ltd*

SpaceGass Version 11 – A general structural analysis program for frame and truss structures. *Integrated Technical Software Pty Ltd*

Ruaumoko – Program for the time history analysis of non-linear two-dimensional structures. *A.J. Carr/University of Canterbury*

Ruaumoko 3D – Program for the time history analysis of non-linear three-dimensional structures. *A.J. Carr/University of Canterbury*

ETABS – A finite element structural analysis program for buildings. *Computers and Structures, Inc.*

SAP 2000 Advanced/Ultimate – A general finite element structural analysis program incorporating non-linear material behaviour. *Computers and Structures, Inc.*