requires that the scaled record is only within a factor of about 1.5 of the target spectrum on average.

Amd 1 Sep '16 The period range from which the scale factors and measure of fit are calculated is $0.4T_1$ to $1.3T_1$, where T_1 is the largest translational period in the direction being considered, using material and section properties appropriate to the limit state under consideration. The upper limit $1.3T_1$ is intended to allow for some lengthening of the period during inelastic response, and the lower limit $0.4T_1$ is intended to allow the period range to include some higher-mode response.

This period range may be inappropriate for some forms of structure that have deliberate severe period-lengthening mechanisms, such as base-isolated or rocking structures, or even those where explicit consideration is being made of the lengthened soil-structure period associated with soil-structure interaction, rather than the rigid-base structural period. In these situations, the upper limit should be some scaling (exceeding 1.0) of the effective secant stiffness-based period of the system (not the structure alone) at the maximum displacement associated with the limit state under consideration. This may be much longer than the effective period in low-amplitude response before yielding (or sliding) of the isolators, before the initiation of rocking, or of the structure alone in the absence of soil-structure interaction. Consideration may also need to be given to the appropriateness of the lower limit for these types of systems.

C5.6 CAPACITY DESIGN

C5.6.1 General

The objective of capacity design is to ensure that in the event of a major earthquake a ductile failure mechanism can develop, which will enable the structure to survive the earthquake without collapse. This process requires the designer to select a suitable ductile failure mode and then proportion the structure so that other non-ductile failure modes cannot develop. With this arrangement, the strength of the potential inelastic zones limits the structural actions imposed on the other structural members or zones of members.

C5.6.2 Structures of limited ductility

For regular structures of limited ductility, some material standards have developed rules, which will ensure that in the event of a major earthquake a ductile failure mechanism will develop to the exclusion of non-ductile failure modes. These rules, however, are only valid for regular structures. For structures of limited ductility that do not meet the requirements set out in Clause 5.6.2, capacity design as set out in Clause 5.6.3 is applied to ensure that the structure has adequate deformation capacity provided to prevent collapse in the event of a major earthquake.

C5.6.3 Capacity design requirements for ductile structures

C5.6.3.1 *Potential inelastic zones*

For multi-storey buildings, where the lateral force resistance is provided by moment resisting frames, the selected potential ductile failure mechanism is generally based on the beam-sway mode (see Appendix C). For buildings where lateral resistance is provided by walls, the selected failure mechanism generally involves the development of plastic hinges at the bases of the walls. For buildings, where lateral resistance is developed by braced frames, the failure mechanism involves the braces or eccentric links in the beams between the offset braces. Permissible ductile failure mechanisms are specified in appropriate material Standards. These mechanisms are both of structural form and material dependent. A key part of capacity design is to identify the potential inelastic zones and then detail these zones so that they can resist the required deformation without significant loss of strength.

C5.6.3.2 *Deformation of potential inelastic zones*

The magnitude of the deformation that an inelastic zone can sustain depends on the level of detailing that is used. For example, the deformation that can be sustained by a steel beam depends upon how potential buckling of the inelastic zone is controlled. Thus the required level of constraint against buckling increases with the magnitude of the inelastic deformation that the zone is required to be capable of sustaining.

The level of detailing required in potential inelastic regions is defined in the appropriate material Standard. In some of these Standards the level of detailing is based on the magnitude of the material strains induced in the inelastic zones by the ultimate limit state seismic actions. In other material Standards the structural ductility factor used in the analysis for ultimate limit state seismic actions is used to define the required level of detailing. As indicated in Appendix C, unidirectional plastic hinges are required to sustain appreciably greater plastic rotation demands than reversing plastic hinges. Allowance for this increase in deformation needs to be made. Material Standards indicate the relative plastic deformation that may be sustained in unidirectional and reversing inelastic zones. Amd 1 Sep '16

C5.6.3.3 *Overstrength actions in potential inelastic zones*

To ensure that the intended ductile failure mechanism develops in preference to other failure mechanisms, the maximum likely strength, known as the overstrength, that each potential inelastic zone can sustain is evaluated. Material Standards define how these overstrengths are calculated. In this calculation, the overstrengths should be assessed from the combinations of actions, which allows the most critical actions to be transmitted to the adjacent zones.

C5.6.3.4 *Design outside inelastic zones*

The remainder of the structure is then proportioned to have a strength greater than the maximum actions that can be induced at any section assuming that overstrength actions act in one or more of the potential inelastic zones.

With the formation of plastic zones in a structure its dynamic characteristics change. Unless this change is recognized and allowed for, non-ductile failure mechanisms may develop. The changes in the actions induced in members, or regions of members outside inelastic zones, is allowed for either by the use of:

(a) Dynamic magnification factors, which are specified in material standards; or by

(b) Nominated distribution of structural actions.

In either case, the values are specified in the appropriate material Standard. An example of the use of a dynamic magnification factor is in the design moments for columns in moment-resisting ductile frames is given in NZS 3101. An example of the use of normal distribution is in the capacity moment diagram specified for multi-storey structural walls in NZS 3101. Appendix C gives further details on dynamic magnification factors for the design of columns in multi-storey ductile frame structures.

As illustrated in Appendix C in multi-storey moment-resisting frames appreciable enhancement of column strengths is required to prevent the premature collapse from occurring due to the formation of a mixed beam-column sway mode involving two or three storeys.

Appendix C outlines the basic concepts behind dynamic magnification factors and methods of defining required strength distribution. The required level of strength enhancement varies with the material that is used. For example with reinforced concrete construction a higher level of strength enhancement is required than for structural steel, as this gives a high level of protection against plastic hinge formation in reinforced concrete columns. This approach allows:

(a) Bars to be lapped in this zone, which is advantageous for construction.

(b) Reduces the amount of confinement reinforcement that is required.

Columns that form part of more than one moment resisting frames are subjected to bi-axial bending from the beams framing into it. In assessing the design actions in the column it should be assumed that over strength actions are sustained simultaneously by beams framing into the column from both frames. However, it is unlikely that the full dynamic magnification factor acts with over strength actions one axis while un-amplified (dynamic magnification factor is 1.0) over strength actions act on the second axis.

Amd 1 Sep '16

C5.7 DIAPHRAGMS

C5.7.1 General

Diaphragms are typically suspended floors that are relatively thin horizontal structural systems capable of resisting lateral force. Diaphragms transfer inertial forces from themselves and connected elements, such as stairs and services connected to them, to the lateral force resisting structural systems. They may also resist differential in-plane movement of the lateral force resisting structural systems.

Diaphragms perform a number of functions while supporting gravity loads and resisting lateral forces: predominantly quoted from Moehle et al (Ref. 10):

(a) Support gravity loads – Most diaphragms are part of the floor and roof framing and therefore support gravity loads;

(b) Provide lateral support to vertical elements – Diaphragms connect to vertical elements of the seismic force resisting system at each floor level, thereby providing lateral support to resist buckling as well as other second-order demands associated with axial forces acting through lateral displacements;

(c) Complete framework for lateral force resistance – By tying together the vertically oriented elements that are lateral force resisting, diaphragms complete the three-dimensional framework to resist lateral loads;

(d) Transfer lateral inertial forces to vertical elements of the seismic force resisting system – The floor system commonly comprises most of the mass of the building. Consequently, significant inertial forces can develop in the plane of the diaphragm. One of the primary roles of the diaphragm in earthquake-resistant buildings is to transfer these lateral inertial forces, including those due to tributary portions of walls, and columns, above and below the diaphragm, to the vertical elements of the seismic force resisting system;

(e) Resist out-of-plane forces – Structural and non-structural elements develop out-of-plane horizontal inertial forces as a building responds to an earthquake. As structural components deform elastically or inelastically, diaphragms and their connections need to resist the imposed forces. The imposed forces may be into, or out of, the building. Out-of-plane forces from wind must also be resisted;

(f) Resist thrust from inclined columns, ramps and stairs – Architectural configurations sometimes require inclined columns, which can result in large horizontal thrust forces acting within the plane of the diaphragms, due to gravity and overturning actions, as well as bending forces. The thrust forces can act either in tension or compression, depending on the orientation of the column. The diaphragm or components within it need to be designed to resist these thrust and bending forces. Such forces from ramps or stairs, spanning between levels of a structure, can induce significant actions in the respective floor diaphragms;

(g) Deformation compatibility forces through the diaphragm – As a building responds to earthquake loading, the diaphragm often transfers forces between the different vertical elements of the seismic lateral force resisting system.

When horizontal displacements of lateral force resisting elements of the structure are constrained to a common profile by the presence of a diaphragm, the diaphragm may be subject to significant forces referred to as "deformation compatibility forces"; Amd 1 Sep '16

(h) Deformation compatibility forces also occur when there is a difference in stiffness and/or strength amongst the vertical elements of the lateral force resisting systems such as when there is:

(i) Vertical irregularity resulting from podium-towers, set-backs, etc.

(ii) Horizontal irregularity when diaphragms are connected to three or more elements of the lateral force resisting systems in any direction, and these elements would have significantly different horizontal displacements if the diaphragms were not stiff. For example, structures with a combination of walls and frames, braced frames and moment frames, or with different length walls that are connected to the diaphragm, would be expected to have significant deformation compatibility forces.
These compatibility forces exist in most diaphragms of most buildings, with different values and orientations (i.e. tension or compression) up the building height.
The largest compatibility forces commonly occur at discontinuities in the vertical elements, including in-plane and out-of-plane offsets in these elements. A common discontinuity is at a ground floor suspended slab, above a basement. The tendency is for a majority of the shear in the stiffest lateral force resisting systems above the ground floor to transfer through the ground floor slab to the basement walls. The deformation compatibility forces of this particular example are traditionally referred to as "transfer forces".

(i) Support soil loads and hydrostatic forces below grade – For buildings with below-ground levels, soil and water pressure bears against the basement walls out-of-plane. The basement walls span between diaphragms, producing compressive reaction forces at the edge of the diaphragms.

A further issue arising from both soil and water pressure is the capacity of the lowest basement slab, which is on-grade, to resist uplift pressures.

In determining the design actions and the in-plane distribution of the internal load paths in diaphragms, the following need to be considered:

(j) Transfer of the required in-plane actions within the diaphragm, through any connections, and to the supporting elements of the lateral force resisting structural system;

(k) Diaphragm stiffness;

(l) Effects of re-entrant corners and floor penetrations;

(m) Reduction of integrity or viability of potential load paths from displacement induced diaphragm damage.

Diaphragms are required to resist the design loads, and also, when required, to redistribute these within the diaphragms in a manner that is consistent with the design assumptions that have been made. The simplest way to achieve this is to design diaphragms to respond in a nominally elastic manner. That is, there should be virtually no plastic deformation within the body of a concrete diaphragm during seismic action. There may be some localized permanent deformation (e.g. some plasticity or sliding) where the vertical elements of the lateral force resisting structural system connect to the diaphragm.

Having limited localized plasticity within the body of a concrete diaphragm may be permitted, providing:

Amd 1 Sep '16

(n) Localized plasticity is not being relied upon for redistribution of actions nor energy dissipation;

(o) Localized plasticity does not negate the outcomes of the analysis of the structure, based on the assumption of modelling the diaphragm as an elastic element;

(p) Localized plasticity does not compromise the gravity supporting role of the floor plate.

If localized plasticity of a diaphragm is expected, this should be considered in the modelling to ensure that element and connection capacities are sufficient to maintain the required load paths. One example where such diaphragm plasticity may occur, due to the beam elongation effect that occurs at that location, is in plastic hinges of reinforced concrete beams – see Figure C5.2 (Report of Canterbury Earthquakes Royal Commission, Ref. 4). The extent of diaphragm plasticity is generally significantly less, but can also occur to some extent in hinges of steel or timber beams depending on the detailing (Kam et al., Ref. 7 and Moehle et al., Ref. 10),

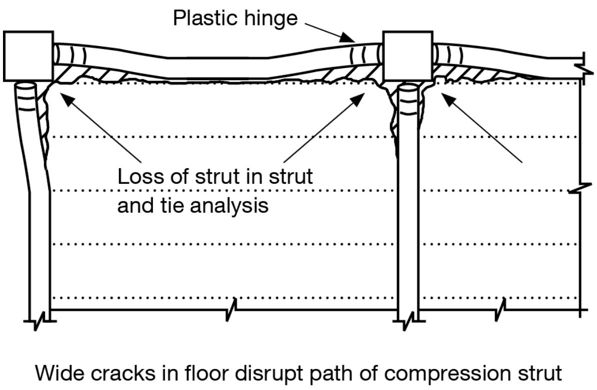

FIGURE C5.2 PLASTIC HINGE ZONES IN THE BEAMS OF A REINFORCED CONCRETE FRAME (REF. 4)

Ductile yielding is permitted in a diaphragm when the diaphragm has been detailed to provide a ductile response. Examples of permitted ductile yielding include nail slip in timber diaphragms, and yielding of diagonal braces in roof structures.

Most computer-based analysis tools work on the premise of elastic diaphragms. Consideration of extensive, widespread inelasticity within a diaphragm may require advanced structural modelling.

Rigid diaphragms, or diaphragms with high in-plane stiffness, will increase the likelihood of lateral deformations in elements of the lateral force resisting system being similar (in structures without significant torsion about the vertical axis). Flexible diaphragms result in lateral force resisting structures deforming to profiles closer to that expected when the elements of the lateral force resisting system are loaded independently of one another. Very flexible diaphragms will result in force distributions to the structural elements following tributary area of floors to each structural element. Rigid and flexible diaphragms are defined in Appendix A and this flexibility should be included in the analysis. The largest compatibility or transfer forces will result in the diaphragms (for a given structure) defaulting to a rigid diaphragm in the modelling. The use of an appropriate stiffness for the diaphragms can alleviate some of the compatibility effects, thereby reducing the diaphragm forces accordingly.

The shape of a diaphragm and the presence of floor penetrations will dictate how the internal load paths are established. Re-entrant corners, in "L", "C", and "T" shaped floor plans, can result in stresses and forces being concentrated at changes of shape of the diaphragm. For example, the behaviour of the area of floor where the two wings of an "L" shaped floor plate meet is akin to a reinforced concrete beam-column joint in terms of high stresses and necessary distribution of reinforcement (as resolved using strut and tie methods). Realistic estimation of the demands at such corners can be complex, being affected by higher modes and non-linearity of the structure. The problem may be avoided by designing structures of this type as rectangular structures connected with seismic gaps as shown in Figure C5.3(d). If this procedure is not adopted, then appropriate modelling should be undertaken to assess demands and appropriate detailing provided to prevent damage. Amd 1 Sep '16

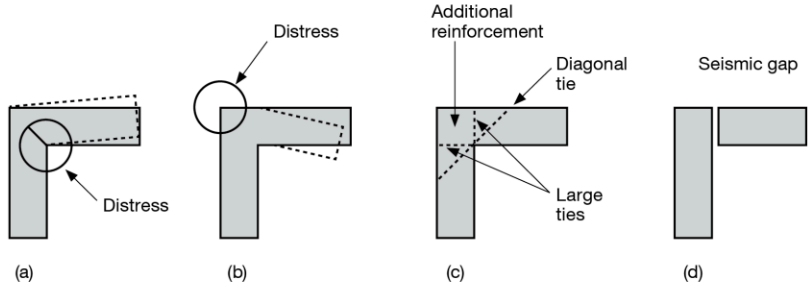

FIGURE C5.3 PLAN OF "L" SHAPED FLOORS – HIGH STRESSES AT JUNCTION OF WINGS (MODIFIED FROM REF. 12)

In some cases, where the displacement compatibility forces are significant, there can be damage to the diaphragm, and loss of potential load paths can occur. Examples of regions where load paths are reduced in capacity are:

(a) In frames with ductile reinforced or precast concrete beams. As lateral displacement occurs, the beams elongate plastically causing damage in the diaphragm at the locations of beam plastic hinges. If the elongation is sufficiently large, the columns supporting the beams will be pushed away from the floor plate resulting in a gap between the floor and supporting beams. Forces cannot cross this gap (Figure C5.2).

(b) Structural concrete walls, or rocking walls of any material, connected to the floors. Here, a ductile wall will elongate, vertically, in the plastic hinge zone at the base of the wall. This results in a temporary uplift displacement of the wall, thereby lifting the floor at the wall-to-floor connection. Similarly, if a wall rocks on its foundations, the wall will displace vertically before returning to the original "at rest" position. In both cases the maximum uplift of the floor is at the ends of the wall element, at maximum lateral drift of the structure. See Figure C5.4.

    In structures with mixed systems of vertical support, when a floor is lifted by a wall yielding flexurally or rocking at its base, the floor and supporting beams framing into the wall are subjected to bending and shear in conjunction with any diaphragm (in-plane) forces. The combined actions need to be considered in design to be provided.

    It should be noted that the increase in height of the wall due to elongation can substantially increase the axial load and the lateral strength of the wall, while at the same time inducing tension, or reducing axial compressive loads, in neighbouring vertical structural elements.

Alternatively, it may be possible to detail the wall-diaphragm connection such that there is no uplift of the floors (Report of Canterbury Earthquakes Royal Commission, Ref. 4).

Amd 1
Sep '16

The floor next to the wall is lifted.

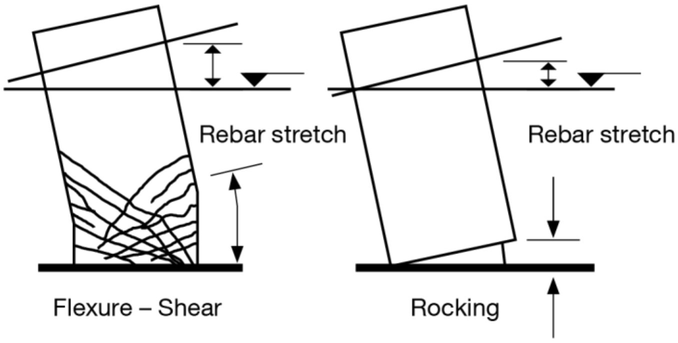

FIGURE C5.4 ELONGATING CONVENTIONAL REINFORCED CONCRETE WALL AND ROCKING WALL (SINGLE CRACK) (REF. 3)

During major earthquakes, the performance of load paths across diaphragms and connections from diaphragms into lateral force resisting structural systems have not always been adequate. A common cause of this poor performance was the lack of strength, or the lack of deformation capacity when the connection was displaced or distorted. The lack of deformation capacity arises from the imposed strains exceeding the strain capacity of the connection (or part of the connection assembly). This effect is more significant when structural materials are inherently brittle (Kam et al., Ref. 7).

The design or assessment methods employed in a design office are approximations of anticipated structural behaviour. The magnitudes and directions of forces in diaphragms are estimations. Considering the simplifications used for determining earthquake-induced forces, robustness of the load paths into, out of and across diaphragms is needed.

One way of enhancing this robustness is to ensure that brittle failures along the load paths are avoided. One aspect of this avoidance is to use materials and details that are not brittle and can accommodate deformations that are imposed. For example: the steel bar reinforcement in the floor plates could be Grade E steel (NZS 4671), as against cold drawn wire mesh, the connection from the structural frame to a concrete cladding panel should have a part of the connection that precludes concrete cone pull-out from the panel – this is done by limiting the force in the load path (e.g. large plasticity, or sliding on interfaces); by a specifically detailed element to accommodate plasticity, such as a cast-in-place concrete infill strip between beams of the frame and precast concrete floor units or nail plates along the edge of a timber diaphragm. Both strength and displacement capacity of a load path need to be considered. Displacement capacity is not synonymous with "ductility":

(a) If there is a relative displacement between elements that needs to be accommodated, simply having a ductile bolt, or reinforcing bar, or structural steel section crossing that gap as a tie, may not be sufficient.

    For example, a gap may form between the topping of a precast floor and the supporting beam, and if wider than 15 – 20 mm, it is likely that the continuity bars, between the beam and the floor topping, will fracture.

(b) There may be a Grade 4.6 bolt, in tension, in the load path. This bolt might be considered ductile, yet 1 – 2 mm extension of the bolt shank will lead to fracture of the bolt.

    Ductility of an element, in a sub-assembly of a connection, will not necessarily mean the connection can act as a force-limiting mechanism. In conjunction with the strength of the elements of a connection, the displacement of the elements needs to be considered in terms of strain or deformation capacity of those elements

    Specially designed connectors need to maintain strength (force transfer) while accommodating the required displacements.

C5.7.2 Design actions

Amd 1
Sep '16

An interim method for establishing floor inertia forces has been proposed in Appendix A of C5.7. The method (pseudo-equivalent static analysis (pESA)) has limitations of use being for concrete frame and dual structures, of nine storeys and less in height (Gardiner, Ref. 6). Currently there are other research/development programmes working on alternative methods that are aiming to deal with tall structures and structures other than reinforced concrete frames and walls. One example is an MBIE project on extending pESA (in preparation).

In taller buildings applying minimum inertias of peak ground acceleration as per Appendix A of C5.7 becomes impractical and inappropriate, being overly conservative. The pESA method may underestimate peak actions from inertia attributed to higher modes (though having little effect of deformation compatibility forces). Depending on the structure type, a pESA-type method similar to that outlined in Appendix A to C5.7 (but without the PGA limitation) may still be useful for the case controlling deformation compatibility forces in a tall structure.

The design actions to be carried by a diaphragm, typically being floors or a roof, arise from a number of sources, related to the functions of diaphragms described in C5.7.1:

(a) Diaphragms must carry gravity loads to supporting structures.

(b) The design accelerations to be resisted by a diaphragm and transmitted to vertical force resisting structural systems arise from inertia of the components of the structure. Typically, the horizontal inertia forces govern the design. Vertical accelerations and horizontal accelerations in a building need not be considered concurrently (Singh et al., Ref. 13). There may be situations where parts of the structure will be sensitive to vertical accelerations, and as such, will need to be considered.

(c) There will be forces that develop internally within a diaphragm arising from vertically oriented elements that are lateral force resisting systems being constrained via the diaphragm to a common lateral cement profile – as described in C5.7.1. These forces are known as "deformation compatibility forces". Depending on the configuration of the building, the compatibility forces may be considerably larger than the inertia force for that particular diaphragm.

(d) Interaction with elements supporting the diaphragm produces actions associated with the functions described in C5.7.1:

    (i) Provide lateral support to vertical elements;

    (ii) Resist out-of-plane forces;

    (iii) Resist thrust from inclined columns, ramps and stairs.

    Further, the elements of the structure supporting the diaphragms can impose deformations on the structure of the diaphragm, over and above the inertia and deformation compatibility forces described here. Deformation compatibility issues between the diaphragm and supporting structure arise when the support element displaces or deforms, imparting those deformations into the adjacent diaphragm element. Examples of issues are, but not limited to:

    (iv) Torsion across the floor plate when one end of that section of floor is resting on a beam, while the other end is lifted by the vertical elongation of a wall or by the vertical rocking of a wall or that of a rocking steel-braced frame.

    (v) The relative rotation between a floor and supporting beam, where the beam is rotated about its longitudinal axis (the beam is transverse to the main lateral displacement of a building, supporting a floor), induces flexure, shear and possibly tension or compression stresses in the plane of the floor (Fenwick et al., Ref. 5).

Amd 1
Sep '16

(vi) Elongating reinforced beams, rocking concrete or structural steel or timber beams can cause flexure, shear, and tension in the plane of the floor.

(vii) Rocking walls, rocking braced frames, the ductile links of an eccentrically braced frame and deformation of a beam next to a floor element will impart flexural and shear deformations to the adjacent floor.

(viii) Concrete coupling beams between structural walls will elongate and push the walls apart. This elongation results in tension forces occurring in the floor plate next to the walls, parallel and at right angles to the walls (these are a result of the tension, parallel to the wall, being eccentric from the plane of the walls and diagonal compression fields transfer from the walls to meet the parallel tension force, thereby being required to be balanced by tension forces at right angles to the walls – this is a strut and tie solution).

(e) As well as connections to the vertical lateral force resisting elements, some structural configurations for diaphragms will have discrete connections between parts of the diaphragm. For example untopped, precast concrete units can be connected by weld-plates. Deformations, as well as strength demands, between the diaphragm elements need to be considered when assessing the load paths.

Where there are abrupt discontinuities, major variations in in-plane stiffness, or major re-entrant corners in diaphragms, the assumption of a rigid diaphragm may not be valid. In some cases, investigation of these effects may require the stiffness of the diaphragm to be modelled in the analysis to ensure that a realistic distribution of lateral force has been obtained.

In some types of mixed systems of walls and moment-resisting frames, shear forces applied to some vertical elements at some levels can be of the opposite sign to the overall inter-storey shear.

The design or assessment of diaphragms is a complex matter. Considerable judgement is needed in the analysis and detailing of diaphragms. The use of simple "deep beam" theory may not be appropriate for floor plates, including roofs, which have re-entrant corners or penetrations. The traditional use of beams as tension chords may be tenuous as these beams are active in frame action during the event and may not be available to act as tension chords as some of the capacity of the longitudinal reinforcement of the beams may be used to resist displacements from oblique seismic attack. Further, as part of any earthquake input to a structure, there are dominant directions of displacement as well as components transverse to the main direction. Therefore, reliance on beams, oriented orthogonal to the theoretical direction of earthquake attack to act as tension chords, may be non-conservative.

It is recommended that bands or zones of tension capacity be provided within the diaphragms to act as chords. Alternatively, if beams are used as tension cords, the additional tensile forces must be included in the design of the beam section and the beam connections (Bull, Ref. 2).

The use of elastic time history analysis or modal analysis that output envelopes of maxima actions will not provide relevant information for detailing the load paths across the diaphragms into the vertical structures. These maxima do not provide actions that occur together at any one point in time, nor produce a vector sense for the action. The designer is interested in floor forces in a real-time context. The same limitations on outputs apply to most non-linear time history analyses.

It is inappropriate to attempt a separate analysis of the inertia effects and deformation compatibility effects. The deformation compatibility actions and the inertias are coupled analytically – that is, inertia causes the building to deform and it is the incompatibility of deformed shapes of each vertical structural system that generates the transfer forces across the diaphragms. These actions occur together.

In a number of instances, however, it is likely that transfer effects may contribute most to the actions in one given diaphragm, whilst inertial actions contribute most in others. These two cases may not occur concurrently. An example would be a tall building fixed into a stiff podium structure. Inertia and displacements in the fundamental sway mode (possibly reaching building overstrength) will usually give rise to the highest actions in the podium diaphragms, and the highest deformation compatibility actions in the adjacent levels. At a different point in the earthquake, higher modes may result in large inertias in upper floors, but which do not sum to high base shears or high overall displacements and therefore do not impose the largest transfer actions on podium levels. The designer may need to consider these cases separately if an equivalent static procedure cannot cover both. Such circumstances are most likely where the building exceeds the current height limitation of the pseudo-equivalent static analysis (pESA) proposed in the following paragraphs, and as described in Appendix A to C5.7, (in particular as the building overstrength base shear drops below PGA). Amd 1 Sep '16

Designing floor plates with the strut and tie solution (Bull, Ref. 2), described sometimes as an "equivalent truss method", deals with the geometries that are variable in architecture and simplifies the determination of load paths across diaphragms and into vertical structural systems.

An equivalent truss method can be applied to light timber frame diaphragms (Malone et al., Ref. 9) and also to solid timber panel diaphragms. Irregular shaped timber diaphragms can be designed using the deep beam analogy in combination with transfer diaphragms (or sub-diaphragms). It is recommended that concrete diaphragms be designed using strut and tie solutions (NZS 3101).

For determining the actions in the structure, it has been suggested in Refs. 2 and 5 that the actions for the structure should be based on capacity design principles. In order to visualize the load paths through the structure, in any design method, it is imperative that equilibrium is maintained across diaphragms, accounting for the interaction of the vertical structure systems via the diaphragms and distribution of the inertia across the floor plates.

It is suggested that a pESA method may be employed. Such a method uses lateral forces applied to the structure, based on amplified forces from the equivalent static analysis (ESA) method of the Standard (see Appendix A to C5.7).

When a structure is required to be ductile, with plastic mechanisms forming during a severe event, the floor forces (the inertia of each floor, in effect, determined for each ultimate limit state (ULS)) should be factored up by the building overstrength factor, as described here:

The building lateral overstrength, accounting for plastic mechanism development in the components of the structure, as designed and detailed, divided by the strength demand of the ULS, determined from this Standard. The building overstrength factor will typically be greater than 2.

For a structure that is designed at ULS as a structure of nominal ductility ($\mu \leq 1.25$), increase the floor forces above that of ULS design earthquake event by the following amplification factor:

$$\text{The amplification factor} = \frac{S_{\text{n,prov}}}{S_{\text{E}}} \cdot \frac{0.93}{S_{\text{p}}} \cdot \frac{\text{95\%ile characteristic yield strength}}{\text{5\%ile characteristic yield strength}}$$

where:

The provided nominal lateral strength of the building, as designed and detailed (the provided nominal capacity) is $S_{\text{n,prov}}$, and the strength demand on the building for the ULS, determined from this Standard, is S_{E}.