# Seismic design of composite metal deck and concrete-filled diaphragms – A discussion paper

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**ABSTRACT:** One of the most neglected elements in the design of buildings is the horizontal floor diaphragm and its interaction with the lateral load resisting systems. Most multi-story structures depend on the floor slab and roof systems to act as horizontal diaphragms to collect and distribute the lateral loads to the vertical framing members, which provide the overall structural stability.

In steel structures, floor diaphragms are most commonly constructed using composite steel deck with concrete fill, although other systems may also be used. Somewhat surprisingly, given the importance of diaphragms to the overall building response, there is no universally agreed design procedure for determining the diaphragm actions and distribution into the seismic-resisting systems. In addition, the specific issues related to beam design for members collecting lateral loads in composite floor systems has gone largely undocumented.

This discussion paper presents a suggested method in determining the design diaphragm actions at a given floor level, how to proportion their transfer into the seismic resisting systems and how to design and detail the supporting beams/composite metal deck for these actions.

## 1 INTRODUCTION

One of the most neglected elements in the design of buildings is the horizontal floor diaphragm and its interaction with the lateral load resisting systems. Most multi-story structures depend on the floor slab and roof systems to act as horizontal diaphragms to collect and distribute the lateral loads to the vertical framing members, which provide the overall structural stability.

This discussion paper presents a suggested method for determining the design diaphragm actions at a given floor level, how to proportion their transfer into the seismic resisting systems and how to design and detail the supporting beams/composite metal deck for these actions. This methodology is based on a North American publication modified for New Zealand practice. This publication is entitled "*Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms*" and was produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg. While this guide (Sabelli et al, 2011) contains much useful information there is still additional guidance required for a New Zealand application. Readers of this paper are encouraged to read the guide, which is available on the internet at: www.nehrp.gov. This discussion paper is written to fill in some of the 'gaps'. This paper only considers simple regular floor plates. Sources of guidance for more complex floor arrangements such as large openings and re-entrant corners are referenced in the body of this paper for both non-transfer and transfer diaphragms.

## 2 DETERMINING THE DESIGN DIAPHRAGM ACTIONS AT A GIVEN FLOOR LEVEL

The floors in a multi-storey building with a composite metal deck are typically designed as rigid elastic diaphragms, which couple all the seismic-resisting systems together and enforce behaviour of the floor as a rigid unit. The forces generated in a diaphragm come from principally two sources simultaneously (Bull 2004) – inertia and the forces generated by the vertical lateral force–resisting systems in the building "fighting" each other in terms of deformation compatibility between each lateral load resisting system and the overall structure. Especially when the deformed shapes of each vertical lateral force-resisting system are inherently incompatible (eg combination of MRFs and shear walls) these incompatibility forces can be very large.

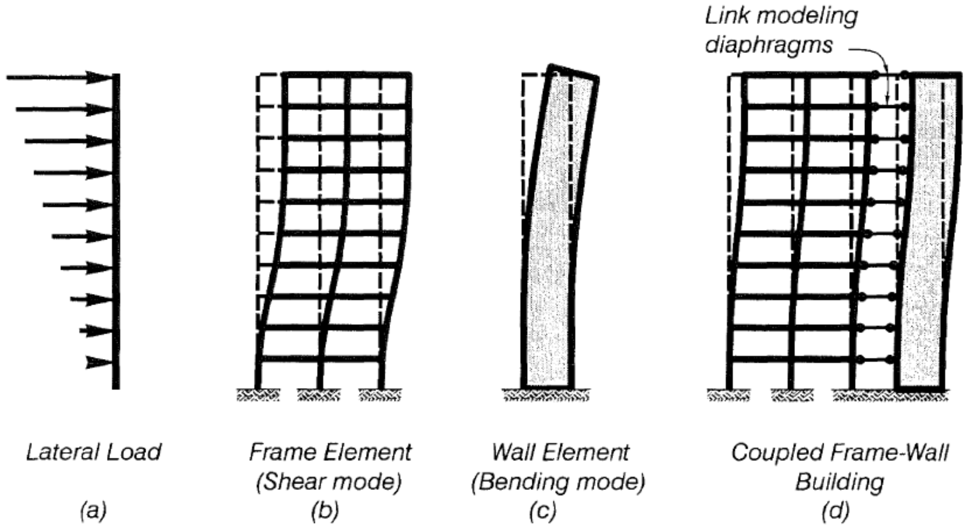


**Figure 1. Deformation compatibility of a dual frame. (Bull, 2004)**

### 2.1 Diaphragm Actions for a Floor that is not a Transfer Structure

One method to determine the diaphragm forces at a given floor level is using the part and components method of NZS 1170.5. This approach is suitable only for floors where the floor does not act as a transfer diaphragm.

The peak ground acceleration is magnified up the building by the influence of the more lightly damped superstructure. The maximum value of the parts floor height coefficient $C_{Hi}$ is given in NZS 1170.5 Clause 8.3. However research presented at the 2009 NZSEE Conference by Uma et al (Uma, Zhao et al. 2009) showed this factor to be overly conservative. Figure 4 of their paper shows that a value of 1.6 would correspond to the maximum floor acceleration for the 4 storey to 11 storey structures they analysed and that value is being recommended to be used. That factor accounts for the inertial force contribution only, not the incompatibility of deformations contribution. However, it gives design shears that are as high or higher than those generated by the method being developed by Bull (2004) and so is likely to be adequate for both contributions. It is also simple to apply and is applied on a floor by floor basis. Because the inertial diaphragm actions are generated by accelerations at the floor level and maximum floor accelerations are near constant over the full height of the building, the value of $C_{Hi,\ diaphragm} = 1.6$ is applied at all floors. Note that in a paper at the 2012 NZSEE Conference by Dantanarayana et al (Dantanarayana et al 2012) this value may not always be conservative and further research is required to confirm the value. In setting this value however it is important to note the duration over which high floor accelerations apply; in an elastically modelled diaphragm, the peak recorded accelerations will occur for very short time intervals (fractions of a second) which are too short to physically generate yield.

The expression for the design diaphragm shear force at level i is given by equation 1:

$$V_{dia,i} = C_{dia} W_{t,dia,i} \tag{1}$$

where:

$V_{dia,i}$ = the design diaphragm shear force for the floor at level i

$$C_{dia} = C_{h0\,\mathrm{mod}al} Z R_u S_p C_{Hi,diaphragm} \tag{2}$$

$C_{h0\,\mathrm{mod}al}$ = the spectral shape factor for T=0 seconds for the modal response spectrum from Table 3.1 of NZS 1170.5 (ie the bracketed values to be used)

$Z$ = the zone factor

$R_u$ = the return period factor from Clause 3.1.5

$S_p$ = the structural performance factor for the category of structural system from NZS 3404 Clause 12.2.2.1(b)

$C_{Hi,diaphragm}$ = 1.6

This diaphragm shear force is applied through the centre of mass at level i in and distributed into all seismic-resisting systems parallel to the direction of motion under consideration.

#### 2.1.1 *Comparison with ASCE 7-10*

In the USA the design forces for diaphragms are contained in ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010). The forces are defined by three equations. The first (Eq 12-10.1) is constructed from the storey force derived from the equivalent lateral force distribution and thus relates to the design base shear. The second two equations (Eq 12-10.2, 12-10.3) are minima and maxima. The maxima diaphragm force equation is

$$F_{px} = 0.4 S_{DS} I_e W_{px} \tag{3}$$

where:

$0.4S_{DS}$ = the peak ground acceleration, which is the spectral acceleration at T = 0 seconds

$I_e$ = the importance factor

$W_{px}$ = the weight tributary to the diaphragm at Level x

The components of equation 3 are compared with equations 1 and 2 above. The result is that the USA maxima equation is based on the value of $S_p C_{Hi,diaphragm}$ equal to one. The value of $S_p C_{Hi,diaphragm}$ in equation (2) is equal to at least 0.7 x 1.6 = 1.12. Therefore the proposed equation for the diaphragm force is larger than the maxima limit stated in ASCE 7-10.

### 2.2 **Diaphragm Actions for a Floor that is Transfer Structure**

For diaphragms that are transfer diaphragms, the overstrength capacity of the seismic resisting system between the levels being transferred are added to the general design actions from above using the square root of the sum of squares approach (SQSS). The maximum inertia and transfer forces are unlikely to occur simultaneously.

## 3 INTERNAL COMPONENT FORCES

Once the building is appropriately modelled and diaphragm inertial and transfer forces are determined, an analysis of the internal forces within the diaphragm must be done in order to determine the design forces for the diaphragm components. There are several analytical methods. The diaphragm analysis method should use diaphragm inertial and transfer forces that are consistent with those in the building analysis.

### 3.1 Deep Beam Analogy

The deep beam analogy considers the diaphragm spans between vertical elements of the lateral load resisting system that act as lateral supports. The diaphragm has in effect tension and compression chords and the web resisting shear.

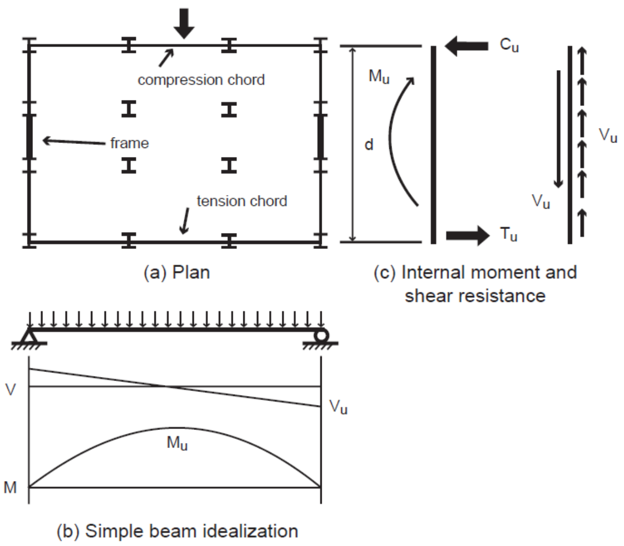


**Figure 2. Deep Beam Analogy (Sabelli et al, 2011)**

The compression chord is through the concrete slab at the compression face of the beam and if there are discontinuities there then the designer must determine an appropriate load path around those discontinuities, in accordance with the following principles:

1. The loads will follow the stiffest load path
2. Be aware of component forces introduced when a compression load changes direction and ensure that slab reinforcement is put in to resist any tension forces introduced when the compression chord force changes direction (e.g. as required to flow around an opening in the slab)

The tension chord must be assessed using the same two principles, but in this case the tension forces can flow either through the steel frame or through reinforcement in the slab. They will follow the stiffest load path. If there is a moment resisting seismic frame along the edge of the slab they will go through that; if it is a gravity system only the stiffest path is through reinforcement in the slab. If the decking is running parallel to the tension chord then the decking will provide a very stiff load path except at the joins.

Collectors, or drag struts, occur where the deck forces are transferred to a frame line over a partial length, that is, where the beams that are part of the braced or moment frame do not extend the full depth of the diaphragm. This is illustrated in Figure 3a at the outer frame lines. The remaining spandrel members in Figure 3a are attached to the deck through fasteners collecting inertial forces from the deck and in turn delivering those forces to the frame members. These collector members must transfer the forces to each other across their connections to the columns. Collector forces are illustrated in Figure 3b.

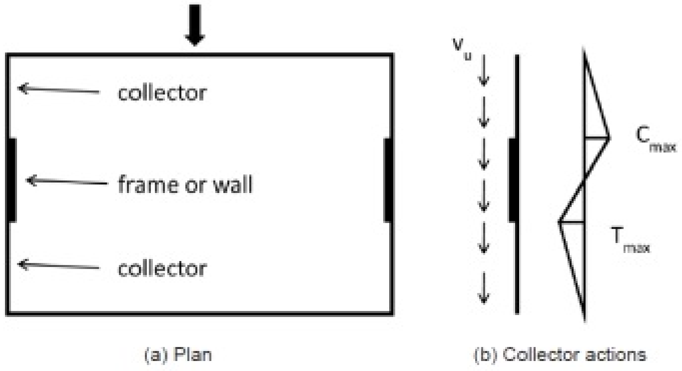


**Figure 3. Diaphragm Collectors. (Sabelli et al, 2011)**

For more complicated floor and lateral system arrangements refer to guidance from (Sabelli et al, 2011). A simplified truss based approach may also be used; refer to a paper by Scarry also being presented at this conference (Scarry, 2014).

### 3.2 **Local Effects at Discontinuities**

Discontinuities in the diaphragm, such as openings, steps, and reentrant corners, require consideration by the designer. The specifics for designing for these are beyond the scope of this paper. For further guidance refer to (Sabelli et al, 2011).

Openings in a diaphragm should be located to preserve as much of the overall diaphragm as possible about any axis. Designers should avoid locating openings in such a way that narrow sections of diaphragm are used to connect different parts of the diaphragm, because of the large forces that must be transferred through the small section of remaining diaphragm. For similar reasons, clusters of diaphragm openings at reentrant corners or along a single edge of the diaphragm should be avoided. It is generally better to locate diaphragm openings so that they are surrounded by substantial portions of the diaphragm or are separated as much as possible. Openings in the diaphragm may be idealized similarly to openings in the web of a steel beam. The impact of the reduced area on the portions of the diaphragm that remain must be taken into account as well as the need to transfer forces around the opening. In some cases, additional walls or frames may be required in order to prevent isolating one section of the diaphragm from another.

## 4 COMPONENT DESIGN

### 4.1 **Should diaphragm component forces be combined with other design checks?**

Diaphragm component actions are considered separate to other design actions checks. All components used in the diaphragm design are checked for adequacy to resist the diaphragm design actions and have a dependable load path for each of these checks. The diaphragm checks are carried out in isolation to any other role of those components. The rationale for this is principally based on four factors:

1. The diaphragm design actions required from above are reasonably conservative
2. It is unlikely that those peak actions will combine with peak design actions from other design scenarios,
3. When it comes to the action effects generated on structural elements from diaphragm actions compared with those generated by other design actions, in some elements the action effects are

in opposition, thereby making the interaction effect less than the effect of each considered separately, while in other elements the action effects are cumulative. Thus in a typical floor system the interaction will only overload some elements in the building system at any one time.

4. Well designed and detailed diaphragms and building systems designed to less than the above proposed provisions have performed well in recent severe earthquakes, especially including the Christchurch earthquake series of 2010/2011 which tested diaphragms designed and built to New Zealand practice.

### 4.2 Longitudinal Shear Strength of Composite Deck

The longitudinal shear resistance of the composite metal deck slab must be checked to ensure that the diaphragm forces can be transferred into the lateral resisting system chords and collectors. Current research is being carried out by University of Canterbury on the influence of any shrinkage or crack effects on the effect of the longitudinal shear strength. In the meantime the longitudinal shear provisions in NZS 3404 (SNZ, 1997) are considered should be used.

According to NZS 3404: 1997, 13.4.10.2, the design shear resistance for normal density concrete along any potential shear surface is given by:

$$V_r = \left(0.80\phi A_{rt} f_{yr} + 2.76\phi_c A_{cv}\right) \leq 0.50\phi_c f_c' A_{cv} \qquad (3)$$

Potential shear surfaces of shear failure when decking is used are presented in Figure 4.

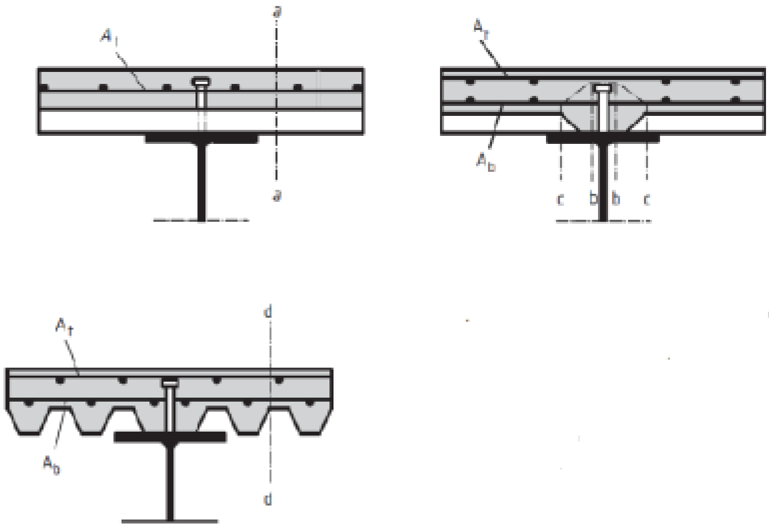


**Figure 4. Typical potential surfaces of shear failure where decking is used. (Hicks, 2011)**

For decking with ribs perpendicular to the beams, which is continuous across the top flange of the beam, its contribution to the transverse reinforcement for a shear surface of type a-a may be allowed for by replacing Equation (3) by:

$$V_r = \left(0.80\phi A_{rt} f_{yr} + 2.76\phi_c A_{cv} + \phi A_{pe} f_{yd}\right) \leq 0.50\phi_c f_c' A_{cv} \qquad (4)$$

where Φ is the strength reduction factor for reinforcement given in NZS3404, Table 13.1.2(1), $A_{pe}$ is the effective cross-sectional area of the decking (neglecting embossments), $f_{yd}$ is the yield stress used in design according to AS/NZS 4600 (SNZ, 2005), 1.5.1.4(b)

Where the decking with ribs perpendicular to the beam is discontinuous across the top flange of the beam, and the stud shear connectors are welded to the steel beam directly through the profiled steel sheets, the term $\phi\ A_{pe}\ f_{yd}$ in Equation (4) should be replaced by:

$$\left(\phi k_{\varphi} d_{do} t f_{yd}\right) / s \leq \phi A_{pe} f_{yd} \tag{5}$$

where $d_{do}$ is the diameter of the weld collar, which may be taken as 1.1 times the diameter of the shank of the stud $d_{sc}$, t is the nominal base steel thickness of the deck, s is the longitudinal spacing centre-to-centre of the stud shear connectors effective in anchoring the deck and $k_{\varphi}$ is:

$$k_{\varphi} = 1 + a / d_{sc} \leq 6.0 \tag{6}$$

where a is the distance from the centre of the stud to the end of the sheeting, to be not less than 1.5 $d_{do}$

Where the ribs of the decking run at an angle Θ to the span of the beam, the effective resistance should be determined from the following expression:

$$V_r = V_1 \sin^2 \theta + V_2 \cos^2 \theta \tag{7}$$

where $V_1$ is the value of $V_r$ for ribs perpendicular to the beam and $V_2$ is the value of $V_r$ for ribs parallel to the beam

The contribution of the decking to the longitudinal shear resistance should always be neglected where it is not properly anchored at discontinuities, or where the decking ribs run parallel to the beam. In theory, when the decking is parallel to the beam and properly anchored, some contribution to the longitudinal shear resistance could be included. However, including this contribution is not recommended because the decking resistance is affected by the (unpredictable) presence of laps on site. Studs fixed in a single line at a butt joint in the decking do not provide sufficient anchorage for the decking to contribute to the transverse reinforcement. However, decking contribution to transverse reinforcement can be taken into account where they are welded alternately on one sheet and then the other as shown in Figure 5

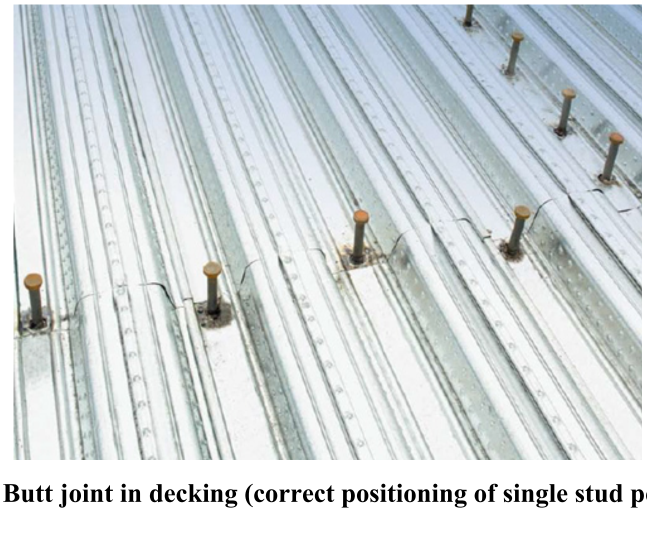

**Figure 5. Butt joint in decking (correct positioning of single stud per trough)**

## 4.3 Slab Edge Beam Detailing

The recommended edge detail for spandrel beams is shown in Figure 6(a) for a secondary spandrel beam and Figure 6(b) for a primary spandrel beam. The decking is terminated with 50mm minimum seating on the top of the beam flange and end closers installed. The outside edge can be formed with either a light gauge steel or a trimmer edge form. The outside edge must be at least six stud diameters beyond the centreline of the nearest stud. In addition to the mesh (longitudinal and transverse) reinforcement, transverse hooked bars together with a lapped longitudinal edge trimmer bar are provided. Figure 7 shows an example in practice

The bottom leg of the hook should extend back past the shear stud into the span of the beam by at least 50mm to help suppress shear splitting at the base of the stud. This is the mechanism that limits the load carrying capacity of a shear stud in a concrete slab, unless the concrete is very strong.

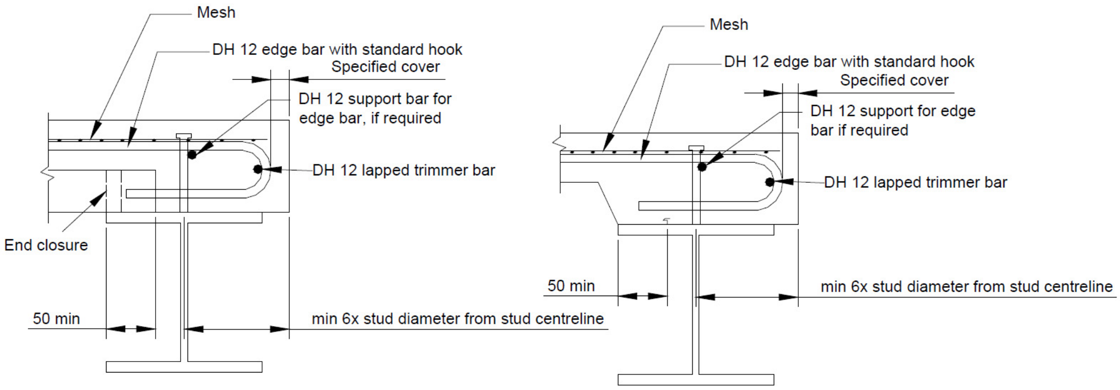


**Figure 6. Spandrel beam reinforcement. (Clifton, El Sarraf, 2005)**

Notes to Figure 6:
(1) The mesh is placed at the specified cover, supported off the deck by chairs.
(2) The spacing of the DH12 edge bars is given by the lesser of the following:
* the distance between shear studs
* in the case of the secondary spandrel beam, the distance between deck troughs
* requirement for fire and seismic resistance (e.g. for fire as given by the Slab Panel Method, the spacing is 250mm which is closer than the minimum spacing requirements for seismic action)
(3) One edge bar should be placed close to each shear stud
(4) The edge bars are tied to the mesh and to the support bar, if this is used
(5) The hook dimensions for the edge bar are from NZS 3101 Clause 7.3.1(a). Lap dimensions for the trimmer bar are to NZS 3101 Clause 7.3.7.
(6) The DH12 reinforcement is grade 500E or 500N to AS/NZS 4671
(7) A typical slab edge form is shown in Figure 9-4
(8) Bars extend into the slab past the edge of the beam by the anchorage length from NZS 3101, which can be taken as 600mm.

**Figure 7. Slab edge form at spandrel beam (Clifton, El Sarraf, 2005)**

### 4.4 **Shear Stud Requirements**

The longitudinal shear connection between the steel section and the concrete is provided by shear connectors, which normally take the form of studs welded to the top of the steel section. Design resistances of shear studs are given in NZS 3404.

Typically, the beams are designed as composite members for the gravity loads applied to the floor system. Diaphragm forces can be considered separately and additional shear studs to transfer the superimposed horizontal load into the beam may not be required. There are two primary reasons for this.

First, the quantity of shear studs selected for a composite beam is usually determined based on a gravity load combination, such as 1.2G+1.5Q. When lateral loads are applied in conjunction with the gravity loads, the load combinations of AS/NZS 1170 reduce the live load levels. Under these reduced live loads, the shear studs provided to develop the composite action required for the gravity loads will be under-used and thus have additional capacity available for the transfer of the diaphragm forces. Second, the interaction of the shear flow from the different loading conditions is additive for some studs but opposite for others. The distribution of horizontal shear from beam flexure is assumed to flow in two directions from the point of maximum moment to the point of zero moment. For a typical simple-span composite beam with uniform gravity loads, this shear flow is as indicated in Figure 8. While the beam shear is greatest at the ends of the beams, it is common practice to assume that the shear studs will deform and redistribute the shear uniformly to all studs.

However, if shear studs are required to develop composite action to resist the loads from (1.0G+1.0Q), the number of shear studs required for this and for diaphragm action must be additive. The use of 1.0Q in this combination instead of $\psi_L Q$ is in accordance with the requirements of NZS 3404 Clause 12.10.2.2 to prevent uniaxial beam hinging (shakedown) in moment resisting frames; however it is applicable in this case to any beam forming a key component of the diaphragm system. Conversely, lateral loads induce shear in only one direction. When these beams are used to collect the diaphragm forces, the shears due to the lateral loads are superimposed on the horizontal shears due to the gravity loads, as indicated in Figure 9. On one side of the beam, the lateral loads increase the horizontal shears over the gravity-induced values, while on the other side of the beam, the lateral loads oppose the gravity-induced horizontal shears.

Assuming the shear studs have sufficient ductility to distribute the horizontal shears evenly along the beam, a composite beam can transfer a horizontal shear due to lateral loads between the floor diaphragm and steel beam that is equal to the summation of the strengths of all the shear studs on the beam regardless of demand on the shear studs from the gravity loads, except as noted above..

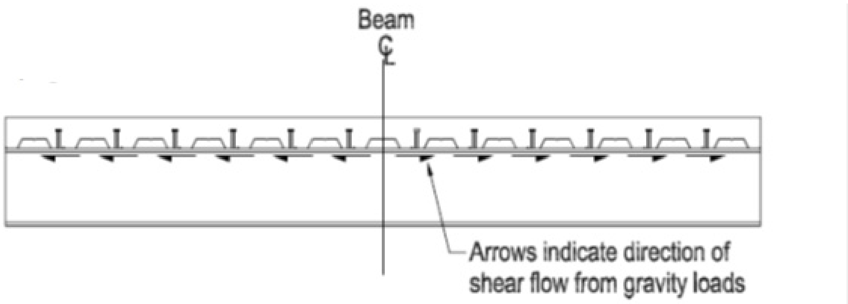


**Figure 8. Shear Flow due to Gravity Loads Only (Burmeister, 2008).**

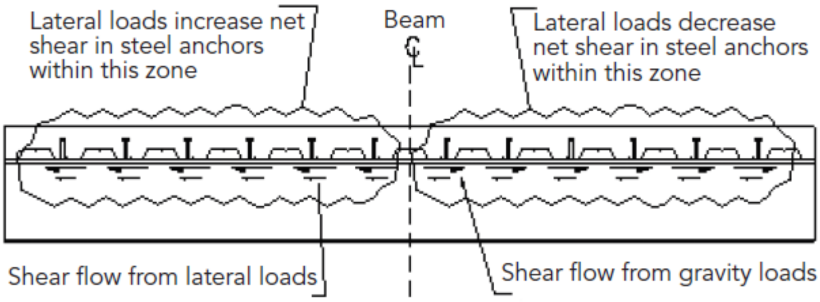


**Figure 9. Shear flow due to Gravity and Lateral Loads in Combination. (Burmeister, 2008).**

### 4.5 Shear stud requirements for "Non-Composite" Chords and Collectors

Designers encounter many conditions where steel beams are designed as non-composite members under gravity loads. Shear studs placed on these beams for transfer of lateral forces still will be subjected to horizontal shears due to flexure from gravity loads. This is unavoidable. Therefore, in order to ensure the shear studs are not overloaded under the gravity loads, it is recommended that all beams that transfer diaphragm forces to the lateral load resisting systems have enough shear studs to achieve a minimum 25% partial composite action, or else have sufficient capacity to resist (1.0G+1.0Q) non-composite. When less shear studs than this are provided, large deformations of the studs may occur under the gravity load case, inhibiting the ability of the beam to function as intended under lateral loading.

### 4.6 Secondary Shears and Moments

Once the designer deals with the transfer of force from the floor diaphragm into the supporting steel beam, the effect of the diaphragm forces on the design of the beam and its connections to the remainder of the lateral load resisting system must be considered. Of particular concern is the effect of the vertical offset (eccentricity) between the plane of the diaphragm and the centreline of the supporting beam as indicated in Figure 9. Intuitively, one would anticipate additional moments imposed on the beam as a result of the eccentricity. However, this is not the case.

As an example, consider a simple-span beam with uniform horizontal shears from the lateral loads and resulting reactions as shown in Figure 10. For this scenario, assume the member is connected to the lateral load resisting system at the left end of the beam only.

The free body diagram in Figure 10 shows the internal member forces that result from this applied uniform load. The axial load in the beam will increase linearly toward the end of the beam designed to transfer the collected force to the lateral load resisting system, but the internal moment, m1. due to this applied lateral load, even considering the d/2 eccentricity, will be zero. The member should be designed as a beam-column, considering the combined effects of the axial forces due to the lateral loads and the flexural forces due only to the gravity loads. The shear is a constant value and must be considered in the connection design at both ends of the member.

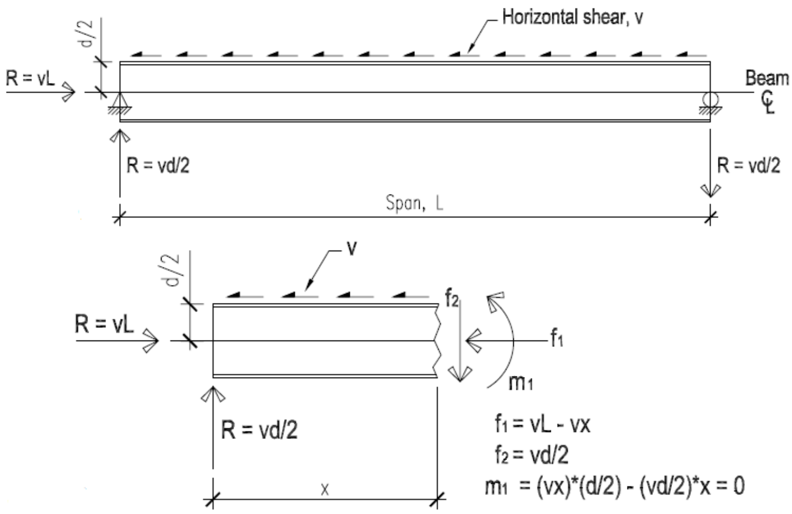


**Figure 10. Consideration of any secondary moments for a simply supported collector beam (Burmeister, 2008).**

### 4.7 Chords and Collectors Connections

If you have an eccentrically braced frame (EBF) or concentrically braced frame (CBF) then diaphragm collector beams will be needed to help drag the diaphragm shear into the seismic resisting system. This requires the diaphragm collector beams to accumulate axial load through the shear studs and transfer that into the columns of the seismic resisting system through a dependable, axially stiff load path but one that does not develop high moments in the columns.

The solution is very simple – use a bolted top flange connection that looks the same as the top flange connection in a Sliding Hinge Joint in conjunction with the flexible end plate (FE) or web side plate (WP) connection to carry the vertical shear. See Figure 11. The net effective tensile area is considered to be the top flange and half the beam web. The connection also has to be detailed to develop inelastic rotation without bolt or weld failure and so will handle the connection rotation due to the top flange pin. Adjacent to the top flange plate in the column must be a continuity stiffener to take the anchorage force into the column and the seismic resisting system. The stiffener must be welded so as to develop the full tension capacity of the stiffener at each flange and transfer this into the column web. This will avoid any crippling or local failure of the column. Alternatively the beam web connection must transfer both vertical and horizontal shear.

Bolts in tension bearing (TB) mode will provide adequate connection stiffness for either a top flange or a web connections.

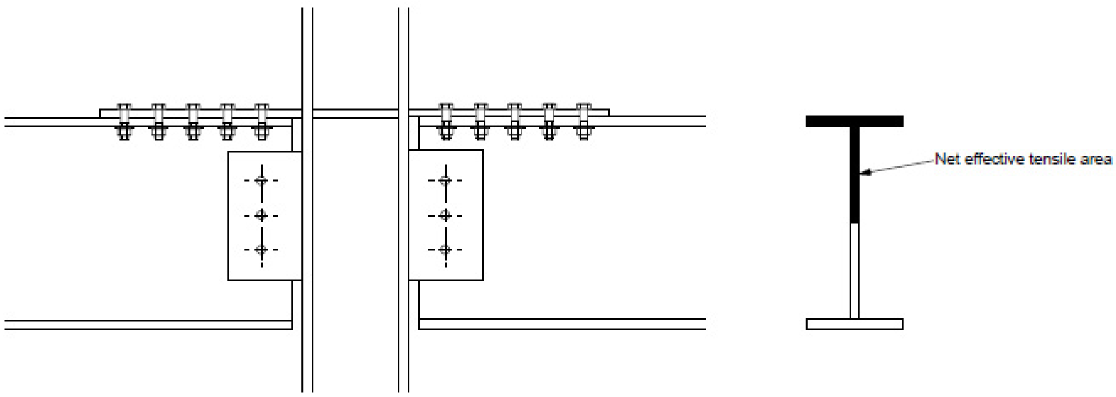


**Figure 11. a) Top flange plate connection to transfer collector beam axial loads, b) Axial loads transferred through top flange and half the beam depth**

## 5 CONCLUSION

Composite metal deck with concrete fill floors has been shown to act as good diaphragms. There is a lack of documented guidance on the design of these diaphragms. This paper provides some additional information which can be used in conjunction with an American publication to design these diaphragms. Specifically, an approach to determining diaphragm actions based on a modification of the parts and components method of the loadings standard is presented. The distribution of these actions into individual components can be made using the deep beam approach. Diaphragm component forces can be considered to be independent of other actions. Design of the interface between the floor and the seismic lateral collector beams is provided using modified provisions of the Steel Structures Standard NZS 3404. A good connection detail between the collector beam and the seismic resisting column is using a bolted top flange connection. This allows for an axially stiff load path but does not develop high moments in the column.

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