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Floor Diaphragms in Timber Buildings

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Contents

1

Introduction

Floor diaphragms transfer horizontal forces to the lateral load-resisting system. Horizontal loads, such as wind, applied to the façade will be transferred as line loads to the edges of the diaphragms. Horizontal loadings cause inertia forces to develop within the flooring system that have to be carried to the frames, walls or lift shafts and stairway cores.

Differences in the behaviour of the lateral load-resisting systems (deformed shape, discontinuous geometry, difference in stiffness) induce additional transfer forces in some diaphragms. Roof and floor diaphragms also carry gravity loads and they link all vertical structural elements together. It is of paramount importance that diaphragms maintain their force transferring and linking behaviour before, during and after any horizontal loadings.

The connection to the lateral load-resisting system can be the weak link in diaphragm design. As discussed in the next section, this force transfer can be compromised by displacement incompatibilities typical of jointed-ductile systems.

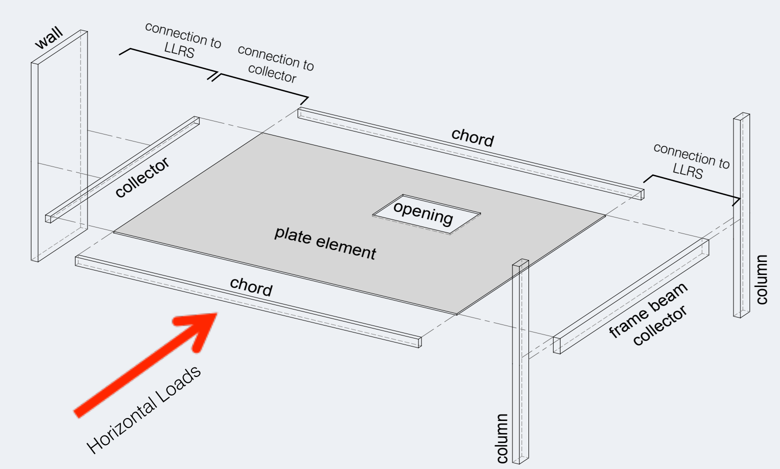

Figure 1.1: Definitions of single diaphragm components.

In this Guide, the horizontal action is considered to be caused by horizontal loads such as wind load.

The first part of the Guide presents the terminology, concept and design of timber diaphragms with their connections to the lateral load-resisting system (LLRS).

The second part reviews a design example of a timber–concrete diaphragm and its connections to the LLRS. The diaphragm is subjected to the wind load applied perpendicular to its long side.

For the calculation of timber concrete composite diaphragms, engineers prefer a grillage method with the use of analysis software. Further information on this grillage method or equivalent concrete truss method can be found in Strut and Tie Seminar Notes[1].

An equivalent truss model is recommended for calculating timber diaphragms. Further information can be found in An equivalent truss method for the analysis of timber diaphragms[2].

New alternative connections and further details including their behaviour in experimental test and also cost comparison can be found in Design of Floor Diaphragms in Multi-Storey Timber Buildings[3], and Seismic design of floor diaphragms in post-tensioned timber buildings[4].

2

Terminology

Diaphragms can be made from many different materials – plywood panels, stressed-skin panels, timber concrete composite floors, cross-laminated timber (CLT), solid floor panels, structural insulated panels – but their main components can be grouped as follows (see Figure 1.1):

- plate element
- chords
- collectors/struts
- connections to the lateral load-resisting system.

The simplest method to design diaphragms is the horizontal steel girder analogy, where the web is made by the plate element and the flanges consist of the chords. The plate element with possible openings transfers the horizontal shear forces. Several single floor elements may have to be linked together and forces carried around openings or re-entrant corners. The resultant shear forces have to be collected and conveyed to the lateral load resisting system via the collectors or struts. The connection of the collector to the lateral load-resisting system has to be designed properly, as it is an essential part of the load path into the foundations.

As shown in Figure 2.1 and according to Malone and Rice[5], the following terminology is suggested:

- Strut: receives shear from one side only
- Collector: receives shear from both sides
- Chord: perpendicular to the applied load and receives axial tension and compression forces.

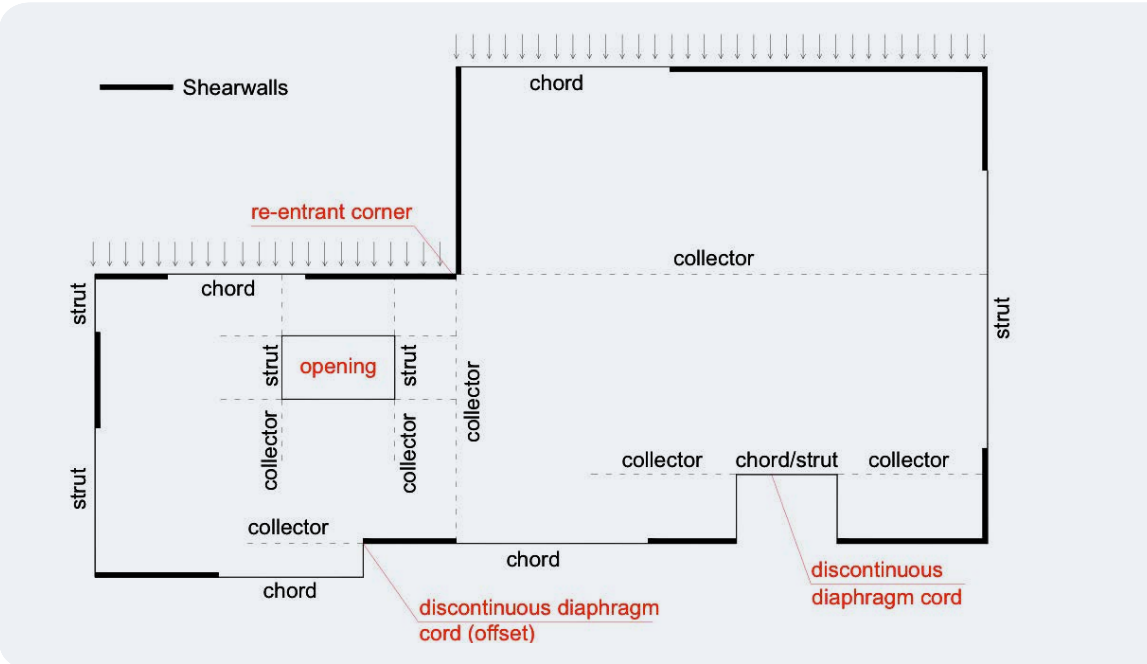

Figure 2.1: Irregular floor geometry with typical diaphragm elements.

Openings are an unavoidable feature of floor plans, as staircases, lift shafts and channels for services need to go along the height of a building. The position of these openings has to be chosen carefully, as they influence the behaviour of a building in multiple ways. First, it influences the load path in the diaphragm, as the shear forces might have to be carried around it. Bigger openings also increase the flexibility of diaphragms, which influences the behaviour of the structure and the load distribution into the lateral load-resisting system. In certain positions, openings also cause a separation of the diaphragm, as the forces cannot be transferred appropriately and the plane element cannot act as a unit.

In this situation, often the concept of sub-diaphragms or transfer diaphragms is introduced[5,6]. These are portions of the main diaphragm and are used to transfer or anchor higher shear stresses (from openings, re-entrant corners or concentrated loads) into the remaining diaphragm or the lateral load-resisting system. Often, closer nail spacing and thicker framing members are adopted, but the sub-diaphragms are essentially designed as regular diaphragms.

3

Displacement Incompatibilities

Displacement incompatibilities within diaphragms or between diaphragms and the LLRS can damage structures. Hence, design and detailing are essential to consider displacement incompatibilities within diaphragms or between diaphragms and the LLRS[7]. The displacement incompatibilities normally associated to concrete and steel structures can also be observed in traditional and innovative timber structures[8].

Experimental testing has shown that the flexibilities of timber members and steel fasteners can, in many cases, accommodate the required displacements without compromising the diaphragm behaviour.

With careful design, well-designed timber diaphragms can easily undergo horizontal loadings without any damage. In the case of TCC floors or any floors with concrete topping, some additional detailing may be necessary as detailed in Section 13.

4

Diaphragm Design for Wind Action

Wind loads are obtained from AS 1170.2. Wind pressures applied to the façade (perpendicular pressure or wind friction) are then simply transferred to the diaphragms according to their tributary areas.

5

Load Paths in Diaphragms

For regularly shaped floor geometries, i.e. rectangular, without big openings or re-entrant corners, timber-only diaphragms can be designed by using the girder analogy. This implies that the shear is taken by the web (diaphragm sheeting) and the bending is taken by the flanges (diaphragm chords). Even though the girder analogy may not be strictly appropriate for deep beams with anisotropic materials, tests have shown that flange stresses are smaller than using that approach, providing a conservative design.9,10 Furthermore, shear stresses develop uniformly over the web, instead of the parabolic shape found in steel girder webs.

Diaphragms can be designed as simply supported or continuous beams, providing that the span-to-depth ratio is greater than 2. For aspect ratios smaller than 1, the girder analogy is quite conservative, as the sheeting and joists contribute substantially in the bending resistance. Considering the high depth of the diaphragm, the chord forces will be small.11 Different authors provide an upper limit for the span-to-depth-ratio, as the diaphragm may become too flexible. If floors are running over internal supports and the different diaphragm parts on each side are connected, they can be analysed as a continuous beam.

As shown in Figure 5.1, diaphragms with simple plan geometries can be calculated by considering a girder analogy, where the bending is taken by the chord elements in form of tension or compression forces and the uniform shear is taken by the plate element.

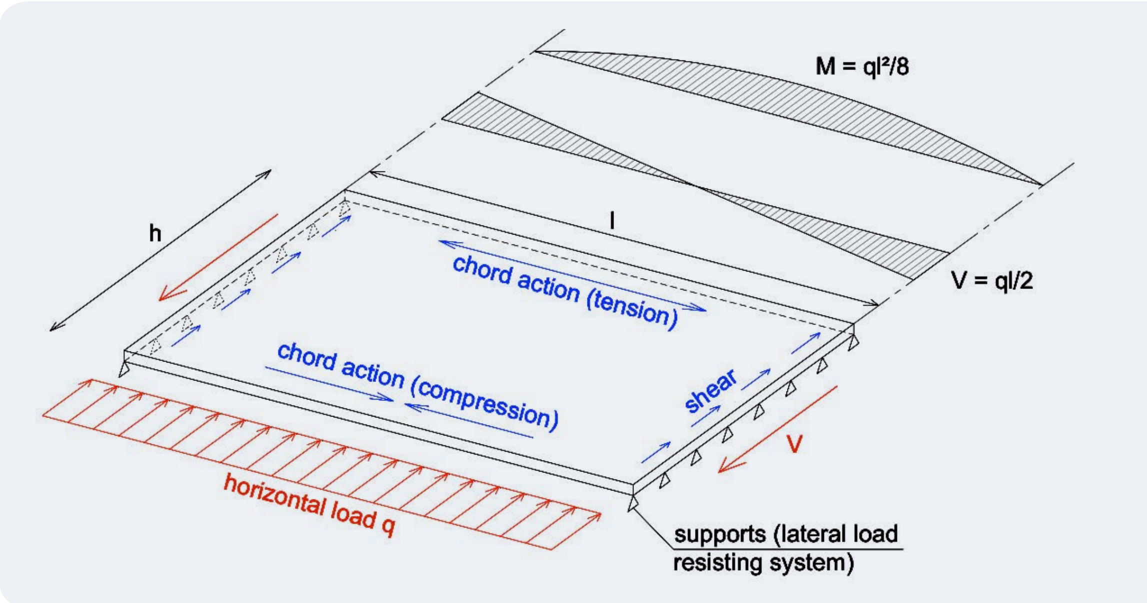

Figure 5.1: Girder analogy for diaphragms.

The tension and compression forces in the chords (T and C) can be calculated as follows, where the terms are shown in Figure 5.1:

$$T = -C = \frac{M}{h} = \frac{ql^2}{8h} \tag{5-1}$$

The shear flow along the edges of the diaphragm is:

$$v = \frac{V}{h} = \frac{ql}{2h} \tag{5-2}$$

where:

q = the lateral load applied to a horizontal diaphragm

l = the span of horizontal diaphragm

h = the width of horizontal diaphragm

V = the shear force applied to a diaphragm

M = the design moment of diaphragm.

The loading of the diaphragms is normally considered as distributed uniformly along the length of the diaphragm. This is the wind load transferred from the façade, or the inertia forces generated by the mass of the floor itself. In the case of multi-storey structures, vertical offsets can introduce concentrated loads. Openings, re-entrant corners, offsets or concentrated horizontal forces will disturb the shear flow and locally higher stresses might arise.

6

Flexible and Rigid Diaphragms

The distribution of forces into the lateral load-resisting system depends on the flexibility of the diaphragm, i.e. rigid or flexible diaphragms. A diaphragm is considered to be flexible if its deformation is more than twice the average inter-story drift at that level.

In the case of rigid diaphragms, the transferred forces depend on the stiffness of the diaphragm with respect to the global stiffness of the lateral load-resisting system. In the likely case that the centres of stiffness and mass are not coincident, torsional effects have to be taken into account.

To define whether the diaphragm is rigid or flexible, the deflection calculation for both the vertical LLRS and the diaphragm is required. For flexible diaphragms (Figure 6.1) the load can be determined by using a tributary area approach.

Depending on the geometry, timber diaphragms often behave somewhere between these two extreme cases. It remains the designer's choice of which approach to use: to calculate both to obtain an envelope; to use a beam on elastic support approach (beam and elastic support have the stiffness characteristics of the diaphragm and lateral load-resisting system respectively); or to use a finite element method.

The beam-on-elastic-support approach is valid as long as the stiffness of the diaphragm and the lateral load-resisting system are similar. The finite element method allows study of the shear stress distribution over the whole plate element, but requires much more effort in modelling and running the simulation.

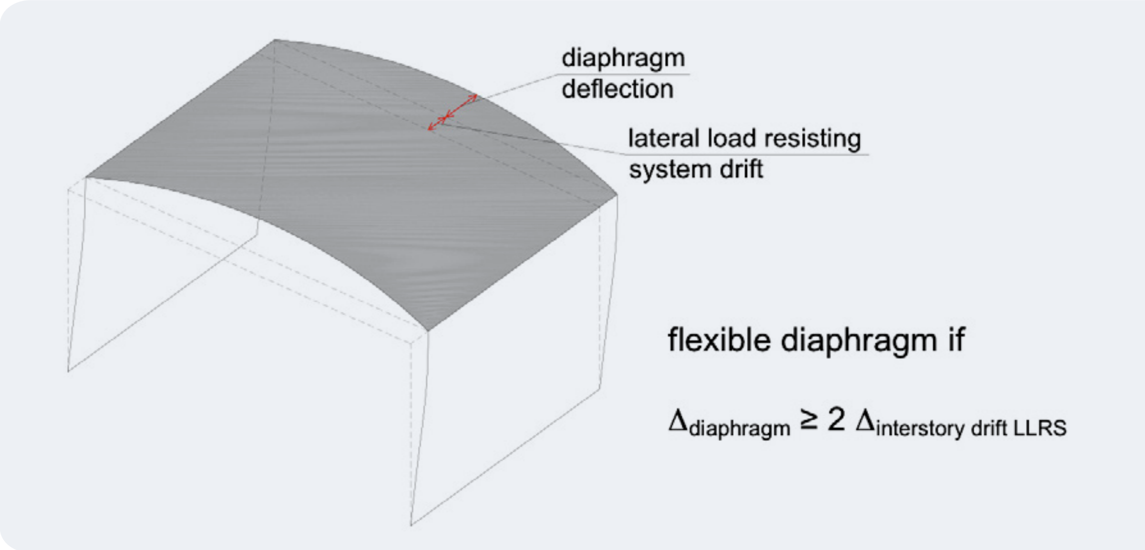

Figure 6.1: Flexible diaphragm.

7

Design of Structural Elements in Diaphragms

The structural elements in a timber diaphragm consist of the plate element, the chords and the collector/strut beams.

Depending on the chosen setup, the plate element consists either of sheeting panels and framing members or thicker solid type panels. They all have to be joined to act as a single unit. For a traditional timber joist floor with panels of particleboard or plywood, the strength of the diaphragm depends on the amount of nailing and the presence of blocking (connection of the panel edges to adjacent panels). Blocked diaphragms have a stiffness and strength two to three times higher than the unblocked equivalent so, for multi-storey timber buildings, such floors should always have solid blocking along all the sheet edges. The buckling of the sheeting is normally prevented by edge connections.

The capacity checks necessary to design a diaphragm should be performed in accordance with the relevant timber code and should include:

- the shear capacity of the nails
- the shear capacity of the panel
- the out-of-plane buckling of the panel.

Nail spacing and sheeting thickness is normally dictated from the stress values at the supports of the diaphragm. As an alternative, the sheeting panels can be glued or screwed to the framing elements; this can provide a higher shear capacity and stiffer connections.

Chord, collector and strut beams have to transfer tension and compression forces and need to be designed accordingly. If they consist of single jointed elements, continuity has to be provided by adequate ductile connections. Chords should be spliced as far as possible away from the point of maximum moment. Often these elements also work under gravity loads and internal stresses have to be combined with the lateral forces. Since the direction of wind forces is arbitrary, chords also act as collectors – and vice versa – depending on the loading direction.

8 Horizontal Deflection of Diaphragms

There are several reasons why the horizontal stiffness of a diaphragm needs to be calculated. The vertical elements supporting or attached to a diaphragm need to maintain their load carrying capacity to guarantee structural integrity, and should not be damaged due to excessive deformation. Furthermore, the distribution of the in-plane loads to the lateral load-resisting system is a function of the stiffness of the diaphragm as discussed above. Finally, the dynamic period of the diaphragm can interfere with the dynamic behaviour of the structure and cause higher modes effect.

The horizontal deflection of diaphragms is the sum of the following single contributions:

Δ_1 = flexural deflection of the diaphragm considering the chords acting as a moment resisting couple

Δ_2 = deflection due to shear in the panels

Δ_3 = deflection of the diaphragm due to fastener slip

Δ_4 = deflection of the diaphragm due chord connection deformation.

Equations for the determination of these contributions are provided in NZS 3603.[12]

$$\Delta_1 = \frac{5ql^3}{192EAB^2} \quad (8\text{-}1)$$

$$\Delta_2 = \frac{ql}{8GBt} \quad (8\text{-}2)$$

$$\Delta_3 = \frac{(1+a)me_n}{2} \quad (8\text{-}3)$$

$$\Delta_4 = \frac{\sum \delta_s x}{2B} \quad (8\text{-}4)$$

where:

q = lateral load applied to a horizontal diaphragm

l = span of horizontal diaphragm

E = elastic modulus of chord member

A = section area of the chord

B = distance between diaphragm chord member

G = shear modulus of the diaphragm sheathing

t = thickness of the diaphragm sheathing

m = number of the sheathing panels along the length of the edge chord

e_n = fastener slip resulting from the shear force V

a = aspect ratio of each sheathing panel given in the NZS 3603[12]

x = distance of the splice from the origin

δ_s = splice slip in the chord.

The fastener slip en is calculated for the maximum unit shear force at the support. Since all diaphragms should be designed as elastic, the slip only depends on the slip modulus and spacing of the fasteners[3]. Equation (8-3) can be modified where different fasteners are used for the panel-to-panel, panel-to-chord and panel-to-collector connections.

Given the variety of wooden construction materials and means of connections, the possible diaphragm setup and the limited amount of experimental tests, the deflection values only provide a rough approximation of the diaphragm deformation. The verification of a certain deflection limit, however, is to be set by the designer with knowledge of the affect deflection will have on the surrounding structure.

The deflection given above is only applicable to simply supported blocked diaphragms with chord beams. To account for diaphragm irregularities such as variable loads, openings, re-entrant corners, changes in diaphragm depth or staggered fastener layouts, these equations can be integrated over parts of the diaphragm[7].

The presence of openings, varying nailing pattern or non-uniform forces should be considered when calculating the horizontal diaphragm deflection. This can be done by modifying the basic deflection equation by changing the coefficients of the single contributions according to basic beam theory or by integrating the equation over segments of the diaphragm. If more precise results are required, finite element analysis might have to be considered.

The design of floors where the diaphragm action is taken by the concrete topping should be in accordance with the relevant concrete code. Special provisions regarding displacement incompatibilities will be given later.

9

Connection Between Single Timber Floor Elements

To connect two single floor panels together, the following alternative connection systems are suggested (see Figure 9.1):

a) nailing (and gluing) of adjacent panels

b) wooden strip in recess between panels with screws or nails

c) inclined fully threaded screws, or regular screws at 90° between joists

d) nailing (and gluing) of panel to the next joist

e) double inclined screws in shear between solid panels

f) tongue and groove with double inclined fully threaded screws.

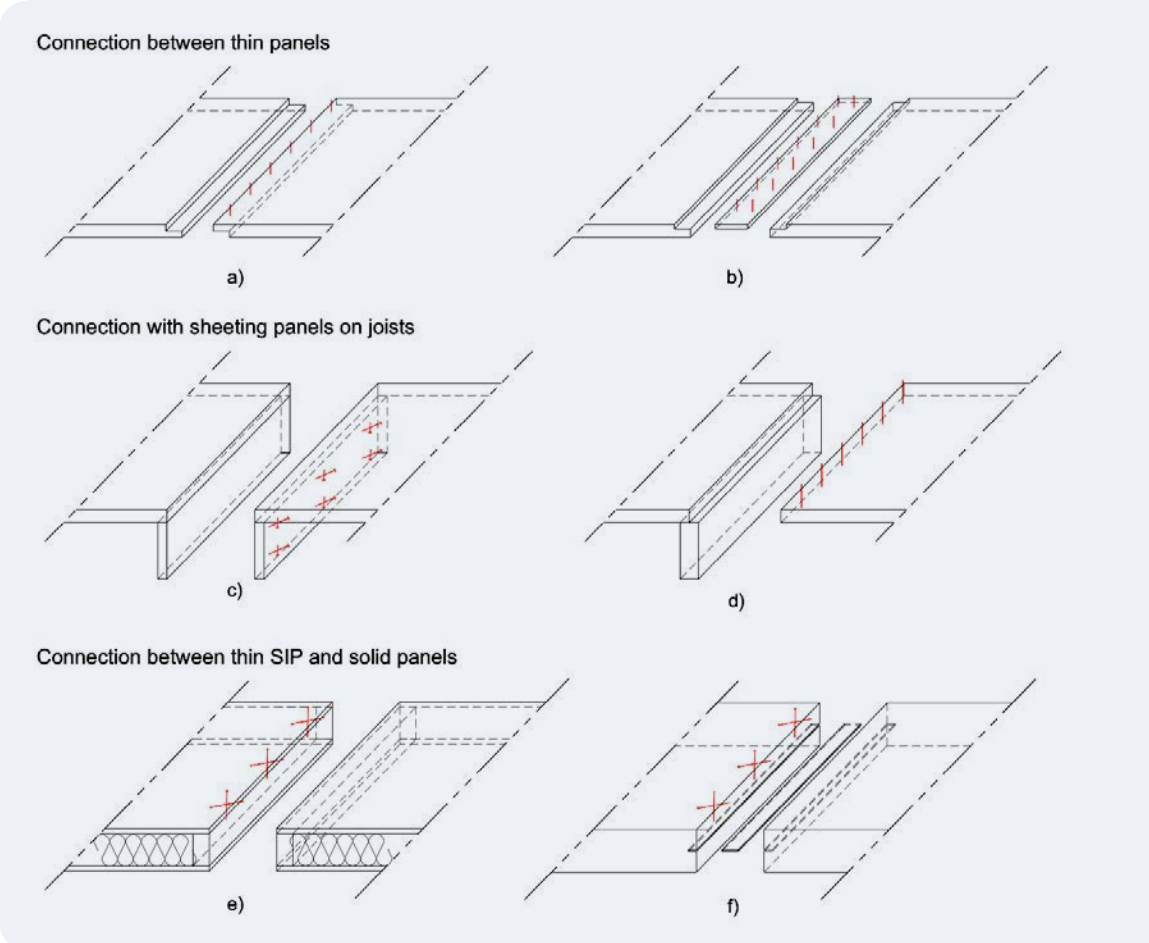

Figure 9.1: Connection details between floor elements.

These connections are generally valid and are to be designed to guarantee adequate shear transfer. Because of the displacement incompatibilities mentioned before, special detailing for floor joints close to the beam–column joints may be necessary.

Gluing must be considered carefully. A nailed joint or screwed joint with glue will become much stronger, but also much more brittle and less deformable under extreme loads, so there will be many cases where glue should not be used.

10 Connection Between Single Timber Floor Elements

Because of the variety of building geometries, lateral load-resisting systems, floor assemblies and the available types of connection, no unique detail solution can be given. Key aspects to consider for the connection design are the kind of required force transfer (horizontal shear only or combined with gravity forces) and the type of diaphragm (timber only or concrete topping).

While the connections mainly have to transfer the horizontal forces deriving from the diaphragm action, out-of-plane forces of the lateral load-resisting system have to be considered as well (see Figure 10.1). These forces can be wind suction at leeward walls, inertia forces on the façade, or dragging forces from the constraint of vertical elements to move with the rest of the structure under a certain drift.

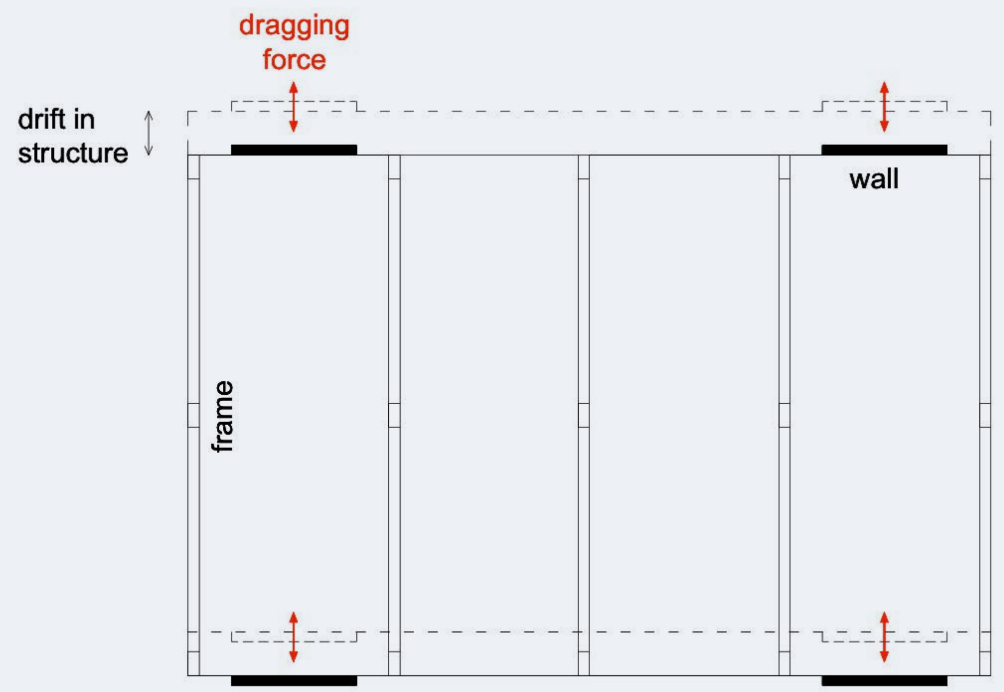

Figure 10.1: Dragging forces on walls from drift in structure with north–south forces resisted by frames.

11 Connection Between Timber Diaphragms and Gravity Frames

11.1 Gravity and Shear Forces

In this section, timber-only floors running perpendicular to the lateral and gravity frames are considered. The floor elements have to transfer vertical gravity forces and horizontal shear forces to the beam, which acts as a collector or strut. For floors sitting between the beams, gravity loads can be transferred by a timber corbel, a pocket in the main beam or steel hanger brackets. The horizontal shear forces can be transferred directly by nailing or screwing the sheeting panels to the beam (see Figure 11.1).

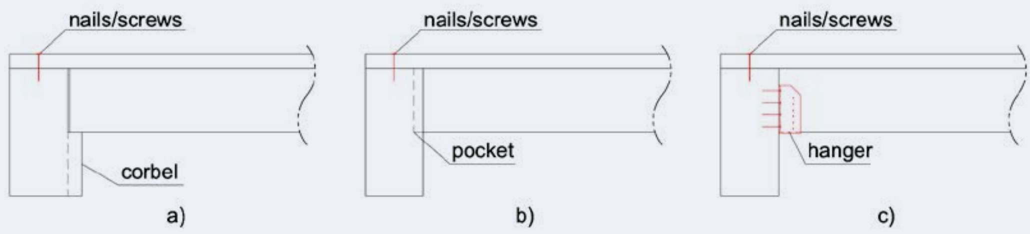

Figure 11.1: Suggested floor-to-frame connections (floor joists flush with beam): a) floor joist on corbel; b) floor joist in pocket; c) steel bracket/hanger.

Where the floors sit on the beams, gravity forces are transferred by direct contact. Shear forces can be transferred by using fully threaded screws at 45° angle or by connecting the sheeting to blocking elements, which are again joined to the beam by screws or steel plate elements (see Figure 11.2).

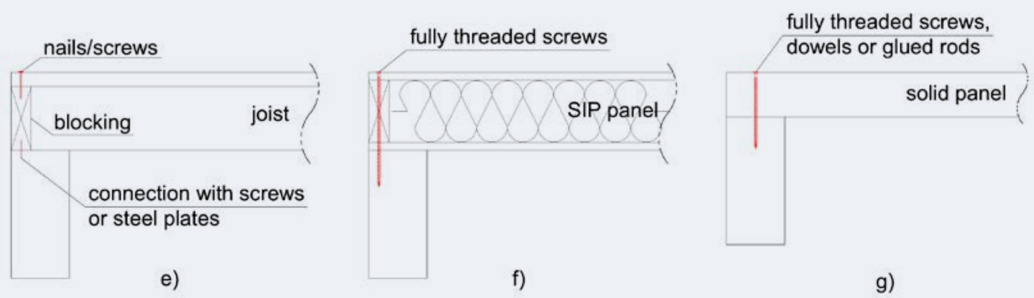

Figure 11.2: Suggested diaphragm to frame connections (floor joists on top of beam): e) floor joist sitting on beam – additional blocking required; f) SIP panel on beam; g) solid timber floor on beam.

11.2 Out-of-Plane Rocking

Where the horizontal load acts perpendicular to the frame, the whole building will undergo a certain drift (depending on the lateral load-resisting system in this direction) and hence the frame will have to rotate out of plane. As indicated in Figure 11.3, it is suggested to leave a construction gap between the floor elements and the beams (also useful for construction tolerance and variances in ambient conditions). This will allow the beam to rotate, without damaging the timber floor or the connection to it. The connection between the beam and the floor also needs to transfer the dragging force to rotate the frame out of plane.

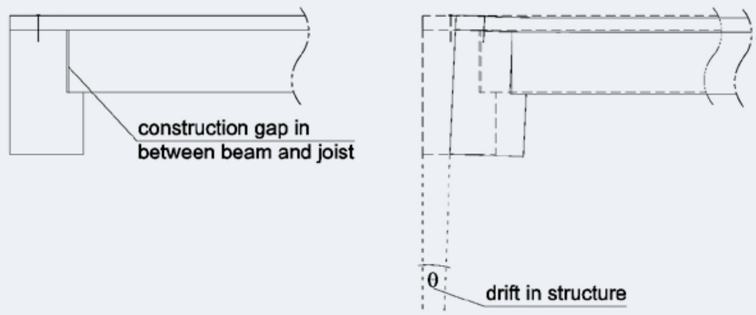

Figure 11.3: Construction gap between a timber floor and supporting beam to allow for rotation: a) undeformed state; b) deformed state.

11.3 Frame Elongation

The formation of gaps at the beam–column joint produces frame elongation. The diaphragm must be able to extend. This behaviour has to be allowed for without a brittle tearing of the plate element, as it would cause permanent damage and compromise the shear transfer. The flexibility of the timber elements and the low stiffness of the steel connections allow for two simple design solutions for engineered timber floors:

Solution 1: Concentrated gap (see Figure 11.4, blue details):

As the required deformation in the floor level occurs only at the beam–column joint, a joint between two adjacent floor panels should be positioned accordingly. This joint needs special detailing, whereas other panel joints can be designed normally.

For floor setups with sheeting panels and slender joists, only the lower part of the joist should be connected, so that the joist can bend along its height, but still guarantees shear transfer (see Figure 11.5a). If a different floor setup is used where the joist are too stiff, special steel elements can be used. These should allow the panels to move apart from each other, but still transfer shear forces (an example is shown in Figure 11.5b). Appropriate gaps in the floor finishing and the wall linings have to be provided to allow these deformations to occur.

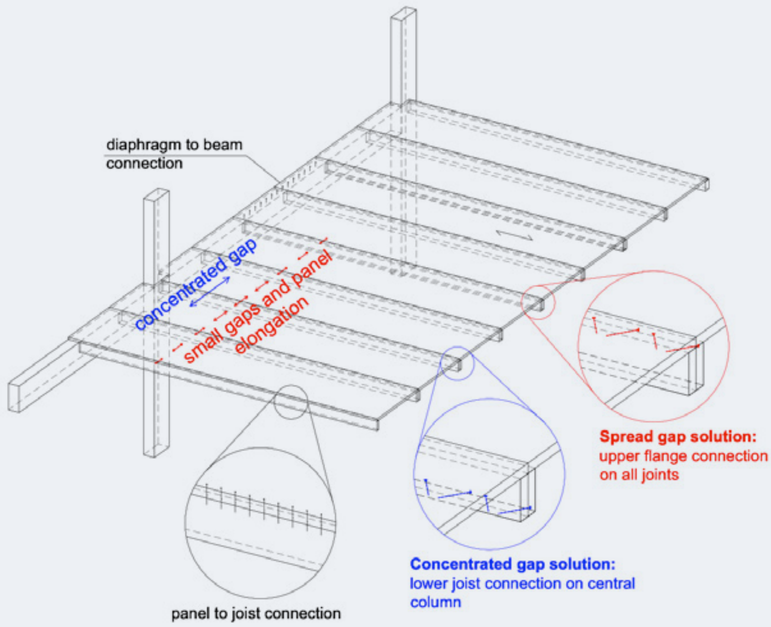

Figure 11.4: Sample design for a concentrated floor gap (blue) and spread gaps and panel elongation (red). Solutions for a timber-only engineered floor.

Solution 2: Spread floor gaps and panel elongation (see Figure 11.4, red details):

As an alternative to a concentrated gap at each beam location, detailing for uniformly spread gaps can be used. The required deformation will be accommodated by a number of small panel gap openings and the elongation of the sheeting panel itself. This implies that the panel is relatively flexible in the direction perpendicular to the span direction).

Two to three floor elements each side of the interested beam–column joint should be connected to each other by means of metallic connectors such as nails or screws (like an upper joist connection shown in Figure 11.5c). The connection needs to guarantee full shear transfer between the elements, but should be flexible enough to allow for a small displacement. Small gaps will hence open in several panel joints and the sheeting panels will elongate. The sum of all contributions will make up the required displacement, as demonstrated in recent testing.

Site gluing to connect floor elements should be avoided, as it results in a stiff and brittle connection that cannot accommodate the required deformations. Furthermore, the panels close to the beam–column joint(s) should not be connected to the beam to transfer diaphragm forces, as this would prevent the development of floor gap openings and panel elongations further away from the area of interest.

The floor finishing should be chosen to be elastic enough to follow the formation of the spread gaps.

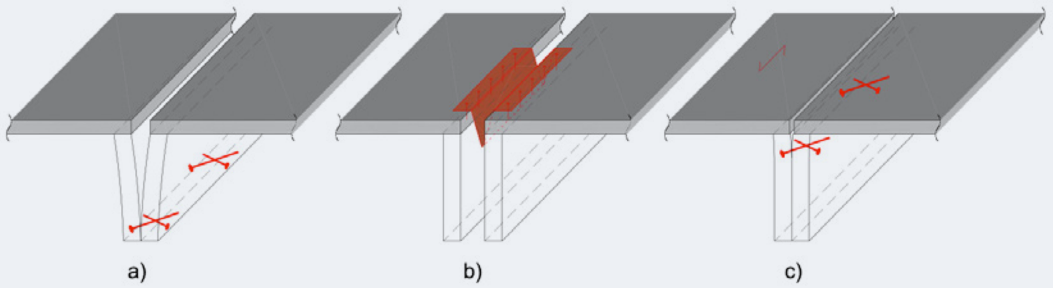

Figure 11.5: a) Lower flange connection; b) connection with thin steel plate; c) upper flange connection.

12 Connections between Timber Diaphragms and Walls

For wall structures, the diaphragm and gravity forces are transferred via the collector/strut beam to the lateral load-resisting system (see Figure 12.1). The most appropriate connection detail to link the collector beam to the walls depends on the span direction of the floor. For floor elements running parallel to the wall, only horizontal forces have to be transferred – otherwise gravity forces have to be taken as well.

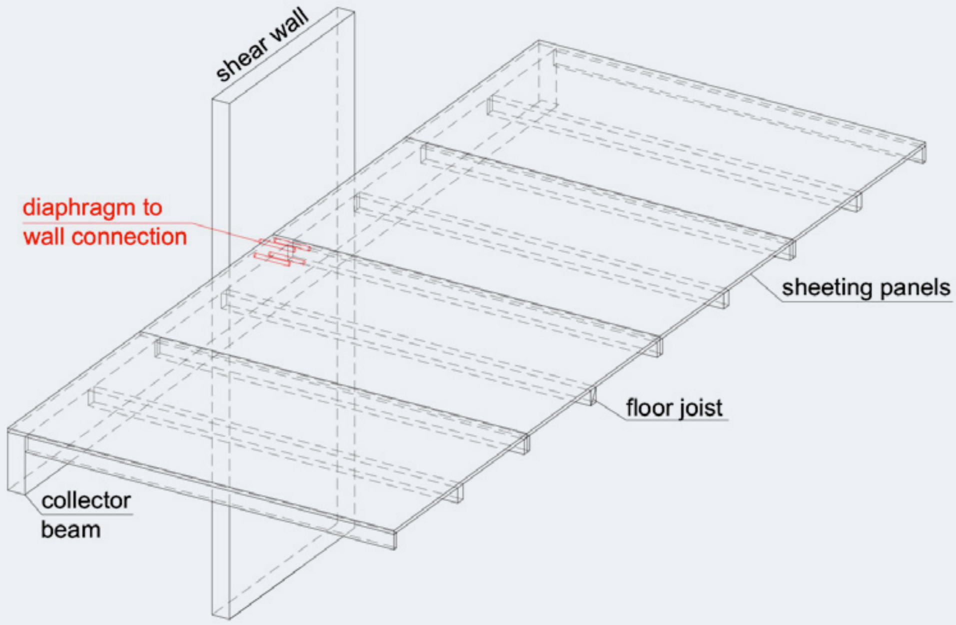

Figure 12.1: Scheme of a typical diaphragm-to-wall connection.

To allow for the required uplift and rotation of the walls, a discrete connection placed at the centre of the wall should be used. Ideally, a single dowel-type connector with a vertical slotted hole would overcome all displacement incompatibilities by still transferring the horizontal forces from the diaphragm. However, a single dowel is not usually suitable because of the magnitude of the forces, possible splitting of wood with large diameter dowels and the difficulty of providing slotted holes in timber.

If only horizontal forces have to be transferred to the wall, connections with steel plates and dowels placed in slotted holes can be used. The plate itself can be fixed by screws, nails, rivets or bolts to the timber elements. Where gravity forces also have to be conveyed to the wall, a vertical restraint is necessary. This solution can be achieved by simply connecting the timber beam and wall together with dowel-type connectors. While a single big diameter dowel is an attractive solution, little is known regarding its embedment strength. As an alternative, a ring of closely spaced dowels will approximate a hinge.

Table 12.1 summarises four connection details and their properties (refer also to Figure 12.2).

Table 12.1: Possible wall to collector beam connections.

Connection type	*Force transfer*	*Displacement incompatibilities*		*Comments*
Big pin connection	Horizontal shear and gravity	Rotation is allowed	Uplift is not allowed	The embedment strength and behaviour of large diameter dowels is not well known
Slotted steel plate with rivets	Horizontal force only	Rotation is allowed	Uplift is allowed	This connection allows for all displacement incompatibilities. Lots of steelwork required
Ring of dowels	Horizontal shear and gravity	Rotation is partially allowed*	Uplift is not allowed	Simple solution, the flexibility of the connection allows for some rotation
Steel profile with slotted holes	Horizontal force only	Rotation is allowed	Uplift is allowed	Possible problems due to friction

** Given the possibility of using oversized holes in the timber and relatively flexible dowel connection, the rotation of the wall normally can be accommodated for limited drift ratios.*

If uplift of the walls is not allowed by the connection and the collector/strut beam is also attached to other vertical elements, such as columns (see Figure 12.3a), the beam and the diaphragm will both need to bend. This can be tolerated if the collector beam is flexible enough (i.e. because of its small section or a long span to the next vertical restraint). The additional re-entering force resulting from bending should be considered when designing the wall.

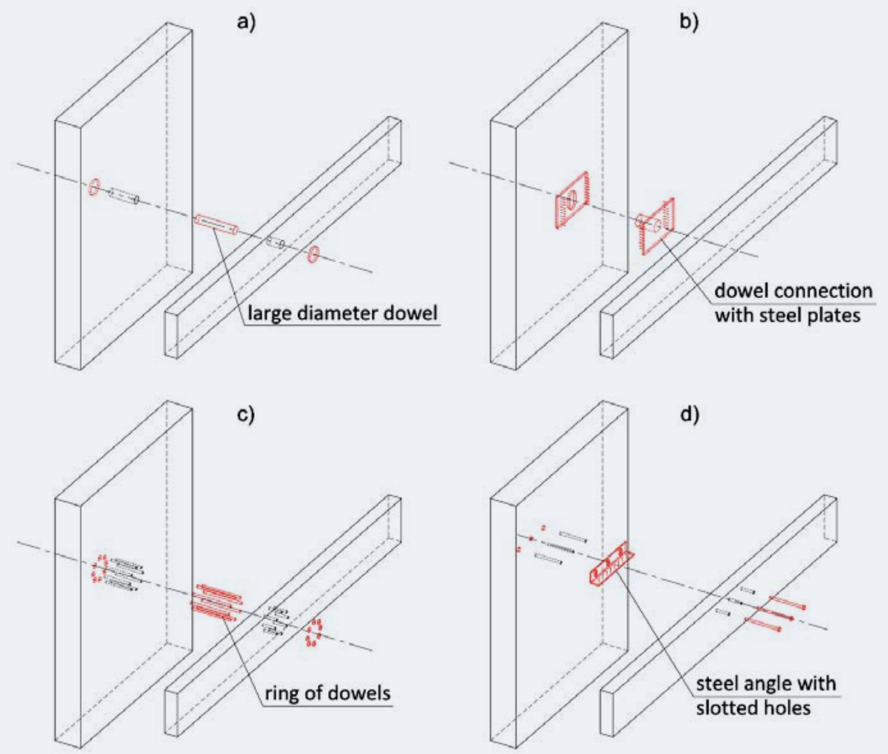

Figure 12.2: Suggested diaphragm-to-wall connection details: a) large diameter dowel connection (timber–timber); b) dowel connection (steel–steel); c) multiple dowel connection (timber–timber); and (d) steel angle with slotted holes.

For multiple dowel connection (ring of dowels), the additional moment coming from the rotational restraint should be checked in the beam and in the wall design. Because of the oversized hole in the timber elements, the compact geometry and the relatively small connection stiffness, this connection should almost behave as a hinge.

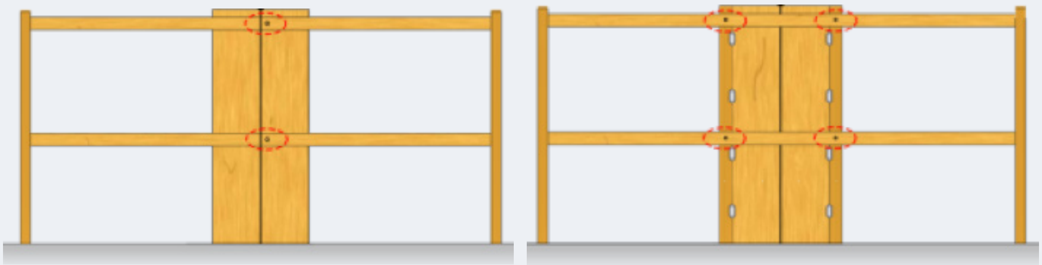

Figure 12.3: a) Single wall; and b) wall with external columns.

Where gravity forces have to be transferred to the wall and the uplift of the collector beam has to be avoided, a wall configuration with external columns as shown in Figure 12.3b can be used. Under horizontal loading, the wall would rock but the columns would only follow rotation without any uplift. In this way, gravity and horizontal forces can be transferred directly to the columns by avoiding any vertical displacement incompatibility. The connection only has to accommodate the rotation of the columns. The horizontal force transfer from the columns to the wall and the buckling restrain of the columns itself must be considered appropriately.

12.1 Out-of-Plane Rocking

As in frame structures, the connections between the floor diaphragm, the collector beam and the wall itself have to transfer not only the shear flow from diaphragm action, but also the drag force from the out-of-plane deformation of the wall. This will occur when the horizontal load act perpendicular to the walls and the whole structure deforms in the out-of-plane direction of the walls. Construction gaps between the floor and the walls must accommodate the displacement incompatibility.

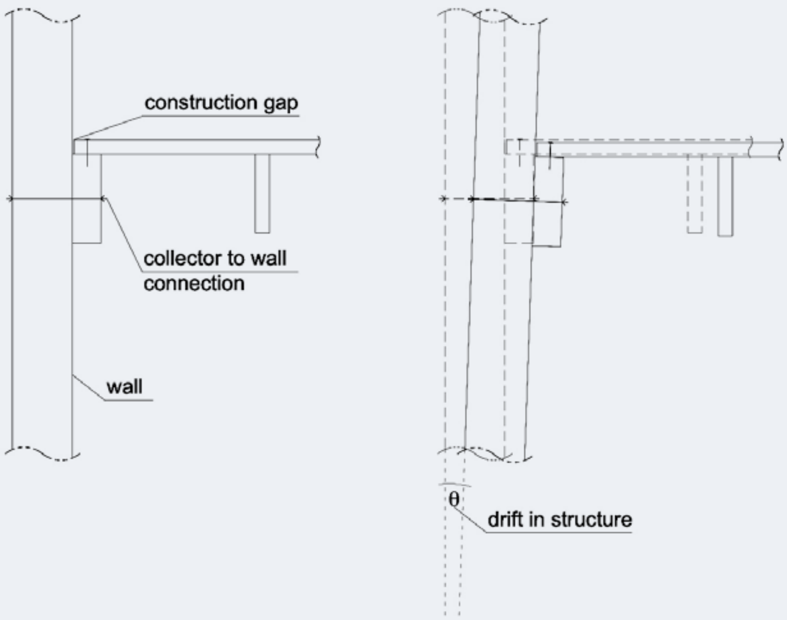

Figure 12.4: Construction gap between the floor and the wall to allow for rotation: a) undeformed state; b) deformed state.

13

Connection Details for Timber Concrete Composite (TCC) Floors

As a result of the low tensile strength of concrete, tearing forces due to frame elongation and bending forces due to uplift and rotation of the walls tend to crack the diaphragm topping. If these cracks become larger, the force transfer is interrupted and the diaphragm action compromised[13]. It is essential to design the diaphragm with its connections accordingly.

For frame structures with TCC floors, the displacement incompatibility required from the beam-column-gap opening can be accommodated similarly to the concentrated floor gap solution already described for timber diaphragms. As suggested in Figure 13.1, the concrete should be pre-cracked along the line of the beam–column joint. Unbonded rebars should be placed across the crack, designed to deform elastically in case of gap opening and to provide shear transfer via dowel action.

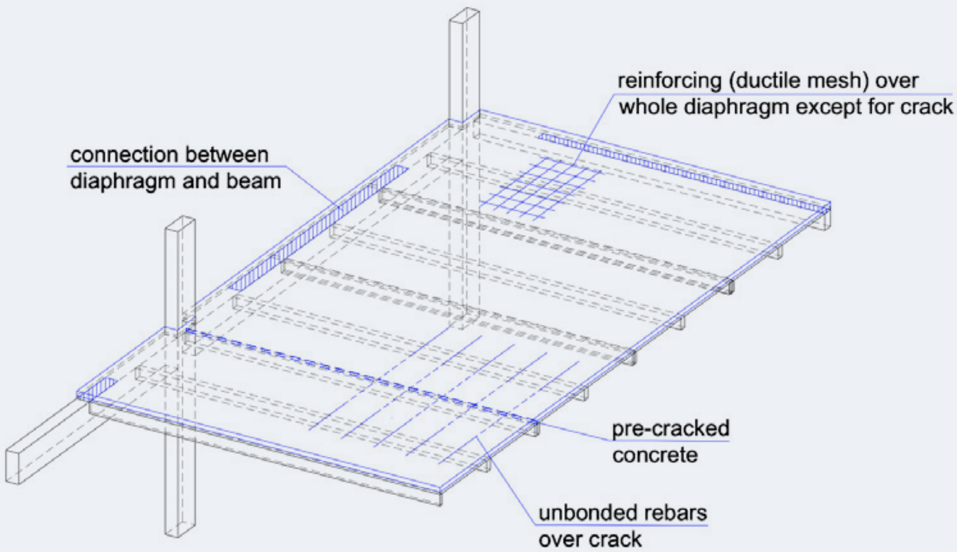

Figure 13.1: Suggested detailing for a TCC floor in a frame system.

The diaphragm has to be tied appropriately to the collector beams. One way to do this is shown in Figure 13.3. The force transfer from the diaphragm to the beam should be guaranteed in the central portion of the beams, leaving it unconnected close to the beam–column joints (in the disturbed areas shown in Figure 13.2). In this way, frame elongation will not compromise the force transfer, which starts away from the disturbed areas where the displacement incompatibility is attenuated. This is especially important on external beams and columns, as no concentrated gap opening can be guaranteed.

A different solution to avoid the frame elongation problem on a multi-bay frame consists in connecting the diaphragm only to one bay and letting the diaphragm slide over the remaining beams. This solution, however, might result in high shear forces at the connection between the diaphragm and the beam, and requires proper detailing to allow for the sliding of the diaphragm in respect to all other elements.

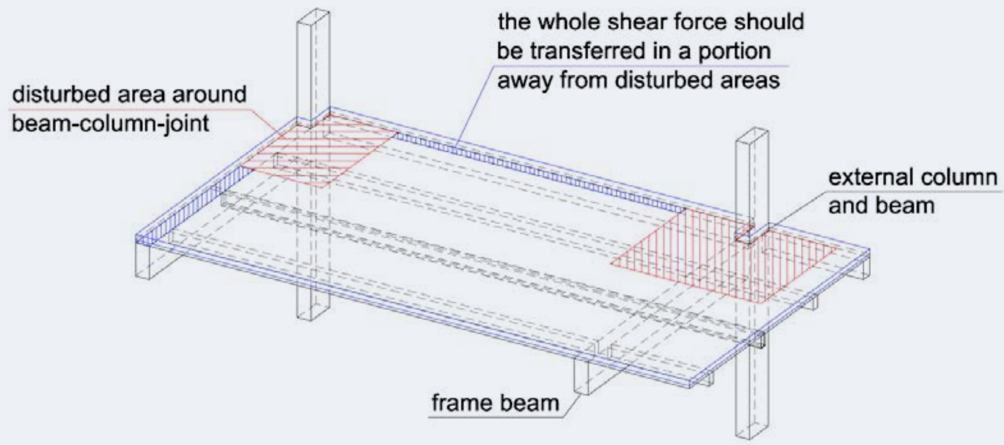

Figure 13.2: Shear transfer between the concrete topping and beams.

If the floor gap opening occurs along a collector beam or tie back, care is needed as cracking of the concrete can compromise the force transfer. Ideally, the pre-crack should be placed away from any connection to the beams.

Figure 13.3 shows a suggested connection between the concrete topping and the collector or frame beam. The diaphragm shear is introduced to the beam via notched connections used for the TCC design (see WoodSolutions Technical Design Guide #30: *Timber Concrete Composite Floors*). If the concrete topping is connected to the beam directly, the beam has to be designed as a composite section. As an alternative, an edge joist from the TCC floor can be connected to the frame beam via a timber-timber connection. Starter bars are required by the code and have to tie the collector/strut beam to the diaphragm as well as carrying the shear in case of a crack along the interface.

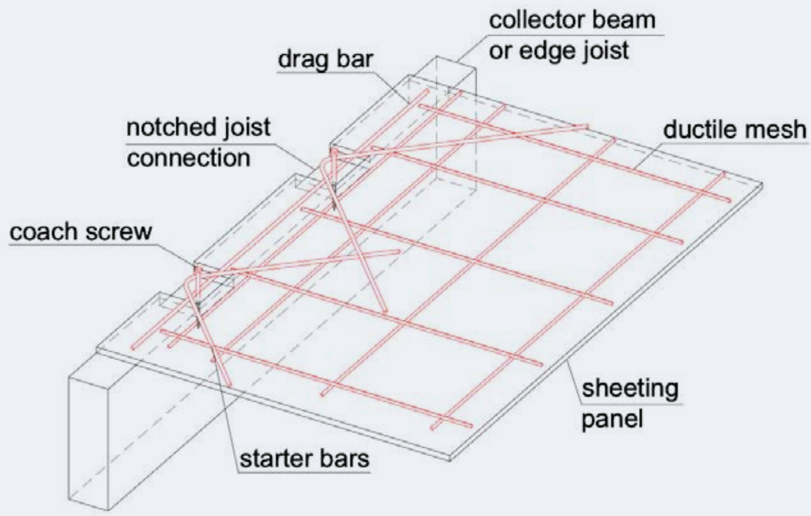

Figure 13.3: Suggested connection between the concrete topping and timber beams.

In wall structures with TCC floors, the force transfer occurs between the wall and the collector beam and is unaffected by the presence of the concrete topping. The connection between the diaphragm topping and the collector/strut beam should be designed as described for frame structures.

If no slotted solution can be adopted for the wall connection, an eventual out of plane bending of the beam has to be considered. Again, if the span to the next vertical restraint is large enough, the bending should be accommodated in the concrete topping without excessive cracking. The additional re-centring force in the wall should be considered, as it might give substantial contribution.

The use of concrete diaphragms in structures that undergo beam elongations cause several complications and special detailing is required[13,14]. The suggestions provided in this Guide have not been fully tested and should be applied with proper engineering judgment.

14 Design of TCC Floor Diaphragm

To design the TCC diaphragm, a strut and tie model as per Section 7 "Strut and Tie Modelling" of the AS 3600 Australian Concrete Code has been adopted. For this design example, only the design for the wind load applied perpendicularly to the long side of the building has been carried out. For wind loads perpendicular to the short side of the building, only a conceptual strut and tie model is shown.

The uniformly distributed loads on the windward and leeward façades (Figure 14.1 and Figure 14.2) for a wind load have been applied on a four metre grid. The resultant forces are transferred by a collector beam running on the inner side of the staircase into the post-tensioned walls. The 100 mm concrete topping is reinforced with a ductile mesh (Ø6.75 mm Grade 500 rebars on 200 mm centres with a resulting reinforcement area of 179 mm² per metre width).

Wind loads at ULS:

$$p = \left(p_{windward} + p_{\text{internal}}\right)h \tag{14-1}$$

$$p_1 = \left(0.35\,{}^{kN}\!/_{m^2} + 0.21\,{}^{kN}\!/_{m^2}\right)3.6m = 2.0\,{}^{kN}\!/_{m} \quad \text{load on windward façade} \tag{14-2}$$

$$p_2 = \left(0.25\,{}^{kN}\!/_{m^2} + 0\,{}^{kN}\!/_{m^2}\right)3.6m = 0.9\,{}^{kN}\!/_{m} \quad \text{load on leeward façade} \tag{14-3}$$

The loads applied to the edge of the diaphragm are carried over compression struts into the collector beam. Several chord beams along the depth of the diaphragm are taking the tension forces, in this way the forces can be kept relatively low.

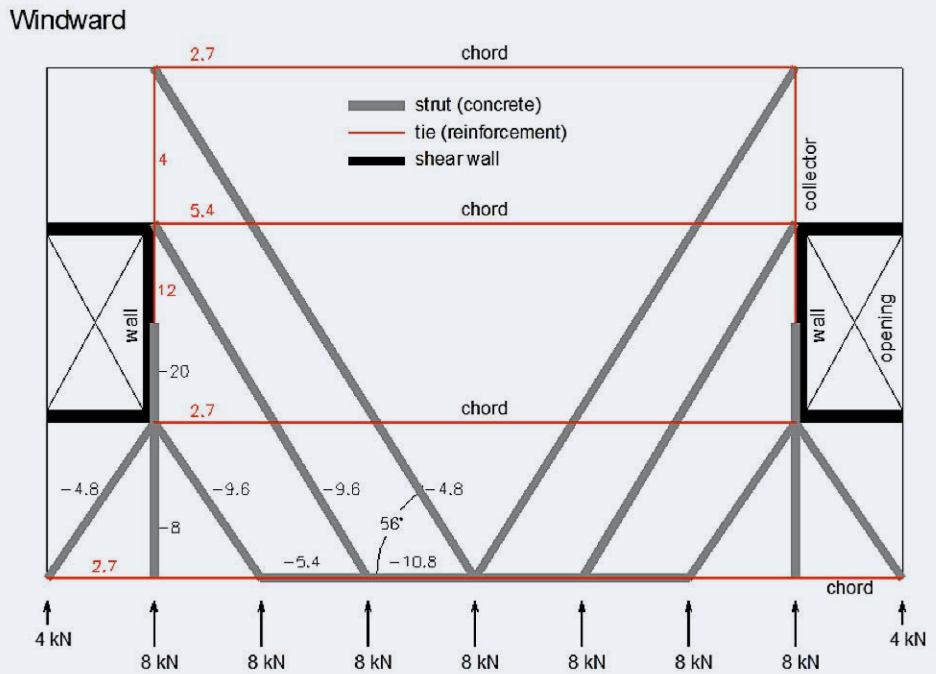

Figure 14.1: Strut and tie model for wind loads on the windward façade.

On the leeward façade, the wind loads are first carried over the tension ties into the diaphragm. From there, compression struts carry the forces into the collector beams.

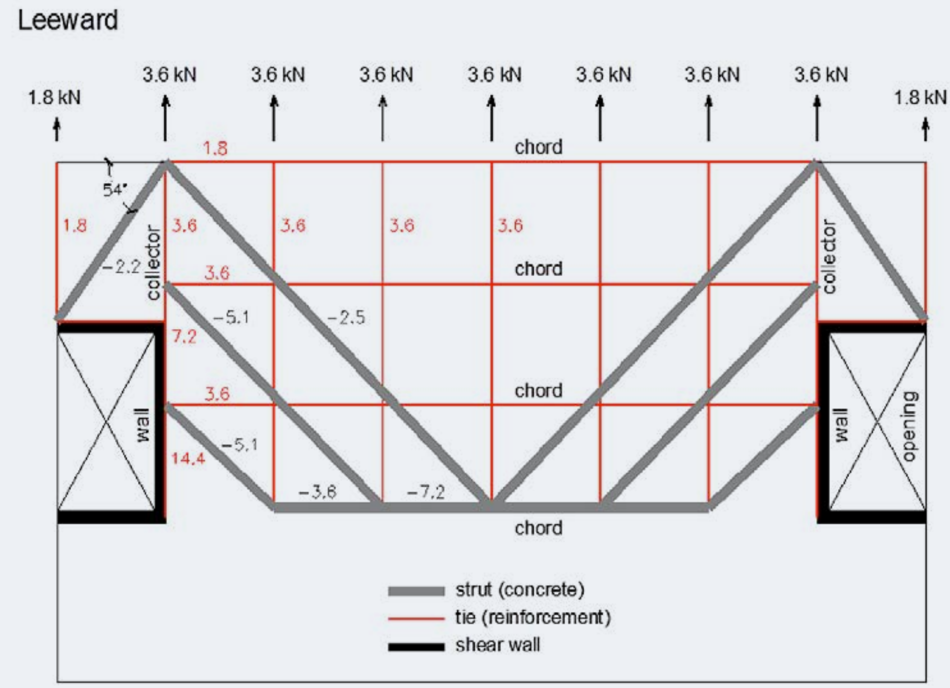

Figure 14.2: Strut and tie model for wind loads on the leeward façade.

There is no unique strut and tie model for a given geometry and load scenario. A minimum of experience is required to set up a well-balanced model, as incomplete or not well elaborated models might lack of equilibrium at the nodes, undergo excessive deformation or require load redistribution because after concrete cracking.

The conceptual strut and tie model for a wind load perpendicular to short sit of the building is depicted in Figure 14.3.

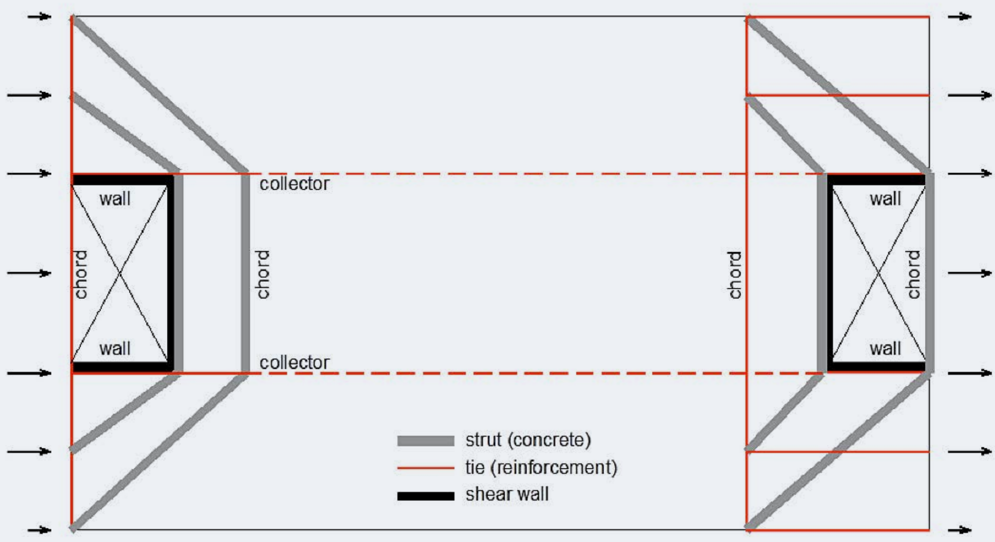

Figure 14.3: Strut and tie model for wind load perpendicular to the short side of the building.

14.1 Tension Ties

The concrete topping is reinforced by a D500DL72 ductile mesh (reinforcing in both direction made of Grade 500 reinforcing bars), which also satisfies the minimum reinforcement for crack control for shrinkage and temperature effects as per Clause 9.4.3 in AS 3600:

$$A_{s,\min} = 75\% \cdot \left(1.75 - 2.5\sigma_{cp}\right) bD \cdot 10^{-3} = 0.75 \cdot 1.75 \cdot 100mm \cdot 1000mm = 131mm^2 \qquad (14\text{-}4)$$

where

$A_{s,min}$ = minimum reinforcement area
σ_{cp} = average intensity of effective pre-stress (0 MPa in this case)
b = width of the diaphragm (taken as 1 m)
D = depth of the concrete topping.

As shown in Figure 14.1 and Figure 14.2, the maximum force in the ties is 12+14.4 = 26.4 kN along the collector beam running parallel to the wall. Along the collector beam, two additional Ø10 Grade 300 reinforcing bars are placed.

$$F_{nt} = \phi_{st} A f_y = 0.8 \cdot \frac{10^2 \cdot \pi}{4} \cdot 300 = 48kN \geq F^* = 26.4kN \therefore OK \qquad (14\text{-}5)$$

where:

F_{nt} = nominal tension capacity of steel tie
A = area of the tension reinforcement
ϕ_{st} = capacity factor for tension struts (0.8).

All other ties have forces of maximum 5.4 kN; therefore, a single leg of the ductile mesh with a Ø6.75 mm Grade 500 rebar provides enough strength to transfer the tension forces.

$$F_{nt} = \phi_{st} A f_y = 0.8 \cdot \frac{6.75^2 \cdot \pi}{4} mm^2 \cdot 500 \, {}^{N}\!/_{mm^2} = 14.3kN \geq F^* = 5.4kN \qquad (14\ 6)$$

To guarantee the force transfer in the tension ties, the reinforcing bars and the ductile mesh have to be placed with the required overlapping as provided by the code or the manufacturer.

14.2 Compression Struts

The design strength of a concrete strut, neglecting the reinforcement, is:

$$F_{nc} = \phi_{st} \beta_s 0.9 f'_{conc} A_{conc} \qquad (14\text{-}7)$$

where

F_{nc} = nominal compression capacity of concrete strut
f'_{conc} = compressive strength of concrete
A_{conc} = cross sectional area at one end of the strut, considering the thickness as the depth of the diaphragm slab (see AS 3600 Australian Concrete Code for more detail)
β_s = efficiency factor for concrete struts
ϕ_{st} = capacity factor for compression struts (0.6).

Considering the maximum force in a strut of only 20kN and a thickness of the slab of 100 mm, all struts are easily verified. Detailed verifications of the struts and the nodal areas are left to the reader.

The reinforcement plan for the concrete diaphragm is shown in Figure 14.4. Appropriate overlapping of the reinforcing bars and the mesh has to be guaranteed.

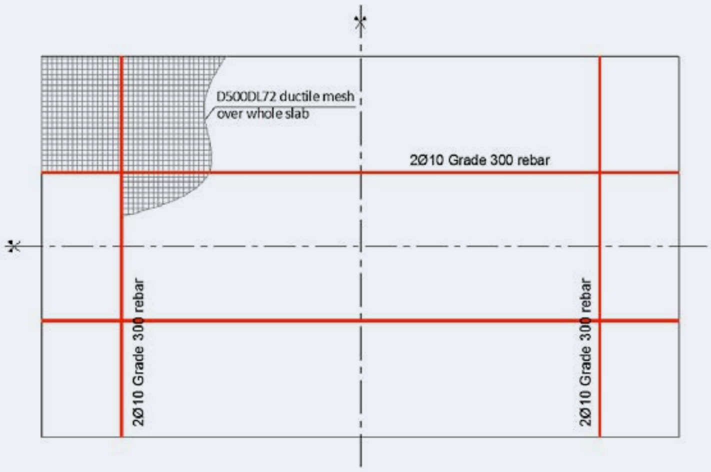

Figure 14.4: Reinforcement plan.

14.3 Connection of the Collector to Walls

The force transfer between the diaphragm and the wall can be realized in different ways. Two design solutions, which are also compatible with the gravity force transfer, are shown. Since the uplift and rotation of the wall is negligible for this design, no special detailing for the connection between the floor and the lateral load-resisting system is required.

Solution 1– Direct Connection between Concrete Slab and Wall

Solution 1 consists in a diaphragm force transfer via coach screws fixed directly to the LVL post-tensioned wall. These are then integrated in the concrete slab when it is cast into place. The floor elements are sitting on corbels or fixed by steel hangers that are directly connected to the wall. The design of the latter is not shown here.

The diaphragm force to be transferred into the wall is 46.4 kN (26.4 kN from the ties and 20 kN from the strut). This force is transferred from the concrete topping to the wall through 6 Ø12 coach screws (Figure 14.5).

According to clause C4.2 of AS 1720.1:2010, and considering an effective timber thickness of $b_{eff} = 2 \times t_p = 192$ mm and embedment strength of $f'_{pj} = 17$ MPa, the connection capacity is as follows:

$$t_p \geq 8D = 8 \cdot 12mm = 96mm \tag{14-8}$$

$$Q_{sk} = Q_{skp} = \min\begin{Bmatrix} b_{eff} f'_{pj} D/2 \\ 15 f'_{pj} \sqrt{D^3} \end{Bmatrix} = \min\begin{Bmatrix} 19.6kN \\ 10.6kN \end{Bmatrix} = 10.6kN \tag{14-9}$$

where:

t_p = penetration length of fastener

Q_{sk} = characteristic capacity for a laterally loaded single bolt in a joint system

Q_{skp} = system capacity for fasteners loaded perpendicular to the grain

f'_{pj} = characteristic value for bolts bearing perpendicular to grain

The second member in the connection is the concrete slab, which can be considered as stiff; hence, the factor for side plates (k_{16}) can be taken as 1.2.

$$N_{dj} \geq N^*$$
$$N_{dj} = \phi k_1 k_{13} k_{16} k_{17} n Q_{sk} = 0.8 \cdot 1.14 \cdot 1.0 \cdot 1.2 \cdot 1.0 \cdot 6 \cdot 10.6kN = 69.6kN \geq N^* = 46.4kN \tag{14-10}$$

where

N_{dj} = design capacity for joints under direct load

k_1 = 1.14 for wind loads as per Clause 2.4.1.1.

k_{13} = 1.0 factor for end grain effects

k_{16} = 1.2 factor for side plates

k_{17} = 1.0 factor for multiple fastener effect.

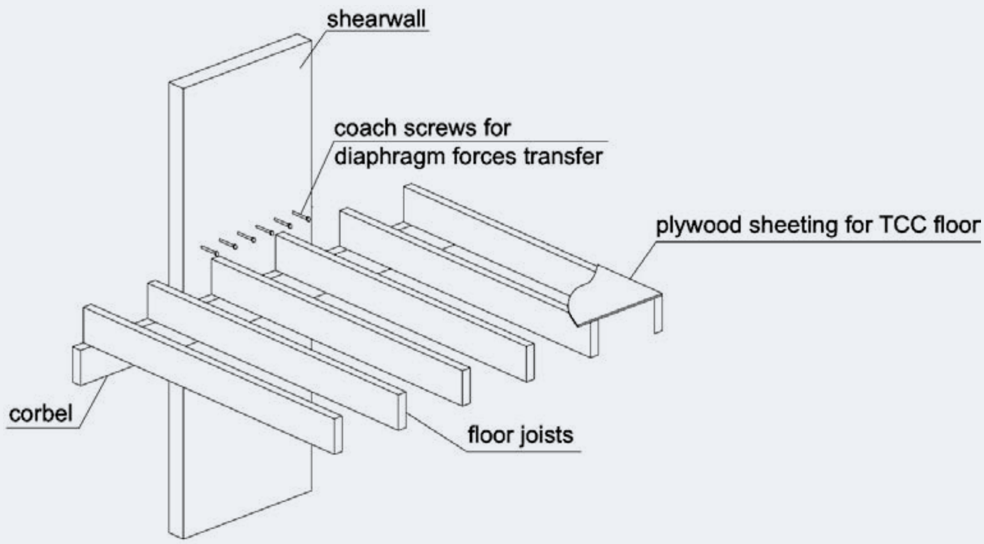

Figure 14.5: Diaphragm force transfer over coach screws.

The strength of the coach screws embedded in the concrete can be checked according Australian Code AS 2327.1:2003 Composite structures. Part 1: Simply supported beams.

Solution 2 – Connection between Concrete Slab and Wall via Strut/Collector Beam

An alternative solution consists in a transverse LVL beam running parallel to the wall, fixed with bolts to it. The diaphragm forces are transferred via notched connections from the concrete topping into the timber beam (Figure 14.6). For the gravity loads, the floor joists are connected to the transverse beam by steel hangers. The connection of the beam to the wall is designed for the combination of gravity and horizontal wind loads.

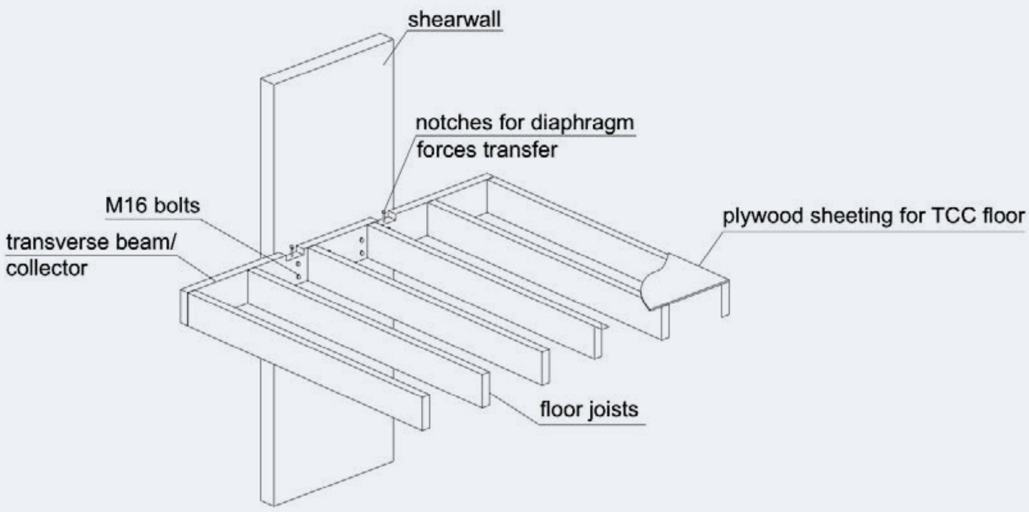

Figure 14.6: Transverse beam with notches for the diaphragm force transfer.

The factored gravity load is:

$$p = 1.2G + 1.5\psi_a Q = 6.7 \, {}^{kN}\!/_{m^2} \tag{14-11}$$

and hence the gravity force on the beam considering a tributary area approach is:

$$F = p \cdot A = 6.7 \, {}^{kN}\!/_{m^2} \cdot \frac{4m}{2} \cdot 7m = 94kN \tag{14-12}$$

From the wind load a horizontal force of 46.4 kN has to be transferred into the wall. This leads to a resultant force and respective angle of:

$$R^* = \sqrt{46.4^2 + 94^2} = 105 \text{ kN} \tag{14-13}$$

$$\vartheta = \arctan\frac{94}{46.4} = 63.7^\circ \tag{14-14}$$

M16 bolts are used to connect the beam to the wall. A joint group JD3 is assumed for the connection in LVL elements. The thickness of the connected members is 90 mm for the beam and 225 mm for the wall. Since the Australian Timber Code AS 1720.1:2010 does not provide the situation of a force transferred on an angle to the grain in between two members running at 90° to each other, as a conservative approach the resultant force is applied perpendicularly to the beam.

Considering b_{eff} = 2 x 90 mm = 180 mm and f'_{pj} = 17 MPa the connection capacity is:

$$Q_{sk} = Q_{skp} = \min\begin{Bmatrix} b_{eff} f'_{pj} D/2 \\ 15 f'_{pj}\sqrt{D^3} \end{Bmatrix} = \min\begin{Bmatrix} 24.5kN \\ 16.3kN \end{Bmatrix} = 16.3kN \tag{14-9}$$

A connection with 8 M16 bolts is chosen to transfers the load from the beam into the wall:

$$\begin{aligned} &N_{dj} \geq R^* \\ &N_{dj} = \phi k_1 k_{16} k_{17} n Q_{sk} = 0.8 \cdot 1.14 \cdot 1.0 \cdot 1.0 \cdot 8 \cdot 16.3kN = 119 \geq R^* = 105kN \end{aligned} \tag{14-10}$$

A different way to connect the collector beam to the wall could be by using inclined fully threaded screws; this is, however, not covered here.

To transfer the diaphragm force from the concrete topping into the beam, two notched trapezoidal connections are provided (Figure 14.7). More information on the design of these connections can be found in the WoodSolutions Technical Design Guide #30:*Timber Concrete Composite Floor Systems*.

$$2N_{dj} = 152.4 > Q^* = 64.4\text{kN} \tag{14-15}$$

$$2N_{dj} = \phi k_1 k_4 k_6 Q_k = 0.8 \times 1.14 \times 1.0 \times 1.0 \times 83.5\text{kN} = 76.2\text{kN} \tag{14-16}$$

$$Q_k = 0.95 \times 90 - 2 = 83.5\text{kN} \tag{14-17}$$

where:

Q_k = characteristic strength of the TCC connection in shear

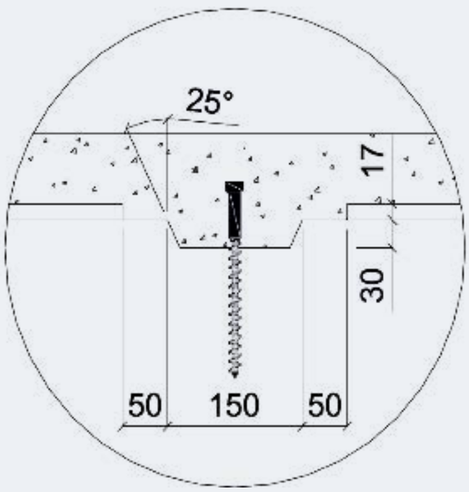

Figure 14.7: Trapezoidal notch.
Source: WoodSolutions Technical Design Guide #30:Timber Concrete Composite Floor Systems.

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Australian Standards

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AS 1170.2:2002 *Structural Design Actions Part 2: Wind Actions*. 2002, Standards Australia, Australia.

AS 2327.1:2003 *Composite structures, Part 1: Simply supported beams* 2003, Standards Australia, Australia.

AS 3600 *Concrete structures*, 2009, Standards Australia, Australia.

WoodSolutions Design Guide

WoodSolutions Technical Design Guide #30, *Timber Concrete Composite Floor System.* WoodSolutions, 2015, Melbourne, Australia.

A

Appendix A - Notations

The symbols and letters used in the Guide are listed below:

a	aspect ratio of each sheathing panel given in the NZS 3603.[12]
A	section area of the chord
A	area of the tension reinforcement
A_{conc}	cross sectional area at one end of the strut, considering the thickness as the depth of the diaphragm slab as per AS 3600
$A_{s,min}$	minimum reinforcement area
b	width of the diaphragm (taken as 1 m)
B	distance between diaphragm chord member
b_{eff}	effective width of member in joint assembly
D	depth of the concrete topping
e_n	fastener slip resulting from the shear force V
F	gravity force on the beam
F_{nt}	nominal tension capacity of steel tie
F_{nc}	nominal compression capacity of concrete strut
F^*	design action in tension
f'_{conc}	compressive strength of concrete
f'_{pj}	characteristic value for bolts bearing perpendicular to grain
f_y	yield strength of steel
G	shear modulus of the diaphragm sheathing
h	diaphragm width
l	length of diaphragm
k_1	modification factors for duration of load given in AS 1720.1
k_{13}	nail connector factor for end grain effects given in AS 1720.1
k_{16}	nail connector factor for plywood or metal side plates given in AS 1720.1
k_{17}	nail connector factor for multiple fastener effect given in AS 1720.1
m	thickness of the diaphragm sheathing
M	design moment of diaphragm
N_{dj}	design capacity for joints under direct load
N^*	design action for joints under direct load
P	wind load at ULS
P_1	wind load on windward façade
P_2	wind load on leeward façade
p	factored gravity load
q	uniformly distributed horizontal load
Q_k	characteristic strength of the TCC connection in shear
Q_{sk}	characteristic capacity for a laterally loaded single bolt in a joint system
Q_{skp}	system capacity for fasteners loaded perpendicular to the grain

R^*	resultant force
t	thickness of the diaphragm sheathing
t_p	penetration length of fastener
V	shear force applied to a diaphragm
v	shear flow along the edges of the diaphragm
x	distance of the splice from the origin
β_s	efficiency factor for concrete struts
σ_{cp}	average intensity of effective pre-stress (0 MPa in this case)
δ_s	splice slip in the chord
Δ_1	deflection due to bending
Δ_2	deflection due to shear in the panels
Δ_3	deflection due to the connection elements
Δ_4	deflection of the diaphragm due chord connection deformation
$\Delta_{diaphram}$	diaphragm deflection at mid-span
$\Delta_{interstory\ drift\ LLRS}$	lateral-ing system drift
ϕ_{st}	capacity factor for tension struts (0.8)
ϑ	angle resultant force transferred into the wall

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