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Briefing

- Panel systems and dimensions
- Stability and temporary bracing
- Roof bracing and lateral support
- Connection to structural steelwork
- Shear walls and moments on panels
- Support of panels
- Ground floor slab plan details
- Connecting suspended slabs to panels
- Shop detailing of panels

Concrete Panel Buildings

Design Considerations for Low-rise Industrial and Commercial Buildings

With the increasing use of concrete panel construction for the entire range of buildings from residential to high-rise, and whether the concrete panels are cast on-site or factory precast, this Briefing Note outlines some of the engineering design issues that should be considered in the use of this type of construction.

Introduction

Concrete panel construction is increasingly being used for the entire range of buildings, from residential to high-rise. This Briefing Note outlines some of the engineering design issues that should be considered in the use of this type of construction for industrial or commercial applications, irrespective of whether the concrete panels are cast on-site or factory precast.

The Cement and Concrete Association of Australia acknowledges the significant contribution of Ian Hymas from H & H Consulting Engineers Pty Ltd (trading as Henry & Hymas) to the content of this document.

Preliminary Considerations

The structural design process is commenced by dividing the walls into panels of appropriate size. This involves:

Site constraints

The constraints of the site may determine the maximum size and weight of panel that can be lifted into position, and whether or not panels can be cast on-site. For example a narrow allotment, basement or slope may not allow crane access or space to cast panels, while overhead power lines may limit the height of the crane boom.

Joint locations

Door and window openings should be considered when determining the location of joints. The panel width next to an opening, and the panel depth over a wide opening, must provide sufficient structural strength to allow for the support of the panel, plus lifting and erection forces.

Economics

- Structural layout - the structural layout, particularly the rafter spacing, will often determine the joint locations and panel dimensions. Panel dimensions should not dictate the frame spacing as the cost per square metre for panels of comparable size is about the same, whereas closer than optimum frame spacing can incur considerable extra cost. If factory precast panels are used, transport restrictions on panel size may dictate the layout.
- Construction at or near the boundary - if walls are located at or near the boundary, this will affect the footing design. Also, if adjacent buildings are on the boundary, consideration needs to be given to matters such as the erection process, base joint details, sealing of wall joints and flashing between buildings, as work must be completed from one side only.
- Rafters supported by panels - can the panel layout and openings allow panels to be designed to carry the roof framing, or are columns required in some locations?
- Casting areas - can the floor slab be used as a casting bed or are temporary casting beds required? To ensure the quality of panels, the surface tolerances, joint details (sealing of joints) and surface finishing requirements should be considered. Panels precast off-site will eliminate the need for areas to be made available for casting, a significant consideration if the site is congested.
- Stack casting - the stacking order to enable an efficient erection sequence should be considered. Panels with openings should preferably be placed on top of stacks to eliminate the need for blocking out the openings to form subsequent panels.
- Cranage - consideration should be given to matters such as crane capacity, lifting locations to minimise the number of setups and access.

Note: While the Engineer needs to consider all these matters, others involved in the process also have responsibility for specific areas, eg casting, delivery sequence, cranage, temporary propping.

Marketability of the building

This involves both the utility and aesthetics of the building. Utility covers items such as the requirements for access, column-free space and flexibility to meet the needs of subsequent owners/occupiers. Aesthetics involves an acceptable level of finish to the panels and facade features to give an appealing exterior. Provision for later removal of panels to create openings or allow the building to be extended may also be a consideration.

Crane Size

While the final practicality of lifting panels must be verified with the crane contractor using appropriate crane charts (for the available crane), the information in **Table 1** can be used as a starting guide to the panel weights that can be lifted by various sized cranes.

Table 1: Crane size to panel weight

Crane Capacity (t)	Panel Weight Typical (t)	Panel Weight Heavy (t)
50 (hydraulic)	10 - 15	18
70	12 - 18	23
120 - 140	18 - 25	35
200	10 - 28	50

'Heavy' panels are panels the crane may lift from a favourable position, ie lifted and erected close to the crane setup position. It is unlikely that all panels will be 'heavy' panels, so a few 'heavy' panels may be able to be accommodated by planning the casting location or delivery access, and crane setup with respect to the final panel location.

Once the crane size has been confirmed, the panel area and size can then be determined. As a guide, for the common panel thicknesses, the panel weights for a concrete density of 2400 kg/m^3 are given in **Table 2**. For heavily reinforced panels, the additional weight of the reinforcement may need to be considered to accurately determine the appropriate cranage.

Table 2: Weight of common panel thicknesses

Panel thickness (mm)	Panel weight (t/m^2)
130	0.32
150	0.36
175	0.42

Panel Systems

Concrete panels can be used either as cladding to the building, or as part of the loadbearing structure, supporting roof and wind loads, **Figure 1.**

Using panels as loadbearing elements generally reduces the overall cost due to a decrease in the weight of structural steelwork required. Even though more roof bracing is required, eliminating the columns usually provides greater savings. Also, in terms of the programming, the panels can be cast and erected prior to the delivery of steelwork, allowing time for fabrication of the steelwork. If the concrete panels are used as cladding, the steelwork is usually on the critical path for the project.

Panel Dimensions

Having assessed the maximum panel weight that can be lifted, the panel dimensions can be determined by considering the wall height and lengths required, and any openings to be included.

For site-cast panels with openings, while the following 'rules of thumb' provide a starting point in determining the minimum dimensions for legs, mullions and spandrels, the requirements for each individual panel need to be considered. Referring to **Figure 2:**

- 600 mm is usually a comfortable minimum leg or mullion width.
- 900 mm is usually a comfortable minimum spandrel depth.

Some leg and mullion widths may need to be more than 600 mm in width, **Figure 3.**

Some leg and mullion widths could be less than 600 mm in width, **Figure 4**. If in doubt, a useful test is to consider if the panel would still be structurally satisfactory without the mullion. If the answer is yes, then it may be possible to reduce the mullion width to less than 600 mm. However, consideration should be given to the vulnerability of thin mullions to damage by accidental loading during erection.

For factory precast panels, minimum leg, mullion and spandrel dimensions may be controlled by transportation requirements rather than lifting stresses alone.

Figure 1: Types of panel systems

(a): Cladding **(b):** Loadbearing

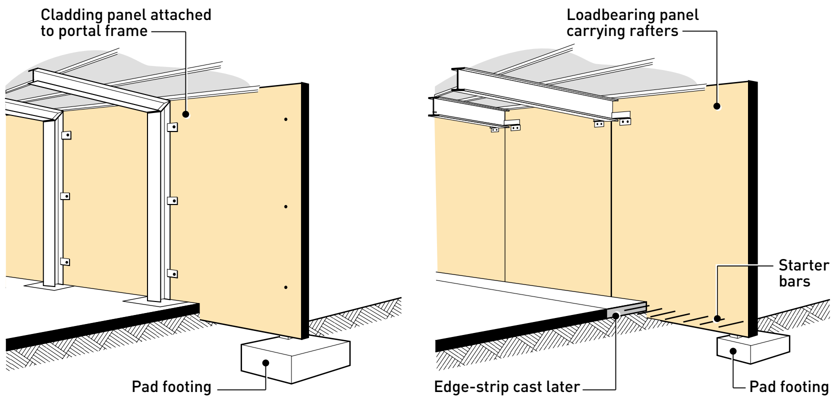


Figure 2:
Minimum recommended widths/depths of panel legs, mullions and spandrels

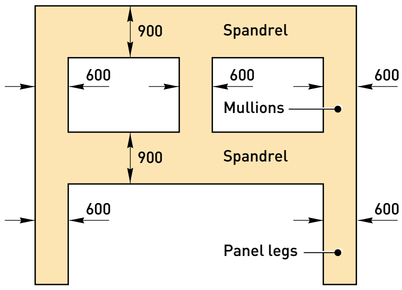


Figure 3:
Situations where leg or mullion widths may need to be more than 600 mm

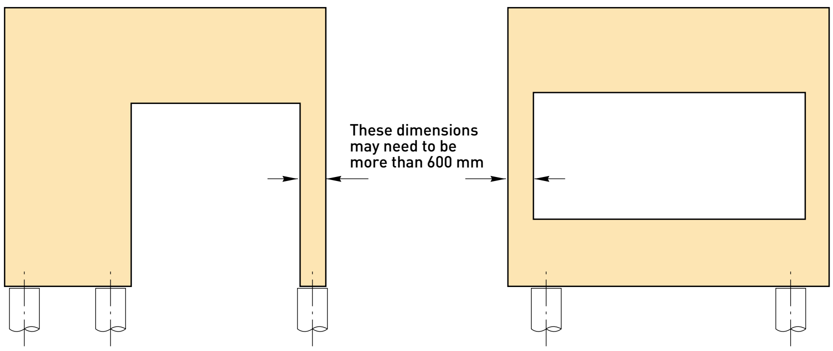
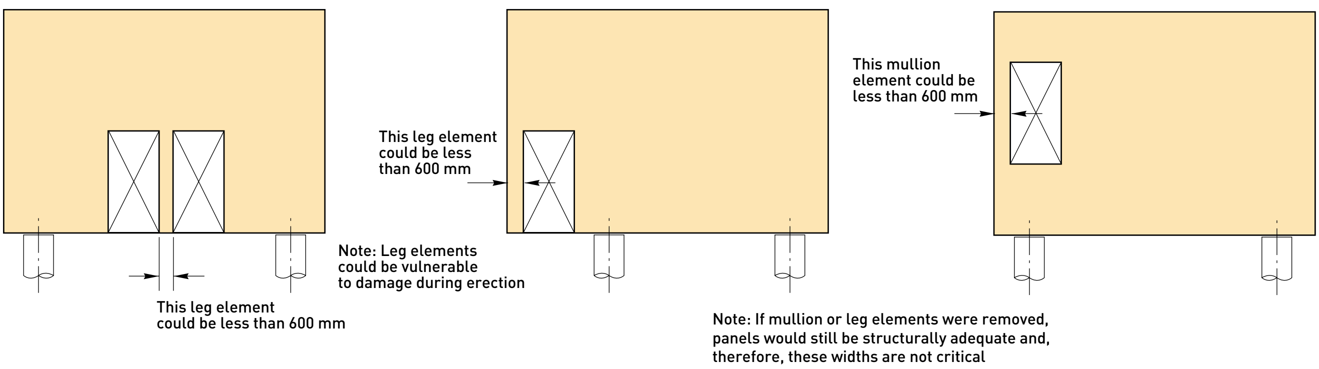


Figure 4: Situations where leg or mullion widths could be less than 600 mm



Building Stability

Concrete panel buildings may be prone to progressive collapse, and redundancy should be incorporated into the structural system that provides stability and robustness. For instance, the failure of a single bracing member due to an event such as a fire, accident or abuse, should not lead to the collapse of the building. Consideration may therefore be given to providing two sets of cross bracing to give an alternative load path, **Figure 5**.

Care should be exercised in using purlins as part of the roof bracing system because of the deflections and secondary moments (particularly if bracing members are located in different planes) that may occur. Also, they are prone to early failure in a fire.

Temporary Bracing (Props)

Temporary bracing is used to support the panels prior to their incorporation into the final structure. This is the stage where stability of the erected panels, and prevention of collapse are critical. Consideration should be given to the following:

- Each panel should typically be supported by a minimum of two braces.
- Braces should be positioned so that they are uniformly loaded, ie if using three braces, they should be spaced along the panel so that the central brace carries one third of the loading.
- Panels should not be braced off other temporarily braced panels unless specified by the Panel Engineer (refer section on shop detailing of panels).

  This also applies to leaning panels against other panels, during say relocation of the crane or while lifting panels further down the stack, which must be erected first.
- If temporary braces are skewed (ie not perpendicular to the panels in plan), then the design and installation of the panel, as well as the support of the brace, need to be checked for the loads induced by the skewed brace.
- Only superimposed loads that have been designed for are to be applied to panels while temporarily braced, as braces and fixings are usually only designed for wind loads.
- The anchorage for braces (usually the slab or a dedicated pier) must be able to resist the temporary loads. If anchoring to the slab, consideration should be given to specifying minimum distances to the slab edge.
- Connections should be checked after installation, to ensure the correct torque settings for anchors have been achieved. Use of expansion anchors near slab edges or joints should be avoided.
- Braces and connections must be regularly inspected to ensure adequacy. Note that the cyclic nature of wind loading can loosen connections.
- The Structural Engineer should inspect and approve the permanent bracing system prior to the removal of any temporary propping.

Figure 5: Designing redundancy into the bracing system

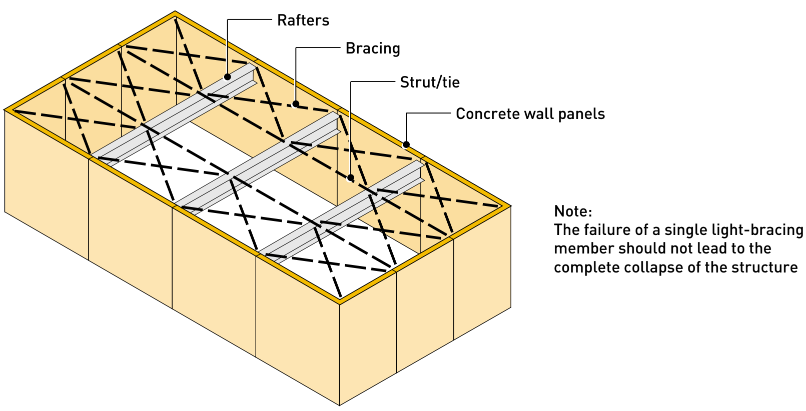
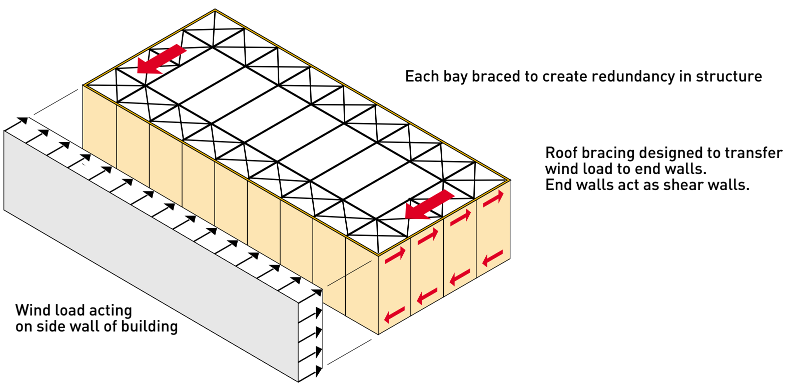


Figure 6: Brace each bay to transfer lateral loads on the walls to the supporting cross walls



Roof Bracing

Typical Layout

Each bay must be braced in order to transfer the lateral loads on the walls to the supporting cross walls, **Figure 6.**

Bracing Design

The 'traditional' model for wind truss analysis involves applying the lateral loads as forces at the truss nodal points and calculating the reactions to be supported by the shear walls at the ends of the roof truss, **Figure 7**. In this type of analysis, the tension and compression loads in the chord members are generally largest at the centre of the truss.

To reduce the load in the chord member adjacent to the panels (and perpendicular to the wind direction), an alternative 'model', which uses the shear wall capacity of the panels perpendicular to the wind direction, can be used, **Figure 8**. Here, the resultant horizontal loads at each of the nodal points are taken out by the shear capacity of the adjacent panels. This chord strut or tie can therefore typically be reduced to a nominal angle section.

Note: As the panel joints are typically located at rafter centres (or nodal points), the angle member along the panel will normally have slotted holes at the ends of the panel to allow for long-term shrinkage of the concrete panels. The horizontal loads are therefore not transferred from the steelwork at the ends of the panels, but carried by one or two rigid connections in the central area of the panel, **Figure 9.** Rigid connections include welding the angle to a cast-in plate, **Figure 10**, bolting to the panel through a non-slotted hole in the angle or using a plate washer welded to the angle over a slotted hole in the angle (to allow for construction tolerances).

Lateral Support of Panels

For large panels, the joints between panels are generally located at rafter centres so that the roof framing (or portal frames when panels are used as cladding only) laterally supports both panels, **Figure 11**.

For smaller panels, an intermediate strut can be provided to laterally support the panels at the intermediate joint location, **Figures 12** and **13**.

Figure 7: Traditional model for wind truss analysis

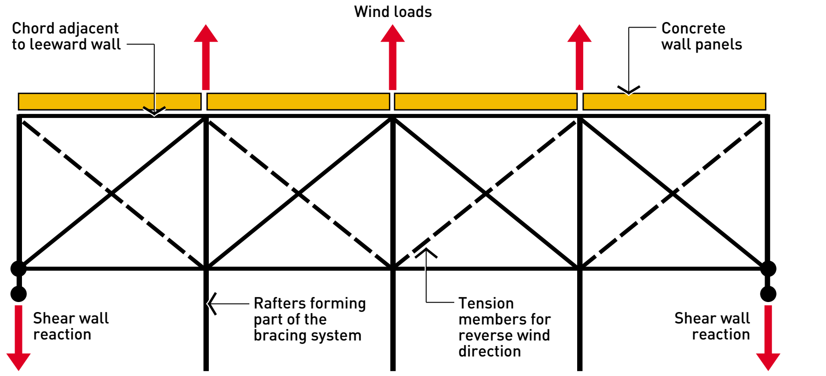


Figure 8: Alternate model for wind truss analysis

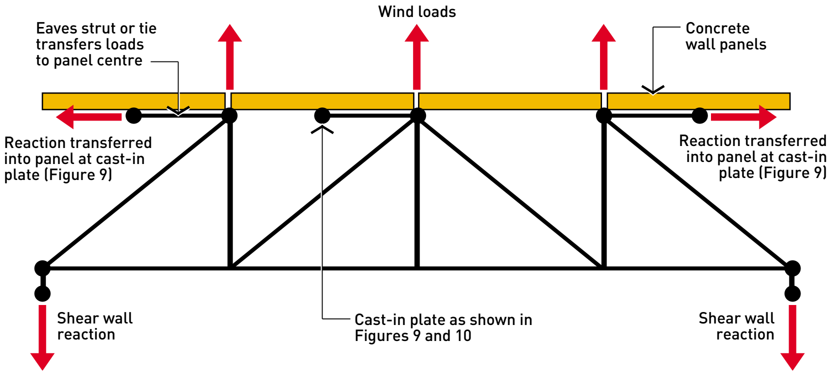


Figure 9: Transfer of loads into panel

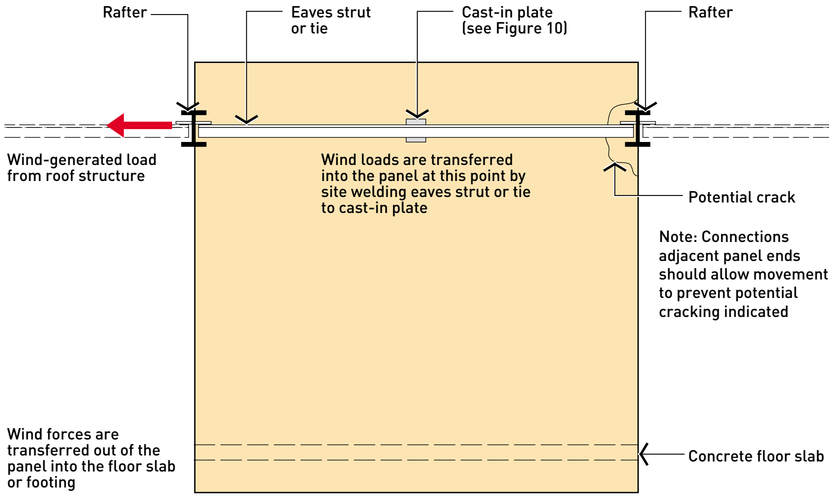


Figure 10: Eaves strut or tie welded to cast-in plate

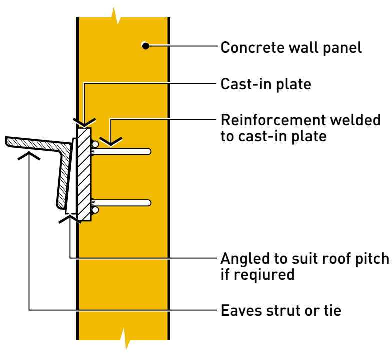


Figure 11: Panel joints located at rafter centres

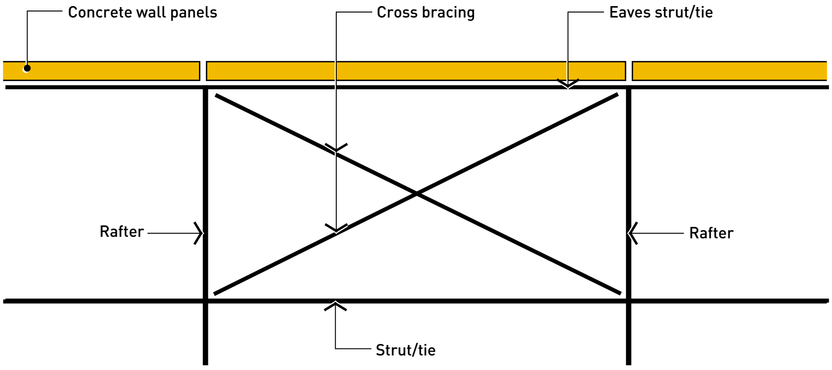


Figure 12 : Strut to support joint between smaller panels

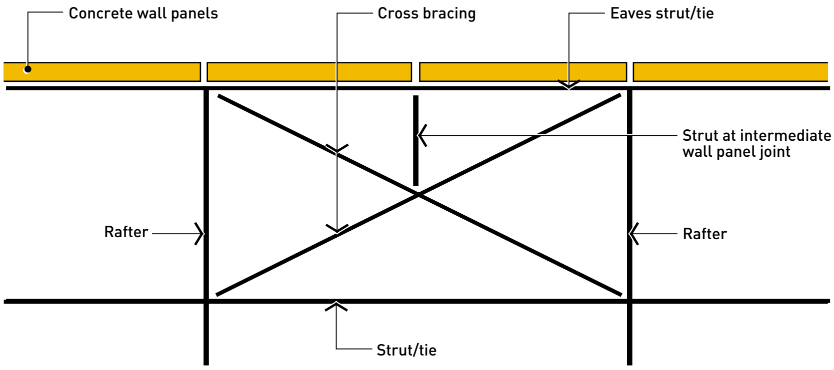


Figure 13: Strut to support joint at smaller panel

Figure 14: Rafter to panel connection at joint

Connection of Structural Steelwork to Panel

Eaves strut or tie to panel

In the fixing of these members to the panels, allowance must be made for erection tolerances and the shrinkage of the panels, **Figure 9**. These members are typically bolted onto the panels via cast-in ferrules. Welding onto cast-in steel plates is also an option. Allowance for movement and construction tolerances can be achieved by providing slotted holes in the eaves strut or tie member, or enlarged holes with plate washers welded onto the member.

Note: Any wind loads that are to be transferred into the panel (to utilise shear wall action) should be transferred in rigid connections in the central area of the panel.

Rafter to panel

If panels are used to support the roof framing, a pin-ended connection between the rafter and the concrete panel should be provided to avoid moments being transferred into the panel. Slotted holes should also be used in the brackets supporting the rafter member if it is located at the edge of the panel, to allow for lateral movement (ie concrete shrinkage and thermal movement) of the panels. A suitable connection detail is shown in **Figure 14**.

Note: The angle supporting the rafter ensures minimum eccentricity of the vertical load being transferred into the panel, and also allows rotation, thus limiting the transfer of moments from the rafter into the panel.

The bolted connection shown in **Figure 15** will transfer some moments from the rafter into the panel, making this detail susceptible to pullout of the top row of ferrules due to end rotation of the rafter. Also, while slotted holes have been left in the fixing plate, care should be taken that the allowance for movement is not taken up with construction tolerances. Insufficient tolerance will allow the transfer of lateral forces into adjacent panels, effectively connecting the panels and imposing high loads on the cast-in inserts adjacent to the edges. Also, no provision for an eaves member appears to have been made, in order to transfer forces to the centre of the panel, see **Figure 9**.

Care should also be taken with connection details similar to that shown in **Figure 16**. As a general rule, stiffness increases eccentricity. The stiffer the supporting bracket the greater will be the eccentricity of the load from the rafter. The bracket shown in **Figure 16** welded to the cast-in plate incorporates three vertical stiffener plates. These stiffener plates mean that the vertical load from the rafter will be located very close to the end of the bracket. If rigid brackets are used, the panels should be designed for the moment resulting from the vertical load being applied at the end of the bracket.
Note: There is no eaves member, and the edge purlins are being used in lieu of it. The diagonal tie is located well below the flange and torsion will be induced in the rafter. As mentioned earlier under building stability, care should be exercised in using purlins as part of the roof bracing system because of the deflections and secondary moments that may occur, particularly if bracing members are not located in the same plane. Also, purlins are prone to early failure in a fire.

Figure 15: Panel/Rafter connection - not recommended

Figure 16: Panel/Rafter connection - not recommended

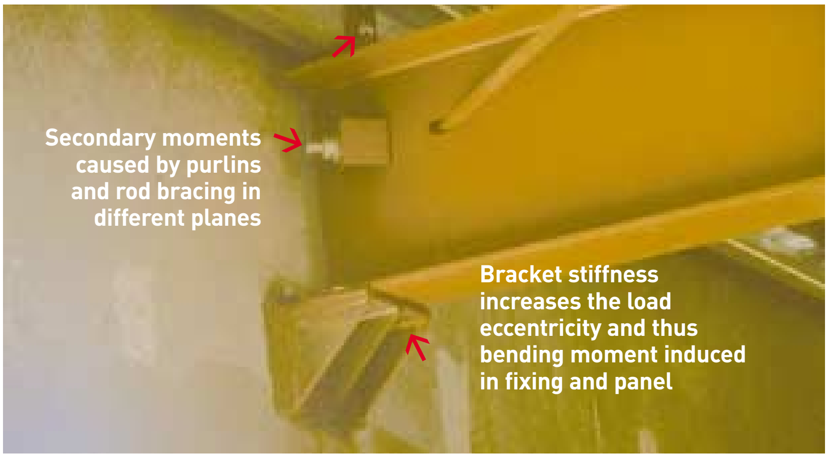


Figure 17:
Multiple panels as shear walls

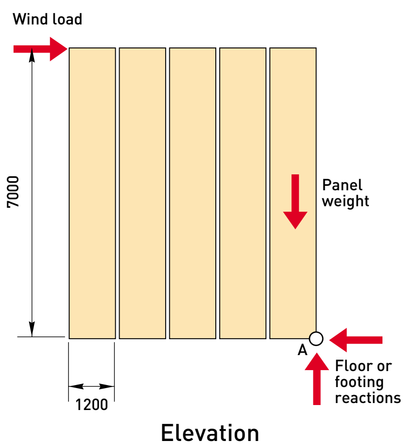
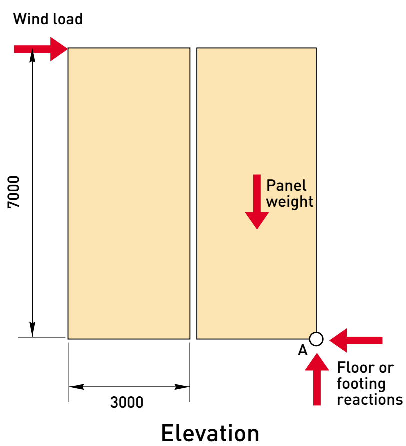


Figure 18:
Two larger panels as shear wall



Panels used as Shear Walls

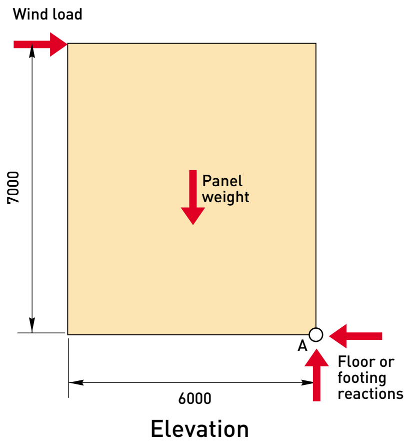
Referring to **Figure 17**, consider firstly the use of a number of narrow panels as the shear-wall elements resisting the lateral forces from the roof framing. The capacity of five panels of size 1200 mm wide, 7000 mm high and 150 mm thick, is as follows:
Panel weight = 30 kN each
Restoring moment about point A
1 panel = 30 x 0.6 = 18 kN.m
5 panels = 90 kN.m

Referring to **Figure 18**, if two larger panels (of typical precast width) of similar dimensions are used (ie total 6000 mm wide, 7000 mm high and 150 mm thick) the capacity is as follows:
Panel weight = 76 kN each
Restoring moment about point A
1 panel = 76 x 1.5 = 114 kN.m
2 panels = 2 x 114 = 228 kN.m

Referring to **Figure 19,** if a single panel of similar dimensions is used (ie 6000 mm wide, 7000 mm high and 150 mm thick) the capacity is as follows:
Panel weight = 150 kN
Restoring moment about point A
= 150 x 3.0 = 450 kN.m

Factory precast panels have a maximum width of about 3 m, which is dictated by transportation limits on public roads. However, it can be seen that these 3-m panels have a restoring moment about 2.5 times that of a series of narrow panels of the same overall dimensions.
The single large panel has a restoring moment about 5.0 times that of the series of narrower panels, or twice that of two smaller panels. Thus it is particularly important to check stability under shear loads when using narrow panels in lieu of wide elements.

Figure 19:
Single large panel as shear wall



Moments on Panels

Figure 20 details the moments on the panel that need to be designed for, after it has been incorporated into its final position within the building.
Note: The moments due to erection loads are not covered here, as the tilt-up stresses are normally allowed for by an Engineer working for the panel contractor. Further information can be found in the Concrete Institute of Australia's publication *Design of tilt-up concrete wall panels. (Z10)*

The P-Δ moment results from the panel deflection and combines with the effect of the out-of-plane wind load and eccentricity of the vertical load. The connection of the rafter to the panels should be designed to keep the load eccentricity to a minimum (see Rafter to panel connection). The combination of the three moments will generally give the reinforcement required. However, impact loads and construction tolerances may also need to be considered.

Figure 20: Moments on panels

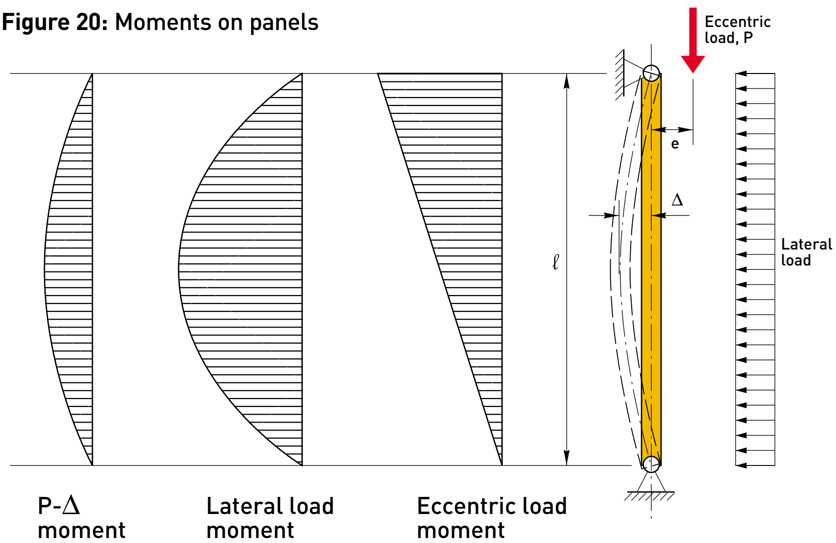
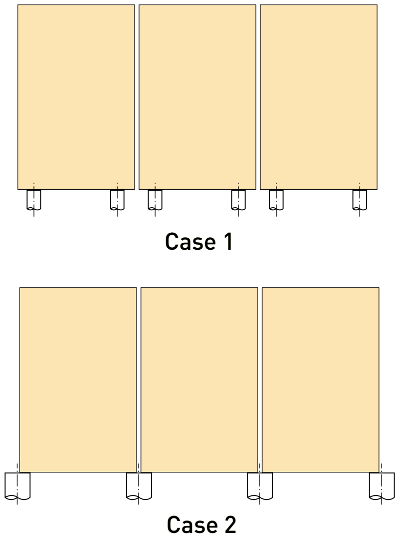


Figure 21: Support of panels on piers



Support of Panels on Piers

Number of Piers

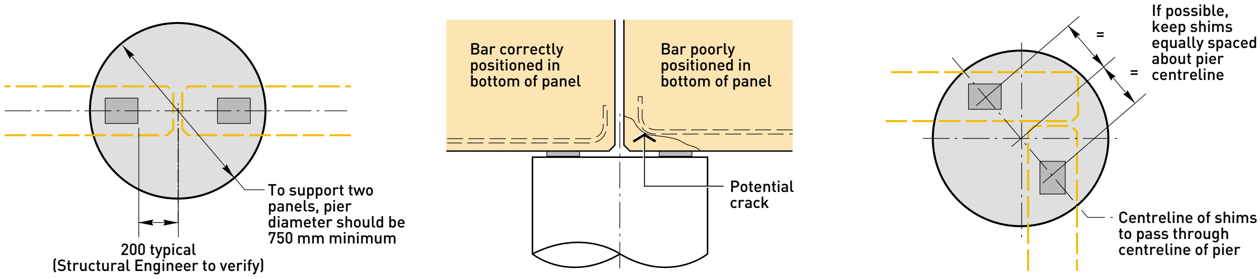
If panels are supported on piers, then there are two cases for the arrangement of the piers. Either two small piers are provided per panel (Case 1), or one larger pier is used to support two panels (Case 2), **Figure 21**.

When evaluating which alternative to use, the following points should be considered:

- Case 1 requires more drilling than Case 2, although the Case-1 pier sizes may be substantially smaller than those of the Case-2 piers.
- Cracking of the bottom corners of panels is less likely with Case 1 because piers can be located away from corners of panels.
- If skin friction on piers is significant then Case 1 can require less concrete.
- If piers hit obstructions while drilling, then for Case 1 they can generally be moved sideways because there is more tolerance in pier locations.

Figure 22: Larger piers supporting two panels

(a): Minimum pier diameter | **(b):** Cracking of panel corner | **(c):** Corner piers



If piers are shallow, the advantages of using two smaller piers would generally make this option the preferred one.

Regarding Case 2, some of the issues to consider when supporting two panels on one larger pier are:

- Minimum diameter of piers should be at least 750 mm in order to allow for the correct edge distances to shims as nominated in the Australian Standard relating to tilt-up construction, AS 3850, **Figure 22 (a)**.
- The potential for cracking of the corner if the bottom reinforcement is poorly placed, **Figure 22 (b)**.
- At a corner, **Figure 22 (c)**, the shims which transfer the loads onto a single pier should be equally spaced about the centerline of the pier so that adverse moments are not introduced into the pier. Note that depending on the pier diameter and requirements for shims, it may not be possible to avoid a moment in the pier, and the pier must be designed accordingly.

Piers at Boundaries

At boundaries, the piers supporting panels will require pier caps because the shims usually can not be satisfactorily located on the piers, **Figure 23 (a)**. If the adjoining building has already been constructed, the proximity of the piers to the boundary (especially for smaller piers) may be determined by the dimensions of the drilling mandrel, **Figure 23 (b)**.

Typically, a pier with a minimum diameter of 750 mm is required along boundaries to achieve the necessary edge distances.

In cases where the pier can not carry the moment induced by the eccentricity and panel load at boundaries, a rectification beam may be required.

Provision of Pier or Pile Caps

If pier/pile caps are required, consideration should be given to the minimum dimensions required to allow for construction tolerances and minimum edge distances required by AS 3850. It doesn't cost much more to make pier caps a 'comfortable' size, **Figure 24**.

Figure 23 : Piers supporting panels at boundaries

(a): Pier location at boundary | **(b):** Drilling machine dimensions

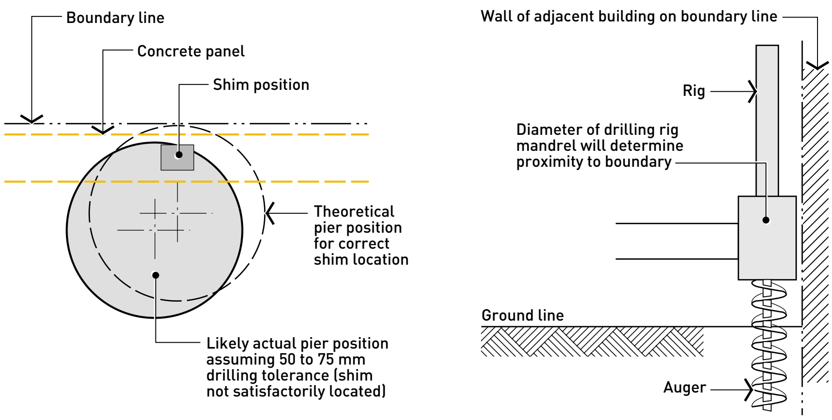


Figure 24:
Minimum size of pier caps

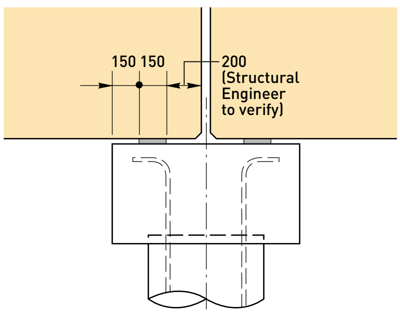
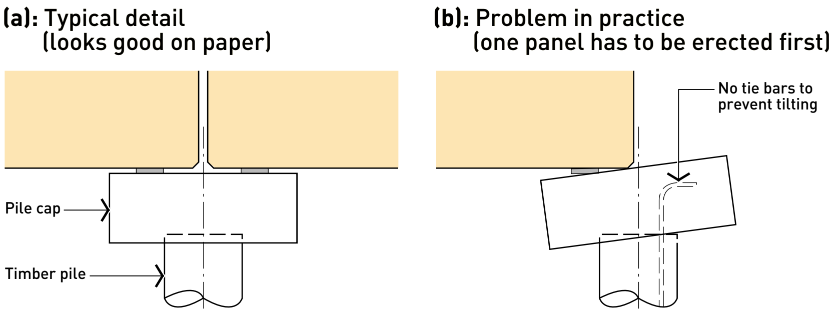


Figure 25: Use of timber piles with pile caps



The starter bars, which connect the pier or pile and the capping member, do their work (resisting eccentric loads) while the panels are being erected. Note that with the use of timber piles, no starter bars are provided. With both panels resting on the pile cap, the load on the pile is balanced and there are no problems. However, if one panel has to be erected first, it will cause an eccentric load and possible movement of the pile cap, **Figure 25.**

One solution for this is to extend the pile cap to encase the top of the timber pile in order to allow some moment transfer during erection of the first panel, **Figure 26**.

Note: With the use of screw-in steel piles, consideration of the unbalanced condition that occurs during panel erection, and possible permanent eccentricity at boundary locations, requires special attention.

Regarding the tops of the piers/piles, caution should be exercised in reading the layout plans so that the pier/pile cut off levels are not confused with the pier/pile cap levels, **Figure 27**.

Footing Layout

It is good practice for the Structural Engineer to provide a dimensioned footing layout drawing to reduce the risk of piers or piles being incorrectly installed. The completed layout should be checked prior to erection being started. With the cost of a crane and crew on site during erection of the panels, discovering a wrongly located pier or pile during erection is not only costly, but will considerably delay the project.

Ground Floor Slab Plan

Frequently, the ground floor slab is used as a casting bed for the wall panels. The slab plan, **Figure 28**, should indicate not only the joint types and locations, but also the construction details adjacent to the

Figure 26:
Extended pile cap for timber piles

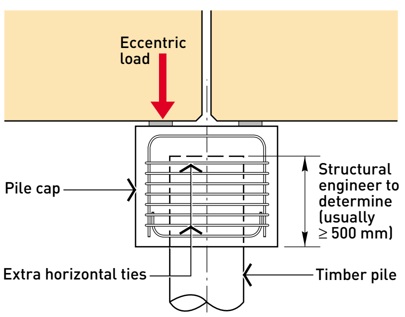


Figure 27: Pier/pile and cap levels

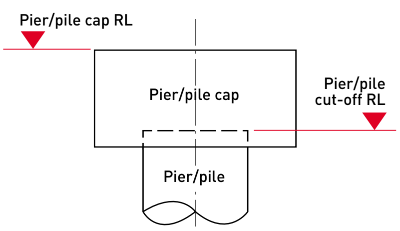
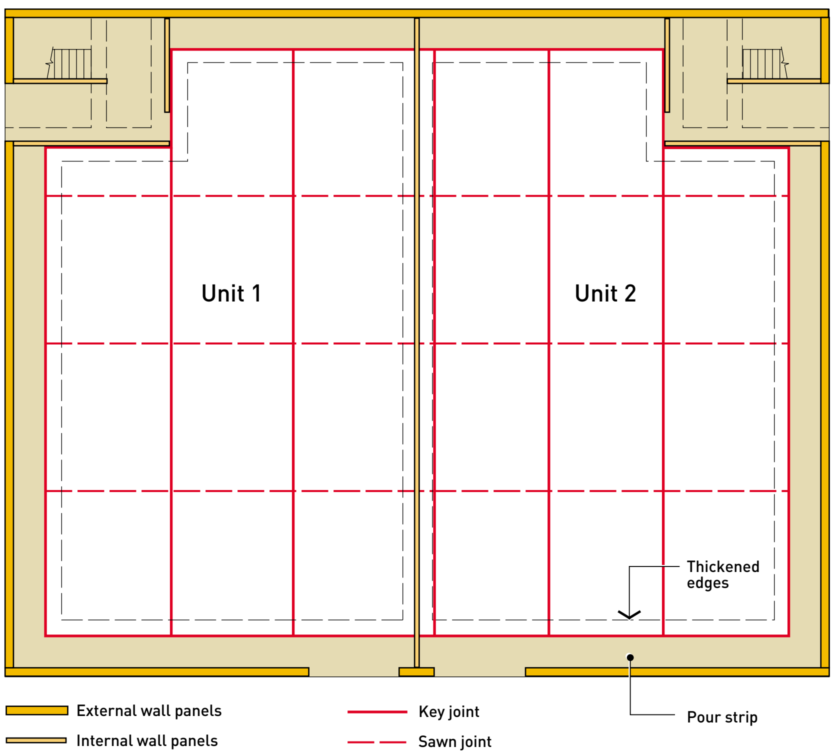


Figure 28: Ground floor slab plan



panels to allow the completion of the floor slab once the panels have been erected, **Figure 29 (a).**

The pour strip as indicated in **Figure 29 (a)** gives the panel some additional stability and is a means of building some redundancy into the structure (ie in case one of the bracing members fails). However, panels have to comply with the requirements of the Building Code of Australia (BCA), relating to the performance of external wall panels in fire.

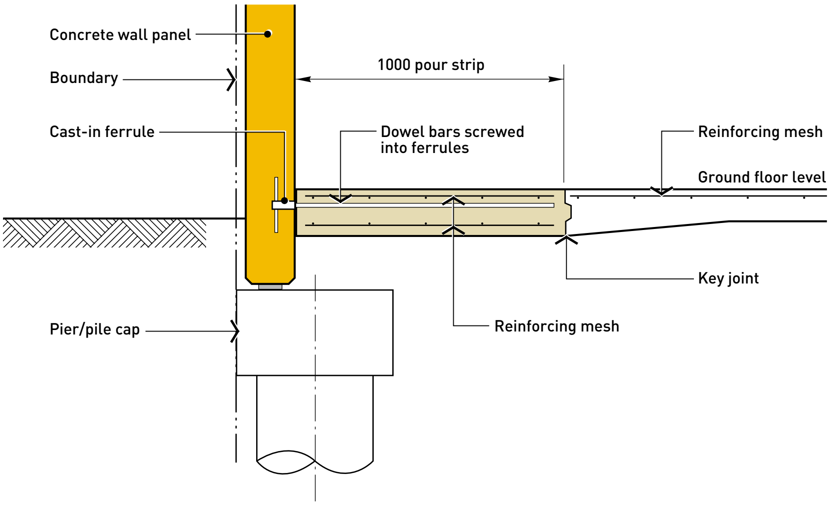
For panel buildings relying on the roof structural steelwork for support, the BCA requires that the top connections are strong enough to draw the panels inward should the supporting steel structure fail in a fire. To comply with this provision, the BCA requires that the connections at the top of the panel must be able to resist an ultimate load of two times the ultimate bending moment capacity of the panel at its base. Considering the typical height of panels, the loads on the top connections to satisfy this requirement are generally not large, even if a tied connection having some moment capacity is provided at the base.

If a pour strip is not incorporated into the slab plan, **Figure 29 (b)**, the layout of floor joints should ensure that they coincide with the panel joints at the perimeter. This prevents locking of the panels at their base and allows free shrinkage and thermal movement of the panels and floor.

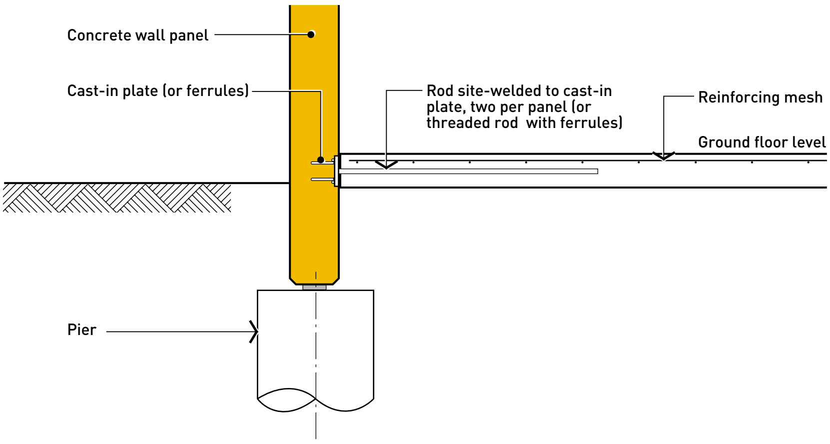
Figure 29 (c) indicates the support of internal wall panels supported on the slab.

Figure 29: Typical ground floor slab details

(a): External wall panel, supported on footing, with pour strip



(b): External wall panel, supported on footing, with no pour strip



(c): Internal wall panel, supported on floor slab

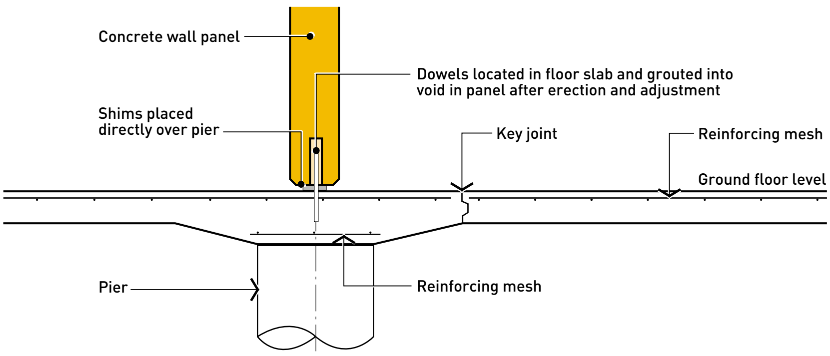
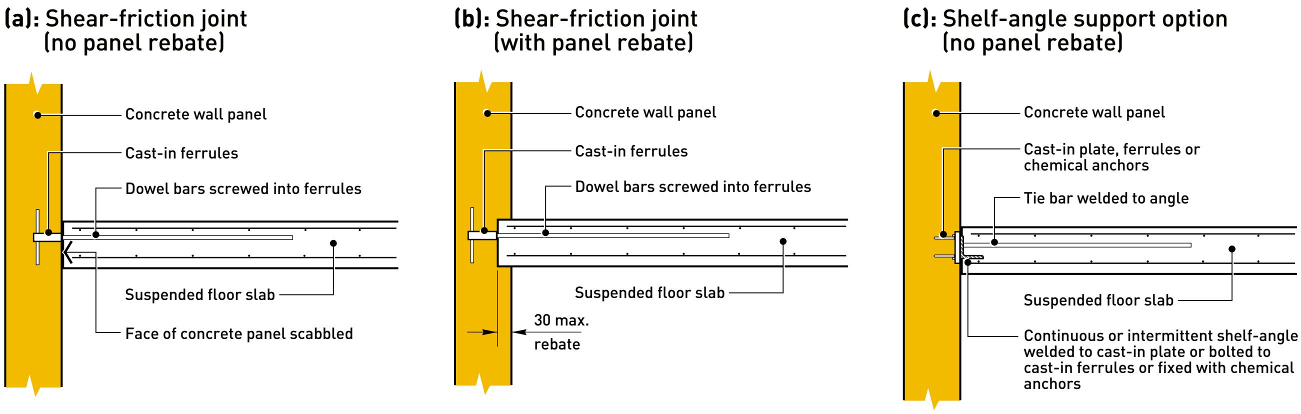


Figure 30: Typical connection of suspended floors



Connecting Suspended Slab to Panels

Typical details for connecting suspended slabs to panels are shown in **Figures 30** and **31**. The design of the connection is normally based on shear-friction theory, which is covered in the ACI code (ACI-318). While no rebate is theoretically required with this design approach, **Figure 30 (a),** many designers prefer to provide a rebate, **Figure 30 (b)**.

The depth of the rebate should be kept to a minimum to reduce the stresses on the panels during lifting, transport and erection. A 17-30 mm rebate and cast-in ferrules with screw-in bars are usually provided.

An alternative to the shear-friction connection, involves directly supporting the slab on, say, a shelf angle, **Figure 30 (c)**. This can be fixed to the panel using cast-in ferrules, chemical anchors or a weld plate and need not be a continuous member.

Shop Detailing of Panels

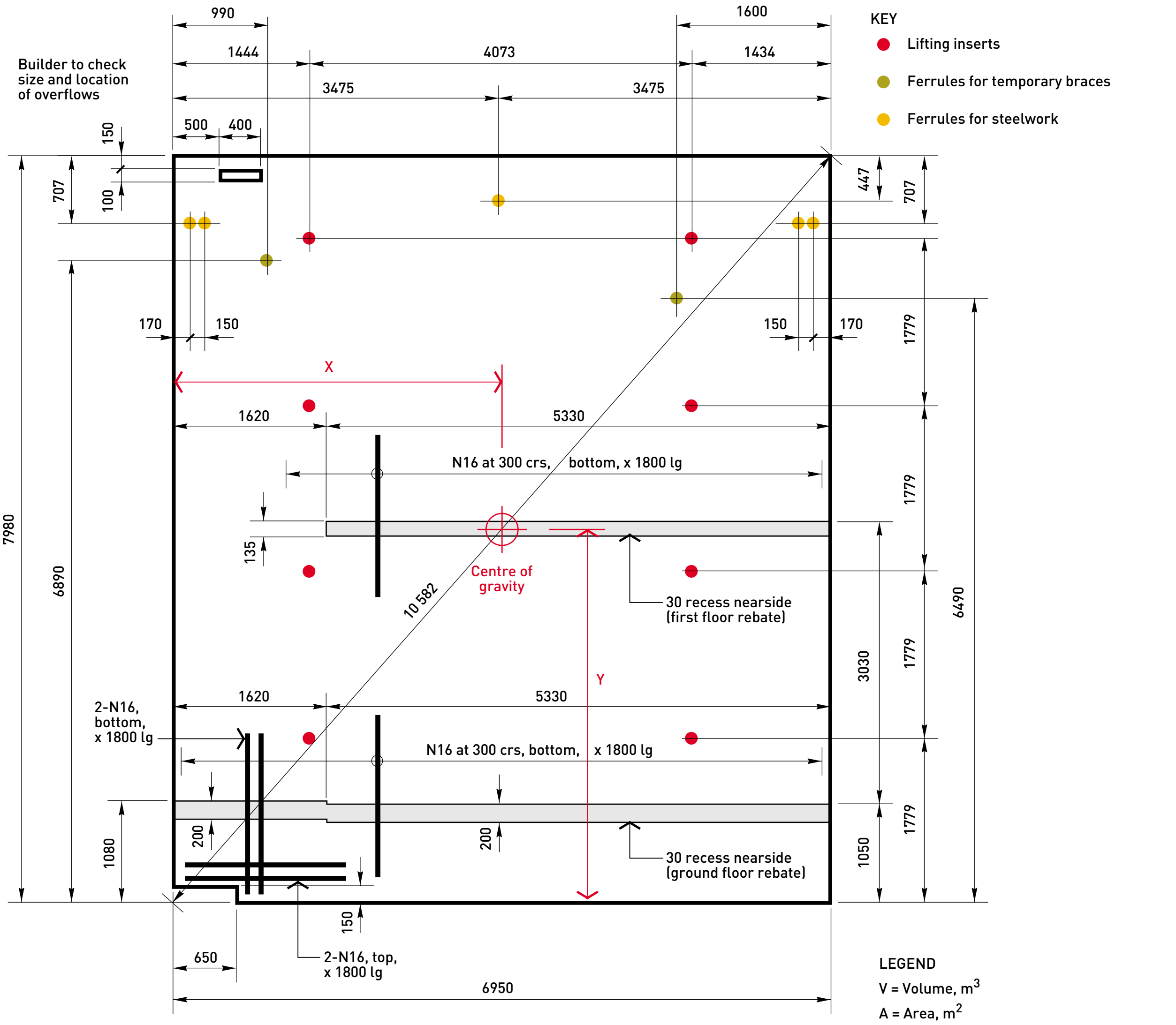
The shop details for panels are usually provided by the panel contractor's Engineer (the Panel Engineer), who is responsible for designing the panels for erection and temporary propping loads. Copies are sent to the Structural Engineer (similar to other shop details) to ensure they reflect the design intent for the completed structure. They should incorporate all the dimensions (including diagonal measurements) and details required for the manufacture of the panel including:

- The location and size of ferrules;
- The location and capacity of lifting anchors;
- Any inserts such as grouting ducts, steel plates;
- Reinforcement details;
- Edge and face rebates;
- Concrete strength at 28 days and time of lifting.

A typical shop detail of a panel is shown in **Figure 32**.

Figure 31: Rebate and cast-in terrules for connection of suspended slabs

Figure 32: Typical shop drawing of panel details



V = 8.26	**PANEL P2**			150 thick
A = 55.36	W = 19 814	P = 29.86	X = 3.480	Y = 4.007

LEGEND
V = Volume, m^3
A = Area, m^2
W = Weight, kg
P = Formwork perimeter, m
X = Distance left side to centre of gravity, m
Y = Distance bottom to centre of gravity, m

Note: Typical items such as reinforcement details, insert capacities, ferrule sizes and concrete details may be covered on other drawings or in the documentation.

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Further Information

While this Briefing Note gives guidance on some specific items that need to be considered, further information can be found on the Cement and Concrete Association of Australia's web site at www.concrete.net.au, and in the following documents.

1 American Concrete Institute Committee 318, *Building Code Requirements for Structural Concrete (318M-02) and Commentary (318RM-02)*, Clause 11.7 - Shear-friction

2 American Concrete Institute, *ACI Manual of Concrete Practice 2002*, 445R-35, Chapter 5 - Shear Friction

3 AS 3600 *Concrete structures* Sandards Australia, 2001

4 AS 3850 *Tilt-up concrete and precast concrete elements for use in buildings* Standards Australia, 1990 Currently under revision

5 Concrete Institute of Australia *Design of Tilt-up Concrete Wall Panels* (Z10), 1992

6 National Precast Concrete Association Australia *Precast Concrete Handbook* (Z48), 2002

7 Ian Hymas, *Design and Erection of Concrete Wall Panels*, The Cement and Concrete Association of Australia, Winning Solutions with Concrete (Building with Concrete Panels) Seminar Preprints, 2000.

8 Ian Hymas, *Fire and Tilt-up*, The Cement and Concrete Association of Australia, Winning Solutions with Concrete (Building with Concrete Panels) Seminar Preprints, 2000.

9 Geoffrey Henry, *Economic Aspects of Tilt-up*, The Cement and Concrete Association of Australia, Winning Solutions with Concrete (Building with Concrete Panels) Seminar Preprints, 2000.

10 Garry Truswell, *Design Methods for a Typical Tilt-up Warehouse*, The Association of Consulting Structural Engineers of NSW, Modern Structural Engineering Seminar Preprints, 15 August 1984.

11 Barry Crisp, *Engineering Design for Tilt-up*, The Cement and Concrete Association of Australia, Tilt-up Design and Construction Seminar Preprints, 1994.

12 The Cement and Concrete Association of Australia *Tilt-up construction notes* (T50), 1997

13 The Cement and Concrete Association of Australia The *Concrete Panel Homes handbook* (T54), 2001

14 *Building Code of Australia*

15 Victoria Workcover Authority, Industry Standard, *Precast and Tilt-up Concrete for Buildings*, 2001

Notes

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Briefing

C&CAA OFFICES

SYDNEY OFFICE:
Level 6, 504 Pacific Highway
St Leonards NSW Australia 2065
POSTAL ADDRESS:
Locked Bag 2010
St Leonards NSW 1590
TELEPHONE: (61 2) 9437 9711
FACSIMILE: (61 2) 9437 9470

BRISBANE OFFICE:
Level 14, IBM Building
348 Edward Street
Brisbane QLD 4000
TELEPHONE: (61 7) 3831 3288
FACSIMILE: (61 7) 3839 6005

MELBOURNE OFFICE:
2nd Floor, 1 Hobson Street
South Yarra VIC 3141
TELEPHONE: (61 3) 9825 0200
FACSIMILE: (61 3) 9825 0222

PERTH OFFICE:
45 Ventnor Avenue
West Perth WA 6005
TELEPHONE: (61 8) 9389 4452
FACSIMILE: (61 8) 9389 4451

ADELAIDE OFFICE:
Greenhill Executive Suites
213 Greenhill Road
Eastwood SA 5063
POSTAL ADDRESS:
PO Box 229
Fullarton SA 5063
TELEPHONE: (61 8) 8274 3758
FACSIMILE: (61 8) 8373 7210

WEBSITE: www.concrete.net.au

EDITOR: Eric Lume
EMAIL: info@ccaa.com.au

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