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RESIDENTIAL PORTAL FRAMES
AN ENGINEER'S PERSPECTIVE

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INTRODUCTION

Portal frames for residential work are an emerging issue for many engineers, who have realised that different methods can produce very different results. The approach in this document has been checked by senior structural engineers from the Engineering General Practitioners Group as well as engineers from several BCAs and judged to be a simplified example of good practice design.

For more complex designs that are outside the scope of this document, for instance, in gable ends where there is no direct connection between the frame and the ceiling diaphragm, we recommend you design per SR337 (Liu, 2015) and the example in HERA DCB No. 75 (Clifton, 2003). A common mistake is to 'fit' a steel portal frame within an external wall, disrupting the integrity of the wall framing under face loading. This often occurs in end walls, where ranch-sliders, folding doors and large windows do not extend to the ceiling diaphragm.

Extra care should also be taken in the following circumstances:

- Wind face loading – Face loading on the portal rafter needs to be checked, and additional stiffening and strengthening may be required to meet SLS and ULS criteria (particularly for skillion ceilings).
- Lateral restraint – Portal frame restraint can become an issue, especially with skillion ceilings. A good option is to run the portal columns up to the roof and provide restraint there. You will need to carefully detail connections for restraint and consider factors such as geometry and load paths.

This document has been prepared for educational and illustrative purposes only and should not be relied on for any other purpose. Always exercise your own professional engineering skill, knowledge and judgement when undertaking engineering activities.

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OVERVIEW

Most portal frames currently designed for residential construction are steel, and, unlike frames in commercial buildings, are frequently comprised of parallel flange channel (PFC) sections and are torsionally sensitive. It is almost impossible to provide full fixity to the base of a residential portal, therefore modelling a pin base portal is advisable.

The example below aims to provide a simple portal frame design that complies with the intent of SR337 (Liu, 2015) without going into calculations for damping or torsion. A damping reduction factor of 0.7 has been applied to the load during the demand calculations to account for damping. Caution must be taken when applying this reduction factor. Where (for example) you have more than one steel bracing system, you should design it as a steel structure with a maximum ductility $\mu = 1.25$, of $S\rho = 0.93$.

Lateral loads of 200BU seismic and 120BU wind are adopted. We are not calculating vertical loads for this example. However, you should take them into account as different load cases (such as wind) may dominate rather than seismic.

The portal frame in this example will have a clear length of 4,000mm and a clear height of 2,000mm. We make allowances for framing timber and ½ the overall depth (d) of the steel member as per Figure 1 .

This portal has direct connection to the ceiling diaphragm. The shear centre of a PFC is offset from the centreline of both the section and the web. It is assumed in this example that the connections between the ceiling members and the portal beam are sufficiently strong and rigid to ensure that there is no torsion on the beam. If your portal does not meet these criteria, then you must consider that potentially your beam has no buckling restraint. This can be resolved by extending the portal to the ceiling and detailing timber framing under it.

It is also assumed that gravity and vertical wind loads are negligible, and second order effects have been checked and are sufficiently small they may be ignored.

Fully elastic earthquake response is assumed.

Lateral load derivation

You can derive loads in several ways, either from specific calculation or from a bracing spreadsheet such as those supplied by manufacturers of bracing systems. If you are using a spreadsheet, then it is good practice to check the numbers and apply engineering judgement as to whether you should increase them. For this exercise, we will assume that loads are from a bracing spreadsheet, are judged to be correct and have a μ of 3.5. Convert loads to kilonewtons and change ductility.

Wind demand $= \frac{120BU}{20} = 6kN_{ULS}$ (as per BRANZ P21 test)

Seismic demand $= \frac{200BU}{20} = 10kN_{ULS}$ for $\mu = 3.5$

Convert load demand based on changing ductility as per SR337

1. $k_{\mu} = 2.43$, $S\rho = 0.7$ for $\mu = 3.5$ as per Engineering Basis of NZS3604
2. $10kN * \frac{2.43}{0.7} = 34.7kN_{ULS}$ now the damping reduction factor can be applied
3. $34.7kN * 0.7 =$ ***24.3kN***

Calculate serviceability loads

Wind (Wellington location) $= 6kN * (43m/s)^2/(51m/s)^2 = \boldsymbol{4.26kN}$

Seismic $= 24.3kN * 0.7(S\rho) * 0.25$ (Return factor) $= \boldsymbol{4.23kN}$

Wind dominates

PORTAL FRAME SIZING AND CAPACITY

The portal should be as efficient as possible. You could calculate deflection demand and then rearrange the stiffness equation to find an appropriate size. In the following method we will try a section and see how capacity compares with demand. The process would be iterated to find an efficient design, however, we do not continue the iterations in this example.

Try using 250PFC, where the clear opening of the portal is 4,000mm. Then add 45mm of packing timber each side, 125mm for ½ depth of the member each side, and the same for the height.

$$L_{cl} = 4{,}000mm + 45mm * 2 + 125mm * 2 = 4{,}340mm$$

$$H_{cl} = 2{,}000mm + 45mm + 125mm = 2{,}170mm$$

Calculating demand

ULS $\delta_{max} = \frac{height}{100} = 22mm$ – As recommended by Liu, SR337

SLS $\delta_{max} = \frac{height}{300} = 7.2mm$ – Taking deflection at mid-height, as per table C1, AS/NZS1170.0-2002

Keep in mind that SLS deflections should also consider in-plane glazing clearances. Usually, this is two times the glass clearance of approximately 3–6mm. Glazing requirements can limit deflections in line with glazed openings to 6–12mm.

$M^* = H_{cl} * \frac{P^*}{2}$ + additional bending moment from vertical actions

$M^* = 2.17m * \frac{24.3kN}{2} = 26.4kNm$ at each knee

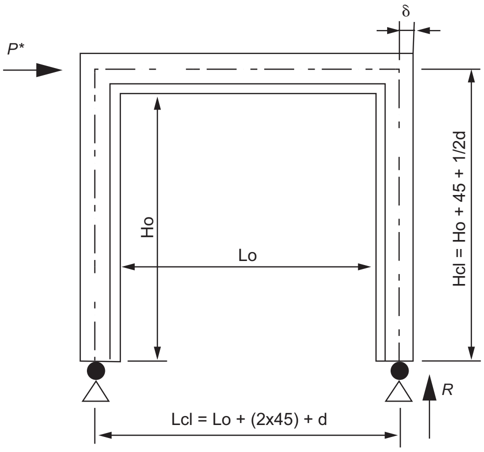
$V^* = \frac{P^*}{2}$ at each leg

$V^* = \frac{24.3kN}{2} = 12.2kN$ at each leg

$$N^*_t = N^*_c = P^* * \frac{H_{cl}}{L_{cl}}$$

$$N^*_t = 24.3kN * \frac{2.17m}{4.34m} = 12.15kN$$

Figure 1



Because by inspection deflections are likely to govern, these centreline actions will be used instead of face moments.

Calculating capacity

Top plate fixings

The load must be able to get into the portal frame. We can transfer 58.2kN (calculated to NZS3603:1993) using ten M12 bolts with square washers at 400crs. Any higher loads should be accompanied by specific detailing for load transfer through the diaphragm.

Check stiffness

Where you specify the same section for the portal columns and beam, then $\delta = \frac{PH^2(2H+L)}{12EI}$

H = frame height	H = 2,170mm
L = frame span, centre to centre	L = 4,340mm
$I_{BM} = I_{COL} = I$	$I = 45{,}100{,}000\text{mm}^4$
E = modulus of elasticity	$E = 205{,}000\ \text{N/mm}^2$
P = lateral load	$P_{ULS} = 24{,}300$ N
	$P_{SLS} = 4{,}300$ N
$\delta_{ULS} = 9\text{mm}$	**9mm ≤ 22mm – OK**
$\delta_{SLS} = 1.6\text{mm}$	**1.6mm ≤ 7.2mm – OK**

Check strength

In this case, we only look at the beam because of the disparity in length between column and beam. Where they are of a similar length, you should also check the column. The beam top flange of this portal has full lateral restraint provided by the roof/ceiling diaphragm, but the designer also needs to consider the unrestrained bottom flange. The designer should check the segment capacity based on the actual restraints available. Take care when your portal frame is parallel to the roof framing, as additional blocking may be required to transfer loads to the diaphragm. For this example, a quick conservative check is to take the full rafter segment, as the rafter length is greater than the column length.

$\phi M_b = \phi M_{sx} . \alpha_m . \alpha_s$ *where* $\alpha_m . \alpha_s \leq 1$ $\phi M_{sx} = 114kNm\ f$ or *250PFC*

$L_e = 4m$

$\alpha_m = 1.00$ (taken conservatively) $\alpha_s = 0.535$

$\phi \boldsymbol{M_{bs}}$ **= 61*kNm*** (Table 5.3.2 AISC Design Capacity tables for structural steel)

26.4kNm ≤ 61kNm – OK

Above is a simplified example of the member bending capacity derived from steel design tables. You can make more accurate calculations by using a program such as SESOC MemDes.

CONNECTIONS

Portal knee

Restraint at the knee

This document assumes lateral restraint at the knee. You can achieve adequate restraint by direct attachment to the ceiling diaphragm or by using a member such as a double stud, for which you should provide calculations, or by running the portal leg full height. In this example, the portal connects to the diaphragm.

The knee joint must be able to develop either:

(a) The overstrength of the beam and column per NZS 3404 if $\mu > 1$ or

(b) The maximum actions from the members, including all load components, for a fully elastic response, including $\mu = 1$ seismic actions (generally the bending moments will govern). However, unless deflection control is very dominant, it is still recommended that the full strength of the members be developed in this case.

For (a), this means that full strength butt welds are required at the interface of the beam and column.

For (b), this means that either full strength butt welds or larger than minimum size fillet welds will be required, subject to specific design.

There usually are two basic acceptable ways to detail the knee joint for strength alone – using rectilinear stiffeners, or an appropriately designed diagonal stiffener.

However, a diagonal stiffener does not provide lateral restraint to the inside corner of the joint. In a conventional portal frame building, this does not matter, because a diagonal fly brace can be attached to the inside corner of the joint, but that is not appropriate within a house. Therefore, you should not use diagonal stiffeners on their own. We recommend rectilinear stiffened joints.

For rectilinear stiffeners of the type shown in Figure 2, the top flange of the beam and the ceiling diaphragm laterally restrains the top of the column. The flanges/stiffeners will ensure that the intersection point of the beam bottom flange and the columns inside flange is also laterally restrained, which significantly improves the performance of these members, and simplifies the design process.

Simple knee capacity check

Residential portal frames are rarely strength governed, and problems with web failure are minimal. For the rectilinear flange/stiffener joint arrangement shown Figure 3, there must be a shear force across the joint to develop the moment in the beam and column.

If $\mu > 1$, consider overstrength actions when assessing the shear forces acting on the joint.

In this example, with an elastic response, they can be derived from the actual bending moments.

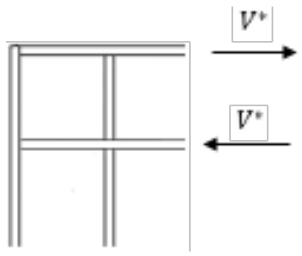
For this example:v

$V^* = N^*_t = N^*_c = 26.4kNm/(0.25m - 0.015m) = 112.3kN$

$ØV = 0.9 * 0.6 * 320\ N/mm^2 * 235mm * 8mm = 304.1kN$ (Cl 5.11.4.1, NZS3404:1997)

112.3kN < 304.1kN **Works OK**

Figure 2

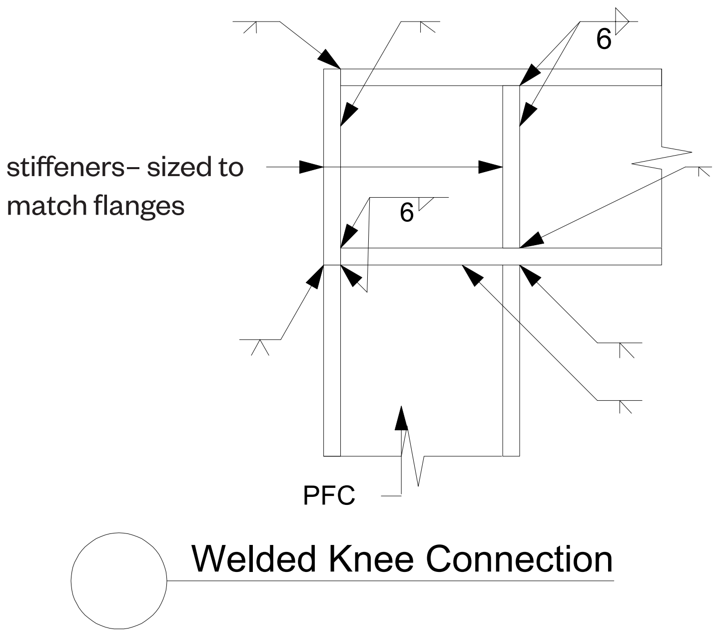


As is normal with residential portals, deflection governs, and therefore web capacity is not critical.

Detailing the knee

There are several ways to detail the knee, and this issue can be contentious. Ensure the full load can be carried around the knee joint.

Figure 3: Example knee detail



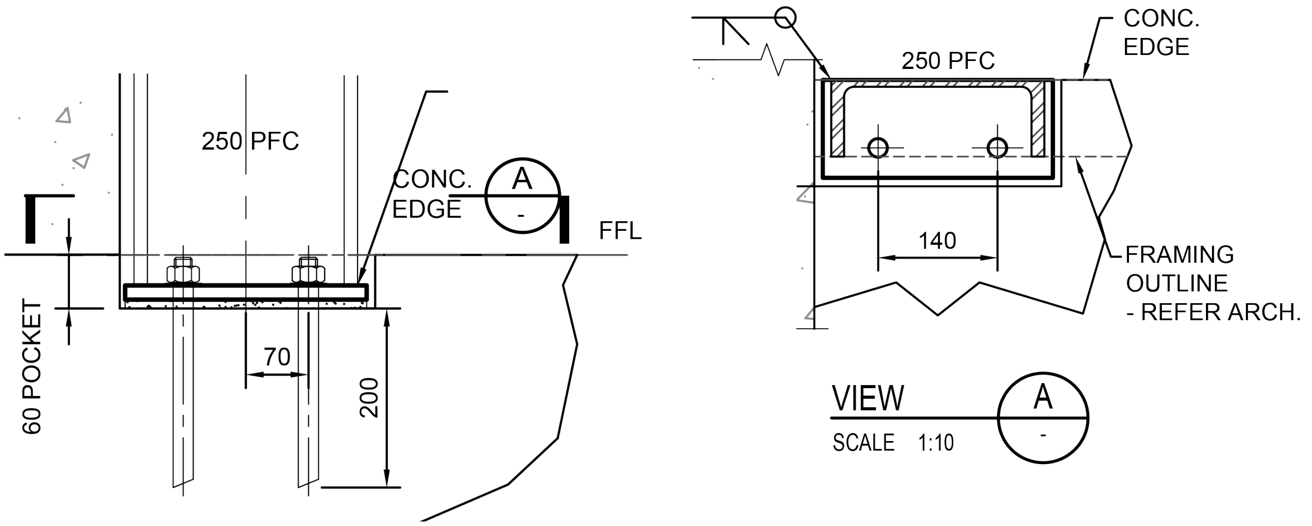
Base plate to foundations

The load must be able to be transferred to the ground. With residential portals, it is usually uplift that is the problem, the foundations must be designed to ensure they can resist the reactions from the portal frame.

Typical connections are chemical anchors and cast-in-place bolts. Both options have potential pitfalls, such as incorrectly placed cast in bolts or improperly installed chemical anchors. This document does not go into the details of calculating out capacities, as it changes by the supplier. There are, however, several factors to consider, including:

- Base plate ductility – the base plate must be stiff enough to yield at ULS but not before. We recommend reading through SESOC's *Anchor Bolts For Steel Structures – Draft Design Guide*[1]. Another useful resource is the ASI Baseplate connection guide – available through HERA.
- Additional bending can occur in anchors when they are not embedded. Embedding the portal feet into the slab can resolve many issues.
- Edge distance – the hold-downs must have sufficient edge distance. Edge distance requirements need to work with the specified cover and allow for clearing the longitudinal reinforcing bar. Epoxied bolts should not rely on cover concrete; you should specify embedment in the reinforcing cage. Obtaining sufficient edge distance can be challenging to achieve within a 90mm wall, especially with a 250PFC. Talking to the architect about using a 140 or 190mm wall can be beneficial.
- Resisting uplift – there must be sufficient mass available to anchor the portal. Assuming the portal must resist the full uplift of 12.15kN, 0.5m^3 of concrete is required. Ensure there is adequate bending capacity in the foundation beam.

Figure 4: Example embedded base



Site welding and assembly of the portal

Avoid having the portal assembled on-site whenever possible. However, there are times when it is necessary. There are different approaches to making it work. We recommend avoiding site welding on the knee; it is preferable to design a bolted welded beam splice connection at the points of contra-flexure on the beam. Be aware of the potential for fire when welding close to timber.

[1] Anchor Bolts For Steel Structures – Draft Design Guide, by John Scarry, downloadable from www.sesoc.org.nz/library/guidelines/anchor-bolts-for-steel-structures-draft-design-guide

Potential site issues

Figure 5: Portal toes to outside, insufficient edge distance

Figure 6: Portal toes to outside, nuts undone

Figure 7: Portal toes to outside, edge distance, studs bent

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