17-19 Gladding Place
P O Box 76-134
Manukau City,
New Zealand

Phone: +64-9-262 2885
Fax: +64-9-262 2856
Email: structural@hera.org.nz
Website: www.hera.org.nz

No. 73 **April/May 2003**

The author(s) of each item in this publication are noted at the beginning of the article.

The material herein has been the subject of review by a number of people. The effort and input of these reviewers is greatly appreciated.

Introduction

The introduction to this issue covers a number of followup matters relating to articles and papers in previous issues. Readers' input and suggestions for research topics for the HERA Structural Division is also sought.

Following this is a design example, on the design of a telescopic boom. Such booms are commonly used in portable lifting platforms such as "cherry pickers". Their simple appearance and ease of operation belie the complexities involved in their design, which invokes most of the general bare steel design provisions of NZS 3404, as well as the delights of combined biaxial bending plus torsion plus axial compression! This makes such a design example perfect for the *DCB*!

This is followed by three short papers or articles relating to topics of current interest.

The issue, as usual, concludes with the references.

Progress on Errata to AS/NZS 2312:2002

As advised in the paper on the new corrosion protection Standard [1] presented on pages 10-15 of *DCB* No. 72, there are a number of minor errors and omissions in this new Standard. These are currently being rectified by the committee. It is intended that the changes be published by July 2003.

Column Splice Cost Comparison Revisited

Page 15 of *DCB* No. 72 presented a short article on a column splice cost comparison between two column options for the top 4 storeys of a multi-storey building. The choices were between introducing a non-standard splice to step the column size down from a 310UC97 to a 250UC73, or using a standard bolted splice from HERA Report R4-100 [2] for the 310UC97 and maintaining the larger column size over the top 4 storeys.

The result showed that carrying the larger column size over the full height was the more economical outcome.

However, a fabricator reader makes the following comments, which are very relevant and may change the overall outcome in specific circumstances:

(1) The larger section size has an increased surface area / metre length of around 20%. If a required surface coating to the column surface for either corrosion protection or fire protection is sufficiently expensive, the extra 20% surface area may alter the economics of the outcome. Given that most multi-storey columns will carry some form of coating for fire protection or corrosion protection purposes, this should have been included.

In This Issue

Charles Clifton's note: This was certainly an omission and should have been included in any comparison made. For spray applied fire protection or for single coat paint systems, which have an applied cost/m^2 of under \$50/m^2, [3], the difference would be unlikely to change the conclusion that maintaining the heavier column section size over the top 4 levels is the more cost-effective outcome. However, for solid board or intumescent paint fire protection systems, which have applied costs/m^2 ranging from \$50/m^2 to over \$200/m^2, the cost saving in coating on the smaller column might well change the outcome.

(2) The extra weight and size of the larger column may slightly increase transport costs, by decreasing the number of lengths of column that can be put on a truck. Thus for every three full loads of columns another trip will be needed to transport the extra weight. This isn't taken account of in the comparison. This would slightly reduce the margin in favour of maintaining the heavier column size given in the *DCB* No. 72 comparison.

This reader's key point was that the most cost-effective option from that comparison is dependent on the input assumptions. While that outcome would be typical, it will not always apply and engineers should not consider that as the definitive answer for all future considerations of this type.

Sound advice…

Corrigenda to the Design of Circular Bolted Flange Annulus Connection

DCB No. 65 contains a design procedure for the design of *Circular Bolted Flange Annulus* connections. These are bolted connections between lengths of circular columns, where the two lengths have an annulus plate welded to their ends, with adjacent plates being bolted via a ring of high strength structural bolts.

A user of the procedure has raised two issues regarding the procedure. These are as follows:

Q1: The dimension m_2 is defined in equation 65.4.2 but not shown in that equation. Is the equation correct?

A1: Yes it is. The dimension m_2 is used in the determination of α, from Fig. 65.16. That variable is then used in equation 65.4.2.

Q2: In section 3.5.2, the equation reference for calculating N^*_{tw} is wrong.

A2: That is correct; N^*_{tw} is calculated from equation 65.3, not equation 65.13 as stated.

Would all *DCB* readers please make this change in their copy of *DCB* No. 65.

Congratulations to Linus Lim on Achieving his PhD.

Linus Lim, the University of Canterbury fire engineering student whose floor slab panel research has been fundamental to the development of the second edition of *the Slab Panel Method* (SPM) fire engineering design procedure, has successfully completed his PhD degree.

His work on membrane action in fire exposed concrete floor systems has been excellent and Charles Clifton congratulates him, personally and on behalf of HERA, on an excellent project well completed.

Linus now joins the Sydney office of a respected fire engineering consultant, where his talents and expertise will be well put to use.

Research Topics Sought

Over the last five years, HERA has received funding from the *Foundation for Research, Science and Technology (FRST)* for our research programme into enhanced steel building performance in high risk events.

This funding has enabled the development of major new design developments, such as:

- The *flange bolted joint* (*DCB* Nos. 58 and 62), for which standard connection details are currently being developed by Raed Zaki, HERA Assistant Structural Engineer
- The *sliding hinge joint* (*DCB* No. 68) for which standard connection details are also currently being developed
- The *slab panel method* (SPM) of floor system design for dependable inelastic response in severe fires.

Their current funding is for a programme of research work through to mid-2004. Despite submitting what the HERA Structural Engineer considers to be our best ever bid to FRST for the next six years built environment funding round, this bid has been declined. This means that all current government funding of HERA research ceases as of mid-2004. As a result, funding for the HERA Structural Division will decrease by over 50% at the end of the 2003/2004 financial year. This will require a complete re-evaluation of HERA's structural steel activities. We need to undertake the following actions:

- Determine which of the current range and scope of activities undertaken by the HERA Structural Division the industry would like to see maintained; then
- Develop funding streams to support these activities

This includes putting up a research programme for the next few years of topics that are a high priority to the industry. We will be seeking input from *DCB* readers on all this in the forthcoming year, however for now we are asking readers to come back with any research topics that they would like to see HERA undertake. Please send a brief description (not more than 100 words suggested) to Charles Clifton, HERA Structural Engineer:
Email: structural@hera.org.nz

Design Example 73.1: Design of a Telescopic Lifting Beam

This design example has been prepared by Murray Landon, Consulting Engineer from Tauranga. It is based on an actual job, but some details have been changed for confidentiality reasons and ease of presentation. Some editorial notes have been added by G Charles Clifton, HERA Structural Engineer.

Introduction and Scope

General details

Fig. 73.1 shows a typical example of a telescopic lifting boom. Despite their simplicity of appearance and operation, they are not the most straight forward item to design. The boom comprises a telescopic beam nested into a main beam. RHS members are used for both the main beam and the telescopic beam.

The sizes of these two members must be such as to allow nesting to occur, clearances for the plastic guides, hydraulic ram to extend and retract the telescopic beam etc. Selection of suitable pairs of RHS sizes for this can be made using the *Design Capacity Tables for Hollow Sections* [4], which covers sizing of nested sections.

Scope of design example

This design example covers:

- Design of the boom for the design actions shown in Fig. 73.2 (but ignoring the self-weight), when raised to an angle of 45^{o} and sitting on ground with a slope of 10^{o} to the horizontal
- The boom is in the fully extended position
- The design actions include a primary torque at the end of the boom of 2.25 kNm.

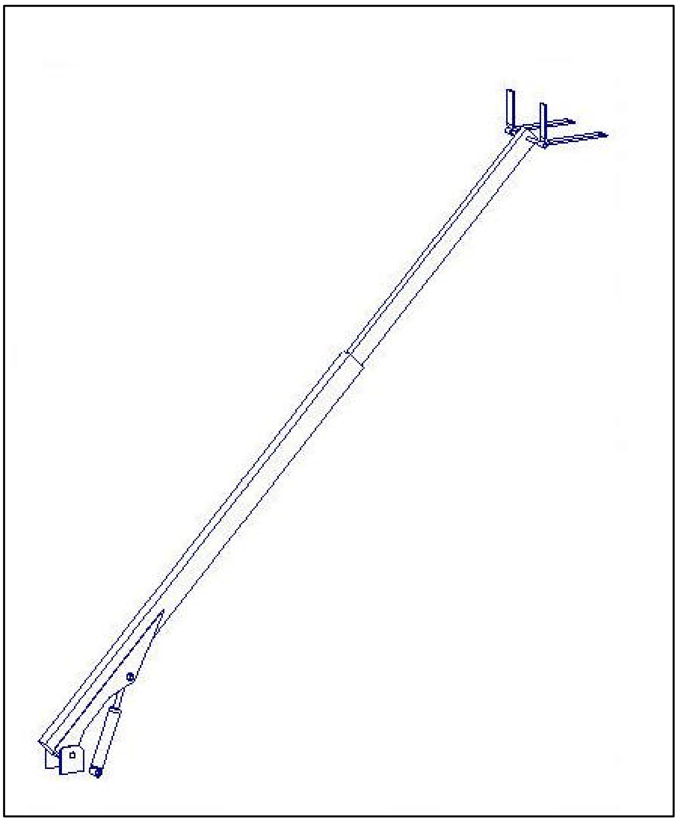


Fig. 73.1
Typical Example of a Telescopic Lifting Boom at Maximum Extension and Raised to 45^{o} from Vertical

The design does not cover:

- Design for other positions: the critical other position to check would be the fully extended boom raised just off the ground, giving maximum bending moment but no axial compression
- Design for concentrated load transfer at the supports and other points.

Design Method

Design is to NZS 3404 [5] using the *Alternative Design Method (Working Load Design)* of Appendix P.

The working load design method is used because the design loads have been evaluated using the working load provisions of AS 1418.1 [6].

Use is also made of *Formulas for Stress and Strain* [7] by Roark. Item references are given to the fifth and sixth editions, which covers the two editions most commonly used by designers.

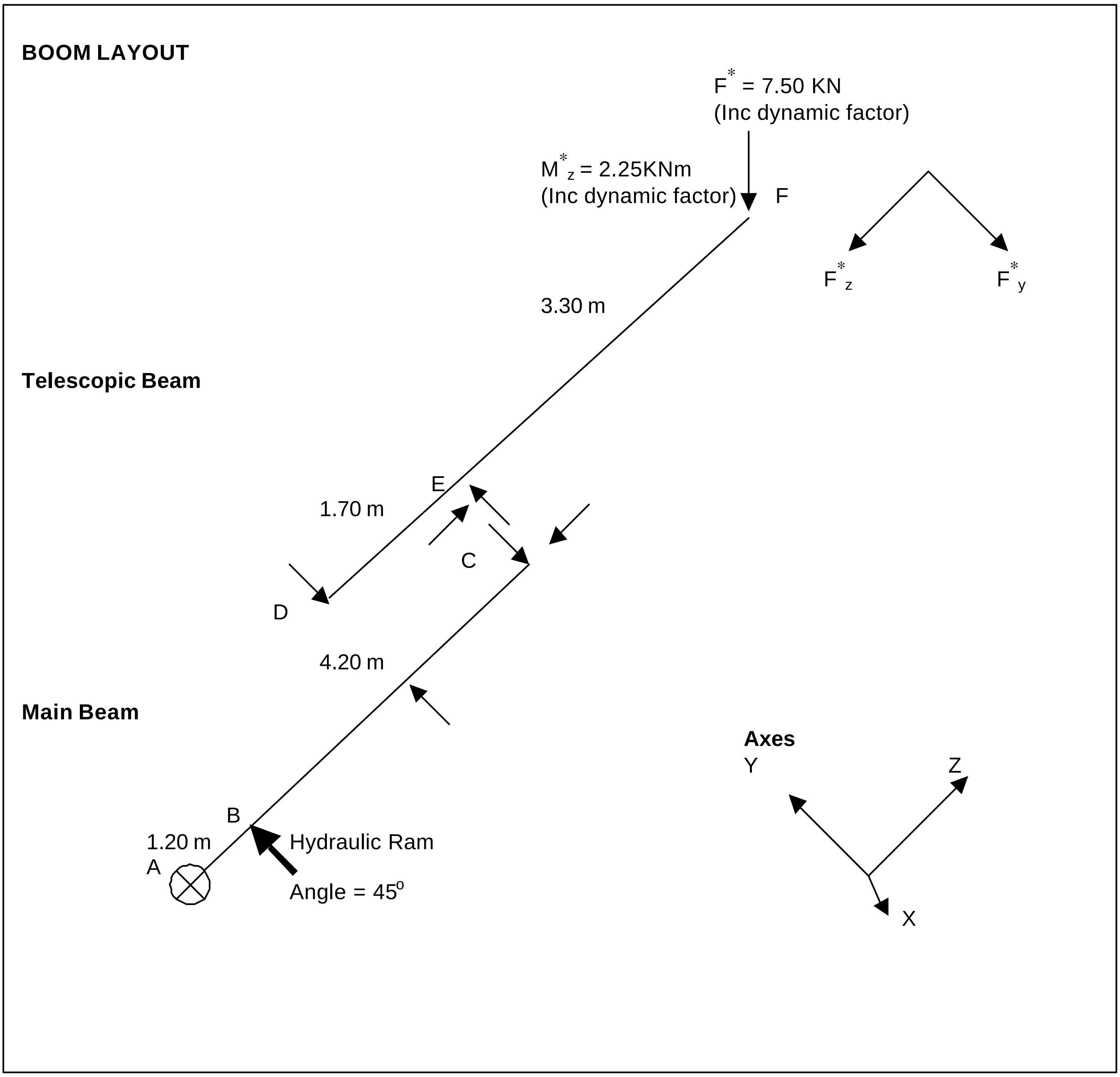


Fig. 73.2
Layout of Telescopic Boom, When Extended and Raised 45^o, Along with Design Actions and Positions for Design Checks

Boom Layout for Design Example

This is shown in Fig. 73.2.

The telescopic beam slides inside the main beam on plastic guides. These are provided on all four sides, to provide the necessary twist restraint.

The ram for the telescopic beam is joined midway between points D and E. It is assumed that the friction of the slides transfers the compressive force from the telescopic beam to the main beam at point C; this is also shown in Figure 73.2.

Section Properties for the Two RHS Members

The members chosen are a 200 x 100 x 5 RHS for the telescopic beam and a 250 x 150 x 6 RHS for the main beam. Both are Grade C350 to AS1163 [8].

Their section properties and nominal capacities that are relevant to this design example are given in Table 73.1.

Table 73.1
Section Properties and Nominal Capacities

Telescopic Beam - 200 x 100 x 5	
I_x = 14.4 x 10^{-6} m^4	I_y = 4.92 x 10^{-6} m^4
r_x = 71.5 mm	r_y = 41.8 mm
J = 12.1 x 10^{-6} m^4	
k_f = 0.925	
N_s = 911 kN	
M_z = 36.0 kNm	
M_{sx} = 62.8 kNm	M_{sy} = 31.6 kNm
V_{vx} = 380 kN	V_{vy} = 189 kN
Main Beam- 250 x 150 x 6	
I_x = 38.4 x 10^{-6} m^4	I_y = 17.5 x 10^{-6} m^4
r_x = 92.0 mm	r_y = 62.2 mm
J = 39.0 x 10^{-6} m^4	
k_f = 0.907	
N_s = 1433 kN	
M_z = 83.0 kNm	
M_{sx} = 131 kNm	M_{sy} = 72.9 kNm
V_{vx} = 577 kN	V_{vy} = 348 kN

Notes:

1. The above are obtained from [4], except that the nominal capacities for compression, moment and shear are given in this table, rather than the design capacities. This is because the alternative design method is being used. That requires determination of nominal capacity, to which a factor of safety, Ω, is then applied. Table P3.3 of [5] gives the required values of Ω, which are repeated in Table 73.2.

2. The nominal capacity equals the design capacity, as given by [4], multiplied by (1/ϕ) or (1/0.9).

Table 73.2
Alternative Design Method Factors of Safety (Ω)

Permissible Strength For	**Factor Of Safety (Ω)**
Bending	0.60
Shear	0.62
Axial Forces	0.60
Combined Actions	0.60

Note: These are the values from NZS 3404 Table P3.3 that are relevant to this design example.

Design Check on Telescopic Beam

Critical location for member capacity

This is, by inspection, at the right hand side of point E.

Design actions in member

Refer to the axes shown in Fig. 73.2 for the signs of the forces.

F^*_y = F x sin 45 (Approx since on 10^o slope)
= 5.30 KN

F^*_z = F x cos 45 (Approx since on 10^o slope)
= 5.30 KN

F^*_x = F x sin 10 (Ground slopes downwards in positive x direction)
= 1.30 KN

$R^*_{y,E}$ = 15.6 KN (Find by taking moments about point D)

$R^*_{z,E}$ = 5.30 KN

$R^*_{y,D}$ = -10.3 KN $\left(F^*_y - R^*_{y,E}\right)$

$M^*_{x,E}$ = F^*_y x 3.30
= 17.5 KNm

$M^*_{y,E}$ = F^*_x x 3.30
= 4.30 KNm

$M^*_{z,E}$ = 2.25 KNm (design torsion; this is applied at the end of the boom, in conjunction with the applied load F^* which is acting through the shear centre of the beam)

$V^*_{y,E}$ = 5.30 KN

$V^*_{x,E}$ = 1.30 KN

Determination of Second-Order Effects

In an elastic analysis to NZS 3404 Clause 4.4, the first requirement is to determine the influence of second-order effects. For the telescopic beam, which is a sway member, this requires the calculation of the elastic buckling load, N_{oms}.

The elastic effective length factor for a sway member with a fixed base is given by NZS 3404 Fig. 4.8.3.2 case number 5, ie. k_e = 2.2.

However, the telescopic beam is not fixed at point E, but is continuous past that point to D. It therefore undergoes rotation at E due to the applied moment over length DE. This rotation will increase the deflection at point F.

To allow for this, the deflection at F due to cantilever plus support rotation must be determined, then converted to an equivalent fixed-ended cantilever length EF that would give the same deflection at F.

Cantilever deflection at F, excluding rotation at E

$$\upsilon = \frac{PL^3}{3EI} = 22.0 \times 10^{-3}\text{m}$$

P = 5.30 kN
L = 3.30 m
E = 200 GPa
I_x = 14.4 x 10^{-6} m^4

Slope at E

$$\theta_E = \frac{M^*_{x,E}\,2L}{6EI} = 3.44 \times 10^{-3}\text{radians}$$ (From Table 3, item 3e, page 103 of [7] or page 107, 6^{th} edition)

$M^*_{x,E}$ = 17.5 KNm
L = 1.70 m

Total deflection at E

$$\upsilon(\text{total}) = \upsilon + \theta_E \times 3.30 = 33.4 \times 10^{-3}\text{m}$$

Equivalent beam length as fixed ended cantilever

$$\upsilon = \frac{PL^3_{eq}}{3EI_x}$$

$$L_{eq} = \sqrt[3]{\frac{\upsilon\, 3EI_x}{P}} = 3.79\text{m}$$

This equivalent length is then used for N_{oms} calculation in the x and y directions.

Calculation of N_{omsx} and N_{omsy}

$$N_{omx} = \frac{\pi^2 EI_x}{(k_{ex} L)^2} = 409\text{KN}$$

I_x = 14.4 x 10^{-6} m^4
k_{ex} = 2.2 (case 5, NZS 3404 Fig. 4.8.3.2; allows for curvature in main beam, etc)
L = 3.79 m

$$N_{omy} = \frac{\pi^2 EI_y}{(k_{ey} L)^2} = 140\text{kN}$$

I_y = 4.92 x 10^{-6} m^4
k_{ey} = 2.2
L = 3.79 m

Calculation of elastic buckling load factors
This is to Clause 4.9.2.3;

$$\lambda_{cx} = \frac{N_{omsx}}{N^*} = \frac{409}{5.30} = 77.2$$

$$\lambda_{cy} = \frac{N_{omsy}}{N^*} = \frac{140}{5.30} = 26.4$$

Consideration of second-order effects

As λ_{cx} and λ_{cy} ≥ 10, from Clause 4.4.2.2.1 of [5] second order effects on the telescopic beam member can be neglected.

Bending about the X-axis

Determining critical cross section along boom for bending moment determination

This involves checking Clause 5.3.3.

At point E; $\frac{M^*}{M_{sx,E}} = \frac{17.5}{62.8} = 0.28$

At point B; $\frac{M^*}{M_{sx,B}} = \frac{39.8}{131} = 0.30$

Hence, in practice, both points need checking as either could be critical.

Section moment capacity at E

$\Omega M_{SX,E}$ = 0.6 x 62.8 = 37.7 kNm
Ω = 0.6, from Table 73.2
M_{sx} = 62.8 kNm, from Table 73.1

Member moment capacity for length EF

This is determined using Clause 5.6.2 of [5]. The segment EF is the relevant segment; this has constant cross section. The end at E has full twist restraint from the support to the main member; the end at F is unrestrained.

$$M_{oa,EF} = M_o = \sqrt{\left\{\left[\left(\frac{\pi^2 EI_y}{L_e^2}\right)\right]\left[GJ + \left(\frac{\pi^2 EI_w}{L_e^2}\right)\right]\right\}} = 929\text{KNm}$$

(Eqn 5.6.1.1(4))

I_y = 4.92 x 10^{-6} m^4
L_e = 3.30 m
k_t = 1.0 (FU)
k_l = 1.0 (Load applied above flange, but this is taken into account by the torsion moment M^*_z)
k_r = 1.0 (FU)

G = 80 GPa
J = 12.1 x 10^{-6} m^4
I_w = 0 (RHS member; see Clause 5.6.1.4)

$$\alpha_{s,EF} = 0.6\left\{\sqrt{\left[\left(\frac{M_{sx}}{M_{oa}}\right)^2 + 3\right]} - \left(\frac{M_{sx}}{M_{oa}}\right)\right\} = 1.0$$

(Eqn 5.6.1.1(3))

$M_{sx,E}$ = 62.8 KNm

$\alpha_{m,E}$ = 1.25 (see Table 5.6.2, case Number 2).

$\Omega M_{bx,E} = \Omega\alpha_s\,\alpha_m\,M_{sx} \leq \Omega M_{sx}$

$\Omega M_{bx,E} = \Omega M_{sx} =$ 37.7 kNm

As $\alpha_s\alpha_m$ > 1.0, the segment EF has full lateral restraint.

(Check on point B is covered in the design of the main member)

Checking x-axis moment adequacy

$M^*_{x,E} \leq \Omega M_{bx,E}$ is required

$M^*_{x,E}$ = 17.5 kNm ≤ 37.7 kNm √ OK

Bending about the y-axis

$M^*_{y,E} \leq \Omega M_{sy,E}$ is required

$M^*_{y,E}$ = 4.30 kNm ≤ 0.6 x 31.6 = 19.0 kNm √ OK

Calculation of equivalent shear

As described in section 8.2.4 of the *Structural Steelwork Limit State Design Guides Volume 1* [9] and also in Commentary Clause C8.5 of NZS 3404: Part 2 [5], the uniform torsion, M^*_z, will interact with the applied shear, $V^*_{y,E}$ to produce an equivalent design shear force.

The second-order influence of M^*_x on the design torque must first be determined, as described in C8.5.4.2 (c) of [5].

$$\delta_z = \frac{1}{1 - \frac{M^*_{x,E}}{M_{oa,EF}}} = \frac{1}{1 - \frac{17.5}{929}} = 1.02$$

$M^*_{z,E} = M^*_z \text{ x } \delta_z = 2.25 \text{ x } 1.02 = 2.30\,\text{kNm}$

Equivalent shear is now given from C8.5.5.2 of [5] as;

$$V^*_{eq} = V^*_{y,E} + \left(\frac{\tau^*_u V_w}{0.60 \times f_y}\right) = 27.7\,\text{KN}$$

$V^*_{y,E} = 5.30\,\text{KN}$

$$\tau^*_u = \frac{M^*_z}{2 \times Ae \times t} = \frac{2.30 \times 10^{-3}}{2 \times 18.5 \times 10^{-3} \times 5 \text{ x } 10^{-3}} = 12.4\text{MPa}$$

$$A_e = (b-t)(d-t) = 18.5 \times 10^{-3}\text{m}^2$$

(see section 8.2.4.1 of [9] or Appendix H, Clause H4 of [5])

b = 100 mm
d = 200 mm
t = 5 mm
$V_w = V_{vx}$ = 380 kN
f_y = 350 MPa

V^*_{eq} = $27.7\text{kN} < \Omega V_w = 0.62 \text{ x } 380 = 236\text{ kN}$

Check interaction of equivalent shear and bending

$$\frac{M^*_{x,E}}{\Phi M_{sx}} = \frac{17.5}{0.6 \times 62.8} = 0.46 < 0.75$$

Therefore don't need to check interaction of equivalent shear and bending (Clause 5.12.2 of NZS 3404 applied to working loads).

The y-axis check is similar and will be OK by comparison.

Calculation of member compression capacity

Member compression capacity is calculated for the length EF.

The actual length EF of 3.3 m is used for this, with an effective length factor of 1.0, as this is now design of the member and second-order effects have already been determined.

$N_c = \alpha_c N_s$
$N_{cx} = 0.895 \times 911 = 815\text{ KN}$
$N_{cy} = 0.675 \times 911 = 615\text{ KN}$

α_{cx} = 0.895 From Table 6.3.3(2) of [5] for α_b = -0.5 and λ_{nx} = 52.5
α_{cy} = 0.675
α_b = -0.5

$$\lambda_n = \left(\frac{L_e}{r}\right)\sqrt{(k_f)}\sqrt{\left(\frac{f_y}{250}\right)}$$

λ_{nx} = 52.5
λ_{ny} = 89.8
r_x = 71.5 mm
r_y = 41.8 mm

L_e = 3.3 m
k_e = 1.0
L = 3.3 m
k_f = 0.925
f_y = 350MPa

Check on member adequacy in compression

$N^* \le \Omega N_s$
$5.30 \le 0.6 \times 911 = 547$ kN √ OK

$N^* \le \Omega N_{cx}$
$5.30 \le 0.6 \times 463 = 278$ kN √ OK

$N^* \le \Omega N_{cy}$
$5.30 \le 0.6 \times 137 = 82.2$ kN √ OK

Check combined bending and compression

The section capacity check at E uses Clause P8.3.4, as applied through Clause 8.3.4.1.

$$\frac{N^*}{\grave{U}N_s}+\frac{M_x^*}{\grave{U}M_{sx}}+\frac{M_y^*}{\grave{U}M_{sy}}=\frac{5.30}{0.6\times 911}+\frac{17.5}{0.6\times 62.8}+\frac{4.30}{0.6\times 31.6}$$
$$=0.701\le 1.0$$

The member capacity check on EF uses Clause P8.4.5, as applied through Clause 8.4.5.1.

$$\left(\frac{M_x^*}{\grave{U}M_{cx}}\right)^{1.4}+\left(\frac{M_y^*}{\grave{U}M_{iy}}\right)^{1.4}=\left(\frac{17.5}{0.6\times 62.1}\right)^{1.4}+\left(\frac{4.30}{0.6\times 31.1}\right)^{1.4}$$
$$=0.475\le 1.0$$

$M_{cx}=M_{ix}$ Full lateral restraint ; see Clause 8.4.5.1

$$M_{ix}=M_{sx}\left(1-\frac{N^*}{\grave{U}N_{cx}}\right)=62.8\times\left(1-\frac{5.30}{0.6\times 815}\right)=62.1\,\text{KNm}$$

$$M_{iy}=M_{sy}\left(1-\frac{N^*}{\grave{U}N_{cy}}\right)=31.6\times\left(1-\frac{5.30}{0.6\times 615}\right)=31.1\,\text{KNm}$$

The design check on the telescopic boom is now complete.

Design Check on Main Member

The critical cross section for bending is, by inspection, over the hydraulic ram at point B.

Design actions in member

$M^*_{x,B} = F^*_y \times 7.50$
$= 39.8$ KNm

$M^*_{y,B} = F^*_x \times 7.50$
$= 9.75$ KNm

$M^*_{z,B} = 2.25$ KNm

$V^*_{y,B} = 5.30$ KN

$V^*_{x,B} = 1.30$ KN

$R^*_{y\,B} = F^*_y \times (3.3+4.2+1.2)/1.2 = 38.5$ kN
(force required at ram for this configuration)

$R^*_{y\,A} = -M^*_{Bx}/1.2 = -33.2$ kN

Determination of second-order effects

The same situation applies over length BC of the main member as applied over length EF of the telescopic boom, in terms of the need to calculate an equivalent fixed ended cantilever length BC.

Cantilever deflection at C, excluding rotation at B

$$\upsilon_c = \frac{PL^3}{3EI_x}=17.04\times 10^{-3}\,\text{m}$$

$P = F_y^* = 5.30$ kN (we are ignoring self weight)
$L = 4.20$ m
$E = 200$ GPa
$I_x = 38.4 \times 10^{-6}$ m^4

Slope at B

$$\theta_B = \frac{M_B^* 2L}{6EI}=1.16\times 10^{-3}\ \textit{radians}$$

$M_B^* = 22.3$ KNm
$L = L_{AB} = 1.2$ m

Total deflection at C

$$\upsilon(\text{total})=\upsilon_c+\grave{e}_B\times 4.20=21.9\times 10^{-3}\,\text{m}$$

Equivalent beam length as fixed ended cantilever

$$\upsilon=\frac{PL_e^3}{3EI_x}$$

$$L_e = \sqrt[3]{\frac{\tilde{o}\,3EI_x}{P}} = 4.57\text{m}$$

This equivalent length is then used for N_{oms} calculation in the x and y directions.

Calculation of N_{omsx} and N_{omsy}

The main beam is a combined section and this needs to be accounted for. The relevant table from Roark, fifth edition [7] is page 534, table 34, item 1a. From the sixth edition it is on page 670.

$$N_{omx}=K_1\times\frac{\pi^2 EI_{x1}}{L^2_{eq,x}}=170\ \text{KN}$$

$K_1 = 0.419$ from table 34 of [5]

$\frac{I_{x2}}{I_{x1}}=\frac{38.4}{14.4}=2.67$ Therefore take as equal to 2.0 to match tables.

$$\frac{a}{L_{eq,x}} = \frac{4.82}{8.36} = 0.58$$ Therefore take as equal to 0.5

$$\frac{P_2}{P_1} = \frac{0}{5.30} = 0$$ (There is no increase in compression load on the larger section)

$$L_{eq} = 4.57 + 3.79 = 8.36 \text{ m}$$

$$N_{omy} = \frac{K_1 \text{ x } \pi^2 \, EI_{y1}}{(2.2/2)^2 L_{eq,y}^2} = 39.8\text{KN}$$

The (2.2/2) multiplier on the denominator for N_{omy} makes allowance for the increase in effective length for a fixed cantilever specified by NZS 3404 Fig. 4.8.3.2 Case 5. Given that K_1 from Roark already takes account of the cantilever configuration, ie. k=2, with regard to elastic compression buckling load, the adjustment for NZS 3404 is (2.2/2).

K_1 = 0.419 from table 34 of [5].

$$\frac{I_{y2}}{I_{y1}} = \frac{17.5}{4.92} = 3.56 \Rightarrow \text{take as equal to 2.0 for tables}$$

$$\frac{a}{L_{eq,y}} = \frac{5.4}{9.2} = 0.59 \Rightarrow \text{take as equal to 0.5}$$

$$\frac{P_2}{P_1} = \frac{0}{5.30} = 0$$

$L_{eq} = 1.20 + 4.2 + 3.79 = 9.2$ (The distance AB is added, because there is no support against buckling about the y-axis offered by the hydraulic support at B. The actual length ABC is therefore used).

Calculation of elastic buckling load factors

$$\lambda_{cx} = \frac{N_{omsx}}{N^*} = \frac{170}{5.30} = 32.1$$

$$\lambda_{cy} = \frac{N_{omsy}}{N^*} = \frac{39.8}{5.30} = 7.51$$

As $\lambda_{cx} > 10$, no need to magnify M_x^*.

As $\lambda_{cy} < 10$, must magnify M_y^*.

$$\delta_m = \delta_s = \frac{0.95}{1 - \left[\frac{1}{\lambda_{cy}}\right]} = 1.10$$

$$M_{y,B}^* = \delta_m \, M_{y\,B\text{ firstorder}}^* = 1.10 \text{ x } 9.75 = 10.73 \text{ kNm}$$

Bending about the x-axis

This involves checking moment capacity at B; section moment capacity at E and member moment capacity over segment length AF. The latter is based on the hydraulic ram not providing effective twist restraint at B, which is probably conservative.

Section moment capacity at B

$\Omega M_{sx,B}$ = 0.6 x 131 = 78.6 kNm
Ω = 0.6, from Table 73.2
$M_{sx,B}$ = 131 kNm, from Table 73.1 for the lower section.

Member moment capacity along length AF

This is determined by using Clause 5.6.2 of [5], in conjunction with Clause 5.6.1.1.2 which accounts for the change in cross section at C.

Calculation of non-uniformity factor, α_{st}

$$\alpha_{st} = 1.0 - [1.2 r_r (1 - r_s)] = 0.777$$

$$r_r = L_r/L = 0.38$$

$$r_s = \frac{A_{fm}}{A_{fc}}\left[0.6 + \left(\frac{0.4 d_m}{d_c}\right)\right] = 0.511$$

A_{fm} = 0.100 x 0.005 x 2 = 1.00 x 10^{-3}m
A_{fc} = 0.150 x 0.006 x 2 = 1.80 x 10^{-3}m
d_m = 200 mm
d_c = 250 mm
L_r = 3.3 m
L = 1.2 + 4.2 + 3.3 = 8.70 m (Actual length of main beam AC and length of telescopic beam EF)

Calculation of uniform elastic buckling moment, M_{oa}, based on cross-section at B (same as at A)

$$M_{oa,u,AF} = \sqrt{\left\{\left[\left(\frac{\pi^2 EI_y}{L_e^2}\right)\right]\left[GJ + \left(\frac{\pi^2 EI_w}{L_e^2}\right)\right]\right\}} = 1193 \text{ kNm}$$

(Eqn 5.6.1.1(4))

$I_{y,A}$ = 17.5 x 10^{-6} m^4
L_e = 1.0L = 8.70 m
k_t = 1.0 (FU)
k_l = 1.0 (Load above flange but taken into account by moment M_z)
k_r = 1.0 (FU)
G = 80 GPa
J = 39.0 x 10^{-6} m^4
I_w = 0 (RHS member; see Clause 5.6.1.4)

Reduction in elastic buckling moment for the segment due to the change in cross-section at C.

$M_{oa,AF}$ = α_{st} x $M_{oa,u,AF}$ = 0.777 x 1,193 = 927 kNm

$$\alpha_{s,AF} = 0.6\left\{\sqrt{\left[\left(\frac{M_{sx}}{M_{oa}}\right)^2 + 3\right]} - \left(\frac{M_{sx}}{M_{oa}}\right)\right\} = 0.96$$

(Eqn 5.6.1.1 (3))

M_{sx} = 131 kNm

α_m = 1.25 (To allow for self weight and the moment distribution in the beam due to the telescopic beam reactions, refer *DCB* No. 16 for improved α_m)

$M_{bx} = \alpha_m \alpha_s M_{sx}$ = 1.25 x 0.96 x 131 = 156 kNm $\leq M_{sx}$ (Eqn 5.6.1.1(1))

$M_{bx,B}$ = $M_{sx,B}$ = 131 kNm

$\Omega M_{bx,B}$ = 0.6 x 131 = 78.6 kNm

(As $\alpha_s \alpha_m$ > 1.0, the segment AF has full lateral restraint).

Checking x-axis moment adequacy

$M^*_{x,B} \leq \phi M_{bx,B}$ is required

$M^*_{x,B} = 39.8\,\text{kNm} \leq 78.6\,\text{kNm}$ √ OK

Bending about the y-axis

$M^*_{yB} \leq \Omega M_{syB}$ is required

$M^*_{y,B}$ = 10.73 kNm (including second - order effects) $\leq$ 0.6 x 72.9 = 43.7 kNm

√ OK

Calculation of equivalent shear

The second-order influence of M^*_x on the design torque on the main member must first be determined.

$$\delta = \frac{1}{1 - \frac{M^*_x}{M_{oa}}} = 1.04$$

M^*_x = 39.8 kNm

M_{oa} = 927 kNm

$M^*_{z,B}$ = M^*_z x δ_z = 2.25 x 1.04 = 2.35 kNm

Equivalent shear is now calculated thus:

$$V^*_{eq} = V^*_{yB} + \left(\frac{\tau^*_u V_w}{0.60 \times f_y}\right) = 20.6\,\text{KN}$$

$V^*_{y,B} = 5.30$ KN

$$\tau^*_u = \frac{M^*_{Z,B}}{2 \times Ae \times t} = \frac{2.35 \times 10^{-3}}{2 \times 35.1 \times 10^{-3} \times 6.0 \times 10^{-3}} = 5.59\,\text{MPa}$$

$A_e = (b - t)(d - t) = 35.1 \times 10^{-3}$ m^2

b = 150mm

d = 250mm

t = 6.0mm

$V_w = V_{vx} = 577$ KN

f_y = 350 MPa

V^*_{eq} = 20.6 kN << ϕV_w = 0.62 x 577 = 358 kN

Check interaction of equivalent shear and bending

$$\frac{M^*_{x,B}}{\phi M_{sx}} = \frac{39.8}{78.6} = 0.51 < 0.75$$

Therefore don't need to consider interaction of equivalent shear and bending.

The y-axis check is similar and will be OK by comparison.

Calculation of member compressive capacity

Member compression capacity is calculated for the length BF for the x-axis and the length AF for the y-axis, as the hydraulic ram does not provide y-axis restraint.

Both lengths comprise the main member and the telescopic member, thus requiring design to Clause 6.3.4.

This first requires calculation of N_{omx} and N_{omy}, so as to calculate the slenderness ratio.

Calculation of N_{omx} and N_{omy}

The relevant table from Roark fifth edition [7] is Table 34, p.535. For the sixth edition it is on page 670.

Earlier in this main member design, the N_{omx} and N_{omy} for the elastic effective lengths for this sway member were calculated in order to determine the second-order effect multipliers. For individual member design, in accordance with NZS 3404 Clause 4.8.3.1 (a) (iv), the member effective length factor is taken as k_e = 1.0. This corresponds to a pin-pin ended member and so means that the item number from Table 34 of [7] corresponding to the k_e = 1.0 (pin-pin) case must be used. That is case number 1b.

$$N_{omx} = K_1 \times \frac{\pi^2 EI_{x1}}{(k_e L_{eq,x})^2} = 577\,\text{KN}$$

$K_1 = 1.297$ From table 34 of [7], Item 1b.

$\frac{I_{x2}}{I_{x1}} = \frac{38.4}{14.4} = 2.67$ Therefore let equal 2.0 to match tables.

$\frac{a}{L_{eq,x}} = \frac{4.2}{7.99} = 0.525$ Therefore let equal 0.5

$\frac{P_2}{P_1} = \frac{0}{5.30} = 0$

$k_e = 1.0$
$L_{eq,x} = 4.20 + 3.79 = 7.99$ m

$$N_{omy} = K_1 \times \frac{\pi^2 EI_{y1}}{(k_e\ L_{eq,y})^2} = 149\ \text{KN}$$

$K_1 = 1.297$ From table 34 of [1], Item 1b

$\frac{I_{y2}}{I_{y1}} = \frac{17.5}{4.92} = 3.56$ Therefore let equal 2.0 to match tables.

$\frac{a}{L} = \frac{5.4}{9.19} = 0.588$ Therefore let equal 0.5

$\frac{P_2}{P_1} = \frac{0}{5.30} = 0$

$L_{eq,y} = 5.40 + 3.79 = 9.19$ m

Calculation of modified member slenderness

This is to Clause 6.3.4 (b)

$$\text{ë}_n = 90\sqrt{\left(\frac{N_s}{N_{om}}\right)}$$

$$\text{ë}_{nx} = 90 \text{ x } \sqrt{\left(\frac{911}{577}\right)} = 113$$

$$\text{ë}_{ny} = 90 \text{ x } \sqrt{\left(\frac{911}{149}\right)} = 223$$

N_s = 911 kN calculated for the smallest cross section, which is the telescoping beam.

N_{omx} and N_{omy} are as calculated above.

Calculation of member compression capacity

$N_c = \alpha_c N_s$

N_{cx} = 0.508 x 911 = 463 kN

N_{cy} = 0.150 x 911 = 137 kN

α_{cx} = 0.508 for λ_{nx} = 113 From Table 6.3.3 (2) of [5]

α_{cy} = 0.150 for λ_{ny} = 223

α_b = -0.5

Check on member adequacy in compression

$N^* \leq \Omega N_s$
$5.30 \leq 0.6 \times 1433 = 860$ kN √ OK

$N^* \leq \Omega N_{cx}$
$5.30 \leq 0.6 \times 463 = 278$ kN √ OK

$N^* \leq \Omega N_{cy}$
$5.30 \leq 0.6 \times 137 = 82.2$ kN √ OK

Check combined bending and compression

The section capacity check at B uses Clause P8.3.4, as applied through Clause 8.3.4.1.

$$\frac{N^*}{\text{Ù}N_s} + \frac{M_x^*}{\text{Ù}M_{sx}} + \frac{M_y^*}{\text{Ù}M_{sy}} = \frac{5.3}{0.6 \times 1433} + \frac{39.8}{0.6 \times 131} + \frac{10.73}{0.6 \times 72.9} = 0.767 \leq 1.0$$

The member capacity check on BF or AF respectively as appropriate (for the x-axis and the y-axis) uses Clause P.8.4.5, as applied through Clause 8.4.5.1.

$$\left(\frac{M_x^*}{\text{Ù}M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\text{Ù}M_{iy}}\right)^{1.4} = \left(\frac{39.8}{0.6 \times 129}\right)^{1.4} + \left(\frac{10.73}{0.6 \times 68.2}\right)^{1.4} = 0.55 \leq 1.0$$

$M_{cx} = M_{ix}$ Full lateral restraint; see Clause 8.4.5.1

$$M_{ix} = M_{sx}\left(1 - \frac{N^*}{\text{Ù}N_c}\right) = 131 \times \left(1 - \frac{5.30}{0.6 \times 463}\right) = 129\ \text{KNm}$$

$$M_{iy} = M_{sy}\left(1 - \frac{N^*}{\text{Ù}N_c}\right) = 72.9 \times \left(1 - \frac{5.30}{0.6 \times 137}\right) = 68.2\ \text{KNm}$$

The design check on the main boom is now complete.

Conclusion

The telescopic lifting boom shown in Fig. 73.2 can carry the applied loads when extended and raised 45^o and situated on a slope of 10^o.

Restraint Issues Relating to Portal Frame Spine Beams

This article has been written by G Charles Clifton, HERA Structural Engineer.

Scope

Fig. 73.3 shows a typical example of a spine beam in a portal frame building. Spine beams are used in propped portal construction to support the portal frame in lieu of a column. They carry the vertical loading (either downwards or uplift) in bending across to supporting columns. This allows the number of supporting columns to be reduced from one every portal frame to one every second or third portal frame, as appropriate. It also means that an extra beam, typically under the apex, is required.

Given the pitch on a portal frame, the clear height from floor to bottom of spine beam at the apex will usually be no less than that from floor to underside of rafter at the knees, so the addition of a spine beam at the apex does not reduce the clear head height within the building.

The use of a spine beam also results in higher column axial loads, which results in larger column sizes that may have greater resistance against accidental impact (but also increase the foundation loads and possible size of foundation pad or pile required).

However, the use of a spine beam introduces restraint issues relating to the spine beam itself, the rafter it supports and the columns that support it, that must be considered.

The scope and purpose of this article is to cover the general restraint issues relating to portal frame spine beams. It addresses the following topics:

- restraint of the portal frame rafter
- restraint of the spine beam
- restraint of the column
- detailed design advice for common configurations of rafter and spine beam
- some general restraint and design issues.

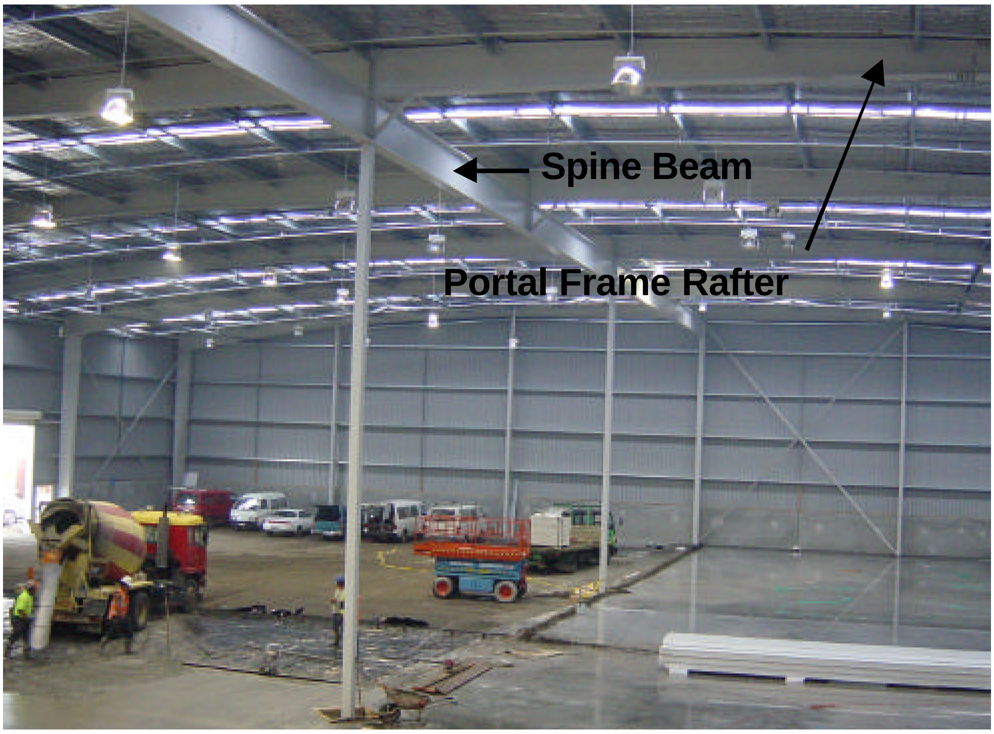


Fig. 73.3
Example of Spine Beam in Portal Frame Building

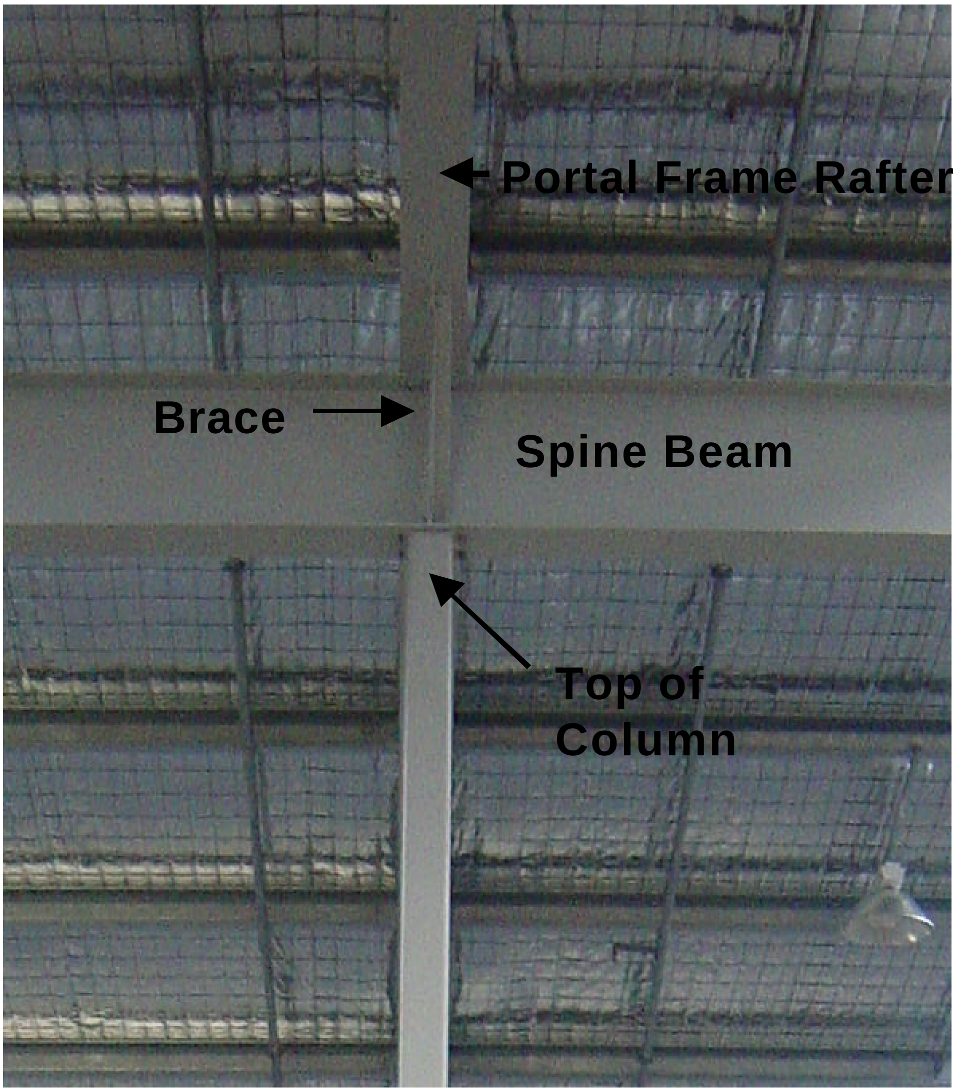


Fig. 73.4
Close up of Spine Beam/Portal Frame/Column Support

The article is written around the restraint provisions of NZS 3404 Clauses 5.4 and 6.7 and the use of HERA Report R4-92 [10] *Restraint Classifications for Beam Member Moment Capacity Determination to NZS 3404*. It covers these items in general terms initially, then goes into detail for common configurations of rafter and spine beam.

Prior to commencing with the first of these topics, some definitions of directions and location are given. These are used repeatedly throughout the article.

Definitions of Directions and Location

These should be read in conjunction with Fig. 73.4, which is a view looking along the span of the rafter, with the spine beam running at right angles underneath, across the picture.

The key directions are:

- parallel to the rafter, which means in the direction of the span of the rafter
- transverse to the rafter, which means at right angles to the span of the rafter and hence parallel to the span of the spine beam
- parallel to the span of the spine beam, which means in the direction of the span of the spine beam
- transverse to the span of the spine beam, which means at right angles to the span of the spine beam and hence parallel to the span of the rafter
- the rafter and spine beam are assumed to be at right angles in this article
- the top of the column for restraint discussions is taken at the bottom of the spine beam, as shown in Fig. 73.4.

Restraint of Portal Frame Rafter

There are no Connection Details from [10] which describe exactly the rafter over spine beam set-up, however Connection Details 11 and 12 come close. The main differences between those details and the rafter/spine beam case is that, in the latter:

(i) The top of the rafter is typically tied into the roof system, which provides a plane of inherent lateral restraint

(ii) The rafter is continuous over the spine beam.

Connection details 19 and 20 are also relevant.

In the case where the rafter is in positive bending going over the spine beam, the rafter top flange is the *Critical Flange* (CF - see NZS 3404 [5] Clause 5.5 for definition). At points of restraint to the roofing system, Connection Detail Nos. 19 or 20 from [10] applies and the spine beam is not required to provide restraint to the rafter.

Where the rafter is in negative bending going over the spine beam, the rafter bottom flange is critical. In that case, from Connection Detail Nos. 19 or 20, the cross section would have less than full twist restraint (F) without the spine beam and the additional restraint offered by that beam can be used to provide full twist restraint. The restraining system must prevent the rafter from twisting - ie. the bottom flange from moving parallel to the span of the spine beam.

If the rafter has one or two full depth stiffeners in this case (as does the system shown in Fig. 73.3), then the restraint against twist can be provided through the flexural restraint of the rafter to spine beam connection and the rafter to purlin connection. The restraining moments are then resisted by in-plane bending resistance of the purlins at the top and the spine beam at the bottom. If there is a typical moment connection between rafter and purlin (see Connection Detail 19 from [10]) and at least two bolts between rafter and spine beam that can resist in-plane moment in the spine beam generated by the out-of plane twist of the supported rafter, then this is a viable solution. It is the solution used in Fig. 73.3.

An alternative is to provide direct lateral restraint to the rafter bottom flange via the spine beam. This doesn't require the rafter to spine beam or the rafter to roof connections to provide moment resistance against rafter twist. However, it puts a lateral restraining force into the spine beam, which travels along the spine beam and accumulates in accordance with Clause 5.4.3.3 of [5]. This restraining force must be anchored back into the roof system, or else there could be a restraint failure caused by the lateral movement of the spine beam as a rigid unit, with all rafters connected to the spine beam rotating in the same direction. In the case of Fig. 73.3, for example, a way of providing this anchorage is to connect the spine beam into an end column in the wind wall, the top of which is tied into the roof bracing system in the end bay.

A final alternative offered is to use a fly brace to the rafter bottom flange, as shown in Connection Detail 21 from [10]. In this case, the fly brace plus spine beam mean that F restraint will be provided, even when the purlin is flexible (see C5.4.2.2 of NZS 3404 [5]).

See the cases described under *Detailed Advice for Common Configurations* for more detailed guidance.

As discussed under *Restraint of the Column,* later in this article, the column restraining forces parallel to the spine beam operate in the same direction as those generated by the rafter bottom flange under negative moment and must be effectively anchored back into the roof plane. The restraining system needs to be designed to resist the greater of these two sets of restraint forces, not the sum of the two.

Restraint of Spine Beam

Fig. 73.3 shows a system where every second portal frame apex is supported by a spine beam, the others being supported by a column. The vertical stiffness offered at the apex by a column is potentially much greater than that offered by the spine beam. In order to control differential vertical deflection at the apex between adjacent portal frames, the spine beam design is likely to be governed by serviceability deflection limitations; eg. differential deflection limits of (frame spacing / 200) or (frame spacing / 250) may apply under the relevant load combinations (see eg. NZS 4203 [11] Table C2.4.1). There will be more stringent limits required where the roof system supports a suspended ceiling. This means that the spine beam is likely to be made continuous over the supporting columns, as indeed is the beam shown in Figs. 73.3 and 73.4, and pinned at the end supporting columns.

Under vertical downwards loading (dead, live, snow, downwards wind), such a spine beam will be subject to negative moment over the intermediate supporting columns and positive moment under the supported rafters.

The moment signs will be reversed for loads involving dead + wind that produce a net uplift.

Under negative moment, the bottom flange is the Critical Flange. Full section restraint should be provided, either via a stiffener and moment system as shown in Connection Detail 12 from [10] or a fly brace as shown in Figs. 73.3 and 73.4. Design of these systems is covered by [10].

Under positive moment, the top flange is the Critical Flange and Full section restraint is provided, by inspection. The lateral restraining force feeds into the rafter and through into the roof system, via the in-plane rigidity of the rafters, without the need for further calculation.

See the cases described under *Detailed Advice for Common Configurations* for more detailed guidance.

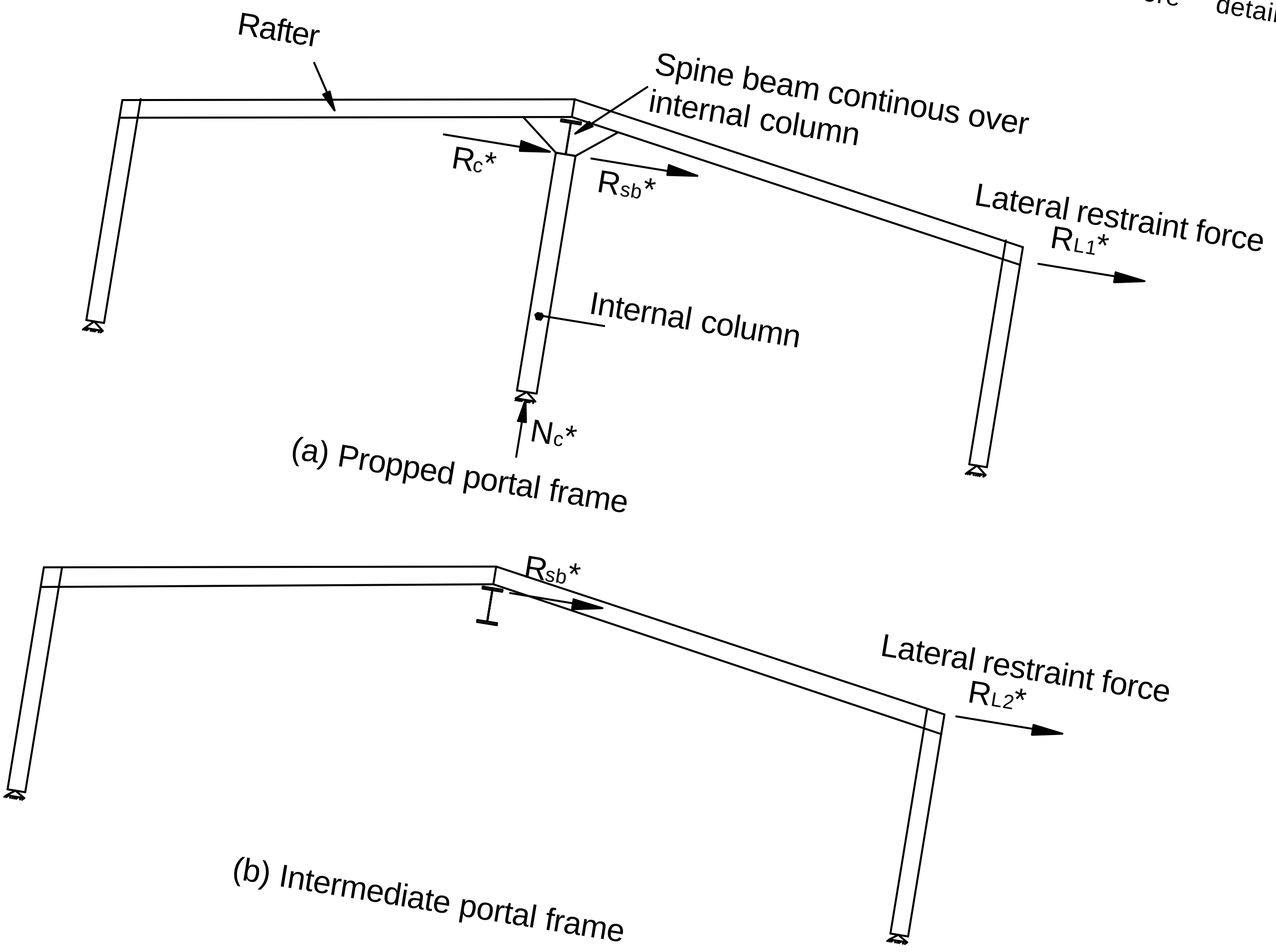


Fig. 73.5

Propped and Intermediate Portal Frames Showing Restraint Forces from Spine Beam and Column

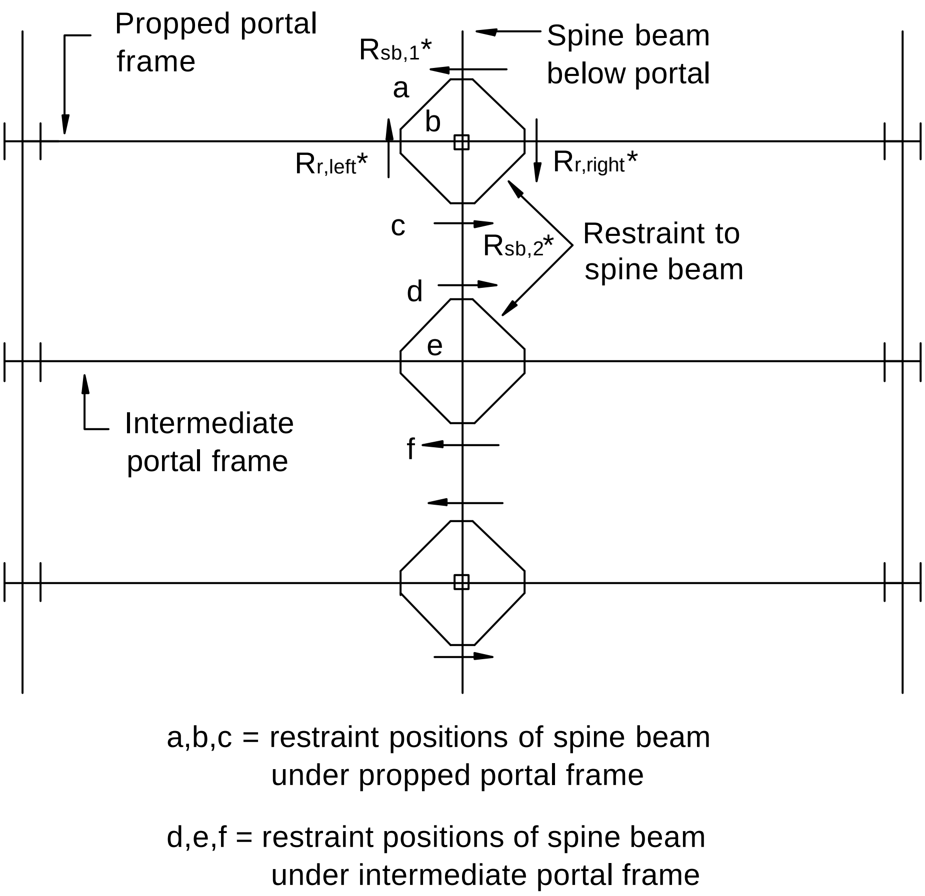


Fig. 73.6
In-Plan Restraint to Spine Beam Critical Flange from the Rafter System

Restraint of the Column

The top of the column, the position of which is shown in Fig. 73.4, must be a point of effective restraint, to NZS 3404 [5] Clause 6.7. This requires restraining forces determined by Clause 6.7.2.1 to be resisted about each of the column's principal axes.

In the direction transverse to the spine beam (parallel to the rafter), the lateral restraining force must be transferred by either of design options 1 or 2 described on pages 18-20 herein. Once this restraining force gets into the rafter, it is resisted by the portal frame/roof system as described on page 17.

In the direction parallel to the spine beam (transverse to the rafter), the lateral restraining force must transfer into the rafter and from there to the roof system. This lateral restraining force is acting in the same direction as the rafter bottom flange restraint force generated under negative moment, solutions for which have been given under *Restraint of Portal Frame Rafter*, on page 14 above. Whichever solution is adopted to resist these forces needs to be designed for the greatest magnitude restraint force from each source, not the sum of the two.

Detailed Advice For Common Configurations

Preamble and assumptions

This section of the article provides detailed advice on the derivation of restraint forces and their resistance by the overall building system for common spine beam/column configurations.

These configurations are:

- propped portal frame (Fig. 73.5 (a))
- intermediate portal frame (Fig. 73.5 (b))
- in-plan restraint to spine beam critical flange from the rafter system (Fig. 73.6).

The assumptions made in this section are as follows:

- The spine beam is continuous over the supporting column of the propped portal frames
- The portal frame building has a normal bracing system for building stability in addition to the cladding system (see pages 2-4 of *DCB* No. 50 for more details on what this requirement entails)
- The roof system provides inherent overall lateral stability (see pages 2-4 of *DCB* No. 50 and/or pages 2.7 to 2.9 of HERA Report R4-92 [10] for more details).

The details on restraint are as follows:

Propped portal frame case

This is shown in Fig. 73.5 (a). In this case the portal frame is providing restraint to the top of the internal column, requiring a restraint force, R^*_c, given by;

$$R^*_c = 0.025\, N^*_c \qquad (73.1)$$

where:

N^*_c = design axial compression on internal column

The portal frame is also providing restraint to the bottom flange of the spine beam, this being the critical flange for vertical load cases - ie. the flange in compression under the negative moment. The spine beam restraint force at the propped portal, $R^*_{sb,pp}$, is given by;

$$R^*_{sb,pp} = 0.025 \frac{M^*_{sb,col}}{(d_b - t_f)_{sb}} \qquad (73.2)$$

where:

$M^*_{sb,col}$ = design negative moment in spine beam at the column

d_b, t_f = spine beam depth and flange thickness

The fly braces shown in that figure must be designed to transfer $(R^*_c + R^*_{sb})$ into the rafter, as shown in Fig. 73.7, design option 1. The design of these braces is covered by section 2.5.3 of R4-92 [10].

For the reasons given in section 2.5.2 (3) of [10], the spine beam restraint force is resisted by in-plane bending in the rafter (see Fig. 2.3 of [10] for the distribution of restraint forces in the fly brace system).

Because the roof system provides inherent overall lateral stability, this force is not required to be resisted by the portal frame as a lateral force, so it does not contribute to R^*_{L1}.

The column restraint force, R^*_c, is carried through into the rafter by the fly braces (or through the stiffener/column system if design option 2 on pages 18-20 is used), where it has to be resisted by the portal frame, ie.:

$$R^*_{L1} = R^*_c \qquad (73.3)$$

The restraint force R^*_{L1} can be resisted by the portal frame, in which case it becomes an additional applied force in the load case generating the design actions on the spine beam and column. The critical load case for this is typically either 1.2G + 1.6Q, or 1.2G + 1.2S_u, from NZS 4203 [11] Clause 2.4.3.3.

However, the fact that the cladding system provides a stiffening effect - as described on pages 4-7 of *DCB* No. 49 and 2-4 of *DCB* No. 50 - can be used to share the resistance of this force between the portal frame and the bracing system in the transverse direction (typically in the gable walls). The split of this force is in accordance with the advice for wind and earthquake given on page 7 of *DCB* No. 49 - ie. for a pinned base portal frame, $0.5 R^*_{L1}$ is resisted by the portal frame and $0.5 R^*_{L1}$ goes into the roof system and hence to the gable walls (or other transverse bracing system).

Intermediate portal frame case

This is shown in Fig. 73.5 (b). In this case, the spine beam is in positive moment and the top flange, which is attached directly to the underside of the portal frame, is the critical (compression) flange. The restraint force, $R^*_{sb,ip}$, is given by:

$$R^*_{sb,ip} = \frac{0.025\, M^*_{sb,ip}}{(d_b - t_f)_{sb}} \qquad (73.4)$$

where:

$M^*_{sb,ip}$ = design positive moment in the spine beam at the intermediate portal frame.

This lateral force must be resisted directly by the portal frame, thus:

$$R^*_{L2} = R^*_{sb,ip} \qquad (73.5)$$

The restraint force, R^*_{L2}, can be resisted solely by the portal frame or split between the portal frame and the roof/gable wall as described above.

Accumulated restraint force in roof/gable wall system

If the decision is made to split the lateral restraining forces R^*_{L1} and R^*_{L2} between the portal frames and the roof/gable wall system, then the gable wall bracing and the roof braced bay that feeds forces into the gable wall bracing must be designed for accumulated restraint forces in accordance with the parallel restrained members provisions of NZS 3404 [5] Clause 5.4.3.3. The restraint forces are those from each portal frame (ie. 0.5 R^*_{L1} and 0.5 R^*_{L2} for pinned portal frames) and the upper limit total restraint force into the gable walls or similar is that from one full quantum of restraint force (ie. 0.5 R* for pinned portal frames) plus six half quanta of restraint forces.

In-plan restraint to spine beam critical flange from the rafter system

These are the situations shown in Fig. 73.6. In these cases, additional restraints are provided, adjacent to the portal frame/spine beam intersection, to reduce the effective length of the spine beams for lateral buckling (so as to increase the member moment capacity). These restraints are triangulated back to the portal.

These restraints may be provided to the spine beam under the propped portal (positions a and c) or under the intermediate portal (positions d and f).

In the first case - ie. at the propped portal - the points of restraint could be in either negative or positive moment regions. It is important that the restraints go to the correct flange of the spine beam (ie. the critical flange), which will depend on the sign of the bending moment.

In the second case - ie. at the intermediate portal - the points of restraint will be in positive moment under vertical loading and the restraints would go to the top flange in this instance. (However, for a load case involving wind uplift, the moment is reversed although the design actions are likely to be lower).

The restraints are required to transfer the restraint force, R^*_{sb}, back to points of anchorage. As the spine beam is continuous under the rafter, the buckled shape of the spine beam critical flange over the length of interest (ie. a-b-c or d-e-f) will involve one flange trying to move to the left and the other to the right, ie. as shown in Fig. 73.6. Thus there will be no net lateral restraint force developed along the portal frame.

The tendency of the spine beam critical flange to rotate in-plan about points b or e, as appropriate, must be resisted by the rafters. The transverse forces exerted on the rafter, $R^*_{r,left}$ and $R^*_{r,right}$, are equal to $R^*_{sb,1}$ and $R^*_{sb,2}$ for the braces at 45° to the spine beam and rafter, as is shown in Fig. 73.6.

If the in-plan braces connect into the bottom flange of the rafter, then this restraint force will tend to twist the rafter at that point and must be carried up to the purlin line where it is resisted by major axis bending in the purlin. If fly braces are used for this - ie. fly braces from the rafter bottom flange up to the purlin, the design of these is covered by section 2.5.3 and Fig. 2.3 of [10].

The other option is to anchor these braces into the rafters near the top flange, where the restraint forces can be effectively anchored into the roof system without further consideration. This will eliminate the need for specific rafter twist restraint but at the expense of longer and more complex in-plan bracing members and connection details.

Because there is no net lateral restraint force from this in-plan bracing into the plane of the portal frame, the portal frame lateral restraint forces, R^*_{L1}, and R^*_{L2}, remain as given by equations 73.3 and 73.5, respectively.

Design options 1 and 2 for spine beam/rafter/column restraint of forces parallel to the rafter

Fig. 73.7 shows two design options for the spine beam/rafter/column restraint. The first of these involves fly braces, which, as previously described, carry the restraint forces R^*_{sb} and R^*_{c} up into the rafter and portal frame/roof system. This option provides a point of dependable restraint at the top of the column. It means that the column need not be designed to resist any restraint forces and therefore is designed to resist only N^*_c and a moment in the plane of the spine beam (ie. about the column axis transverse to the spine beam) generated by the eccentric transfer of vertical load requirement of NZS 3404 Clause 4.3.4.2.

Design option 1, in contrast, is based on the top connection transmitting restraint actions due to (R^*_{sb} + R^*_{c}) into the rafter system, although as described above only R^*_{c} goes into R^*_{L1}, which is required to be resisted by the portal frame/roof system.

Design option 2, in contrast, is based on the column/spine beam resisting the moment generated by the restraint actions shown in Fig. 73.8. The peak magnitude of this moment, which occurs at the top of the column, is given by:

$$M^*_{max} = \frac{\left(R^*_{sb} + R^*_{c}\right) d_{sb}\, L_{col}}{\left(d_{sb} + L_{col}\right)} \qquad (73.6)$$

where all variables are as defined in Fig. 73.8. That moment must be able to be developed by the stiffened spine beam cross-section at the bottom of the spine beam, the connection between the spine beam and column and the column itself. The moment in the column acts in a plane transverse to the spine beam - ie. about the column's other principal axis to the moment generated through the eccentric transfer of vertical load requirement of NZS 3404 Clause 4.3.4.2. This means that the column is subject to biaxial bending plus axial compression for design option 2. For a column section that complies with NZS 3404 Clause 8.1.5, the critical condition will be section capacity to Clause 8.3.4.2.

General Issues to Consider

Restraint stiffness requirements

Editions of NZS 3404/AS1250 prior to 1992 had requirements for restraining systems to meet both strength and stiffness requirements. As a result of studies that are referenced from Commentary Clause C5.4.3.1 of NZS 3404 Part 2 [5], the stiffness requirement has been removed. However, this is only the case where the restraining elements are made of steel or involve anchorage into a reinforced concrete slab or structural system. The key is to provide the strength requirements to "transfer to anchorage or reaction points" [5] the design restraint actions.

Restraint actions based on design actions

The restraint actions required by Clauses 5.4.3.1, 5.4.3.2, 5.4.3.3 or 6.7.2 are based on the design actions, not the member capacities. For load combinations not including earthquake this means that, where the member requiring restraint is oversized for strength, the restraining forces to be resisted are less than those generated by the member moment or compression capacity.

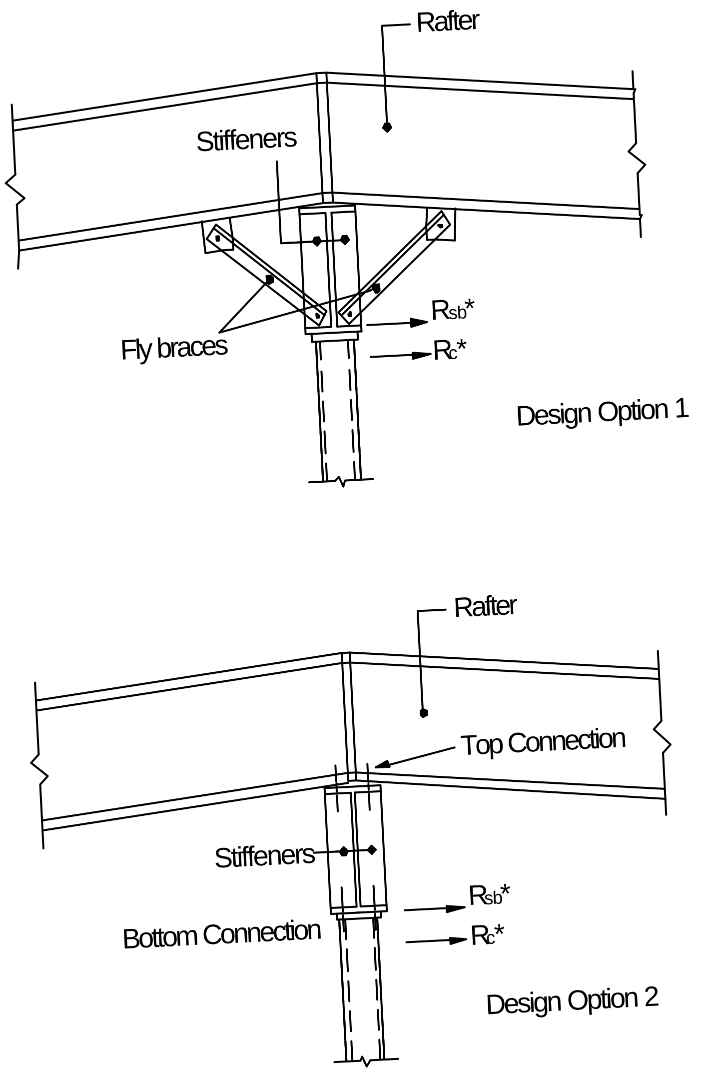


Fig. 73.7
Design Options 1 and 2 for Spine Beam/Rafter/Column Restraint

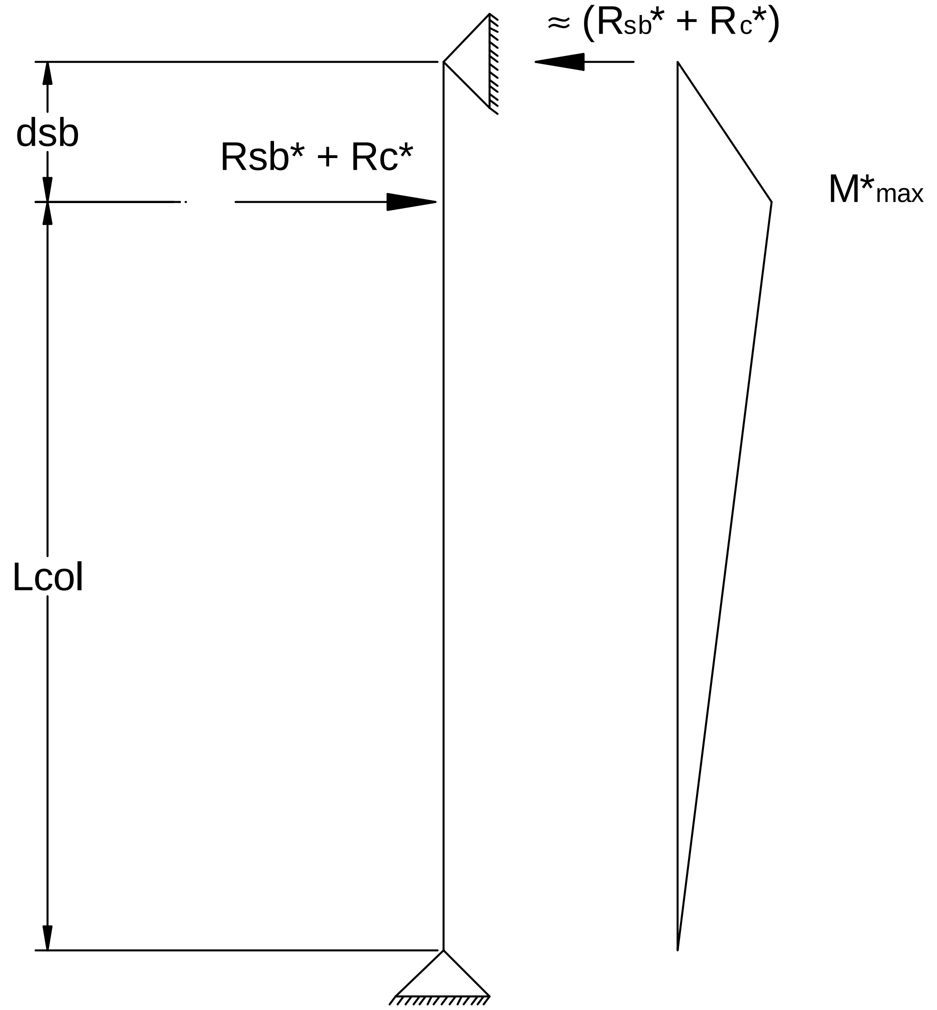


Fig. 73.8
Restraint Force Application and Moments for Design Option 2

Load height classification for spine beam supports to rafter

Where the spine beam supports the rafter, the rafter load is applied to the spine beam via the spine beam top flange. The structural system transferring this load is laterally restrained, so that, for a spine beam with a vertical axis of symmetry, k_1 = 1.0 applies from the NOTE (1) to Table 5.6.3 (2). In most instances, such a support is also the end of a segment, both ends of which are restrained, hence k_1 = 1.0 from Table 5.6.3 (2) irrespective of the load height position.

Bolts fully tensioned

The high strength structural bolts used in and around the spine beam should be fully tensioned to increase the rigidity of the overall system.

Where slotted or oversize holes are used, these bolts must be fully tensioned; see NOTE to Clause 5.4.2.1 and 5.4.2.2 of NZS 3404 [5].

Judicious use of beam web stiffeners

The designer should ascertain carefully where stiffeners to the beam web are needed in a rafter/spine beam system and only specify them where required.

As described in *DCB* No. 52, pp. 11-13, stiffeners to the (spine) beam over the supporting intermediate columns will almost always be needed.

However, stiffeners in either the spine beam or the rafter over the locations where the rafter is supported by the spine beam alone may not be necessary, especially where fly braces are also used on the spine beam, and these applications should be checked more closely.

NZFI-Vib1	Programmed by: Y. Khwaounjoo, HERA		Date: 1 May 2002 Version: 1

Project: **Example 2: HiBond slab (Location 2)**

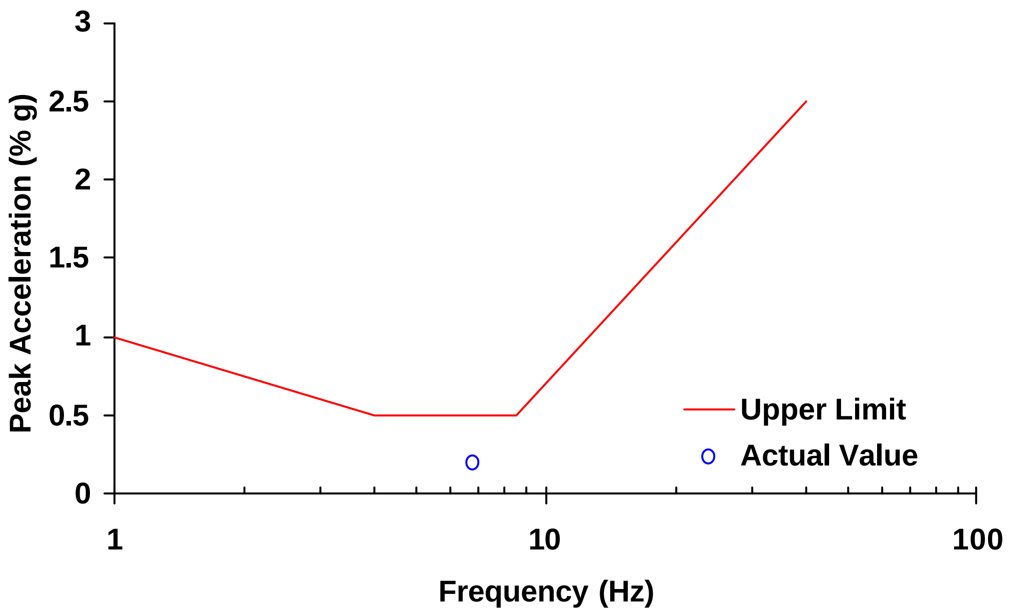
Designed by: **YRK**

b = 0.05		Slab Span (m) =	2.75
Beam/Joist Span 1 (m) =	9.00	Beam/Joist Span 2 (m) =	9.00
Girder 1 Span 1 (m) =	8.25	Girder 1 Span 2 (m) =	8.25

Vibration analysis based on the recommendation of AISC/CISC Design Guide 11- Floor Vibration to Human Activity, 1997, Murray et al., NZS3101:Part1:1995 and NZS 3404:Part 1:1997.

E. FINAL RESULT: **FOR SLAB SPAN OF 2.75m**

Member	Fundamental frequency (Hz)	Peak Acceleration (% g)
Slab	*18.18*	*0.02*
Beam/Joist	*8.18*	*0.10*
Girder 1	*12.04*	*0.05*
Girder 2		
Combined Floor	6.767	0.199
Limiting Combined Floor Acceleration (% g) = 0.50		



Floor response against limiting criteria

Fig. 73.9
Example of Output from Floor Vibration Program

Floor Vibration Program *NZFl*_Vib 1: What the Slab Output Represents

This article is written by Yadav Khwaounjoo, the developer of the program *NZFl_Vib 1*, with additional material by G Charles Clifton, HERA Structural Engineer.

General

The program *NZFl_Vib 1* is a program for the analysis of floor vibration of floor systems comprising steel beams supporting a concrete slab. It is a spreadsheet based program and comes with a comprehensive users manual, HERA Report R4-112 [12].

That manual contains seven worked design examples. Fig. 73.9 shows the output from Design Example No. 2, which comprises a concrete slab on *Hibond* decking, supported on a network of secondary and primary beams.

The program implements two published procedures for floor vibration assessment. The first of these is the AISC Design Guide Series 11 [13] and the second is the Applied Technology Council Design Guide 1 [14].

The reason for using both procedures is because neither covers the full range of floor systems built in New Zealand for which vibration checks are required. The first and most widely used procedure [13] covers slabs supported on beams, where the response of the slab and the response of the beams will contribute to the overall vibration response.

The second procedure [14] covers the vibration of the slab when spanning between stiff supports, such as walls, where the vibration response of the floor is solely dependent on the slab.

A brief overview of the scope and coverage of these two floor vibration design procedures is given in *DCB* No. 56, pp. 25-27.

Because *NZFl_Vib 1* incorporates and presents results from both procedures and these procedures treat the vibration response of the slab differently, the input and output needs to be appropriate for the support conditions (eg. slab on walls or slab on beams) and needs to be read and interpreted appropriately. Guidance on this is now given.

Applying The Results from *NZFl_Vib1*

Fig. 73.9 on page 21 shows an example of the output. As previously stated, this is for Design Example 2 from the User's Manual [12], in which all the dimensions of the floor system are shown.

The output given in the *Table E: Final Result* of Fig. 73.9 is for the slab, beam/joist, girder 1, girder 2 (none in this instance) and combined floor.

The first of these, the slab output, is for the slab on rigid supports as determined by [13]. This would apply to the slab close to supporting columns, for example. This value does not consider combined action effects of the slab with the supporting system and should be considered in isolation from the rest of the items in that table.

The rest of these items are determined using [13] and relate to each floor system component in turn, followed by combined actions.

The graphical output is that for the combined floor system.

Fig. 73.9 shows a design example where the slab spans only 2.75m onto supporting beams (secondary beams), which span onto supporting girders (primary beams), which are supported by columns. Fig. 73.10 then shows the output for the same slab thickness but for the slab span increased to 5m between secondary beams. The example is not a realistic practical solution, because in this instance the size of the beams would need to be increased, as would the slab thickness, however these have been deliberately kept the same to illustrate the affects of changing only the slab span.

E. FINAL RESULT: **FOR SLAB SPAN OF 5 m**

Member	Fundamental frequency (Hz)	Peak Acceleration (% g)
Slab	***4.54***	***0.70***
Beam/Joist	***6.01***	***0.19***
Girder 1	***12.24***	***0.05***
Girder 2		
Combined Floor	**5.462**	**0.257**
Limiting Combined Floor Acceleration (% g) = 0.50		

Fig. 73.10
Result for the Slab Span Doubled to 5m

The effect on the slab alone is considerable; the peak acceleration for the slab alone close to rigid supports is now critical; which would be the case in practice if this system were built.

Increasing the slab span also affects the beam and girder stiffnesses and participating masses. In this case, doubling the slab span decreases the beam frequency and increases the peak acceleration, because the increase in participating mass is greater than the increase in stiffness. However, it slightly increases the girder frequency, although this does not change the calculated girder peak acceleration.

Modelling the Effect of Stiff Supports

For a slab spanning directly onto stiff supports, eg. walls, type NR into the Sec. Beam/Joist and the Pri. Beam 1 boxes. The slab response is then calculated in accordance with [14], which is the appropriate procedure in this instance (ie. with no supporting beams).

However, when the slab spans onto a mixture of stiff supports and supporting beams, then the details of these beams must be entered and the calculation is undertaken to [13]. Fig. 73.11 shows the results for a 150 mm thick slab on Hi-bond, spanning from a wall onto a *530UB82* supporting beam - ie. one end of the deck span is supported by the wall, the other runs over the supporting beam.

E. FINAL RESULT: **FOR SLAB SPANNING 5m FROM WALL ONTO A SUPPORTING BEAM**

Member	Fundamental frequency (Hz)	Peak Acceleration (% g)
Slab	***7.50***	***0.48***
Beam/Joist	***8.09***	***0.05***
Girder 1		
Girder 2		
Combined Floor	**5.632**	**0.241**
Limiting Combined Floor Acceleration (% g) = 0.50		

Fig. 73.11
Result for 5 m Span Slab on Hi-Bond, 150 mm Thick, Spanning from Wall to over a Supporting Beam

In this instance, what the results show is:

- the response of the slab alone close to the supporting wall (which in this instance has the highest peak acceleration value)
- the response of the supporting beam, and
- the combined response of the slab and supporting beam, where the slab contribution is that away from the supporting wall.

These responses occur at different regions of the floor and, for a complying floor system, all peak accelerations must be below the limiting floor acceleration criteria. As previously mentioned, the upper limit line shown in the graphical output of *NZFl_Vib 1* gives the limiting floor acceleration as a function of frequency that each component of the floor system must meet. The combined floor response is plotted on that graph. However, the slab response is not shown there and must be checked seperately. In the case of the slab shown in Fig. 73.11, the limiting acceleration (for this type of occupancy) for the slab with a frequency of 7.5 Hz is 0.5% g, so the slab is satisfactory.

Design of Multi-Storey Steel Buildings for Satisfactory In-Service Wind-Induced Acceleration Response: Update on *DCB* No. 66

This update is written by G Charles Clifton, HERA Structural Engineer.

Background

DCB No. 66, pp. 1-10, February 2002 contains a procedure for the design of multi-storey steel buildings for satisfactory in-service wind induced acceleration response. This procedure is based on a preliminary design technique developed by Cenek et. al. [15] and the Joint Australian/New Zealand wind loadings standard, AS/NZS 1170.2:2002 [16].

The *DCB* paper also gives the background to this topic, a commentary to the procedure and a worked design example.

Between then and now, the following developments have occurred which call for an update on the *DCB* No. 66 procedure:

- AS/NZS 1170.2 [16] has been published (it was in draft when that paper was written)
- Some errors and ambiguities have been noted in that paper
- Clark Hyland has undertaken further research into the comparison of design provisions for calculating the peak acceleration at the top of the building and found a more accurate expression for this. Details are given in session 1 of [17] and briefly discussed on the next page.

These aspects are all covered in the update details, which are as given below.

Details of Update

(1) The building mass to be used, m_o, should be expressed in kg/m height and is the average building mass/unit height over the top one-third of the building, when applying the provisions of *DCB* No. 66 or Appendix G of AS/NZS 1170.2 [16]

(2) In the example of application given on pages 7,8 of *DCB* No. 66, m_o is also expressed as tonnes/m height when making the level 1 assessment. For the equation used in that assessment, $h^{1.3}/ m_o > 1.6$, this change from kilogrammes to tonnes is correct. Equation G1 from [16] is the same equation, but with the 1.6 changed to 0.0016 to allow m_o in kg/m height to be used.

(3) The design acceleration and the acceleration limit must be in the same units. The a_{max} from equation 66.2, *DCB* No. 66, is calculated in m/s/s and converted to milli-g through the (1000/9.81) factor at the end of the equation. The equation given in Fig. 66.3 is equation 66.7 without the (1000/9.81) adjustment factor. Its units are therefore m/s/s, although the vertical axis of that figure is given in milli-g

(4) Also in equation 66.7, f_o = fundamental frequency, not fundamental period as stated

(5) Note the frequency scale in Fig. 66.2 is in steps of 20, not 2!

(6) It is recommended that readers use Equation G3(1) from AS/NZS 1170.2 [16] instead of equation 66.2 from the *DCB* No. 66 paper to calculate the (cross-wind) design maximum acceleration at the top of the building. This is the critical wind acceleration for design. This recommendation comes from Hyland's comparison study, which shows Equation G3(1) giving the best match in cross-wind acceleration with experiment of all the procedures available. This equation gives the acceleration in m/s/s directly, which is then compared with a_{limit} from equation 66.7, taking account of the units. Equation G3(1) is more complex to use than the Cenek et. al. equation 66.2, but gives more accurate answers. See details from Session 1 of [17] for more information. When using Equation G3(1), be careful in determining the value of C_{fs} to use from Clause 6.3.2.3 of AS/ZNS 1170.2 and make sure the appropriate option for the building's shape and proportions is being used.

SPM0103: Potential Problem and Solution

This short article is written by G Charles Clifton, HERA Structural Engineer.

The second edition of the *Slab Panel Method* has been published in *DCB* No. 71. It comes with a computer program, SPM0103, which allows designers to rapidly design the slab panels.

The program is available as a single executable file entitled SPM0103. It is accompanied by a sample calculation file, being the design example presented in section 9, pages 12-14 of *DCB* No. 71.

To date, no one to whom the program has been sent has reported any problems with its installation. One firm, however, has reported a problem with trying to run new jobs. What happens is that they can input the data on the input screen, however the program does not perform the calculations when the *calculate* button is pressed, displaying instead a run time error.

The SPM0103 version of the program is written as a design tool, however the coding for the program actually contains code for the design version and for a research version, which was developed for HERA research use and allows more variables to be altered than are required for design. In the design version, the coding for the research version is supposed to be switched off. However, what has happened, in this instance, is that modules of the research version have been activated by the design example and have tried to run in conjunction with modules of the design version, causing incompatibility problems and a run-time error to develop. Why we don't know.

However, if a user encounters this problem, the solution is straight forward. It is to overwrite the data, in the design example supplied, with the design data for the case under consideration, do the calculation, then save as under a new filename. That approach has worked in the two instances where this problem has occurred.

If any users have encountered this problem, please advise Charles Clifton, email address: structural@hera.org.nz

References

1. AS/NZS 2312:2002, Guide to the Protection of Structural Steel Against Atmospheric Corrosion by the Use of Protective Coatings; Standards New Zealand, Wellington.

2. Hyland C; Structural Steelwork Connections Guide Incorporating Amendment No. 1; HERA, Manukau City, New Zealand, 1999/2001, HERA Report R4-100.

3. Hyland, C; Structural Steelwork Estimating Guide; HERA, Manukau City, 1998, HERA Report R4-96.

4. Design Capacity Tables for Structural Steel, Second Edition, Volume 2: Hollow Sections, Australian Institute of Steel Construction, Sydney, Australia, 1999.

5. NZS 3404: 1997, plus Amendment No. 1: 2001, Steel Structures Standard; Standards New Zealand, Wellington, New Zealand.

6. AS 1418.1 : 2002 (Plus Amendment No1 : 1997), Cranes (Including Hoists and Winches) Part 1 : General Requirements; Standards Australia, Sydney, Australia.

7. Roark, RJ and Young, W C; Formulas for Stress and Strain (Sixth Edition); McGraw-Hill International, Tokyo, Japan, (1975).

8. AS 1163:1991, Structural Steel Hollow Sections; Standards Australia, Sydney, Australia.

9. Clifton, GC; Structural Steelwork Limit State Design Guides Volume 1; HERA, Manukau City, 1994, HERA Report R4-80.

10. Clifton, GC; Restraint Classifications for Beam Member Moment Capacity Determination to NZS 3404:1997; HERA, Manukau City, 1997, HERA Report R4-92.

11. NZS 4203:1992, General Structural Design and Design Loadings for Buildings; Standards New Zealand, Wellington, New Zealand.

12. Khwaounjoo, YR; Report and User's Manual for *NZFl*_Vib 1 Program (Program for the Analysis of Floor Vibration); HERA, Manukau City, New Zealand, 2002, HERA Report R4-112.

13. Murray, TM et. al.; Floor Vibration due to Human Activity; American Institute of Steel Construction, 1997, Steel Design Guide Series 11.

14. Allen, DE et. al.; Mnimising Floor Vibration; Applied Technology Council, Redwood City, USA, 1999, ATC Design Guide; 1.

15. Cenek, P et. al.; Designing for Dynamic Serviceability Under Wind Loading; Recent Advances in Wind Engineering, Volume 1, TF Sun (Editor); Pergamon Press, 1989, pp. 399-406.

16. AS/NZS 1170.2: 2002, Structural Design Actions Part 2; Wind Actions; Standards New Zealand, Wellington.

17. Hyland, C et.al; Notes Prepared for the Steel Structures Seminar 2003; HERA, Manukau City, New Zealand, 2003, HERA Report R4-119.