(ii) 50 + years loads – k_1 = 0.57

$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f'_b\, Z$ <3.2.1.1>

$= 0.90 \times 0.57 \times 1.0 \times 1.0 \times 1.0 \times 0.695 \times 37.6 \times 3780{\times}10^3$ Nmm

$= 50.7$ kNm $> M^* = 26.6$ kNm ✓ <3.2.1.1>

or load ratio $\dfrac{M^*}{M_d} = \dfrac{26.6}{50.7} = 0.525$ (*equation 5.44*)

Both gravity load cases are satisfied.

Uplift load cases

For the case of wind uplift, the load direction is reversed, so the top edge will be in compression. The critical edge therefore has lateral restraint provided by each purlin at 1200 mm centres and $k_{12} = 0.695$ as above.

For wind uplift $k_1 = 1.0$ and

$M_d = \phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f'_b\, Z$ <3.2.1.1>

$= 0.90 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 0.695 \times 37.6 \times 3780{\times}10^3$ Nmm

$= 88.9$ kNm $> M^* = 50.9$ kNm ✓ <3.2.1.1>

or load ratio $\dfrac{M^*}{M_d} = \dfrac{50.9}{88.9} = 0.573$ (*equation 5.44*)

Under wind uplift, the load ratio for the portal is around 57% at the knee.

Load case	k_1	M^* (kNm)	M_d (kNm)	$\dfrac{M^*}{M_d}$
Construction loads $1.2G + 1.5Q$	0.94	−53.0	83.6	0.634
50+ years loads $1.2G + 1.5\psi_\ell Q$	0.57	−26.6	50.7	0.525
Wind uplift $0.9G + W_u$	1.0	50.9	88.9	0.573

BD10. Check Shear Capacity

Shear at critical section – 12.8 kN including the load within 1.5 d of the face of the support. (This is a conservative assumption. If there is a problem with the shear capacity, the load applied within 900 mm of each knee can be ignored in the calculation of V^*.)

For this LVL product, the shear strength supplied by the manufacturer = 4.6 MPa.

The rafter has previously been classed as a primary member in a normal structure – capacity factor (for primary member, normal structure) is 0.9

$\phi = 0.9$ <Table 2.1>

Modification factors remain unchanged from the bending calculations

factor	**value**	**Reference**
k_1	0.94	<Table 2.3>
k_4	1.0	<8.4.3>
k_6	1.0	<2.4.3>

$A_s = 0.667\, b\, d = 0.667 \times 63 \times 600 = 25.2{\times}10^3$ mm^2 *equation 5.21*

with $g_{shape} = 0.667$

$V_d = \phi\, k_1\, k_4\, k_6 f'_s\, A_s$ *equation 5.20*

$V_d = 0.90 \times k_1 \times 1.0 \times 1.0 \times 4.6 \times 25.2{\times}10^3$ N

$V_d = 104\, k_1$ kN

Load case	k_1	V^* (kN)	V_d (kN)	$\frac{V^*}{V_d}$
Construction loads $1.2G + 1.5Q$	0.94	12.8	97.8	0.131
50+ years loads $1.2G + 1.5\psi_\ell Q$	0.57	6.4	59.3	0.108
Wind uplift $0.9G + W_u$	1.0	−12.3	104	0.118

For example, for the first load case,

$V^* = 12.8$ kN, therefore $\phi V \geq V^*$ ✓ *equation 5.19*

The load ratio for shear effects $\frac{V^*}{V_d} = \frac{12.8}{97.8} = 0.131$ *equation 5.45*

The beam is only used to 13% of its shear capacity. ✓

The beam is safe for all load cases for shear, even with a conservative estimate of the shear forces to be resisted. Had the load applied within 1.5*d* of the support been ignored in the calculation of V^*, then the load ratios would have been even smaller (safer).

BD11. Check Bearing Capacity

Here there is no problem with bearing. The reaction to the rafter is via the connectors in the knee joints rather than by bearing onto the underside of the member.

The design on the basis of serviceability produced a deep rafter 600 × 63 LVL. The calculation of the bending strength at the knee of this member showed that it had ample capacity in bending provided the section could be laterally and torsionally restrained.

- For the case of compression on the inside edge of the LVL rafter (gravity loads produce this at the knee), the restraint had to be provided by fly-braces at every purlin in the vicinity of the knee joint.
- At mid-span of the rafter, the compression on the inside edge will be produced by net uplift forces. The appropriate spacing for fly-bracing at mid-span has still to be determined.

The rafter has ample capacity for the shear effects.

Note that the MoE used in these calculations is the average value. Hence the deflections calculated are the average deflections, and may be exceeded in some members. If it was vital to the performance of the building that clearance to the services underneath be maintained, a 5th percentile value of MoE should be used. Table 2.10 can be used to estimate these values. In the case of LVL material, there will only be a small change in using the 5th percentile MoE.

A deflection critical situation where a more reliable value for the MoE is required is illustrated in Example 5.5. The serviceability guidelines in <Appendix B> help designers to recognise "critical" elements. These include beams where absolute clearance is required for deformations to ensure that the element serves its intended function and does not cause loss of function or damage to other adjoining/related elements in the structure.

✍ **Example 5.5 Serviceability design of a floor support beam .**

The glulam beam designed in Example 2.8 is being used to span an opening in which a large sliding glass door and window panel unit will be constructed. If the beam sags or deflects excessively, the glass door will become inoperable and the fixed panes in the panel unit may crack or break. The door manufacturer has specified an absolute clearance of 15 mm from the top of the glass door/window reveal to the underside of the beam. The beam is to be redesigned to meet this “critical” serviceability limit state.

From Example 2.8:

- Design span = 4000 mm
- “Comfort” limit = span/250 for short-term imposed load alone. This ensures that people walking across the floor won’t “feel” movement under foot. This was satisfied by a 330 × 65 mm GL12 beam as shown in Example 2.8
- GL12 has average short-term MoE of 11 500 MPa.

Additional information

- The beam will be supplied with a camber of span/300.
- The total deflection under any load case (long or short-term imposed load) should allow adequate clearance over the top of the window.
- Serviceability loads on the beam:

permanent load (G)	=	3.0 kN/m	(est > 1 year)
shorter-term serviceability imposed load (Q_s)	=	8.0 kN/m	(est < 1 day)
longer-term serviceability imposed load (Q_l)	=	4.6 kN/m	(est > 1 year)

✍ **Solution**

Only ***Step BD1*** to ***Step BD5*** will be documented here. Strength checks (***Step BD6*** to ***Step BD11***) will also need to be followed later.

BD1 Deflection limits and load combinations

Exceeding the deflection limit in this problem may cause some damage to elements in the building. Although the damage probably will not endanger life, taking remedial measures after such a problem has surfaced may prove very costly. Rather than designing to give deflections right on the limit, a factor of safety will be used so that if the design scenario is just a little worse than that assumed, there will still be clearance to the partition.

There are no defined safety factors for the serviceability limit state. However, it is reasonable to use a factor between 1.2 and 1.5. The decision is arbitrary.

Allowing a factor of safety of 1.25 on the clearance below the depth of the deflected beam, the limiting beam deflection is 15 / 1.25 = 12 mm

The camber also needs to be calculated, as it affects the net total displacement of the beam. Over the 4000 mm span, the camber is

4000 / 300 = 13.3 mm

The total deflection minus the camber must be < clearance/factor of safety

$$\delta_{total} - 13.2 \leq 12 \text{ mm}$$

or $$\delta_{total} \leq 25.3 \text{ mm}$$

This limit is associated with the two total load combinations:

- permanent load + longer-term imposed load
- permanent load + shorter-term imposed load

The comfort limit under short term imposed actions only is span/250

= 4000/250 = 16 mm

BD2 Selection of design Modulus of Elasticity

In this case an "absolute" limit for deflection must be maintained for the glass door and window panel in the building to remain functional. It is therefore important that we obtain as reliable a prediction of the maximum deflection as possible. As discussed in Section 2.3.1, the values for Modulus of Elasticity specified in <Table 7.1> are average values, which are representative of the total population of the grade of timber being considered.

There is a 50% probability that a beam complying with the specification will have a value of MoE lower than this average. This means that there is a significant chance or risk that the actual beam deflections will be higher than those predicted using the average MoE in the design. In this case, there are serious consequences if the limiting deflection is exceeded.

In order to reduce the "risk" of damage to the glass doors and windows to an acceptable level, the Serviceability Guidelines in <Appendix B> recommend that a "lower bound" value of MoE (e.g. the 5th percentile value) should be used.

Table 2.10 of this Handbook can be used to make an estimate of the 5th percentile value for the MoE of GL12, by assuming a design value that is 75% that of the average, so that the value for E now becomes:

$$E_{0.05} = 0.75 \times 11\,500 = 8630 \text{ MPa}$$

As before, the comfort limit does not have the same serious consequences of exceedance, so E = 11 500 will be used for this load case.

BD3 Load combinations

<u>longer-term loads (permanent + imposed load)</u>

i.e. Serviceability load $= G + Q_\ell = 3.0 + 4.6 = 7.6$ kN/m <AS/NZS 1170.0 4.3>

duration: > 1 year Limit = 25.3 mm

<u>total load with shorter-term imposed loads (permanent + shorter-term imposed)</u>

i.e. Serviceability load $= G + Q_s = 3.0 + 8.0 = 11.0$ kN/m <AS/NZS 1170.0 4.3>

duration: short-term assume <1 day. (Floor short-term serviceability loads are generally from crowd events which rarely last longer than one day.) Limit = 25.3 mm

BD4 Selection of critical serviceability load case and calculation of I

All of the loadings in this problem are uniformly distributed loads over a simply supported span. Hence the deflection function $f_1(L) = \dfrac{5\,L^4}{384}$

Equation 5.4 reduces to: $I \geq \sum_{i\,=\,\text{each distr load}} \left[\dfrac{5\,L^4}{384}\,\dfrac{j_2\,w_i}{E\,\delta_{\text{lim}}}\right]$

Load combination	Loads (N/mm)	j_2 <T 2.4>	$\sum(j_2\,w_i)$ (N/mm)	E (MPa)	δ_{lim} (mm)	I (mm^4)
total longer-term loads (permanent + imposed)	G =3.0 Q_ℓ = 4.6	2.0 2.0	2.0 × 3.0 + 2.0 × 4.6 = 15.2	8630	25.3	232×10^6
total shorter-term loads (permanent + imposed)	G = 3.0 Q_s = 8.0	2.0 1.0	2.0 × 3.0 + 1.0 × 8.0 = 14.0	8630	25.3	214×10^6
Shorter-term imposed load	Q_s = 8.0	1.0	8.0 × 1.0 = 8.0	11 500	16	145×10^6

The maximum value of I is given by the longer-term load case. It will be used as the critical load for the design of the deflection limited beam.

$$I \geq 232\times10^6 \text{ mm}^4$$

BD5 *Selection of cross-section*

Appendix A gives section properties for standard sized glulam beams. Table A1 shows that a 360×65 GL12 beam ($I = 237\times10^6$ mm^4) or a 330×85 GL12 beam ($I = 237\times10^6$ mm^4) would both be suitable.

Of these the 360×65 has a smaller cross-sectional area and so would be lighter and probably cheaper. Unless lateral torsional buckling limits the behaviour, use the 360 × 65.

This represents an increase in the size required for the serviceability limit state. Example 2.8 which had a deflection limit of span/250 under short-term imposed load, appropriate to maintain comfort, only required a 330 × 65 GL12 beam. Note that the strength limit state still has to be checked. (See Example 5.6).

To illustrate the potential problem which could occur if the average MoE had been used, the required depth for a beam with the average MoE of 11 500 MPa would be 317 mm. A 330 × 65 mm GL12 would have been selected.

However, if the beam delivered to the site had an actual MoE of 10 000 MPa or lower (which is still within grade), the absolute deflection criteria for the serviceability limit state would have been violated. In probability terms, the chance of this happening in the life of the building would be about 20%.

5.7.2 Design for the strength limit state

All members must have adequate performance to meet the strength limit state loads. While there may be some members that have no deflection limits, so require no checking for serviceability, every beam must be checked for its performance under strength limit state loads. Some designers size all members for strength and only perform a serviceability check on those for which there are deflection limits. In some cases, where the serviceability limits are not onerous, the strength limit state may govern the design. Also, beams with shorter spans and higher load levels are often limited by the strength limit state, and so for these configurations it is efficient to size them to satisfy strength requirements and then check serviceability.

If a section is sized for the serviceability limit state and the strength checks show that the strength limit state performance of the selected member was not adequate, then the cross-section needs to be increased. Where the strength of the member was only a little less than the factored strength load, the next sized member up may be able to provide an adequate level of performance. If there was a larger shortfall, the member may have to be redesigned for the strength limit state.

Design for the strength limit state should be undertaken first in the following cases:

- Short span beams, where the limit is provided by bending capacity, or for very short span beams that are limited by shear.
- Long span beams with sparse lateral restraint, where the limiting behaviour may be provided by bending capacity with poor lateral torsional stability.
- Beams that normally have downwards loads and are laterally restrained on the top face, but under some loading conditions may have upward loads. In these cases, the restraints are on the tension face and less effective in preventing lateral torsional buckling.

- Beams that have no serviceability limits or for which the serviceability limits are very generous.

Design summary—Bending members selected for strength

Each of the steps in this design summary is headed ***Step BU***. This stands for beam design for ultimate (or strength) limit state.

Step BU1 Examine each loading combination and identify the shortest term load in that combination. For the shortest term load in each combination, determine the appropriate duration and hence duration of load factor k_1 from <2.4.1.1> or <Table 2.3>.

Step BU2 Calculate $D_L = \dfrac{w^*}{k_1}$ or $D_L = \dfrac{M^*}{k_1}$ for each strength limit state load combination. The critical load case for the strength limit state is the one with the highest value of this parameter. The design will target w^* or M^* for the critical load case.

Step BU3 Select the appropriate characteristic strength of the material f'_b from manufacturer's information or from an appropriate table in AS 1720.1. An assumption must be made about the size used.

Step BU4 Determine the capacity factor (ϕ) from <Table 2.1>. This requires the classification of the structural element as either primary or secondary and classification of the structure into housing, structures other than housing, or post-disaster function structures.

Step BU5 Evaluate the relevant modification factors k_1 k_4 k_6 and k_9 for the ambient conditions and the critical load case selected in ***Step BU2***. (Where unseasoned timber is used, it is conservative to assume that $k_4 = 1.0$.) At this stage, assume that the stability factor $k_{12} = 1.0$. Use *equation 5.18* to calculate the required elastic section modulus Z. Choose an appropriate member using a table of cross-section properties such as that in Appendix A of this Handbook.

Step BU6 Evaluate the actual bending strength f'_b for the selected member.

Examine lateral restraint on the member. Either:

- Use Tables 5.4 to 5.7 as appropriate to find the spacing of restraints required to ensure that $k_{12} = 1.0$ as assumed. Design appropriate restraints for these positions; or
- Use the closest practical positioning of restraint and evaluate the slenderness coefficient (S_1) for that configuration of restraints. This can be used to give the stability factor (k_{12}).

Step BU7 Calculate the bending capacity of the selected member using the appropriate value for f'_b and k_{12} in *equation 5.6*. Compare this with the critical load combination using *equation 5.5*. Other load combinations may have less lateral restraint on the member (for example, wind uplift in which case the purlins restrain the tension edge not the compression edge over most of the length of the member). Hence, all load combinations should also be checked using *equation 5.6*.

Step BU8 Check the serviceability load combinations for which deflection limits δ_{lim} have been assigned using the appropriate j_2 value and expression for deflection.

Step BU9 Check the shear capacity of the beam using *equations 5.19* and *equation 5.20*.

Step BU10 Check the bearing capacity of the beam using *equations 5.22 - 5.23*.

The floor beam (bearer) in Example 5.5 was designed to meet serviceability limit states for deflection using ***Step BD1*** to ***Step BD5***. Under normal circumstances, this would have been followed by ***Step BD6*** to ***Step BD11*** in which the beam is checked for the strength limit state.

As an example of strength design, the problem detailed in Example 5.5 will be revisited:

Example 5.6 Design of a floor beam for the strength limit state

Design a GL12 floor beam for the strength limit state, using the methodology explained in Section 5.7.2.

The beam has a clear span of 3900 mm with 100 mm bearing at each end, giving a design span of 4000 mm. Joists are attached to the top of the beam with nailed straps. The joist spacing is 450 mm.

The beam will support a mezzanine office floor for a warehouse in Dandenong, Victoria and has the following unfactored (nominal) loadings determined from AS/NZS 1170.1:

- Permanent load 3.0 kN/m (estimated/50+ years)
- Imposed load 11.4 kN/m (estimated/5 day loading – crowd)

The *radiata pine* GL12 material can be sourced from a local supplier who manufactures to AS/NZS 1328.

The following information about serviceability is provided.
A deflection limit of span/250 is required for total permanent and imposed loads for both shorter-term and longer-term serviceability load combinations.
Shorter-term serviceability imposed load only $= \psi_s Q = 0.7 \times 11.4 = 8.0$ kN/m

Longer-term serviceability imposed load $\psi_\ell Q = 0.4 \times 11.4 = 4.6$ kN/m
Permanent load (serviceability) 3.0 kN/m

permanent + longer-term serviceability imposed load total defl limit = 25.3 mm
permanent + shorter-term serviceability imposed load total defl limit = 25.3 mm

(The total limit was determined in Example 5.5 and takes into account camber in original beam. It must be used with a conservative estimate of E)

Solution

Here ***Step BU1*** to ***StepBU10*** will be presented for a strength limit state design process.

BU1 Strength limit state load combinations and duration of load factors

The nominal imposed load has been given, and now a longer-term strength limit state imposed load and a shorter-term strength limit state imposed load must be estimated. The techniques outlined in Figure 2.7 of this Handbook will be used to obtain the estimates of imposed load for different durations of loading.

Shorter-term imposed load

In this case, there are no known imposed loads. All imposed loads are estimated. The imposed load given is a shorter-term imposed load with a typical strength limit state scenario of a crowd loading superposed on a smaller 50+ years loading given by furniture and storage.

$$Q = 11.4 \text{ kN/m}$$

Longer-term imposed load

In the light of no other information, the 50+ years imposed load will be given by

$$\psi_\ell Q = 0.4 \times 11.4 = 4.6 \text{ kN/m}$$

This would correspond with a 50+ years loading given by furniture and storage in the offices.

The strength limit state load combinations for permanent and imposed load combinations are:

Load case	Calculation	Effect (kNm)
50+ years (permanent + imposed)	$1.2G + 1.5\psi_{\ell}Q = 1.2 \times 3.0 + 1.5 \times 4.6$ $= 10.4$ kN/m	20.8
5 day (permanent + imposed)	$1.2G + 1.5Q = 1.2 \times 3.0 + 1.5 \times 11.4$ $= 20.7$ kN/m	41.4

load cases from <AS/NZS 1170.0 4.2.2> and moment from $M^* = \dfrac{w^* L^2}{8}$

BU2 Selection of critical load case for strength

Load case	Total duration (over lifetime of building)	k_1
50+ years (permanent + imposed)	50 + years	0.57
5 day (permanent + imposed)	short duration irregular load (5 days)	0.94

Load combination	Design moment M^* (kNm)	k_1	D_L (from ***Step BD5***.)
50+ years (permanent + imposed)	20.8	0.57	36.5
5 day (permanent + imposed)	41.4	0.94	44.0

The "critical" load combination to be considered is shaded in the summary table above. Note that in this case, it corresponded to the highest load combination with $M^* = 41.4$ kNm and $w^* = 20.7$ kNm.

The equation for D_L here was $D_L = \dfrac{M^*}{k_1}$

From analysis of the load combination, the following strength limit state load effects must also be considered:

Shear at critical section: 40.4 kN taking load over the clear span of the member (3.9 m, conservative)

Bearing (reaction): 42.4 kN taking the load over the full design length of the member (4.1 m)

(Shear and bearing both taken from $N^* = \dfrac{w^* L}{2}$ but using the spans indicated above

Both are used in later calculations)

BU3 Bending strength f'_b

GL12 has the following bending strength

		$f'_b = 25$ MPa	(independent of size)	<Table 7.1>
	shear strength	$f'_s = 4.2$ MPa		<Table 7.1>
radiata pine SD6	bearing strength	$f'_p = 10$ MPa		<Table H2.2>

Bearing and shear strengths are used in later steps.

BU4 Capacity factor ϕ

For the capacity factor (ϕ), the appropriate strength of the GL12 product that conforms to AS/NZS 1328 is used.

The floor beams are principal structural elements in a warehouse, so the members can be classed as primary elements in a normal structure.

For glulam to AS/NZS 1328 as primary elements in normal structures

$\phi = 0.85$ <Table 2.1>

BU5 Strength modification factors for bending and required Z

Duration of load k_1

The critical load case for the strength limit state was given by a 5 day load for which $k_1 = 0.94$ <Table 2.3>

Partial seasoning k_4

The glulam beam is a seasoned timber product and will be used indoors. No partial seasoning is expected $k_4 = 1.0$ <2.4.2.3>

Temperature k_6

Dandenong (Vic) is well South of 25° S $k_6 = 1.0$ <2.4.3>

Strength sharing k_9

The beam will be a glulam beam $k_9 = 1.0$ <7.4.3>

(Even if the member had been sawn timber, the floor support beam will not have any closely spaced members to share strength with so $n_{mem} = n_{com} = 1$, so $k_9 = 1.0$ anyway.)

Stability factor k_{12}

It is assumed that $k_{12} = 1.0$ **Must check**

Material constant ρ_b

For glulam beams, ρ_b is found from the bending strength and stiffness and the nature of load.

For a conservative estimate, and one that is applicable to all load combinations, assume that $r = 0.25$. $\rho_b = 0.84$ <Table 7.2(A)>

Note that for the critical load case, $r = \dfrac{1.5\times 11.4}{1.2\times 3.0 + 1.5\times 11.4} = 0.83$

and this gives $\rho_b = 0.79$

This value would not be conservative for all load combinations, so the design and checking of all load combinations will use $\rho_b = 0.84$.

Using AS 1720.1, the following modification factors have been selected:

Factor	Value	Reference
k_1	0.94	<Table 2.1>
k_4	1.0	<2.4.2.3>
k_6	1.0	<2.4.3>
k_9	1.0	<7.4.3> ($k_9 = 1.0$ for glulam)
k_{12}	1.0	**Assumed**
ϕ	0.85	<Table 2.1>
ρ_b	0.84	<Table 7.2(A)>

$$Z_{min} \geq \frac{M^*}{\phi\, k_1\, k_4\, k_6\, k_9\, k_{12}\, f'_b} \text{ mm}^3 \qquad \textit{equation 5.18}$$

$$Z_{min} \frac{41.4\times10^6}{0.85\times0.94\times1.0\times1.0\times1.0\times1.0\times25.0} = 2070\times10^3 \text{ mm}^3$$

Must have a Z_x greater than the 2070×10^3 mm^3.

Use a 395 × 85 mm cross-section with design values of

d = 391 mm

b = 82 mm ⇒ $Z_x = 2090\times10^3$ mm^3. (Appendix A)

However, it is now necessary to check that there is adequate lateral restraint to ensure that $k_{12} = 1.0$ and that the capacity is still satisfactory with the appropriate size factor.

BU6 Actual value of f'_b and k_{12}

There is no modification to f'_b for size of glulam products.

f'_b of GL12 = 25 MPa for all sizes. <Table 7.1>

The floor joists are fastened to the top or compression edge. This is the critical edge. To quickly check the assumption that k_{12} = 1.0, Table 5.4 of this Handbook could be used. A 395 × 85 mm cross-section is not included in Table 5.4, but a 395 × 80 GL12 glulam beam (which is more slender as it has a higher aspect ratio) requires restraints at closer than 1380 mm centres to give full lateral restraint. The restraint at 450 mm centres provided by the joists will be adequate for continuous lateral restraint and will give k_{12} = 1.0.

Alternatively, we could have used <3.2.4> to determine:

$$S_1 = 1.25\,\frac{d}{b}\left(\frac{L_{ay}}{d}\right)^{0.5} = 1.25\times\left(\frac{391}{82}\right)\left(\frac{450}{391}\right)^{0.5} = 6.39 \qquad \text{(comp edge) <3.2.3.2(a)>}$$

and $\rho_b = 0.84$ <Table 7.2(A)>

⇒ $\rho_b\, S_1 = 0.84 \times 6.39 = 5.37 < 10$ => $k_{12} = 1.0$ <3.2.4(a)>

BU7 Check of strength limit state flexural capacity for all load cases

5 day load combination (critical load case)

$M_d = 0.85 \times 0.94 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 25 \times 2090\times10^3$ Nmm

M_d = 41.7 kNm

M^* = 41.1 kNm , therefore $M_d \geq M^*$ ✓ *equation 5.5*

The load ratio for bending strength = $\dfrac{M^*}{M_d} = \dfrac{41.4}{41.7} = 0.993, < 1.0$ ✓

50 + year load combination

This is a gravity load case as well, so k_{12} will be the same value as the other gravity load case. (If the net load was in uplift k_{12} would have to be re-evaluated.)

$M_d = 0.85 \times 0.57 \times 1.0 \times 1.0 \times 1.0 \times 1.0 \times 25 \times 2090\times10^3$ Nmm

M_d = 25.3 kNm

M^* = 20.9 kNm , therefore $M_d \geq M^*$ ✓ *equation 5.5*

The load ratio for bending strength = $\dfrac{M^*}{M_d} = \dfrac{20.9}{25.3} = 0.826, < 1.0$ ✓

A 395 × 85 mm GL12 beam with design dimensions 391 × 82 is satisfactory for strength

f'_b assumption **checked** ✓

k_{12} assumption **checked** ✓

