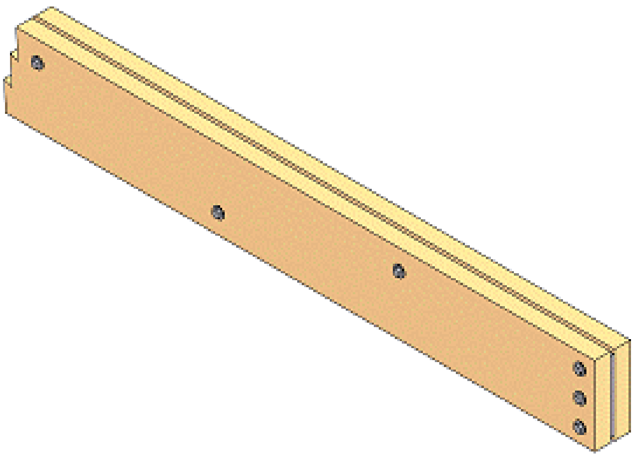
# GD9 How to design a bolted steel flitch beam



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Cover photographs are from a TRADA research project which investigated steel flitch beams fabricated with shot-fired dowels.

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# GD 9 How to design a bolted steel flitch beam

Flitch beams can be used for lintels, attic beams and other situations where solid timber beams are not strong enough or stiff enough to support design loads adequately. A bolted steel flitch beam is a vertically laminated beam consisting of timber stringers and steel plates bolted together.

This Guidance Document provides a design for a flitch beam which consists of two identical timber members with one steel plate between them. Eurocode 5 does not provide a specific design method for flitch beams, but the method given here is based on the principles given in Eurocode 0, Eurocode 3 and Eurocode 5. It assumes that the loads and reactions will be applied equally to both of the timber members and that the steel plate runs the full length of the beam.

A design example to BS 5268, based on this document, can be found in TRADA Technology's Timber frame housing: UK Structural Recommendations. TRADA's flitch beam design software (free to TRADA members in the 'lite' version) is recommended for regular designers of these components to BS 5268.

## Introduction

The normal design method for flitch beams is based on the assumption that the three members act as a composite beam, in which all three move together and are subject to the same vertical loads at any point along their length, which they share between them.

To ensure that the loads are applied to the timber rather than the steel, the depth of the steel plate should be less than that of the timber stringers. The difference in the depth of the steel plate and the stringers should be sufficient to cater for any small shrinkage which may occur in the timber and any lack of straightness in the timber or steel. A total height difference of 25 mm is common. Steel plate thicknesses from 8 mm to 20 mm are commonly used.

The timber is normally solid softwood or hardwood. However there is no reason why a structural timber composite such as laminated veneer lumber (LVL), parallel strand lumber (PSL) or laminated strand lumber (LSL) should not be used instead, except that characteristic values for its mechanical properties are not always available.

## Load distribution

To calculate the distribution of the loads between the steel and timber members, it has traditionally been assumed that the deflection of each member is due entirely to bending, ie shear deflection is ignored.

If $k_{timber}$ is the proportion of an applied load supported by the timber and $k_{steel}$ is the proportion of an applied load supported by the steel and shear deflection is ignored, it can be shown that for any point or distributed load the proportion of the loads taken by the timber members is:

$$k_{timber} = \frac{E_t I_t}{E_t I_t + E_s I_s} \quad (1)$$

and the proportion of the loads taken by the steel is:

$$k_{steel} = \frac{E_s I_s}{E_t I_t + E_s I_s} \quad (2)$$

where $E_t$ is the modulus of elasticity of the timber
$I_t$ is the second moment of area of the timber
$E_s$ is the modulus of elasticity of the steel
$I_s$ is the second moment of area of the steel

This method for calculating the load distribution between the timber and steel members was shown to be reasonably accurate by experiments carried out at the University of Bath in 2002 (Ref: Alam).

## Timber properties

Because the properties of timber members vary, even when they are of the same structural grade, timber design codes provide two values of the elastic modulus for each grade: $E_{50}$ the mean value, and $E_0$ the 5 percentile characteristic value which is used as the minimum value likely to occur in practice.

For a given beam, stringers with an actual elastic modulus equal to $E_{50}$ will take a larger share of the load than stringers with an actual elastic modulus of $E_0$ because they are stiffer. This increase in load share will generally outweigh the corresponding increase in strength of the stiffer members, so failure of the timber will occur at a lower total load if the timber is stiffer.

With less stiff timber the proportion of load transferred to the steel is greater, so the load on the bolts and steel is greater.

Table 1 shows the appropriate values of $E_t$ to use.

**Table 1 Timber elastic modulus to calculate load distribution**

| Check | $E_t$ value to use and the reason | |
|---|---|---|
| Stresses in the timber | $E_{50}$ | combined with a 1.04 enhancement of the timber strength properties produces a safe result, provided that the bending stiffness of the steel is between 20% and 80% of the total bending stiffness of the beam, otherwise apply no enhancement |
| Stresses in the steel and loads on the bolts | $E_0$ | produces higher stresses in the steel and greater loads on the bolts |
| Deflection of the beam | $E_{50}$ | Eurocode 5 states that $E_{50}$ should be used to calculate deformations |

Although timber grading rules limit distortions such as cup, bow and spring, they do not eliminate them. Therefore timber used for flitch beams should be selected for straightness, and if there is any cupping or bowing in a member, it should be positioned with the convex side on the outside.

# Effects of shear deflection, joint slip and creep

Shear deflection and joint slip affect to some extent the traditional assumptions about load-sharing between the steel and the timber.

At the supports, the relative stiffness of the steel and timber members and therefore the load distribution between them is proportional to their shear stiffness, GA, rather than their bending stiffness, EI. Since the shear modulus of steel in relation to its bending modulus is about 6 times higher than that of timber, the proportion of load taken by the steel when calculated using shear stiffness is very much higher than that predicted by formula (2). Overall however, bending deflection is much more significant than shear deflection, and it has been shown that an increase of 10% in the calculated load in the steel should always cater for the effects of the shear deflection.

Slip in the joints, either from deformation within the joint or from oversize holes, tends to have the opposite effect, and increase the load taken by the timber. However, it appears that any such increase will be offset by a corresponding reduction due to shear effects, so it seems reasonable to ignore the effects of slip in this case.

Creep in the timber effectively reduces its stiffness and hence the proportion of load that it takes. It increases the loads in the steel and bolts and increases the deflection by a factor of 4% to 40% in service class 1 and by 5% to 50% in service class 2. Consequently, stresses in the timber should be checked using the initial or instantaneous value of $E_{50}$ as shown in Table 1, and the stresses in the steel and the bolt loads should be checked using the final value of $E_0$. The instantaneous and final deflections should be checked using instantaneous and final values of $E_{50}$ respectively.

In general

$$E_{fin} = E_{inst} \frac{u_{inst}}{(u_{inst} + u_{creep})} \qquad (3)$$

where $E_{fin}$ = final value of $E_0$ or $E_{50}$ as appropriate
$E_{inst}$ = instantaneous value of $E_0$ or $E_{50}$ as appropriate
$u_{inst}$ = instantaneous deflection
$u_{creep}$ = creep deflection

$$u_{inst} = \sum_{i>0} u_{instG,j} + u_{instQ,1} + \sum_{j>1} \psi_{0,j} u_{instQ,i}$$

for strength checks with $E_0$

$$u_{inst} = \sum_{i>0} u_{instG,j} + u_{instQ,1} + \sum_{j>1} \psi_{1,j} u_{instQ,i}$$

for deflection checks with $E_{50}$ when the frequent combination is required

$$u_{creep} = k_{def} \left( \sum_{i>0} u_{instG,j} + \sum_{j>0} \psi_{2,j} u_{instQ,j} \right)$$

$u_{instG,j}$ = instantaneous deflection produced by a permanent action
$u_{instQ,i}$ = instantaneous deflection produced by a variable action
$u_{instQ,1}$ = instantaneous deflection produced by the principal variable action
and the other symbols are as defined in *BS EN 1990: 2002 Eurocode 0* or in *TRADA Technology GD2*.

Note that the value of $E_{fin}$ will generally vary according to the load case.

The formula for $u_{creep}$ assumes that the proportion of load taken by the timber remains constant, whereas it actually reduces over the life-time of the structure as the steel takes up a larger proportion of the load. A close approximation to the proportion which should be used to calculate the creep deflection might be the mean of the initial and final values. On this basis, it is possible to calculate $u_{creep}$ by means of a quadratic equation or an iterative method, but it is simpler to assume that the proportion of load taken by the timber remains constant. This leads to loads in the steel and final deflections which are not more than 1% higher than they would be using a more exact approach.

Parametric studies have demonstrated that a safe and close approximation to the final value of $E_0$ or $E_{50}$ can be determined more simply as:

$$E_{fin} = E_{inst} \frac{u_{instG} + u_{instQ1}}{u_{instG} + u_{instQ1} + k_{def}\left(u_{instG} + \psi_2 u_{instQ1}\right)} \qquad (4)$$

for both ultimate and serviceability limit states.

Formula (4) rarely overestimates the load in the steel or bolts or the final deflection by more than 2% and never by more than 5%, even in service class 3.

Table 2 shows values of $k_{def}$ for solid timber and glulam taken from Eurocode 5 and Table 3 values of $\psi$ taken from the National Annex to Eurocode 0.

### Table 2 Values of $k_{def}$ for solid timber and glulam

| Service class | $k_{def}$ for solid timber and glulam |
|---|---|
| 1 | 0.6 |
| 2 | 0.8 |
| 3 | 2.0 |

### Table 3 values of $\psi$ for final values of E

| Imposed load | $\psi_0$ | $\psi_1$ | $\psi_2$ |
|---|---|---|---|
| Floors; dwellings and office areas | 0.7 | 0.5 | 0.3 |
| Storage areas | 1.0 | 0.9 | 0.8 |
| Roofs (not to be combined with snow and wind) | 0.7 | 0.0 | 0.0 |
| Snow ≤ 1000 m above sea level, and wind | 0.7 | 0.2 | 0.0 |

Modifying formulae (1) and (2) in the light of the above, we obtain new load sharing factors:

- To check the bending and shear strength of the timber, and the instantaneous deflection:

  $$k_t = \frac{E_{50} I_t}{E_{50} I_t + E_s I_s} \qquad (5)$$

  with timber strength properties enhanced by 1.04 for $0.2(EI)_{total} \leq (EI)_{steel} \leq 0.8(EI)_{total}$.

- For strength checks on the bolts and the steel plate:

  $$k_s = \frac{1.1 E_s I_s}{E_{0,fin} I_t + E_s I_s} \qquad (6)$$

  with $E_{0,fin}$ from Formula (3) or (4).

To check the final deflection:

$$k_{t,def} = \frac{E_{50,fin} I_t}{E_{50,fin} I_t + E_s I_s} \qquad (7)$$

with $E_{50,fin}$ from Formula (3) or (4).

## Load transfer

Since all the loads are applied to the upper surface of the two timber members, the proportion of these loads carried by the steel member must be transferred through the bolts.

For uniformly distributed loads, if the bolts are spaced equidistantly and we assume that the load transfer per unit length is constant, then the design load on each bolt will be:

$$F_d = \frac{k_s \sum p_{i,d}}{n} \qquad (8)$$

where $\sum p_{i,d}$ is the total design load on the beam
n is the number of bolts.

For a point load $F_d$ the load transfer from the timber to the steel will occur in the vicinity of the load, so a sufficient number of bolts should be provided to transfer a load of $k_s$ $F_d$ from the timber to the steel at that point. This also applies at each end of the beam, where the load on the steel plate has to be transferred back into the timber which bears on the supports.

In general, we may say that UDLs are catered for by bolts spaced equidistantly along the beam, and point loads and reactions by bolts positioned as close as possible to the point at which these loads are applied.

The self-weight of the timber and steel affects the stresses and deflections, and allowance for these effects should be made by converting the combined weights of the timber and steel to an additional full-length permanent UDL (or long-term UDL for structures with a design life not exceeding 10 years). Although the weight of the steel is not shared between the timber and steel, it is usually relatively small (typically 2.5% of the total load on the beam), so it is sufficiently accurate to treat the self-weight as a simple UDL.

## Bolting requirements

The strength of a bolted timber-steel-timber connection depends on the embedding strength of the timber, the bending strength of the bolt, the bearing strength of the steel, and the dimensions of its components. The formulae for calculating the strength of such a joint are given in Eurocode 5, and they take account of all these factors, except the bearing strength of the steel. This should be checked separately, but is it is unlikely to prove critical. Note that the bolt loads are perpendicular to the grain.

Formula (8) gives the load on each bolt. Hence the number of bolts required to support the UDLs for a particular load case is

$$n = \frac{k_s \sum W_{d,i}}{R_d} \qquad (9)$$

where $W_{d,i}$ is a UDL within the load case
$R_d$ is the design resistance of the connection for two shear planes

The bolts to support a partial UDL should in theory be distributed equidistantly beneath it, rather than along the full length of the beam, but if the bolting is specified in this way and the load is not symmetrically distributed about the centre of the beam, then there would be a problem if the beam were installed the wrong way round. Also to specify separate bolting patterns for separate UDLs is more complicated for the designer, the specifier and the manufacturer. So for bolting purposes only, the designer may wish to consider partial UDLs as full-length UDLs with the same load per unit length, basing the permissible bolt loads on the load duration for each case as normal.

The bolting requirements for the reaction at each support and for any other point load should be calculated separately using a load of:

$k_s F_d$

where $F_d$ is the load at that point.

The load duration for an applied point load should be used to calculate the number of bolts required to support it, not the load duration for the load case, since the duration effects on the embedment stress are local.

If a point load is positioned at a distance of half the timber depth or less from the structural support, it will not require any bolts, either at the point of application or at the reaction.

At each support, a sufficient number of bolts should be provided to transfer a force of:

$k_s R_i$

where $R_i$ is the total reaction at the support.

Alternatively the number of bolts required to support the larger of the two forces may be specified at both ends, so that the beam can be installed safely either way round. In any case a minimum of two bolts should be specified at each support to prevent twisting of the plate, unless the timber or steel is so shallow that there is room for only one.

Where the bolting pattern is not symmetrical about the mid point for any reason, it is essential to ensure that the beam is assembled and installed the right way round, by marking the plate or beam in an unambiguous manner.

# Timber strength

## Shear strength

The shear strength of the timber should be checked using a shear force of:

$k_t R_{d,max}$

where $R_{d,max}$ is the greater of the two reactions.

Formulae to calculate the reactions to a simply supported beam with a UDL or point load are given in *Formulae for beams* at the end of this document.

## Bending strength

The bending strength of the timber is checked using $k_t$. Formulae to calculate the bending moment at any point along a simply supported beam with a UDL or point load are given in *Formulae for beams*.

## Bearing strength

The bearing strength of both the beam and the bearing plates should be checked using $R_{d,max}$, the greater of the two reactions. Masonry bearings may require a spreader plate.

## Steel strength and stability

Both theory and experiment demonstrate that the bending stress in the steel can be significant, so this must be checked. It is normally assumed that as the steel plate is clamped between two timber stringers these will prevent buckling of the plate under design loads, provided that:

- the stringers are not cupped away from the top or bottom edges of the steel
- they do not cup in this way as a result of drying – hence the timber should be conditioned to its in-service moisture content before manufacture
- the nuts remain tight – hence they should be checked after 6 months in service
- each timber member is itself stable.

Design specifications should include these requirements.

The stability of the timber members is assured if $k_{crit}$ = 1 (Eurocode 5 Clause 5.2.2 This is true if:

(a) the lateral displacement of the compression edges is prevented throughout their length, by their attachment to floor decking etc, and torsional rotation is prevented at the supports

or

(b) $\sqrt{f_{m,k} / \sigma_{m,crit}} \leq 0.75$

where $f_{m,k}$ = characteristic bending strength

$$\sigma_{m,crit} = \frac{0.75 E_0 b^2}{L_{ef} h}$$

Eurocode 5, Table 6.1 gives appropriate values for the effective length, $L_{ef}$ , depending on the degree and type of restraint.

In theory the bearing strength of the steel beneath the bolts should be checked, since this is not covered by the Eurocode 5 joint design formulae, but it is unlikely to prove critical. The shear strength, which is even less likely to prove critical, may be checked using a cross section reduced by the sum of the bolt hole diameters (d + 1 mm) at the support. Provided however that the stiffness of the steel is between 20% and 80% of the total beam stiffness, its design strength in bearing and shear may be assumed adequate.

All checks on the steel should be made using $k_s$ to determine the proportion of the total load that it supports. Note that in some cases the critical load cases for the timber and steel will not coincide.

## Deflection

The deflection of the complete beam can be ascertained by calculating the deflection of the timber, using the share of load taken by the timber given by formulae (5) and (7). The total deflection is calculated by adding together the bending and shear deflections produced by each load in a load case.

In typical timber beams, the shear deflection usually accounts for less than 10% of the total deflection, although in short members, such as deep lintels, it can be as much as 20%. For desk-top calculations, ignoring shear deflection gives a reasonably accurate estimate of the total deflection, particularly in longer members where it might be critical. If it is found that the beam just fails or passes the appropriate deflection limit, then a more accurate calculation of deflection can be made by working out the shear deflection.

Formulae to calculate the bending and shear deflections at the mid-point of a simply supported beam with a UDL or point load are given at the end of this document.

## Bolt holes and bolt positions

The diameter of the bolt holes should be as close as possible to the diameter of the bolts, in order to maintain the composite action of the members. In accordance with Eurocode 5, a maximum of 1 mm oversize on the diameter is recommended. The holes should be accurately positioned in relation to one another, which means that either all three members should be drilled through in one operation or the steel plate should be predrilled and used as a template. In either case, the three members should be securely clamped together, or the first few bolts should be inserted and tightened as soon as the holes are drilled.

Tests on nailed flitch beams in which two 6 mm thick steel plates were nailed to two 51 mm thick x 100 mm deep LVL stringers demonstrated that at failure, buckling of the steel plate can occur even with three rows of nails 25 mm apart with the nails spaced at only 1.2 times the depth of the beam, although buckling was not necessarily the initial mode of failure (Alam and Ansell).

It is therefore recommended that the bolts for the UDLs should be staggered, with their centres alternately h/4 above and below the centre line, where h is the depth of the stringers. They should not be positioned further from the centre line than this, or the holes will significantly reduce the bending resistance of both the steel and timber members.

To ensure composite action of the timber and steel, the bolts should be equidistantly spaced at no more than 2.5 times the depth of the timber members or 600 mm. However the tests mentioned previously indicated that if fasteners are spaced too closely they reduce the failure load by causing the timber to split, so ideally the bolt diameter should be selected so that the required number of bolts can be spaced at the minimum of $2.5h_t$ and 600 mm.

To stabilise the steel, a minimum of two bolts should be used at each support, one above the other and spaced as widely apart as possible. These should be positioned at a distance of $0.5\ell_b$ from the inner edge of the support, where $\ell_b$ is the minimum bearing length required to prevent the permissible compression stress in the stringers and bearings being exceeded.

With large loads, two columns of bolts may be used at the supports or beneath a point load, spaced 4d apart, where d is the bolt diameter, positioned equidistantly on each side of the line of action. Similarly, two straight rows of bolts may be used along the beam if one staggered row is insufficient to support the UDLs.

The bolts for a point load should be positioned beneath the point load and be as close as possible to the centre line in order to reduce as little as possible the bending strength of the members. If the distance between two point loads is less than 4d it is impossible to put bolts for each load immediately beneath it, so for bolting purposes the two loads should be combined into one load positioned at a load-weighted distance between them. If the bolts for two point loads are combined in this way and the load durations differ then the sum of the number of bolts required for each load and its duration may be used conservatively.

According to Eurocode 3 the minimum distance between the holes and the edges of the steel plate should be $1.2d_0$, and the minimum distance between the holes should be $2.4d_0$, where $d_0$ is the hole diameter, ie d + 1 mm. The minimum end and edge distances in the timber and the minimum spacing perpendicular to the grain should be 4d, as recommended in BS EN 1995-1-1. The minimum spacing parallel to the grain should be 7d.

The bolts at the supports should be positioned so that they do not interfere with any joist or beam hangers which may be used.

## Effect of holes on bending stress and shear stress

Holes in the members reduce their section modulus and therefore increase the bending stress at the extremities. Where a single hole is offset from the centre line, as in staggered bolting, the centroid moves away from the hole, increasing the bending stress on the hole side still more.

The factor by which the bending stress in the timber and steel should be increased for a single offset hole is:

$$\frac{h^2\left(h^2 - hd' + 2d'y\right)}{h^4 - h^3d' - hd'^3 + d'^4 - 12hd'y^2} \qquad (10)$$

where h = depth of member
d’ = diameter of hole (1 mm more than the diameter of the bolt)
y = distance between centre of hole and centre of member

A single 11 mm diameter hole 55 mm from the centre line in two 220 mm deep timber members and a 195 mm deep steel plate increases the bending stress in the steel plate by 10% and by 7% in the timber members.

With two holes on opposite sides of the centre line and at an equal distance of y from it, the bending stress is increased by a factor of:

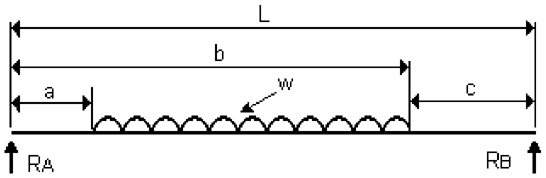
$$\frac{h^3}{h^3 - 2d'\left(d'^2 + 12y^2\right)} \qquad (11)$$

With the dimensions given above the bending stress is increased by 12% in the steel plate and by 8% in the timber members.

At the supports the cross-section of each member is reduced by the holes in it. The reduced cross-section should be used to calculate the shear stress in the timber and steel at the supports.

# Fomulae for beams

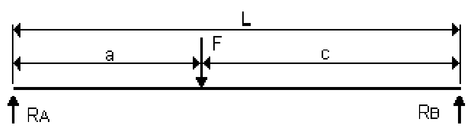
## Uniformly distributed loads




where w is the load per unit length.

| | Reaction at A | Reaction at B |
|---|---|---|
| | $R_A = \dfrac{w(b-a)(2L-a-b)}{2L}$ | $R_B = \dfrac{w(b^2-a^2)}{2L}$ |
| **Value of x** | **Bending moment at a distance x from $R_A$** | **Maximum bending moment** |
| $x \le a$ | $M_x = R_A x$ | |
| $a < x \le b$ | $M_x = R_A x - \dfrac{w(x-a)^2}{2}$ | $M_{max} = R_A\left(\dfrac{R_A}{2w}+a\right)$ at $x = \dfrac{R_A}{w}+a$ |
| $x > b$ | $M_x = R_A x - \dfrac{w(b-a)(2x-a-b)}{2}$ | |
| **Configuration** | **Bending deflection at centre** | **Shear deflection at centre** |
| $0 \le a \le L/2$<br>$b \le L/2$ | $y_b = \dfrac{w}{EI}\left\{b^2\left[\dfrac{L^2}{32}-\dfrac{b^2}{48}\right]-a^2\left[\dfrac{L^2}{32}-\dfrac{a^2}{48}\right]\right\}$ | $y_s = \dfrac{3w}{10GA}\left\{b^2-a^2\right\}$ |
| $0 \le a \le L/2$<br>$b > L/2$ | $y_b = \dfrac{w}{EI}\left\{\dfrac{5L^4}{384}-c^2\left[\dfrac{L^2}{32}-\dfrac{c^2}{48}\right]-a^2\left[\dfrac{L^2}{32}-\dfrac{a^2}{48}\right]\right\}$ | $y_s = \dfrac{3w}{20GA}\left\{L^2-2c^2-2a^2\right\}$ |
| $a > L/2$<br>$b > L/2$ | $y_b = \dfrac{w}{EI}\left\{(L-a)^2\left[\dfrac{L^2}{32}-\dfrac{(L-a)^2}{48}\right]-c^2\left[\dfrac{L^2}{32}-\dfrac{c^2}{48}\right]\right\}$ | $y_s = \dfrac{3w}{10GA}\left\{(L-a)^2-c^2\right\}$ |

## Point loads




| | Reaction at A | Reaction at B |
|---|---|---|
| | $R_A = \dfrac{Fc}{L}$ | $R_B = \dfrac{Fa}{L}$ |
| **Value of x** | **Bending moment at a distance x from $R_A$** | **Maximum bending moment** |
| $x \le a$ | $M_x = R_A x$ | $M_x = R_A a$ at $x = a$ |
| $x > a$ | $M_x = R_A x - F(x-a)$ | $M_x = R_A a$ at $x = a$ |
| **Configuration** | **Bending deflection at centre** | **Shear deflection at centre** |
| $a \le L/2$ | $y_b = \dfrac{Fa}{EI}\left\{\dfrac{L^2}{16}-\dfrac{a^2}{12}\right\}$ | $y_s = \dfrac{3Fa}{5GA}$ |
| $a > L/2$ | $y_b = \dfrac{Fc}{EI}\left\{\dfrac{L^2}{16}-\dfrac{c^2}{12}\right\}$ | $y_s = \dfrac{3Fc}{5GA}$ |

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