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**TECHNICAL NOTE**

# FLEXURAL TIMBER DESIGN TO EUROCODE 5 AND THE MALAYSIAN TIMBER CODE MS 544: 2001

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**Abstract:** For years, MS 544 has served as the code of practice for Malaysian engineers designing structural timber members using local timber species. The permissible stress design (PSD) code, published by the Department of Standards Malaysia, is closely modelled after the now obsolete British Standards code of practice CP 112. However, as the world moves rapidly towards globalization and the international engineering community having long since embraced limit state design philosophy, it is high time that Malaysian engineers do the same for structural timber by adopting the modern code of practice EN 1995-1-1. This code, also known as Eurocode 5, has been used in the United Kingdom and much of Western Europe for at least the past five years. This paper seeks to compare MS 544: 2001 and Eurocode 5 in terms of design philosophy and methodology, and highlights the similarities and differences between the two codes of practice, in particular for flexural timber design.

**Keywords:** *MS 544, Eurocode 5, permissible stress design, limit state design, flexural timber design.*

## 1.0 Introduction

In Malaysia, a country covered with lush tropical rainforests, 2500 species of trees attain sizes for sawn timber. Of these, 10% can be used as structural elements with unlimited supply if re-plantation is carried out (Yusof, 2010). Structural timber design in Malaysia is generally carried out in conformance to MS 544: 2001 – *Code of practice for structural use of timber*. This code of practice, developed by the Department of Standards, Malaysia, is in essence modeled on the design philosophies outlined in the British code of practice CP 112 and its successor BS 5268: Part 2. Even though the design approach follows the British practice, the strength and mechanical properties of 94 species of locally available timber are listed in MS 544: 2001. Furthermore, in order to aid the designer in specifying local timber for structural use, the code further categorizes these 94 timber species into seven strength groups and two subcategories according to their durability characteristics (MS 544: Part 2: 2001).

However, in the United Kingdom, BS 5268: Part 2 has been fully replaced since 2009 by Eurocode 5 (EC 5), a timber structural design code issued by the European Committee for Standardization. A number of beautiful buildings there have already been designed using EC 5, including the Sheffield Winter Gardens and the roof of Portcullis House, London.

It is obvious that the British designers have been using EC 5 for the past decade and there is no reason why Malaysian engineers must continue to stick to obsolescent or even obsolete design codes. It is therefore high time that local designers embrace EC 5.

This paper seeks to compare EC 5 and MS 544: 2001, and to highlight the salient features which make the former so much different from the latter. The comparison is focused on flexural member design as these members occur in most civil engineering structures, for example as floor joists in timber houses, girders for bridges, rafters and purlins as part of roof support systems, and as joists and stringers which form part of concrete formwork as shown in Figure 1.


Figure 1: Timber formwork for concrete bridge crossbeam construction.

## 2.0 Design Approach

MS 544: 2001 is a permissible stress design (PSD) code which means that two aspects of structural behaviour are dealt with simultaneously, namely:

(a) Stresses experienced by a structural member are not allowed to exceed the permissible stresses (McKenzie, 2000):

$$\textit{Applied stress} \leq \textit{Permissible stress} \tag{1}$$

(b) Once condition (a) has been satisfied, the deformations experienced by the structure are also limited.

Elastic theory is used to analyze structures under various loading conditions to give the worst design case. Then timber sections are chosen so that the permissible stresses are not exceeded at any point throughout the structure (Kermani, 1999).

Permissible stresses in timber are governed by the particular conditions of service and loading. For example, a thick piece of timber with high moisture content and which sustains loads over a period of time may paradoxically be able to carry lesser flexural stresses compared with a similar piece of timber of the same strength group but which is thinner, drier and is sustaining loads over a shorter period of time. In MS 544: 2001, permissible stresses are calculated as (Basri, 2007):

$$\textit{Permissible stress} = \textit{grade stress} \times \textit{(modification factors)} \tag{2}$$

Grade stresses are stresses which can safely be permanently sustained by a piece of timber of a specific size. Grade stresses are dependent on the particular species of timber in question and are given in Tables 1 and 2 of MS 544: Part 2: 2001 for 94 Malaysian timber species, and in Table 4 of the same document for seven timber strength groups.

The above mentioned modification factors, sometimes also referred to as the K-factors, are listed in Appendix A of MS 544: Part 1: 2001.

The design of timber structures according to EC 5 follows a totally different approach when compared with MS 544: 2001. The design is based on the limit state design philosophy which means that any given timber structure must meet two groups of limit states (Handbook 1 – Timber structures, 2008):

(a) Ultimate limit states – ultimate limit states are reached when the structure or a part of it collapses.

(b) Serviceability limit states – when these limits are breached, the structure does not experience catastrophic failure. However, it becomes no longer suitable for its intended use. In EC 5, there are two main serviceability conditions which must be satisfied, namely:

(i) Maximum deflections of timber members should be smaller than the prescribed ultimate values deemed to be acceptable, and

(ii) Vibrations of timber members should be within a specific range in order to avoid any unacceptable discomfort to users.

## 3.0 Design of Flexural Members

Main considerations when designing flexural members to MS 544: 2001 are (Zakaria, 1992):

(a) lateral stability of the member,

(b) limiting flexural stresses,

(c) limiting shear stresses,

(d) ensuring that the deflection of the flexural member is not excessive and is therefore kept below a stipulated value, and

(e) Limiting localized bearing stresses at supports and at other contact points.

(a) Lateral stability – When loaded, deep timber beams not only bend downwards as is usually the case, but also tend to experience out-of-plane deformation and twisting which causes lateral instability (Technical Report 14, 2003). This is known as lateral torsional buckling (LTB) and is depicted schematically in Figure 2. A real life example of this phenomenon is shown in Figure 3. What this means is that the actual moment carrying capacity of a beam is lesser that the theoretical value calculated by merely considering the beam cross section without taking lateral instability into account.

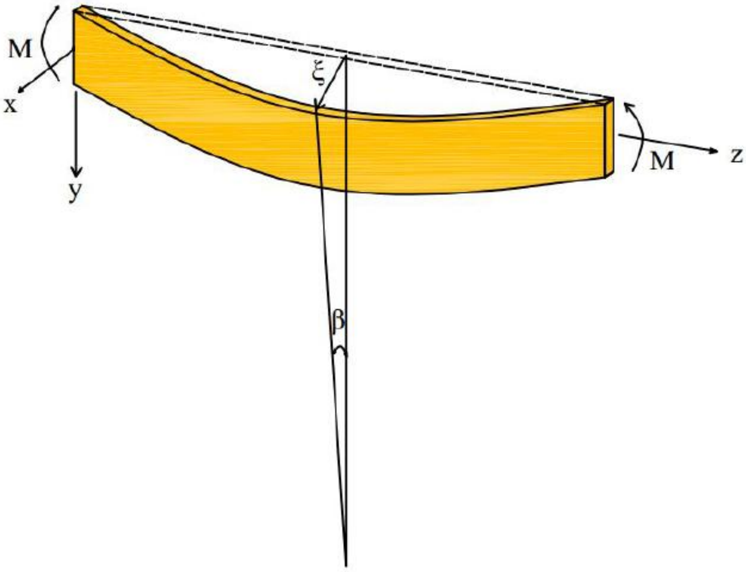


Figure 2: Lateral torsional buckling in timber beams. Notice that the beam has experienced an out-of-plane displacement, ξ, and has twisted by an angle β (Xiao, 2014).

Figure 3: Lateral torsional buckling of a deep composite timber I-beam.

Clause 11.8 of MS 544: Part 2: 2001 ensures that solid and laminated beams of rectangular cross-sections do not experience lateral torsional buckling by limiting the depth-to-breadth ratio of these beams (Johan Afandi Bin Hassan Basri, 2007):

$$(D/B)_{actual} \leq (D/B)_{allowable} \tag{3}$$

Depth-to-breadth ratios for various degrees of lateral restraint are listed in Table 7 of MS 544: Part 2: 2001.

EC 5 eschews this prescriptive approach to solving the issue of lateral torsional bucking in timber beams. Instead, a very scientific approach is applied where a parameter called the relative slenderness for bending, $\lambda_{rel,m}$, is calculated which is a measure of the degree of LTB experienced by a given timber beam. The design bending strength of the beam is then reduced according to a factor, $k_{crit}$, which is a function of $\lambda_{rel,m}$.

(b) Limiting flexural stresses – the maximum flexural stress induced in the flexural member under consideration should not under any circumstances exceed the permissible bending stress as stipulated in Clauses 9, 10 and 11 of MS 544: Part 2: 2001. Mathematically, this can be expressed as (Zakaria, 1992):

$$f_s \leq f_p \tag{4}$$

where,

$f_s$ = actual maximum flexural stress due to applied loads
$f_p$ = permissible flexural stress

$f_s$ is determined using simple elastic bending theory (McKenzie, 2000):

$$f_s = \frac{M}{Z} \tag{5}$$

where,

$M$ = maximum applied service bending moment
$Z$ = section modulus

According to Clause 11.1 of MS 544: Part 2: 2001, the value of permissible flexural stress, $f_p$, is determined by multiplying the grade stress of the timber species or timber strength group in question with the appropriate modification factors:

$$f_p = f_g \times K_1 \times K_2 \times K_5 \times K_6 \tag{6}$$

where,
$f_g$ = grade flexural stress value as stipulated in Clause 7 of MS 544: Part 2: 2001 for individual hardwood and softwood species, or for Malaysia structural timber grouped into seven so-called strength categories (Johan Afandi Bin Hassan Basri, 2007)

Therefore, according to MS 544: 2001, a timber flexural member is capable of sustaining bending stresses safely and adequately if the following condition is met:

$$f_s \leq f_g \times K_1 \times K_2 \times K_5 \times K_6 \quad (7)$$

Similarly, EC 5, or using its formal designation, EN 1995-1-1, calculates maximum flexural stress by using simple elastic bending theory as follows (McKenzie and Binsheng, 2007):

$$\sigma_{m,d} = \frac{M_d}{W} \quad (8)$$

where,
$\sigma_{m,d}$ = design flexural stress parallel to grain
$M_d$ = design bending moment
$W$ = elastic section modulus about the axis of bending

However, this is where the similarities between MS 544 and EN 1995-1-1 end. The design bending moment is calculated from design loads at the ultimate limit state with different partial safety factors being applied to dead loads – dubbed “permanent actions” in the Eurocode parlance – and to live or imposed loads – the corresponding Eurocode terminology being either “leading variable actions” or “accompanying variable actions” (Mosley, *et al.*, 2012).

In stark contrast, MS 544: 2001 being a permissible stress design code, calculates the design bending moment by multiplying a factor of unity to both the dead and live loads acting on a flexural timber member without taking into consideration any likelihood of these loads exceeding their assumed values throughout the working life of the structure.

EN 1995-1-1 stipulates in Clause 6.1.6 that a timber member in bending shall satisfy:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (9)$$

and,

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (10)$$

where,

$\sigma_{m,y,d}$ = design flexural stress about the y-y axis
$\sigma_{m,z,d}$ = design flexural stress about the z-z axis
$f_{m,y,d}$ = design bending strength about the y-y axis
$f_{m,z,d}$ = design bending strength about the z-z axis
$k_m$ = a factor which makes an allowance for re-distribution of stresses and the effect of inhomogeneities of material in a cross-section. Refer to Figure 4 for the orientations of y-y and z-z axes.

For rectangular sections of solid timber and glulam, Clause 6.1.6(2) of EN 1995-1-1 stipulates a value of 0.7 for $k_m$.

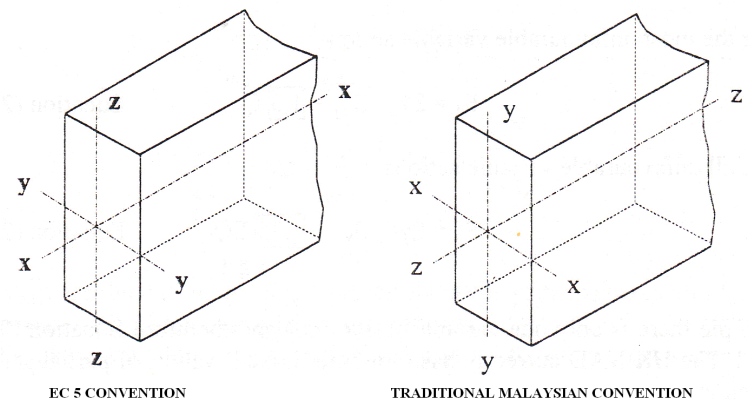


Figure 4: Flexural member axes according to EN 1995-1-1 (McKenzie, 2000).

These formulae, numbered 6.11 and 6.12 respectively in EN 1995-1-1 assume that the flexural member is experiencing bi-axial bending about both the major y-y axis and the minor z-z axis (McKenzie and Binsheng, 2007). As most flexural members such as timber beams and girders normally experience uniaxial bending about the major y-y axis, equation (10) can be disregarded and by taking $\sigma_{m,z,d}$ = 0, equation (9) can be simplified to:

$$\sigma_{m,y,d} \leq f_{m,y,d} \tag{11}$$

where,

$$f_{m,y,d} = (f_{m,y,k} \times k_{\text{mod}} \times k_h \times k_{m,\alpha} \times k_{sys}) / \gamma_M \tag{12}$$

and,

$f_{m,y,k}$ = characteristic bending strength of the timber stress class chosen for the flexural member

$k_{mod}$ = a modification factor which takes into account the effects of the duration of load and moisture content. Values of $k_{mod}$ are listed in Table 3.1 of EN 1995-1-1

$k_h$ = a modification factor which takes into consideration the influence of the depth or thickness of a timber member on its strength. Formula (3.1) of EN 1995-1-1 is for rectangular solid timber members, equation (3.2) for rectangular glulam members and equation (3.3) is for laminated veneer lumber structural members of rectangular cross-section

$k_{m,\alpha}$ = a modification factor related to the bending strength of single-tapered beams. Take $k_{m,\alpha}$ = 1.0 for rectangular prismatic cross-sections

$k_{sys}$ = system strength factor, normally taken as 1.1. EN 1995-1-1 recognizes that when several equally spaced similar members, components or assemblies are laterally connected by a continuous load distribution system, the member strength properties can be increased by as much as 10%. This is akin to the modification factor $K_2$ for load sharing systems stipulated in Clause 10 of MS 544: Part 2: 2001 which incidentally has the same value of 1.1

Therefore, according to EN 1995-1-1, a rectangular timber beam sustains bending stresses adequately if the following condition is met:

$$\sigma_{m,y,d} \leq (f_{m,y,k} \times k_{\mathrm{mod}} \times k_h \times k_{sys}) / \gamma_M \tag{13}$$

where, $\gamma_M$ is given in Table 1:

Table 1: Partial factors for material properties and resistances, $\gamma_M$ (Porteous and Kermani, 2007).

| States/combinations | $\gamma_M$ |
|---|---|
| Ultimate limit states (fundamental combinations) | |
| Solid timber – untreated or treated with a preservative | 1.3 |
| Glued-laminated timber | 1.25 |
| LVL, plywood and OSB | 1.2 |
| Particleboard | 1.3 |
| Fibreboards – hard, medium, MDF, soft | 1.3 |
| Punched metal plate fasteners | |
| Anchorage strength | 1.3 |
| Plate (steel) strength | 1.15 |
| Connections – excluding punched metal plate fasteners | 1.3 |
| Ultimate limit states – accidental combinations | |
| Any material and connection | 1.0 |
| Serviceability limit states – all combinations | |
| Any material and connection | 1.0 |

(c) Limiting shear stresses – the maximum shear stress induced in the flexural member must not exceed the allowable shear stress for the given timber species or timber strength group (Zakaria, 1992):

$$q_s \leq q_p \tag{14}$$

where,

$q_s$ = actual maximum shear stress

$= 1.5\dfrac{V}{A}$ (for a rectangular cross-section)

$V$ = maximum shear force

$A$ = effective cross-sectional area of the timber flexural member

Allowable shear stress according to MS 544: 2001 (Johan Afandi Bin Hassan Basri, 2007) is given as:

$$q_p = q_g \times K_1 \times K_2 \times K_5 \tag{15}$$

where,
$K_1$, $K_2$ and $K_5$ are the above mentioned K-factors, and
$q_g$ = grade shear stress given in Tables 1 and 2 for 94 Malaysian timber species and in Table 4 for seven strength groups of timber (MS 544: Part 2: 2001)

Therefore, according to MS 544: 2001, a timber beam can safely sustain shear stresses when:

$$q_s \leq q_g \times K_1 \times K_2 \times K_5 \tag{16}$$

EN 1995-1-1 takes a similar approach as stipulated in equation 6.13 of the code:

$$\tau_d \leq f_{v,d} \tag{17}$$

where,

$\tau_d$ = design shear stress at the ultimate limit state (ULS)

$$= \frac{1.5V_d}{bh} \text{ (for a rectangular cross-section)} \tag{18}$$

$V_d$ = design vertical shear force at ULS

$b$ = width of beam

$h$ = depth of beam

The design shear strength is given as:

$$f_{v,d} = (f_{v,k} \times k_{\mathrm{mod}} \times k_v \times k_{sys}) / \gamma_M \quad (19)$$

where,
$f_{v,k}$ = characteristic value of shear strength. This is obtained from Table 1 of BS EN 338: 2009: *Structural timber – Strength classes*.

and,
$k_v$ = reduction factor for notched beams as stipulated in Clause 6.5.2 of EN 1995-1-1. For a beam without notches or notched at the opposite side to the support, take $k_v$ = 1.0.

Therefore, according to EN 1995-1-1, a timber beam without notches is deemed to be able to resist shear stresses adequately if:

$$\tau_d \le (f_{v,k} \times k_{\mathrm{mod}} \times k_{sys}) / \gamma_M \quad (20)$$

(d) Limiting deflection – Clause 11.7 of MS 544: Part 2: 2001 stipulates that a flexural member should be designed in such a way as to restrict its deflection within limits which ensure that surfacing materials, ceilings, partitions and finishes supported by or attached to the flexural member are not damaged. Deflection is also restricted to conform to functional needs and aesthetic requirements. The clause further stipulates that the deflection of a flexural member when fully loaded should not exceed 0.003 of the span. For domestic floor joists, the deflection under full load should not exceed 0.003 times the span or 14 mm, whichever is the lesser. The 14 mm deflection limitation is to avoid undue vibration under moving or impact loading.

Mathematically, Clause 11.7 can be expressed as (Zakaria, 1992):

$$\Delta_{actual} \le \Delta_p \quad (21)$$

where,

$\Delta_{actual}$ = actual deflection

$\Delta_p$ = permissible deflection
= 0.003 x span
= lesser of (0.003 x span, 14 mm) for domestic floor joists

The deflection of any beam is a combination of bending deflection and shear deflection (Baird and Ozelton, 1984). Shear deflection is usually a fairly small percentage of the total deflection of solid sections, but Clause 11.7 deems shear deflection to be significant in glulam beams and exhorts the structural designer to take it into account. Therefore (Zakaria, 1992):

$$\Delta_{actual} = \Delta_{bending} + \Delta_{shear} \tag{22}$$

$$\Delta_{actual} = \frac{5wL^4}{384EI} + \frac{FM_0}{GA} \tag{23}$$

where,

$\Delta_{bending}$ = bending deflection

$\Delta_{shear}$ = shear deflection

$w$ = uniformly distributed load per unit length of the flexural member

$L$ = span of the flexural member

$E$ = modulus of elasticity. MS 544: Part 2: 2001 stipulates that for a single solid timber beam, use the minimum modulus of elasticity value, $E_{\min}$, for the given timber species or strength group. However, for flexural members which form part of a load sharing system, deflection values should be calculated using the mean modulus of elasticity, $E_{\text{mean}}$

$I$ = second moment of area of the beam cross-section about the axis of bending

$F$ = a form factor dependent on the cross-sectional shape of the beam (equals to 1.2 for a solid rectangle)

$M_0$ = bending moment at mid-span

$G$ = modulus of rigidity, normally taken as $\frac{E}{16}$ (Davalos *et al.*, 1991)

$A$ = cross-sectional area

EN 1995-1-1 takes an approach similar to that of MS 544: 2001 in dealing with deflection of a flexural member by limiting deflection values to those stipulated in Table 7.2. According to Clause 7.2 of the code, deflection is given as:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \tag{24}$$

where,

$w_{net,fin}$ = net final deflection

$w_{inst}$ = instantaneous deflection

$w_{creep}$ = creep deflection

$w_{fin}$ = final deflection

$w_c$ = precamber (if applicable)

These parameters are illustrated in Figure 5.

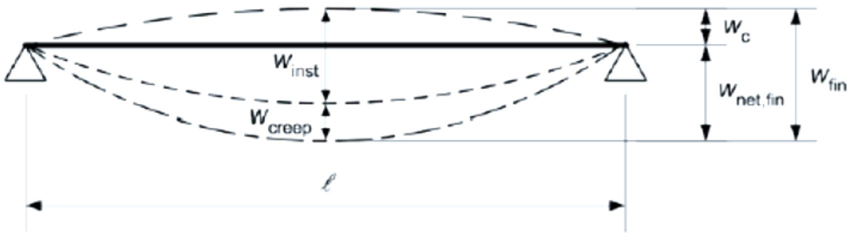


Figure 5: Components of deflection according to EN 1995-1-1.

(e) Limiting localised bearing stresses – Localised compressive stresses induced at support locations or due to concentrated loads can cause failure to occur as these stresses act in a direction perpendicular to the grain. The compressive strength of Malaysian timber perpendicular to grain is only in the range of 10 to 20% of the corresponding strength value parallel to grain (Table 4, MS 544: Part 2: 2001). The applied bearing stress is calculated from the following equation (Kermani, 1999):

$$\sigma_{c,a,\perp} = \frac{F}{A_{bearing}} \tag{25}$$

where,

$\sigma_{c,a,\perp}$ = applied compressive stress perpendicular to grain
$F$ = reaction force at the support or applied concentrated load
$A_{bearing}$ = bearing area (= bearing length x breadth of the section)

MS 544: 2001 stipulates that:

$$\sigma_{c,adm,\perp} \geq \sigma_{c,a,\perp} \tag{26}$$

where,

$\sigma_{c,adm,\perp}$ = permissible value of bearing stress

$$\sigma_{c,adm,\perp} = \sigma_{c,g,\perp} \times K_1 \times K_2 \times K_3 \tag{27}$$

$\sigma_{c,g,\perp}$ = compression perpendicular to grain values obtained from Tables 1 and 2 for 94 Malaysian timber species or from Table 4 for seven Malaysian timber strength groups.

$K_1$, $K_2$ and $K_3$ = K-factors as mentioned earlier

EN 1995-1-1 uses the same approach as MS 544: 2001 in order to guard against localised bearing failure at beam supports or at the points of application of concentrated loads. Clause 6.1.5 stipulates that the following condition shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} \tag{28}$$

with,

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{cf}} \tag{29}$$

where,

$\sigma_{c,90,d}$ = design compressive stress in the effective contact area perpendicular to the grain

$F_{c,90,d}$ = design compressive load perpendicular to the grain

$A_{cf}$ = effective contact area in compression perpendicular to the grain

$f_{c,90,d}$ = design compressive strength perpendicular to the grain

$k_{c,90}$ = a factor which takes into account the load configuration, the possibility of splitting and the degree of compressive deformation. Clauses 6.1.5(2), 6.1.5(3) and 6.1.5(4) stipulate values for this factor depending on support conditions, whether the supports are continuous or discrete in nature, support geometry and type of member, whether it is made of solid softwood timber or glued laminated softwood timber

## 4.0 Design Example

A design example (adapted from Zakaria, 1992) is used to illustrate the workings of both MS 544: 2001 and EN 1995-1-1:

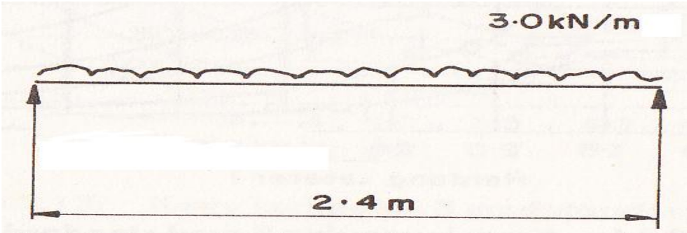

Figure 6: Timber beam (Zakaria, 1992).

Determine whether the beam shown in Figure 6 above can withstand the long-term load of 3.0 kN/m (uniformly distributed throughout the span). The beam consists of a piece of timber with a nominal size of 50 mm x 200 mm and is planned on all four sides. The timber used is Standard Structural grade of Balau. Beam ends are supported on 125 mm wide blockwork walls. The solution is depicted in Table 2 below:

Table 2: Design of the timber beam depicted in Figure 6 according to MS 544 and EC 5.

| Parameter | MS 544:2001 | EC 5 | Remarks |
|---|---|---|---|
| Design load | 3.0 kN/m | 4.05 kN/m | EC 5 calculates the design load at ultimate limit state. In this case, the load is multiplied by 1.35. |
| Actual beam cross-section | 45 mm x 190 mm | 45 mm x 190 mm | Size reduced due to planing. Refer to Appendix B of MS 544: Part 2: 2001. No such guidance given in EC 5. |
| Design flexural stress | 7.98 N/mm$^2$ | 10.77 N/mm$^2$ | EC 5 calculates design stresses at ultimate limit state. |
| Allowable flexural stress | 26.5 N/mm$^2$ | 37.69 N/mm$^2$ | Refer to Table 2 of MS 544: Part 2: 2001 for grade flexural strength of Balau.<br>EC 5 calculates $f_{m,y,d}$ by assuming Balau to fall under strength class D70 of BS EN 338:2009 as characteristic density of Balau, $\rho_k$ > 900 kg/m$^3$. |
| Design shear stress | 0.63 N/mm$^2$ | 0.85 N/mm$^2$ | EC 5 calculates design stresses at ultimate limit state. |
| Allowable shear stress | 2.28 N/mm$^2$ | 2.69 N/mm$^2$ | Refer to Table 2 of MS 544: Part 2: 2001.<br>EC 5 refers to BS EN 338:2009 for value of $f_{v,k}$. |
| Design bearing stress | 0.64 N/mm$^2$ | 0.86 N/mm$^2$ | EC 5 calculates design stresses at ultimate limit state. |
| Allowable bearing stress | 3.74 N/mm$^2$ | 7.27 N/mm$^2$ | Refer to Table 2 of MS 544: Part 2: 2001.<br>EC 5 refers to BS EN 338:2009 for value of $f_{c,90,k}$..<br>$k_{c,90}$ is taken as unity in accordance with Clause 6.1.5(2) of EC 5. |

## 5.0 Conclusion

From the foregoing discussion, it can be concluded that:

1. MS 544: 2001 is a "permissible stress design" code whereas EC 5 conforms to the "limit state design" philosophy.

2. MS 544: 2001 presents mechanical properties of timber as "grade stresses" which already have built-in safety factors. EC 5 on the other hand, presents mechanical properties as "characteristic values" with no built-in safety factors. The designer must supply appropriate partial factors in order to convert these values to design values (TRADA, 2007).

3. MS 544: 2001 gives mechanical properties of 94 species of Malaysian timber and for seven timber strength groupings. However, there are no such values given in EC 5. The designer has to look elsewhere for these, for example, by referring to BS EN 338: 2009 or to other published test data.

4. MS 544: 2001 treats the phenomenon of LTB in a simplistic and prescriptive manner, whereas EC 5 employs physics to actually calculate the reduction in moment of resistance of a timber beam due to LTB.

5. The K-factor approach of MS 544: 2001 is similar to the use of modification factors in EC 5. For example, $k_{sys}$ of EC 5 is akin to $K_2$ of MS 544: 2001 and incidentally both parameters have the same value.

6. Unlike EC 5, MS 544: 2001 does not consider the effects of creep (TRADA, 2007).

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