Building Industry Consultative Council

DEMYSTIFIYIG THE EUROCODES

‘Course A’
Module 1
-
INTRODUCTION TO THE EUROCODES

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OUTLINING THE STRUCTURAL EUROCODES

These are an unrivalled set of unified international codes of practice for designing buildings and civil engineering structures. They embody the vast experience and research output of 19 member states.

Anyone opening the Eurocodes for the 1st time may find them complicated.

As the Eurocodes refer to 2nd order effects,

HISTORY OF STRUCTURAL EUROCODES

The idea to develop models for an international set of Codes for structural design for the different materials used in construction and applicable to all kinds of structures was born in 1974 based on an agreement between several technical-scientific organisations.

The scope was *"standardization of structural design rules for building and civil engineering works taking into account the relationship between design rules and the assumptions to be made for materials, execution and control."*

EUROCODE PROGRAMME

MSA EN 1990 Basis of Design
MSA EN 1991 Eurocode 1: Actions on structures
MSA EN 1992 Eurocode 2: Design of concrete structures
MSA EN 1993 Eurocode 3 : Design of steel structures
MSA EN 1994 Eurocode 4 : Design of composite steel and concrete structures
MSA EN 1995 Eurocode 5 : Design of timber structures
MSA EN 1996 Eurocode 6 : Design of masonry structures
MSA EN 1997 Eurocode 7 : Geotechnical design
MSA EN 1998 Eurocode 8 : Design of structures for earthquake resistance
MSA EN 1999 Eurocode 9 : Design of aluminium structures

MSA – Malta Standards Authority – WWW.Msa.org.mt The national annexes are available for purchasing or free viewing from the Standards Library of MCCAA – contact: standard@mccaa.org.mt

FORMAT OF THE STRUCTURAL EUROCODES

The Eurocodes contain a considerable number of parameters for which only indicative values are given. Each country may specify its own values for these parameters which are indicated by being enclosed by a box (|___|).

The appropriate values which are at least equivalent with regard to the resistance, serviceability and durability achieved with present Eurocodes, are set out in the National Application Document (NAD).

A BICC working group has been working on these NAD's over the past 1½ years.

2014-12-11 Eurocodes NA MSA status - 1

Reference	Title
EN 1990:2002	Eurocode: Basis of structural design
EN 1991-1-1:2002	Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight and imposed loads for buildings
EN 1992-1-1:2004	Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings
EN 1992-1-2:2004	Eurocode 2: Design of concrete structures - Part 1-2: General rules - Structural fire design
EN 1992-2:2005	Eurocode 2 - Design of concrete structures - Concrete bridges - Design and detailing rules
EN 1992-3	Eurocode 2 - Design of concrete structures - Part 3: Liquid retaining and containment structures
EN 1994-1-1:2004	Eurocode 4: Design of composite steel and concrete structures – Part 1: General rules and rules for buildings
EN 1994-1-2:2005	Eurocode 4 - Design of composite steel and concrete structures – Part 2: General rules - Structural fire design
EN 1994-2:2005	Eurocode 4 - Design of composite steel and concrete structures - 2: General rules and rules for bridges

2014-12-11 Eurocodes NA MSA status - 2

EN 1996-1-1:2005	**Eurocode 6 - Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures**
EN 1996-1-2:2005	**Eurocode 6 - Design of masonry structures - Part 1-2: General rules Structural fire design**
EN 1996-2	**Eurocode 6 - Design of masonry structures - Part 2: Design considerations, selection of materials and execution of masonry**
EN 1996-3	Eurocode 6 - Design of masonry structures - Part 3: Simplified calculation methods for unreinforced masonry structures
EN 1999-1-1	Eurocode 9: Design of Aluminium Structures - Part 1-1: General rules
EN 1999-1-2	Eurocode 9: Design of aluminium structures - Part 1-2: General - Structural fire design
EN 1999-1-3	Eurocode 9: Design of Aluminium Structures - Part 1-3: Additional rules for structures susceptible to fatigue
EN 1999-1-4	Eurocode 9: Design of Aluminium Structures - Part 1-4: Supplementary rules for trapezoidal sheeting
EN 1999-1-5	Eurocode 9: Design of Aluminium Structures - Part 1-5: Supplementary rules for shell structures

UNUSUAL DEFINITIONS

BS 8110 differ from EC2 in that they contain a considerable amount of material which those drafting EC2 would have considered to belong more properly in a manual. E.g. bending moment coefficients for beams and slabs, design charts, etc.

One area where the EC2 terminology differs is its use of the word 'actions'. This is a logical term used to describe all the things that can act on a structure. The definition states that it includes 'direct actions' (loads) and ' indirect actions' (imposed deformations).

Self weight and dead loads are permanent actions normally represented by a unique value.

Superimposed loads are variable actions having different values depending on combination value Ψ, rare load combination Ψo, frequent value Ψ_1, and quasi-permanent value Ψ_2, found in EC1.

An accidental action normally has a unique value.

Rules for Application: Indicative Values

The common basic rules of structural design follow the requirements for public safety and serviceability of structures based on the principle of risk in terms of reliability conditions. Construction works should be fit for their intended use and offer:

- adequate durability under normal maintenance conditions;
- an economically reasonable working life;
- the structure should also be designed so that it will not sustain damage disproportionate to the original cause.

On the other hand, the Eurocodes give the necessary liberty to the designers whilst allowing innovation in the construction industry.

EN 1990 EUROCODE – *BASIS OF STRUCTURAL DESIGN*

This was approved in October 2001. It is the world's first 'material-independent' design code. The large number of materials include concrete, steel, masonry, timber and aluminium, whilst the disciplines incorporate fire, geotechnics, earthquake, bridge design etc. This Eurocode introduces the principles and requirements for safety, serviceability and durability, whilst providing an introduction to reliability and risk management and its limit-state design philosophy based on partial safety factors. It also summarises the loading combinations for the assessment of structures.

A novel combination value gives the ultimate value for actions that cannot occur simultaneously, such as the proportion of the live load to be considered in combination with seismic forces, the predominant permanent action being established in such cases.

EN 1991 EUROCODE 1:
Actions on Structures

This is in an advanced state of development, forming one of the key documents in the suite of 19 structural Eurocodes. It is in four parts, the first part being divided into sections covering self and imposed loads and actions due to fire, snow, wind, heat, construction and accidents. The remaining three parts cover traffic loads on bridges, actions by cranes and machinery and actions in silos and tanks. For the first time in an international standard, annexes provide models for more realistic calculation of thermal actions.

Guidance on wind actions is provided for the structural design of buildings, chimneys and bridges. The data on wind velocity to be provided as a national annex corresponds to the 10 minute wind speed, with an annual probability of exceedance of 0.02 (50 year period), taken at 10.0m above ground.

EN 1992 Eurocode 2:
Design of concrete structures

The first part of the code in plain, reinforced and prestressed concrete, covering common design rules and design requirements for. The second and third parts covers design of bridges and liquid-retaining structures.

All the expressions in the code relate to cylinder strength, not cube strength of concrete. The published first part contains national annexes which deal with matters such as partial factors for material. The items to be covered in the national annex are very limited, with six in the design rules section and three in the fire section.

EN 1993 Eurocode 3: *Design of steel structures*

This code is wider in scope than most other Eurocodes due to diversity of steel structures, the need to cover both plastic and elastic design, the use of both bolted and welded joints and the possible slenderness of construction.

It codifies semi-rigid joints, sheetpiles, shells, silos and stainless steel structures for the first time. For cold formed steelwork, more advanced methods of design are included. It is also unusual in having a partial material safety factor of 1.0, since a recent survey of European steel products shows they are generally around 20% stronger than their nominal value.

EN 1994 Eurocode 4:
Design of composite steel and concrete structures

This code applies to composite structures and members made of structural steel, and reinforced or prestressed concrete connected together to resist loads.

- Part 1-1: General-common rules and rules for building
- Part 1-2: Structural fire design
- Part 2 : Bridges.

The scope of this code is to be wider than any previous codes. For buildings, web-encased beams, columns joints and frames are included. For bridges, double composite action, trusses, tied arches, filler beam decks and prestressing by tendons or by jacking at supports are included.
Further reference is made to partially encased composite beams, highstrength structural steels, composite joints, composite columns and composite slabs.

EN1995 Eurocode 5:
Design of timber structures

Unlike BS 5268 based on the permissible stress, this code, to be consistent with the other Eurocodes, adopts the limit state.

The code is divided into two parts, with the first part giving general rules for buildings and the second devoted to bridges.

Serviceability is considered in great detail, particularly creep deflection and floor vibrations.

More important is the CE marking given to timber, which has to rely on a Eurocode for validation.

EN 1996 Eurocode 6:
Design of masonry structures

The first part of this code relates to buildings and other civil engineering works in unreinforced, prestressed and confined masonry. The first part of the code applies to the design of buildings and civil engineering works. Only the requirements for resistance, serviceability and durability of structures are dealt with, including also fire and lateral load design.

The second part of the code deals with the design selection of materials and execution of masonry.

The third part relates to simplified and simple rules for masonry structures. Examples include the thickness of basement walls of a certain height, a simplified method for obtaining the eccentricity of walling on unreinforced walls, together with the factors to be used in lateral load design.

EN 1997 Eurocode 7: *Geotechnical design*

This code aims to bridge the gap between traditional geotechnical calculations relying on highly subjective assessments of design parameters, with greater emphasis on serviceability and how this is satisfied through ultimate-limit-state design. This implies a rectangular soil foundation stress block, instead of the traditional triangular or trapezoidal stress block, doing away with the middle third criteria, thus simplifying bending moment and shear force calculations.

The first class in geotechnical limit state design is the ultimate limit state, in which either a mechanism is formed in the ground or in the structure, or even severe structural damage occurs due to movements in the ground. Five ultimate limit states are to be considered: (EQU), (STR), (GEO), (UPL) & (HYD).

The second class is the serviceability limit state at which deformation in the ground will cause loss of serviceability in the structure. This includes settlements which affect the appearance or efficient use of the structure, or cause damage to finishes or nonstructural elements, or vibration which causes discomfort to people or damage to the content of the building. By dividing geotechnical tasks into various categories the code specifies the various geotechnical risks encountered as related to hazard and vulnerability levels. The low, moderate and high-risk category then goes on to outline the necessary procedure to be adopted.

Geotechnical Categories & Geotechnical Risk Higher Categories satisfied by greater attention to the quality of the geotechnical investigations and the design

Table 1: Geotechnical Categories related to geotechnical hazard and vulnerability levels

Factors to be considered	Geotechnical categories		
	GC1	GC2	GC3
Geotechnical hazards /vulnerability /risk	Low	Moderate	High
Ground conditions	Known from comparable experience to be straightforward. Not involving soft, loose or compressible soil, loose fill or sloping ground.	Ground conditions and properties can be determined from routine investigations and tests.	Unusual or exceptionally difficult ground conditions requiring non-routine investigations and tests.
Regional seismicity	Areas with no or very low earthquake hazard	Moderate earthquake hazard where seismic design code (EC8 Part V) may be used	Areas of high earthquake hazard
Surroundings	Negligible risk of damage to or from neighbouring structures or services and negligible risk for life	Possible risk of damage to neighbouring structures or services due, for example, to excavations or piling	High risk of damage to neighbouring structures or services

Table 1 (cont.)	Geotechnical Categories		
	GC1	GC2	GC3
Expertise required	**Person with appropriate comparable experience**	**Experienced qualified person – Civil Engineer**	**Experienced geotechnical specialist**
Design procedures	**Prescriptive measures and simplified design procedures e.g. design bearing pressures based on experience or published presumed bearing pressures. Stability of deformation calculations may not be necessary**	**Routine calculations for stability and deformations based on design procedures in EC7**	**More sophisticated analyses**
Examples of structures	**- Simple 1 & 2 storey structures and agricultural buildings having maximum design column load of 250kN and maximum design wall load of 100kN/m - Retaining walls and excavation supports where ground level difference does not exceed 2m**	**Conventional: - Spread and pile foundations Walls and other retaining structures - Bridge piers and abutments Embankments and earthworks**	**- Very large buildings - Large bridges - Deep excavations - Embankments on soft ground Tunnels in soft or highly permeable ground**

Ultimate Limit State (ULS) partial factors (persistant & transiet situations)

Table 2 - Partial factors for ultimate limit states in persistent and transient situations

Parameter	Factor	Case A	Case B	Case C	Case C2	Case C3
Partial load factors (γF)		(UPL)	(STR)	(GEO)	(EQU)	(HYD)
Permanent unfavourable action	γG	1.00	1.35	1.00	1.35	1.00
Variable unfvaourable action	γQ	1.50	1.50	1.30	1.50	1.20
Permanent fvourable action	γG	0.95	1.00	1.00	1.00	1.00
Variable favourable action	γQ	0	0	0	0	0
Accidental action	γA	1.00	1.00	1.00	1.00	1.00

Values in red are partial factors either given or implied in ENV version of EC7
Values in green are partial not in the ENV that may be in the EN version

EN 1998 Eurocode 8:
Design of structures for earthquake resistance

This code has five parts which cover a range of structures including buildings, bridges, towers, tanks and geotechnical structures. The life-safety objective is followed in the code, implying that the structure may be damaged, but it must not collapse in order to prevent loss of life.

Structures are to be designed to resist an earthquake which has a 10% chance of exceedance in 50 years, otherwise known as a 475-year return period. Each state is responsible for defining an appropriate seismic hazard map. The philosophy behind the code is that areas with a design ground acceleration less than 0.1g are treated as regions of low seismicity, with simplified design procedures being implemented. For areas where the design ground acceleration is less than 0.04g the provisions of Eurocode 8 do not need to be observed.

Another part of the code covers seismic strengthening and repair of buildings. This reflects the importance of seismic evaluation and retrofitting of existing structures.

Malta’s Seismic Zoning - EC8

•Design grd. Acceleration for a return period of [475] yrs (EC8) taken at 0.06g (being the ground motion level which is not going to be exceeded in the 50 years design life in 90% of cases.

MM – Earthquake Intensity	Return Period (years)	Base Shear Design % of g
VI	125	2-5
VII	1000	5-10
VIII	10,000	10-20

Defined as a low seismicity zone as <0.10g but > 0.04g
EC2 concrete provisions to be catered for - not EC8.

EN 1999 Eurocode 9:
Design of aluminium structures

Owing to the increasing use of aluminium alloys in construction this code has been added as an alternative to steel. With only a third of the weight, $2700 kg/m^3$, together with a comparable strength varying between 150 to $350 N/mm^2$ and a self-protecting surface, the material has clear advantages over steel but it also behaves very differently. It has a high deflection and buckling tendency due to its Young's modulus also being a third that of steel, $70{,}000 N/mm^2$, no yield plateau and complex strain hardening characteristics, with the importance of ductility on local and global behaviour being given.

Fire design included in all Eurocodes is very relevant for aluminium as it is generally less resistant to high temperatures than steel and reinforced concrete. Nevertheless, by introducing rational risk-assessment methods, the analysis of a fire scenario might in some cases, result in a more beneficial time-temperature relationship and thus make aluminium more competitive.

COMPARISON OF PROPERTIES FOR STRUCTURAL MATERIALS

Material	Ultimate Stress (N/mm^2)	Modulus of Elasticity (N/mm^2)	Density (KN/m3)	Coeff of Thermal Expansion *10-6/°C	Embodied Energy MJ/kg (Embodied CO_2)(kg/t)	Material Factor of Safety (EC's & PrEN) γ_m
Mild steel	275	205000	70	10.8	35(2030)	1.0
High Yield steel	460	200000	70	10.8	35(2030)	1.0
Pre-stressing wire	1570	200000	70		35(2030)	1.15
Aluminium Alloy	255	70000	24	23.0	300(17000)	1.2
Timber: Softwood Hardwood	10-30** 35-70**	8000** 12000**	6	3.5** 3.5**	2(1644) 3(2136)	1.3
Reinforced concrete	20-60	28000 - 40000	24	10.8	8(203)	1.5
Glass fibre composite	250	20000	18		100(8070)	1.7
Limestone Masonry	7.5	17000	20	4.0	3(2136)	2.3-3.0
Annealed glass	13(45*)	70000	25	8.3	15(1130)	1.8
Prestressed glass	45(150*)	70000	25	8.3	20(1130)	1.2 – 1.8

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DEMYSTIFIYING THE EUROCODES

‘Course A’
Module 2
AS REFERRING TO HEAD CODE EN1990 – BASIS OF STRUCTURAL DESIGN

DESIGN WORKING LIFE EXAMPLES

Design working life	**Examples**
1-5 years	Temporary structures
25 years	Replacement structural parts e.g. handrails, small canopies, protective features (slats, caps, etc.)
50 years	Buildings, footbridges and other common structures
100 years	Monumental buildings and other special or important structures
120 years	Highway and rail bridges

DESIGN SITUATIONS

(1)P The relevant design situations shall be selected taking into account the Circumstances under which the structure is required to fulfil its function.

(2)P Design situations shall be classified as follows :

– persistent design situations, which refer to the conditions of normal use ;
– transient design situations, which refer to temporary conditions applicable to the structure, *e.g. during execution or repair ;*
– accidental design situations, which refer to exceptional conditions applicable to the structure or to its exposure, *e.g. to fire, explosion, impact or the consequences of Localised* failure ;
– seismic design situations, which refer to conditions applicable to the structure when subjected to seismic events.

NOTE Information on specific design situations within each of these classes is given in EN 1991 to EN 1999.

(3)P The selected design situations shall be sufficiently severe and varied so as to Encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure.

Ultimate Limit State Verification

The following ultimate limit states shall be verified as relevant:

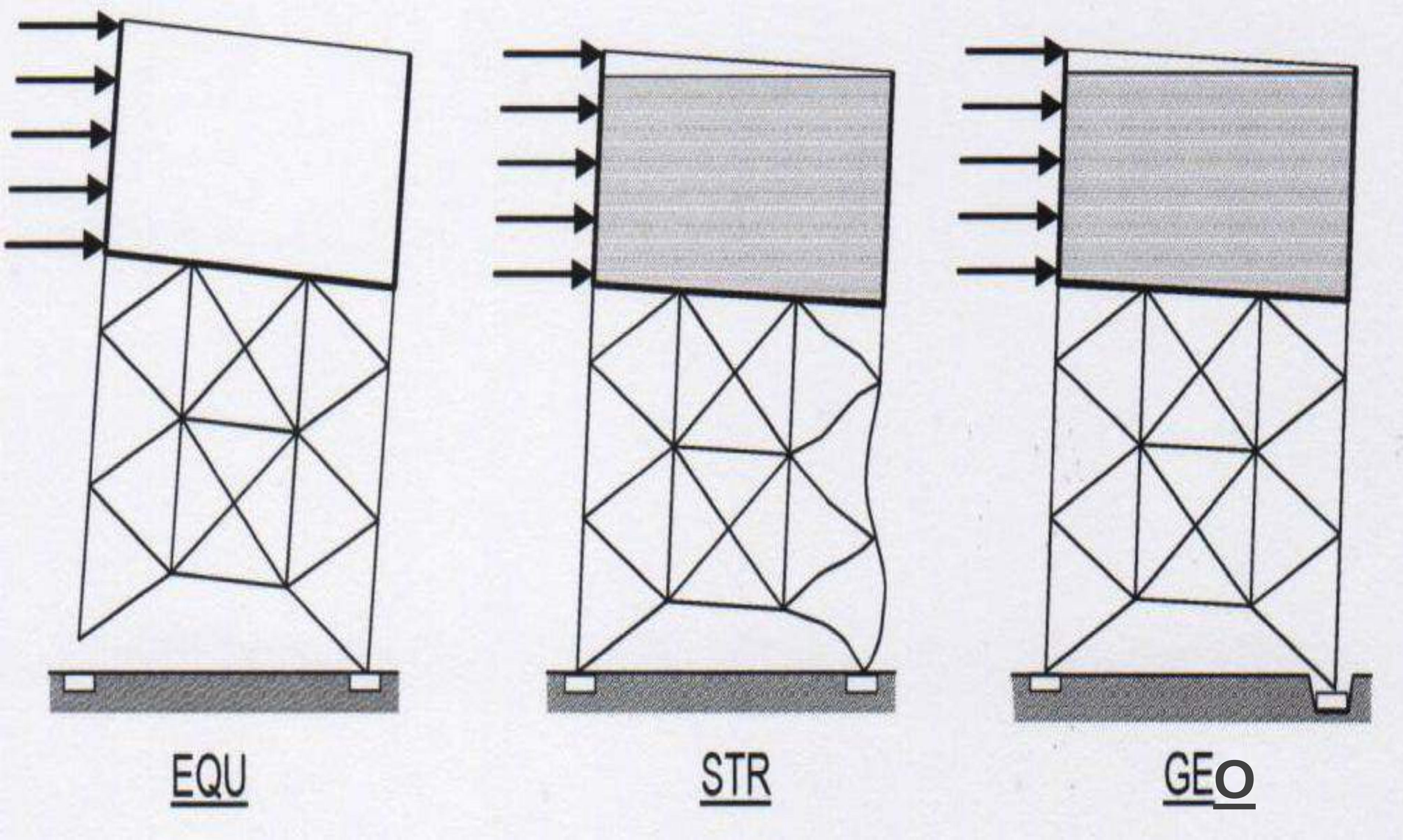
EQU: Loss of equilibrium of the structure. (considering for slding overturning or uplift).

STR: Internal failure or excessive deformatgion of the structure of structural member (Design of structural for strength of members and frames).

GEO: Failure due to excessive deformation of the ground (Design of structural members such as footing, piles, basement walls, etc.)

FAT: fatigue failure of the structure or structural member.

EQU, STR & GEO Conditions



European Model Codes in the 60s and 70s

The principles of partial safety factors was proposed in 1927, by the Danish Moe.

An early example of the result of this work is in a British standard CP110. Any condition that a structure might attain, which contravened the basic requirement was designated a Limit State.

The most important innovation in CP110 was the explicit use of probability theory in the selection of "characteristic" values of strength which – according to some notional or measured distribution – would be exceeded in at least 95% of standardised samples.

In 1978 the Nordic Committee on Building Regulations (1978) issued a report on Limit State Design containing "Recommendation for Loading and Safety Regulations of Structural Design"

– NKB report No 36.

It introduces a concept of Structural Reliability dealing in safety and control class

LIMIT STATE DESIGN – CHARACTERISTIC VALUE & DESIGN STRENGTH

CHARACTERISTIC STRENGTH OF A MATERIAL is the strength below which not more than 5% (or 1 in 20) samples will fail.

CHARACTERISTIC STRENGTH = MEAN VALUE – 1.64 X Standard Deviation

DESIGN STRENGTH =

$$\frac{\text{CHARACTERISTIC STRENGTH}}{\text{MATERIAL FACTOR OF SAFETY}} = \frac{f_u}{\gamma_m}$$

EXAMPLE:

Ten concrete cubes were prepared and tested by crushing in compression at 28 days. The following crushing strengths in N/mm2 were obtained:

44.5 47.3 42.1 39.6 47.3 46.7 43.8 49.7 45.2 42.7

Mean strength x_m $= \dfrac{448.9}{10} = 44.9$ **N/mm2**

Standard deviation $= \sqrt{[(x-x_m)2/(n-1)]} = \sqrt{(80/0)}$

$= 2.98$ **N/mm2**

Characteristic strength $= 44.9 – (1.64 \text{ X } 2.98)$

$= 40.0$ **N/mm2**

Design strength $= \dfrac{40.0}{\gamma_m} = \dfrac{40.0}{1.5}$

$= 26.7$ **N/mm2**

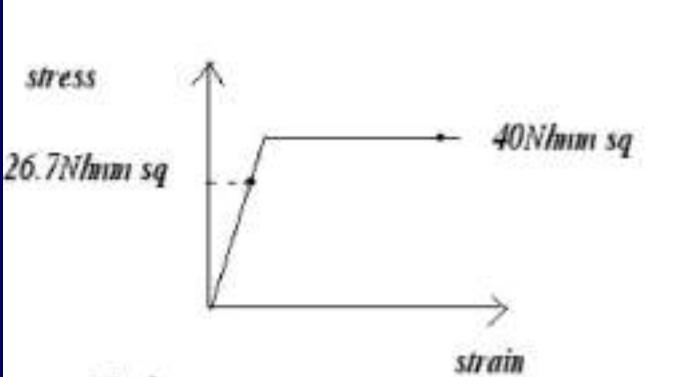


Fig 3

Combination of Actions for Persistent/Transient Design Situation

COMBINATION OF ACTIONS FOR PERSISTENT/TRANSIENT DESIGN SITUATION
(EN1990,cl.6.4.3.2)

Persistent and transient design situation – EQU Equation 6.10 (Set A)				
		Action	**Favourable γ**	**Unfavourable γ**
6.10	$E_d=\Sigma\gamma_G G_k+\gamma_Q Q_{k1}+\gamma_Q \psi Q_{k2}$	Permanent (dead, earth), γ_G	0.9	1.1
		Variable (imposed, wind), γ_Q	0	1.5
Note: Single source is not applicable for EQU design situation. Different γ factors can be used in favourable and unfavourable areas.				

Persistent and transient design situation – STR/GEO Equation 6.10, 6.10a & 6.10b (Set B)				
6.10	$E_d=\Sigma\gamma_G G_k+\gamma_Q Q_{k1}+\gamma_Q \psi Q_{k2}$	**Action**	**Favourable γ**	**Unfavourable γ**
6.10a	$E_d=\Sigma\gamma_G G_k+\gamma_Q \psi_{0,1} Q_{k1}+\gamma_Q \psi Q_{k2}$	Permanent (dead, earth), γ_G	1.0	1.35
6.10b	$E_d=\Sigma\xi\gamma_G G_k+\gamma_Q Q_{k1}+\gamma_Q \psi Q_{k2}$	Variable (imposed, wind), γ_Q	0	1.5
Note: Single source is applicable for STR/GEO design situation.				

Persistent and transient design situation – GEO Equation 6.10 (Set C)				
		Action	**Favourable γ**	**Unfavourable γ**
6.10	$E_d=\Sigma\gamma_G G_k+\gamma_Q Q_{k1}+\gamma_Q \psi Q_{k2}$	Permanent (dead, earth), γ_G	1.0	1.0
		Variable (imposed, wind), γ_Q	0	1.3
Note: Single source is applicable for STR/GEO design situation.				

Source:- Valentinos Neophytou

TABLE A1.1/ NA.2 VALUES OF Ψ FACTORS FOR BUILDINGS

Action	J0	J1	J2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1	0.9	0.8
Category F: traffic area,			
Vehicle weight < or = 30 kN	0.7	0.7	0.6
Category G: traffic area,			
30 kN < vehicle weight < or = 160 kN	0.7	0.5	0.3
Category H: roofs	0.6	0	0
Snow loads on buildings (see EN 1991-1-3)	0.5	0.2	0
Wind loads on buildings (see EN 1991-1-4)	0.6	0.2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0.6	0.5	0

Ψ_0 Factor for combination value of a variable action – takes account of reduced probability of simultaneous occurrence of two actions

Ψ_1 Factor for frequent value of a variable action – load exceeded for short period only; used for accidental ULS and reversible limit states (e.g. cracking in pre-stressed concrete)

Summary Table of Partial, combination & reduction factors

Expression	Unfavourable Permanent action	Unfavourable Variable actions		
	Self-weight	Imposed floor loads	Wind loads	Snow loads
6.10	γ_G=1.35	$\gamma_{Q,1}$ =1.5	$\gamma_{Q,2}\psi_{0,2}$ =1.5x06=0.9	$\gamma_{Q,3}\psi_{0,3}$ =1.5x05=0.75
	γ_G=1.35	$\gamma_{Q,2}\psi_{0,1}$ =1.5x07=1.05	$\gamma_{Q,1}$ =1.5	$\gamma_{Q,3}\psi_{0,3}$ =1.5x05=0.75
	γ_G=1.35	$\gamma_{Q,2}\psi_{0,1}$ =1.5x07=1.05	$\gamma_{Q,3}\psi_{0,2}$ =1.5x06=0.9	$\gamma_{Q,1}$ =1.5
Less favourable equations 6.10a & 6.10b				
6.10a	γ_G=1.35	$\gamma_{Q,1}\psi_{0,1}$ =1.5x07=1.05	$\gamma_{Q,2}\psi_{0,2}$ =1.5x06=0.9	$\gamma_{Q,3}\psi_{0,3}$ =1.5x05=0.75
	γ_G=1.35	$\gamma_{Q,2}\psi_{0,1}$ =1.5x07=1.05	$\gamma_{Q,1}\psi_{0,2}$ =1.5x06=0.9	$\gamma_{Q,3}\psi_{0,3}$ =1.5x05=0.75
	γ_G=1.35	$\gamma_{Q,3}\psi_{0,1}$ =1.5x07=1.05	$\gamma_{Q,3}\psi_{0,2}$ =1.5x06=0.9	$\gamma_{Q,1}\psi_{0,3}$ =1.5x05=0.75
6.10b	$\xi\gamma_G$=0.85*1.35	$\gamma_{Q,1}$ =1.5	$\gamma_{Q,2}\psi_{0,2}$ =1.5x06=0.9	$\gamma_{Q,3}\psi_{0,3}$ =1.5x05=0.75
	$\xi\gamma_G$=0.85*1.35	$\gamma_{Q,2}\psi_{0,1}$ =1.5x07=1.05	$\gamma_{Q,1}$ =1.5	$\gamma_{Q,3}\psi_{0,3}$ =1.5x05=0.75
	$\xi\gamma_G$=0.85*1.35	$\gamma_{Q,2}\psi_{0,1}$ =1.5x07=1.05	$\gamma_{Q,3}\psi_{0,2}$ =1.5x06=0.9	$\gamma_{Q,1}$ =1.5

Note: Shaded boxes indicate the *'leading variable action'*.

Source:- Valentinos Neophytou

APPLICATIONS OF EQU 6.10, 6.10A & 6.10B

Example

Dead, Imposed, Wind – all unfavourable

6.10 1.35*D + 1.5*I + 1.5*0.7*W **or** 1.35*D + 1.5*0.7*I + 1.5*W

6.10a 1.35*D + 1.5*0.7*I + 1.5*0.7*W

6.10b 0.85*1.35*D + 1.5*I + 1.5*0.7*W **or** 0.85*1.35*D + 1.5*0.7*I + 1.5*W

Example;

Dead load: 6 kN/m ^{2} Imposed Load: 5 kN/m ^{2}

Eqn 6.10: 1.35 * 6 + 1.5 * 5 = 15.6 kN/m^2

Eqn 6.10a: 1.35 * 6 + 1.5 * 0.7 * 5 = 13.35 kN/m ^{2}

Eqn 6.10b: 0.85 * 1.35 * 6 + 1.5 * 5 = 14.385 kN/m ^{2}

A SAGGED TIMBER JOIST

Combination of Actions for Serviceability Limit State

Characteristic combination Damage to finishes/partitions	
Equation 6.14b	$E_d = G_k + Q_{k,1} + \psi_0 Q_{k,2}$
Frequent combination Comfort criteria such as vibrating machinery	
Equation 6.15b	$E_d = G_k + \psi_1 Q_{k,1} + \psi_2 Q_{k,2}$
Quasi-permanent combination Appearance as effected by Shrinkage/creep	
Equation 6.16b	$E_d = G_k + \psi_2 Q_{k,1}$

Source:- Valentinos Neophytou

INDICATIVE LIMITING VALUES FOR VERTICAL DEFLECTIONS

(Manual of EC0 &EC1, Table D.1)

	Serviceability Limit States Vertical deflections
Serviceability Requirement	Characteristic Combination (Expression 6.14b in EC0) w_{max}
Function and damage to non-structural elements (e.g. partition walls claddings etc) –Brittle -Non-brittle	 **≤L/500 to L/360** **≤L/300 to L/200**
Function and damage to structural elements	**≤L/300 to L/200**

Source:- Valentinos Neophytou

INDICATIVE LIMITING VALUES FOR HORIZONTAL DEFLECTIONS

(Manual of EC0 &EC1, Table D.2)

	Serviceability Limit States **Vertical deflections**
Serviceability Requirement	Characteristic Combination (Expression 6.14b in EC0) w_{max}
Function and damage to non-structural elements -Single storey buildings top of column -Each storey in a multi-storey building -The structure as a whole for a multi-storey building	 **u≤H/300** **u≤H/500 to H/300** **u≤H/500**

Source:- Valentinos Neophytou

ACCIDENTAL & SEISMIC COMBINATIONS

ULS accidental design situation, BS EN 1990: 2002, **6.4.3.3**:

$$E_d = E\left\{\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} A_d \text{"+"} (\psi_{1,1} \text{ or } \psi_{2,1}) Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} Q_{k,i}\right\}$$

EN1990 (6.11b)

EN1990 suggests that the choice of which ψ factor (ψ_1 or ψ_2) to use for the leading variable action depends on the particular situation being considered

ULS seismic design situation, BS EN 1990: 2002, **6.4.3.4**:

$$E_d = E\left\{\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} A_{Ed} \text{"+"} \sum_{i\geq 1} \psi_{2,i} Q_{k,i}\right\}$$

EN1990 (6.12b)

Summary of combination equations Table 1. Factors used for combinations of actions

	Equation	Permanent action $G_{k,j}$	Prestress P	Accidental or seismic A_d	Leading variable action $Q_{k,1}$		Accompanying variable action $Q_{k,i}$	
		$\gamma_{G,j}$	γ_P	A	$\gamma_{Q,1}$	$\psi_{Q,1}$	$\gamma_{Q,i}$	$\psi_{Q,i}$
ULS persistent/transient	6.10	γ_G	γ_P	0	γ_Q	1.0	γ_Q	ψ_0
ULS accidental	6.11b	1.0	1.0	A_d	1.0	ψ_1	1.0	ψ_2
ULS seismic	6.12b	1.0	1.0	A_{Ed}	1.0	ψ_2	1.0	ψ_2
SLS characteristic	6.14b	1.0	1.0	0	1.0	1.0	1.0	ψ_0
SLS frequent	6.15b	1.0	1.0	0	1.0	ψ_1	1.0	ψ_2
SLS quasi-permanent	6.16b	1.0	1.0	0	1.0	ψ_2	1.0	ψ_2

STRUCTURAL LOADS
Imposed Loads - 1

Category of use

(EN1991-1-1:2002,Table 6.1)

Category	Specific Use	Example
A	Area for domestic and residential activities	Rooms in residential buildings and houses bedrooms and wards in hospitals, bedrooms in hotels and hostels kitchens and toilets
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D1))	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1**: Areas in general retail shops **D2**:Areas in departments stores

Source;- Valentinos Neophytou

STRUCTURAL LOADS
Imposed Loads - 2

Imposed loads
(EN 1991-1-1:2002, Table 6.2)

Category Of loaded areas	q_k (kN/m²)	Q_k (kN)
Category A		
-Floors	1.5-2.0	2.0-3.0
-Stairs	2.0-4.0	2.0-4.0
-Balconies	2.5-4.0	2.0-3.0
Category B	2.0-3.0	1.5-4.5
Category C		
-C1	2.0-3.0	3.0-4.0
-C2	3.0-4.0	2.5-7.0
-C3	3.0-5.0	4.0-7.0
-C4	4.5-5.0	3.5-7.0
-C5	5.0-7.5	3.5-4.5
Category D		
-D1	4.0-5.0	3.5-7.0

Imposed load on Roof
(CYS NA EN1991-1-1, Table 6.10)

Sub-category	Actions	Imposed load,	
		q_k (kN/m²)	Q (kN)
H	Roof (inaccessible except for normal maintenance and repair)	0.4	1.0

Source:- Valentinos Neophytou

DEMYSTIFIYING THE EUROCODES

'Course A'
Module 3
THE WIND CODE EN1: PART IV, AS APPLIED TO PV PANELS

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24th /26th March 2015

NATIONAL CLIMATIC DATA TO ASSESS BASIC WIND

the Maltese Islands are definitely windy with only 7.7% of the days, on average, being calm with a wind speed of 0m/s. Most other days have a wind speed between 0.5m/s and 11m/s (1 & 21 knots).

The most common wind in all seasons is the cool N-W (Majjistral) which blows on an average of 19% of the days in a year. Next in frequency are winds blowing for the NNW & W (Punent). All other winds are nearly equally represented and none show any dominance

A gentle/moderate breeze is given by a wind speed of 5m/s, a fresh/strong breeze with whistling of telephone wires heard at 10m/s, with a strong gale causing slight structural damage at 20m/s as noted in table 1

TABLE 1 - THE BEAUFORT LAND SCALE

BEAUFORT FORCE	HOURLY-AVERAGE WIND SPEED (m/s)	DESCRIPTION OF WIND	NOTICEABLE WIND EFFECT
0	<0.45	Calm	Smoke rises vertically
1	0.45 – 1.55	Light Air	Direction shown by smoke drift but not by vanes
2	1.55 – 3.35	Light Breeze	Wind felt on face; leaves rustle; wind vane moves
3	3.35 – 5.60	Gentle Breeze	Leaves and twigs in motion; wind extends a flag
4	5.60 – 8.25	Moderate Breeze	Raises dust and loose paper small branches move
5	8.25 – 10.95	Fresh Breeze	Small trees, in leaf, sway
6	10.95 – 14.10	Strong Breeze	Large branches begin to move; telephone wires whistle
7	14.10 – 17.20	Near Gale	Whole trees in motion
8	17.20 – 20.80	Gale	Twigs break off; personal progress impeded
9	20.80 – 24.35	Strong Gale	Slight structural damage; chimney pots removed
10	24.35 – 28.40	Storm	Trees uprooted; considered structural damage
11	28.40 – 32.40	Violent Storm	Damage is widespread
12	>32.40	Hurricane	Countryside is devastated; only occurs in tropical countries

MALTA GALES

Days with gusts of wind greater than 18m/s (35 knots), termed as gale force winds, occur throughout the year with a maximum frequency in December and a minimum in the months of June to September.

Gales of force 8: 23m/s – 30m/s (45 to 58 knots) are much rarer and only occur in an average of 0.1 days during the months of January, February and October.

In other words, only one day of January, February and October in a period of 10 years has force 8 winds. The strongest gale recorded was in December 1988 at 34m/s (66 knots).

EUROCODE PROVISIONS

To be noted that Eurocode (EN1991-1-4) dealing with Wind Loads stipulates 2 methods of design –

the Simplified Method and the Detailed Method, with the Detailed Method taking notice to vibrational response of the slender structures.

As the majority of buildings require only a simple rule as not sensitive to wind load, the simplified method is sufficient as dynamic effects are negligible.

ITALY WIND CLASSIFICATION

Table 2 – Italy (refer to map in ENV 1991-2-4 for details of zones)

ZONES	DESCRIPTION	CLASSIFICATION
1,2	Northern Italy (25 m/s)	II
3	Central & Southern Italy (27m/s)	II
4,5,6	Sardinia & Sicily (28 m/s)	II
7	Liguria (29 m/s)	II
8,9	Trieste & Islands (31 m/s)	III

Italy was divided into 9 zones with 5 basic wind speeds in the draft Eurocode (EN 1994). These are 10 – min mean speeds with a 50-year return period, ranging from 25 to 31 m/s (Table 2 above)

EUROPE WIND MAP

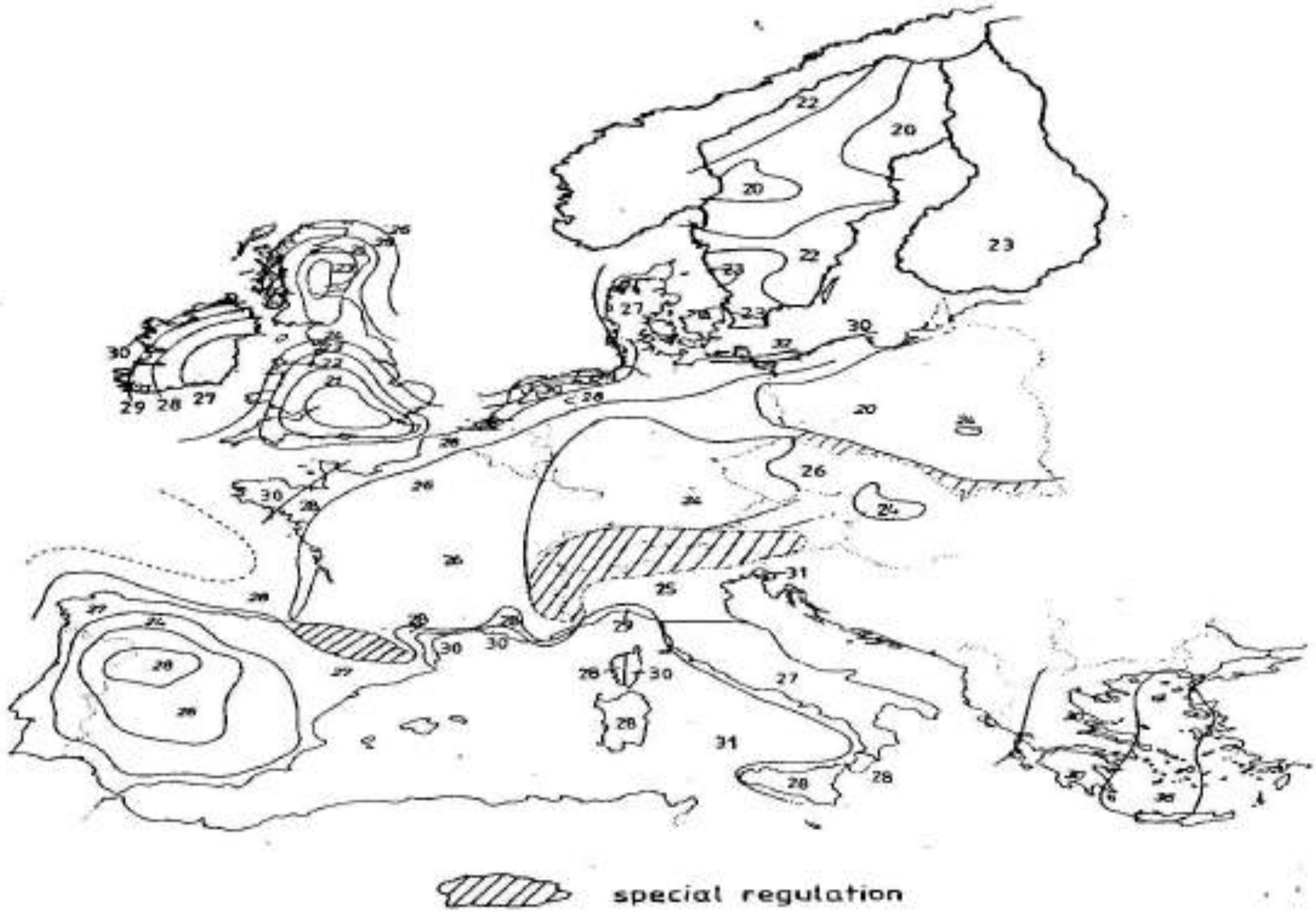

Figure 1. Overlook of the European wind map for basic wind velocities $v_{b,0}$
(indicative values only)

German and UK National Annexes suggest that mean hourly wind speed be multiplied by 1.06 to obtain 10 minute mean wind velocity & by 1.5 to obtain 3 sec gust speed.

MALTA'S DESIGN WIND SPEED?

BICC's unpublished "Structural Integrity Handbook – 2000" gives guidance with regards to Malta's basic wind speed which according to CP 3: Ch V Pt 2 1972 is taken at 47m/s for a 3-sec gust speed.

MSA/EN 1991-1-4 refers to a 10min wind speed at 10m above open country at sea-level which is likely to be exceeded on average only once in 50 years.

The National Annex value for Malta's basic wind speed has not as yet been computed, but it appears that this value approximates to 28m/s, according to Italian date (24.5m/s UK data).

It may be recommended that 90% of the wind pressure as obtained from CP 3: Ch V Pt 2 1972 should comply with requirements of MSA/EN 1991-1-4.

EN 1991-1-4 WIND

1.1 (2) Buildings < 200 m in height

4.2 (1) Fundamental basic wind velocity $v_{b,0}$ (National Annex) characteristic 10 minutes mean wind velocity at 10 m above ground in open country (terrain category II)

4.2 (2) Basic wind velocity $v_b = c_{dir} * c_{season} * v_{b,0}$ (Eq 4.1)

The relationship existing between basic velocity and basic pressure is:

$$q_b = \rho/2 \cdot v_b^2$$

where: ρ = density of air at 1,25 kg/m^3 thus: $q_b = 0.613 \cdot v_b^2$

c_{dir}: Direction factor (recommended value 1)

c_{season}: Season factor (recommended value 1)

4.3.1 Mean wind velocity $v_m(z) = c_r(z) * c_o(z) * v_b$

$c_r(z)$ roughness factor

$c_o(z)$ orography factor – 1 generally except where hills, cliffs result in increased velocities. Annex A3 gives guidance on calculation of $c_o(z)$

4.3.2 Terrain roughness

$c_r(z) = k_r \ln(z/z_0)$ $z_{min} < z < z_{max}$ Eq 4.4

Determination of peak velocity pressure, $q_p(z)$ [BS EN 1991-1-4:2005, 4.5 (1) Note 1]

When orography is not significant $c_o = 1{,}0$:

$q_p(z) = c_e(z)q_b$ for sites in Country terrain; and

$q_p(z) = c_e(z) . c_{e,T} . q_b$ for sites in Town terrain.

The values of exposure factor $c_e(z)$ are given in Figure NA.7 and the values of exposure correction factor for Town terrain $c_{e,T}$ are given in Figure NA.8.

Then the EN goes on to wind pressure & force coefficients

dhi PERITI
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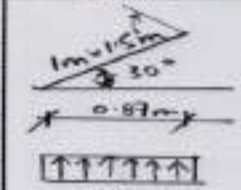
job no.: BICC CPD 2015		sheet no.: F01	
member / location: uplift to PV Panel			
job title: PV - stability to uplift	drg ref.:		
	made by: DHC	date: 02/15	

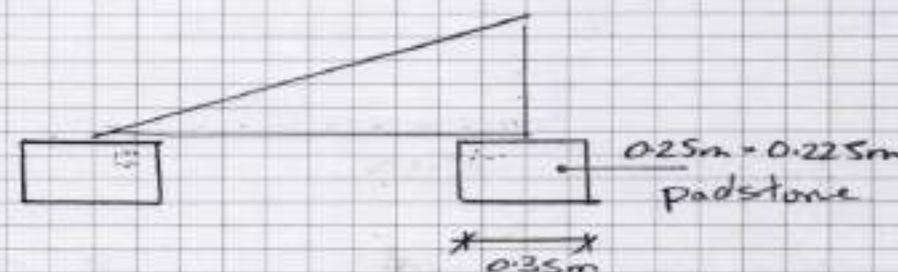
Ref.	Calculations	Outputs
	DESIGN to EN1 - Part IV.	
	basic wind velocity $V_{b,0} \simeq 28\,m/s$ (10m above grd)	
	Basic wind velocity $V_b \Rightarrow c_{dir} \cdot c_{season} \cdot V_{b,0}$	
	$\Rightarrow 1 \cdot 1 \cdot 28 \Rightarrow 28\,m/s$	installed at 15m ht — 2km away from shoreline
	Basic wind pressure $q_b \Rightarrow 0.613 \cdot V_b^2$	
	$\Rightarrow 0.613 \cdot 28^2$	
	$\Rightarrow 480\,N/m^2 \Rightarrow 0.48\,kN/m^2$.	
	PEAK VELOCITY PRESSURE in Country terrain $q_{p(z)} \Rightarrow c_{e(z)}\, q_b$	
	$c_{e(z)} \Rightarrow 2.9$	Fig NA.7
	$q_{p(z)} \Rightarrow 0.48\,kN/m^2 \times 2.9$	
	$\Rightarrow \underline{1.39\,kN/m^2}$	
	in Town Terrain $q_{p(z)} \Rightarrow c_{e(z)}\, c_{e(T)}\, q_b$	
	$c_{e(t)} \Rightarrow 0.96$	Fig NA.8 1km inside town terrain.
	$q_{p(z)} \Rightarrow 2.9 \times 0.96 \times 0.48$	
	$\Rightarrow \underline{1.33\,kN/m^2}$.	
	Force Coeff $c_f \Rightarrow 0.65$ (30° incline)	Romanian study
	$F_{w\uparrow} \Rightarrow c_f \cdot q_{p(z)} \cdot A_{(projected)}$	
	$\Rightarrow 0.65 \cdot 1.39 \cdot (0.87m \times 1.5m/2)$	
	$\Rightarrow 0.59\,kN$ (Country Terrain)	
	$\Rightarrow 0.65 \cdot 1.33\,(0.87m \times 1.5m/2)$	
	$\Rightarrow 0.56\,kN$ (Town Terrain)	



dhi PERITI
STRUCTURAL CONSULTANTS

job title:	PV – stability to uplift
job no.:	BICC CPD 2015
sheet no.:	F02
member / location:	balast to PV panel
drg ref.:	
made by:	PHC
date:	02/15

Ref.	Calculations	Outputs
	ENO – overall equilibrium of structure	
	(EQU) $0.9\,G_k + \underline{1.5}\,Q_k$	
	$1.5\,Q_k \Rightarrow 0.59 \cdot \underline{1.5} \Rightarrow 0.885\,kN$	Country terrain
	$\Rightarrow 0.56 \cdot \underline{1.5} \Rightarrow 0.84\,kN$	Town terrain.
	wt. of a standard padstone 0.225m × 0.25m	
	$wt/m \Rightarrow 0.225 \times 0.25 \times 24\,kN/m^3 \Rightarrow 1.35\,kN/m$	
	length of padstone required to counteract uplift	
	$0.885\,kN / (1.35\,kN/m \times 0.9) \Rightarrow 0.73\,m$	Country Terrain
	$0.840\,kN / (1.35\,kN/m \times 0.9) \Rightarrow 0.69\,m$	Town Terrain



DEMYSTIFIYING THE EUROCODES

'Course A'
Module 4

DESIGN OF A RESTRAINED STEEL BEAM FOR FORMING AN OPENING IN A CELLULAR MASONRY OFFICE BLOCK.
- LOAD PATHS IN STRUCTURAL MASONRY INTRODUCED -

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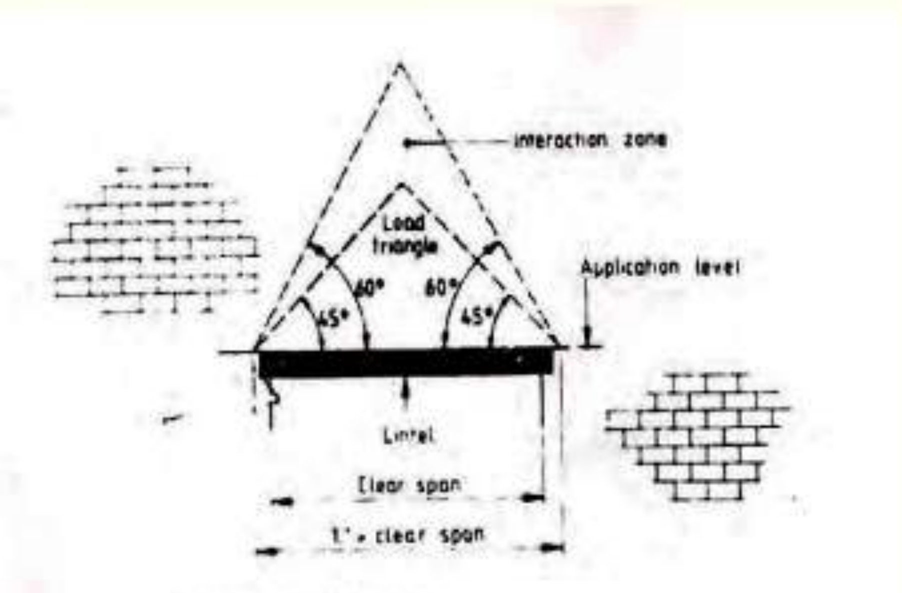
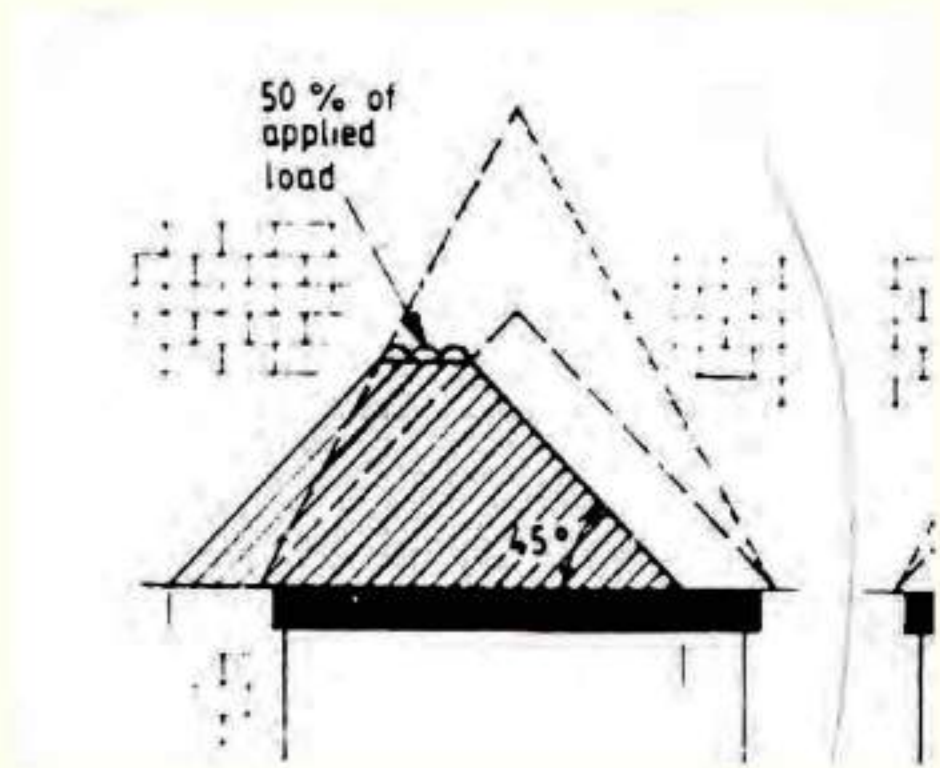
24th /26th March 2015

LOAD TRIANGLE & INTERACTION ZONES

BS5977:PT1:1981 Lintels



Load triangle and interaction zone



THE COMPOSITE ACTION TO BRICK PANEL WALLS SUPPORTED ON RC BEAM – RH Wood BRE 1952 - I

No shear connection appears necessary when the depth of masonry panel is > 0.6.span.

Arching effects come into play via the creation of a composite beams, much deeper than the existing beam, with the provision of a dpm not preventing this latter effect from occurring.

Testing was carried out to RC beams carrying house walls & spanning short bored piles. However, analysis undertaken caters for any spans to be used.

THE COMPOSITE ACTION TO BRICK PANEL WALLS SUPPORTED ON RC BEAM – RH Wood BRE 1952 - II

Method for calculating amount of steel reinforcement in the supporting beam is given at design moment of WL/50 where there are door or window opening near the supports and WL/100 for panels where door and window openings are absent or occur at mid-span.

During testings these moments ranged from WL/960 to WL/130.

- When using this method the ratio of beam depth to span should range between 1/15 & 1/20.

Eg. LOAD TRIANGLE OR COMPOSITE ACTION METHODS

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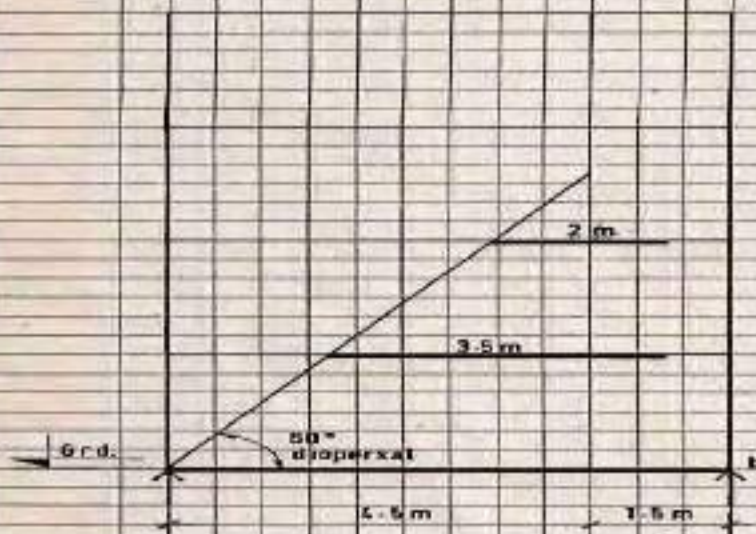
Structural Consultant

Job No.	Sheet No.	Rev.
XX91	Appendix A	

Member / Location LOAD ANALYSIS
Drg Ref.
Made by DHC Date JULY 91 Chd.

Job Title GENERAL

BEAMS AT GRD. FLR
at 4.5 m ≠
DESIGN LOAD taken at
14 KN/m² to cater for
TRANSVERSE PARTITIONS



63KN/m.2.0m/6m => 21.00KN/m

63KN/m.3.5m/6m => 36.75KN/m

14KN/m².4.5m => 63KN/m
20crs 1.35KN/cr=> 37.8 KN/m
158.55KN/m

B.M(beam) => $158.55.6^2/8$ => 713.47KN-m

Calculation by method proposed by Wood (8)
loadings from upper flrs. + b/w
($14KN/m^2$.4.5m + 11crs.1.35KN/cr 1.4) 4flr => 335KN/m
B.M => $63.6^2/8 + 335.6^2/40$
=> 585KN-m

The 2 values for B.M equate for a 6 storey building.

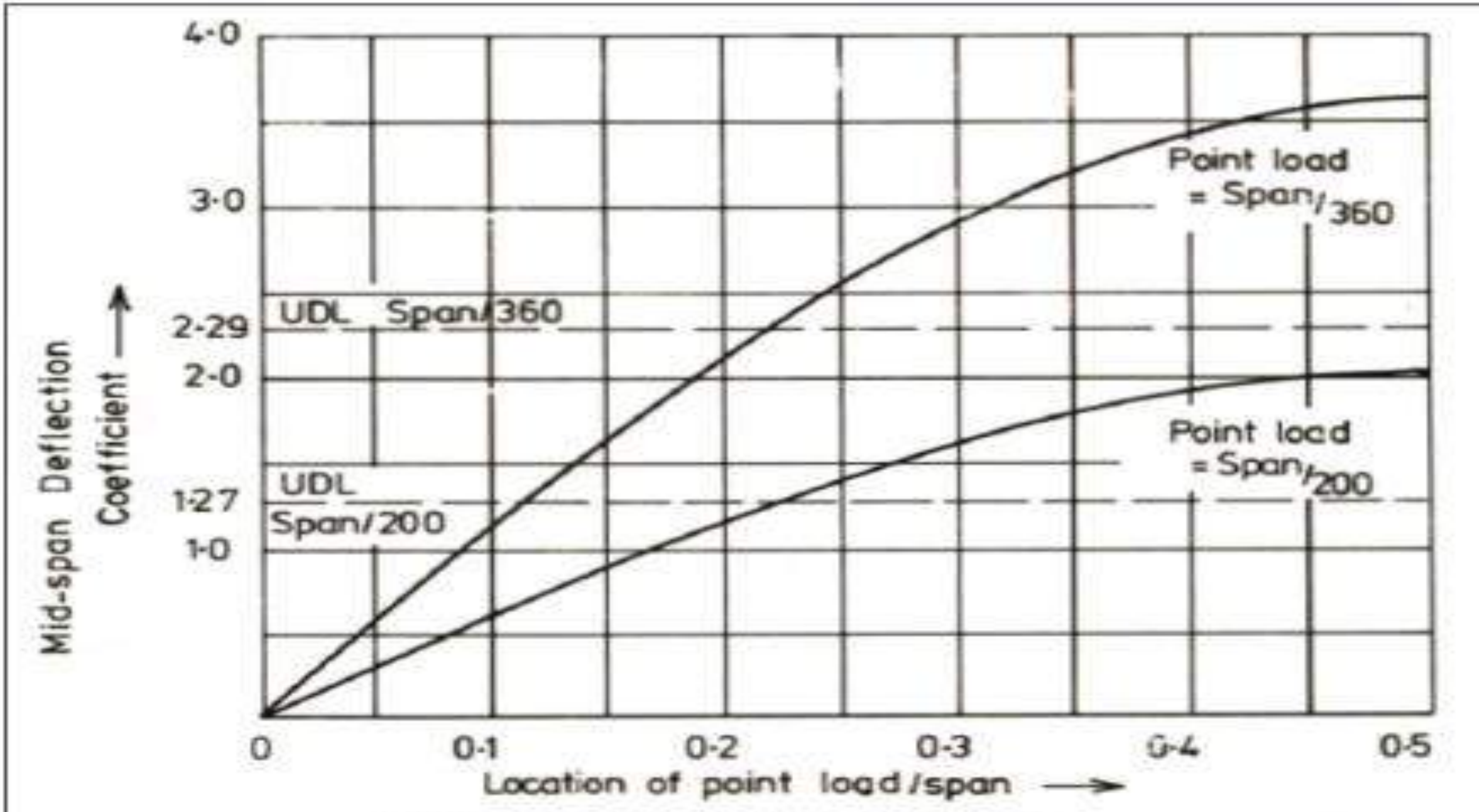
5, Europa Centre, Floriana-Malta. Tel. 233376

Table 1. Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steel.

Steel grade	Thickness t [mm]			
	t<= 40 mm		40 mm <= t <= 100 mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN10025				
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	335	490
EN 10113				
Fe E 275	275	390	255	370
Fe E 355	355	490	335	470

The partial factor of safety for steel is taken at 1.0, unless for resistance of cross-sections in tension to fracture where this is increased to 1.1.

DEFLECTION COEFFICIENT C – to calculate M of I for steel sections in cm^4 (ref BSCA publication)



Deflection coefficient for inertia

Table 2: 'C' deflection coefficient for I cm^4 Calculation for a simple support span condition for udl's & central point loads

Span to deflection ratio	Steel E= 210kN/mm^2		Timber E = 8kN/mm^2	
	udl	Pt. load	udl	Pt. load
1/200-warehouse	1.24	1.98	43.3	52.0
1/360-normal	2.23	3.57	77.9	93.7
1/500-brittle	3.10	4.96	108.2	130.2
1/800-bridges	4.96	7.94	173.2	208.4
1/1000-shear	6.20	9.92	216.5	260.4

SERVICEABILITY DEFLECTION CALCULATION

The required moment of inertia I in cm^4 for a udl is obtained from:

$$I = CWL^2 \qquad \text{i.}$$

The required moment of inertia I in cm^4 for a central point load is obtained from:

$$I = CWL^3 \qquad \text{ii.}$$

Where C is a factor obtained from table 1, dependent on the span/deflection ratio adopted, w is the serviceability load in kN/m, W is the central point load in kN and L is the effective span in m.

The units thus employed are consistent with the value of the constant C in cm^4 adopted.

EXAMPLE: CONSIDER THE DEFLECTION, TO BE LIMITED TO SPAN/200 OF A SIMPLY SUPPORTED STEEL BEAM WITH YOUNG'S MODULUS E =210kN/mm^2.

For a simply supported beam of effective span L in mm as subjected to a uniformly distributed load UDL of w in kN/m, the central deflection in mm is given by: *assuming consistent units throughout in mm and then converting moment of inertia I in cm^4.*

$$\delta = \frac{L}{200} = \frac{5wL^4}{384EI}$$

$$I = \frac{5 \times 200 \ \times w \times L^3}{384E}$$

$$I = \frac{5 \times 200 \times w \times (L \times 1000)^3}{(384 \times E \times 1000) \times 10^4} = CwL^3 \textit{ note that I is given in cm}^4.$$

$$C = \frac{5 \times 200 \times 10^9}{(384 \times 210 \times 10^{7})} = 1.24$$

When using I = CwL^3, w is in kN/m and L is in m, which then calculates I in cm^4 by dividing by 10^4.

The 1.24 C value conforms to the figure quoted in table 2.

Poutrelles normales européennes

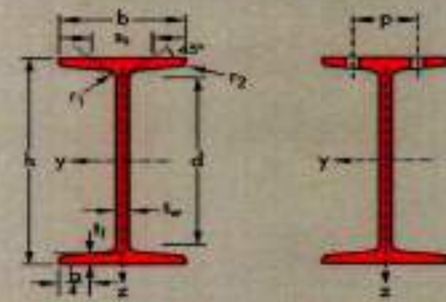
Inclinaison des ailes: 14%
Dimensions: IPN 80 - IPN 550 DIN 1025-1: 1995, NF A 45-209: 1983
IPN 600 DIN 1025-1: 1963
Tolérances: EN 10024: 1995
Etat de surface: conforme à EN 10163-3: 2004, classe C, sous-classe 1

European standard beams

Flange slope: 14%
Dimensions: IPN 80 - IPN 550 DIN 1025-1: 1995, NF A 45-209: 1983
IPN 600 DIN 1025-1: 1963
Tolerances: EN 10024: 1995
Surface condition: according to EN 10163-3: 2004, class C, subclass 1

Europäische Normalträger

Flanschneigung: 14%
Abmessungen: IPN 80 - IPN 550 DIN 1025-1: 1995, NF A 45-209: 1983
IPN 600 DIN 1025-1: 1963
Toleranzen: EN 10024: 1995
Oberflächenbeschaffenheit: Gemäß EN 10163-3: 2004, Klasse C, Untergruppe 1



Désignation Designation Bezeichnung		Dimensions Abmessungen							Dimensions de construction Dimensions for detailing Konstruktionsmaße				Surface Oberfläche	
	G kg/m	h mm	b mm	t_w mm	t_f mm	r_1 mm	r_2 mm	A mm^2 $x10^2$	d mm	Ø	p_{min} mm	p_{max} mm	A_L m^2/m	A_G m^2/t
IPN 80*	5,94	80	42	3,9	5,9	3,9	2,3	7,57	59	-	-	-	0,304	51,09
IPN 100*	8,34	100	50	4,5	6,8	4,5	2,7	10,6	75,7	-	-	-	0,370	44,47
IPN 120*	11,1	120	58	5,1	7,7	5,1	3,1	14,2	92,4	-	-	-	0,439	39,38
IPN 140*	14,3	140	66	5,7	8,6	5,7	3,4	18,2	109,1	-	-	-	0,502	34,94
IPN 160*	17,9	160	74	6,3	9,5	6,3	3,8	22,8	125,8	-	-	-	0,575	32,13
IPN 180*	21,9	180	82	6,9	10,4	6,9	4,1	27,9	142,4	-	-	-	0,640	29,22
IPN 200*	26,2	200	90	7,5	11,3	7,5	4,5	33,4	159,1	-	-	-	0,709	27,04
IPN 220*	31,1	220	98	8,1	12,2	8,1	4,9	39,5	175,8	M 10	50	56	0,775	24,99
IPN 240*	36,2	240	106	8,7	13,1	8,7	5,2	46,1	192,5	M 10	54	60	0,844	23,32
IPN 260*	41,9	260	113	9,4	14,1	9,4	5,6	53,3	208,9	M 12	62	62	0,906	21,65
IPN 280*	47,9	280	119	10,1	15,2	10,1	6,1	61,0	225,1	M 12	68	68	0,966	20,17
IPN 300*	54,2	300	125	10,8	16,2	10,8	6,5	69,0	241,6	M 12	70	74	1,03	19,02
IPN 320*	61,0	320	131	11,5	17,3	11,5	6,9	77,7	257,9	M 12	70	80	1,09	17,87
IPN 340*	68,0	340	137	12,2	18,3	12,2	7,3	86,7	274,3	M 12	78	86	1,15	16,90
IPN 360*	76,1	360	143	13	19,5	13	7,8	97,0	290,2	M 12	78	92	1,21	15,89
IPN 380*	84,0	380	149	13,7	20,5	13,7	8,2	107	306,7	M 16	84	86	1,27	15,12
IPN 400*	92,4	400	155	14,4	21,6	14,4	8,6	118	322,9	M 16	86	92	1,33	14,36
IPN 450*	115	450	170	16,2	24,3	16,2	9,7	147	363,6	M 16	92	106	1,48	12,83
IPN 500*	141	500	185	18	27	18	10,8	179	404,3	M 20	102	110	1,63	11,60
IPN 550*	166	550	200	19	30	19	11,9	212	445,6	M 22	112	118	1,80	10,80
IPN 600*	199	600	215	21,6	32,4	21,6	13	254	485,8	M 24	126	128	1,92	9,89

IPN

Notations pages 205-209 / Bezeichnungen Seiten 205-209

Désignation Designation Bezeichnung	G	I_y	$W_{el,y}$	$W_{pl,y}$ ♦	i_y	A_{vz}	I_z	$W_{el,z}$	$W_{pl,z}$ ♦	i_z	s_s	I_t	I_w	Classification EN 1993-1-1: 2005 Pure bending y-y S235	S355	Pure compression S235	S355	EN 10025-2:2004	EN 10025-4:2004	EN 10225:2001
		Valeurs statiques / Section properties / Statische Kennwerte: axe fort y-y / strong axis y-y / starke Achse y-y					axe faible z-z / weak axis z-z / schwache Achse z-z													
	kg/m	mm^4 $x10^4$	mm^3 $x10^3$	mm^3 $x10^3$	mm x10	mm^2 $x10^2$	mm^4 $x10^4$	mm^3 $x10^3$	mm^3 $x10^3$	mm x10	mm	mm^4 $x10^4$	mm^6 $x10^9$							
IPN 80	5,94	77,8	19,5	22,8	3,20	3,41	6,29	3,00	5,00	0,91	21,6	0,87	0,09	1	1	1	1	✓		
IPN 100	8,34	171	34,2	39,8	4,01	4,85	12,2	4,88	8,10	1,07	25,0	1,60	0,27	1	1	1	1	✓		
IPN 120	11,1	328	54,7	63,6	4,81	6,63	21,5	7,41	12,4	1,23	28,4	2,71	0,69	1	1	1	1	✓		
IPN 140	14,3	573	81,9	95,4	5,61	8,65	35,2	10,7	17,9	1,40	31,8	4,32	1,54	1	1	1	1	✓		
IPN 160	17,9	935	117	136	6,40	10,83	54,7	14,8	24,9	1,55	35,2	6,57	3,14	1	1	1	1	✓		
IPN 180	21,9	1450	161	187	7,20	13,35	81,3	19,8	33,2	1,71	38,6	9,58	5,92	1	1	1	1	✓		
IPN 200	26,2	2140	214	250	8,00	16,03	117	26,0	43,5	1,87	42,0	13,5	10,5	1	1	1	1	✓		
IPN 220	31,1	3060	278	324	8,80	19,06	162	33,1	55,7	2,02	45,4	18,6	17,8	1	1	1	1	✓		
IPN 240	36,2	4250	354	412	9,59	22,33	221	41,7	70,0	2,20	48,9	25,0	28,7	1	1	1	1	✓		
IPN 260	41,9	5740	442	514	10,40	26,08	288	51,0	85,9	2,32	52,6	33,5	44,1	1	1	1	1	✓		
IPN 280	47,9	7590	542	632	11,1	30,18	364	61,2	103	2,45	56,4	44,2	64,6	1	1	1	1	✓		
IPN 300	54,2	9800	653	762	11,9	34,58	451	72,2	121	2,56	60,1	56,8	91,8	1	1	1	1	✓		
IPN 320	61,0	12510	782	914	12,7	39,26	555	84,7	143	2,67	63,9	72,5	129	1	1	1	1	✓		
IPN 340	68,0	15700	923	1080	13,5	44,27	674	98,4	166	2,80	67,6	90,4	176	1	1	1	1	✓		
IPN 360	76,1	19610	1090	1276	14,2	49,95	818	114	194	2,90	71,8	115	240	1	1	1	1	✓		
IPN 380	84,0	24010	1260	1482	15,0	55,55	975	131	221	3,02	75,4	141	319	1	1	1	1	✓		
IPN 400	92,4	29210	1460	1714	15,7	61,69	1160	149	253	3,13	79,3	170	420	1	1	1	1	✓		
IPN 450	115	45850	2040	2400	17,7	77,79	1730	203	345	3,43	88,9	267	791	1	1	1	1	✓		
IPN 500	141	68740	2750	3240	19,6	95,60	2480	268	456	3,72	98,5	402	1400	1	1	1	1	✓		
IPN 550	166	99180	3610	4240	21,6	111,3	3490	349	592	4,02	107,3	544	2390	1	1	1	1	✓		
IPN 600	199	139000	4630	5452	23,4	138,0	4670	434	752	4,30	117,6	787	3814	1	1	1	1	✓		

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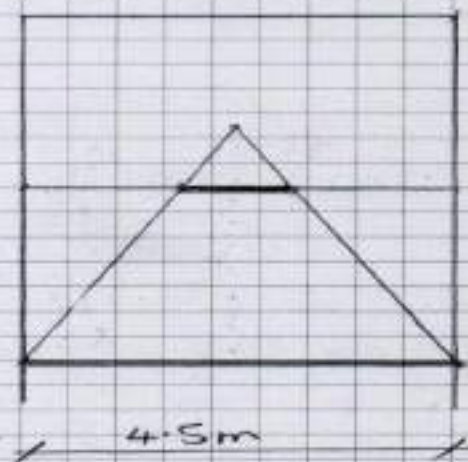
job no.:	BICC CPD2015	sheet no.:	F01
member / location:	REMOVAL of MASONRY WALL		
drg ref.:			
job title:	OPENING IN EXISTING OFFICE		
made by:	DHC	date:	02/15

Ref.	Calculations	Outputs



13m

OFFICE Flr LOADING – STRUCT

		SLS		ULS
self-wt	0.15 . 25	→ 3.6	$\xrightarrow{1.35}$	4.86 kN/m²
finish	0.1 . 18	⇒ 1.8	$\xrightarrow{1.35}$	2.43 kN/m²
L.L	2.5	⇒ 2.8	$\xrightarrow{1.5}$	3.75 kN/m²
		8.2		11.04 kN/m²

EC0 eqn 6.10

9" masonry walling taken at 1.35 kN/m/ful

△ ht taken at 17 as giving a W of

⇒ 17 as . 1.35 kN/m/ful /2 × 4.5m $\xrightarrow{1.35}$ 69.71 kN

BM ⇒ 11.04 . 4m . $\frac{5.5}{4.5}$. 4.5²/8 + 69.7 . 4.5/6

⇒ 136.62 + 52.28 ⇒ 188.90 kN-m

I ⇒ CWL³

⇒ 2.23 (8.2 . 4m . $\frac{5.5}{4.5}$. 4.5³ + 69.7 . 4.5²)

⇒ 11,293 cm⁴

From deflⁿ to be limited to d/span ⇒ 360

C ⇒ 2.23 (table 1)

IPN 260 I_x ⇒ 5,740 cm⁴ × 2 → 11,480 cm⁴ ✓

W_{plx} ⇒ 514 cm³

MR → 514 × 275 N/mm²/1,000 ⇒ 141 kN-m

t < 16mm

f_y ⇒ 275 N/mm²

IPN 260×2 MR ⇒ 282 kN-m > 188.90 kN-m ✓

DEMYSTIFIYING THE EUROCODES

‘Course A’
Module 5

DESIGN OF A TIMBER JOIST SPANNING 6M AT 1.2M SPACING FOR A RESIDENTIAL INTERMEDIATE FLOOR IN AN OLD MILLROOM.

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BICC Eurocodes CPD

24th /26th March 2015

EC5 STANDARDS FOR DESIGNING TIMBER STRUCTURES

It is to be noted that the behavior of timber is not ductile and design therefore is different to steel and concrete. The EC5 standard for designing timber structures is based on a simplified method of limit state, whereby characteristic values of load actions, material characteristics are adjusted by partial coefficients.

Timber structures are analysed using elastic structural analysis techniques in ultimate & serviceability limit states. Thus whilst the ULS loading is adopted as per EC5, the section modulus applied is the elastic not the plastic modulus.

As noted, for the rectangular section $\frac{bd^2}{6}$ the elastic modulus, not the plastic modulus $\frac{bd^2}{4}$ is to be applied

DEFLECTION LIMITS

Ignoring vibration effects, timber deflection to reduce damage to brittle finishes is to be limited to $L/250$, otherwise $L/150$.

The *Manual for the design of timber building structures to EC5*, however, notes that these deflection ratios are there to limit the curvature, rather than the absolute deflection.

Table 1: Updated 'C' deflection coefficient for I cm^4 calculation for a simple support span condition for udl's & central point loads

Span to deflection ratio	Steel E= 210kN/mm^2		Timber E = 8kN/mm^2	
	udl	Pt. load	udl	Pt. load
1/200	1.24	1.98	43.3	52.0
1/360	2.23	3.57	77.9	93.7
1/500	3.10	4.96	108.2	130.2
1/800	4.96	7.94	173.2	208.4
1/1000	6.20	9.92	216.5	260.4

CALCULATING a TIMBER C-deflection constant.

Note that the timber C values for light weight timber floors account for a 33% increase in value over the interpolated E values of the materials, to cater for creep effects.

To allow for creep (Technical Note 2012/11) notes the instantaneous deflection due to permanent loads is to be increased by a factor (1 + k_{def}*), whilst for imposed loads a reduced factor of (1 +*$\psi_{2.1} k_{def}$*) is applied. Shear deflection can also be accounted for by adding 10% to the calculated deflection.*

With k_{def} *given at 0.8 for internal environment and* $\psi_{2.1}$ *taken at 0.3, the calculated deflection has to be increased by 33% to cater for all these effects. To be noted that the effect of the DL on the deflection calculation is considered insignificant.*

For a timber section on a simply supported span, to restrict the deflection to $\frac{Span}{500}$*,*

C works out at: $C = \frac{3.10 \times 1.33 \times 210}{8} = 108.2$

TIMBER VIBRATION CHECK

Overall, excessive vibrations can be avoided by designing floor systems to have fundamental frequencies typically above 8Hz (Mouring & Ellingwood 1993). For office buildings this is limited to 4Hz as minimum, with for stages and dance floors this minimum is increased to 8.4Hz.

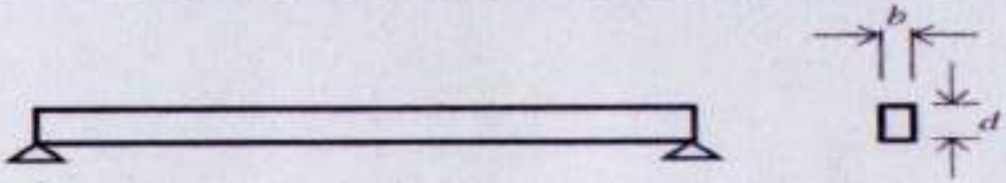
The timber Eurocode EC5 notes that a residential timber floor may be considered to satisfy vibration criteria if the natural frequency of the floor exceeds 8Hz.Further the immediate deflection under a 1kN point, which represents a person walking on the floor should not exceed the deflection (δ) given by:

$$\boldsymbol{\delta = 16{,}500/l^{1.1} \text{ or } 1.8\text{mm if } l < 4\text{m}}$$

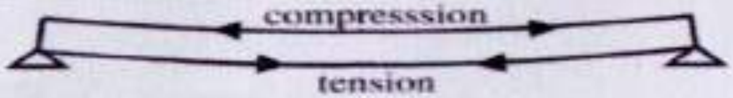
where I is the span given in mm.

On site, the effect of a human footfall, a non-intrusive preliminary cheap testing plan is useful for characterizing the global performance of a floor in terms of human annoyance. A heel drop is generated by an 80kg person arching his heels up by 60mm on the balls of his feet and then free-falling onto the floor. The peak force is about 2.2kN and the duration of the impulse is 50 milliseconds (Nash 1993).The heel drop does not require the assessment of the flooring damping system, as the (viscous) human body absorbs mechanical energy whenever it is in contact with the floor.

ELASTIC & PLASTIC BENDING STRESSES IN A RECTANGULAR BEAM

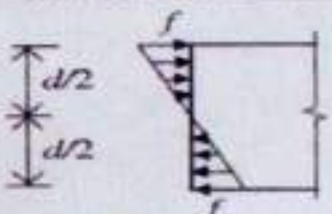


(a) beam unloaded — beam cross-section

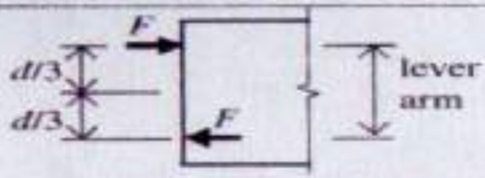


(b) deformation under load

Elastic Behaviour



Stresses (viewed from the side)



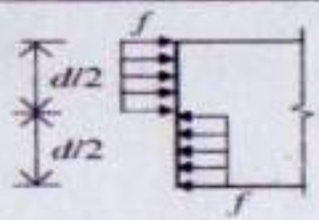
F = average stress × area
= $(f/2) \times (bd/2)$
= $fbd/4$

lever arm = $2 \times d/3 = 2d/3$

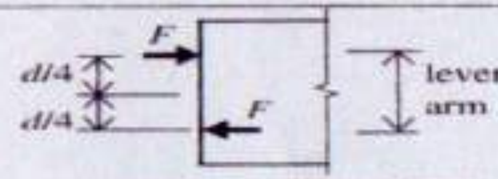
MR = F × lever arm
= $fbd/4 \times 2d/3 = fbd^2/6$
= $f W_{el}$

So $W_{el} = bd^2/6$

Plastic Behaviour



Stresses (viewed from the side)



F = average stress × area
= $f \times (bd/2)$
= $fbd/2$

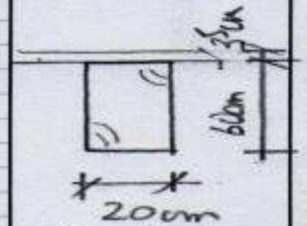
lever arm = $2 \times d/4 = d/2$

MR = F × lever arm
= $fbd/2 \times d/2 = fbd^2/4$
= $f W_{pl}$

So $W_{pl} = bd^2/4$

Ref.	Calculations	Outputs

INTERMEDIATE 6.0m Timber Joist with timber boarding flooring at 1.2m centres



3.5cm timber boards at 1.2m

Softwood timber class C14

Bending $f_{m,k} \Rightarrow 14 N/mm^2$

$E_{0,mean} \Rightarrow 7$

$\rho_{mean} \Rightarrow 3.5 kN/m^3$.

Mezzanine flr. loading (timber flooring) – STRUCT

ECO equ 6.10

		SLS		ULS
Self-wt	0.035 . 3.5 ⇒	0.1225	$\xrightarrow{1.35}$	0.1654
L.L	1.5 ⇒	1.5	$\xrightarrow{1.5}$	2.25
demountable partitions	1.0 ⇒	1.0	$\xrightarrow{1.5}$	1.5
		2.6225 kN/m²		3.915 kN/m²

Mezzanine screeded flooring

screed	0.075×24 ⇒	1.8 kN/m²	$\xrightarrow{1.35}$	2.43 kN/m²
		4.4225 kN/m²		6.345 kN/m²

$BM_{ULt} \Rightarrow (3.915 \times 1.2m).6^2/8 \Rightarrow 21.14 kN\text{-}m$

$BM_{ult} \Rightarrow f \times Z / \gamma_m$

$\gamma_m \Rightarrow 1.3$

$Z \Rightarrow bd^2/6$ (elastic modulus)

$\Rightarrow 0.2 . 0.3^2/6 \Rightarrow 0.003 m^3$

$BM_R \Rightarrow 14,000 kN/m^2 . 0.003 m^3 / 1.3$

$\Rightarrow 32.31 kN\text{-}m > 21.14 kN\text{-}m$ ✓

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job no.: BICC CPD/A. 2015		sheet no.: F02	
member / location: Intermediate Floor			
drg ref.:			
made by: DHK		date: 02/15	

job title: TIMBER BEAM in old Mill Room

Ref.	Calculations	Outputs

<u>Defl Check for span/ δ -> 250</u>

$I \rightarrow CWL^3$

where C from table 1 is given at 43.3

for E => 8 = span/δ - 200

updated C -> (43.3. 7/8 × 250/200)

=> 47.36

$I \rightarrow 63.(2.6225 \times 1.2).6^3 \Rightarrow 32,192\ cm^4$

$I \Rightarrow bd^3/12 \rightarrow 20.30^3/12 \Rightarrow 45,000\ cm^4$ ✓

<u>VIBRATION CHECK</u>

central point load deflection limited to:

1.8mm for L < 4m

$\delta \Rightarrow 16,500/L^{1.1}$ for L > 4m

$\delta - 16,500/6^{1.1} \Rightarrow 1.152\ mm$

for point load table 1 C => 52

span/δ => 6000/1.152 => 5,207

updated C => (52. 7/8) × 5207/200 => 1,185

$I \Rightarrow 1,185.1.6^2 \Rightarrow 42,645\ cm^4 < 45,000\ cm^4$ ✓

The same workings for a screeded flooring notes that the same timber beam size of 20cm × 30cm may be adopted, but on a spacing of 0.75m, instead of the 1.2m spacing adopted.