Demystifying the masonry Eurocode 6 & 8 (seismic)

‘Course C’ Module 1 - Introduction to the Masonry Eurocode

http://www.eurocode6.org/index.htm

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ABOUT EUROCODE 6

Eurocode 6, or to use the more formal title, BS EN 1996, consists of four documents:

BS EN 1996-1-1: Rules for reinforced and unreinforced masonry

BS EN 1996-1-2: Structural fire design

BS EN 1996-2: Selection and execution of masonry

BS EN 1996-3: Simplified calculation methods for unreinforced masonry structures

The four documents which make up BS EN 1996 were first published in 2005 and 2006. The supporting MSA National Annexes were first published 2013.

BS EN 1996-1-1: 2005; Rules for reinforced and & unreinforced masonry

In developing Eurocode 6 a way had to be found to deal with the wide range of masonry units used across Europe. This range not only includes different material such as clay, concrete and stone, but also a variety of configurations based upon the proportion and direction of any holes or perforations, web thickness etc. This has resulted in four grouping of masonry units.

The characteristic compressive strength of masonry is presented in the form of an equation (3.1). This equation includes the normalised strength of the masonry and the strength of the mortar.

The normalised strength relates the compressive strength of the unit determined by test to a standardised shape and moisture content. The designation of mortars has also changed with the need for a declaration based on strength rather than mix proportions. Thus an M12 mortar may be expected to have a strength of 12N/mm^2.

BS EN 1996-1-2:2005 Structural fire design

Fire design is largely in the form of tables. The fire resistance of a loadbearing wall now comprises two values depending upon how highly loaded the wall is and is further enhanced if the wall is plastered.

BS EN 1996-2:2006 Selection and execution of masonry

Part 2 of Eurocode 6 contains limited information of a very general nature on materials and execution. The projected exposure conditions of new masonry play an important part in their durability. Water ingress impacts on the strength of the material and must therefore be considered during the design. Five new exposure classifications MX1 to MX5 are defined.

BS EN 1996-3:2006 Simplified calculation methods for unreinforced masonry structures

Part 3 deals with simplified calculation methods for unreinforced masonry but probably does not produce more cost effective outcomes.

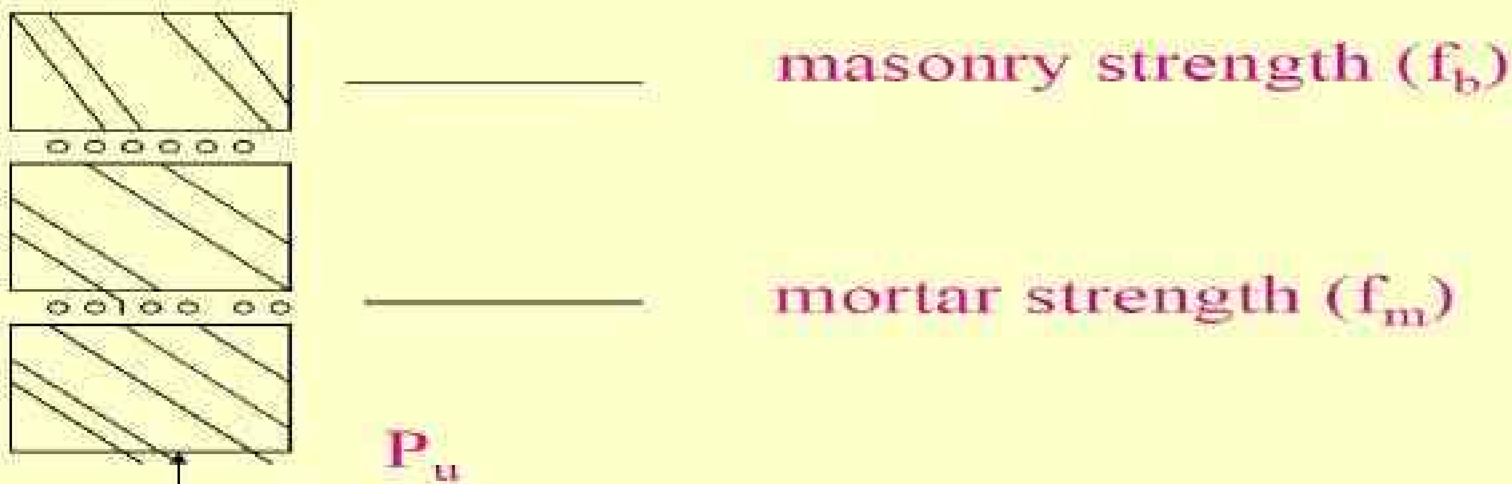
BS EN 1998-1 :2004 E Ch. 9 SPECIFIC RULES FOR MASONRY BUILDINGS

Includes for seismic requirement wrt to min strength of block & mortar.

Thickness of block & effective lengths are reduced further.

Rules for simple masonry buildings also given.

Fig 1 - MASONRY AS A COMPOSITE MATERIAL



P_u as a combination of masonry unit & mortor strengths

$f_k = 0.45 f_b^{0.7} \cdot f_m^{0.3}$ (EC6)

Where f_b is the normalised mean compressive strength of the units in N/mm^2.

*f_m is the compressive strength of mortar in N/mm^2

*f_b => 1.2 X compressive strength (dry) X shape factor

PROPERTIES OF MORTAR

Mortar is the glue that binds masonry together and is typically 10mm thick, although it is possible to have mortar as thin as 0.5mm. Originally it was a clay based mud that eventually became a lime and sand based mixture that remained in use in some form until the early 20th century, when cement based mortar became prevalent.

The change to cement based mortars occurred because they are less weather dependent during construction than lime based mortars. They also gain early strength rapidly, speeding up the building process. Cement based mortars do not self-heal as well as lime based ones, which means that greater attention needs to be paid to movement. Additionally, cement based mortars will force any moisture in the wall to evaporate from the face of the brick, and not from the mortar, which can lead to damage of the brick surface.

When working on historic structures, it is important to ensure that appropriate mortar is used, with probably lime-based mortars required to be specified.

Table 1 - Acceptable assumed equivalent mixes for prescribed masonry mortars

Compressive Strength Class	Prescribed mortars (proportion of materials by volume) (See Note)			
	Cement : Lime : sand with or without air entrainment	Cement : Sand with or without air entrainment	Masonry Cement (inorganic filler) : sand	Masonry cement (lime) : sand
M2	1 : 2 : 8 - 9	1 : 7 - 8	1 : 5,5 - 6,5	1 : 4,5
M4	1 : 1 : 5 - 6	1 : 5 - 6	1 : 4 - 5	1 : 3,5 - 4
M6	1 : 0,5 : 4 - 4,5	1 : 3 - 4	1 : 2,5 - 3,5	1 : 3
M12	1 : 0 - 0,25 : 3	1 : 3	not suitable	not suitable

Note: The number following the M is the compressive strength for the class at 28 days in N/mm2

BS EN 1996-1-1 Design of Masonry Structures divides mortar into four classes: M2, M4, M6 and M12. The lower the class number, the weaker it is, which is inversely proportionate to its flexibility. Class M12 mortar has a compressive strength of 12 N/mm2 and is quite brittle when compared to Class M2 mortar, which has a compressive strength of 2 N/mm2, yet is the most flexible of the cement based mortars. The most commonly used mortar is Class M4 as it offers sufficient flexibility without sacrificing too much in the way of compressive strength (4 N/mm2).

Table 2 - strengths of Maltese Mortars from tests carried out by Debattista (1985)

MORTAR CONSTITUENTS	PROPORTION BY VOLUME	COMPRESSIVE STRENGTH 28DAYS-N/mm^2	FLEXURAL STRENGTH	W/C
Cement, Carolline Sand, Fine Globigerina sand	1:2:10	1.86 (M2)	0.58	3.5
Cement, Carolline Sand, Fine Globigerina Sand	1:2:6	4.48 (M4)	1.30	2.0
Cement, carolline Sand, Coarse Globigerina sand	1:3:12	0.92	0.20	4.4
Cement, White lime, carolline Sand, course globigerina sand	1:1.14:2:4	1.43	0.29	2.5
White lime, fine globigerina sand	1:2	1.32	0.56	2.1

MASONRY MOVEMENT JOINTS

Joints should be provided to minimize the effects of movement cause by drying shrinkage, moisture expansion, temperature variations, creep and settlement.

To be noted that from Table 3, the low movement characteristics of limestone. Compared with most other materials used in the structure of a building, masonry is relatively stiff and brittle.

It does not readily absorb distortions arising from movement or displacement nor readily redistribute high localized stresses.

Table 3 – Guide to the Properties

Properties	Dense concrete blockwork	Lightweight concrete blockwork	Aerated concrete blockwork	Globigerina Limestone	Lower Coralline Limestone
Weight (kN/m^3)	15 - 21	7 - 16	4-9	17	21
Compressive strength (N/mm^2)	7 - 35	3.5 - 10.5	2.8 - 7	15 - 37.5	35 - 75
Flexural strength (N/mm^2)				1.1 - 4.7	
Elastic modulus (kN/mm^2)	10 - 25 or $300f_k$*	4-16	1.7-8	17	
Reversible moisture movement (%)	0.02 – 0.06(-)	0.03 – 0.06 (-)	0.02 – 0.03 (-)	0.01 (+)	
Initial moisture expansion (+) or drying shrinkage (-) (%)	0.02 – 0.06 (-)	0.05 – 0.06(-)	0.05 – 0.09 (-)	0.01	
Coefficient of thermal expansion ($X10^{-6}/^oC$)	6 - 14	7 – 12	8	4	
Long-term natural water absorption (%)				15.6	6.7
Thermal conductivity at 5% moisture content (W/m^oC)	0.6 – 1.3	0.20 – 0.44	0.10 – 0.27	1.3	

Table 4 - DIFFERENT MATERIAL RATES OF THERMAL & MOISTURE

MATERIAL	COEFFICIENT OF THERMAL EXPANSION/ ^0C X 10^{-6}	APPROXIMATE DRYING SHRINKAGE - % IN AIR AT 65% RH
Wood	3.6 to 5.4	2.0 to 4.0 (across the grain) 0.1 (along the grain)
Glass	9.0	-
Steel	10.8	None
Concrete	10.8	0.3 to 0.12
Plastic	17.0	-
Copper	17.2	None
Aluminium	23.0	None
Limestone	4.0	0.1
Mortar	11-13	0.04 – 0.1

Where different materials are connected together or connected to parts of a building not subject to external changes of temperature, care has to be taken in design to accommodate the expansion and contraction of one relative to another, limit and control cracking. Many constructional materials shrink on drying and expand again on wetting, this process being partially or wholly reversible.

MOVEMENT IN FRANKA

To determine the movement likely to take place it is necessary to combine the individual effective movement due to thermal moisture & other effects.

The effective thermal & moisture effects are not directly additive

The moisture expansion of limestone is given at +0.01%

The coefficient of thermal expansion is given at $4/^{o}C \times 10^{-6}$

Considering a 1.0mn length for $\Delta t = 20^{o}C$

Increase in length (mm)

$1000 \times 4 \times 10^{-6} \times 20 = 0.08mm$

representing a 0.008% increase in length

Total temperature + effective moisture movement

= 0.008% + 0.01%/2 = 0.0.13%

Assuming modern filler can compress to 50% for a 10mm movement,

a joint width of 20mm is required at a spacing given by 10mm/0.13mn

= 75m spacing

MOVEMENT IN CONCRETE B/W

This 75m spacing is to be compared to the 6m – 10mm joint spacing specified for concrete hollow blockwork due to its high irreversible drying shrinkage.

For reinforced concrete hollow blockwork this joint spacing may be increased to

12m for an L/h = 2

18m for an L/h = 4

Non-loaded unrestrained parapet walls should be provided with twice the amount of movement provision.

Table 5 - Maximum horizontal distance l_m between vertical movement joints in external unreinforced non-load bearing walls [1]

Type of Masonry	l_m m
Clay masonry	12
Calcium silicate masonry	8
Aggregate concrete and manufactured stone masonry	6
Autoclaved aerated concrete masonry	6
Natural Stone masonry	20 [2]

Note 1: The value for masonry walls containing bed joint reinforcement conforming to EN 845-3 may be increased. Guidance may be obtained from the manufactures of bed joint reinforcement.

Note 2: When using this figure, movement joints should be located at not more than 8 m from the corner.

Sizes of vertical chases and recesses in masonry, allowed without calculation Table 6

	Chases and recesses formed after construction of masonry		Chases and recesses formed during construction of masonry	
Thickness of wall mm	max depth mm	max width mm	minimum wall thickness remaining mm	max width mm
85 – 115	30	100	70	300
116 – 175	30	125	90	300
176 – 225	30	150	140	300
226 – 300	30	175	175	300
> 300	30	200	215	300

NOTE 1 The maximum depth of the recess or chase should include the depth of any hole reached when forming the recess or chase.

NOTE 2 Vertical chases which do not extend more than one third of the storey height above floor level may have a depth up to 80 mm and a width up to 120 mm, if the thickness of the wall is 225 mm or more.

NOTE 3 The horizontal distance between adjacent chases or between a chase and a recess or an opening should not be less than 225 mm.

NOTE 4 The horizontal distance between any two adjacent recesses, whether they occur on the same side or on opposite sides of the wall, or between a recess and an opening, should not be less than twice the width of the wider of the two recesses.

NOTE 5 The cumulative width of vertical chases and recesses should not exceed 0,13 times the length of the wall.

Sizes of horizontal and inclined chases in masonry, allowed without calculation – Table 7

Thickness of wall mm	Maximum depth mm	
	Unlimited length	Length ≤ 1 250 mm
85 - 115	0	0
116 - 175	0	15
176 - 225	10	20
226 - 300	15	25
over 300	20	30

NOTE 1 The maximum depth of the chase should include the depth of any hole reached when forming the chase.

NOTE 2 The horizontal distance between the end of a chase and an opening should not be less than 500 mm.

NOTE 3 The horizontal distance between adjacent chases of limited length, whether they occur on the same side or on opposite sides of the wall, should be not less than twice the length of the longest chase.

NOTE 4 In walls of thickness greater than 175 mm, the permitted depth of the chase may be increased by 10 mm if the chase is machine cut accurately to the required depth. If machine cuts are used, chases up to 10 mm deep may be cut in both sides of walls of thickness not less than 225 mm.

NOTE 5 The width of chase should not exceed half the residual thickness of the wall.

FIRE RESISTANCE OF FRANKA

Building stones have low thermal diffusivity. Hence temperatures rise, within body of wall is correspondingly low. The high temperature would not exist within a moderate depth below surface. A steep temperature gradient exists between the outer and inner parts causing splitting. Splitting is more pronounced in hollow blocks.

For temperatures up to 400^{o}C pink or reddish brown coloration occurs for Franka containing $Fe_2\ O_3$. Free of $Fe_2\ O_{3,}$ a greyish colour develops with the depth of coloration rarely exceeding 20mm.

Around 600^{o}C, colour disappears & calcinations occur with depth rarely exceeding 1cm. Calcinated limestone has a dull earthly appearance.

FURTHER TO FIRE RESISTANCE OF FRANKA

No significant reduction in crushing strength occurs up to 400/450^{o}C.

At 600^{o}C the masonry retains 60% of original strength thus it is expected safe to re-build on existing walls except those stressed in tension.

Moulded glass soften or flow at 700^{o}C/800^{o}C cast iron forms drops or sharp edges are rounded at 1,100^{o}C/1,200^{o}C, 650^{o} for aluminium 1,000^{o} for bronze.

Table 8 Manufactured stone masonry minimum thickness of separating non-loadbearing separating walls (Criteria EI) for fire resistance classifications

Row number	material properties normalized strength [N/mm²] gross density ρ [kg/m³]	Minimum wall thickness (mm) t_F for fire resistance classification EI for time (minutes) $t_{fi,d}$					
		30	60	90	120	180	240
1	**Group 1 units**						
1.1	Mortar: general purpose, thin layer, lightweight 1 200 ≤ ρ ≤ 2 200						
1.1.1 1.1.2		50 (50)	70 / 90 (50/70)	90 (70)	90 / 100 (70 / 90)	100 (90/100)	100/170 (100/140)

Table 9 Manufactured stone masonry minimum thickness of separating loadbearing single-leaf walls (Criteria REI) for fire resistance classifications

row number	material properties normalized strength [N/mm²] gross density ρ [kg/m³]	Minimum wall thickness (mm) t_F for fire resistance classification REI for time (minutes) $t_{fi,d}$					
		30	60	90	120	180	240
1	**Group 1 units**						
1.2	Mortar: general purpose, thin layer, lightweight 1 200 ≤ ρ ≤ 2 200						
1.2.1 1.2.2	$\alpha \leq 1{,}0$	90/170 (90/140)	90/170 90/140	90/170 (90/140)	100/190 (90/170)	140/240 (100/190)	150/300 (100/240)
1.2.3 1.2.4	$\alpha \leq 0{,}6$	70/140 (60/100)	70/140 (70/100)	90/170 (70/100)	90/170 (70/140)	100/190 (90/170)	140/240 (100/190)

Fire resistance classifications

In the tables the thickness referred to is that of the masonry itself, excluding finishes, if any. The first row of pairs of rows defines the resistance for walls without a suitable surface finish (see 4.2(1)). Values in brackets ( ) in the second row of pairs of rows are for walls having an applied finish in accordance with 4.2(1), of minimum thickness 10mm on both faces of a single leaf wall, or on the fire-exposed face of a cavity wall.[deleted rendering or plaster again, ref to 4.2(1) is enough]

Masonry made with units having high precision dimensions and having unfilled vertical joints more than 2 mm, but less than 5mm, wide, may be assessed using the tables providing render or plaster of at least 1 mm thickness is used on at least one side. In such cases, the fire resistance periods are those given for walls without a layer of surface finish. For walls having vertical joints with a thickness less than or equal to 2 mm, no additional finish is required in order to be able to use the Tables appropriate to walls with no applied finish.

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'Course C' Module 2 - Design of Vertical Wall Elements

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Classification of masonry units & workmanship class, most applicable in Malta

2 levels of attestation of conformity are recognised:
Category I and Category II.

• Category I masonry units, which have a declared compressive strength with a probability of failure to reach it not exceeding 5%.

• Category II masonry units, which are not intended to comply with the level of confidence of Category I units, relates to Malta.

5 classes of execution control are also recognised,
Class 2 applicable to Malta.

Applicable Material Partial Safety Factors – Table 1

	Material	γ_M Class 1	2	3	4	5
	Masonry made with:					
A	Units of Category I, designed mortar[a]	1,5	1,7	2,0	2,2	2,5
B	Units of Category I, prescribed mortar[b]	1,7	2,0	2,2	2,5	2,7
C	Units of Category II, any mortar[a, b, e]	2,0	2,2	2,5	2,7	3,0
D	Anchorage of reinforcing steel	1,7	2,0	2,2	2,5	2,7
E	Reinforcing steel and prestressing steel	1,15				
F	Ancillary components[c d]	1,7	2,0	2,2	2,5	2,7
G	Lintels according to EN 845-2	1,5 to 2,5				

a Requirements for designed mortars are given in EN 998-2 and EN 1996-2.

b Requirements for prescribed mortars are given in EN 998-2 and EN 1996-2.

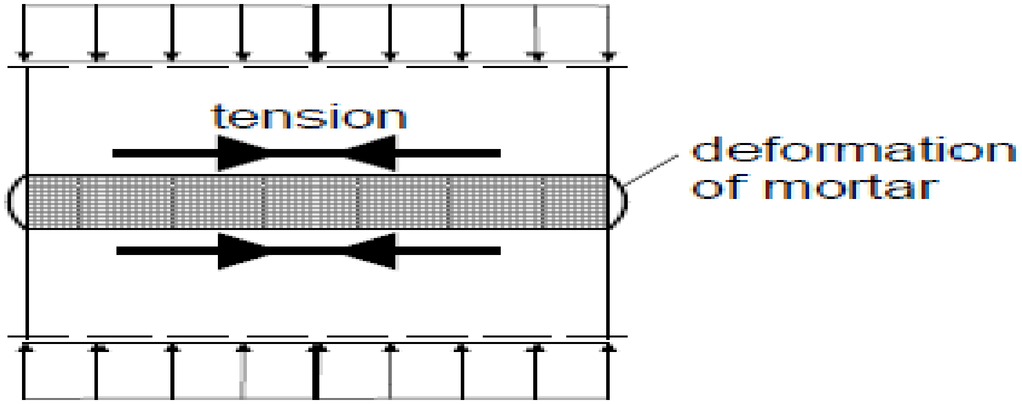
c Declared values are mean values.

d Damp proof courses are assumed to be covered by masonry γ_M.

e When the coefficient of variation for Category II units is not greater than 25 %.

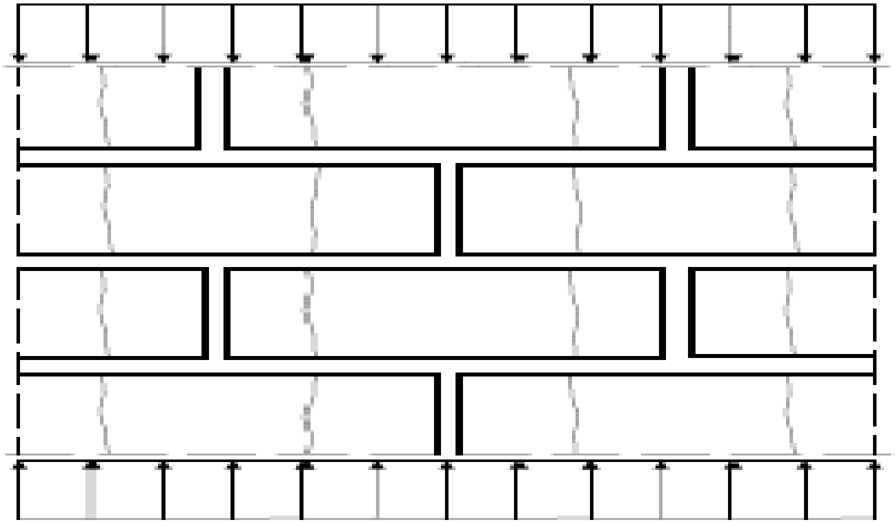
Failure mode of masonry

In masonry under compressive load the transversal strain of the mortar in the bed joints is normally larger than that of the units.



This causes transversal tensile stresses in the units.

This leads to the effect that the compressive strength of masonry is limited by the tensile strength of the units.



When the compressive load is increased up to the bearing capacity, the units will crack normal to the mentioned tensile stresses.

So the compressive strength of masonry mainly depends on:

— the tensile strength of the masonry units
(units with holes and also grip slots are disadvantageous in this regard),

— the compressive strength of mortar
(as a higher strength of mortar reduces the transverse strain).

Note:
Otherwise certain deformability of mortar is advantageous, so that masonry may accommodate induced deformations, for example resulting from unequal settlements, without cracking.

Additional parameters influencing on the strength of masonry:

— Masonry bond:

Walls, in which every unit goes through the whole wall thickness are stronger than walls which are built up of several units, laying side by side over the wall thickness.
In the latter case a sufficient number of through units is very important.

— Thickness of bed joints:

Too thick bed joints are unfavourable.
Therefore their thickness is limited (normally from 8 mm to 15 mm).

— Number of bed joints over the height of the wall:

Blocks are better than smaller units in this respect.

Characteristic compressive strength of masonry other than shell bedded – Figure 1

- Alternative i: based on test acc. EN 1052-1 by project / database
  - in table format or
  - in formula format: $f_k = K\, f_b^{\alpha}\, f_m^{\beta}$ with K, α and β to be given in the NA
- Alternative ii: formulae as given:

$f_k = K \cdot f_b^{0,7} \cdot f_m^{0,3}$ (3.2) masonry with general purpose mortar

$f_k = K\, f_b^{0,85}$ (3.3) thin bed masonry with CS and AAC units

$f_k = K\, f_b^{0,7}$ (3.4) thin bed with group 2 and 3 clay blocs

f_b is not taken to be greater than 75 N/mm^2 when units are laid in general purpose mortar
f_m is not taken to be greater than 20 N/mm^2 nor greater than 2 f_b when units are laid in general purpose mortar;

Table 2 – Values of *K* for use with general purpose, thin layer and lightweight mortars

Masonry Unit		General purpose mortar	Thin layer mortar (bed joint $\geq 0,5$ mm and ≤ 3 mm)	Lightweight mortar of density	
				$600 \leq \rho_d \leq 800$ kg/m^3	$800 < \rho_d \leq 1\,300$kg/m^3
Clay	Group 1	0,55	0,75	0,30	0,40
	Group 2	0,45	0,70	0,25	0,30
	Group 3	0,35	0,50	0,20	0,25
	Group 4	0,35	0,35	0,20	0,25
Calcium Silicate	Group 1	0,55	0,80	‡	‡
	Group 2	0,45	0,65	‡	‡
Aggregate Concrete	Group 1	0,55	0,80	0,45	0,45
	Group 2	0,45	0,65	0,45	0,45
	Group 3	0,40	0,50	‡	‡
	Group 4	0,35	‡	‡	‡
Autoclaved Aerated Concrete	Group 1	0,55	0,80	0,45	0,45
Manufactured Stone	Group 1	0,45	0,75	‡	‡
Dimensioned Natural Stone	Group 1	0,45	‡	‡	‡

‡ Combination of mortar/unit not normally used, so no value given.

Definition of Declared & Normalised Compressive Strength f_b

- Declared values – The mean value of a test sample must not be less than the declared value

– E.g. declared compressive strength for masonry units

- Mean compressive strength of 10 units must be greater than the declared value
- Any individual result must not be less than 80% of the declared value

- Normalised mean compressive strength – Conditioning regimes
- Air dry and 6% m_c – used as reference method
- Oven dry m_c – X 0.8
- Immersion in water m_c – X 1.2

– Shape factor λ

$f_b = m_c$ X (manufactured declared compressive strength) X λ.

Table 3 – Normalised strength

DESIGN NOTE 5:Shape factors for Normalised Strength

Shape Factors for Normalised Strength

Height mm	Width mm (Historically called Thickness for some UK masonry units) 50	75	90	100	115	125	140	150	190	200	215	225	≥250
40	0.80	0.75	0.72	0.70									
50	0.85	0.80	0.77	0.75	0.74	0.73	0.71	0.70					
65	0.95	0.90	0.87	0.85	0.82	0.80	0.77	0.75	0.71	0.70	0.69	0.68	0.65
100	1.15	1.08	1.03	1.00	0.97	0.95	0.92	0.90	0.82	0.80	0.79	0.78	0.75
140	1.27	1.22	1.18	1.16	1.13	1.11	1.08	1.06	0.98	0.96	0.95	0.94	0.91
150	1.30	1.25	1.22	1.20	1.17	1.15	1.12	1.10	1.02	1.00	0.99	0.98	0.95
190	1.42	1.37	1.34	1.32	1.29	1.27	1.24	1.22	1.14	1.12	1.11	1.10	1.07
200	1.45	1.40	1.37	1.35	1.32	1.30	1.27	1.25	1.17	1.15	1.14	1.13	1.10
215	1.48	1.43	1.40	1.38	1.35	1.33	1.30	1.28	1.20	1.18	1.16	1.15	1.12
≥250	1.55	1.50	1.47	1.45	1.42	1.40	1.37	1.35	1.27	1.25	1.22	1.20	1.15

Linear interpolation between values is permitted.

NOTE: The BS EN 771 series requires masonry unit manufactures to declare the strength of the unit in the air dry condition and it is not necessary for the designer to apply any correction for the moisture content of the units at test

Compression Crushing Strengths of Local Masonry Units

Cachia (1985) noted in testing franka crushing values of:

Dry testing 15.0 – 37.84N/mm^2.

Saturated testing: 7.95 – 22.0N/mm^2

The stress in the N-direction (ie normal to the stratification) is generally higher than in the P-direction.

On average the strength in the P-direction is 8% less.

This value is lower in the fully saturated state than in the dry state, Loss of strength is on average 39%.

In general the compressive strength decreases as one goes down in the quarry.

Table 4 – Blockwork Characteristic Strength f_k Data

Blockwork type mm	*Average Characteristic Strength N/mm2*	*Average Coefficient of variation %*	*Period*	*Best Year %*	*Worst Year %*
115	5.86	18.23	1991 1994	1992 13.37%	1991 25.29%
150	7.51	16.25	1991 1996	1993 12.58%	1991 20.28%
225 singlu	7.50	13.01	1991 -1996	1993 9.43%	1996 19.61%
225 dobblu	8.67	12.93	1991 -1996	1995 10.92%	1996 14.86%

Source: Grech (1997)

An important concept to introduce is shell bedding, with mortar laid on the 2 outer edges only. The design strength should be reduced by the ratio of the bedded area to the gross area.

Characteristic compressive strength of shell bedded masonry

(1) The characteristic compressive strength of shell bedded masonry, made with Group 1 and Group 4 masonry units, may also be obtained from 3.6.1.2, provided that:
– the width of each strip of mortar is 30 mm or greater;
– the thickness of the masonry is equal to the width or length of the masonry units, so that there is no longitudinal mortar joint through all or part of the length of the wall;

– the ratio *g/t is not less than 0,4;*
– K is taken from 3.6.1.2 when g/t = 1,0 or K is taken as half of those values when g/t = 0,4, with intermediate values obtained by linear interpolation.

g is the total of the widths of the mortar strips;
t is the thickness of the wall.

(2) The characteristic compressive strength of shell bedded masonry made with Group 2 and Group 3 masonry units, may be obtained from 3.6.1.2, provided that the normalised mean compressive strength of the units, f_b, *used in the equation is that obtained from tests on units tested in accordance* with EN 772-1 for shell bedded units.

Properties of concrete infill

(1)P The characteristic compressive strength and shear strength of concrete infill shall be determined from tests on concrete specimens.

NOTE: test results may be obtained from tests carried out for the project, or be available from a database.

(2) Where test data are not available the characteristic compressive strength, f_{ck}, *and the* characteristic shear strength, f_{cvk}, of concrete infill may be taken from table 4 below:-

Table 5. — Characteristic strengths of concrete infill

Strength class of concrete	C12/15	C16/20	C20/25	C25/30, or stronger
f_{ck} (N/mm^2)	12	16	20	25
f_{cvk} (N/mm^2)	0,27	0,33	0,39	0,45

Where to go – Load table for EC6? – Table 6

Material	*Crushing strength N/mm2*	*Mortar type IV KN/m*	*Mortar type III KN/m*	*Mortar type II KN/m*
225 franka	20	537	602	
225 qawwi	75			1640
180 franka	20	493	551	
150 franka	20	469	522	
225 block dobblu	8.5	283	319	
225 block singlu	7	268	297	
150 block	7	217	246	
115 block	5	163	185	
225 infilled block	15	457	522	551
225 infilled block with 12mm bar at 225 centres	15			944
225 infilled block with 20mm bar at 225 centres	15			1206

The above table demonstrates the low load bearing capacity of concrete b/w of crushing strength 7N/mm^2, as being approximately 50% for equivalent thick franka of crushing strength 20N/mm^2. (Source – Structural Integrity Handbook BICC 2001)

Wall Geometry

- Effective height, h_{ef}
  - $h_{ef} = \rho_n h$,

    $\rho_n = 0.75$ or 1.0

    depending on top and bottom restraints, further reductions are permitted for vertical restraints.

- Effective thickness, t_{ef}
  - $t_{ef} = (t_1^3 + t_2^3)^{1/3}$, t_1 & t_2 are actual thickness of each leaf

- Slenderness ratio
  - $h_{ef}/ t_{ef} \leq 27$ (EC 6) ≤ 15 (EC 8)

Stiffness coefficient

When a wall is stiffened by piers the effective thickness is enhanced by using the following equation:

$t_{ef} = r_{tt}$

where

t_{ef} = *effective thickness*

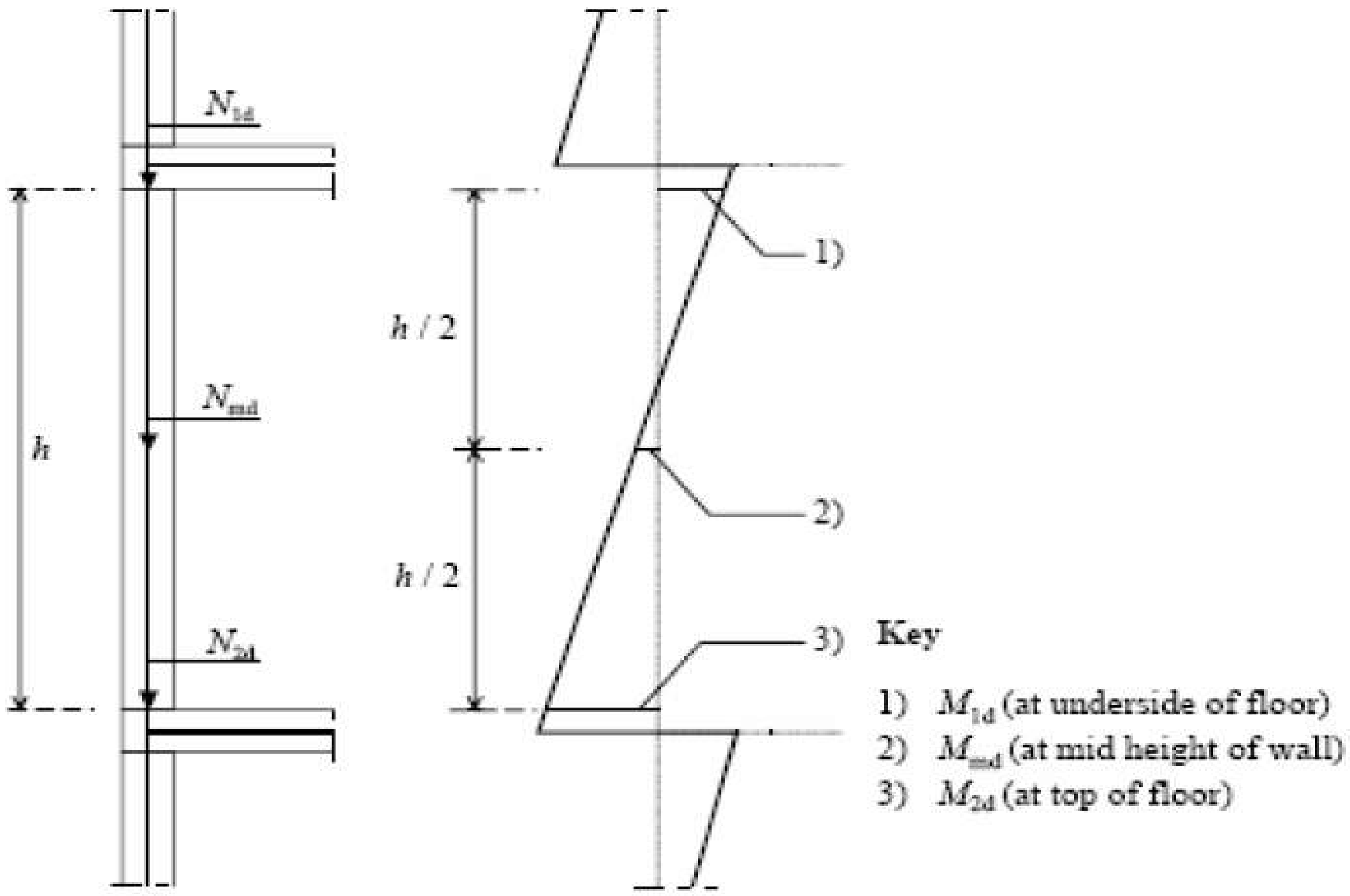
r_{tt} = coefficient obtained from Table 7 Below

t = *thickness of the wall*

Stiffness coefficient, r_{tt}, for walls stiffened by piers			
Ratio of pier spacing	Ratio of pier thickness to actual		
(centre to centre)	thickness of wall to which it is bonded		
to pier width	1	2	3
6	1.0	1.4	2.0
10	1.0	1.2	1.4
20	1	1	1
Note: Linear interpolation is permitted in this table			

Eccentricity – Figure 2

- Assessed at top, middle and bottom of wall using a sub-frame analysis



Eccentricity Continued – Figure 3

- At top or bottom of wall
  - $e_i = \frac{M_{id}}{N_{id}} + e_{he} + e_{init} \geq 0.05t$
  - M_{id}= design moment at top or bottom of wall
  - N_{id}= design vertical load at top or bottom of wall
  - e_{he}= load related eccentricity at top or bottom of wall from lateral loads
  - e_{init}= h_{ef}/450 when SR≤ 27

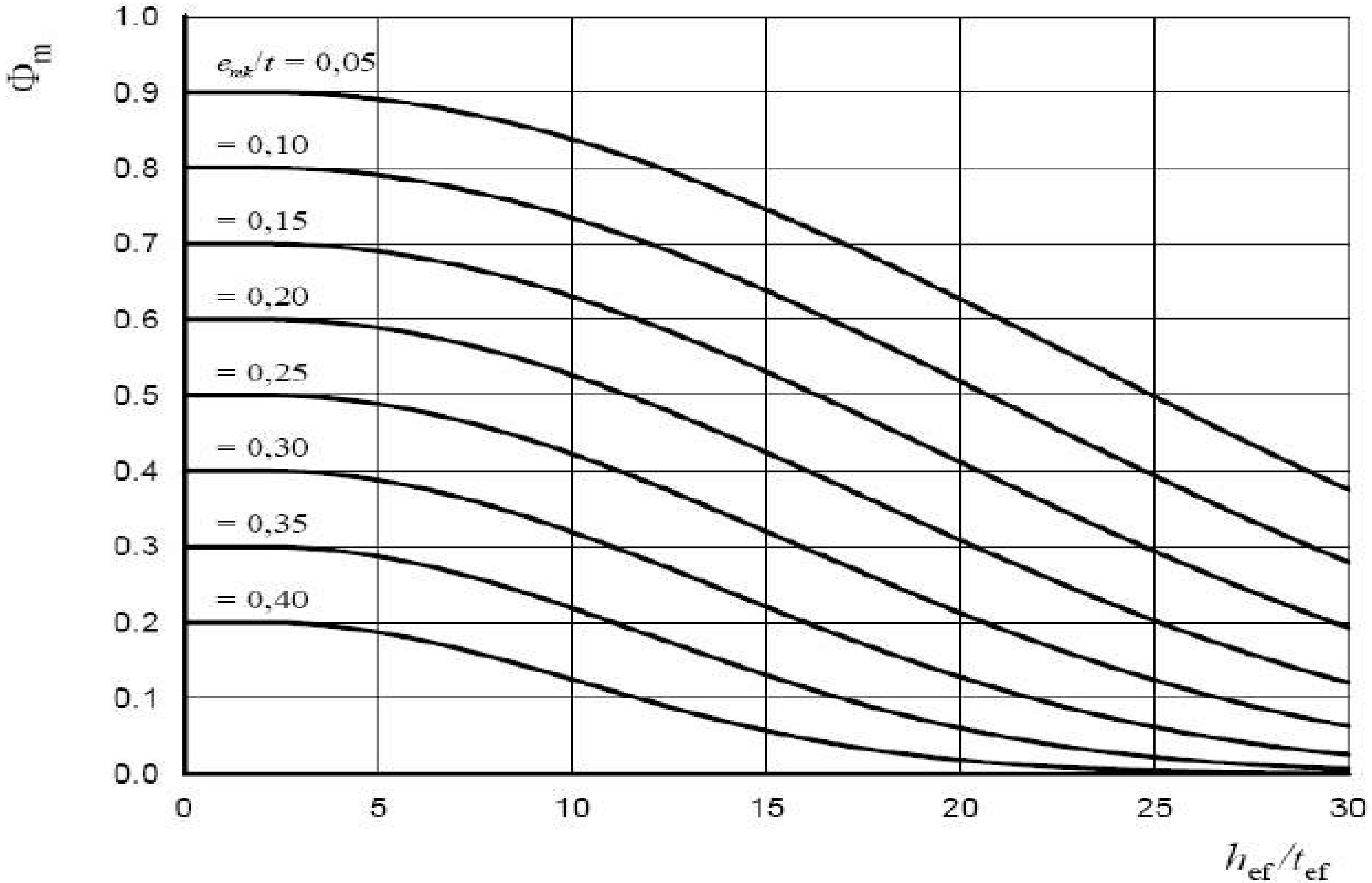
Eccentricity Continued – Figure 4

- Middle of wall
  - $e_{mk} = e_m + e_k \geq 0.05t$
  - $e_m = \dfrac{M_{md}}{N_{md}} + e_{hm} \pm e_{init}$
  - $e_k = 0.002\Phi_{\infty} \dfrac{h_{ef}}{t_{ef}} \sqrt{te_m}$
    - Usually taken as 0

The creep eccentricity, e_k, may be taken as zero – for all walls built with clay and natural stone units.

Capacity Reduction Factor – Figure 5

- $\Phi_i = 1 - 2e_i/t$ top or bottom of wall
- Φ_m = use graphs



Vertical Load Resistance - Figure 6

- $N_{RD} = \Phi \, t \, f_d$
  - $f_d = f_k/\gamma_m$
- Therefore
  - $N_{RD} = \Phi \, t \, f_k/\gamma_m$ - EC6

    $= \beta \, t \, f_k/\gamma_m$ - BS 5628 -1

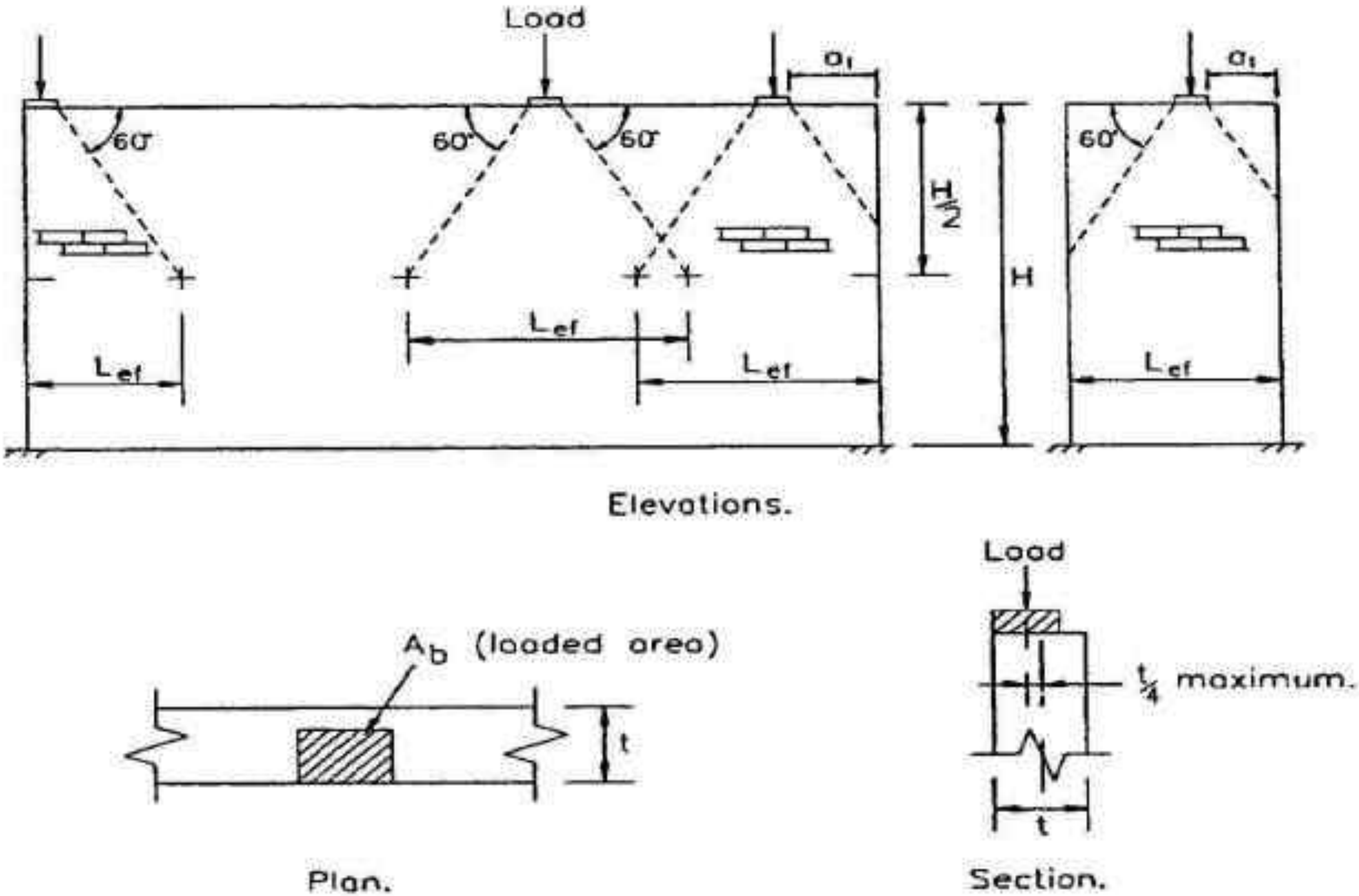
Bearings under Concentrated loads

(1) Concentrated loads should bear on a wall a minimum length of 90 mm or such distance as is required from calculations according to 6.1.3, whichever is the greater.

(2)Where the concentrated load is applied through a spreader beam of adequate stiffness and of width equal the thickness of the wall, height greater than 200 mm and length greater than # X the bearing length of the load, the design value of the compressive stress beneath the concentrated load should not exceed 1,5 f_d.

(3) For walls built with Groups 2, 3 and Group 4 masonry units and when shell bedding is used, it should be verified that, locally under the bearing of a concentrated load, the design compressive stress does not exceed the design compressive strength of masonry, f_d (i.e. β is taken to be 1.0).

Concentrated loads under Bearings – 1 – Figure 7



Concentrated loads under Bearings – 2 – Figure 8

Locally under the bearing of the concentrated load, the design compressive stress shall not exceed the following values:

- Walls built with Group 1 masonry units (not shell bedded):

$$\frac{f_k}{\gamma_M}\left[(1+0{,}15\,x)\cdot\left(1{,}5-1{,}1\frac{A_b}{A_{ef}}\right)\right] \geq \frac{f_k}{\gamma_M}$$

$$\leq 1{,}25\,\frac{f_k}{\gamma_M} \quad \text{where } x = 0$$

$$1{,}5\,\frac{f_k}{\gamma_M} \quad \text{where } x = 1{,}0$$

- All other cases:

$$\frac{f_k}{\gamma_M}$$

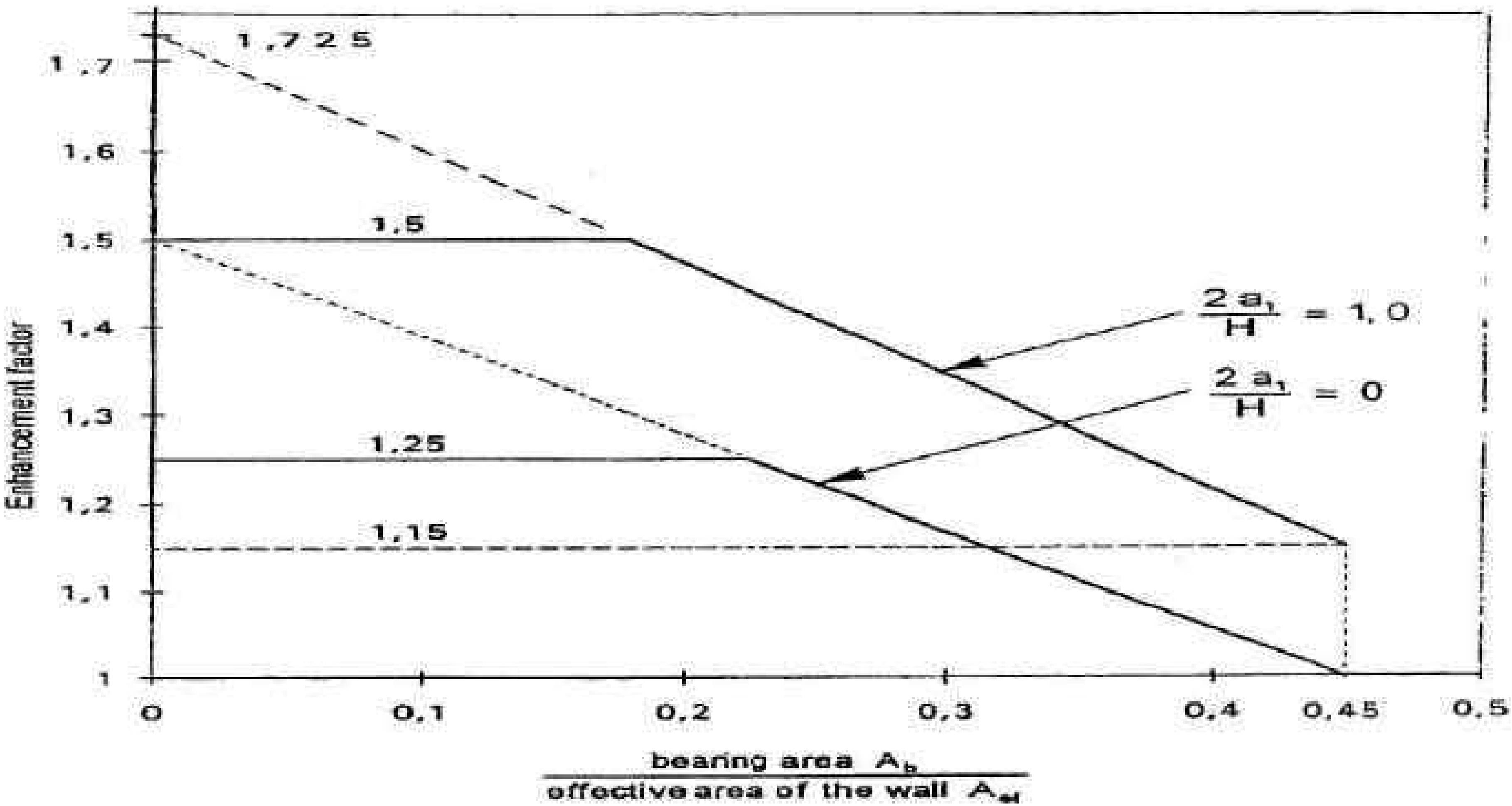
where: $x = \dfrac{2a_1}{H}$

A_b is the bearing area, not taken to be greater than $0{,}45\,A_{ef}$;

A_{ef} is the effective area of the wall $L_{ef}\,t$ (see figure 4.4).

Concentrated Loads under Bearings – 3 – Figure 9

Graph showing the enhancement factor as given in 4.4.8: Concentrated loads under bearings.



Ref	Calculations	Outputs

230 franka × 265mm depth table 3 shape factor $X \Rightarrow 1.13$
180 franka × 265mm depth table 3 shape factor $X \Rightarrow 1.04$

f_b (230 franka) $\Rightarrow 1 \times 20\,N/mm^2 \times 1.13 \Rightarrow 22.6\,N/mm^2$
f_b (180 franka) $\Rightarrow 1 \times 20\,N/mm^2 \times 1.04 \Rightarrow 20.8\,N/mm^2$
f_b (230 b/w double) $\Rightarrow 1 \times 8.67\,N/mm^2 \times 1.13 \Rightarrow 9.8\,N/mm^2$

$f_k \Rightarrow K \cdot f_b^{0.7} \cdot f_m^{0.3}$
230/F/M2 $f_k \Rightarrow 0.45 \cdot 22.6^{0.7} \cdot 2^{0.3} \Rightarrow 4.91\,N/mm^2$
230/F/M4 $f_k \Rightarrow 0.45 \cdot 22.6^{0.7} \cdot 4^{0.3} \Rightarrow 6.05\,N/mm^2$
180/F/M2 $f_k \Rightarrow 0.45 \cdot 20.8^{0.7} \cdot 2^{0.3} \Rightarrow 4.64\,N/mm^2$
230/bw/M2 $f_k \Rightarrow 0.45 \cdot 9.8^{0.7} \cdot 2^{0.3} \Rightarrow 2.74\,N/mm^2$

Design Axial Loads* kN/m for various wall types

	Mortar	kN/m (EC2)	kN/m - BS (table 4)
230 franka	M2	513	557
230 franka	M4	632	602
180 franka	M2	485	493
225 bricks double	M2	286	283

* Axial load kN/m
$f_k \cdot 0.23m / \gamma_m$

where $\gamma_m \Rightarrow 2.2$ (table 1 - Category 2/ Class 2 unit)

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job no: BICC CPD/2 2015		sheet no: F02	
member / location: 180 mm Franka block - 3m ht			
drg ref:			
made by: DHC		date: 05/15	

job title: STRENGTH of WALL PANEL

Ref.	Calculations	Outputs

$h_{ef} = \rho_n h = 0.75 \times 3000 \Rightarrow 2250\,mm$

$t_{ef} \Rightarrow t \Rightarrow 180\,mm$

$h_{ef}/t_{ef} \Rightarrow 2250/180 \Rightarrow 12.5$

Eccentricities - top & bottom of wall

$e_i \Rightarrow (M_{id}/N_{id}) + e_{he} \pm e_{init} \geq 0.05t$

$M_{id}/N_{id} \Rightarrow 0$ concentric load capacity required

$e_{he} \Rightarrow 0$ no horizontal loads

$e_{init} \Rightarrow h_{ef}/450 \Rightarrow 2{,}250/450 \Rightarrow 5\,mm$

$e_i \Rightarrow 0 + 0 + 5\,mm = 5\,mm < 0.05t = 9\,mm$

$\Phi_i \Rightarrow 1 - 2(e_i/t)$

$\Rightarrow 1 - 2(0.05) \Rightarrow \underline{0.9}$

Middle of wall

$e_m \Rightarrow M_{md}/N_{md}) + e_{hm} \pm e_{init} \geq 0.05t$

$M_{md}/N_{md} \Rightarrow 0$

$e_{hm} \Rightarrow 0$

$e_{init} \Rightarrow h_{ef}/450 \Rightarrow 2{,}250/450 \Rightarrow 5\,mm$

$e_{mk} \Rightarrow 0 + 0 + 5 \Rightarrow 5\,mm < 0.05t \Rightarrow 9\,mm$

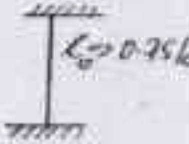
therefore $e_{mk} \Rightarrow 0.05t$

$\Phi_m \Rightarrow 0.79$ (from fig 5) - governs design

Vertical load capacity

$\Rightarrow \Phi\, t\, f_k/\gamma_m$

$\Rightarrow 0.79 . 0.23 . 4.64/2.2 \Rightarrow 383\,kN/m$



Demystifying the masonry Eurocode 6 & 8 (seismic)

'Course C' Module 3 - Arching, shear stresses & Stability Moments in Masonry

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7th /9th July 2015

UNREINFORCED MASONRY WALLS SUBJECT TO LATERAL LOADING

Figure 1: Walls arching between supports

- resistance:
  - design load resistance under arch action
  - design strength of the support
  - analysis may be based on a three-pin arch

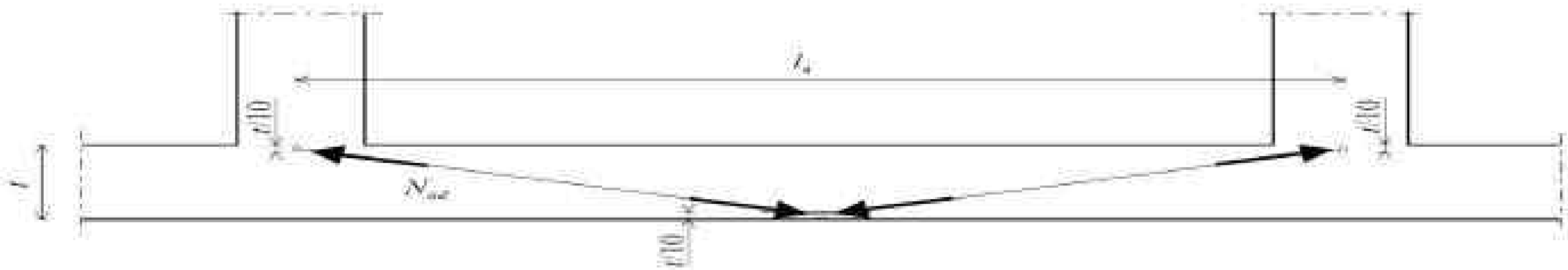
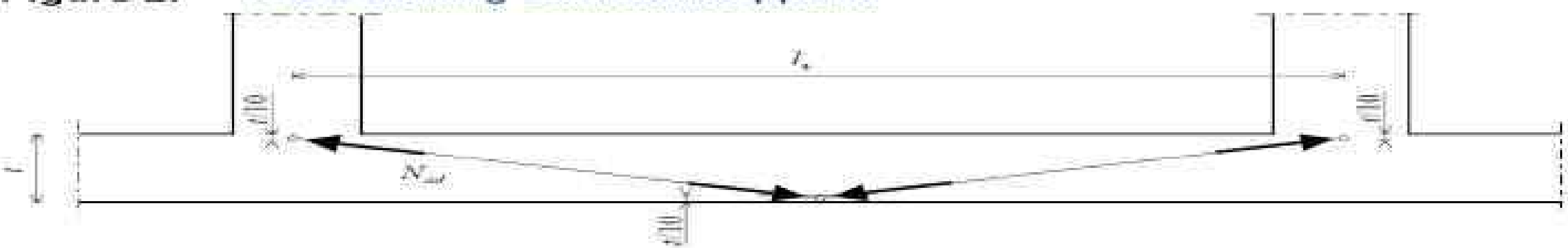


Figure 2: Walls arching between supports



- arch rise: $r = 0{,}9\,t - d_a$
- maximum design arch thrust per unit length: $N_{ad} = 1{,}5\, f_d \dfrac{t}{10}$
- with small lateral deflection: $q_{lat,d} = f_d \left(\dfrac{t}{l_a} \right)^2$
  - design value of vertical stress ≥ 0,1 N/mm²
  - slenderness ratio ≤ 20

Source Rob Van Der Plijm – Apr 2009

The arch rise is given by:

$$0{,}9\,t - d$$

where:

d is the deflection of the arch under the design lateral load; it may be taken to be zero for walls having a length to thickness ratio of 25 or less.

The maximum design arch thrust per unit length of wall may be assumed to be:

$$1{,}5\,\frac{f_k}{\gamma_M}\,\frac{t}{10}$$

and where the lateral deflection is small, the design lateral strength is given by:

$$q_{lat} = \frac{f_k}{\gamma_M}\left[\frac{t}{L}\right]^2$$

where:

q_{lat} is the design lateral strength per unit area of the wall.

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job no.: BICL CPD/C 2015	sheet no.: F01
member / location: Intermediate floor of the mill room	
drg ref.:	
made by: DHC	date: 05/15

job title: MASONRY XOROK DESIGN

Ref.	Calculations	Outputs
	Xorok slab taken at 45mm thick x 230mm wide	
	Compressive strength	
	$f_k \Rightarrow K f_b^{\alpha} f_m^{\beta}$	
	$\Rightarrow 0.45 . 20^{0.7} . 4^{0.3}$	
	$\Rightarrow 5.55\ N/mm^2$	
	$\delta \Rightarrow 0.65$ for 45mm x 230mm	
	$f_d \Rightarrow f_k\, \delta / \gamma_m$	
	$\Rightarrow 5.55 . 0.65 / 2.2 \Rightarrow 1.64\ N/mm^2$	
	$q \Rightarrow f_d \left(\frac{t}{L_0}\right)^2 \Rightarrow 1640 . \left(\frac{0.045}{1}\right)^2 \Rightarrow 3.32\ kN/m^2$	
	Residential loading - $1.5\ kN/m^2$	
6.10	$0.125 . 18 . 1.35 + 1.5 . 1.5 \Rightarrow 5.28\ kN/m^2$	
6.10a	$0.125 . 18 . 1.35 + 0.75 . 1.5 \Rightarrow 4.61\ kN/m^2$	
6.10b	$0.85 . 0.125 . 18 . 1.35 + 1.5 . 1.5 \Rightarrow 4.83\ kN/m^2$	
	so $4.61\ kN/m^2 > 3.32\ kN/m^2$ ✗ ?	

Characteristic of shear strength of Masonry

Shear strength
- Principle: determination by test, but no test available

Initial shear strength f_{vko}
- Principle: determination by test.
- EN 1052-3 (masonry)
- EN 1052-4 (dpc layers)

Shear strength of masonry will filled head joints
- $f_{vk} = f_{vko} + 0,4\,\sigma_d$
- $f_{vk} \leq 0,065\,f_b$ or $f_{vk} \leq f_{vlt}$

Shear strength of masonry with unfilled head joints
- $f_{vk} = 0,5\,f_{vko} + 0,4\,\sigma_d$
- $f_{vk} \leq 0,045\,f_b$ or $f_{vk} \leq f_{vlt}$

Shear strength of shell bedded masonry
- $f_{vk} = \frac{g}{t} f_{vko} + 0,4\,\sigma_d$
- Not greater than with unfilled bed joint

Table 1 — Values of the intitial shear strength of masonry, f_{vko}

Masonry units	f_{vko} (N/mm²)			
	General purpose mortar of the Strength Class given		Thin layer mortar (bed joint ≥ 0,5 mm and ≤ 3 mm)	Lightweight mortar
Clay	M10 - M20	0,30	0,30	0,15
	M2,5 - M9	0,20		
	M1 - M2	0,10		
Calcium silicate	M10 - M20	0,20	0,40	0,15
	M2,5 - M9	0,15		
	M1 - M2	0,10		
Aggregate concrete	M10 - M20	0,20	0,30	0,15
Autoclaved Aerated Concrete	M2,5 - M9	0,15		
Manufactured stone and Dimensioned natural stone	M1 - M2	0,10		

Tests carried out on franka (Saliba 1990) gives an unconfined shear strength varying from 2.2 to 3.85N/mm^2

MINOR MASONRY ARCH DESIGN

The arch is likely to adopt a statically determinate 3-hinge formation. The 3-hinge method simplifies the application of engineering judgment in the assessment of simple masonry arches.

Treat the arch as a simply supported beam of the same span. Determine the vertical reactions under the loads concerned and the bending moments due to the horizontal thrust H, i.e.

$Hy = M$

Where y is the maximum height of the arch above the line of the horizontal thrust, at a point distance from the support.

Once the horizontal thrust has been determined the maximum compressive stress in the masonry is determined from

$F_m = H/bd$

Where fm is the characteristic compressive stress in the masonry which should not exceed the masonry bearing stress, given above at 1.5_{fk}.

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job no: BICC CPD/C 2015		sheet no:	F02
member / location: bm intermediate arch to			
drg ref: f11 m mill room			
made by:		date:	05/15

job title: SIMPLE MASONRY ARCH DESIGN

Ref.	Calculations	Outputs

Load from above

$N \Rightarrow 10m \cdot 1.35 kN/m/ld \times 1.35 + 5.25 kN/m^2 \cdot 6m/2 \Rightarrow 40.35 kN/m$

$f_{vk} \Rightarrow f_{vko} + 0.4\,\sigma_d$

$\sigma_d = 40.35 kN/m / 0.23 \cdot 1m \Rightarrow 0.175 N/mm^2$

$f_{vk} \Rightarrow 0.1 + 0.175 \cdot 0.4 \Rightarrow 0.175 N/mm^2$

$BM \Rightarrow 4.61 kN/m^2 \cdot 6^2/8 \times 1.2m \Rightarrow 24.89 kN\text{-}m$

$H \Rightarrow BM/L_a \Rightarrow 24.89/0.55m \Rightarrow 45.25 kN$

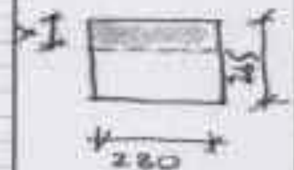
$x \Rightarrow 45.25/(2.32 \times 1.5)/0.23 \Rightarrow 5.65 cm$

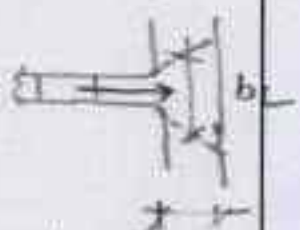
$b_L \Rightarrow 0.23m + 2 \cdot 0.45/2 \Rightarrow 0.68 m$

$h_{pier} \Rightarrow \frac{H}{2} \Big/ \frac{f_v}{\gamma_m} \Big/ \left(t + \frac{b_L}{2}\right)$

$\Rightarrow \frac{45.25}{2} \Big/ \frac{0.175}{2.2} \Big/ 0.68$

$\Rightarrow 0.418 m$





$h \Rightarrow 0.45 m$

Walls subjected to lateral earth pressure

(1)P Walls subject to lateral earth pressure shall be designed using acceptable engineering principles.

Note: the flexural strength of masonry f_{xk1} should not be used in the design of walls subjected to lateral earth pressure.

FREE STANDING WALLS

Walls over 1.80m in height should be referred to a perit for checking.

Table 2 - Height to thickness ratio related to wind speed.

Wind Pressure KN/m2	*Height to thickness ratio*
0.30	Not exceeding 10
0.60	7
0.85	5
1.15	4

When damp-proof courses incapable of developing adequate bond are used, the height to thickness ratio should not exceed 75% of the appropriate value in table 18. The use of such dpc's are not generally recommended.

The following rule of thumb may be followed for wall panels 225mm thick subjected to wind speed of 47m/s. the maximum wall area for a panel fixed on 3 sides is to be limited to 20m^2 and to 16m^2 for a panel pinned on one or more of the three supported sides.

EARTH RETAINING WALLS

Ideally retaining walls should have an impervious lining on the side adjacent to the retained material to prevent moisture damaging the mortar and the masonry. All earth-retaining walls should be provided with weep holes of 50mm minimum diameter at 3.00m centers to allow for adequate drainage. An alternative is drainage at the rear of the wall with open joints (French drain), surrounded by crushed stone.

Table 3 Height to thickness ratios for retaining walls

Height of retained material - m	***Height to thickness ratio***
0.90	4
1.20	3.75
1.50	3.5
1.80	3.25

The above details are based on no surcharge and slope of retained earth not greater than 1:10. unless walls are constructed in a flexible mortar, i.e. not containing cement but lime, movement joints are necessary if cracking is to be avoided.

EARTH RETAINING WALLS (cont.)

The economy of constructing masonry retaining walls is to be stressed, but above a height of 2.00m reinforced masonry retaining walls tend to become more economical, with a stepped reinforced masonry retaining wall offering further economies above a height of 4.00m.

Provided that the top of the wall is unrestrained, the earth pressure will be equal to the active pressure. It is recommended that walls in cohesive soils are never designed for a pressure (KN/m2) of less than 4.8 times the height in metres of the retained material. In addition to the active earth pressure, allowance must be made for water pressure where it develops and any surcharge on the retaining side of the wall.

As partial safety factors are included in the limit state approach the factors of safety for stability analysis are not required, other than in the sliding analysis where a factor of safety of 2 is to be adopted.

Design moment of resistance of free-standing wall without flexure

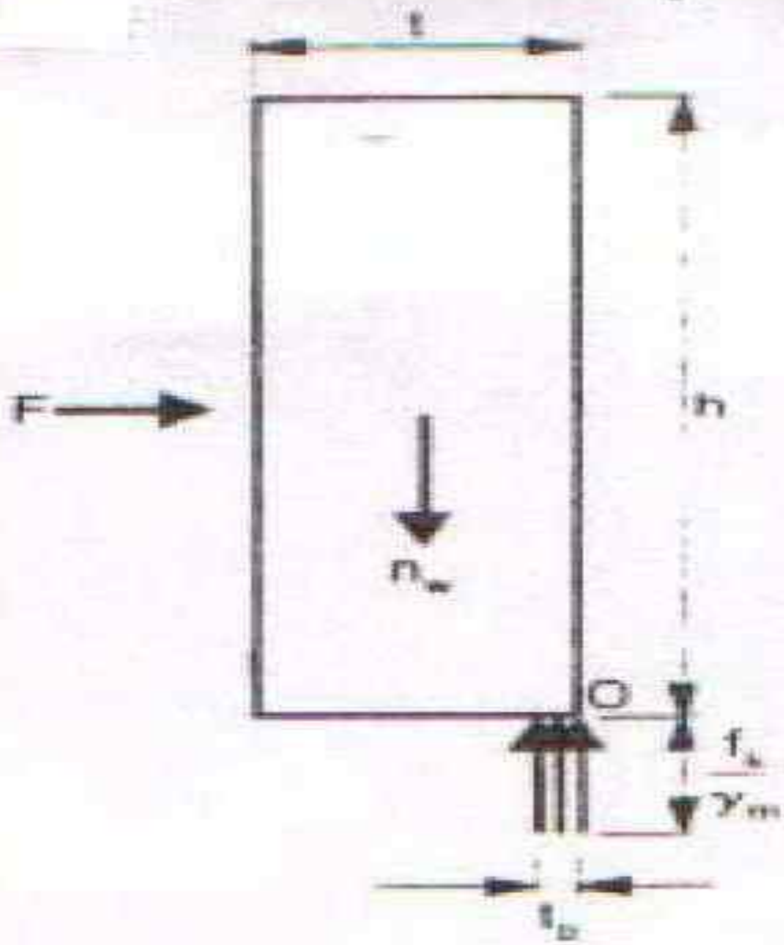


Figure 3

the design vertical load n_w is resisted by a rectangular stress block of width t_b

on the point of failure by rotation about O, under the action of the applied load F, the stress block is limited by the design compressive strength f_k/γ_m

the moment of resistance about O available to resist the applied moment due to F is:

$$n_w \times \frac{t}{2} - \left(\frac{f_k}{\gamma_m} \cdot t_b\right) \frac{t_b}{2}$$

but $n_w = t_b \times \frac{f_k}{\gamma_m}$ or $t_b = n_w \frac{\gamma_m}{f_k}$

$\therefore$ design moment of resistance =

$$\frac{n_w}{2}\left(t - \frac{n_w \gamma_m}{f_k}\right)$$

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job no.: BICC CPD/C 2015		sheet no.: F03	
member / location: 2.5m high wall retaining rock fill			
drg ref.:			
job title: MASONRY RETAINING WALL	made by: DHC	date: 05/15	

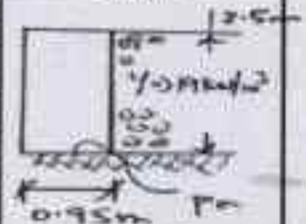
Ref.	Calculations	Outputs

$p_a \Rightarrow k_a \rho h \Rightarrow 0.23 . 19 . 2.5 \Rightarrow 10.925 \text{ kN/m}^2$

$P_a \Rightarrow \frac{1}{2} p_a h \Rightarrow 0.5 . 10.925 . 2.5 \Rightarrow 13.656 \text{ kN/m}$

$BM \Rightarrow P_a . h/3 \Rightarrow 13.653 . 2.5/3 \Rightarrow 11.38 \text{ kN-m/m}$

$N \Rightarrow \rho b h \Rightarrow 19 . 0.95 . 2.5 \Rightarrow 45.125 \text{ kN-m/m}$

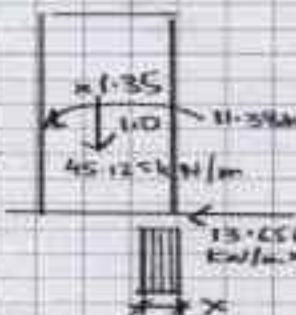


For rock-fill
$\phi \Rightarrow 35^\circ$
$\delta = 20^\circ$
$k_a \Rightarrow 0.23$

Stability Calculation according to EC7
Design Approach 1 (Str/Geo) Combination 1.

$e \Rightarrow BM/N \Rightarrow (11.38 \times 1.35)/45.125 \Rightarrow 0.34\text{m}$
(middle third today outdated)

$x \Rightarrow P_a \Big/ \frac{f_k}{\gamma_m} \Rightarrow 45.125 \Big/ \frac{4.91}{2.2} \Rightarrow 0.2022\text{m}$



$f_k \Rightarrow 4.91 \text{ N/mm}^2$
(module 2 calc sheet)
for 230 masonry in M2 mortar.

$BM_{ult} \Rightarrow 11.38 \times 1.35 \Rightarrow 15.36 \text{ kN-m/m}$

$MR \Rightarrow \frac{N}{2}\left(t - \frac{N.\gamma_m}{f_k}\right) \rightarrow \frac{N}{2}(t - x)$

$\Rightarrow 45.125/2 \times (0.95 - 0.2022) \Rightarrow 16.87 \text{ kN-m/m} > 15.36 \text{ kN-m/m}$ ✓

for $t \Rightarrow 0.9$ $MR \Rightarrow 15.14 < 15.36 \text{ kN-m/m}$

Check Sliding at the base

$V \Rightarrow 13.653 \times 1.35 \Rightarrow 18.44 \text{ kN/m}$

$v \Rightarrow 18.44/0.95 . 1\text{m} \Rightarrow 0.0194 \text{ N/mm}^2$

$f_{vk} \Rightarrow f_{vko} + 0.4 \sigma_d$

$\Rightarrow 0.1 + 0.4 . 45125/0.95$

$\Rightarrow 0.119 \text{ N/mm}^2 > 0.0194 \text{ N/mm}^2$ ✓

Demystifying the masonry Eurocode 6 & 8 (seismic)

‘Course C’ Module 4 - Tensile Stresses in Masonry

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7th /9th July 2015

Characteristic flexural strength of unreinforced masonry

· SYMBOL: f_{xk}

· determined from the results of tests on masonry produces

· two different values:

– f_{xk1}: failure parallel to the bed joints,

– f_{xk2}: failure perpendicular to the bed joints.

Flexural strengths f_{xk1} and f_{xk2}.

· use of f_{xk1}:

– only for transient loads (for example wind)

– $f_{xk1} = 0$, where failure of the wall would lead to a major collapse.

Walls subjected to lateral wind loads – 1 - Support conditions and continuity

(1)P In assessing the lateral resistance of masonry walls subjected to lateral wind loads, the support conditions and continuity over supports shall be taken into account.

(2) The reaction along an edge of a wall due to the design load may normally be assumed to be uniformly distributed when designing the means of support. Restraint at a support may be provided by ties, by bonded masonry returns or by floors or roofs.

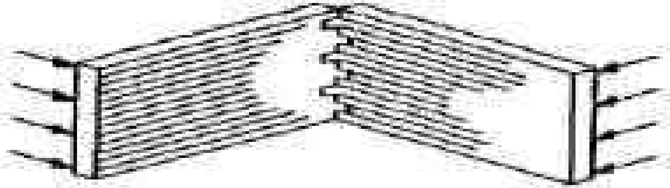
Walls subjected to lateral wind loads - 2 - Method of design for a wall supported along edges

Masonry walls are not isotropic
and there is an orthogonal strength ratio
depending on the unit and the mortar used.

The calculation of the design moment, M_d,
should take this into account
and may be taken as either:

$$M_d = \alpha W_k \gamma_F L^2 \quad \text{per unit height of the wall}$$

when the plane of failure is
perpendicular to the bed joints,
ie. in the f_{xk2} direction,

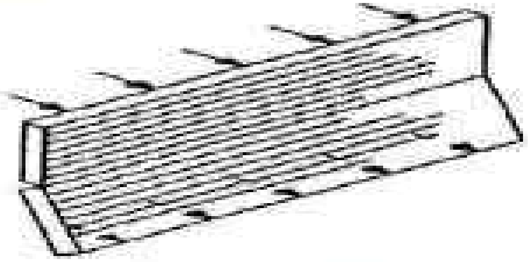


f_{xk2}: Plane of failure perpendicular to bed joints

or:

$$M_d = \mu \alpha W_k \gamma_F L^2 \quad \text{per unit length of the wall}$$

when the plane failure is
parallel to the bed joints,
ie. in the f_{xk1} direction;



f_{xk1}: Plane of failure parallel to bed joints

Walls subjected to lateral wind loads - 2 - Method of design for a wall supported along edges - Continued

where:

α is a bending moment coefficient which depends on:

- the orthogonal ratio, μ,
- the degree of fixity at the edges of the panels
- and the height to length ratio of the panels.
  (it is implicit that a suitable theory is given in the National Application Documents.)

γ_F is the partial safety factor for loads;

μ is the orthogonal ratio of the characteristic flexural strength of the masonry, f_{xk1}/f_{xk2};

L is the length of the panel between supports;

W_k is the characteristic wind load per unit area.

When a vertical load acts so as to increase the flexural strength f_{xk1}, the ortogonal strength ratio may be modified:

$$f_{xk1} + \gamma_M \sigma_{dp}$$

where:

σ_{dp} is the permanent vertical stress of the wall at the level under consideration.

The design moment of lateral resistance of a masonry wall, M_{Rd}, is given by:

$$M_{Rd} = \frac{f_{xk} \cdot Z}{\gamma_M}$$

where:

Z the section modulus of the wall.

Table 1 – BENDING MOMENT COEFFICIENT FOR TWO WAY SPANNING PANELS SUBJECTED TO LATERAL LOADS ($\mu = 0.35$)

Values of α

$\frac{h}{L}$	1	2	3	4	5	6	7	8	9	10	11
0·30	0·045	0·035	0·029	0·022	0·018	0·016	0·014	0·009	0·024	0·021	0·016
0·50	0·064	0·049	0·039	0·032	0·035	0·029	0·025	0·017	0·055	0·045	0·035
0·75	0·080	0·059	0·045	0·040	0·052	0·041	0·033	0·026	0·098	0·075	0·060
1·00	0·089	0·065	0·049	0·044	0·064	0·050	0·039	0·032	0·144	0·104	0·084
1·25	0·095	0·068	0·052	0·048	0·074	0·055	0·043	0·037	0·194	0·129	0·108
1·50	0·100	0·071	0·053	0·050	0·081	0·060	0·046	0·040	0·244	0·152	0·129
1·75	0·103	0·073	0·054	0·051	0·086	0·063	0·048	0·043	0·296	0·173	0·148

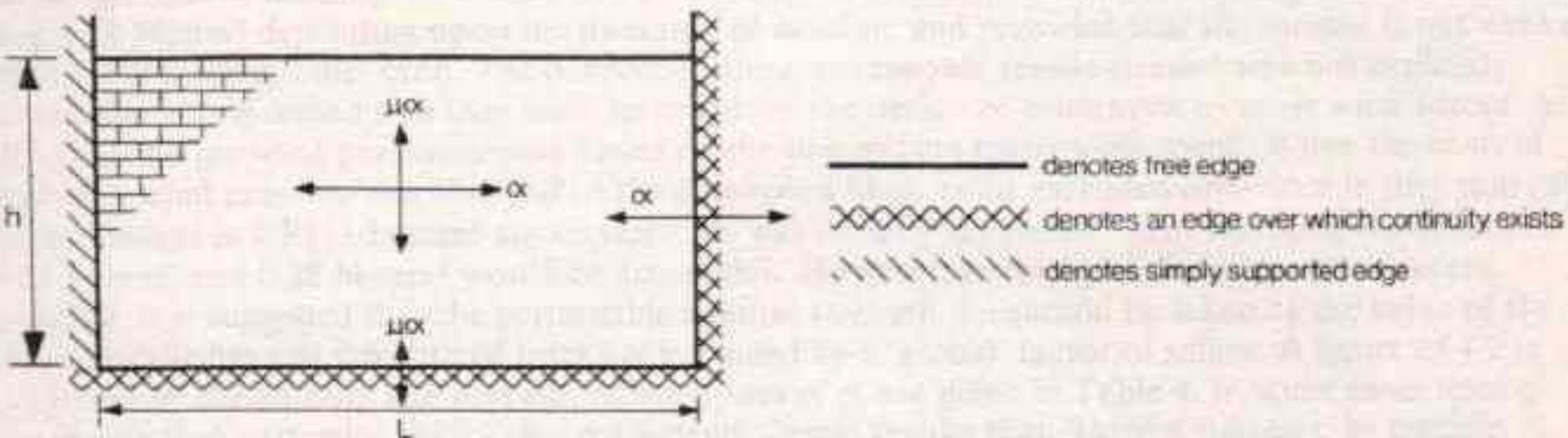
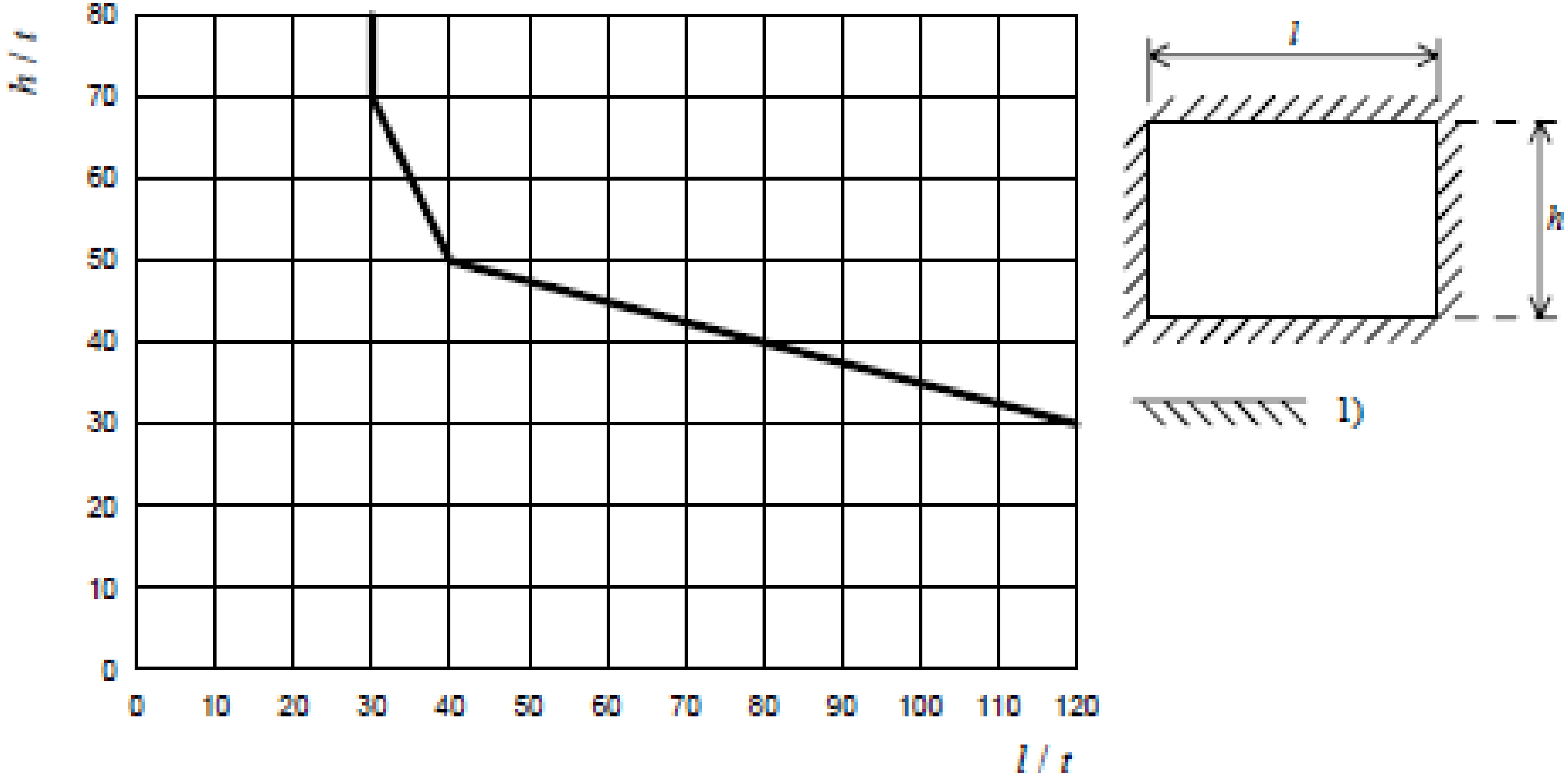


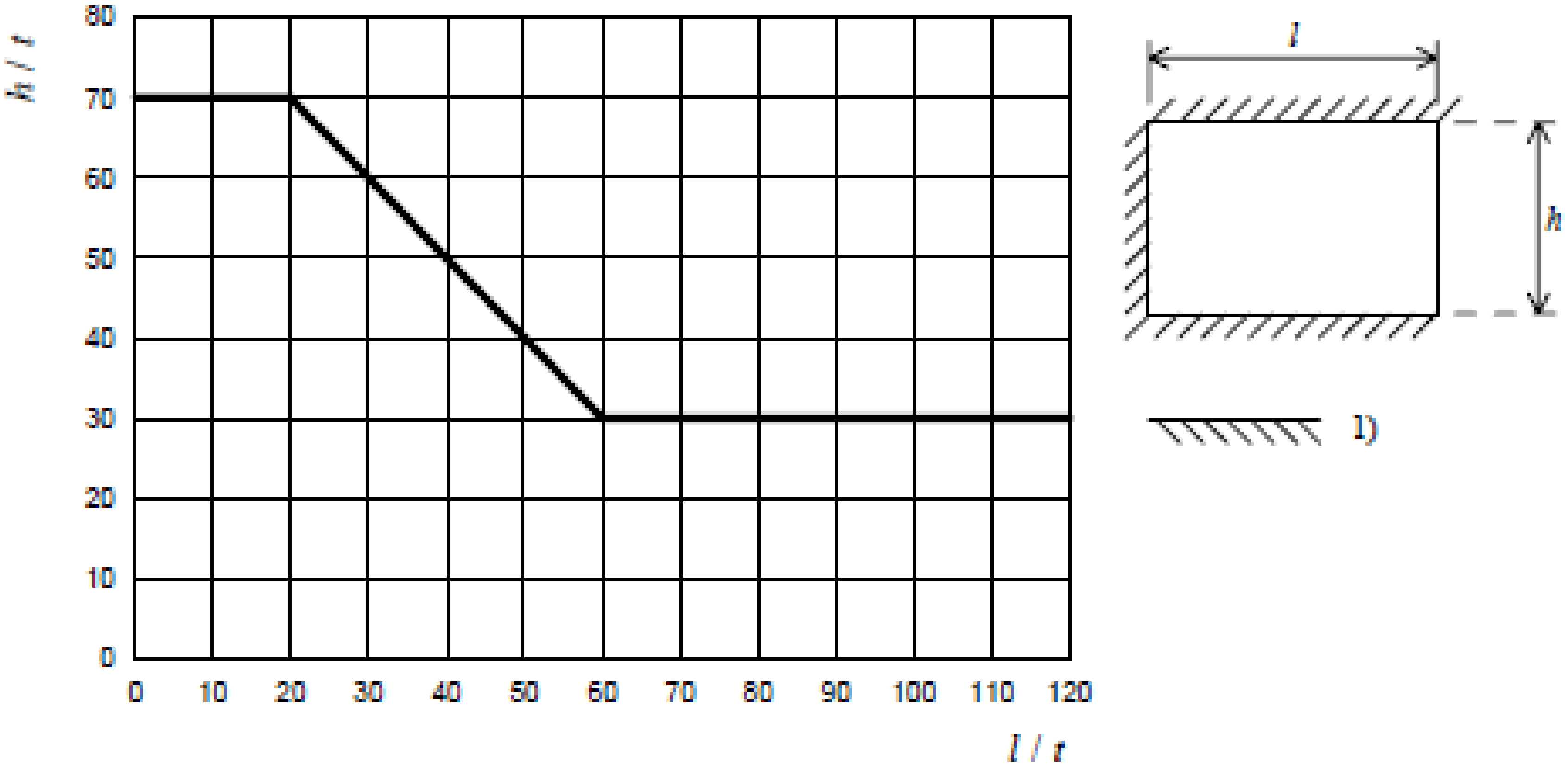
Figure 1 - Limiting height and length to thickness ratios of walls restrained on all four edges



Key

1) simply supported or with full continuity

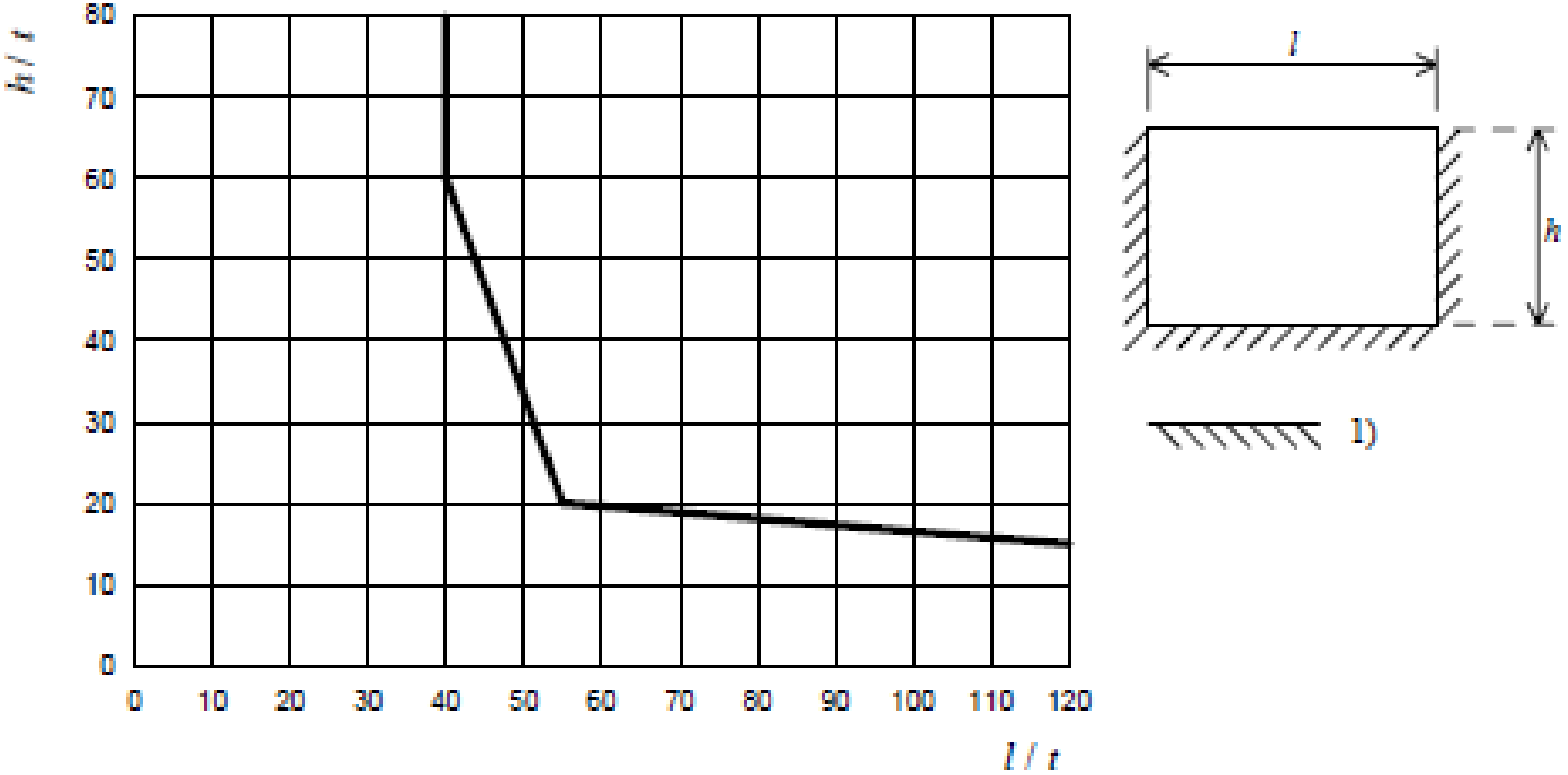
Figure 2 - Limiting height and length to thickness ratios of walls restrained at the bottom, the top and one vertical edge



Key

1) simply supported or with full continuity

Figure 3 - Limiting height and length to thickness ratios of walls restrained at the edges, the bottom, but not the top



Key

1) simply supported or with full continuity

Calc Sheet

Demystifying the masonry Eurocode 6 & 8 (seismic)

‘Course C’ Module 5 – Stability & seismic action in Masonry Structures

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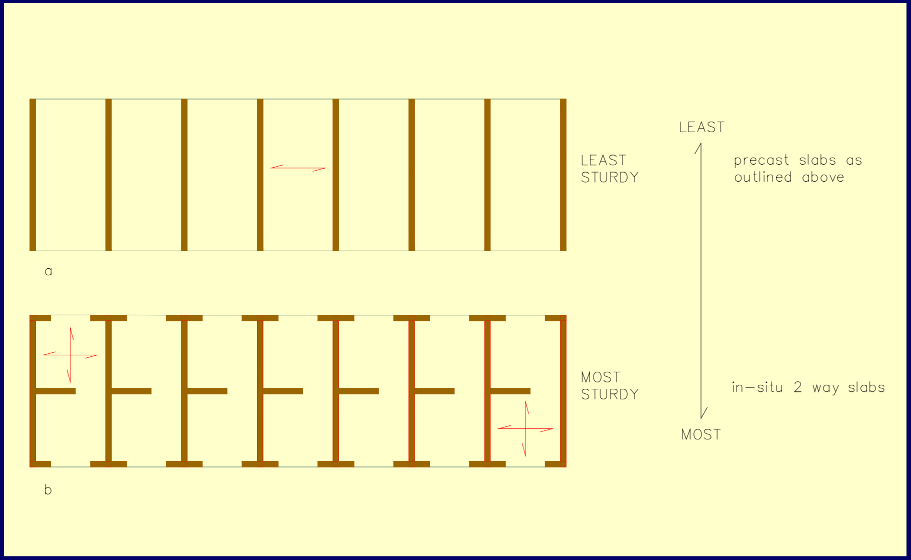
7th /9th July 2015

Stability of Walls & Piers subject to Vertical Loading - Figure 1

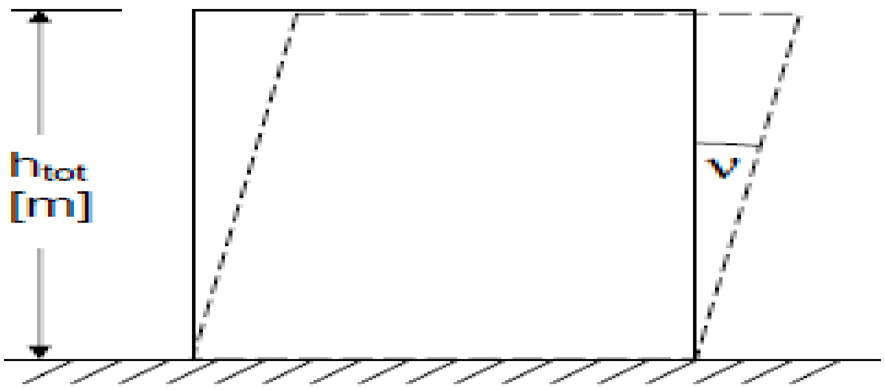


- THE EXTENT OF DAMAGE SHOULD NOT BE DISPROPORTIONATE TO ITS CAUSE
- EC6 gives this at 1% of the combined vertical characteristic dead and imposed load at the particular floor divided by $\sqrt{h_{tot}}$
- Their effect may be ignored, if less onerous than other horizontal actions eg. wind

Structural behaviour and overall stability

Design models for structural behaviour

- To ensure stability and robustness,
  it is necessary for the layout of the structure on plan and section,
  the interaction of the masonry parts,
  and their interaction with other parts of the structure,
  to be such as to produce a properly braced arrangement.
- Structures incorporating masonry walls,
  should have their parts suitably braced together,
  so that sway of the structure will not occur.
- The possible effects of imperfections should be allowed for,
  by assuming that the structure is inclined at an angle ν



$$\nu = \frac{1}{100 \cdot \sqrt{h_{tot}}}$$

Second order effects

(1)P Structures incorporating masonry walls designed according to this EN 1996-1-1 shall have their parts braced together adequately so that sway of the structure is either prevented or allowed for by calculation.

(2) No allowance for sway of the structure is necessary if the vertical stiffening elements satisfy equation (5.1) in the relevant direction of bending at the bottom of the building:

$$h_{tot} \sqrt{\frac{N_{Ed}}{\sum EI}} \quad \begin{array}{ll} \leq 0{,}6 & \text{for } n \geq 4 \\ \leq 0{,}2 + 0{,}1\, n & \text{for } 1 \leq n \leq 4 \end{array} \tag{5.1}$$

where:

h_{tot} is the total height of the structure from the top of the foundation;

N_{Ed} is the design value of the vertical load (at the bottom of the building);

$\sum EI$ is the sum of the bending stiffnesses of all vertical stiffening building elements in the relevant direction;

NOTE Openings in vertical stiffening elements of less than 2 m^2 with heights not exceeding 0,6 h may be neglected.

n is the number of storeys.

(3) When the stiffening elements do not satisfy 5.4(2), calculations should be carried out to check that any sway can be resisted.

Structural Analysis

Imperfections

Structure is supposed to be inclined under an angle $\upsilon = \dfrac{1}{\left(100\sqrt{h_{tot}}\right)}$

The resulting horizontal action should be added to the other actions

Second order effects

No influence when:

$$h_{tot}\sqrt{\frac{N_{Ed}}{\sum EI}} \quad \begin{array}{ll} \leq 0{,}6 & \text{for } n \geq 4 \\ \leq 0{,}2 + 0{,}1\,n & \text{for } 1 \leq n \leq 4 \end{array}$$

When the stiffness is not large enough: calculation (e.g. by Annex B)

DEFLECTION & ROTATION COEFFICIENTS FOR A CANTILEVER

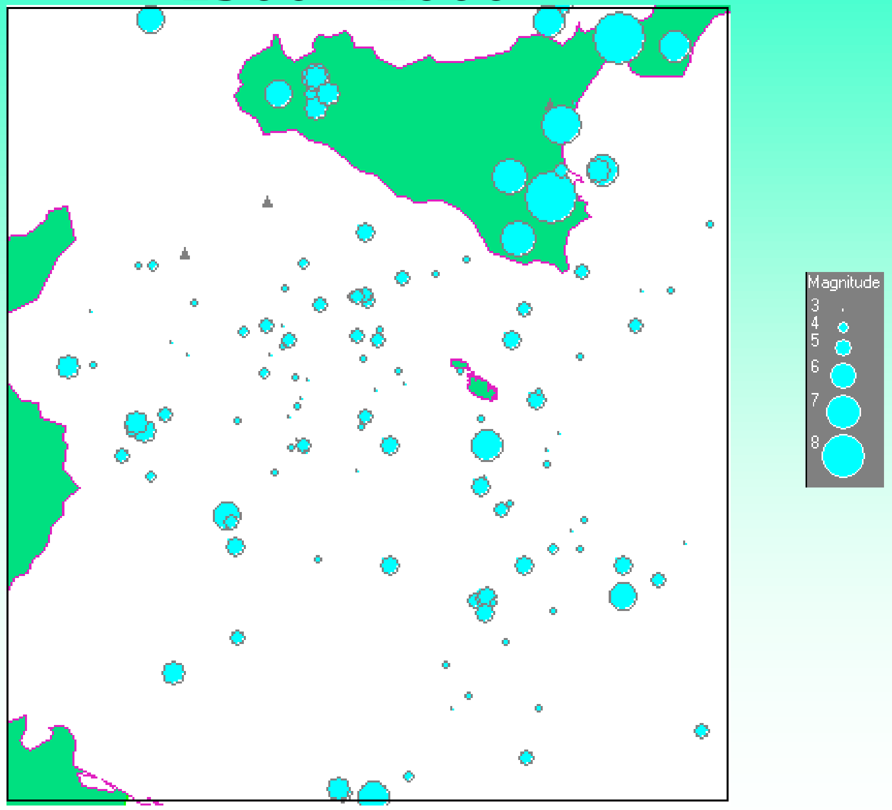
Table 1: Updated 'C' deflection coefficient for moment of inertia calculation for a cantilever span condition

Span to deflection ratio	Steel $E = 205kN/mm^2$	Concrete $E = 30kN/mm^2$	Timber $E = kN/mm^2$
1/300 - udl	18.3	125	469
1/300 – pt load	48.3	330	1,238

Rotation in rad (udl) = 1.33 X span : deflection ratio
Rotation in rad (point load) = 1.50 X span : deflection ratio

Instrumental seismicity sicily channel 1900-2000 – figure 2

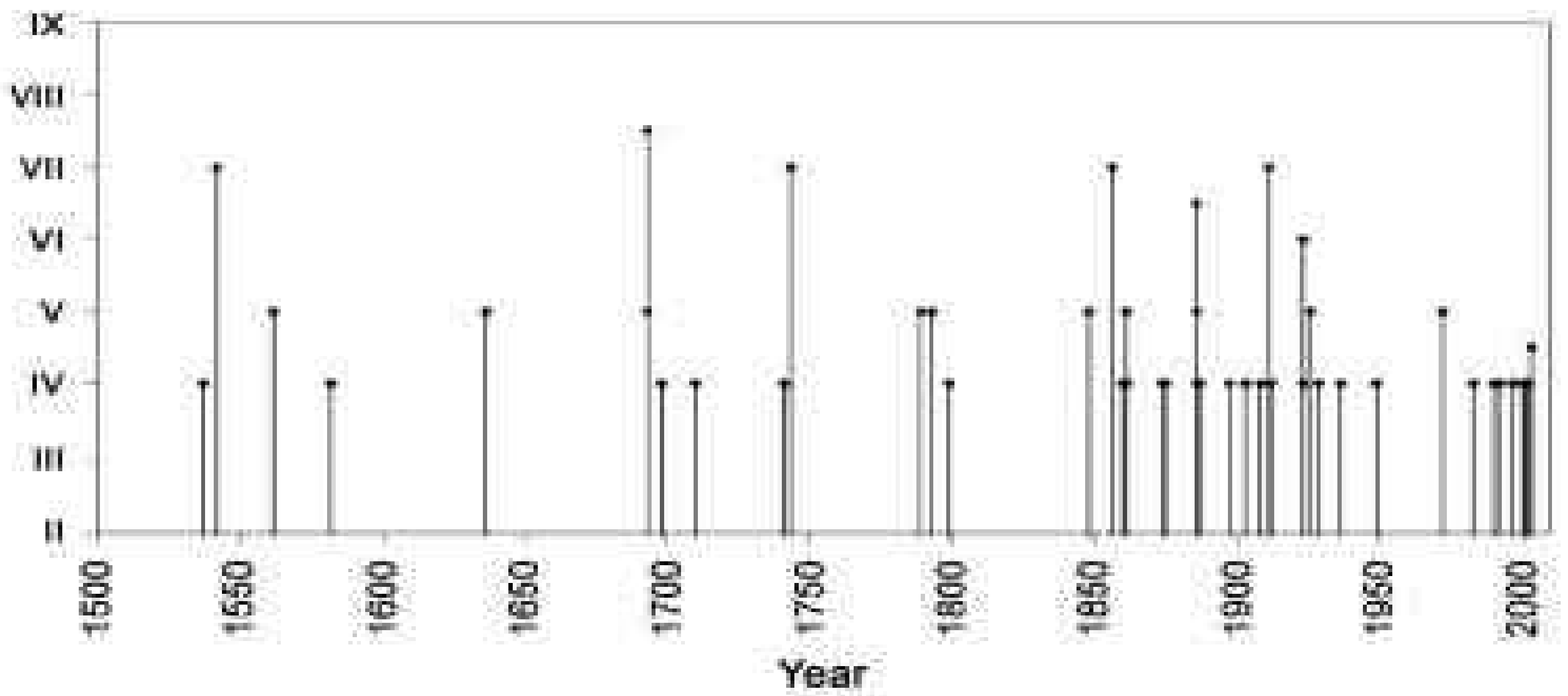
Instrumental Seismicity Sicily Channel
1900 - 2000



Source: ISC Bulletin, INGV, EMCS

Malta is situated on the stable plateau at the edge of the African plate, with seismic effects arriving from Sicily or Greece.

Figure 3: Site seismic history for the Maltese islands since 1500, showing EMS-98 I ≥IV.



Source:- Dr. Pauline Galea – Annals of Geophysics, Vol. 50, N. 6, December 2007

Table 2 Subset of felt earthquake catalogue, showing only events that produced EMS-98 I=V and over on the Maltese islands.

Year	Month	Day	Hour	Lat	Long	Region	I_{max} on Maltese islands	I_o	M	Parameter reference
1542	12	10	15:15	37.20	14.90	E. Sicily	VII	XI	M_w 6.6	Gruppo di Lavoro CPTI (2004)
1562	3	8	Morning			Sicily Channel?	V?			
1636	9	1				Sicily Channel(?)	V?			
1693	1	11	13:30	37.18	15.02	E. Sicily	VII-VIII	XI	M_w 7.4	Boschi *et al.* (2000)
1743	2	20	16:30	39.87	18.78	Ionian Sea	VII	IX	M_w 6.9	Gruppo di Lavoro CPTI (2004)
1789	1	19	Morning			Sicily Channel(?)	V?			
1793	2	26	Morning			Sicily Channel?	V?			
1848	1	11	12:00	37.20	15.20	E. Sicily	V	VIII-IX	M_w 5.5	Gruppo di Lavoro CPTI (2004)
1856	10	12	00:45	35.60	26.00	Crete	VII		M_w 7.7	Papazachos *et al.* (2000)
1861	2	8	23:45			Sicily Channel(?)	V?			
1886	8	15	02:45			Sicily Channel(?)	V			
1886	8	27	22:00	37.00	27.20	Aegean Sea	VI-VII	XI	M_w 7.3	Papazachos *et al.* (2000)
1911	9	30	09:25	36.47	13.57	Sicily Channel	VII			
1923	9	18	07:30	35.57	14.57	Sicily Channel	VI			ISC (2001)
1926	6	26	19:46	36.50	27.50	Aegean Sea	V		M_w 7.6	Papazachos *et al.* (2000)
1972	3	21	23:06	35.80	15.00	Sicily Channel	V		M_b 4.5	ISC (2001)

Source:- Dr. Pauline Galea – Annals of Geophysics, Vol. 50, N. 6, December 2007

Figure 4 - Estimated return periods, following the methodology of Magri *et al. (1994).*

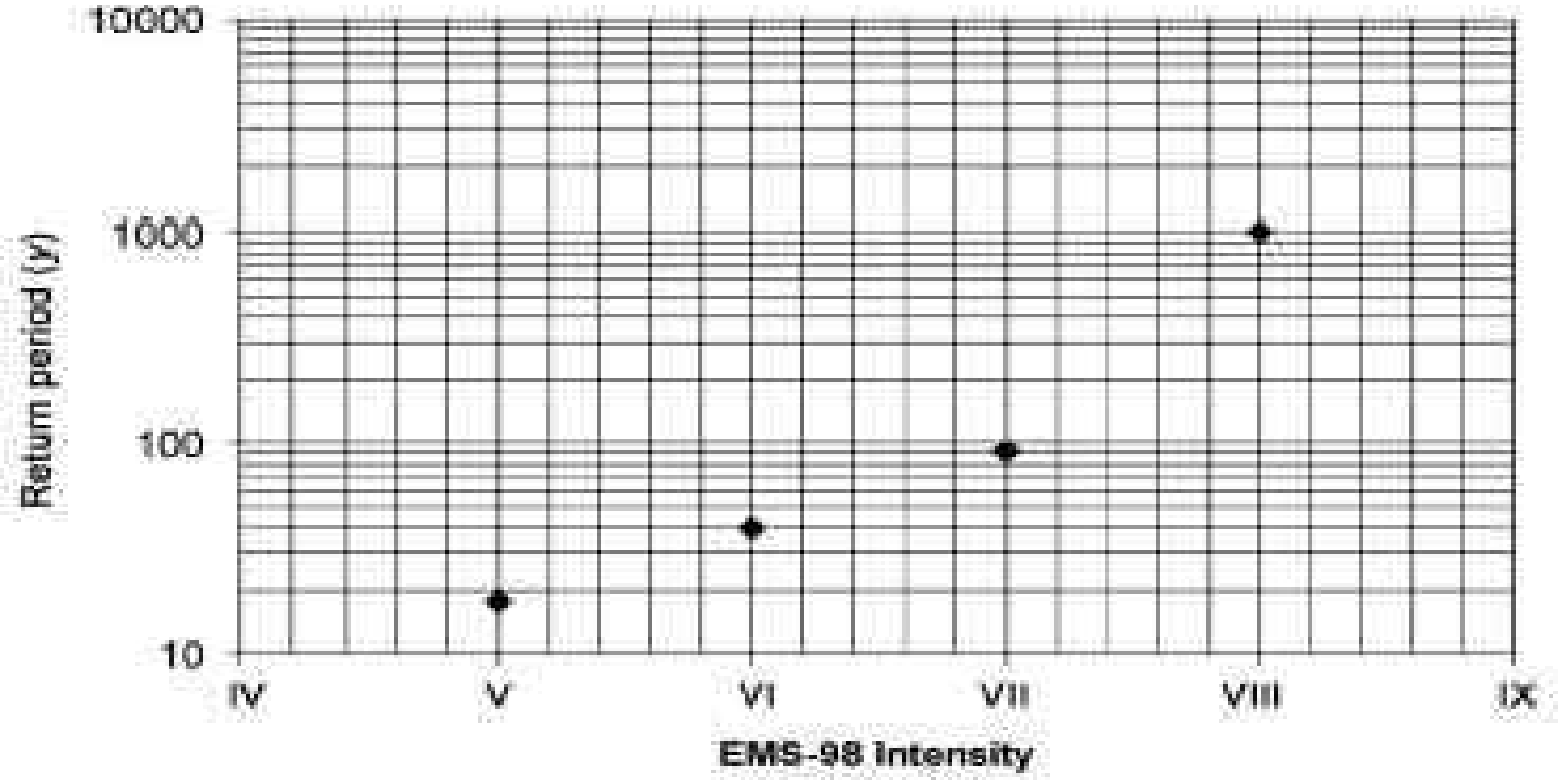
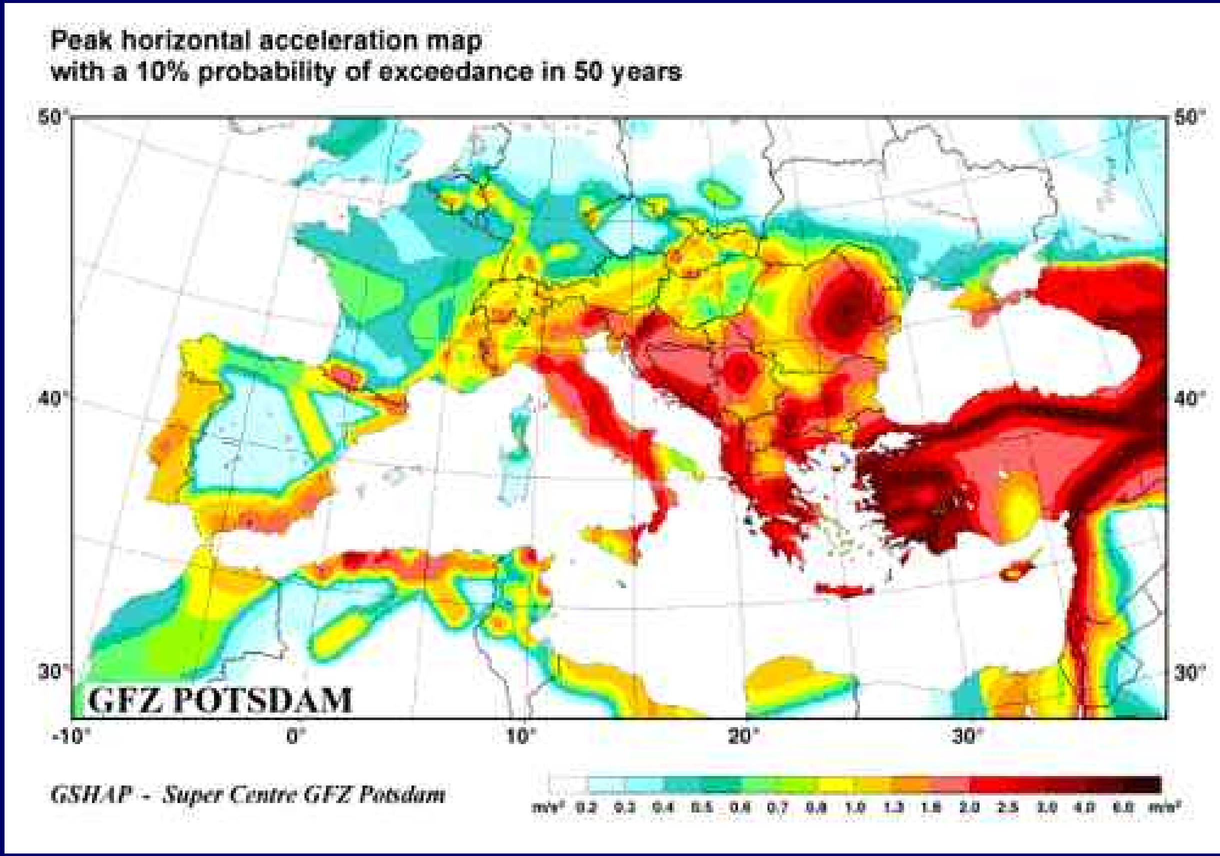


Figure 5 – GSHAP – (Global Seismic Hazard Assessment project) map for Europe



Malta is a green colour corresponding to 0.05g – 0.06g. But the data on which this was complied was probably very sparse for Malta

Table 3 - 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 for no collapse requirement. For damage limitation exceedence this is to be based on a 95 yr return period, which signifies a 10% chance of exceedence.

MM – Earthquake Intensity	Return Period (years)	Base Shear Design % of g
V	8	1-2
VI	40	2-5
VII	90	5-10
VIII	1000	10-20

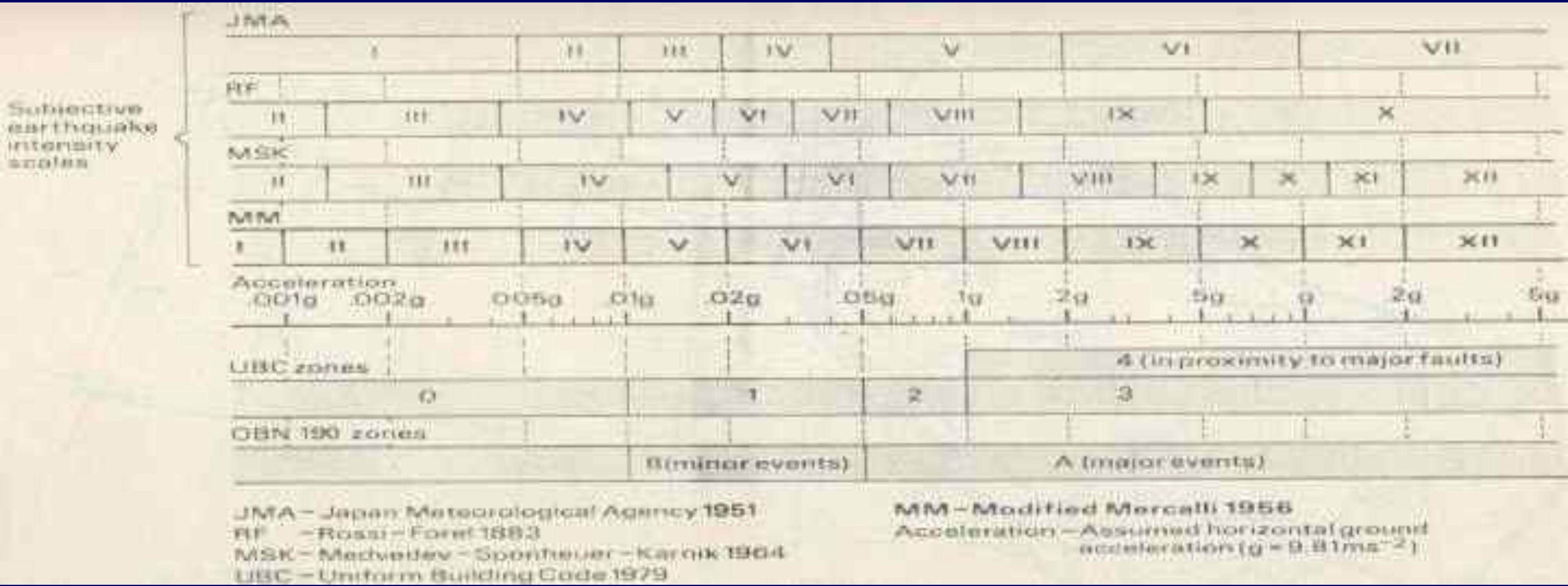
Defined as a low seismicity zone as <0.08g but > 0.04g EC2 concrete provisions to be catered for not EC8

Table 4 – Classification of Building according to anticipated Earthquake Intensity Damage

Type	Description	Base shear design % of gravity
A	**Building of fieldstones, rubble masonry, adobe and clay**	**0.5%**
B	***Ordinary unreinforced brick buildings, buildings of concrete blocks, simple stone masonry and such buildings incorporating structural members of wood;***	***0.7%***
C	***Buildings with structural members of low-quality concrete and simple reinforcements with no allowance for earthquake forces, and wooden buildings, the strength of which has been noticeable affected by deterioration;***	***0.9%***
D_1	***Buildings with a frame (structural members) of reinforced concrete***	***2-3***

Buildings found in Malta are mostly found in types C & D, buildings deteriorated at B. Further buildings classified as D_2 up to D_5 with a D_5 building frame able to withstand a 20% gravity base shear.

Figure 6 – Comparative chart of earthquake intensity scales, ground acceleration levels and design requirements



Several empirical formulae have been proposed linking intensity with magnitude. Often these refer specifically to studies in limited areas, such as the work done by Richter in California. In the assessment of intensity used in this Note, the formula used is:

$$I = 8.0 + 1.5\,M - 2.5\,\log_e (h^2 + d^2 + 400)^{0.5}$$

where I = *Intensity (MM)*

M = *Magnitude (Richter)*

h = *Focal depth (km)*

d = *Distance from the epicentre (km)*

Table 5 – Mean Damage Ratio (MDR) & Death Rates for building types B & C founded on rock

Building Type	**B**			**C**		
Earthquake Intensity MM	MDR	Death Rate	Mean damage costs as % of re-building costs	**MDR**	Death Rate	Mean damage costs as % of re-building costs
5	2%	-	2.5%	-	-	-
6	4%	-	6%	**1%**	-	1.25%
7	20%	0.03%	40%	**10%**	-	15%
8	45%	1%	135%	**25%**	0.4%	62.5%

Source: Swiss Re (1992)

For a type ‘B’ building non structural damage would amount to 50% of MDR, increasing to 70% for a type ‘C’ building.

As the quality of a building goes up, the contribution of non-structural damage increasing, the death rate reduces, but a higher number of injuries occur.

Table 6 – Amended damage Ratio Matrix for Higher Irregularity & Asymmetry founded on rock

Building Type	C	D_1
EARTHQUAKE INTENSITY		
V	10%	5%
VI	30%	18%
VII	60%	40%
VIII	100%	72%
IX	100%	95%

If founded on clay move up to higher intensity if on fill material to a further higher intensity

TABLE 7 - DAMAGE PROBABILITY MATRIX FOR BUILDING (DPM)

Damage class % of value			Mean Damage Ratio (%)									
			1.5	3	5	10	25	37.5	50	60	70	85
0	- 1.5	(A)	83	73	60	36	9	2				
1.5	- 3	(B)	17	25	26	23	9	3				
3	- 6	(C)		2	10	18	11	5	2			
6	- 12.5	(D)			3	12	18	12	6	2	1	
12.5	- 25	(E)			1	8	24	24	15	7	3	
25	- 50	(F)				3	19	28	29	23	18	10
50	- 100	(G)				1	10	29	48	68	78	90

Source : Swiss Re (1992)

TABLE 8 - PERCENTAGE OF BUILDINGS WIH 80-100% DAMAGE DEPENEDING ON MDR

MDR	10	20	30	40	50	60	70	80	90
Percentage	0.25	3.5	10	20	30	45	56	70	85

As a rule of thumb about 1/4 - 1/8 of the population in the 80% - 100% damage class will be killed

SEISMIC DESIGN TO EC8 – PART 1

EC8 relates to 6 parts with -
Part 2 relating to bridges
Part 3 relating to seismic assessment & retrofitting of existing buildings.
Part 4 relating to silos, tanks & pipelines.
Part 5 relating to foundation design.
Part 6 relating to towers, masts & chimneys.

Part 1 relates to specific structural materials - concrete, steel, timber & masonry.

The FS to masonry for seismic design is to be taken at 2/3's of the FS for the permanent load design, but not less than 1.5.

MASONRY DESIGN CRITERIA FOR ZONES OF LOW SEISMICITY (EC8)

1. Shear walls in unreinforced manufactured stones units

   $t \geq [175]$mm

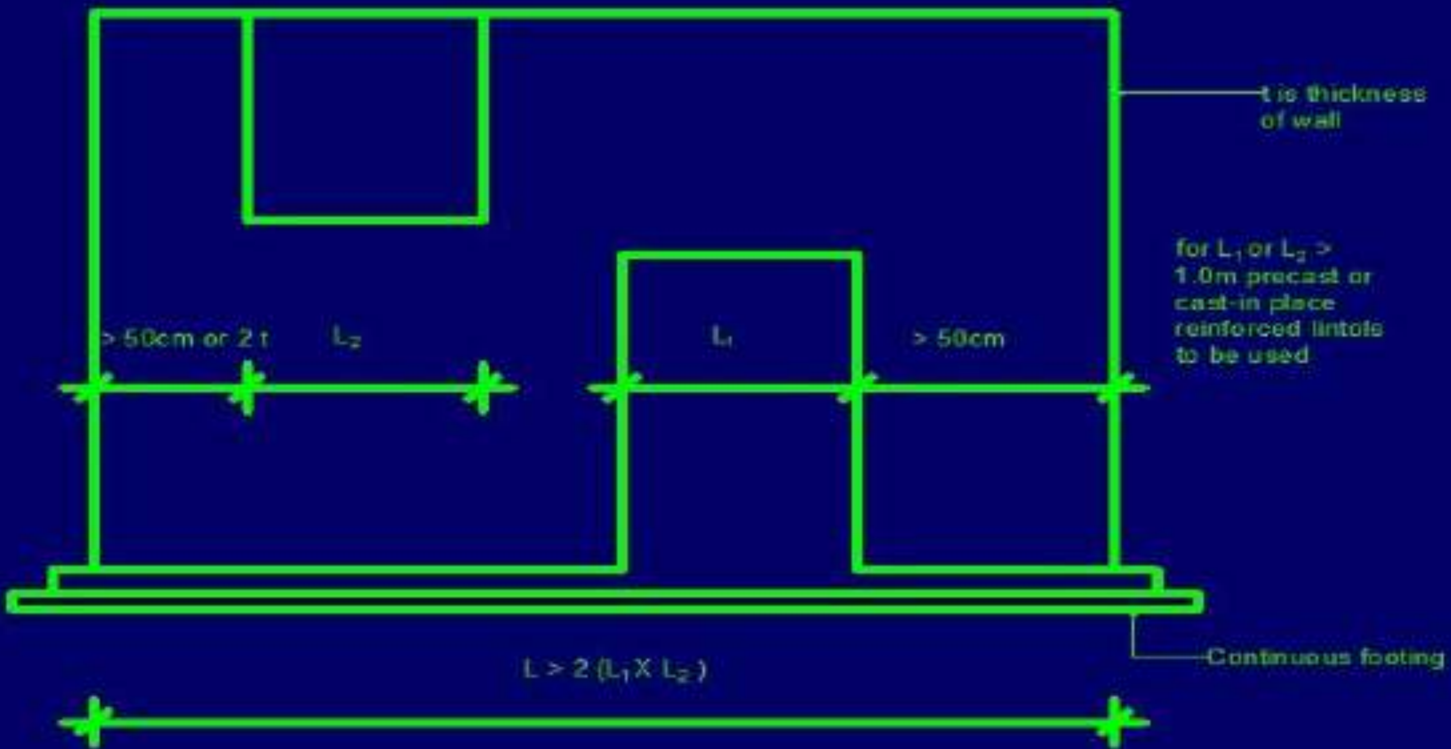
   $h_{ef}/t \leq [15]$

2. A min of 2 parallel walls is placed in 2 orthogonal directions. The cumulative length of each shear wall > 30% of the length of the building. The length of wall resisting shear is taken for the part that is in compression.

3. For a design ground acceleration < 0.2g the allowed nº of storeys above ground allowed is [3] for unreinforced masonry and [5] for reinforced masonry, however for low seismicity a greater nº allowed.

4. Mortar Grade (M5) although lower resistance may be allowed. Reinforced masonry type (M10). No need to fill perp. Joints.

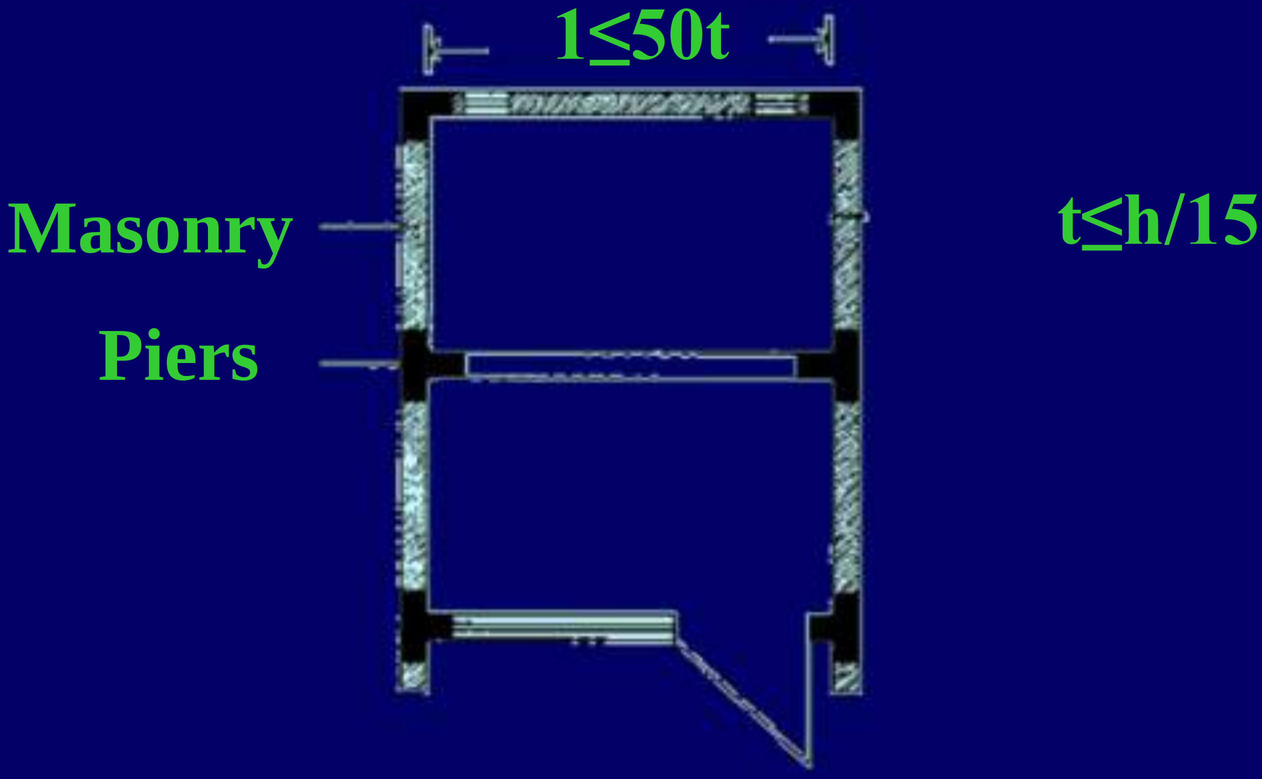
MASONRY IMPROVED STURDINESS FOR ASEISMIC DESIGN - FIG. 7



Unreinforced masonry shear walls t_{ef} = 350mm h_{ef}/t_{ef} = 9 L/h = 0.5
Reinforced masonry shear walls t_{ef} = 240mm h_{ef}/t_{ef} = 15

Example of overcoming unsymmetrical requirements when large opening required on one side - FIG. 8

Forming stiffening piers at 7m centres, with min outstand of h/5



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made by: DHC	date: 06/15

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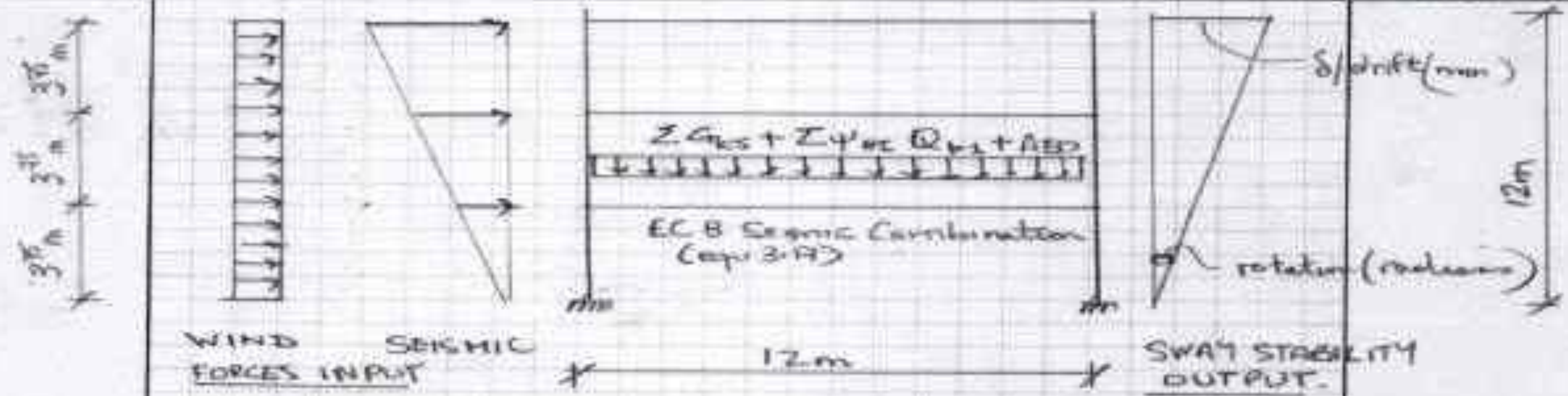

SEISMIC CHARACTERISTICS - EC8

Fundamental Period of vibration period T_1

$T_1 \Rightarrow C.H^{3/4}$ (equ. 4.6 EC8)

where $C \Rightarrow$ 0.085 for moment resistant steel frames
0.075 for moment resistant concrete frames
0.050 for all other structures

$T_1 \Rightarrow 0.05 . 12^{3/4} \Rightarrow 0.32s$

or $T_1 \Rightarrow 2.\sqrt{\delta}$ (equ. 4.9/EC8 - to be checked later)

Ground Conditions - Table 3.2 / EC8 - type 1 response

Gnd type	S	$T_{B(s)}$	$T_{C(s)}$	$T_{D(s)}$
A - rock	1.0	0.15	0.4	2.0
B - clay*	1.2	0.15	0.5	2.0

* c_u > 250 kN/m² N_{SPT} > 50 (blows/30cm)

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Ref.	Calculations	Outputs

For $T_B < T \leq T_C$ $0.15 \leq 0.32 \leq 0.4$ (Grd types A & B)

$S_{e(T)} = a_g . S . \eta . 2.5$ (eqn 3.3/EC8)

$a_g = 0.08g$ (Malta - NA)

$S = 1$ (rock foundation)

$\eta = \sqrt{10/(5+\xi)}$

η is the damping correction factor, with $\eta = 1$ for 5% viscous damping ratio

for masonry to be taken at 13%

$\eta = \sqrt{10/(5+13)} = 0.745$

$S_{eT} = 0.08 . 1 . 0.745 . 2.5 = \underline{0.149}$

SEISMIC LOAD COMBINATION FACTOR

$\Sigma G_{kj} + \Sigma \psi_{Ei} . Q_{ki}$

$\psi_{Ei} = \phi . \psi_{2i}$

$\phi = 1$ (table 4.2/EC8 retail units)

$\psi_{2i} = 0.6$ (table A.1.1. CPD A/module 2)

$DL = 0.1.25 + 0.1.18 kN/m^2$

$LL = 1.0.6 . 5 kN/m^2$

$7.3 kN/m^2$

$\Sigma N / fl = 7.3 kN/m^2 . 2.5m . 12m/2$

b/w $15cm . 1.35 kN/fl/m^2 . 2.5m$

$160 kN/fl$

ΣN $160 kN/fl . 3 fl$ $\underline{480 kN}$

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	made by: DHC	date: 06/15

Ref	Calculations	Outputs

SEISMIC FORCE OUTPUT

$F_b \Rightarrow S_d(T_1)\, m\, \lambda$ (equ. 4.5/EC8)

$\lambda \Rightarrow 0.85$ as $T_1 \Rightarrow 0.32s < 2T_C$
where $2T_C \Rightarrow 2 \cdot 0.4 \Rightarrow 0.8s > 0.32s$
& has more than 2 stories (EC8/para 4.3.3.2.2)
An importance factor is also to be considered (table 4.3/EC8)
which for this ordinary building may be taken at 1, which could increase to 1.2 for schools, 1.4 for hospitals and 0.8 for agricultural buildings

$F_b \Rightarrow 0.149 \cdot 480kN \cdot 0.85 \Rightarrow 60kN$ at 2.5m ¢

Distribution of F_b over ht of bldg at 2.5m ¢'s

$F_i \Rightarrow F_b \cdot z_i m_i / \Sigma z_j m_j$ (equ 4.11 EC8)

$z_1 m_1 \Rightarrow 12m$ $\quad F_1 \Rightarrow \frac{12}{23.25} \cdot 60$

$z_2 m_2 \Rightarrow 7.5m$ $\quad F_2 \Rightarrow \frac{7.5}{23.25} \cdot 60$

$z_3 m_3 \Rightarrow 3.75m$ $\quad F_3 \Rightarrow \frac{3.75}{23.25} \cdot 60$

$\Sigma z_j m_j \Rightarrow 23.25$

$BM \Rightarrow 31 \cdot 12 + 19 \cdot 7.5 + 9.5 \cdot 3.75 \Rightarrow 550\ kN\text{-}m$

$N \Rightarrow 160\ kN/flr \times 3 fl \Rightarrow 480\ kN$

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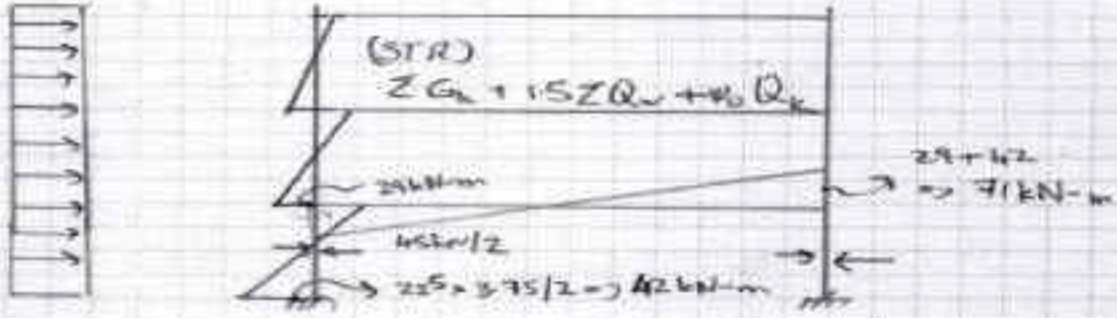
job title: WIND/SEISMIC STABILITY	made by: DHC	date: 06/15	

Ref	Calculations	Outputs

The basic wind pressure is taken at 1kN/m².
(refer to CPD/A - module 3 for calculations of wind pressures according to EC1/part4 + NA)

1kN/m² x 2.5m x 12m
⇒ 45kN



PORTAL FRAME METHOD OF DESIGN
(dependent on fixity moment of 71kN-m developing?)

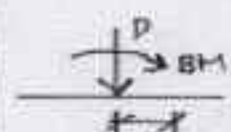
otherwise go for free cantilever design

$BM \Rightarrow \frac{1kN/m^2 . 2.5m . 12^2/2}{2}$

$\Rightarrow$ 135kN-m

3 load cases to be catered for:

	BM kN-m	Load kN	e-m ⇒ BM/P
seismic	550	480	1.15
wind (portal)	42	480	0.09
wind (cantilever)	135	480	0.28



$e \Rightarrow \frac{BM}{P}$

The high seismic e obtained, tends to overrule the construction

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	made by: DHC	date: 06/15	

Ref	Calculations	Outputs

DESIGN CHECK to be undertaken for the Cantilever (Wind) by the Stability Moment Calculation.

f_k (for 230 masonry) → 4.91 N/mm² (Module 2 Calc. Sheet F01)

f_d → f_k / γ_m where γ_m → 0.67 × 2.2 → 1.47 ≈ 1.5

→ 4.91/1.5 → 3.27 N/mm².

x → 480kN / 3,270 kN/mm² / 0.6m = 0.245m

MR → P L_a

l_a → H/2 − 0.245/2 → (H − 0.245)/2

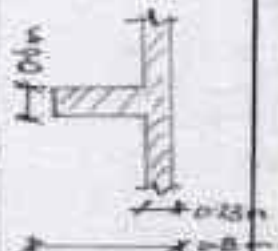
135 → 480 (H − 0.245)/2

H → 0.8075m (say 0.8m)

MASONRY PIER at 2.5m c/s

width of 0.6m

DEFLECTION CHECK



$I = CWL^3$

$E_{masonry}$ → 1,000 × 20 N/mm² → 20 kN/mm²

C → 125 × 30/20 → 187.50 (span/δ = 300)

I → 187.50 (1kN/m² × 2.5m). 12³ → 810,000 cm⁴

I → $bd^3/12$

d → (12 . 810,000/0.6)$^{1/3}$ → 54.3cm < 80cm ✓

Ref | Calculations | Outputs

$I_{row} \Rightarrow bd^3/12$

$\Rightarrow 60.80^3/12 \Rightarrow 2,560,000\ cm^4$

actual span/δ ⇒ $300 \times 2,560,000/840,000 \Rightarrow 980$

$\delta \Rightarrow 12m/980 \Rightarrow 12.25\ mm$

rechecking from page F01

$T_1 \Rightarrow 2\sqrt{\delta} \Rightarrow 2/(0.01225)^{\frac{1}{2}}$

$\Rightarrow 0.16s > 0.15s$ ✓

ROTATION CALCULATION

$\theta \Rightarrow 1.333/(\text{span}/\delta)$

$\Rightarrow 1.333/980 \Rightarrow 0.00136\ rad$

$\theta_{all} \Rightarrow 1/100\sqrt{h_{1st}}$

$\Rightarrow 1/(100.\sqrt{12}) \Rightarrow 0.00289\ rad$

$> 0.00136\ rad$ ✓