Technical Guidance Handbook

Building Regulations 2000

A 114

STRUCTURAL INTEGRITY HANDBOOK

BICC

BUILDING INDUSTRY

CONSULTATIVE COUNCIL

Prepared by

PERIT DENIS H CAMILLERI

1.00 BACKGROUND AND EXPLANATION

1.01 References to the Building Regulation in the text are prefixed with the word 'regulation', whilst prefix para. refers to this handbook.

1.02 The manual is intended to be used by the perit in the preparation of structural design calculations. The first decision to be made is whether to adopt a loadbearing masonry design or to provide a structural frame.

1.03 The range of structures covered by this manual include a simple design building with suggestions given how this type may withstand better the effects of earthquakes by opting for a more robust layout and a tied structure with the vertical elements linked to the rigid horizontal plain. A medium rise structure is also included subjected to the wind and earthquake actions.

1.04 The individual elements are not designed as particular emphasis is based on data particular to the Maltese Islands. For structural components of an international nature reference should be made to the relative Codes of Practice (Regulation 3.00-5.00).

2.00 GENERAL

2.01 Basic guidance for the application of the Building Regulation on Structural Integrity is given indicating the context in which the Regulation should be used and set down certain criteria relating to its objectives.

2.02 The scope of this handbook is to give guidance in some places, but in others only draws attention to factors to which the designer should attend when devising a structural scheme for a specific building. In doing so the designer will have of necessity to make assumptions appropriate to the circumstances in addition to those inherent in the recommendations of the Regulations. In order to ensure the satisfactory realization of a design it is essential that these assumptions are justified in practice by the provision of the necessary supervision.

2.03 The concept of 'good practice' embodied in the Regulations does not necessarily represent an exclusive approach to the design of structures and to the use of appropriate materials limiting the use of alternative materials and methods of design and construction. Such a rigid view would prejudice and inhibit development and innovation preference. However, the Regulations do set or indicate required standards and guiding principles, which may be used as basis of comparison against which to judge the use of alternative procedures and materials.

The prime constraint on the use of alternative methods or materials is that their suitability should be judged on the basis of tests which are designed to represent as far as possible the significant factors which would influence their performance in a real building.

CHAPTER 1 – STABILITY, MOVEMENT & COMPONENTS

3.00 STABILITY (Regulation 1.04.1)

3.01 Masonry (Regulation 1:02.5) is a traditional material which lends itself to layouts on plan which may have irregular outlines and a variety of internal walls. The traditional layout has become known as cellular planform and, due to the high degree of buttressing afforded by intersecting walls, seems a desirable form of construction.

3.02 The sturdiest form of construction being masonry with reinforced concrete slabs. Precast slabs with no lateral ties, continuity and tie bars at supports are the least sturdy. Changes in practice due to economic pressures, shortages of craftsmen and materials, changing standards for lighting, heating and appearance have led to simpler planforms with fewer and lighter weight intersecting walls, and larger openings. The degree of redundancy afforded by cellular planforms has been eroded considerably.

3.03 *'The designer responsible for the overall stability of the structure should ensure the compatibility of the design and details of parts and components. There should be no doubt of this responsibility for overall stability when some or all of the design and details are not made by the same designer.*

To ensure a robust and stable design it will be necessary to consider the layout of structure on plan, returns at the ends of walls, interaction between intersecting walls and the interaction between masonry walls and the other parts of the structure.'

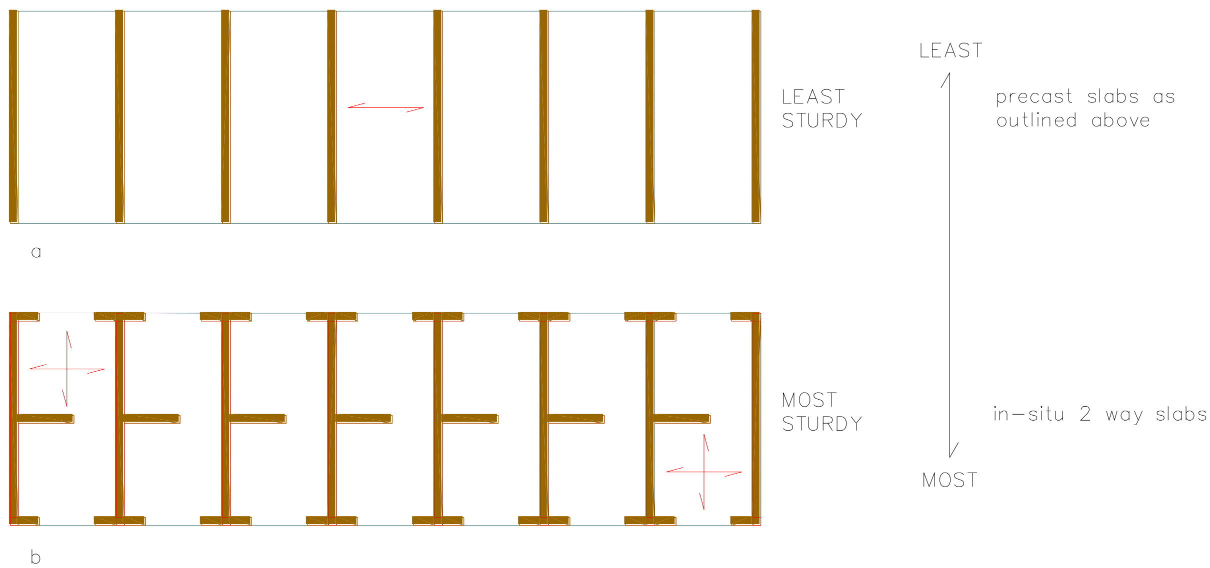


FIG 1 – STURDINESS OF FLOOR PLAN

3.04 A philosophy has developed (Regulation 2.03.1) that while it is not generally economic or even possible to design structures to withstand totally the effects of likely or foreseeable extreme loads, it is possible to design structures to accommodate the effects of such loads and so limit the spread of damage. So has arisen the expression *'the extent of damage should not be disproportionate to its cause.'* Here again it is difficult to set objective requirements. A given explosion which in a reinforced concrete framed structure with infill walls of varying strength might blow out only the lightest weight or weakest panel, might almost demolish a detached house. Yet the same masonry walls at the base of a 4 storey building could well withstand the explosion.

3.05 Regulations1.03.4 & 1.02.6 relate to site or demolition works as affecting adjacent structures. Guidance may be sought from Building Regulation 2000 – Technical Guidance, Approved Document A [(1)] & BICC Guidelines 3 [(2)].

4.00 DESIGN: ACCIDENTAL DAMAGE (Regulation 1.04)

4.01 The general precepts for design of accidental damage are given in Cl. 37 of BS5268 pt 1[3], which should be considered in conjunction with Cl.20 of BS5268 pt 1[3]. The first question for all buildings is to establish that their layout and method of construction have been arranged to provide the best resistance to spread of damage. Although the Regulation 6.02.4e and para. 34.03 do give guidance, the following features which specifically contribute to robustness, and are found in cellular constructions, may be considered advantageous: avoidance of relatively thin or light-weight walls; limitation of floor spans; walls buttressed at both ends except for occasional free ends to minor internal walls; limitation of length of unbuttressed wall; and limitation of size of openings. There may be, of course, other functional or architectural requirements which conflict with these features and the designer must establish a desirable balance. The application of this philosophy, is common to all buildings, is emphasised by the format of Table 12 in BS5268 pt. 1[3]. The minimum lateral load is also specified at 1.5% of the total characteristic dead load above any level. Over and above these somewhat general exhortations for robustness, specific recommendations are made for buildings of five storeys and above, as well as clear spans exceeding 9.00m, in line with the Regulation 2.03.1.

4.02 A number of points are clear on both sides of the argument. The taller the building the more significant the structural aspects become as part of the total cost, and introducing the additional measures generally becomes relatively easier and cheaper. The possibility of extensive vertical progressive collapse is much greater in a taller building. In most cases, it is possible to design low-rise masonry buildings for normal loads in a manner which will provide adequate robustness. However, single-storey long span buildings appear to form a class of buildings which may be particularly sensitive to abnormal events unless particular care is given by the designer to their structural behaviour. The possibility of extensive horizontal progressive collapse, eg. in a crosswall building, should not be ignored (see Fig. 1a).

4.03 Table 12 of BS5628 pt1[3] lists 3 options for buildings of 5 stories or more. In many circumstances Option 3 will be selected because it prescribes horizontal and vertical tying similar to BS 8110[4] without the need for any further consideration of structural behaviour. In other words, it is assumed that improved ability to accommodate local damage of any kind will result. Minimum mortar designation for this option to be III.

4.04 Option 1 presents a more objective approach, sometimes known as the alternative path method, in which each loadbearing element is considered to be removed in turn, and the structure then checked for its ability to accommodate the loss. This more fundamental approach relies to a substantial extent on engineering judgement, as members capable of withstanding a pressure of 34 KN/m^2 in any direction, as classified as '*protected members'* (Regulation 2.03.1) and are not required to be removed.

Perhaps the most commonly adopted solution will be the recommendations of Option 2. This option combines the specific provisions of Option 3 with regard to horizontal elements with the more general approach of alternative paths of Option 1 for vertical elements. This option will find favour because buildings have concrete floors in which it is relatively easy to accommodate any additional horizontal ties, whereas vertical tying may present difficulties.

In Sections 3.00 & 4.00 extensive reference has been made to Handbook to BS 5628 pt1[3].

5.00 MOVEMENT JOINTS (Regulation 1.05.4)

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

The effectiveness of movement joints depends on their location. In masonry construction there are two distinct types of movement joint: the first is a primary movement joint that should divide the structure into individual sections; the second consists of secondary movement joints that divide the elements into individual portions. The structure or element on each side of the joint should be independently stable and robust.

In all forms of movement joint it is essential to continue the joint through any finishes (e.g. plaster), attached cladding and similar elements.

5.02 Primary movement joints are used to reduce the influence of overall dimensional changes or distortions of the total structure, and are usually positioned at changes in direction, significant changes in dimension of plan or height, or changes in the form of construction either of the structure or of its foundations. In long uniform structures these joints would normally be provided at 40 to 50m centers and be at least 25mm in width.

Primary movements joints should pass through the whole of the structure above ground level and be in one plain. Consideration should be given to the need to carry the joint through the foundations.

5.03 The purpose of secondary movement joints is usually to accommodate differential movements arising from material behaviour and/or local structural distortions.

5.04 To be noted from table 1, 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.

5.05 Some examples requiring attention are:

- masonry panels on suspended beams or slabs that may crack because of the support deflections
- diaphragm action of floors transmitting lateral forces to strongpoints or shear walls
- lateral restraint to walls by floors and vice-versa. The bearing length of precast prestressed slabs being important as too much fixity may cause cracking to the top face.
- infill masonry panels (which should be individually supported and connected to the surrounding frame)
- uplift and suction arising from wind for lightweight roof construction (special attention needed at roof/wall junctions)
- shrinkage of *in situ* concrete where supporting or supported by masonry units.

Particularly in cases of precast concrete floor units, the designer must satisfy him/herself that the elements can act as horizontal diaphragms where so assumed and that the connections can transmit the forces resulting from the interaction.

Lateral deflections of a reinforced concrete or steel frame may induce cracking of infill cladding; frame shortening may impose load on infill masonry unless a horizontal compression joint is provided.

Masonry infilling may be used to provide the bracing to reinforced concrete or steel framed structures. In such circumstances the walls are not usually required to carry gravity loads from the structure but are subjected to in-plane loads. Where the infill also provides the cladding to the building it will also need to resist wind loads normal to the wall. Due consideration must be given to the effects of possible removal of such walls at a later date.

Infill masonry panels when used as bracing should be fixed tightly to the surrounding structural frame for the efficient bracing to the structure. Regard should be paid to the possible shrinkage of concrete block masonry panel making the pinning ineffective. Movement joints within the panel, either primary or secondary, should be avoided. Similarly, openings that might impair the ability of the panel to brace the structure should be carefully examined. Load sharing arising from secondary effects (e.g. frame shortening) must be considered.

Infill masonry panels that resist only laterally imposed loads should be adequately restrained. This may be on two opposite sides to avoid an unrestrained corner. The methods of restraint must make due allowance for any relative movement between the masonry infill and the structural frame.

Unless the walls are designed to provide principal or secondary stability, it is rarely necessary to consider the influence of accidental damage to masonry infilling since its removal should not precipitate collapse

Table 1 - 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	

Note -* *Broadly but not linearly related to f_k, the characteristic compressive strength*

5.06 After construction, buildings are subject to dimensional changes, which may be caused by one or more of the following factors:

(a) change in temperature

(b) seasonal change in moisture content

(c) long-term absorption of water vapour

(d) chemical actions e.g. carbonation

(e) deflection of supporting structure under loads/creep

(f) ground movement/differential settlement.

In general, because restraints are often present, masonry is not completely free to move, and forces may develop that may lead to bowing or cracking. Masonry units of markedly different characteristics should not be bonded but should be effectively separated by a movement joint or slip plane. It is essential to consider provision for movement at the design stage.

5.07 Proper movement joints need, therefore, to be included at appropriate intervals to allow for thermal and other types of movement in the structure. Such movement will, of course, act in the vertical as well as horizontal direction, although units do not restrain the mortar in the vertical direction. The determination of movement is complex as is not merely a summation or subtraction of extremes of thermal and moisture movement, creep, deflection and so on. Additional shrinkage of concrete units and mortar can occur as a result of carbonation, although it is extremely small.

Materials used in buildings have different rates of thermal and other types of movement including moisture shrinkage as per table 2.

Table 2 - gives approximate coefficients of thermal expansion per ^{o}C change in temperature & range of moisture shrinkage for different materials.

MATERIAL	COEFFICIENT OF THERMAL EXPANSION/ ^{0}C 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.

5.08 Referring to Regulation 1.05.5 for fire requirements reference is to be made to Building Regulations 2000, Technical Guidance Approved Document C.

In section 5.00 extensive reference has been made to "Manual for the design of the plain masonry in building structures"[7].

6.00 DAMPPROOF COURSES (dpcs) (Regulation 6.02.2)

Despite the widespread use of damp proof causes in masonry elements, their structural properties, particularly in tension, have not been widely studied. Current British Standards do not define structural performance requirements.

The principal factors to be considered are:

- resistance to squeezing out due to compressive loads
- ability to resist sliding and/or shear stresses
- adhesion to mortar so that flexural stresses may be transmitted.

In general, advice on the resistance to compression, tension, sliding and shear should be sought from the manufacturers. In particular it should be noted that the flexural strengths of dpcs are particularly suspect.

Dpcs, whether flexible or rigid, should not be pointed or rendered over since this will allow water to by-pass the dpc. Changes in directions of dpcs whether horizontal or vertical and the junctions between horizontal and vertical dpcs, may, if not properly designed or considered, direct water into the building.

7.00 MORTARS (Regulations 6.02.3)

7.01 Mortars should be selected on the ground of strength, durability and economy. There is no evidence to suggest that the use of a weaker cement mortar gives an increasing ability to accommodate movement. However, where cracking is likely to occur, the use of strong (cement-rich) mortars with weak units can give rise to cracking of the units and should generally be avoided (see Table 3).

7.02 Choice and grading of the sand has a significant effect on workability. Sands not conforming to BS1200 seem acceptable, provided that the strength requirements are met. Plasticisers are often used in lieu of lime to improve the workability and divisibility of mortars. They do not, however, provide the extra gain of strength with time, possible with lime.

Table 3 - Mortar mixes from BS5628 Pt 1[3]

Mortar designation	Types of mortar (proportion by volume)		Mean compressive strength at 28 days (N/mm^2)	
	Cement: lime: sand	Cement: sand with plasticiser	Preliminary (laboratory) tests	Site tests
(i)	1:0 to ¼: 3	-	16.0	11.0
(ii)	1:1/2:4 to 41/2	1:3 to 4	6.5	4.5
(iii)	1:1:5 to 6	1:5 to 6	3.6	2.5
(iv)	1:2:8 to 9	1:7 to 8	1.5	1.0

Table 4 gives the strengths of Maltese Mortars from tests carried out by Debattista (1985)[8]

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 (iv)	0.58	3.5
Cement, Carolline Sand, Fine Globigerina Sand	1:2:6	4.48 (iii)	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

7.03 The inclusion of lime (Regulation 6.02.3a) in our mortars is to be advocated as it improves workability, water retention and bonding properties. Lime mortar is softer and less rigid than cement, and can accommodate slight movement and settlement. Lime is more porous and allows the wall to breathe, reducing the effects of rising damp. Lime mortar takes longer to achieve strength and so limits the speed of rate of laying.

7.04 Mortar joints may be finished in a number of ways. When this is carried out while the mortar is still fresh it is termed 'jointing'. When the mortar is allowed to stiffen and some is then removed and replaced with fresh mortar (sometimes coloured) before finishing, the process is referred to as 'pointing'. Jointing is preferable to pointing because it leaves the bedding mortar undisturbed.

Mortar used for pointing should have mix proportions similar to those used in the bedding mortar.

For all types of masonry, it is essential to fill all the joints to minimise the risk of rain and fire penetration (Regulations 1.05.3 & 1.05.5).

It is also important to avoid pointing over dampproof courses (dpcs). This could provide a passage for water to bypass the dpc and cause mortar to crumble as the dpc settles.

8.00 WALL TIES (Regulation 6.02.4n)

Wall ties should comply with BS 1243 [9]. In situations of severe exposure, or where required by building regulations, suitable stainless steel or non-ferrous ties should be used. The most frequently specified ties are either of low carbon steel protected with a zinc coating to BS 729 or minimum weight of coating 940g/m^2, or grade 304 austenitic stainless steel.

Serious consideration should be given to the selection of ties of adequate durability, particularly when a life of at least 60 years required, during which the minimum margin of safety is not reduced.

CHAPTER 2 – MASONRY STRENGTH CRITERIA

9.00 LOAD BEARING PROPERTIES OF MASONRY WALL PANELS

Masonry is a composite material. Its strength is dependent on the crushing strength of the masonry block and of the infilling mortar used. It also depends on the workmanship. The most common workmanship defects are:

a. The horizontal bed joins should be filled completely with mortar. Incompletely filled bed joints may reduce the strength of masonry panels by 33%. Failure to fill vertical joints has little effect on the compressive strength but are undesirable for weather and force, exclusion and sound insulation.

b. Mortar bed joints should not be thicker than 10mm (Regulation 6.02.3b). Bedjoints of 16 –19mm thickness, result in a reduction of compressive strength of up to 25% as compared with 10mm thick joints. 12mm bow or out of plumb also reduces compressive strength by 15%.

c. Before laying mortar the block is to be well wetted to reduce its suction rate, plus a proportion of lime in the mortar mix will help the mortar mix to retain its water. A high absorbent block will result in a weaker mortar, with a resulting weaker wall panel.

10.00 CHARACTERISTIC COMPRESSIVE STRESS f_k OF NATURAL STONE MASONRY (Regulation 5.05)

10.01 Where masonry is constructed from large, carefully shaped pieces with relatively thin joints, its loadbearing capacity is more closely to the intrinsic strength of the stone than is the case where small structural units are used. Design stresses in excess of those obtained from tables 5-7 below may be allowed in massive stone masonry, provided the designer is satisfied that the stone warrants an increase.

10.02 Tests by Buhagiar (1985)[10] on 26 1/3 scale wall panels crushed to destruction with mortar beds fully filled, were shown to abide by tables 5-7. BS5628 Pt 1[3] recognises the effect of the shape factor on the strength of a block., with the greater the proportion of mortar per unit area of lock the lower the strength of the wall panel. The following tables cater for the effect of different block thicknesses.

Table 5 - Characteristic Compressive stress f_k of 225mm thick masonry N/mm^2 for specified crushing strength – as per BS 5638 pt 1[3]

Mortar Designation	*Globigerina*				*Coralline*
	Compressive Strength of Unit (N/mm^2)				
	15	***17.5***	***20***	***35***	***75****
I	8.6	9.6	10.6	16.3	27.4
II	7.6	8.4	9.2	13.4	22.6
III	7.2	7.7	8.3	12.2	
IV	6.3	6.8	7.4	10.4	

** as per BS 5628 pt2[11]*

Table 6 - Characteristic Compressive stress f_k of 150mm thick masonry N/mm^2 for specific crushing strength – as per BS 5628 pt1[3]

Mortar Designation	*Globigerina*				*Coralline*
	Compressive Strength of Unit (N/mm^2)				
	15	***17.5***	***20***	***35***	***75****
I	11.4	12.5	13.7	21.2	36.4
II	9.8	10.8	11.9	17.5	28.6
III	9.3	10.0	10.8	15.8	
IV	8.2	8.9	9.7	13.5	

**as per BS 5628 pt2[11]*

Table 7 - Characteristic Compressive stress f_k of 180mm thick masonry N/mm2 for specified crushing strength – as per BS 5628 pt1[3]

Mortar Designation	*Globigerina*				*Coralline*
	Compressive Strength of Unit (N/mm^2)				
	15	***17.5***	***20***	***35***	***75****
I	9.9	11.0	12.2	18.7	31.6
II	8.7	9.6	10.5	15.4	24.8
III	8.2	8.8	9.5	14.0	
IV	7.2	7.8	8.5	12.0	

** as per BS5628 pt2[11]*

10.03 Regulation 6.02.1c stipulates the minimum characteristic compressive stress of load bearing masonry to be 15N/mm^2. From tests carried out by Cachia (1985)[12] on local masonry, the highest crushing value on a dry sample was 32.9N/mm^2, with the corresponding lowest at 15N/mm^2. The highest value was obtained on a "sol" sample, being the densest and having the lowest void ratio and porosity. The stress in the N direction (i.e. 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%. Internal walling may be considered to be in a dry condition, whilst for external walling an intermediate value to be taken.

10.04 Porosity is the volume of pores within a stone, expressed as a % of the total volume. Values range around 10 –20%, although they may be as low as 10% and as high as 40%. The value for franka is around 35%. A sol sample has a low at 27.8% Cachia (1985)[12]. Values for coral limestone are in the region of 16% Bonello (1988)[13].

10.05 Microporosity is the proportion of the total pore space of pores having an effective diameter less than 5 microns. A stone with high proportions of very fine pores is less durable than a stone that has mainly coarse pores. The value for franka samples falls between a grey middle of 50 - 80 %, which on its own merit may not be used to classify its durability characteristic. An improved indication of durability may sometimes be obtained by combining two properties. Camilleri (1988) [14].

10.06 For the franka samples tested by Cachia (1985)[12] it was concluded that a wet/dry compressive strength ratio of 0.58 appears to mark a dividing line between a better and a poorer stone. For the franka samples tested by Cachia (1985)[12] this appears to be confirmed, however a dividing line between a very poor sample (0.56) and a very good sample (0.59) is too fine and a better indication of durability appears to be obtained by dividing the wet/dry strength ratio by microporosity and multiplying the result by a factor. Camilleri (1988)[14].

11.00 RANDON RUBBLE MASONRY

The characteristic compressive strength f_k is to be taken at 75% of the corresponding strength for natural stone, built in similar materials. For the case built in lime mortar to be taken at 50% for masonry in mortar designation iv.

12.00 CHARACTERISTIC COMPRESSIVE STRESS f_k OF HOLLOW CONCRETE BLOCK WALLS (Regulation 5.04)

12.01 For hollow blocks, the characteristic compressive strength quoted when tested according to the relevant BS, the gross plan area is referred to, as though it were solid. The panel strength is obtained from tables 8-10. Blocks less than 100mm thickness are intended for non-loadbearing partitions, with the lowest crushing strength being not less than 2.8N/mm^2. Regulations 6.02.1c specify that for simple design the characteristic compressive stress has to be not less than 7N/mm^2. It is important to bond the units in a pattern, which ensures that the webs are aligned vertically, with the maximum height that should be normally built in a day not exceeding 1.5m.

12.02 For infilled blocks, the unit is treated as solid with the characteristic compressive stress now calculated on the net instead of the gross area. Its panel characteristic stress is then taken from the appropriate table 4, or 5. For a stronger infill, the strength of the hollow blockwork assumed, whilst for a weaker infill the strength of the infill taken for calculating the panel characteristic strength.

12.03 The average value of the drying shrinkage should not exceed 0.06%.

Table 8 - Characteristic Compressive stress f_k of 225 thick concrete hollow blockwork in N/mm^2

Mortar Designation	***Compressive Strength of Unit (N/mm^2)***							
	2.8	***3.5***	***5.0***	***7.0***	***10***	***15***	***20***	***35***
I	2.0	2.5	3.6	4.4	5.1	6.3	7.4	11.4
II	2.0	2.5	3.6	4.2	4.8	5.6	6.4	9.4
III	2.0	2.5	3.6	4.1	4.7	5.3	5.8	8.5
IV	2.0	2.5	3.1	3.7	4.1	4.7	5.2	7.3

Table 9 - Characteristic Compressive stress f_k of 150 thick concrete hollow blockwork in N/mm^2

Mortar Designation	***Compressive Strength of Unit (N/mm^2)***							
	2.8	***3.5***	***5.0***	***7.0***	***10***	***15***	***20***	***35***
I	2.6	3.2	4.6	5.4	5.9	6.7	7.4	11.4
II	2.6	3.2	4.6	5.2	5.5	6.0	6.4	9.4
III	2.6	3.2	4.6	5.1	5.3	5.6	5.8	8.5
IV	2.6	3.2	4.1	4.5	4.7	5.0	5.2	7.3

Table 10 - Characteristic Compressive stress f_k of 115 thick concrete hollow blockwork in N/mm^2

Mortar Designation	***Compressive Strength of Unit (N/mm^2)***							
	2.8	***3.5***	***5.0***	***7.0***	***10***	***15***	***20***	***35***
I	2.8	3.5	5.0	5.7	6.1	6.8	7.5	11.4
II	2.8	3.5	5.0	5.5	5.7	6.1	6.5	9.4
III	2.8	3.5	5.0	5.4	5.5	5.7	5.9	8.5
IV	2.8	3.5	4.4	4.8	4.9	5.1	5.3	7.3

12.04 Grech (1997)[15] carried out a study on local concrete blockwork. He notes that the production of the strength of the blocks depends on the year of manufacture, leading him to conclude that strength is dependent on the weather, related to the amount of rainfall during the winter and the hot weather in summer.

The compressive strength of the blocks was analysed for various suppliers over the period 1991 –1996. The following table lists the average characteristic strength and coefficient of variation over the period.

Table 11 – 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)[15]

13.00 DIMENSIONS & TOLERANCES OF CONCRETE BLOCKS

The maximum deviation on the sizes of units are as follows.

Length +3mm and –5mm
Height +3mm and –5mm
Thickness +2mm and –2mm average
+4mm and –4mm at any individual point.

14.00 CHARACTERISTIC COMPRESSIVE STRESS f_k OF REINFORCED CONCRETE INFILLED BLOCKWORK

14.01 BS5628 Pt2[11] specifies that it is preferable to use grade I or grade II mortar, although grade III may be used in walls incorporating bed joint reinforcement. The concrete infill should consist of the following proportions by volume

1: 0 to ¼ : 3 : 2 cement :lime : sand, or else a prescribed mix of grade 25, with 10mm maximum aggregate size. Jointing of successive pours should be made about 5 cm below the concrete block surface.

14.02 Concrete infill for pre-tensioned prestressed masonry should be a minimum grade of 40 and of 25 for post-tensioned prestressed masonry work

14.03 The compressive strength of the infilled concrete block is calculated as outlined above for infilled hollow blockwork. The following tables give the characteristic compressive stress of the infilled blockwork for use in reinforced blockwork masonry.

Table 12 - Characteristic Compressive stress f_k of 225 thick infilled concrete hollow blockwork in N/mm^2

Mortar Designation	***Compressive Strength of Unit (N/mm^2)***			
	7	***10***	***15***	***20***
I	4.9	6.3	8.6	10.6
II	4.6	6.0	7.6	9.15

** as per BS 5628 pt2*[11]

Table 13 - Characteristic Compressive stress f_k of 150 thick infilled concrete hollow blockwork in N/mm^2

Mortar Designation	***Compressive Strength of Unit (N/mm^2)***			
	7	***10***	***15***	***20***
I	6.3	8.2	11.2	13.8
II	6.0	7.8	9.9	11.9

** as per BS 5628 pt2*[11]

15.00 BEARING STRESSES

Increased local stresses may be permitted beneath the bearing of a concentrated load. For the normal type of bearing above stresses may be increased by 1.5, although the range varies from 1.25 up to 2.0 as outlined in BS 5628 pt1[3]. It also permits the load to be dispersed at 45^{O} through the masonry for the purpose of checking the design strength at 0.4h.

16.00 DESIGN STRENGTH

16.01 The design strength is equal to the characteristic strength divided by the partial factor for material strength. The partial safety factors listed in BS 5628 Pt 1 & 2[3&11] are as in table below.

Table 14 - Partial Safety factors γ_m for material strength for normal design loads.

Material	*Special Category*	*Normal Category*	*BS 5628*
Masonry			
Compression	2.5	3.1	Pt1
Compression/flexure	2.0	2.3	Pt 2
Flexure	2.8	3.5	Pt1
Shear	2.5	2.5	Pt1
Shear	2.0	2.0	Pt 2
Bond	1.5	1.5	Pt2
Strength of steel	1.15	1.15	Pt 2
Wall ties	3.0	3.0	Pt 1

16.02 When considering the probable effects of misuse or accident, the values given should be halved.

16.03 Special Category may be assumed when preliminary compressive strength tests carried out on the mortar indicate compliance with the strength requirements and regular testing of the mortar on site shows compliance with the strength requirements. The compressive strength of the structural units supplied, with not more than 2.5% falling below the acceptance limit.

Normal category applies when the requirements of the special category are not met.

17.00 DESIGN LOADS IN KN/M FOR NORMAL CATEGORY – $f_k t/\gamma_M$

Table 15 - Design axial loads for various wall types

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			1301

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.

18.00 VERTICAL DESIGN LOAD RESISTANCE – $F_K A/\gamma m$

18.01 For walls or columns with a plan area less than 0.2m^2, above loads are to be reduced by (0.7 + 1.5A), where A is the loaded cross-sectional area in m^2.

18.02 Table 15 applies for short walls, defined as having a slenderness ratio (effective height or effective length / effective thickness) less than 8. For slender walls a reduction coefficient is obtained from table 7 in BS 5628 pt1[1]. This table also gives reduction coefficients for slenderness combined with load eccentricities on walling, although eccentricities less than 1/20 of the thickness (0.05t) may be ignored. The primary assumption is that the load transmitted to a wall by a single floor or roof acts at 1/3 of the depth of the bearing area from the loaded face of the wall. *However in the case where the floor or roof is very stiff (eg concrete) the load may be considered to be axial provided the loads and spans on each do not exceed the other by 50%.*

The slenderness ratio should not normally exceed 27, where the slenderness coefficient reduces to 0.4 from a coefficient of 1.0 at a slenderness ratio of 8, considering no eccentricities. For walls less than 90mm thick the slenderness coefficient should not exceed 20, in agreement with Regulation 6.02.4b.

18.03 The effective thickness for double walling where bonding is by a bondstone is taken as the total thickness of the construction where the air cavity is less than 100mm. (Regulation 6.02.4m). Where metal ties are used as an alternative to bondstones the effective thickness is taken at 2/3 the total thickness. (Regulation 6.02.4n) Guidance exists regarding the effective thickness for piered wall construction in BS 5628 Pt1[3].

The effective height of a wall may be taken at 0.75 times the clear distance between lateral supports that provide resistance to lateral movement, this being the case for heavily loaded walls. With simple lateral supports this is taken at 1.0.

Again the effective length is taken at 0.75 or 1.0 times the clear distance, as above, or else 2 ½ times the distance between a support and a free end.

18.04 For masonry compression members of irregular planform the capacity reduction factors should be written in terms of L/r slenderness ratios and Z/A eccentricity ratios. For the method refer to Morton (1991) [16].

19.00 CHARACTERISTIC SHEAR STRENGTH f_r OF MASONRY

19.01 There are several types of shear failure of masonry. Vertical shear may occur, particularly at the junction of the intersecting walls, in which masonry units bonding the walls together will suffer shear failure. Horizontal shear may occur along bedding surfaces, particularly at the level of the damp-proof membranes. Both diagonal and horizontal shear resistance are dependent on vertical stress in the masonry and recommendations relate to this condition.

19.02 Tests carried out on franka (Saliba 1990)[17] gives an unconfined shear strength varying from 2.2 to 3.85 N/mm^2.

The characteristic shear strength of masonry in the horizontal direction is given by BS5628 pt1[3] at

$0.35 + 0.6g_a$ N/mm^2 with a max of 1.75N/mm^2 for walls in mortar designation i, ii &iii
$0.15 + 0.6g_a$ N/mm^2 with a max of 1.4 N/mm^2 for walls in mortar designation iv

where g_a is the design vertical load per unit area.

Horizontal shear may occur along bedding surfaces, particularly at the level of damp-proof membranes (Regulation 6.02.4i). Further guidance may be obtained from (Saliba 1992)[18].

19.03 In the vertical direction shear failure may occur particularly at the junction of intersecting walls and is given by

For masonry	0.7N/mm^2 for mortar designations i,ii & iii. 0.5N/mm^2 for mortar designation iv.
For blockwork	0.35N/mm^2 with a minimum strength of 7N/mm^2.

Alternatively for reinforced sections, as per BS 5628 pt 2[11] the characteristic shear strength of masonry is given by 0.7N/mm^2, provided that the ratio of height to length of the wall does not exceed 1.5.

20.00 COEFFICIENT OF FRICTION

This may be taken at 0.6 between clean concrete and masonry faces. The main use of friction probably lies in design to resist accidental damage.

21.00 CHARACTERISTIC FLEXURAL STRENGTH - f_{xk}

21.01 In general direct tension should not be allowed for in masonry. The design methods outlined in BS 5628 pt1[3] for laterally loaded wall panels and freestanding walls rely on a knowledge of the flexural strength of masonry, obtained from tests carried out in bending or flexure. Where direct tension is to relied upon, such as resisting wind uplift or accidental loads, then the direct tensile stress should be limited to ½ the flexural strength. Flexural tensile stresses should not generally be allowed at damp-proof courses, but partial fixity may be provided due to the action of dead loads.

21.02 Tests carried out by Saliba (1990)[17],found that flexural strengths on dry franka samples varied from 1.1 – 4.7 N/mm^2 with an average value of 3.8 N/mm^2. In general this value varied from 1/5 to 1/6 of the compressive strength. For saturated samples the values varied from 1.2 – 3.7 N/mm^2.

21.03 BS 5628 pt1[3] defines two principal directions of flexural failure. The weaker direction is along the bedding plane, with the stronger direction being perpendicular to the bed joint. μ is the ratio of flexural strength, when failure is parallel to the bed joints to the flexural strength when failure is perpendicular to the bed joints.

Table 16 gives the flexural f_{xk} values in the relative directions in N/mm^2.

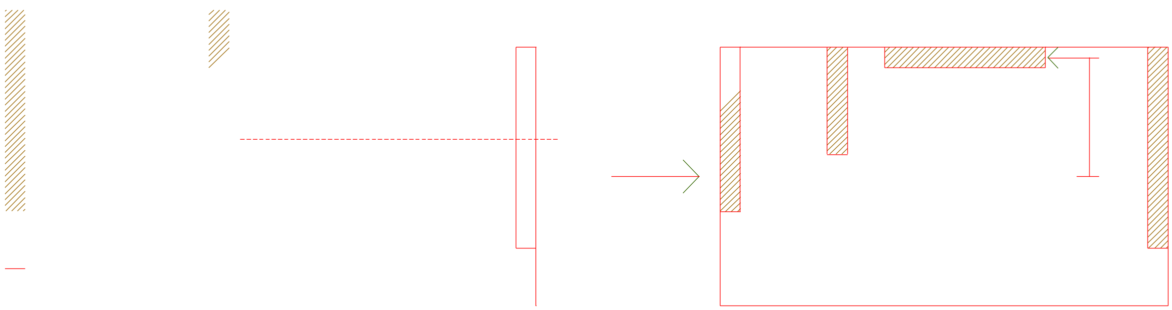
Concrete blocks of compressive strength N/mm^2	***Plane of failure parallel to bed joint***		***Plane of failure perpendicular to bed joint***	
Mortar Designation	I, ii & iii	iv	I, ii & iii	iv
2.8	0.25	0.20	0.40	0.40
3.5	0.25	0.20	0.45	0.40
7.0	0.25	0.20	0.60	0.50
10.5	0.25	0.20	0.75	0.60
14.0 and over	0.25	0.20	0.90*	0.70*

When used with flexural strength assume an orthogonal ratio of 0.30

21.04 BS 5628 pt1[3] table 9 gives coefficients for the calculation of bending moments Mxx in the plain vertical to the bed joint due to lateral loading given by $Mxx = \alpha\, W_k \gamma_f . L^2$.

These are worked for panels of various sizes supported on 3 or 4 sides with varying conditions of fixity, according to the yield line theory, which has been found as a reasonable method for predicting the capacity of walls. The support conditions have to be assessed first. Table 17 is an abridged version of the coefficients found in BS 5628 pt1[1].

Table 17 – Bending moment coefficient for two way spanning panels subjected to Lateral loads (μ= 0.35)



A free edge is easily identified, but some judgment is necessary in deciding between simply supported or fixed. The effects of dpcs needs to be considered in lateral loading. Their presence complicates the design since they generally act as a discontinuity in a laterally loaded wall. Some continuity is however, still possible because of vertical stresses induced due to loading from above.

21.05 Table 17 gives the flexural strengths for an orthogonal ratio of 0.35, however when vertical load acts so as to increase the flexural strength in the parallel direction, the orthogonal strength ratio may be modified by adding the stress due to the design vertical load to the horizontal flexural stress and coefficient obtained from BS 5628 pt1[3] table 9.

The lateral load is to be taken at uniformly distributed, so for water pressure in a built-up reservoir the triangular water pressure distribution is to be averaged out to a udl.

For guidance on reinforced & prestressed wall panels subjected to lateral loading refer to Golding (1991)[19].

21.06 For free-standing walls BS 5628 pt1[3] Cl 36.5, the design moment of resistance is given by:

$$\left(\frac{F_{xk}}{\gamma_m} + g_d\right) Z$$

where flexural strength cannot be relied upon because of the type of dpc used, use:

$$\frac{nw}{2}\left[t - \frac{n_w \gamma m}{f_k}\right]$$

where f_{xk} is the characteristic flexural strength

g_d is the design vertical dead load per unit area

Z is the section modulus

n_w is the design vertical load per unit length of wall taken at $0.9G_k$

f_k is the compressive characteristic strength of masonry

γm is the material factor of safety

CHAPTER 3 – STRUCTURAL ELEMENTS PRELIMINARY GUIDANCE

22.00 FREE STANDING WALLS (Regulation 6.02.4i & j) & Wall Panels

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

Table 18 – 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.

22.02 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.

23.00 EARTH RETAINING WALLS (Regulation 6.02.4k)

23.01 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 19 – 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.

23.02 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.

23.03 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/m^2) 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.

23.03 As partial safety factors are included in the limit state approach, refer to para. 32.01, 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.

24.00 MASONRY ARCHES (Regulation 6.02.5a)

24.01 There is ample evidence that masonry arches tend to deform when centering is removed and that 3 hinges can form under the action of dead load alone. Sometimes this is due to shortening of the arch itself under compression, especially in the case of flat arches. At other times, it may be due to abutment spread at the springings. Whatever the cause, 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.

24.02 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 vertical reactions and applied loads. These moments are balanced by the 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 f_m is the characteristic compressive stress in the masonry, which should not exceed the masonry bearing stress, given above as $1.5f_k$. (obtained from tables 5-7), not normally a design constraint. The minimum of the masonry stressblock to support this thrust may be calculated, from the maximum depth masonry bearing stress. Hence with a given altered height from the springing to the centre of stress at the crown, gives a new reduced thrust value $H_{A.}$

b is the breath and d is the depth of the arch section.

24.03 The thrust of the arch at the springing attempts to move a volume of masonry and thus it is necessary to check the resistance of the wall to the horizontal thrust. This thrust is resisted by 2 plains and the width of abutment may be calculated from

$X = \gamma H_A / 2 f_v t$

Where X is the width of abutment
f_v is the characteristic shear strength of the masonry – refer to para19.03.
γ_m the material partial safety factor as per table 14
t is the thickness of the abutment
& H_A is as para. 24.02.

25.00 PROPERTIES OF MALTESE CLAYS (Regulation 6.01 table B.01)

25.01 Referring to Mr. A. Cassar A&CE, from various insitu tests carried out using SPT and laboratory tests on recovered samples, Maltese clays may be described as stiff to very stiff in its natural state, having an average C value of 100KN/m^2, with a lower limit of 50 and an upper limit of 200. Also the plastic limit (PL) of clay is given at 23%, with the liquid limit (LL) at 70% (Bonello 1988)[13]. The plasticity index (PI) is thus given by

PI = LL – PL = 47%

From the Casagrande plasticity chart this is classified as an inorganic clay of high plasticity.

From BS 8004[20] table 1, stiff clays have a presumed alloweable bearing value of 150 to 300KN/m^2, whilst very stiff clays have values varying from 300 to 600 KN/m^2.

For a PL at 23% and a high clay content, the shrinkage and swelling potential of Maltese clays is classified at high, usually showing cracks on drying.

25.02 Due to the heaving and shrinkage characteristics, the top layer of a clay formation is to be removed for a minimum depth of 750mm and a duly compacted layer of hard spalls laid prior to casting of any foundation works. The foundation and fully compacted fill tend to act compositely and therefore resist the heave forces being applied, providing a more uniform bearing that will cushion the heave effects.

26.00 BASIC WIND SPEED (Regulation 4.02.1)

This is taken as the maximum gust speed likely to be exceeded on the average only once in 50 years at 10m above the ground in open level country as defined in CP3 : ChV : Pt2 :1972[21].
Abdelnaby & El-Heweity (2001)[22] from various tests at Luqa over a 20 year period have calculated this basic wind speed for Malta at 47m/s.

Note that the basic wind speed in BS6399 pt2:1997[23], is defined as the mean hourly wind speed at 10m above open country at sea level, estimated to have an annual probability of exceedance of 0.02, irrespective of direction. From this basic wind speed the site wind speed is calculated with an appropriate probability of exceedance, then taking the terrain category and the structural factor into account. From the UK maps of the relevant wind speeds, it is to be noted that for a maximum gust speed of 47m/s, the mean hourly wind speed is taken at 23m/s.

Table 20 gives the wind pressure in KN/m^2 for various building heights and various terrains for a basic wind speed of 47m/s and where the greater horizontal and vertical dimension do not exceed 50m, as per CP3 : ChV[21].

H – m	***Sea front with a long fetch***		***Countryside with scattered wind breaks***		***Outskirts of towns and villages***		***Town centers***	
		cladding		***cladding***		***cladding***		***cladding***
3 or less	1.05	1.12	0.90	0.97	0.81	0.86	0.70	0.76
5	1.12	1.19	1.00	1.07	0.88	0.95	0.74	0.81
10	1.28	1.35	1.19	1.26	1.00	1.05	0.84	0.90
15	1.34	1.39	1.28	1.35	1.12	1.19	0.93	1.00
20	1.36	1.43	1.32	1.39	1.22	1.28	1.01	1.07
30	1.42	1.47	1.39	1.44	1.31	1.36	1.15	1.21
40	1.46	1.51	1.43	1.48	1.36	1.42	1.26	1.31
50	1.49	1.54	1.46	1.49	1.40	1.46	1.32	1.38

The cladding values in table above, apply to all units of cladding, glazing and their immediate fixings.

The distribution of the wind forces into the various vertical structural elements distributed via the rigid floor elements is discussed in Section 30.00 as for Earthquake forces, which however has a triangular distribution in elevation, together with possibly a top force, whilst for wind loading a stepped vertical loading is more appropriate.

27.00 SEISMIC ZONING (Regulation 4.02.2)

The Zone 2 specification of the UBC - 85 code building[24] is equivalent to an earthquake intensity of MMVII., subjected to an acceleration varying from 0.05g to 0.10g.

These regulations by referring to Regulation 2.02.1 are mandatory only to a limited range of buildings. The basic philosophy being the continuance of the infrastructure and hospital services, and least disturbance to the prisons and people with some impairment. Buildings with large assemblies of people, exceeding 100 persons together with freestanding buildings exceeding 24m in height also fall under this category.

28.00 EARTHQUAKE DATA

28.01 The following facts ought to guide the perit in advising his client on the advantages a particular building not listed above may gain by being made earthquake resistant or the advantages of retrofitting an existing building.

28.02 Presently a seismic risk hazard analysis has not yet been drawn up for the Maltese Islands, but from the limited data available, the return periods are approximated as per table below.

Table 21 – Return Periods for Earthquake Intensity

MM – Earthquake Intensity	***Return Period (years)***	***Base Shear Design % of g***
VI	333	2 –5
VII	1800	5 –10
VIII	100,000	10- 20

Camilleri (2001)[25]

Table 22 - Types of Building for damage due to Earthquake Exposure

Type	Description	Base shear design % of gravity
A	Building of fieldstones, rubble masonry, adobe, and clay. Buildings with vulnerable walls because of decay, bad mortar, bad state of repair, thin cavity brick walls, etc.,	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 noticeably affected by deterioration;	0.9
D_1	Buildings with a frame (structural members) of reinforced concrete	2-3
D_2	Buildings with a frame (structural members) of reinforced concrete	3-4
D_3	Buildings with a frame (structural members) of reinforced concrete	6
D_4	Buildings with a frame (structural members) of reinforced concrete	12
D_5	Buildings with a frame (structural members) of reinforced concrete	20

Source: Swiss Re (1992)[26]
NOTE: the subscript to a D Building denotes the base shear to be resisted, as given in adjacent column.

28.03 In Malta a few buildings are classified as type B. These would be restricted to old rural deteriorated dwellings exceeding 150 years in age or old deteriorated buildings in Valletta, which due to little maintenance, stability has been impaired due to ingress of water. Type A are limited to deteriorated old agricultural sheds found in fields. Most masonry buildings and most buildings in concrete frame would be classified as conforming to type C. The more rigid buildings, satisfying stiffness regularity and symmetry in plan/elevation layout, are classified $D_{1.}$

By comparing the base shear as a % of 'g' to be resisted in an earthquake of particular intensity from tables 21 & 22, it is to be noted that for no damage to be suffered during an MMVI, building type to be D2/D3, during MMVII building type D3/D4 and at MMVIII building type D5. The above reinforces the fact quoted in codes that unreinforced masonry is disadvantageous against earthquakes, with types A to C buildings only resisting a nominal base shear. Consequently, it is not feasible with masonry construction to design an aseismic building above a certain level. It is recommended that reinforced blockwork construction, reinforced concrete or steel construction be used instead.

28.04 **The Mean Damage Ratio (MDR)** table 23 is the average damage to buildings of about identical vulnerability and architectural characteristics, expressed as a percentage of their new value.

Table 23 - Mean Damage Ratio (MDR) For Building Type Against Earthquake Intensity founded on rock, being moderately asymmetrical & irregular.

BUILDING TYPE	**A**	**B**	**C**	**D_1**	**D_2**	**D_3**	**D_4**
EARTHQUAKE INTENSITY	MDR	MDR	MDR	MDR	MDR	MDR	MDR
V	4%	**2%**					
VI	10%	**4%**	**1%**				
VII	45%	**20%**	**10%**	**3%**	*2%*		
VIII	60%	**45%**	**25%**	**12%**	*6%*	*3%*	*1%*
IX	80%	**60%**	**45%**	**30%**	*17%*	*12%*	*6%*
X	100%	**80%**	**65%**	**55%**	*35%*	*25%*	*17%*
XI	100%	**100%**	**100%**	**85%**	*60%*	*50%*	*35%*

Source: Camilleri (1999)[27]

The present majority range of Maltese buildings fall within types B-D_1 represented in bold in table 23.

For buildings founded on softer material than limestone, the **MDR** is taken as the progressively corresponding higher value on the scale. For example if a type C building founded on clay it is subjected to MM-VI, its **MDR** is to be taken at 10%. Further, if founded on a poorly back-filled disused quarry, an **MDR** of 25% to be taken.

From table 23 it is noted that retrofitting a type C building from a type B would reduce the MDR at MMV, from 2% to nil, at MMVI from 4% to 1%, at MMVII from 20% to 10% and for a MMVIII from 45% to 25%. These damage savings may be achieved by modifying our method of construction, with the room corners being in reinforced blockwork, for vertical reinforcement to tie in with the reinforced concrete floor slabs. For aseismic design it is normal for reinforced concrete collar beams to be provided over the load bearing walling at every level, however in case where cast-in-place floor slabs are provided adjoining the top of the walls, collar beams may be omitted as the slabs serve to maintain rigidity to the top of the wall, taking over the transmission of horizontal forces.

28.05 An improvement to robustness in masonry construction may be obtained by: (refer to Fig 2)

1. *openings in exterior walls should be at least 500mm from corners, with the sum of the width of the of openings made less than or equal or equal to ½ of the sum of the wall length in respective directions. Also, for the whole building, total sum of width of openings of each storey should be made less than or equal to 1/3 of the total sum of the length of walls;*
2. *interior doorways should be at least 2 wall thicknesses away from the end of the wall;*
3. *openings in walls should be at least 500mm apart.*
4. *Openings in masonry lintels should be limited to 1.0m. For larger openings precast or cast-in-place reinforced concrete with sufficient bearing should be used.*
5. *Despite the recommendations given in Regulation 6:01, for the purpose of making masonry construction earthquake resistant, it is appropriate to use continuous footings tying the*

bottom of each wall into one body, with the height of footing being not less than 40cm and enough for uniform contact soil pressure and adequate to span large openings.

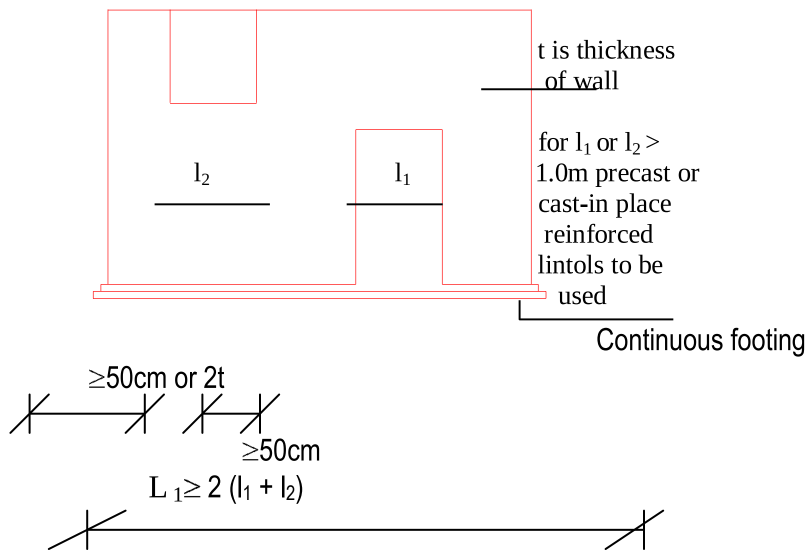


FIG 2 – MASONRY IMPROVED STURDINESS MASONRY FOR ASEISMIC DESIGN

28.06 It is recognised that an asymmetric or irregular design in buildings will suffer a higher mean damage ratio (MDR) than regular structures exposed to the same shaking.

A building may be slightly irregular or asymmetric due to the following factors:

- A small part is of different elevation
- The floor area is reduced from a certain storey upwards
- Elevator shafts or columns are asymmetrically arranged
- A part is of different stiffness

If a building has an "L"- shaped elevation or an "L"-shaped floor plan, or if foundations are resting on different sub-soil, the earthquake exposure is greater.

Elevations are easy to evaluate as regards asymmetry, but it is important to inspect all sides of a building. The inspection of floor plans should take all into consideration, as there could be major differences in plan between the ground and upper floors.

More difficult to assess are irregularities and asymmetries, associated with the internal properties of buildings, e.g. mass, stiffness or dampness.

An elevated water tower is an example of a non-uniform distribution of mass and thus irregularity. A cantilevered canopy could be another example.

28.07 An enhanced factor F_r shall be obtained for a highly irregular building, with abrupt change of stiffness between floors. The MDR's in table 23 are worked out for a weighting factor F_{r1} of 1.3 for irregularity and asymmetry in relation to a recessed elevation of building (shape A1 in table 24a) a similar value for F_{r2} (shape B1 in table 24b) of 1.3 in relation to an L-shaped floor plan whilst a value F_{r3} of 1.5 in relation to internal irregular spans and layout of walls of building (shape C1in table 24c) giving a global factor of

$$\mathbf{F_{rA} = 1.3 \times 1.3 \times 1.5 = 2.5}$$

Table 24 - Amplification factor for anticipated damage to structures, depending on irregularity and asymmetry

(a) Irregularity and asymmetry effects on damage in relation to building elevation

Shape	Elevation	F_{r1}
A1	L-Shaped frame with increased height	1.3
A2	A soft structure introduced at ground level for majority of foot print area, overlying a rigid masonry structure above	4.0

(b) Irregularity and asymmetry effects on damage in relation to floor plan

Shape	Floor plan	F_{r2}
B1	A trapezoidal or L-shaped plan as opposed to rectangular	1.3
B2	A T-shaped plan	1.5
B3	A U-shaped plan	1.8

(c) Irregularity and asymmetry effects on damage in relation to internal features

Shape	Internal properties	F_{r3}
C1	Different spans of irregular arrangements of substantial internal walls	1.5
C2	Continuous window-bands interrupt fill-in wall, producing a short pier effect or substantial transitions in stiffness at ground level, due to large open spans	2.5

**Abridged version of tables obtained from Appendix A of Swiss Re (1992)[26]*

Soft designs encountered locally could incorporate a partial soft ground floor, yielding a F_{r1} factor of 4 (shape A2 in table 24a). A T-shaped floor plan with increased damage probability at both sides of intersection yields a F_{r2} factor of 1.5 (shape B2 in table 24b). For the continuous window bands at upper level yields a F_{r3} factor of 2. 5(shape C2 in table 24c), giving a global factor of

$$F_{rB} = 4 \times 1.5 \times 2.5 = 15$$

The effects of asymmetry lead to an amplification of MDR given by

$$\frac{F_{rB}}{F_{rA}} = \frac{15}{2.5} = 6 \text{ times}$$

The local buildings which fall into this category are Buildings Type C, and D1 and an amended damage ratio matrix (table 25) is proposed to cater for higher asymmetry and irregularity.

Table 25 - Amended Damage Ratio Matrix for Higher Irregularity & Asymmetry

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

28.08 The absence of walls at ground floor implies a substantial transition in stiffness and some difference in mass and damping between the ground and upper floors. During the past 25 years the building construction in Malta has been subjected to changes, brought about from the economic expectations of landed property. A further irregularity in stiffness, frequently found in commercial and public buildings is due to the greater height of the ground floor. Unfortunately this feature is often combined with a soft ground floor, as there are few or no walls lending lateral support to the columns. Such designs make a building a potential death trap.

UBC 88[28] defines a soft storey as one in which the lateral stiffness is less than 70% of that in the storey immediately above or less than 40% of the combined stiffnesses of the 3 stories above. Mass irregularity is considered where the effective mass of any storey is more than 150% of the effective mass of an adjacent storey.

The commercialisation of buildings has opened up the layout especially at ground floor level, obtaining a flexible soft structure, whilst on the upper levels rigid structures in masonry are still being constructed, due to the economic availability of good building stone. Another recent innovation is the availability of precast prestressed slabs, which are ideal for obtaining large open spans necessary for the societal car parking facilities. These slabs, normally sit freely on the supporting structure, with no tying provided to the rest of the structural system. In earthquake design the tying of the various structural system is a requisite to obtain a rigid diaphragm tying the whole building together.

Reference to Camilleri (2000)[29], indicates tying calculations, for prestressed hollow slabs to an underlying garage in a terraced construction, according to BS8100[4]

29.00 EARTHQUAKE FORCES

29.01 From the Chilean experience (Villablanca Frolov, 1988)[30], Chilean engineered masonry buildings have generally behaved well in strong earthquakes. The basic lateral resisting force system consists of numerous structural walls. The actual behaviour of low rise masonry buildings is controlled by the shear failure of their wall elements, with the masonry takes all shear approach feasible to about 5 stories high, which crack in shear at spectral accelerations ranging from 0.30g to 0.40g, being resistant in the MMVIII-IX range. The low rise buildings studied had a wall area ratio varying from 4.6% to 8.6%. This approximates to 2% wall area ratio per floor.

29.02 According to the Uniform Building Code (UBC-85)[24], the minimum total lateral seismic forces assumed to act nonconcurrently in the direction of each of the main axes of the structure is calculated in accordance with the following formula. Further for a Zone 2 location only reinforced masonry is to be adopted with reinforcement placed centrally at 0.60m centers.

V = ZIKCSW where
For Zone 2 buildings classified as per Regulation 2.02.1 this force shall be increased by 1.25.

1. For Zone 1, Z = 3/16
   Zone 2, Z = 3/8
   Zone 3, Z = ¾
   Zone 4, Z = 1.

2. I is the Occupancy Importance Factor, given as

   1.5 for essential facilities
   1.25 for any building where the primary occupancy is for the assembly use for more than 300 persons, in one room.
   1.0 for all others

3. Value of K depends on type of arrangement of resisting element
   Buildings with a box type system = 1.33
   Buildings with a dual bracing system = 0.80 (combination of frame & shear-wall)
   Total ductile frame system = 0.67 (frame resists total lateral force)
   Elevated tanks on 4 or more legs = 2.5

All other building frames not listed = 1.0

4. The product CS need not exceed 0.14
   For a refined value refer to UBC 85[(24)].

5. W is the total dead load due to the weight of all permanent structural and nonstructural components of a building, such as walls, floors, roofs and fixed service equipment. In other codes such as EC8, the total dead load taken plus an estimate of the possible live load that could reasonably be expected. The %'s taken vary from 20% for residential loading to 30% for quasi-permanent storage values up to 60% for frequent storage loadings. UBC88[28] takes 25% for floor load to storage and warehouse loadings.

29.03 Two seismic design procedures exist. The equivalent-static-force procedure and the dynamic analysis. In the equivalent-static-force procedure the inertial forces are specified as static forces using empirical formulae. The formulae are developed to adequately represent the dynamic behaviour of regular structures having a reasonably uniform distribution of mass and stiffness. Dynamic analysis should be used for irregular structures by taking account of its irregularities, including natural frequencies, mode shapes and damping. The notion of irregularity is based on vertical structural and plan structural irregularity as outlined in paras 28.06 & 28.07. UBC 85[24] considers that buildings with setbacks not exceeding 75% in each plan dimension of the corresponding plan dimension of the lower part, may be considered as uniform buildings without setbacks, provided other irregularities do not exist.

30.00 DISTRIBUTION OF SEISMIC LATERAL FORCES FOR REGULAR STRUCTURES

30.01 The total lateral force V shall be distributed over the height of the structure in accordance with the following.
For structures over 7 storeys high a concentrated force at the top shall be calculated from
$F_t = 0.007NV$
Where N is the number of storeys
V is obtained from para 29.02
& F_t should not exceed 0.25V and may be considered 0, when storey height is less than 7.

The remaining portion of the total base shear shall be distributed over the height of the structure according to

$F_x = (V- F_t)w_x h_x / \Sigma wh$

Where w_x is the weight at a particular level designated by x and h_x is the height of a particular level above the shear base to level x. At each floor the force is located at the center of the mass. The Σwh is the summation of the products of all $w_x h_x$'s for the building.

For equal storey heights and weights, this lateral force distributes linearly increasing towards the top (Fig 3). Any significant variation from this triangular distribution indicates an irregular structure.

30.02 The storey shear at level x, V_x is the sum of all the lateral forces at and above that level given by

$$V_x = F_t + \sum_{i=x}^{n} f_i$$

The overturning moment at a particular level M_x, is the sum of the moments of the storey forces above, about that level (Fig 3). Hence

$$M_x = f_t (h_n - h_x) + \sum_{i=x}^{n} f_i (h_i - h_x)$$

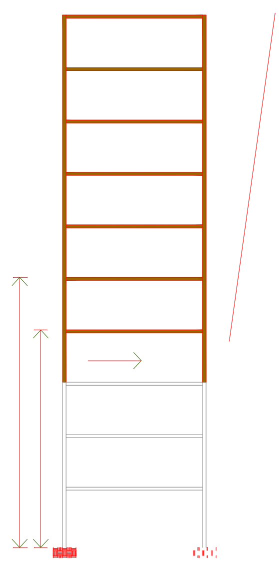


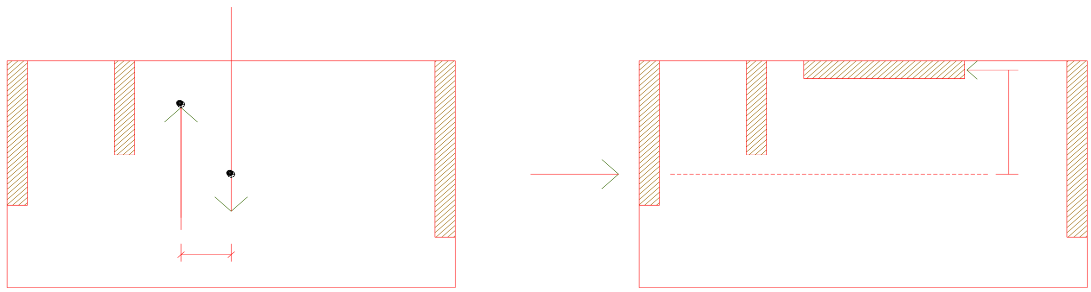
FIG 3 LATERAL FORCE DISTRIBUTION & OVERTURNING MOMENT

30.03 The total shear in any horizontal plane shall be distributed to the various elements in proportion to their rigidities. If the bracing system without torsion consists of both shear walls and frames, the load cannot simply be distributed between them in proportion to their stiffness factors because their modes of deflection are not the same. A shear wall deflects predominantly in bending and shear deflection is predominant in a frame. Estimates of the lateral displacements of frame-shear-wall systems may be obtained using the charts developed by Khan & Sbarounis (1964) [31]. These curves do not include for secondary effects for axial deformation in the columns or finite member sizes and as such may be used at the preliminary design stage. A further difficulty arises where a wall is pierced by a series of openings, so that it is not clear whether it can be considered as a single unit or whether it should be considered as separate walls. Charts such as by Pearce and Matthews (1972)[32] may be used to distribute the relative bending moments, together with calculating the induced shear in the connecting beam.

31.00 HORIZONTAL TORSIONAL MOMENTS

Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity (fig 4). Negative torsional shears shall be neglected. Where the vertical resisting elements depend on a rigid diaphram action for shear distribution at any level, such as a concrete floor slab, the shear- resisting elements shall be capable of resisting an accidental torsional moment assumed to be equivalent to the storey shear acting with an eccentricity of not less than 5% of the maximum building dimension at that level.

This accidental torsion in addition to the normal torsion is due to uncertain live load distribution, inelastic behaviour of bracing elements, such as cracking of walls, subsequent alterations that may be done, such as the addition of walls, which not only change the dead load but may change the position of the center of rigidity.



Calculated Torsion M_T = We (distributed into 3 walls according to angular rotation & displacement)

M_T = We (distributed into the orthogonal walls by couple action)

FIG 4- ACCOUNTING FOR TORSIONAL DIAPHRAGM EFFECTS

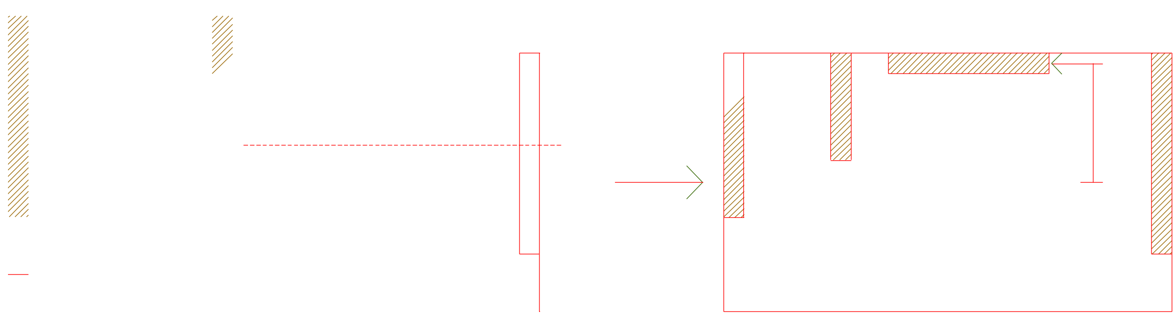
32.00 COMBINATION OF FORCES

32.01 U = 1.4D + 1.6L

U = 0.9D ± 1.4W *

* for infill walls where removal of the wall does not impair stability factor taken at 1.2

U = 1.2D + 1.2 L + 1.2W



$U = 0.9D \pm 1.4E$*
$U = 1.2D + 1.2L + 1.2E$*
* The philosophy of earthquake design in most codes is to resist moderate earthquakes without structural damage, but with some non-structural damage.
$U = 1.4A + 0.9P + 1.2H + 0.9D$
Where U is the ultimate strength, D is the effect of the dead load, L is the effect of the live load, W is the effect of the wind load, E is the effect of the earthquake load, A is the active earth pressure, P is the passive earth pressure and H is the assumed water head pressure.

32.02 Complete certainty is statistically impossible and a probability of building collapsing is postulated low enough to be acceptable, with a probability of 10^{-4} assumed, i.e. a chance of 1 in 10,000 on which the present code Load Factors are based. This over an assumed design life of 50 years may be compared with the number of people killed in traffic accidents with a probability of 130.10^{-4} in 50 years, i.e. 130 times safer. The tolerable limit in a nuclear plant is given at 10^{-5}, i.e. 1 in 100,000. If the probability of failure is to be lowered to 10^{-6} for a ductile material the present load factor adopted at a probability of 10^{-4} would have to be increased by 15%, whilst for a brittle material for same lower probability, the load factor would have to be increased by 125%. Gero & Cowen (1976)[33]

CHAPTER 5 – DEFINING BUILDING CATEGORY

33.00 BUILDING CATEGORIES (Regulation 2.00)

33.01 Regulation 2.01 defines a simple building where verification of the Structural Integrity may be complied without the preparation of structural calculations, but by complying with the recommendations given in the Technical Guidance.

33.02 The relevant sections to follow are Regulation 6.01 dealing with foundations, Regulation 6.02 dealing with masonry where amongst other matters the minimum crushing strength for load-bearing masonry walls of minimum thickness 180mm is given at $15N/mm^2$, for load bearing concrete hollow blockwork of minimum thickness 225mm at $7N/mm^2$. Guidance is then given on mortar mixes to be adopted together with further masonry detailing outlining maximum height thickness ratio and limitation on wall outstands. Regulation 7 gives guidance on reinforced concrete, with table B.03 giving reinforcement details for slab types, table B.04 gives staircase details, whilst table B.05 gives lintol reinforcement details.

33.03 To be noted that in table B.03, two respective spans have been specified, the 1st specified as structurally safe, with the 2nd considering the effect of deflection. The structurally safe span ignores deflection consideration due to the cellular masonry construction with rooms spaces limited to $60m^2$ when enclosed on all sides and to $30m^2$ when enclosed on 3 sides, as per Regulation 6.02.1a. Furthermore, considering the provision of transverse reinforcement as per Regulation 7.01.5, it is considered that the slab is supported by the 4 walls and due to this distribution, deflection is not a limiting factor, when the length to breath ratio of the respective sides does not exceed 1.75. Specification of concrete to be adopted given in Regulation 7.01.2.

33.04 Regulation 7.02.1 gives the minimum bearing onto a double leaf wall of 75mm on the inner leaf for spans up to 4.00m, whilst for spans between 4.00m to 6.00m this is increased to 100mm (Regulation 7.02.3). The external leaf has to be properly bonded to the internal one, with Regulation 6.02.4l&m, giving guidance on this.

34.00 SEISMIC CONSIDERATIONS OF SIMPLE BUILDINGS

34.01 Although the following are not mandatory they should be considered good practice, considering that plain masonry construction is disadvantageous against earthquakes, for while it has great weight and large compressive strength, its strength for tension, bending, shear is less, whilst if the work is also poorly executed, joints connecting each unit become structurally weak points.

34.02 Bearing walls should be proportionately arranged in the plan. If the distribution of walls is one-sided, divergence of the location of center of mass of the building from that of rigidity of the walls become large and the building as a whole is twisted at the time of the earthquake with dangerous stresses occurring (see fig 4). At the corners and intermediate positions of importance, bearing walls should be arranged at right angles so that the plans developed have L, T or cross-shapes (see Fig 5). Balance in elevation is also important especially where a large opening occurs at the lower storey, where a stiff beam is to be provided, with parts of the walls placed unsymmetrically not considered load bearing, for the wall at the upper floor not to be tilted during an earthquake.

EXAMPLES OF OVERCOMING UNSYMMETRICAL REQUIREMENTS WHEN LARGE OPENING REQUIRED ON 1 SIDE

FIG 5

34.03 In masonry construction, the smaller the internal divisions the stronger it becomes. In an ordinary design it is taken at $60m^2$, as per Regulation 6.02.1a. The thickness of the load bearing wall is to be taken as not less than 1/15 of the storey height, as opposed to $1/20^{th}$ in Regulation 6.02.4b. This means that for a 3.0m storey height the minimum thickness of load bearing wall is to be taken at 20cm. Further if a load bearing wall is made extremely long, it becomes dangerous against bending and twisting. The distance between the adjoining cross walls is to be 50 times the thickness of the wall, as per fig 5. Thus for a 20cm thick load bearing wall this distance is to be not greater than 10m. Earthquake resistance is larger where longer walls are placed in key locations than where there are many shorter walls. In short walls, effect of bending is large, thus horizontal cracks easily develop, causing deformation of the wall, and the resistance of the wall against shear becomes unreliable. The 6 recommendations given in section 28.05 on Earthquake Data, relating to robustness of masonry construction may be further used as a guide.

35.00 SEISMIC CALCULATIONS FOR SIMPLE BUILDINGS

35.01 A rough calculation may be carried out by the Wall Rate method. In wall construction, the respective values of bearing wall length of each storey in widthwise and lengthwise direction divided by the floor area of the storey is called the wall rate. That is to say the wall rate is the length of wall in a certain direction per unit floor area.

The required wall rate L_o is expressed by

$L_o = 1.4.0.9.V.\alpha\, \gamma_m /(A.f_v.t)$

Where V is the storey force calculated as per section 29.02 on Earthquake Forces.

⇒ 1.4 & 0.9 are load combination factors taken from para 32.01

⇒ α is the concentration coefficient of shearing stress, taken as 1 when there is no unbalance in the arrangement of the walls, but ordinarily takes the value of 1.5 – 2.0.

⇒ A is the storey floor area.

⇒ f_v is the characteristic shearing stress of wall given in section 19.02, Characteristic Shear Strength of Masonry & γ_m is shear strength factor of safety

⇒ t is the thickness of the wall.

Wall rates of 20cm/m^2 have been quoted as performing satisfactorily. Wall area ratios of 2% per floor, as per para 29.01, should also be adhered to although table 26 Moroni & al (2000)[34] refines the walls ratios necessary depending on the level of damage and number of storeys.

Table 26 - Relation Between the Level of Damages and the Wall Density Per unit Floor.

Level of Damage	Damage Category (as per table 27)	Wall Density d/N(%)
Light	0-1	≥ 1.15
Moderate	2	0.85 - 1.15
Severe	3	0.5 – 0.85
Heavy	4 – 5	≤ 0.5

Where wall density d defined as the ratio between the total shear wall area in one direction and the floor area. N is the number of floors

Table 27- Damage Categories

Category	Damage Extension	Action
0 No damage	No damage – hairline crack widths 0.1mm	No action is needed
1 Light non-structural damage	Fine cracks on plaster, falling of plaster on limited zones. Typical crack widths up to 1mm	It is not necessary to evacuate the building. Only architectural repairs are needed internally.
2 Moderate structural damage	Small cracks on masonry walls, falling of plaster block in extended zones. Damage is non-structural members, such as chimneys, tanks, pediment, cornice. The structure resistance capacity has not been reduced noticeablement. Generalized failures in non-structural elements. Typical crack widths up to 5mm	It is not necessary to evacuate the building. Only architectural repairs are needed in order to ensure conservation, such as external re-pointing to ensure weather tightness and easing/adjusting of sticky doors and windows.
3 Severe structural damage	Large and deep cracks in masonry wall, widely spread cracking in reinforced concrete walls, columns and buttress. Inclination or falling of chimneys, tanks, stair platforms. The structure resistance capacity is partially reduced. Typical crack widths exceed 15mm.	The building must be evacuated and shored. It can be re-occupied after retrofitting. Before architectural treatment is undertaken, structural restoration is needed. Service pipes fractures and some loss of bearing in beams. Apertures distorted.
4 Heavy structural damage	Wall pieces fall down, interior and exterior walls break and lean out of plumb. Failure in elements that join buildings portions. Approximately 40% of essential structural elements fail. The building is in a dangerous condition. Typical crack widths exceed 25mm.	The building must be evacuated and shored. It must be demolished or major retrofitting work is needed before being re-occupied. Beams lose bearing
5 Collapse	Collapse of part or complete building	Clear the site and rebuild.

35.02 As plain masonry is not adequate for seismic forces, it would be prudent to adopt the stability clauses in the masonry codes providing the various tying requirements required. The vertical ties would be provided in the re-entrant T or L shaped infilled concrete blockwork piers provided as per Section 34.02, Fig 5. For lintols over a 1.0m in span filling the supporting jambs in concrete is also advisable. To be noted that this type of construction adopted in Chile known as "confined masonry", was observed to have taken the severe shaking of the 1985 earthquake in a satisfactory manner (Villablanca Frolov 1988)[30] and on which tables 26 & 27 are based for MM≥7. To be further noted that buildings in the greater damage category had a weak mortar and lack of reinforcement. It is to be noted however, that Chilean engineered masonry buildings designed by comparatively primitive codes, low-strength strength masonry, small reinforcement ratios and little or no special detailing for ductility in an apparent contradiction have generally behaved well in strong earthquakes. A word of caution given by Villablanca Frolov (1988)[30], when applied to other countries, the high wall area ratios alluded to previously are to be taken note of.

CHAPTER 6 – DESIGN EXAMPLES

The following chapter contains two worked examples which attempt to cover as much as possible of the design aspects of this Handbook.

The examples are cross referenced in the right-hand margin to the relevant clause numbers in the various codes, together with reference to this handbook denoted by H to the Regulations denoted by R.

The 1st example incorporates a simple design for a 4-storey residential building in load bearing masonry, complying with the recommendations of the Structural Integrity Document. A simple arch analysis, followed by a built-up well construction subjected to lateral soil pressures is also analysed, together with a rule of thumb for a piered garden wall. Basic seismic recommendations by the wall rate and density methods are also given for this design, followed by the Stability Clause of BS 8110 intended for buildings over 4 storeys. However, these tying requirements should achieve "confined masonry" buildings outlined in para. 35.01

The 2nd example is for the design of an 8-storey free-standing office building incorporating wind and seismic calculations. The equivalent static force procedure, together with accidental torsion, has been analysed distributing the horizontal forces to the various wall elements, whilst the corner columns are effectively designed for vertical loading only. The main vertical load bearing elements are taken in reinforced blockwork, changing over to reinforced concrete when the design implies the necessity. A foundation stress distribution is carried out to the main central core elements, whilst encircling basement wall in infilled blockwork has been checked for active earth and surcharge pressures.

The 3rd example outlines the procedure necessary in the calculation for the joint spacing required in long walling.

REFERENCES

1. Building Regulations 2000, Technical Guidance, Approved Document A – Site Preparation, Construction & Demolition Works

2. Health & Safety Guidelines – issue No 3, 20001, BICC

3. BS 5628: Code of Practice for use of masonry
   Part 1: 1992: Structural use of unreinforced masonry. British Standards Institution London

4. BS 8110: Pt1:1985 Structural use of Concrete, Pt 1 Code of Practice for Design & Construction- British Standards Institution London.

5. Handbook to BS5628: Structural Use of Masonry Pt 1: unreinforced masonry, 1981, Haseltine & Moore.

6. Building Regulations 2000, Technical Guidance Approved Document C – Fire.

7. Manual for the design of plain masonry in building structures – The Institution of Structural Engineers – July 1997.

8. Debattista W. (1985) Masonry Mortars (unpublished B.E.& A. (Hons) dissertation).

9. BS 1243: 1978: Specification for metal ties for cavity wall construction, British Standards Institution London.

10. Buhagiar P. (1985) structural Masonry: experimental determination of the compressive strength of a wall (unpublished B.E & A. (Hons) dissertation).

11. BS 5628. Pt2, 1985 Structural Use of reinforced & prestressed masonry British Standards Institution London.

12. Cachia J. (1985) The Mechanical & Physical Properties of Globigerina Limestone as used in local Masonry Construction (Unpublished B.E. & A. (Hons) dissertation).

13. Bonello S. (1988) “Engineering Properties of Rock & Soils of the Maltese Islands”, (unpublished B.E. & A. (Hons) dissertation).

14. Camilleri D.H. (1988) “Structural Franka in Design Offices” The Architect March 1988, No 9.

15. Grech C. (1997) Quanlity Classification of Hollow Concrete Blockwork (unpublished B.E. & A. (Hons) dissertation).

16. Morton J. (1991) The Design of Masonry compression Members of irregular Planform, The Structural Engineer Vol 69, No 4.

17. Saliba J (1990) "The shear strength of globigerina limestone" (unpublished B.E. & A. (Hons) dissertation).

18. Saliba S (1992) "The shear strength of the masonry joint" (unpublished B.E. & A. (Hons) dissertation).

19. Golding J.M (1991) Practical Design of laterally loaded masonry panels, The Structural Engineer Vol 69, No4.

20. BS 8004: 1986 Code of Practice for Foundations, Brisih Standards Institution London.

21. CP3 Ch.V. Pt. 1972 Basic Data for the Design of Buildings – Wind Loads, British Standards Institution London.

22. Abdelnaby & El-Hewerty (2001) "Calculation of Design Wind Speed in Malta" International Conference "Safety, Risk & Reliability, Trends in Engineering, Malta

23. BS 6399: Pt 2. 1997 Loading for Buildings, Wind Loading, British Standards Institution London

24. UBC – 85, 1985 edition, Uniform Building Code, issued by the Internal Conference of Building Officials.

25. Camilleri D.H. (2001) "Malta's Risk Minimisation to Earthquake, Volcanic & Tsumani Damage" Internationl Conference "Safety, Risk & Reliability, Trends in Engineering" Malta.

26. Swiss Re Zurich (1992) "Earthquake & Volcanic Eruptions: A Handbook on Risk Assessment" in-house publication.

27. Camilleri D.H. (1999) "Volnerability of Buildings in Malta to earthquake, volcano & tsunami hazard" The Structural Engineer Nov 99, Vol 77, No 22.

28. UBC – 88, 1988 edition, Uniform Building Code, issued by the International Conference of Building Officials.

29. Camilleri D.H. (2000) Local Precast Construction under Earthquake Loading, BICC Conference on "Precast Technology, what you need to Know".

30. Villablanca Frolor R (1998) Performance of Engineered Masonry in the Chilean Earthquake of March 3, 1985: Lessons for US Design Practice UMI Dissertation Services, Michigan.

31. Khan F.R. & Sbarounis J.A. (1964) "Interaction of shear walls & frames" Proc Am Soc Civil Engineers, 90 ST3

32. Pearce D.J. & Matthews D.D. (1972) “An appraisal of the design of shear walls in box frame structures” Property Services Agency, London.

33. Gero J.S. & Cowen H.S. (1976) “Design of Building Frames”, Applied Science Publisher Ltd, London

34. Moroni M, Astroza M & Caballew R (2000) “Wall Density & Seismic Performance of Confined Masonry Buildings”, the Masonry Society Journal, July 2000.