EC8 – implications for masonry

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This technical note follows a keynote presentation "Outlining the Seismic Vulnerability of Malta's Buildings - is it an Issue" to a seismic RELEMR workshop held in Malta last year, followed by a Kamra tal-Periti CPD course on "The use of local sustainable masonry as a structural material", presenting the terraced house seismic analysis case study.

Terraced housing two or three storeys high was traditionally considered robust and stable, but the needs of the motor-car have introduced a soft open storey at ground or basement level. An economical structural system to span this 6 to 7m soft storey is by utilising hollow prestressed slabs, with thicknesses varying from 280mm to 450mm, supported on 230mm thick masonry laid in grade IV mortar. Table 1 outlines the characteristic compressive stress for 230mm thick masonry units with a height of 265mm, as outlined in BS5628PtI (1).

MALTA'S SEISMIC ZONING

EC 8 (2) specifies that a design ground acceleration for a return period of 475 years has to be specified in the National Annex. The 475 return period is based on the proviso that this ground motion is not to be exceeded in the assumed 50 years' design life of the structure in 90% of the cases.

With reference to Fig. 1, return periods may be identified for earthquakes of intensity MMV and MMVI, whilst an MMVII was noted to have occurred in 1693, when a strong MMXI had hit the Eastern side of Sicily. It is noted that an MMVII in Malta requires an MMXI in Sicily with a return period of 1,000 years.

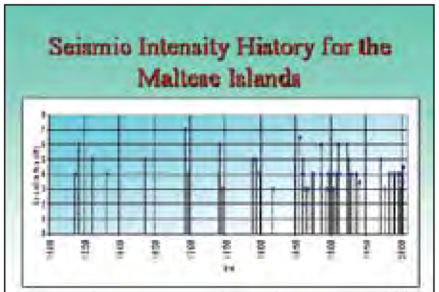

Figure 1 *Source - Pauline Galea, Seismologist, 2006*

Although a seismic risk hazard has not as yet been undertaken for the Maltese Islands, considering above data, Table 2 proposes return periods for expected seismic activity in Malta for various earthquake intensities.

From Table 2 and plotting a log-log graph, the 475 return period works out at 0.06g. This figure is also confirmed by the GSHAP – (Global Seismic Hazard Assessment project) map, Fig. 2, for Europe, with Malta identified as a green colour corresponding to 0.05g -0.06g, although the data on which this was compiled for Malta was probably very sparse.

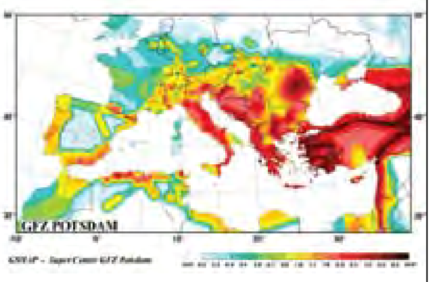

Figure 2. Peak horizontal acceleration map with a 10% probability of exceedance in 50 years

According to EC8, with a design ground acceleration of 0.06g, Malta is classified as a low seismicity zone, as falling within <0.10g but >0.04g, with the following provisions to be catered for.

MASONRY EC8 DESIGN CRITERIA FOR ZONES OF LOW SEISMICITY

1. Shear walls in manufactured stone units are to have thickness t >175mm. This fortunately is the thickness for internal partitioning adopted at 180mm. Further, $\mathbf{h_{eff}/t < 15}$ and $\mathbf{h/l < 2.5}$, where **t** is the thickness of the wall, $\mathbf{h_{eff}}$ the effective height of the wall, **h** the greater clear height of the openings adjacent to the wall, **l** the length of the wall.
2. For a design ground acceleration <0.2g, the allowed number of storeys above ground is three for unreinforced masonry and five for reinforced masonry, however for low seismicity a greater number of storeys are allowed.
3. Mortar type to be adopted should be at least Grade III, although lower resistance may even be allowed, whilst for reinforced masonry grade IV may be used. Further there is no need to fill the perpendicular joints.
4. Floor diaphragms may be considered rigid, if they consist of reinforced concrete. The connection between the floors and walls shall be adequately provided by steel ties at every floor level, spaced at not more than 4m centres.

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

Table 1: Characteristic Compressive stress f_k of 225mm thick masonry N/mm^2 for specified crushing strength – as per BS 5628 pt 1

MM - Earthquake Intensity	Return Period (years)	Base Shear Design % of g
VI	125	2-5
VII	1,000	5-10
VIII	10,000	10-20

Table 2: Malta's Seismic Return Period

LOAD PATH ANALYSIS FOR A 4-STOREY MASONRY BUILDING

The floor plans indicated in Fig. 3, show an open garage plan constructed in 1995, when the allowable storey height stood at two floors. The prestressed planks inserted at first floor level were 230mm thick to support the overlying single floor. In 1998 the allowable height was increased to three floors. Thus to support the additional floor, hollow block concrete slabs were inserted at 2nd floor level spanning over the 6.25m opening from party wall to party wall in 230mm thick masonry, as indicated in Fig. 4. In 2005 an additional receded floor was further allowed.

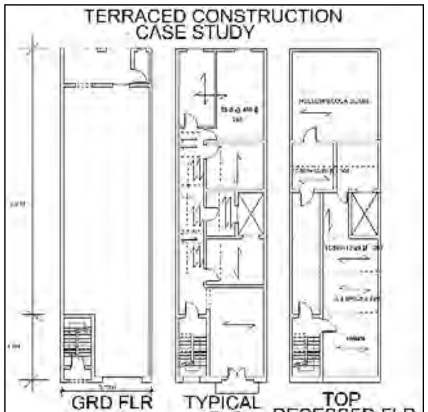

Figure 3

Fig. 4 gives an indication of the load paths adopted. Arching was provided for at 1st floor level, up to the underside of the hollow block concrete slabs, while at the upper levels the loads went directly as reactions onto the party walls. The reaction from the prestressed planks was eccentric (3) onto the party wall, whilst the loading coming from the remaining upper floors is centrally located onto the party wall.

For the load analysis shown in Fig. 4, the total characteristic loading from the three

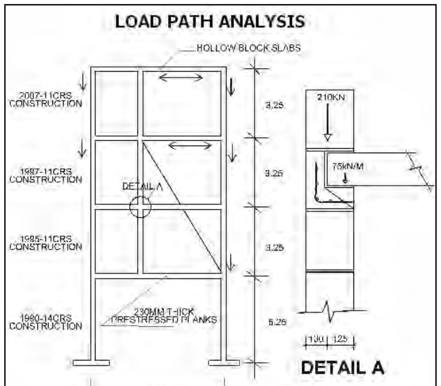

Figure 4

construction in Malta

habitable floors and roof construction totalling 285KN/m is split as 210 KN/m as direct loading onto the party wall, with the remaining 75KN/m being the eccentric reaction from the 6.25m spanning precast prestressed slab, with 125mm bearing.
At the prestressed planks seating level, the resultant eccentricity is calculated thus:
e = 75 (125/3 – 112.5)/(210 + 75) = 18.6mm
e/t = 18.6/225 = 0.08
l_{eff}/t = (4000*1.0)/225 = 17.8 (dpm at ground level not considered stiff enough for this soft storey layout)
β = 0.72, from table 7 in BS5628 PtI giving capacity reduction factors β, only up to a value of 0.3t.
f= 285KN/m/0.225m = 1.27N/mm^2
From Table 1 for a masonry unit of compressive strength 20N/mm^2
f_k = 7.4N/mm^2*β/γm =7.4 * 0.72/3 = 1.78N/mm^2 > 1.27N/mm^2
Thus under vertical loading, with resulting eccentricities the structure is stable.

WIND STABILITY CALCULATIONS:

With a basic wind speed of 47m/s, a ground roughness of 3 as no building dimension exceeds 50m, thus classified as a Class B structure, according to CP3 ChV (4)
S_3 = 0.83, thus $V_s = VS_1S_2S_3$ = 47*1*1*0.83 = 39m/s
Wind pressure q = 0.93KN/m^2
Force coefficient C_f for buildings with flat roofs (Table 10 CP3 ChV):
l/w = 24/6 = b/d = 24/6 = 4
height/breath = 15/24 =0.625
Thus **C_f** = 1.3
F = $C_f q\ A_e$ = 1.3*0.93*15 = 18.15KN/m
Considering the load combination 1.2γ_D +1.2γ_L +1.2γ_w as being the more onerous
BM = 18.15/2 * 4m/2 * 1.2 = 21.8KN-m/m
P_w = 18.15* 7.5m/6.25m = 22.8KN/m
ΣN(75 + 210) * 1.2/1.45 + 22.8 *1.2 = 263KN/m, where the vertical loading has been factored down to 1.2, instead of the previous average load factor of 1.45.
e = 21.8KN-m/m / 263KN/m + 0.0186m = 0.101m
e/t = 101/225 = 0.45 (Table 7 in BS5628 PtI gives capacity reduction factors β, only up to a value of 0.3t.)
l_{eff}/t = (4000*1.2) / 225 = 21.33 (under wind load, masonry structure considered to sway sideways.)
From Table 1 for a masonry unit of compressive strength 20N/mm^2
f_k = 7.4N/mm^2 with γm =3.0
Width of stress block x to which direct load subjected to:
x = 263KN/m * 3 /7.4N/mm^2 = 107mm
Stability moment for wall section = 263KN/m * (225 – 107)/2 = 15.52KN-m/m.
This stability moment is less that the moment induced under the wind load condition at 21.8KN-m
However, such constructions are known to be stable under such wind load conditions, thus besides the sway action analysed, diaphragm action comes into play whereby the end walls take the outstanding % of wind action via torsional redistribution, with the centre of gravity of the wind loading in plan not coinciding with the shear centre of the walling at ground floor level.

SEISMIC STABILITY CALCULATIONS:

Period of vibration for this 4-storey building given as per Annex C2 EC8:
T = $C_t H3/4$ = 1.5* 153/4 =0.38s
As 0.1s < T < 0.4s
$S_{d(t)}$ = αSβo/q
where α, the ratio of the design ground acceleration to the acceleration of gravity, is taken as 0.06 for Malta, as referred to above.
S and βo are dependent on the soil conditions, for type A sub-soil, which refers to rock founding material, as from Table 4.1 (EC8), given at 1 and 2.5 respectively.
Q is a behaviour factor, which for unreinforced masonry as per Table 5.1 (EC8) is given at 1.5.
$S_{d(t)}$ = 0.06 * 1.0 * 2.5 / 1.5 = 0.1, corresponding to 10% of vertical load.
The EC8 seismic load combination is $\Sigma G_{kj} + \Sigma \psi_{Ei}.Q_{ki}$
Where ψ_{Ei}, is the combination coefficient for variable action given by: ψ_{Ei}, = ð * ψ_{2i}
With combination coefficient taking into account the likelihood of the loads being not present over the entire structure during the occurrence of the earthquake.
Value of ψ_{2i} is given in Pt1 of EC1 at 0.2 for domestic loading and the value of ð given at 0.5 from Table 3.2 of EC8. Thus only 0.1 of the total live load is to be catered for the seismic condition.
The total dead load for the upper floors is given at 4KN/m^2 (self weight) + 2KN/m^2 (finish) + 4KN/m^2 (masonry partitions) = 10KN/m^2
The total load transferred onto the two supporting party walls at just below the 1st floor level is given by:
[{10KN/m^2 * 3floors + 6KN/m^2 * 1 roof slab + 0.1* 1.5KN/m^2 (LL) * 4 floors} * 6.25m + 55crs * 1.35KN/m/crs * 2 in No. = 377KN/m.
The seismic horizontal force is thus given at 10% of the total vertical seismic load combination 0.1 * 377KN/m = 37.75KN/m on each supporting party wall, as opposed to 18.15KN/m for the wind load condition.
BM = 37.75/2 * 4m/2 = 37.75KN-m/m
P_s = 37.75* 10m/6.25m = 60.4KN/m
ΣN 377/2 + 60.4 = 248.9KN/m
e = 37.75KN-m/m / 248.9KN/m + 0.0186m = 0.17m
e/t = 170 / 225 = 0.76 (Table 7 in BS5628 PtI gives capacity reduction factors β, only up to a value of 0.3t)
l_{eff}/t = (4000*1.2) / 225 = 21.33 (under seismic load, masonry structure considered to sway sideways.)
Again from table 1 for a masonry unit of compressive strength 20N/mm^2
f_k = 7.4N/mm^2 with γm =1.7 from table 5.3 EC8
Width of stress block x to which direct load subjected to:
x = 248.9KN/m * 1.7 /7.4N/mm^2 = 57mm
Stability moment for wall section = 248.9KN/m * (225 – 57)/2 = 20.91KN-m/m.
This stability moment is less than the moment induced under the seismic load condition at 37.75KN-m

OBSERVATIONS

For Malta's masonry construction, the seismic horizontal force at 37.75 KN/m is just more than double the horizontal wind force at 21.8 KN/m. From above, it is noted that the seismic force is 10% of the vertical load, thus wind load equates to just under 5% of vertical load. These figures are well above what BS 5628 specifies, for the minimum lateral load given at 1.5% of the characteristic dead load above that level.
Again the seismic stability moment at 20.91 KN-m/m is 55% of the overturning seismic moment at 37.75Kn-m/m. The wind stability moment at 15.5 KN-m/m is 71% of the overturning wind moment at 21.8KN-m/m. Although the wind stability moment works out less than the overturning wind moment, the rigid diaphragm action of these constructions appears to transfer the outstanding bending moment, not catered for by frame action by couple action into the transverse supporting wall system. For this to occur the tying clauses stipulated in BS 5628 pt 1 – Table 12 are to be adhered to.

Building Type	B	C
Earthquake Intensity MM	MDR	MDR
5	2%	-
6	4%	1%
7	20%	10%
8	45%	25%

Table 3 –Mean Damage ratio (MDR) for building types founded on rock (5).

However the stability, under the 475 year seismic return period, with the stability moment equating only to 55% of the overturning moment is under question. Table 3 for a symmetrical building in layout gives mean damage ratios for MMVI varying from 1% up to 4% and for an MMVII from 10% up to 20%, depending on the quality of the building. For buildings of higher irregularity and asymmetry, these ratios are even known to go higher.
These mean damage ratios outlined in Table 3, compared with the above analysis seem to indicate that masonry soft buildings in Malta will suffer damage if subjected to a seismic tremor as indicated by the 475 year return period as specified in EC8.

References:

BS5628: Code of Practice for use of masonry, PtI: 1992: Structural use of unreinforced masonry. British Standards Institution London

Eurocode 8: DD ENV 1998-1-1:1996, Design provisions for earthquake resistance of structures, Part 1.1 General rules – Seismic actions and general requirements for structures. CEN.

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