DATE: 01/11/23

2023_UPDATE TO MALTA'S SEISMIC RISKS – technical note

This technical note should provide guidance to practicing structural engineers in Malta on how to guide their clients on what should be acceptable to counteract the seismic risks of the Maltese Islands. Government funded projects & those acquiring European funding are to abide by the Structural Eurocodes, in particular Eurocode 8 and Malta's National Annex.

On the other hand private projects/developments are not required presently to abide by above requirements. This note should guide the local practitioner following upon his/her research on which construction methodology should be acceptable to undertake.

INTODUCTION

A historical catalogue of felt earthquakes on the Maltese islands has been compiled dating back to 1530. Although no fatalities were officially recorded during this time as a direct consequence of earthquake effects, damage to buildings occurred several times.

However, it is to be stressed that historical knowledge of damaging earthquakes is sorely deficient. Not only is the information after 1530 incomplete, but damaging earthquakes before this date – though absent from the record – may be inferred to have affected the Islands. For example, the 1169 Sicilian earthquake had the same source, intensity at the source (I = XI) and probably a similar epicentral location as that of the 1693 earthquake, yet no records on of its impact on the Islands exist.

Out of the 100 events catalogued, 6 in number were damaging.The Islands lie in the middle of an extensive fault system affecting the central Mediterranean from Tunisia to Sicily. The faults, along which earth-quakes occur, are continuous through Malta and Gozo. Some of the faults are extinct, but others are young and still active. Malta, situated in the Sicily Channel, forms part of the relatively stable northernmost platform of the African continent.

Observations of recent earthquake activity around the Islands indicate, at first sight, a low level of activity, with small-magnitude events (<3.5) occurring at low rates. In the catalogue time period, the islands experienced EMS-98 intensity VII-VIII once (11 January 1693) and intensity VII, or VI-VII five times[1]. The occurrence of a magnitude 4.5 event close to Malta in 1972, together with the reported 1693 earthquake, shows the importance on outlining Malta's current Seismic Zoning. This may offer advice on the advantages for a particular building being made earthquake resistant or the advantages of retrofitting an existing building.

[1] Galea P., Seismic history of the Maltese islands and considerations on seismic risk, ANNALS OF GEOPHYSICS, VOL. 50, N. 6, December 2007

It is however to be noted that it is the large distant earthquakes, especially from eastern Sicily & the Aegean sea, with others only located within the Sicily Channel to the north of the Islands that have caused the most significant damage. It is further noted that it is not those generated either in the Sicily Channel or south of the Islands.

Eurocode 8[2] specifies that design ground acceleration on firm ground for a return period of 475 years has to be specified in the National Annex. The 475 year 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.

EARTHQUAKE FORCES

The energy dissipated by earthquakes is expressed in horizontal and vertical acceleration forces acting on the skyscrapers. The immense forces transmitted from underground must be absorbed by the supporting structures of the buildings. These dynamic loads are replaced by structural equivalent loads in horizontal and vertical direction when a structural analysis of the building is performed. The highest acceleration forces measured to date in an earthquake, were recorded during the Northridge earthquake in Los Angeles (17th January 1994) and amount to 2.3 times the acceleration due to gravity "g" ($g = 9.81m/s^2$) in horizontal direction and 1.7 times the acceleration due to gravity in vertical direction. In simplified terms, this means that the planning engineers would additionally have to apply roughly 2.3 times the dead weight in horizontal direction and roughly 1.7 times the dead weight in vertical direction to the building when dimensioning the supporting structure so that these earthquake forces can safely be absorbed. Fortunately as noted further down these seismic forces are noted to be much less for the Maltese Islands.

In simplified form, earthquake loads can be represented by horizontal and vertical equivalent loads acting on the mass centre of gravity of the building. The magnitude of these equivalent loads depends directly on the mass of the building. This leads to the conclusion that as the height of the building increases, the mass centre of gravity normally wanders upwards and the flexural effect on the building is intensified by the longer lever arm.

The far-field effects of large earthquakes are known to differ from those felt close to epicentres and whereas many types of buildings are effected in the latter locations, in the former, damage is more selective. During historic earthquakes, low-frequency structures (e.g., low rise complex buildings like cathedrals, large churches and palaces) suffered far greater damage from large distant earthquakes than high-frequency structures (e.g., stiff buildings such as low chapels and low rise houses with simple square or rectangular shapes). This is due to high frequency waves being more attenuated with increasing distance from a given epicenter than low frequency waves[3].

The most important and most serious effects are as outlined below, together with the possible protective measures.

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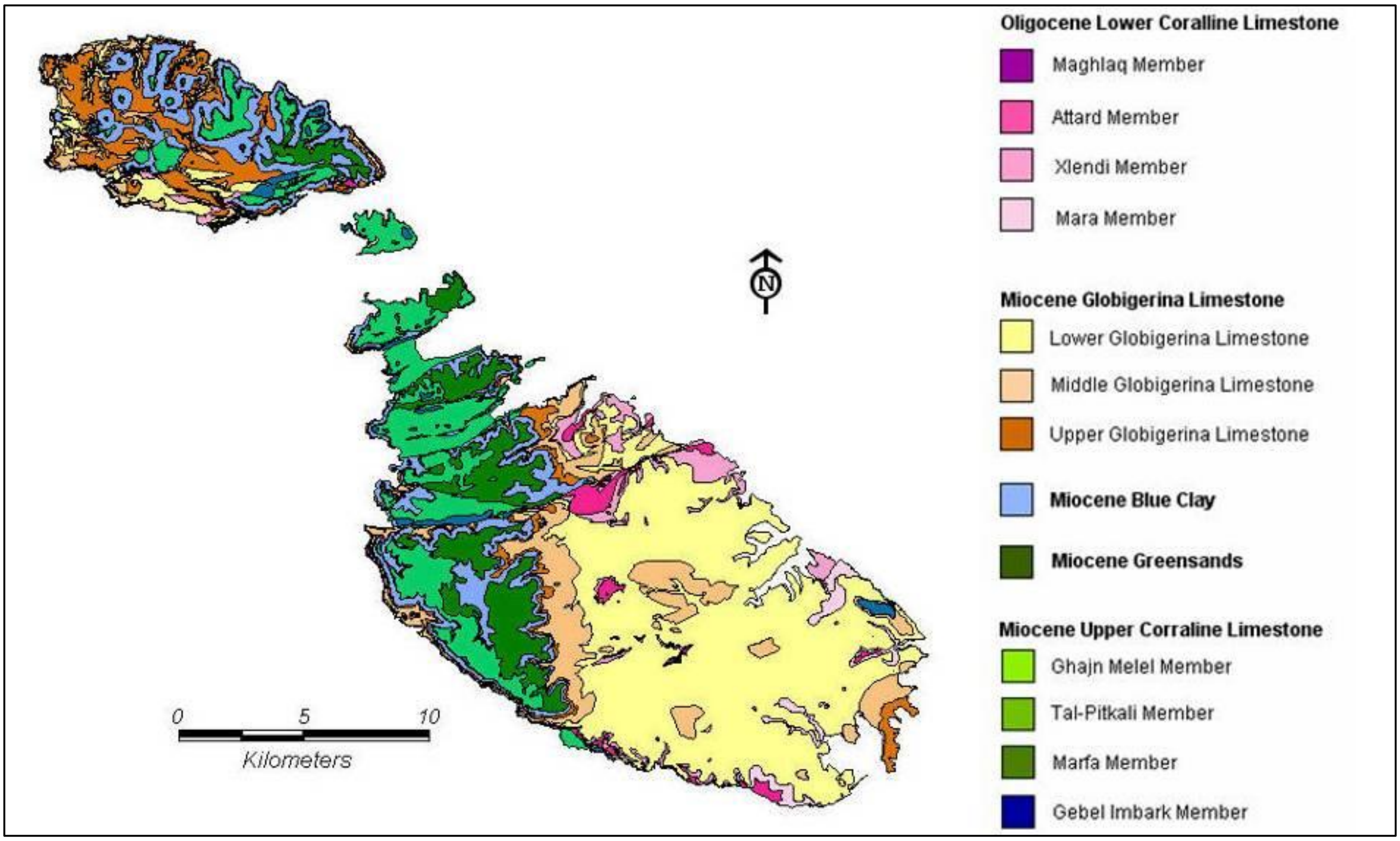
[2] Eurocode 8. Design of structures for earthquake resistance. General rules, seismic actions and rules for Buildings: MSA EN 1998-1:2004

[3] Main G. et alia, "The hazard of the Maltese Islands"' Nat Hazards, Nat Hazards, Springer 2018.

SUBSOIL

Natural rock is the best subsoil from the point of view of its earthquake properties. Sandy soils saturated with water and artificially backfilled land are considered to be particularly critical. The widely-feared liquefaction effects (plasticization of the soil) can occur if an earthquake coincides with high groundwater levels.

Figure 1: Geological Map of Malta (Source: https://goo.gl/P1c67p)



The other shades of colouring refer to different varieties of stable rock formations.

Fortunately most of Malta's buildings are founded on a stable limestone formation. Clay sites do exist as highlighted in green in figure 1, limited to some areas in northern Malta and more frequent in Gozo.

The blue clay formation is a particularly problematic formation, as seismic waves are known to have triggered historic slope failures. Another problematic situation develops with blue clay cropping out below the upper coralline limestone. This causes instability of upper coralline limestone outcrops lying above the blue clay. Some rock spreads on plateau or sloping surfaces develop large limestone block slides. These however are locations where very few of Malta's building stock is located, with future buildings discouraged.

SUPPORTING BUILDING STRUCTURE

A distinction can generally be made between rigid and elastic supporting systems. Rigid systems, such as solid masonry wall and ceiling elements, are difficult to deform and transmit the seismic loads through their rigidity. Due to the stiffness and lack of ductility in the supporting structure, however, shear cracks can develop in the building. The problem is that more and more energy must be absorbed through the high rigidity and that more and more material is required for this purpose.

Elastic supporting structures, such as reinforced concrete or steel frames, are highly deformable and absorb the applied seismic energy in this way. The nodes connecting the horizontal and vertical elements of the supporting structure are highly stressed however, and peak loads occur both here and on the reinforcing elements (bonds) which must be taken into account when producing these connections.

The most common structural system adopted in Malta relates to cellular load-bearing masonry. A compact workable *franka* limestone building stone (crushing strength approx. $20N/mm^2$) is an important natural resource. Terraced housing previously 2 or 3 stories high was considered robust and stable, but the needs of the motor-car, have introduced a soft open storey at ground or basement level. However with height relaxations ongoing over the past 18 years, these 2 – 3 storey buildings now account for 5 – 8 storey buildings in structural masonry, with even pencil buildings 10 – 11 storeys high in load bearing structural masonry having been undertaken.

The most economical structural system to span this 6 to 7m soft storey between ground & basement level is by utilizing precast hollow prestressed slabs, with thicknesses varying from 230mm up to 525mm, supported on 230mm thick masonry laid in an M2 type mortar.

The concrete buildings constructed to date generally fall short of recommendations given by earthquake Codes for detailing methods of reinforcement, mostly in the tying of column-beam joint layouts and in the positioning of stirrups in beams and columns. Special care is also needed with prefabricated buildings, as their very design is often deficient, unless tying provisions are undertaken to tie the individual structural elements.

SYMMETRY & SHAPE OF THE BUILDING

Symmetric layouts, rigidity and mass distribution lead to a considerably better seismic response than asymmetric layouts, rigidity and mass distribution. This is because asymmetric buildings are subjected to stronger torsion (twisting) around the vertical axis by horizontal seismic loads.

When parts of different height are permanently connect to one another as, for example, is often found in high-rise buildings with atriums, then the various structures in the building can be subjected to considerable torsional stresses by the seismic loads.

Resonance effects can also cause buildings to oscillate so strongly that they hammer against one another. Another effect observed in high-rise buildings is the soft-storey effect: due to lobbies, atriums or glazed shopping passages, some floors - usually near the ground floor – are distinctly "softer" than those above them. These "soft" floors then collapse in an earthquake.

CLASSIFICATION OF BUILDING TYPES FOUND IN MALTA

Rubble masonry buildings, as found in old buildings in Valletta or in the villages, being over 200 years old, are brittle. These buildings are made of stone set in earth, and such structures tend to fall apart even if shaking is moderate at MSK V.

The method depends on the 12" MSK intensity scale' which is roughly equivalent to the MM scale in actual values, varying as to degree of sophistication, including building types (Table 1), damage grades and quantities. The arrangement of the scale includes the effects on humans, natural objects, and the damage to buildings.

Table 1: Classification of Building according to anticipated Earthquake Intensity Damage

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

In Malta, a few buildings are classified as type B. These would be restricted to old, deteriorated rural dwellings exceeding 150 years in age or old, deteriorated buildings in Valletta in which, owing to little maintenance, stability has been impaired because of water ingress. Type A are limited to old, deteriorated rubble buildings, with age exceeding 200years, utilised as agricultural annexes. Most masonry buildings and most concrete frame buildings would be classified as conforming to type C. The more rigid buildings, satisfying stiffness regularity and symmetry in plan elevation layout, are classified D_l.

For buildings founded on softer material than limestone, the MDR* is taken as the progressively corresponding higher value on the scale: e.g. if a type C building, founded on clay, is subjected to MSK VI, its MDR* is to be based on an MDR of MSK VII, whilst if founded on a poorly backfilled, disused quarry, its MDR is to be based on MSK VIII.

* The mean damage ratio (MDR - Table 2) is the average damage to buildings of roughly similar vulnerability and architectural characteristics, expressed as a percentage of their new value.

Table 1 notes that the partially collapsed buildings in the 1693 event were classified as old and neglected, falling under Type A (5% PGA). On the other hand the buildings in Valletta suffering damage are classified as type B (7% PGA). At 7.5% PGA seismic activity the MM for this event falls between VII & VIII. For type D1 buildings in the 20% - 30% range damage occurs mid-MM VIII.

SEISMICITY & VULNERABILITY OF MASONRY CONSTRUCTIONS IN MALTA.

Eurocode 8 specifies that design ground acceleration on firm ground for a return period of 475 years has to be specified in the National Annex. The 475 year 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 for no collapse requirement. For damage limitation exceedance this is to be based on a 95 year return period, which signifies a 10% chance of exceedance.

It is recommended to consider as very low seismicity cases those in which the design ground acceleration on firm ground, a_g is not greater than 0,04 g (0,39 m/s^2). It is then recommended to consider as low seismicity cases those in which the design ground acceleration on firm ground, a_g, is not greater than 0,08 g (0,78 m/s^2), or those where the product $a_g.S$ is not greater than 0,1 g (0,98 m/s2). For very low seismicity the provisions of EN 1998 need not be observed. For low seismicity reduced or simplified seismic design procedures for certain types or categories of structures may be used. For all other design ground accelerations, the recommendations of Eurocode 8 are to be abided by.

The worst recorded damage was during the 1693 event, which caused 60,000 deaths in Sicily. In Valletta it is reported that there was not one house that did not need some repair. The facades of some major buildings were detached from the main structure, and needed immediate repair. Some churches suffered major damage to their domes and severe cracks in walls. Serious damage was done to the old mediaeval city of Mdina. Here the Cathedral suffered partial collapse and many other buildings suffered serious damage. It should be noted that there are several remarks in the reports that show that many of the buildings in the city were very old and had been neglected for many years. In particular, the 13th century cathedral was already showing serious signs of disrepair before the earthquake, and plans had in fact already been drafted for its rebuilding. In Gozo, it was noted that the damage to the fortified Cittadella, was most probably due to long years of neglect, as was the damage to coastal towers. Further during Mgr Pietro Duzina's apostolic visit in 1575, he went to 430 churches in Malta and Gozo. They were mostly small, bare and dirty. Forty-nine were on the verge of collapsing. Two hundred and twenty-two had no doors and animals entered regularly.

Most of the houses were extremely shattered and deserted by the inhabitants who then lived in grottos and under tents in the fields. It is also mentioned that the Grand Master was hunting presumably in the Buskett-Girgenti area and was in great danger by the falling of a mountain near him . Agius de Soldanis in his manuscript Gozo Antico & Moderno, recounts how the sea at Xlendi rolled out to about one mile and swept back a little later *"con grande impeto and mormorio"* in the earthquake of 1693[4].

Considering the above damage, rock falls in addition, together with a tsunami, it appears that intensity of the 1693 Earthquake works out at MSK VII - VIII. However, noting that no deaths have been recorded in the catalogues, it appears as from table 2, that this should tend within the MSKVII range.

Table 2: Mean Damage Ratio (MDR) & Death Rates for building types B & C

Building Type	**B**		**C**	
Earthquake Intensity MM	**MDR**	**Death Rate**	**MDR**	**Death Rate**
5	2%	-	-	-
6	4%	-	1%	-
7	20%	0.03%	10%	-
8	45%	1%	25%	0.4%

Return periods may be identified from catalogues for earthquakes of intensity MMV & MMVI, whilst an MMVII/VIII was noted to have occurred only once, as in 1693, when a strong MMXI had hit the Eastern side of Sicily. Reference is also made to the the 1169 Sicilian earthquake noted in Introduction, with the same source and intensity at the source (I = XI). It is noted that an MMVII in Malta requires an MMXI in Sicily with a return period of 1,000 years.

Figure 2: Site seismic history for the Maltese islands since 1500, showing EMS-98 I ≥IV1[1].

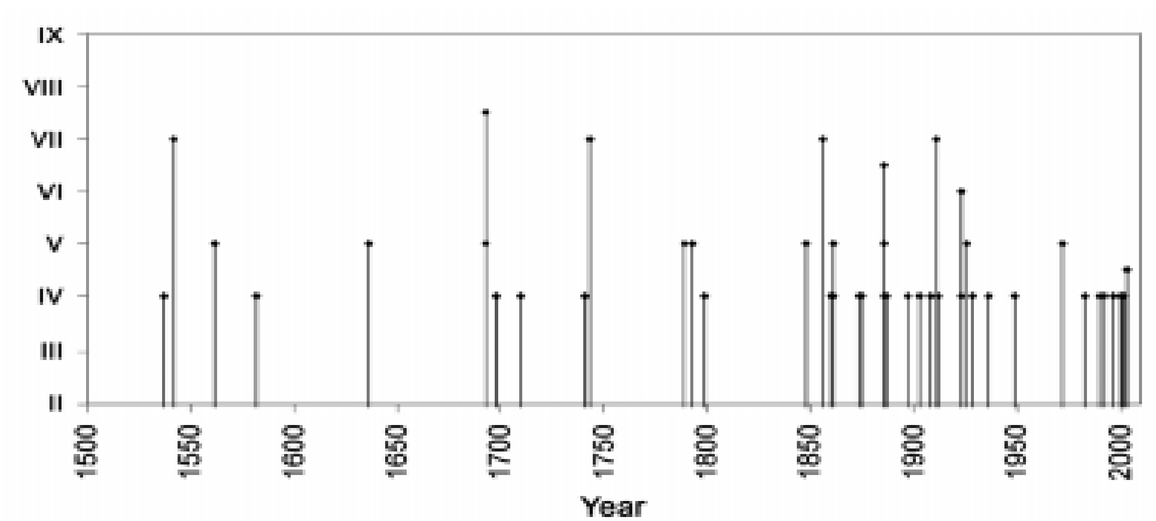
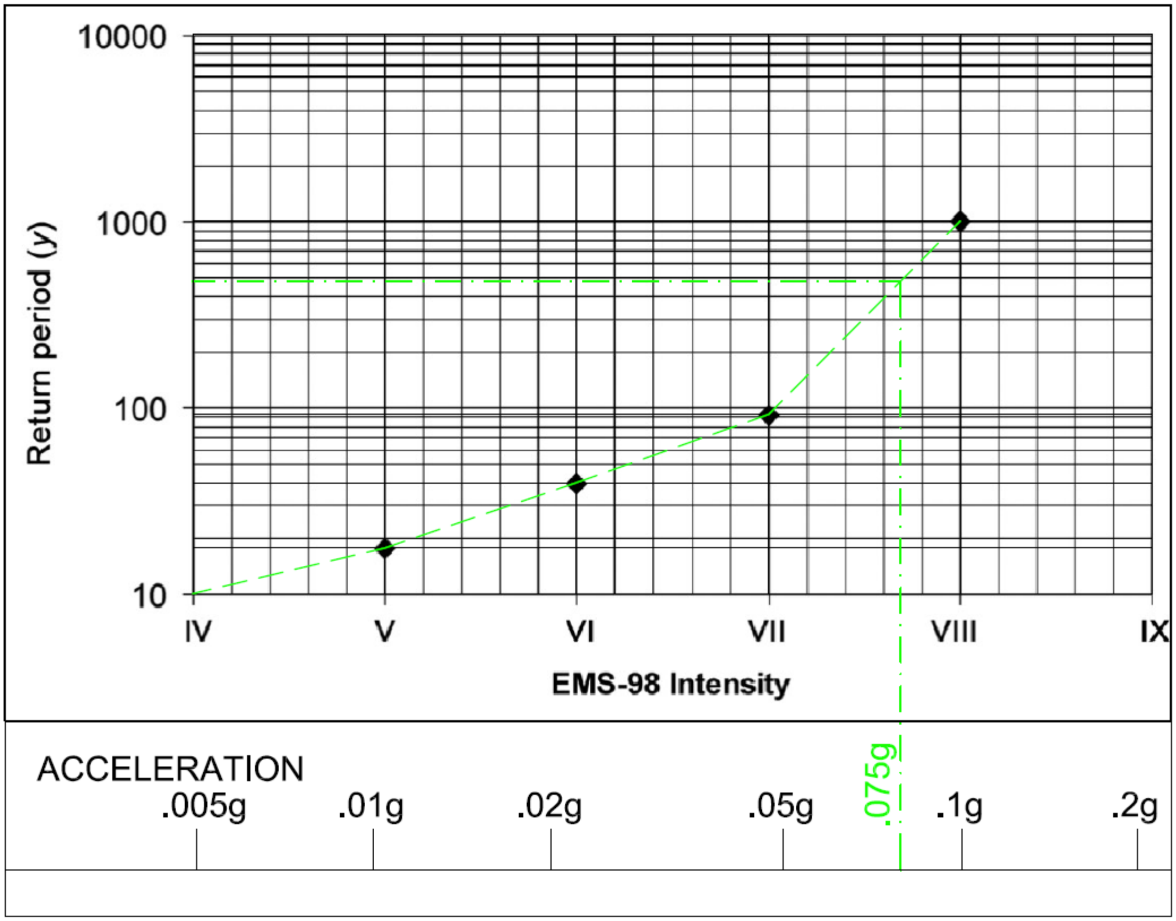


Figure 3 indicates return periods in rock or firm soil for expected seismic activity in Malta for various earthquake intensities. These return periods have taken cognizant of historical data as per figure 2.

[4] SHOWER, J. (1693): Practical reflections on the late earthquakes in Jamaica, England, Sicily, Malta, etc., London.

The Reference Peak Ground Acceleration on type A* Ground, a_g

Fig. 3 Estimated return periods, following the methodology of Magri *et al.* (1994)[i]



* Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.

From *figure 3,* as plotted on a log-log graph, the 475 return period works out at 0.075g. This is also confirmed by the GSHAP – (Global Seismic Hazard Assessment project) map no in *Figure 4* below.

Figure 4: GSHAP – (Global Seismic Hazard Assessment project) showing Malta

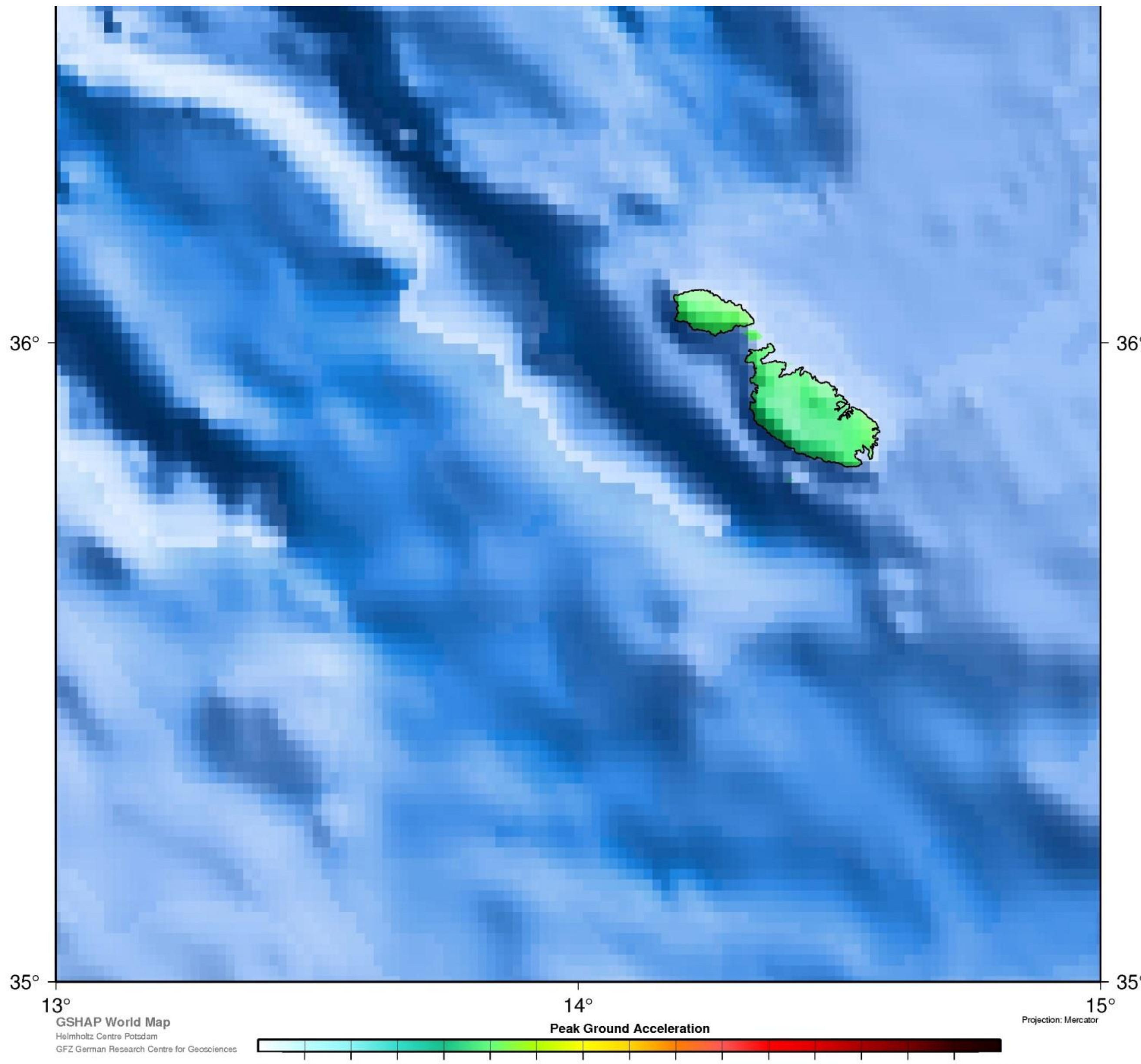


Figure 4 notes peak accelerations PGA as varying from 0.7m/s^2 down to 0.55m/s^2, with the south eastern cliff face being the more stable location. The above ground acceleration values note the Maltese Islands according to EC8 to be classified for low seismicity.

Figure 5: GSHAP – (Global Seismic Hazard Assessment project) showing the Maltese Islands, Siciliy and Parts of Africa

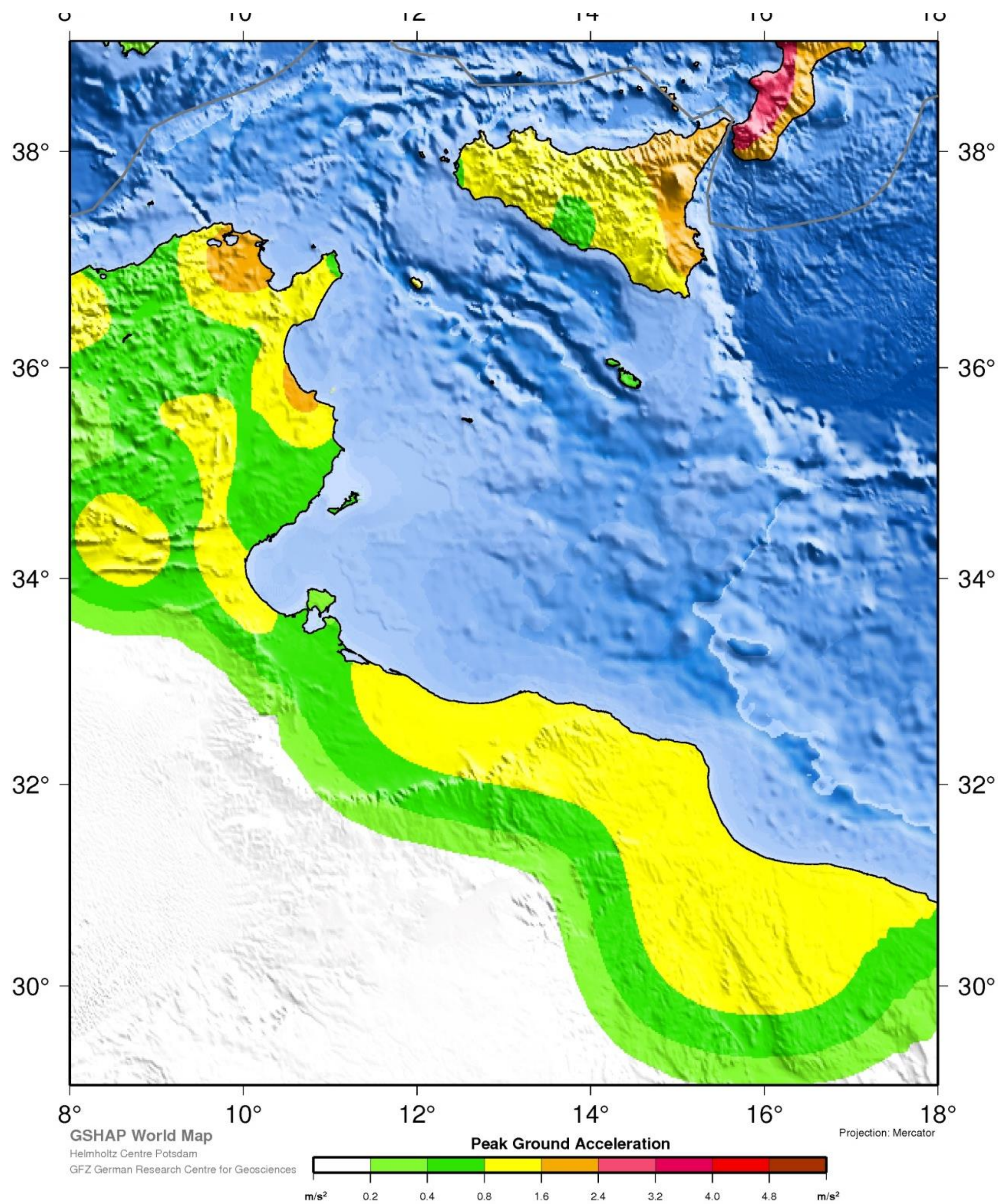


Figure 5 notes that the areas with a high seismic risk which are those noted with a brownish colour are: NE parts of Sicily (Syracusa, Catania, Messina) and parts of North Africa (Tunis, Ariana, Bizerta). The areas shaded in yellow have a lower seismic risk then the brown areas whilst the areas shown in light green have the lowest seismic risk. When compared to its surroundings, Malta has a low seismic risk as it is shown in green.

It is further to be noted that the updated seismic Eurocode 8 Annex G, as per fig A.1 below, still in draft form classifies Malta on the lower end of the Low Seismic Hazard at 1.2 m/s^2(c PGA - 0.06g).

prEN 1998-1-1:2022 (E)

Malta 1.2

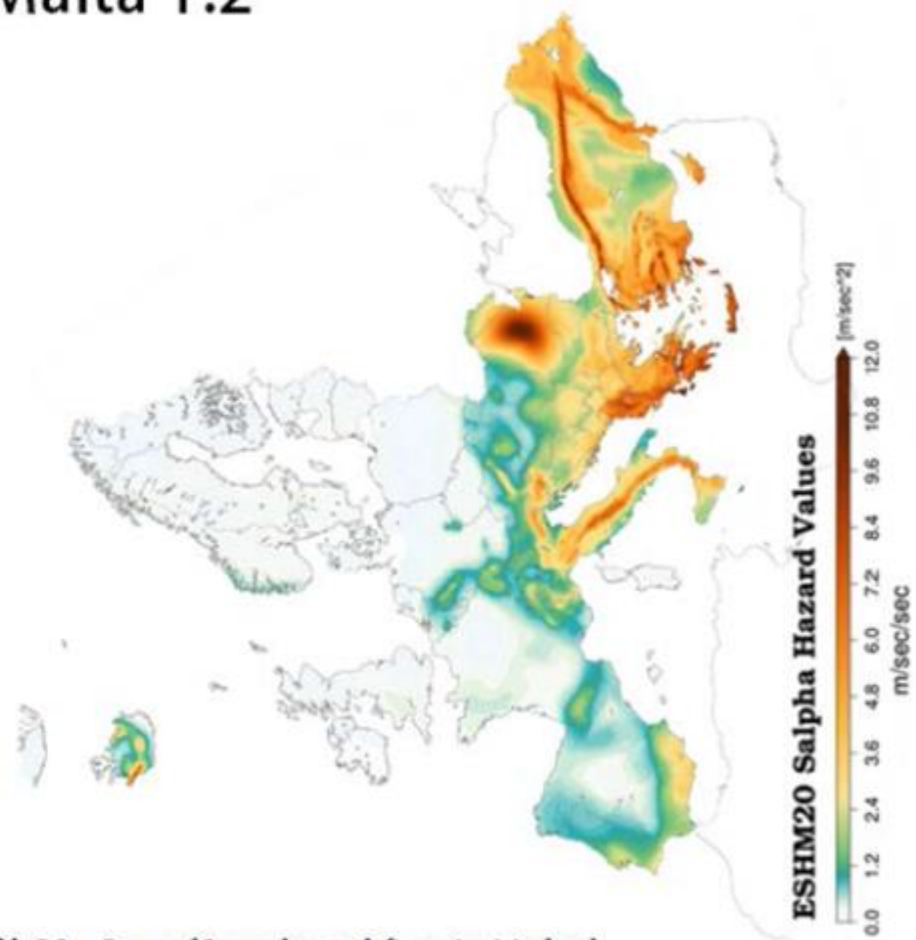


Table 5.2 — Range of $S_{\alpha,475}$ values to define seismicity levels

Seismicity level	$S_{\alpha,475}$ (m/s^2)
Very low	$S_{\alpha,475} < 1{,}0$ m/s^2
Low	$1{,}0$ m/s$^2 \leq S_{\alpha,475} < 2{,}5$ m/s^2
Moderate	$2{,}5$ m/s$^2 \leq S_{\alpha,475} < 5{,}0$ m/s^2
High	$S_{\alpha,475} \geq 5{,}0$ m/s^2

- Low Hazard: PGA < 0.1g
- Moderate Hazard: 0.1g < PGA < 0.25g
- High Hazard: PGA > 0.25g

Figure A.1 — A representation of $S_{\alpha,475}$ European map for rock sites, based on ESHM20

Further studies note that according to the SHARE map and data, Malta is listed with PGA falling between 0.05g and 0.075g. The upper range of 0.075g signifies that Malta may experience seismic activity of MSKVII+. This classifies Malta as a location with Low Seismic Hazard Risk, which agrees with above classification.

It is further noted that Malta's proximity to Sicily and its higher earthquake hazard have sparked in recent years a debate on the likely risk for the archipelago. A series of probabilistic seismic hazard assessment (PSHA) have been conducted and published in 2013[5] [6], showing results of PGA between 0.09-0.18 g for 10% probability in 50 years (475 return period), changing Malta's classification risk from "Low" to "Moderate".

Following a buildup of an instrumental earthquake offshore catalogue 1995 - 2014[7], this catalogue notes:

i) that the highest concentration of seismic activity is located 80–120 km SSE of Malta. The southeastward extensions of the Melita and Linosa grabens are seismically active, showing that extension in the Sicily Channel is ongoing. In January 2023 a seismic swarm of circa 100 events occurred 100km south-west of Malta facing Libya at a depth of 10km below the seabed, in a zone known as Melita Graben. Events noted order of magnitude varying between 5.2 (MMVI) down to 3.6 (MMIII), with a number measuring 4.3 (MMV).

ii) 40 km to the east of Malta there is another cluster of epicentres, which have generated felt earthquakes in the past. This region is known to have generated sporadically occurring earthquakes, as was the case on the 24th of April 2011, when a seismic swarm of around 15 events occurred over a period of 4 days. The largest event had a magnitude > 4 (intensity IV) and was clearly felt all over the archipelago, but especially on the eastern side of Malta;

iii) many earthquakes take place in swarms. We also conclude that the earthquake catalogue for this region is complete down to magnitude 3.0 earthquakes and that this region has the potential of an earthquake with magnitude >5. In November 2022 a seismic strike of magnitude 4.6 was registered quite close to Gozo. The public then reacted to the rumbling noise & shaking of premises which shook for circa 10sec. Considering this to be a clay site magnitude 4.6 could relate to within the range MMV -MMVI.

It has been reported from seismologists that the present intensified seismic activity is considered normal for Malta having occurred on other occasions. To be noted that over the past 20 years Malta has become better equipped to measure the ongoing activity, than over previous years. The recorded activity used to

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[5] D'Amico, S., Galea, P., & Panzera, F. (2013). Seismic Hazard Maps for the Maltese Archipelago: Preliminary Results. American Geophysical Union.

[6] D'Amico, S., Panzera, F., Akinci, A., Galea, P., Agius, M., & Lombardo, G. (2015). Seismic hazard maps for the Maltese archipelago (Central Mediterranean). 26th IUGG General Assembly. Prague

[7] An instrumental earthquake catalogue for the offshore Maltese islands region, 1995–2014 - Matthew R. Agius, Pauline Galea, Daniela Farrugia and Sebastiano D'Amico, ANNALS OF GEOPHYSICS, 63, 6, SE658, 2020; doi:10.4401/ag-8383

be limited mostly to people's reaction during a quake, or from data provided by foreign nearby stations, such as in Sicily. These cluster seismic events were previously not picked up.

It is further noted that Malta's proximity to Sicily and its higher earthquake hazard have sparked in recent years a debate on the likely risk for the archipelago. A series of probabilistic seismic hazard assessment (PSHA) have been conducted and published in 2013[8] [9], showing results of PGA between 0.09-0.18 g for 10% probability in 50 years (475 return period), changing Malta's classification risk from "Low" to "Moderate". Recent Seismic activity in the Mediterranean over the past months is also indicating a PGA of 0.1g.

The above all converges to define the Maltese Islands as a low seismic area as per EC8, falls within <0.1g but >0.04g, with simplified design provisions to be undertaken.

This is further confirmed as noted previously, that in the existing catalogue of Malta's seismic events, no mention is made of any deaths to have occurred. As further noted from table No. 2, the earthquake intensity should have been lower than MMVIII.

Further when historical evidence refers to houses being extremely shattered, these should be referring to very old constructions in rubble masonry, which as per table No. 1 are classified as type A constructions, which at a base shear design % of gravity, suffer damage to an MDR of 2% for MMV, 10% for MMVI, 45% for MMVII & 60% for MMVIII, as noted from table No. 5.

Masonry EC8 Design Criteria for Zones of Low Seismicity.

EC8 notes the conditions under which unreinforced masonry that follows solely the provisions of EC 6[10], may be used in a country. It is to be noted that such use is recommended only in low seismicity cases. This implies that the normal masonry Eurocode 6 may be adopted for Malta.

1/- Shear walls in manufactured stone units are to have thickness t >175mm. **This fortunately is the thickness for internal partitioning adopted at 180mm.** For party walls due to acoustic considerations and for improved thermal capacity, a thickness of 230mm adopted.

Further h_{eff}/t <15

l/h_{min} < 0.35

where t is the thickness of the wall

h_{eff} effective height of the wall

h greater clear height of the openings adjacent to the wall

[8] D'Amico, S., Galea, P., & Panzera, F. (2013). Seismic Hazard Maps for the Maltese Archipelago: Preliminary Results. American Geophysical Union.

[9] D'Amico, S., Panzera, F., Akinci, A., Galea, P., Agius, M., & Lombardo, G. (2015). Seismic hazard maps for the Maltese archipelago (Central Mediterranean). 26th IUGG General Assembly. Prague

[10] Eurocode 6 - Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures

l length of the wall

For $S_\delta < 3$ m/s^2, unreinforced masonry buildings should be designed to DC2

- Normal to bed face: $f_b = f_{bv} \geq 3$ MPa
- Normal to bed face: $f_b = f_{bv} \geq 3$ MPa
- General purpose mortar: $f_{m,min} = 2.5$ MPa

2/-a minimum of two parallel walls is placed in two orthogonal directions, with a spacing of these walls not smaller than 75% of the building dimension orthogonal to the wall plane, at least for the walls in 1 direction. The primary walls should be placed far from the centre of mass so that the building is not classified as torsionally flexible.The cumulative length for each shear wall should be >30% of the length of the building. In cases of low seismicity, the wall length required may be provided by the cumulative length of the shear walls in one axis, separated by openings. In this case, at least one shear wall in each direction should have a length, *l,* not less than that corresponding to twice the minimum value of $l/h < 0.35$ as above. The length of wall in resisting shear is taken for that part that is in compression.

3/- For unreinforced masonry buildings, walls in one direction should be connected with walls in the orthogonal direction at a maximum spacing of 7 m.

4/- The plan should be approximately rectangular. The ratio between the length of the small side and the length of the long side in plan should be not less than a minimum value, $\lambda_{min} = 0.25$.
The area of projections of recesses from the rectangular shape should be not > $p_{max} = 0.15$
of the total floor area above the level considered.

4/- For a design ground acceleration <0.07g, the allowed number of storeys above ground is 4 floors for unreinforced masonry and 5 for reinforced masonry, however for low seismicity a greater number of stories are allowed.

5/- Mortar type to be adopted should be at least M2, although lower resistance may even be allowed, whilst for reinforced masonry M5 may be used. Further there is no need to fill the perpendicular joints (in the updated version of EC 8 M3 & M10 respectively are being suggested).

6/- 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 vertical centres, or reinforced concrete ring beams, reinforced with a minimum longitudinal reinforcement of 200mm^2. The axial resistance of ring ties and ring beams should not be smaller than 150 kN.

7/- pA,$_{min}$ is the absolute minimum ratio of the area of walls in one direction as a % of the floor area. The value for pA,$_{min}$ is 2% for unreinforced masonry buildings and 1.5% for reinforced and confined masonry buildings

It is to be noted that when the proposed floor plans of buildings are being undertaken, the above masonry stiffening elements should be catered for. Layout plans should not be based solely on planning criteria, such as the lower especially basements levels catering for parking facilities which necessitate open spaces, however a stiff geometrical layout is to be provided for, lateral resisting elements extending for the full depth of the building, not being cut short in some instances at ground level.

Item 1 above notes that the thickness of shear walls is not to be less than 175mm. However for Malta it should be considered that at the lower levels, solid walls thinner than 230mm are not adopted.

The Case for Lean Seismic Design

Seismic design is now being undertaken by EN 1998 parts 1 – 6, with however the parts relating to masonry building include for EN1998-1:2004[ii] & EN1998-3:2005[iii]

The following 2 equations from ref vii, note the importance of the q factor on the calculation of the seismic horizontal force. Eq 3.14, then notes that the higher the material value of q, the lower will be the seismic horizontal force.

$$F = S_d(T_1) \cdot m \cdot \lambda \qquad (4.5)$$

Where $S_d(T) = a_g \cdot S \cdot 2{,}5/q \qquad T_b < T < T_c \qquad (3.14).$

or $S_d(T) = a_g \cdot S \cdot 2{,}5/q \,\{T_c/T\} \qquad T_c < T < T_D \qquad (3.15).$

F is the horizontal seismic force acting on the structure, m is the seismic mass of the building & a correction factor $\lambda = 0.85$ is applied if the building has more than 2 storeys, otherwise $\lambda = 1$.

$S_d(T_1)$ is the design spectrum as calculated from equations 3.14 or 3.15 depending on the period of vibration at period T_1, whilst the S factor depends on the type of ground in existence.

It is thus noted that the seismic force F is dependent on the

1/- PGA a_g, determined for a mean return period with a value recommended in EC8 of 475 years. It is further to be noted that this PGA is to be determined for rock or other rock-like formation, **including mostly 5 m weaker material at the surface.** Is this bold statement which appears to cater for the alluvial river plains, considered when the PGA of a region is established?

2/- the S factor which depends on the type of founding material.

3/- the behaviour q factor is a structure-dependent parameter used to reduce seismic design forces below those corresponding to elastic response. This masonry seismic force reduction factor or behaviour factor, known as the q-factor, accounts in an approximate way, for inelastic response at ultimate.

4/- the seismic mass which as quoted in[11] approximates to ball park figures of 1.2ton/m^2 for concrete buildings & at 0.6ton/m^2 for steel buildings.

[11] Manual for the seismic design of steel and concrete buildings to Eurocode 8, The Institution of Structrual Engineers, 2010.

For a seismic lean design the factors as noted in item Nos. 1 & 3 do not have to carry any over design element in them. Regarding the seismic mass as this does not provide for any safety factors, whilst the live loads have massive reductions applied, no over design should occur here, if the dead loads in place are known.

On the other hand the estimation of the fundamental frequency T_1 of the structure has a bearing on the calculation of the seismic force F. The fundamental frequency will dictate whether equation 3.14 or 3.15 in ref 2 will have to be applied.

Ref 2 notes: For buildings with heights of up to 40 m the value of T_1 (in sec) may be approximated by the following expression:

$$T_1 = C_t \cdot H^{3/4} \qquad (4.6)$$

Where C_t is 0,085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0,050 for all other structures.

Now does not all other structures also refer to shear walls, but then equation 4.7 notes:

Alternatively, for structures with concrete or masonry shear walls the value C_t in expression (4.6) may be taken as being

$$C_t = 0{,}075/ A_c\text{^}0.5 \qquad (4.7)$$

The fundamental frequency of a 11.6m 4-storey high masonry building of rectangular plan layout 13m x 20m is calculated at 0.314s by equation 4.6 and 0.143s from equation 4.7 on type C ground formation, such as clay.

The 4-storey buildings of seismic mass 12,067kN have been utilised for PGA's from 0.012g – 0.24g.

Due to the differing fundamental frequency obtained, for T_1 = 0,314s equation 3.14 is utilised, whilst for T_1 =0.143s equation 3.15 is now utilised. Table No. 3 now notes a substantial difference in the seismic horizontal force F as calculated, whence by utilizing equation 3.14, the seismic force works out at 30% lower than for equation 3.15.

TABLE No. 3 - F in kN.

PGA	T=0.143s q = 1.5 type C soil	% F - seismic mass	T=0.314s q = 1.5 type C soil	% F - seismic mass
0.012g	334	2.55%	232	1.77%
0.024g	669	5.10%	464	3.54%
0.048g	1337	10.20%	929	7.08%
0.096g	2675	20.40%	1858	14.17%
0.16g	4458	34.00%	3096	23.61%
0.24g	6687	66.90%	4644	35.42%

Now the % difference is even higher at 58%, if the q-value is also varied with q =2.5 applied to equation 3.14, whilst applying a q = 1.5 for equation 3.15, as noted in Table No. 4, which outlines the horizontal forces for structures founded on rock.

TABLE No. 4 - F in kN.				
PGA	T=0.143s Q = 1.5 Type A rock	% F - seismic mass	T=0.314s Q = 2.5 Type A rock	% F - seismic mass
0.012g	223	1.70%	93	0.71%
0.024g	446	3.40%	186	1.42%
0.048g	892	6.80%	372	2.83%
0.096g	1783	13.60%	743	5.67%
0.16g	2972	22.67%	1238	9.44%
0.24g	4458	34.00%	1858	14.17%

Table Nos. 3 & 4 note that the seismic force on rock approaches 13.6% of the seismic mass for a PGA of 0.096g increasing to 20.4% if founded on clay. These %'s decease to 6.8% & 10.2% respectively for a PGA of 0.048g.

Further work on masonry structures in Slovenia[iv] noted that as these are rigid structures with natural periods of vibration ranging between periods where the response spectrum is flat, therefore, the ordinate of the design spectrum for masonry buildings can be determined from equation 3.14.

The horizontal seismic force as established from equ 4.6 above, will subject the masonry structure to rocking & sliding effects. For relatively small axial loads, slender piers tend to overturn and exhibit rocking, while stocky piers tend to exhibit sliding due to a lower normal force along horizontal joints. On the contrary, high axial loads tend to provide sufficiently large resisting moments to resist overturning and a sufficiently high normal stress to prevent sliding, so shear failure tends to govern for most aspect ratios.

To calculate moment & shear resistance of the masonry piers, the mechanical properties of masonry are obtained from Eurocode 6.1.1 ref 10. The characteristic shear strength of masonry using general purpose mortar and having the perpend joints unfilled, but with adjacent faces of the masonry units closely abutted together, may be taken as:

$$f_{vk} = 0{,}5\, f_{vko} + 0{,}4\,\sigma_d \quad (3.6)$$

where f_{vko} is taken at 0.1N/mm^2 (table 3.4 of EC6.1.1) for manufactured/natural stone with M2 mortar, but not greater than 0,045 f_b or f_{vlt}
where $f_{vk} \leq f_{vlt} / \Upsilon_m$

Lateral force method of analysis

Both references ii, note when taking into account the regularity of masonry buildings whose response is not significantly affected by contribution from higher modes of vibration, the lateral force method of analysis based on equation 4.5 will provide adequate results.

This method is deemed to be satisfied in buildings which fulfil both of the two following conditions.

a) they have fundamental periods of vibration T_1 in the two main directions which are smaller than the following values: $T1 < 4T_c$ or 2s

Where T_c is obtained from Table Nos 3.2 (Type 1 elastic response) or 3.2 (Type 2 elastic response).

b) they meet the criteria for regularity in elevation.

Literature Review for q-values in Unreinforced Masonry Construction

The following Literature Review will now discuss, a masonry seismic force reduction factor or behaviour factor, known as the q-factor which may vary between 1.5 & 2.5+, accounting in an approximate way, for inelastic response at ultimate.

The findings from the latest update on the q-factor is to be noted in reference[12]. The text in italics is from *"Latest Findings on the Behaviour Factor q for the Seismic Design of URM Buildings 2020".*

Recent earthquakes as the 2012 Emilia earthquake sequence showed that recently built unreinforced masonry (URM) buildings behaved much better than expected and sustained, despite the maximum PGA values ranged between ***0.20 - 0.30g,*** *either minor damage or structural damage that is deemed repairable. Especially low-rise residential and commercial masonry buildings with a code-conforming seismic design and detailing behaved in general very well without substantial damages.*

However, the results of the safety checks adopting linear methods of analysis applied to common real structural configurations of masonry buildings using a q-factor equal to 1.5-2.0, as suggested by some seismic codes like the current version of EC8, were found to be overly conservative and in contradiction with the experimental and post-seismic evidence. It was evident that using a q-factor equal to 1.5-2.0 as suggested by some seismic codes (e.g. the current version of EC8, CEN 2005a), it was practically impossible to satisfy strength safety checks for any configuration of two- or three storey URM buildings for PGA greater than 0.10g. In many cases, the strength safety checks would not be satisfied even for a_gS *greater than 0.05g.*

As a result of the investigations, rationally based values of the behaviour factor q to be used in linear analysis in the range of 2.0 to 3.0 are proposed for well-constructed box behaviour URM buildings. A strong irregularity can produce a decrease of the behaviour factor of about 30%.

A previous paper 2008[13], now notes that this preoccupation on workings of URM buildings had long been forthcoming.

It was evident that, using a q-factor equal to 1.5-2.0 as suggested by some seismic codes (e.g. EC8) it is practically impossible to satisfy strength safety checks for any configuration of unreinforced 2 or 3 storey masonry buildings for peak ground acceleration a_gS *greater than 0.1g. In many cases the strength safety checks would not be satisfied even for* a_gS *greater than 0.05g.*

An even earlier paper[14] 2004, defines the research project as undertaken in Slovenia on q-values.

[12] Latest Findings on the Behaviour Factor q for the Seismic Design of URM Buildings – Morandi, Butenweg, Breis, Beyer, Maganes – 2020.

[13] Some issues on seismic design & assessment of masonry buildings based on linear elastic analysis – Magenes & Morandi – 2008.

[14] STRUCTURAL BEHAVIOR FACTOR FOR MASONRY STRUCTURES – Tomazevic, Bosiljkov, Weiss - 2004

Low-rise unreinforced masonry buildings URM - family houses represent the major part of masonry construction in Europe. The study indicated that the values depend not only on the system of construction, but also on the **properties of masonry materials** and structural configuration of the building under consideration.

The seismic resistance needs to be verified by calculation, unless the buildings are in conformity with the requirements for simple masonry buildings in the case of which the calculations are not mandatory.

A range of values of q factor for different systems of masonry construction is proposed in the recent draft of EC 8:
• for unreinforced masonry: q = 1.5 - 2.5,
• for confined masonry: q = 2.0 - 3.0,
• for reinforced masonry: q = 2.5 - 3.0.

*Following the simple definition and the observed behaviour, some estimates regarding the validity of the proposed values of q-factor have already been carried out on the basis of the results of models of masonry buildings tested on the shaking-table. The values of **q = 2.84,** 2.69 and 3.74 have been obtained for the cases of **unreinforced,** confined and reinforced masonry buildings, respectively.*

*However, without systematic analysis of the seismic behaviour of masonry buildings during recent earthquakes, it is not possible to define the values of q-factors. **Otherwise we face the risk that the design situation will not be realistic.***

This 2004 statement called for the correlation of the damage observed following a seismic event, whether this truly corresponds with the structural parameters being adopted. This has been adopted over the following years, as noted from more recent writings.

To account for the actual ability of masonry to redistribute loads, the Italian code NTC 2008,specifies a q-factor of 2.0 but recommends an over-strength ratio OSR of 1.4 & 1.8 for single & multi-storey buildings, resulting in an effective q-factor of up to 3.6[15]

The Mean Damage Ratio (MDR) & the effects of Irregularity and asymmetry [16]

Table 5 is the average damage to buildings of about identical vulnerability and architectural characteristics, expressed as a percentage of their new value.

[15] Seismic Design of buildings to Eurocode 8_Design of Masonry Structures – Dejong, Penna, Taylor & Francis Group - 2016.
[16] Camilleri D. H., Vulnerability of buildings in Malta to earthquake, volcano and tsunami hazard" The Structural Engineer Volume 77/No 22 36 November 1999.

Table 5 - Mean Damage Ratio (MDR) For Building Type Against Earthquake Intensity founded on rock, being moderately asymmetrical & irregular[17].

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

The present majority range of Maltese buildings fall within types B-D_1 represented in bold in table 5.

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 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 5 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 the method of construction and providing tying provisions.

It is recognized 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 enhanced factor F_r shall be obtained for a highly irregular building, with abrupt change of stiffness between floors. The MDR's in table 7 are worked out for a weighting factor F_{r1} of 1.3 (shape A1 in table 6a) for irregularity and asymmetry in relation to a recessed elevation of building, a similar value for F_{r2} (shape B1 in table 6b) 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 8c) giving a global factor of:

F_{rA} = 1.3 X 1.3 X 1.5 = 2.5

Table 6 -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

Soft designs encountered locally could incorporate a partial soft ground floor, yielding a F_{r1} factor of 4 (shape A2 in table 5a). 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 5b). For the continuous window bands at upper level yields a F_{r3} factor of 2. 5(shape C2 in table 5c), giving a global factor of:

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

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

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

The local buildings which fall into this category are Buildings Type C, and D1 and an amended damage ratio matrix (table 6) is proposed to cater for higher asymmetry and irregularity.

Table 7 - 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%

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 35 years the building construction in Malta has been subjected to changes, brought about from the economic expectations of landed property. The commercialization of buildings has opened up the layout especially at ground floor level, obtaining a flexible soft structure.

STRUCTURAL ROBUSTNESS[18]

This concept was introduced post 1968, following the Ronan Point gas explosion disaster that occurred in East London on the 16th May 1968. This concept has to form part & parcel of Malta's construction method in order to achieve lower MDR's, following a seismic occurrence.

Structural robustness involves structures that:

(1) don't fail like a house of cards;

[18] Mann A. P., et alia, *"Practical guide to structural robustness and disproportionate collapse in buildings"*, The Institution of Structural Engineers, London, 2010.

(2) minor errors not to have a disproportionate effect;

(3) structures not to fail to any great degree under accidental loading

A building's structural form will significantly affect its robustness. Traditional cellular forms with many loadbearing walls assure a sensible level of robustness because loss of any one wall will generally not lead to the collapse of a large proportion of the structure. In contrast, having a large span supported on an easily dislodged and/or vulnerable single column would not be a robust structure.

Additionally, a robust structural concept will be one which avoids situations where damage to small areas or failure of any single element progresses to widespread collapse. Notwithstanding that ideal, there are clearly occasions when reliance does have to be placed on single elements, but at least once this is recognised, their robustness can be improved by making such elements substantial. Experience has shown certain arrangements to be potentially vulnerable; examples include:

– significant transfer beams, these are single beams which support a number of columns or hangers

– apparently minor elements that are required to ensure the stability of more significant elements

– significant cantilevers

– long span, simply supported beams*.

The last two forms have no redundancy but that need not imply unacceptable vulnerability.

Prescriptive rules for robustness include for the provision of horizontal ties both internal & peripheral for buildings up to 4 storeys in height. For buildings higher than 4 storeys, vertical ties are also to be provided for. On the other hand for buildings not higher than 4 floors instead of the horizontal tie requirements this may be supplanted by the provision of effective anchorage of the floors. Lack of anchorage would lead to instability in walls running parallel to floor spans. Most designers opt for providing effective floor anchorage to the walls/beams rather than specifying ties. Such anchorage can be achieved in most cases simply by the friction between the floor and the wall/beam although consideration of parameters such as temperature and camber may negate this approach. Where a floor unit is prestressed, its camber prevents proper contact with walls below running parallel to the span and so friction can only be relied on when bedding mortar is used.

Where in situ topping is not provided, reinforcement can be provided within plank end pockets with bars then grouted up to link units together.

SENSITIVITY TESTING OF THE SEISMIC VARIABLES EFFECTING MALTA's MASONRY BUILDINGS.

The 2 variables considered shall be limited to the PGA & the q-factor.

The discussion as undertaken in section *SEISMICITY & VULNERABILITY OF MASONRY CONSTRUCTIONS IN MALTA*, notes a PGA between bands of 0.55g – 0.1g.

The discussion as undertaken in section *Literature Review for q-values in Unreinforced Masonry Construction*, notes a q-factor limited between a band of 1.5 – 2.85. However this is being limited to within 1.5 – 2.5, as noted in the new suite of EN 1998-1-2:2020, still in draft form.

Table No. 8 now includes for initial preliminary understandings for the number of storeys that may be constructed in masonry as founded on rock or clay. This sensitivity analysis includes for PGA's of 0.055g, 0.075g & 0.1g, whilst q-factors of 1.5 & 2.5 have been adopted.

TABLE No:8	**ROCK**	**ROCK**	**CLAY**	**CLAY**
PGA	q = 1.5	q = 2.5	q =1.5	q = 2.5
0.055g_VI MSK	9flr	13flr	6flr	10flr
0.075g_VII$^-$MSK	7flr	11flr	4flr	7flr
0.100g_VII$^+$MSK	5flr	8flr	3flr	5flr

For masonry buildings higher than 7 - 8 floors other structural criteria may come into play, such as stability in the shorter dimension, axial shortening amongst others which have not been considered. These additional floors as highlighted in red, would have to be treated with caution. Also a proportion of height to width of the building should possibly be limited to being not greater than 3. This proportion has not affected the storey heights as outlined in table No. 8 for a 13m wide building, but could reduce the storey heights alloweable on a building width of 6m.

This analysis has been based on varying-storey height masonry buildings on a plan dimension of 13m width for a depth of 20m, having internal masonry partitions for residential use & separate party walls. Concrete floors tie into the masonry walls according to the stability internal, peripheral & vertical ties noted in previous section *STRUCTURAL ROBUSTNESS.*

Further the masonry load bearing requirements according to Eurocode 8 noted in section *Masonry EC8 Design Criteria for Zones of Low Seismicity*, item Nos. 1 - 5 are to be undertaken.

It is further suggested that although shear walls in item No. 1 is noted to have a minimum thickness of 175mm, that this is topped to 230mm for Malta.

In item No.5 stiffening cross walls are noted to be placed at a maximum of 7m centres. This is not possible if the basement rampway is placed by the party wall. In this instance further stiffening of the party wall is here necessary – is it possible to increase the thickness of the party wall in this location to achieve the desired stiffness?

This elastic linear analysis is based on the seismic restoring moment in the longitudinal stiff direction, whilst in the transverse direction couple action comes into play, with no party wall being subjected to uplift forces. The party wall is taken to be loaded by a 3m span of slab. The alloweable shear force is noted not to be exceeded for both the in-plane & out of plane directions. The vertical load resistance for the party wall will not be exceeded if constructed in franka masonry or hollow core blockwork of minimum compressive strength of 7N/mm^2, infilled 12N/mm^2 for the lower floors.

Our superior building material geometry in ashlar construction may be further taken into consideration. In Europe these URM buildings could be constructed in fragile brittle clay/calcium silicate hollow blocks with very thin shells and webs in the 5mm region, whilst the local webs are in the 30/50mm region for the single & double blockwork types. The local hollow block is classified as a Group 1 unit (void 25%, thickness of shell/web 18mm), whilst the European counterpart also consists of Group 4 units (void 70%, thickness of shell/web 5mm) in EC6.1.1.

These Group 4 units though not significantly influencing the collapse mechanisms when subjected to gravity loads, significantly influence the behaviour of masonry structures of all systems in seismic conditions. It has been shown that they reduce the robustness of masonry units (due to thin shells and webs) and homogeneity of masonry walls (due to masonry bond) as structural elements.

Table No. 9 now notes the vertical load bearing characteristics of the local masonry units adopted.

Table 9 - vertical load bearing characteristics of the local masonry units adopted, as per EC6-1-1 & EC8-1-1.

Material	Group type/k value to equation 3.1 in MRA EN 1996-1-1	Crushing strength N/mm²	M2 -EC6 KN/m	M2 -EC6 KN/m accidental/ seismic	M5 - EC6 KN/m	M5 -EC6 KN/m accidental/ seismic
225 franka	1/0.45	20	536	786	705	1034
180 franka	0.45	20	436	640	574	842
150 franka	0.45	20	379	557	499	733
225 block dobblu	1/0.55	8.5	360	528	474	695
225 block singlu	2/0.45	7	257	377	338	496
180 block	0.45	7	209	307	275	404
150 block	0.45	7	182	267	240	351
115 block	0.45	5	114	167	150	220
225 infilled block dobblu	0.55	11.3	440	645	579	850
225 infilled block singlu	0.55	10.8	425	623	559	820

The characteristic alloweable load is undertaken as per Eurocode 6 ref. 10, by applying a factor of safety of 2.2, whilst the characteristic accidental/seismic load is undertaken as per Eurocode 8 ref. 2, which notes that the factor of safety to masonry for seismic design is to be taken at 2/3's of the factor of safety for the permanent load design, but not less than 1.5.

This sensitivity testing goes beyond the recommendation of Eurocode 8 in item No. 3 *Masonry EC8 Design Criteria for Zones of Low Seismicity,* namely:

For a design ground acceleration <0.07g, the allowed number of storeys above ground is 4 floors for unreinforced masonry and 5 for reinforced masonry, however for low seismicity a greater number of *stories are allowed. This statement should presumably be updated when the new suite of Eurocode 8 will come into force.*

A study[19] as undertaken on the M5.4 Kraljevo earthquake 4km from the epicenter & focal depth of 13Km MM 8.15 caused 2 deaths. 5 storey unreinforced masonry buildings are said to be damaged, with the founding material includes for medium-dense and dense soils, noted as being weaker than stiff clay.

Table No. 8 is also to be guided by[20] which notes that isolated contemporary loadbearing unreinforced masonry buildings, would not be damaged by an earthquake with a design PGA of 0.10 g if their overall height is not greater than three floors, if built on rock and, two floors, if built on clay. The q-factor has not been discussed but is assumed that these findings relate to q =1.5. Table No. 8 refers to 4floors & 3 floors respectively for a PGA of 0.10g for a q = 1.5.

The analysis of ref 20 has been complied via the introduction of various factors, as noted in the section *The Mean Damage Ratio (MDR) & the effects of Irregularity and asymmetry*. It is then noted that the vulnerability index intervals given in the GNDT document were altered in order to reflect better the different classes of seismic vulnerability for the contemporary loadbearing unreinforced masonry building typology in the Maltese Islands. This could have a bearing on the differences on the alloweable floor heights, as noted above.

CONCLUSIONS & RECOMMENDATIONS

From the above by classifying Malta's seismic risk hazard as low for the period under which a seismic catalogue has been drawn up, the same may not be said for the seismic vulnerability of Malta's building stock.

EC8 threads carefully for masonry buildings above 4 stories heights for low seismic risk. For moderate seismic risk the number of load bearing masonry floors is reduced to 2. It is however to be noted that the updated EC7 still in progress, presently noted for a PGA of 0.16g URM constructions may be up to 4 stories in height. On the other hand in the case of terraced development where shear walls may be 20m deep table No. 8 notes that this 4 storey classification may be exceeded.

By comparing the base shear as a % of 'g' to be resisted in an earthquake of particular intensity from table 5, it is to be noted that for no damage to be suffered during an MMVI, building types to be D_1, and to higher specifications at MMVII & MMVIII. 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.

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[19] Predrag Blagojevic´, Svetlana Brzev & Radovan Cvetkovic, "Simplified Seismic Assessment of Unreinforced Masonry Residential Buildings in the Balkans: The Case of Serbia", Buildings 2021, 11, 392.

[20] Torpiano A., Bonello M., Borg R.P., Sapiano P. & Ellul A., M., "The development of a seismic vulnerability assessment methodology for contemporary loadbearing masonry buildings in the Maltese Islands", Int. J. Sustainable Materials and Structural Systems, Vol. 2, Nos. 3/4, 2016.

It has been noted that prior to the past 15-year period, Malta's height limitation had been limited to 2 to 3 storeys. In Valletta and Sliema buildings up to 5 stories high have been in existence over a long period of time. In Valletta 2 in number 8-storey buildings have been in existence for over 100 years.

Since 15 years ago the previous height limitations of 2 to 3 floors have gradually increased varying from 5 to 8 floors depending on location. Unfortunately these increased storey limitations have not been upped with improved construction methods. Unlike the high rise constructions presently being undertaken in Malta, which adopt a reinforced concrete cantilevered core construction around the stair/liftwells, these 5 to 8 floor constructions (in some locations even upped to 11 floors, considering the additional 2-floor hotel policy in place) still adopt the usual load bearing masonry constructions as undertaken in the previous 2 – 3 floors constructions.

This all points towards, that whilst the Maltese Islands could be noted as a low seismic risk, the seismic vulnerability of buildings over the past 35 years in the medium rise masonry category of 5 to 8 floors heights together with additional basement floors has been increasing. This signifies that although no deaths have been recorded during Malta earthquakes over a c. 500-year period, the same cannot be said for the future due to the increased vulnerability of Maltese buildings. It has been noted in the section *The Mean Damage Ratio (MDR) & the effects of Irregularity and asymmetry*, that the effects of asymmetry, compounded via the creation of soft storeys can lead to an amplification of MDR[21] given by 6 times. Hence the importance of obtaining symmetrical plan layout to achieve robust layout, to reduce the amplification factor noted of 6 times to the MDR's quoted in table No. 5 for buildings being moderately asymmetrical & irregular.

With the introduction of deep hollow core prestressed slabs in the early 80's, being utilized as transfer slabs, supporting a number of masonry overlying cellular residential floors, the incidence of soft storeys at ground floor was increased. These slabs, normally sit freely on the supporting structure, with no tying provided to the rest of the structural system.

Tying at corners for these medium rise buildings together with provisions for progressive collapse as outlined in EC1-7[22] Annex A, are not presently commonly undertaken. The tying of the various structural systems is a requisite to obtain a rigid diaphragm tying the whole building together.

On the other hand robustness requirements as issued post 1968 relate. These for all floor elements to be anchored to masonry walls so as to form effective horizontal ties in a similar manner to reinforced concrete structures: these are considered to have withstood the test of time in providing adequate robustness for masonry structures, which are not higher than 4/5 storeys. The lack of stiff floors & poor wall-slab connections, has led onto façade collapses during seismic events[23].

Thus in masonry, it is usual to require the external walls and piers to be adequately connected to the floor construction to prevent their premature failure under outward pressure. This can be achieved by relying upon the shear strength of the connection, based on the type of masonry unit, mortar

[21] D.H. Camilleri, *Malta's risk minimisation to earthquake, volcanic and tsunami damage*, Disaster Prevention & Management Vol. 12 No. 1, 2003 pp 37 -47 MCB UP Ltd.
[22] MSA EN 1991-1-7:2006, Actions on structures. General actions. Accidental actions
[23] Beyer K., EN 1998-1-2 Buildings Specific Rules for Masonry Buildings, 2023.

strength class and design vertical loading, or on its frictional resistance based on design vertical loading and appropriate coefficient of friction if the wall is loadbearing. This anchorage obtained in practice works instead of providing for the horizontal ties as stipulated, for buildings 4 floors high or lower.