Seismic Unreinforced Masonry URM Design & Climate Emergency

Much has been written over the past year to be lean in our structural designs. This due to most of the embodied carbon tending towards 67% of the building developments being undertaken nowadays, in relation to foundation & frame works[i].

This started off with *Rethinking Floor Loadings*[ii]. Here it is quoted that based on a MEICON survey[iii], it is known that imposed loadings are vastly greater than reached in real buildings. A plea is made that lower loads leading to smaller structures leads to potential embodied carbon savings. Reductions in dead loads as stipulated in equation 6.1b of EN1991-1:2002[iv] together with floor area & storey height reductions are to be fully pursued.

Then a paper on *Vertical extensions: technical challenges & carbon impact*[v], noted that a solution that does not rely on the existing building, involves almost 50% more embodied carbon than one that can justify the increase in load on the existing structures & foundations. Further providing an exoskeleton & new foundations to an additional 2 floors, was further noted as being still a lower carbon option, than demolishing & constructing a new building on the same site. Ref i had also referred to moving away from new structures towards increased reuse & retrofit, the benefits of lighter loads, becomes highly significant towards creating opportunities for vertical extension & foundation reuse.

Another paper on justifying an existing structure[vi], notes that retrofitting & extending existing buildings minimises waste of the materials & energy already invested in these structures & the amount of additional material used. For the concrete building, noting that concrete continues to gain strength after construction, roughly 10% over its 1st few years, a 10% loading increase is then considered acceptable. It was then considered that the expense undertaking for the testing of the concrete testing, more than compensated for the strengthening to the concrete members as undertaken for the additional floors imposed on this exiting building. It was acknowledged that the additional surveys, opening-up undertaken resulted in a significant reduction of necessary strengthening works.

A steelwork fabricator then made a plea for greater material efficiency in response to the climate emergency. Here in ref i, lean design calls for minor 1 - 2% overstress may be acceptable, which goes against codified design. The days of overdesign must now become a thing of the past. There is a place for marginal overdesign at concept design stage, where much detail remains outstanding, but overdesign has no place at detailed design stage.

The Case for Lean Seismic Design

Now with engineers being in the forefront for Climate Change, the advantages of traditional masonry construction is truly to be advocated. It provides better thermal, acoustic, fire rating properties, together with added stability against gales, hurricanes, floods, whilst being a more economic form of construction, together with delivering substantial carbon benefits.

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[vii] & EN1998-3:2005[viii]

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. This refers to the lateral force method, whilst other more advanced methods are referenced to in EC8.

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

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 λ = 0.85 is applied if the building has more than 2 storeys, otherwise λ = 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 at most 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[ix] approximates to ball park figures of 1.2ton/m^2 for concrete buildings & at 0.6ton/m^2 for steel buildings.

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 vii will have to be applied.

Ref vii 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 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 11.60m 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 4.7s on type C ground formation.

The 4-storey buildings of seismic mass 13,112kN have been utilised for PGA's from 0.012g – 0.16g. For a PGA of 0.24g a 3-storey building of building height 10.875m with a seismic mass of 9,834kN has been adopted with the respective fundamental frequencies working out at 0.253s & 0.115s.

Due to the differing fundamental frequency obtained, for T_1 = 0,314s (0.253s) equation 3.14 is utilised, whilst for T_1 =0.143s (0.115s) equation 3.15 is now utilised. The bracketed figures are for the 3-storey building.

Table No. 1 now notes a substantial difference in the seismic horizontal force F as calculated.

TABLE No. 1 - 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	5015	51.00%	3483	35.42%
* this is for a 3-storey, whilst the above are for 4-storey constructions				

Table No. 1 notes substantial differences existing depending on the fundamental period adopted, whether computed according to equation 4.6 or 4.7. A 4-storey unreinforced masonry design example in[x] refers to a recommended approximate natural period of 0.3s by adopting equation 4.6.

Further work on masonry structures in Slovenia[xi] 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.

Lateral force method of analysis

Both references x & xi, 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.

Recent research into q-values in Unreinforced Masonry Construction URM

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[xii]. 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.

As a result of the investigations, rationally based values of the behaviour factor q to be used in linear analyses 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[xiii] 2008, 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, CEN 2005a), 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 ref xi 2004, defines the research project as undertaken in Slovenia on q-values.

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.

EN1998.1.2004 and 'simple masonry buildings'

The above literature review notes that a European problem exists with URM buildings. With the present q-factors as quoted in EC8.1, these buildings cannot be certified as seismically stable unless the PGA is in the 0.05g region. On the other hand use of non-linear analysis methods, such as push-over and time-history can overcome this issue. Table 9.3 in EN1998.1.2004 then circumvents this dilemma, as simple buildings are allowed, although not higher than 4 storeys.

Table 9.3: Recommended allowable number of storeys above ground and minimum area of shear walls for "simple masonry buildings"

Acceleration at site $a_g.S$		≤ 0,07 *k*.g	≤ 0,10 *k* .g	≤ 0,15 *k*.g	≤ 0,20 *k*.g
Type of construction	Number of storeys (*n*)**	Minimum sum of cross-sections areas of horizontal shear walls in each direction, as percentage of the total floor area per			
Unreinforced	1	2,0%	2,0%	3,5%	n/a
	2	2,0%	2,5%	5,0%	n/a
masonry	3	3,0%	5,0%	n/a	n/a
	4	5,0 %	n/a*	n/a	n/a

Ref x had commented on the rules for *'simple masonry buildings in EC8 para 9.7 essentially provide seismic safety by ensuring a conservatively large amount of lateral resistance in terms of total shear* *wall area, limiting the height as a function of the design ground acceleration, and eliminating irregularities (in plan, elevation and height) that are known to cause amplified seismic demands. The height limits are quite restrictive for unreinforced masonry structures. For example, for ground acceleration levels greater than 0.1 g for building heights greater than two storeys, following these rules alone is not sufficient, and detailed analysis is required.*

The above is what the literature review has commented adversely upon, as recent earthquakes have demonstrated that URM buildings have withstood seismic effects in the 0.2 – 0.3g region satisfactorily.

Reading intently note 9.3.5 in relation to table 9.1 of EN1998.1.2004, this notes the behaviour factor q at a platonic value of 1.5, not even distinguishing between ashlar & rubble infill masonry material. It is then noted that: *If the building is non-regular in elevation the q-values listed in Table 9.1 should be reduced by 20%, but need not be taken less than q = 1.5.*

So for a soft-storey structure a q-factor of 1.5 is to be applied, which also then notes this to be not less than. Why has not a range of q-factors been provided, with only the worst case scenario given serving for all situations?

To put the masonry q-factor into perspective quoted within the 1.5 – 2.85 range (ref x - Italy also notes a q-value 3.6, as an additional factor added on, that caters for the capacity of masonry to redistribute loads):

Concrete medium ductility DCM 3.9 high ductility DCH 6.75,

Steelwork medium ductility DCM 4.0 high ductility DCH 8.

On hindsight now, EC8 as noted above has been complacent on the q-factor for over 20 years. Possibly the agenda was for URM buildings to be discouraged from seismic areas, not compatible with the lean structural design presently being advocated.

Table 14.3 being the present revised version of table 9.3 above, as now included in the revised version to EN8-1-2[xiv], notes that 4 storey URM buildings are now stable under 3.6m/s^2 (0.16g), with the % of walling at grd level given at 6.5%, whilst 3 storey buildings are stable up to 4.8m/s^2 (0.24g) with the % of walling at grd level given at 6%.

Table 14.3 — Values of parameter $p_{A,ref}$ characterising the minimum area of shear walls for simple masonry buildings

Seismic action index S_δ		< 0,3 m/s^2	< 0,6 m/s^2	< 1,2 m/s^2	< 2,4 m/s^2	< 3,6 m/s^2	< 4,8 m/s^2	< 6,0 m/s^2
Type of masonry	Number *N* of storeys	Values of $p_{A,ref}$ as function of the total floor area per storey						
Unreinforced masonry	1	0,020	0,025	0,030	0,035	0,040	0,045	0,055
	2	0,020	0,025	0,030	0,035	0,045	0,050	0,060
	3	0,030	0,035	0,040	0,045	0,055	0,060	NA*
	4	0,040	0,045	0,050	0,055	0,065	NA*	

*NA means “Not Acceptable”.

It is further noted that the previous table 9.3 & the proposed table 14.3, make reference to the founding material for these simple buildings via the S factor as attached in Ag.S. The minimum masonry unity compressive strength is to be of 12N/mm^2.

Reference vii notes 4 Classes of masonry, the Group 1 unit has a void 25% of thickness of shell/web 18mm, whilst Group 4 has a void of 70%, thickness of shell/web at 5mm). 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. Hence in seismic regions, it is desirable that only Class 1 & 2 units are adopted.

Workings Undertaken in relation to proposed Table 14.3

Delving into Table 14.3 if for URM buildings the %'s quoted on a PGA of 0.16g over 4 floors is taken as correct and further correct on a PGA of 0.24g over 3 floors at 6%, it appears very strange that on a PGA of 0.3m/s^2 (0.12g), a 4-storey building requires 4% of walling.

Noting this, the same building height and plan dimensions as applied in calculating seismic horizontal forces in table No. 1 is being reused for the build-up of Table No. 2. Table No.1 was based on soil type C & a q-value of 1.5, whilst table No. 2 is worked on rock type A, for varying q-values of 1.5 & 2.5.

TABLE N0. 2 – F in kN.

PGA	T=0.143s Q = 1.5 Type A rock	% F - seismic mass	T=0.143s Q = 2.5 Type A rock	% F - seismic mass
0.012g	223	1.70%	134	1.02%
0.024g	446	3.40%	267	2.04%
0.048g	892	6.80%	535	4.08%
0.096g	1783	13.60%	1070	8.16%
0.16g	2972	22.67%	1783	13.60%
0.24g*	3344	34.00%	2006	20.40%
* this is for a 3-storey, whilst the above are for 4-storey constructions.				

Basic structural design undertaken, appear to indicate that the % of walling to be applied for simple buildings as noted in table 14.3, may be tentatively updated as follows:

6.00% at a PGA of 0.24g, as compared to 6%

6.5% at a PGA of 0.16g, as compared to 6.5%

5.00% at a PGA of 0.096g, as compared to 5.5%

2.50% at a PGA of 0.048g, as compared to 5%

1.25% at a PGA of 0.024g, as compared to 4.5%

0.75% at a PGA of 0.012g, as compared to 4%

These workings as undertaken via the lateral force method indicate a much steeper decline from 6.5% to the 4% in table 14.3. Could it be that factors such as from the p-delta method is the reason for requiring 4% of wall even with such a low PGA?

On the other hand for the very low seismic, especially for PGA of 0.12g, the prescriptive rules for robustness[xv] may be more cumbersome than the proposed % walling updates. These 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 could lead to instability in walls running parallel to floor spans. Designers can opt for providing effective floor anchorage to the walls rather than specifying ties. Such anchorage can be achieved in most cases simply by the friction between the floor and the wall/beam. Tying at corners for these medium rise buildings together with provisions for progressive collapse as outlined in reference[xvi] Annex A, are to be further undertaken. The tying of the various structural systems is a requisite to obtain a rigid diaphragm tying the whole building together.

The seismic effect along the shorter dimension of the building is not noted here, as this is not the dominant scenario, with couple action kicking in on the separate party walls & the existing vertical loading being sufficient to counteract the induced uplift forces during a seismic jolt.

Recommendations

To go for lean seismic design, the above notes that the lateral seismic force induced depends largely proportionally on the peak ground acceleration for the region under consideration together with the q-factor in an inversely proportional manner.

Hence the importance of not overdesigning for the peak ground acceleration as noted above in the live load scenarios, as otherwise this creates repercussions to our Climate Emergency strategy. The literature review has noted that over the past 20 year + period, overdesign has been placed on URM buildings, as the seismic Eurocodes ref vii & viii, have limited the q-value for masonry to within 1.5. This to the detriment that URM buildings may have to be discarded, notwithstanding their positive impact in the other aspects as outlined above, in seismic regions. Should not a distinction be undertaken in deciding on the upper range of the q-factor, whether masonry is in ashlar or deteriorated infilled masonry constructions?

Following references xi - xiii, updates to structural masonry is visible in the present ongoing updates to the seismic Eurocodes. The recent table No. 14.3 as compared to the previous Table No. 9.3 is a case in point. The % of walling quoted for simple buildings, appear however, to be on the high side for low seismic regions, with proposed updating given.

Further to the above, improved lean seismic design may further be obtained if it is clear for masonry shear walls which equation, either 4.6 or 4.7 in ref vii is applicable, for calculating the fundamental period. Table No. 1 notes the huge discrepancy in calculating the seismic horizontal force, dependent on the equation applicable.

It may also be the opportune time to demystify seismic design for simple buildings. Table No. 1 notes the seismic horizontal force for buildings on a type C soil to increase from 2.5% for PGA 0.12g up to 35% for PGA 0.16g of the seismic mass. Then Table No. 2 notes the seismic horizontal force for buildings on a type A rock to increase from 1.75% for PGA 0.12g up to 22.5% for PGA 0.16g of the seismic mass. These %'s are based on a q-value of 1.5. These %'s go even lower for a q-factor of 2.5.

By simplifying the design process, it will be ascertained that more seismic design checks are undertaken in the structural design offices.

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