Fixity moment induced on masonry walling

Dennis Camilleri reports on the feedback from experience in Malta

BS 5628: Part1: 1978, Structural use of unreinforced masonry in Cl. 31, dealing with eccentricity at right angles to the wall, permits at the discretion of the designer, the assumption that the load acts at $1/3$ of the depth of the bearing area from the loaded face of the wall as shown in Fig 1. Furthermore, the resultant eccentricity of the load at any level may be calculated on the assumption that the total vertical load on a wall is axial immediately above a lateral support.

The code initially suggests that preferably the eccentricity should be calculated; however no method is suggested, although the most obvious appears to be the moment distribution method.

Analytical methods

Since 1978, analytical methods have evolved, such as Hendry's work[1], and a method is also included in Appendix C of Eurocode 6 Part 1-1 (presently referenced as DD ENV 1996-1-1) giving a general basis for the design of buildings and civil engineering works in unreinforced, reinforced, prestresssed and confined masonry. Here, as stated in a BMS publication[2], the structural eccentricity at right angles to the wall has to be assessed.

A full or partial frame analysis may be used provided that it takes into account that excessive wall moments will need to be redistributed. It is further implied that the eccentricities are to be limited to not greater than 0.4 times the wall thickness, with the rectangular stress block limited to <0.2t, t being the thickness of the eccentric loaded wall. A reduction factor given by $(1-k/4)$, applied to wall moments takes into account that full joint rigidity is not achieved in practice, where k is taken as the ratio of the sum of the slab stiffnesses to the wall stiffnesses. The value of k is limited to a maximum value of 2. This value is assumed as being too conservative, with a value of 3 approximating to Hendry's research work.

At low values of design vertical load the calculated eccentricity can become excessive and the method is assumed valid only when the average vertical stresses exceeds 0.25N/mm², which limits the eccentricity due to redistribution to 0.4 times the wall thickness, mentioned above.

Maltese construction feedback

The most common residential structural system adopted in Malta relates to cellular load-bearing masonry. A soft, workable limestone building material is the only natural resource available. Terraced housing two or three stories high was considered robust and stable, but the needs of the motorcar have introduced a soft open storey at ground or basement level. Originally, beams supporting the overlying floors spanned this opening, but for about 20 years, hollow prestressed slabs spanning from 6m to 7m have overtaken this structural system.

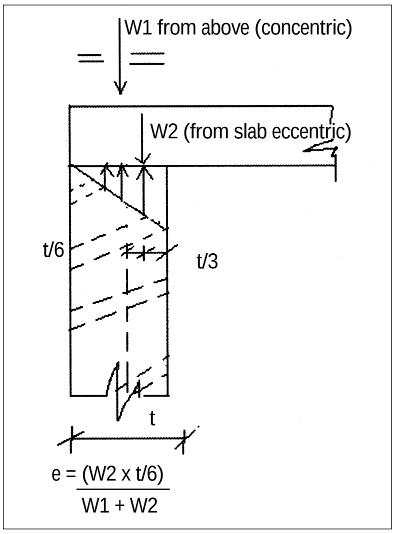

Fig 1. Simplified method

These precast slabs, with thicknesses varying from 280mm to 400mm, depending on overlying loadings, are supported on 225mm thick masonry party walls. These limestone wall panels have an average crushing strength of 20N/mm², with a grade IV mortar, which according to BS 5628 Pt 1 has a characteristic compressive stress of 7.4N/mm², for the particular geometric height-to-width block ratio.

The seating of these precast slabs on the supporting walling, averaging 2.5m to 4.0m in height, does not generally exceed 125mm. The reaction from the precast slab is thus eccentric onto the walling. From Fig 2, a vertical load distribution from the upper floors is carried out. Either corbelling of the masonry walling is allowed for, or else the vertical wall element is assumed to act as a deep beam design, with arching action catered for according to Wood[3].

Some structural tying integrity is achieved for the soft storey at ground level, by providing tying reinforcement in the joint of every 3rd grouted up precast section, i.e. at 3.6m centres. These are linked to a horizontal edge bar and connected to vertical bars inserted into concrete infilled blockwork, again at 3.6m centres[4].

For the load analysis shown in Fig 2, the total characteristic loading from the 2 upper floors and partial roof construction totals 250kN/m run, split as 100kN/m run as direct loading onto the party wall, with the remaining 150kN/m run, as the reaction from the 6.50m spanning precast prestressed slabs.

According to the above loading, Table 1 tabulates the various eccentricities onto the 225mm thick party walling, resulting from the various analytical methods outlined.

a) e = 150kN/m*(225/2–125/3) /(150kN+100kN) = 42.5mm

b) E for concrete is assumed at 25kN/mm² and from tests on limestone walling is taken at 17kN/mm².

If no tests values are available, EC6 gives E values at $1000f_k$ for ULS & $600f_k$ for SLS. For an average f_k value of 20N/mm², this works out at 20KN/mm² and 12kN/mm² respectively.

The Moment Distribution method, on calculating the respective slab and wall stiffnesses, for a fixed ended slab bending moment of 171kN-m/m, yields a restraint bending moment onto walling of 42kN-m/m. This is considered applicable, as the load compression due to the loading, is high enough to contribute to rigidity of joints. The eccentricity e is calculated at 42kN-m/m/250kN/m = 168mm.

c) This value is obtained from the Moment Distribution in the previous row, multiplied by the reduction factor $(1-k/4)$ equated at 0.85, for a k-value of 0.6. An accidental eccentricity is however, further added on, calculated at the effective height/450.

For a ground floor clear height of 4.00m, the effective height assuming a pin joint at base of walling works out at 0.85*4000mm. The eccentricity e is thus calculated at 168mm*0.85 + 0.85*4000mm/450 = 150mm

d) This is a more sophisticated method, allowing for the relative rotation of the wall and slab at the joints, together with changing wall stiffness due to tension cracking in flexure. The bending moments for the superimposed load and dead load are calculated separately at 14.9kN-m/m and 66.2kN-m/m respectively. The joint fixity factor is calculated at 0.74.

The eccentricity e is thus calculated at (14.9 + 66.2)*0.74/250 = 240mm

e) Table 7 in BS 5628 Pt 1 gives capacity reduction factors b, only up to a value of 0.3t, with EC 6 giving values up to 0.33t. The most critical section is located not under the seating, but towards the centre of the wall height, due to the compounding

Table1: Eccentricities obtained on 225mm thick walling according to various methods outlined (e=BM/W)

	Simplified Method[a]	Moment Distribution[b]	EC6[c]	Hendry[d]
e-mm	42.5	168	151	240
e/te	0.19t	0.75t	0.67t	1.067t

slenderness effects. The eccentricity from the loading is taken to decrease linearly, from a maximum value at the top to no eccentricity at base of walling. Within the middle fifth of walling this eccentricity thus decreases to 0.19t*0.6. For a wall slenderness ratio calculated at 0.85*4000mm/225mm given at 15 and an eccentricity of 0.114t, the β value from table 7 is given at 0.775. This gives a safe, allowable loading on walling, for a material factor of safety of $\gamma_m = 3.1$ calculated at $\beta * f_k * t/\gamma_m = 0.775*7.4*225/3.1 = 416$kN as compared to existing loading of 250kN/m, thus making above construction stable under vertical loading.

Observations

From Table 1, it is to be noted that the above vertical loading scenario appears only stable under the Simplified Method with an eccentricity of 0.22t, subjected to a characteristic rectangular stress block of loaded length given by t(1–2e/t)=0.225m(1–2*0.19)=140mm, with $f_c = 250\text{kN/m}/0.140\text{m} = 1.79\text{N/mm}^2 < f_k/\gamma_m = 7.4\text{N/mm}^2/3.1 = 2.39\text{N/mm}^2$

For the other more rigorous analysis under such high eccentricities, the wall in question should not be stable. Even if the EC 6 condition stipulating that the effect of eccentricity is not to exceed 0.4t, extrapolating table 7 of BS 5628 Pt1, which a member of the BS committee has confirmed as being a valid assumption, although only interpolation is mentioned, gives a β value of 0.31. This yields a design vertical load resistance of $\beta * f_k * t/\gamma_m = 0.31*7.4*225/3.1 = 167$kN/m < characteristic load 250kN/m.

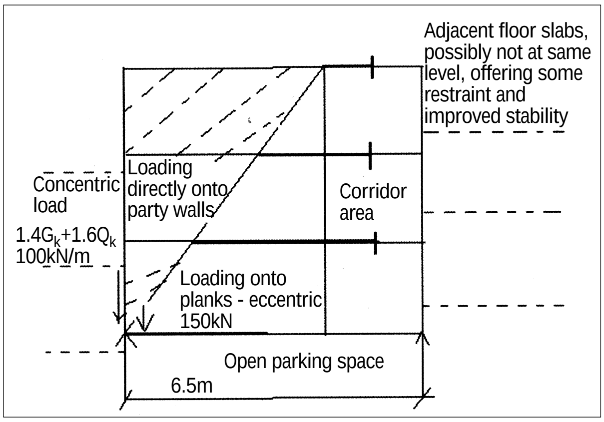

Fig 2. Wall elevation

Such walls have been in existence for around 15 years under vertical loading and are not showing any sign of distress. If the higher eccentricities were present, this would introduce further complications, as they would create a higher fixity moment. The FIP Recommendations[5] state that hollow core units should normally be designed as simply supported, although the design and detailing of the connections may involve restraining effects. Measures particularly important with large wall loads are to be taken to reduce the restraining effects to an acceptable level. If not considered, the actual state of stresses affected by the restraint may cause cracks in the top of the hollow core units. A deep crack will result in a remarkable reduction of the shear capacity of the hollow core unit.

These prestressed hollow core units are further subjected to high shear loads. High eccentricities on the load-bearing wall would thus further reduce the shear load capacity of the prestressed units, something that has not been observed.

The more refined methods of analysis, unlike the initial simplified method under highly loaded hollow core units, appear not to be giving realistic results, as they are being disputed in practice. Note has to be taken of any possible restraint achieved when adjacent terraced constructions are completed, which as noted in Fig 2, may also not be at the same level. se

REFERENCES

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