CADS A3D MAX

How to model frames with masonry infilled panels

Version: 1.0

Revision history

Date	Version	Description	Author
02/03/2015	1.0	Draft for review	RCH

1 Introduction

Ever since fully framed building structures were pioneered in the nineteenth century, designers have been infilling selected frames with brickwork or blockwork masonry. Usually the infilling has served primarily as external cladding or internal compartmentation but competent engineers have always been aware of its potential function as bracing and frequently relied on it either implicitly or explicitly. The stiffness and strength of infilled frames has been a popular research topic with a large catalogue of theses and published papers. It is therefore surprising that very little design guidance is available in British or European standards. The subject seems to have fallen into an `information hole' between the steel and reinforced concrete codes and the structural masonry codes.

In recent decades the trend to tall buildings, open-plan offices and lightweight moveable partitioning has led to designers concentrating resistance to lateral loads into steel bracing bays or reinforced concrete shear walls and stair/lift cores. Nevertheless masonry infilled frames still provide an economic alternative for low and medium rise structures and it is therefore important to have structural analysis and design guidance for this method of construction. Fortunately a relatively simple model and method was published by Hendry[1] based on the original `equivalent strut' proposed by Polyakov[2] and developed by Holmes[3] , Stafford-Smith[4] and others. This has been adopted as the basis of the the following guidance for users of CADS A3D MAX.

This guide has been prepared with appropriate professional engineering logic and care but it has no official status and its interpretation and application is the responsibility of the user.

2 Modelling masonry infilled frames in A3D MAX

2.1 The equivalent diagonal strut and modes of failure

This method is based on the premise that the contribution of a masonry panel infilling a structural frame may be represented by equivalent diagonal bracing as illustrated in fig 1.

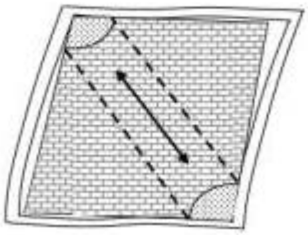


Fig 1 Corner crushing

Fig 1 shows a frame restrained against sway by the infill acting as compression brace. The diagram shows corner crushing which is a common mode of failure in tests.

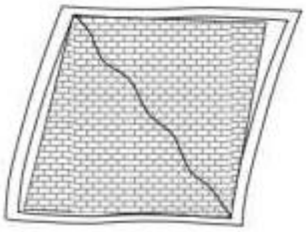


Fig 2 Diagonal cracking

Diagonal cracking may occur in some circumstances if the infill has high crushing and shear strength. This is less common.

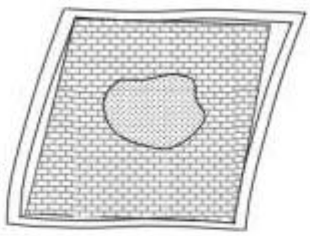


Fig 3 Compression buckling

Out-of-plane compression buckling may occur if the infill is so thin relative to the panel size that second order effects become significant.

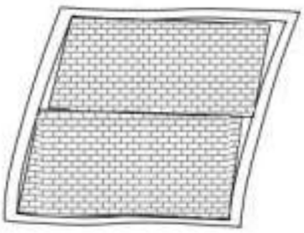


Fig 4 Shear/sliding

Fig 4 shows another common mode of failure – shear/sliding which may be horizontal as shown or stepped. Shear failure of the masonry does not cause collapse because the sheared panel jams into the frame so as to provide continued resistance. However it is safe to treat it as an ultimate limit state.

Failure modes 1, 2 and 3 may be checked using the BS 5628 or EN 1996-1-1 methods for members primarily in compression with effective length equal to the diagonal length. Failure mode 4 may be checked using the code methods for in-plane horizontal shear. Obviously the beam and column frame must be made adequate to resist the forces induced by the system.

All of the above modes of failure can be investigated by modelling the infill acting as a diagonal strut in frame analysis. Obviously if the applied horizontal force reverses, the opposite diagonal becomes the compression strut so the infill should be modelled as compression-only cross bracing.

This guide excludes consideration of infill panels with openings. Although there are research papers dealing with infill panels with openings it is considered that designers would not normally rely on these as contributing to the overall stability of a building.

Similarly as masonry walls are relatively easily dismantled in error compared with concrete shear walls and steelwork bracings, it is advisable to rely only on walls which are unlikely to be disturbed during the life of the building eg: fire compartment walls, lift and stair walls. Consideration should be given to building in durable warning labels beneath the finishes to minimise the possibility of unauthorised or unqualified alterations. Construction drawings should be clearly labelled to show essential infill wall panels and clear instructions given as to the latest stage at which they must be installed. The temporary stability and wind resistance of the frame prior to installation of bracing panels should be carefully considered.

2.2 Properties of the equivalent diagonal strut

2.2.1 Properties for frame analysis

For the purposes of frame analysis it is necessary to define the properties of the equivalent strut.

- E_w Modulus of elasticity. The E value may be obtained from the relevant structural masonry codes of practice eg: BS 5628-2 Annex C; BS EN 1996-1-1 clause 3.7.2. and UK NA.2.9. Alternatively a value may be obtained for selected masonry using CADS Wallpanel MAX.
- B Width of section is the wall panel thickness t_w.
- D Depth of the section is the width of the equivalent strut w. A method for calculating w based on stiffness/contact theory is given by Hendry[1] and summarised in section 3.1. However a conservative first estimate is given by:

  $$w = \text{panel diagonal length}/10$$

- End condition. The member should be considered as pin-ended.
- Directionality. The member should be considered as compression-only (CO).
- Location. If the infill wall is built up tight to the beam above it, the CO diagonals may for simplicity connect diagonally opposite nodes/joints of each panel. Alternatively a more elaborate model with offset nodes may be adopted as discussed in section 3.3. In the special case whereby the infill wall is not built tight to the beam above additional nodes should be introduced just below the

main nodes in the columns to connect the diagonals and the effective width of the strut should be calculated to allow for lack of beam contact. In this case the possibility of a sliding failure between the wall panel and columns should be considered. However this option is not considered further in this document.

- Self weight For reasons related to the analysis procedure, A3D MAX ignores the self weight of members specified as compression-only (or tension-only) This is not a problem because we do not want the weight of the `equivalent strut' to be included. We want the self weight of the whole panel to be applied to the frame. This can be achieved either by applying relevant loads to the beam below each panel or by creating loading panels. These should be specified as `non-rigid'. Self weight of a panel is applied as an in-plane dead load with `bearing' load distribution.

- Wind loads In order to apply lateral wind pressures and suctions to the panels it is recommended to create non-rigid loading panels which can also be used for self weight as noted above.

2.2.2 Properties for design checking

Following frame analysis the wall panel should be checked for compliance with the relevant code of practice for structural masonry. The principal checks are:

- Compression resistance of the equivalent masonry strut. This requires the characteristic compressive strength of the masonry as given by eg: BS 5628-1 clause 19 or BS EN 1996-1-1 lause 3.6.1. This is then used to calculate the compression resistance using eg: BS 5628-1 clause 28 and Annex B or BS EN 1996-1-1 clauses 5.5.1 and 6.1.2.
- Shear resistance at the mid height of the panel. This requires the characteristic shear strength as given by BS 5628-1 clause 21 or BS EN 1996-1-1 clause 3.6.2. The BS EN 1996-1-1 calculation is given in clause 6.2.

Further details of these design checks and the relevant properties of the wall panel are discussed in section 3 of this guide and a worked example is given in section 4.

3 Summary of Equivalent strut calculations

The following text is a summary and interpretation of that given by Hendry[1] adapted for use with BS EN 1996-1-1.

3.1 Effective width of equivalent strut

At an early stage of in-plane loading of an infilled frame, cracks open between the frame and the infill except in the vicinity of the loaded corners. The contact lengths at the corners are governed by the relative stiffnesses of the frame and the infill masonry. Using the theory of beams on elastic foundation and assuming a rigid jointed frame, the relative stiffness of the column and wall is expressed by the parameter λ_c

$$\lambda_c = [E_w \,.\, t \,.\, \text{Sin}2\theta \,/\, (4 \,.\, E_c \,.\, I_c \,.\, H)]^{0.25}$$

where:

E_w = the modulus of elasticity of the wall masonry in kN/mm^2

E_c = the modulus of elasticity of the column in kN/mm^2

t = the thickness of the wall in mm

θ = $\tan^{-1}(H/L)$

H = height of the panel in mm

L = length of the panel in mm

The contact length is then given by:

$W_c = 0.5.\pi/\lambda_c$ in mm

Similarly for the beam/wall interface:

$$\lambda_b = [E_w \,.\, t \,.\, \text{Sin}2\theta \,/\, (4 \,.\, E_b \,.\, I_b \,.\, L)]^{0.25}$$

where:

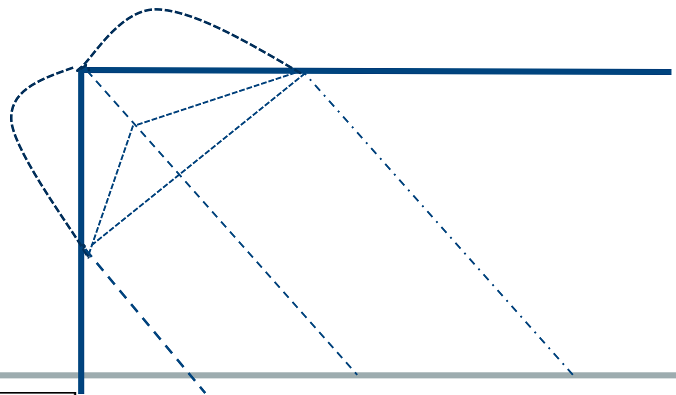
E_b = the modulus of elasticity of the beam in kN/mm^2

The contact length is then given by:

$W_b = 0.5.\pi/\lambda_b$ in mm

Fig 3.1 shows approximate contact stress profiles, and the linear idealisation of compressive stresses in the equivalent strut according to Hendry[1] :-

Fig 3.1



Overall width of strut	w_o	=	$[w_c^2 + w_b^2]^{0.5}$	mm

Assuming that stress distribution is approximately linear/triangular:

Effective width of strut with beam and column contact	w	=	$0.5.\ [w_c^2 + w_b^2]^{0.5}$	mm
Or if there is no beam contact:	w	=	$0.5\ .\ w_c\ .\ \mathrm{Cos}\ \theta$	mm

If the above seems somewhat idealised especially as it makes no allowance for lack of fit and other workmanship issues, some reassurance may be gained from the researchers' reports of good correlation with experimental results.

3.2 Masonry check calculation

If the force in the strut given by frame analysis at the ultimate limit state is F_a kN

Shear force on panel bed joints	F_h	=	$F_a\ .\ \mathrm{Cos}\ \theta$	kN
Normal force on panel bed joints	F_n	=	$F_a\ .\ \mathrm{Sin}\ \theta$	kN

Shear failure surfaces extend across the full length of the panel so find the length in compression:

Taking moments about leeward side of panel at mid-height:

Overturning moment of horizontal force component	M_o	=	$F_h\ .\ H/2$	kNmm
Restoring moment of vertical force component	M_{rv}	=	$F_n\ .\ L$	kNmm
Restoring moment of panel weight above mid height	M_{rw}	=	$W_w\ .\ L/4$	kNmm

Where W_w is the weight of the wall panel.

Position of centroid of loads from leeward edge:

$X = (M_{rv} + M_{rw} - M_o)/(F_n + W_w/2)$ mm

Length in compression L_c = Lesser 3X and L mm

Following BS EN 1996-1-1 clause 6.2 and UK NA for shear resistance at wall panel mid height:

Design compressive stress at mid height $\sigma_d = 1000\ .\ (F_n + W_w/2)/(L_c\ .\ t)$ N/mm²

Characteristic shear strength from clause 3.6.2

f_{vk} = Lesser of: $f_{vk0} + 0.4\ \sigma_d$ and $0.065 f_b$ N/mm²

Where: f_b = normalised compressive strength of the masonry *units*. N/mm²

f_{vk0} = characteristic initial shear strength N/mm²

Design shear resistance: $V_{Rd} = f_{vk}\ .\ L_c\ .t/(1000\ .\ \gamma_M)$ kN

Where: γ_M = partial safety factor for shear resistance

From UK NA table NA.1 γ_M = 2.5 -

Design shear load $V_{Ed} = F_h$ kN

Following BS EN 1996-1-1 clause 6.1.2.1 for compressive resistance of the equivalent diagonal strut:-

Design compression load	N_{Ed}	=	F_a	kN
Design compressive resistance	N_{Rd}	=	$\varphi.k_a.w.t.f_k/(1000.\gamma_M)$	kN
Where:	γ_M	=	partial safety factor for compression resistance	
Eg: from UK NA table NA.1 for category 1 units execution class 2	γ_M	=	2.7	-
small area reduction factor:	k_a	=	lesser of: 1.0 and 0.7 + 3A	-
where:	A	=	$w.t/10^6$	m^2

From clause 6.1.2.2 reduction factor for slenderness and eccentricity:

	ϕ	=	$1 - 2e_{mk}/t$	-
where: total eccentricity at mid height				
	e_{mk}	=	Greater: $e_m + e_k$ and 0.05t	mm
where: eccentricity due to loads	e_m	=	$e_{me} + e_{hm} + e_{init}$	mm
where: eccentricity at mid height due to top and bottom moments				
	e_{me}	=	M_{mmd}/N_{md}	mm
moment at mid height due to top and bottom moments			M_{mmd}	kNmm
axial force at mid height			N_{md}	kN
eccentricity at mid height due to lateral loads eg wind			e_{hm}	mm
initial eccentricity (from clause 5.5.1.1)	e_{init}	=	$h_{ef}/450$	mm
effective length/height	h_{ef}	=	$[L^2 + H^2]^{0.5}$	mm
eccentricity due to creep e_k	=		$0.002.\ \varphi_\infty\ .\ (H_{ef}/t)\ .\ [t\ .\ e_m]^{0.5}$	mm
where: final creep coefficient	φ_∞		given in UK NA table NA.7	-

The above expressions from BS EN 1996-1-1 cover the general case.

For infill panels:	e_{me}	=	0	mm
According to the UK NA, if H_{ef}/t <= 27,	e_k	=	0	mm
If there is no co-existent lateral wind load on the panel:	e_{hm}	=	0	mm

3.3 Effect on frame members

The interaction between the infill masonry and the frame members at the corners implies shear forces acting on the columns and beams. If the members modelling the equivalent struts connect direct to the nodes at the beam/column intersection, these shear forces will not appear in the member effects. This can be dealt with by manual intervention in the design checks. Alternatively the equivalent strut may be split into two each connecting to intermediate nodes in the beams and columns. Relevant shear and axial effects will then appear in the member effects and can be taken into account in the checking software.

4 Worked example

Note that this example only considers lateral stability in the direction of the Z axis. Obviously stability in the X direction must also be ensured.

4.1 Unbraced frame without masonry panels

Fig 4.1 shows the A3D MAX stick model of a simple four storey cast insitu reinforced concrete frame.

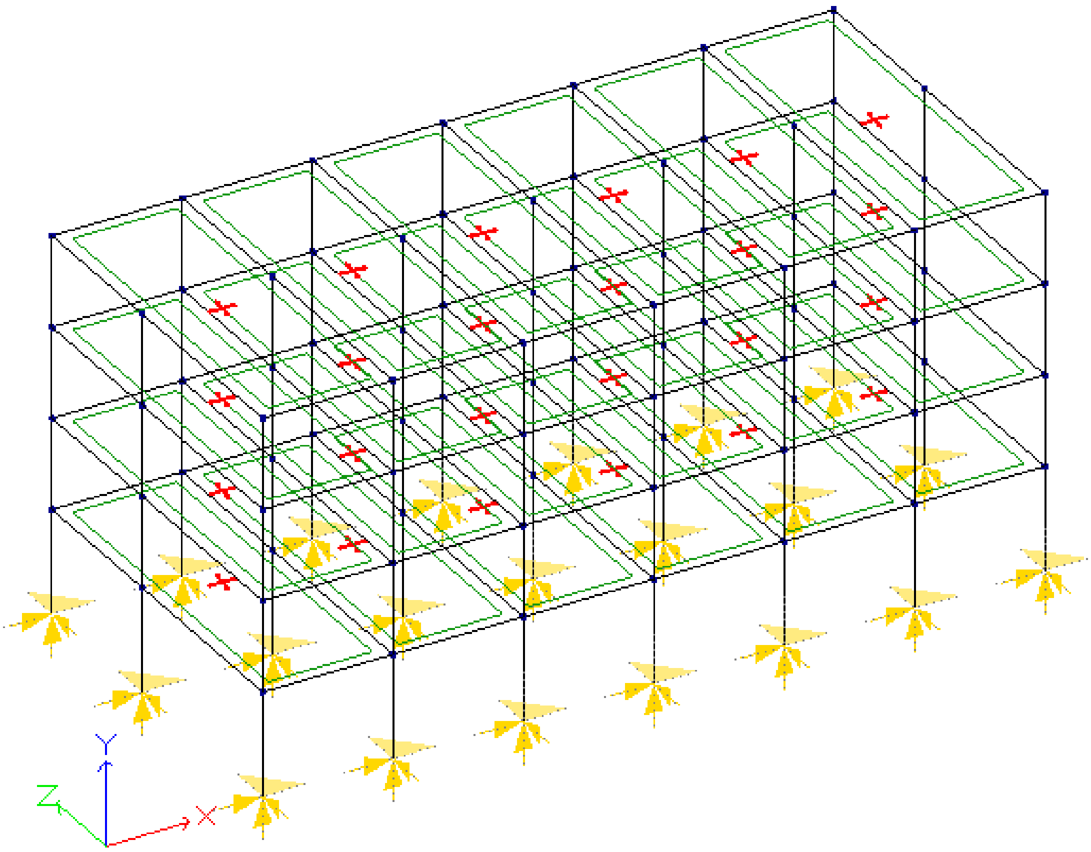


Fig 4.1

The frame has seven bays of 5m, two spans of 8m and 6m, three upper storeys of 3.5 m and ground storey 4.0 m. Columns are 300 x 300 mm section. Main beams are 500 mm overall depth x 300 mm. Edge beams are 500 mm overall depth x 300 mm width except the edge beams parallel to Z axis are 200 mm wide. The roof and floor slabs are 200 mm thick considered as providing rigid horizontal diaphragms. The frame could also have been constructed in steelwork.

The frame is subjected to dead and imposed floor and roof loads applied as panel area loads. Wind and notional horizontal loads are applied as point loads at the joints but could be applied as panel area loads. The frame is first considered as unbraced relying on the rigidity of the beam to column connections for lateral stability.

Fig 4.2 shows the bending moment graphics for the ULS load combination 1.35 x dead + 1.05 x imposed + 1.5 x Zwind. This is a Eurocode load combination with wind as the leading variable and dead load acting unfavourably but the equivalent BS 8110/BS 5950 load combination 1.2(dead + imposed + Z wind) would give similar effects. Maximum moment in main beam at first floor is 345 kNm. Maximum moment in internal columns lowest storey is 108 kNm. Note the reversed moments in the windward (near) first storey columns.

Fig 4.3 shows the deflection graphics for the equivalent serviceability load combination: 1.0 (dead + imposed + wind Z). The deflection at roof level is 40 mm.

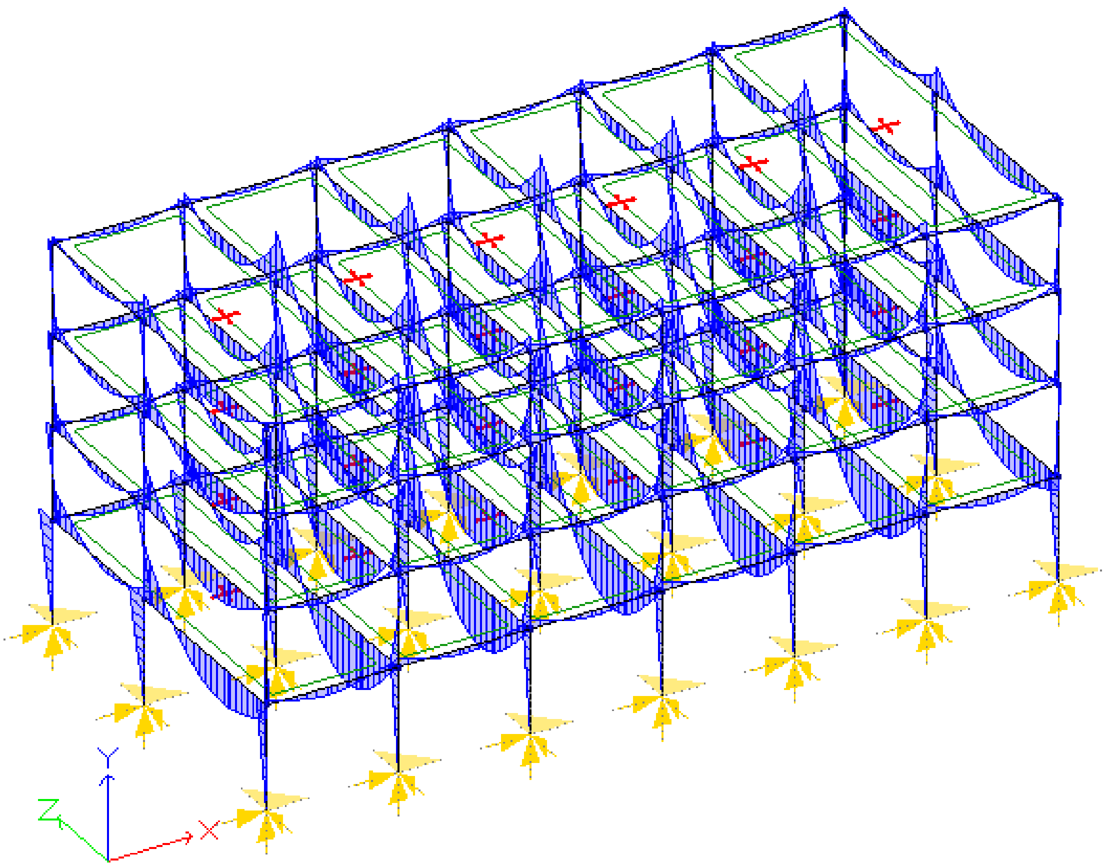


Fig 4.2 Moments in frame for ULS load combination 1.35xdead + 1.05ximposed + 1.5xZwind.

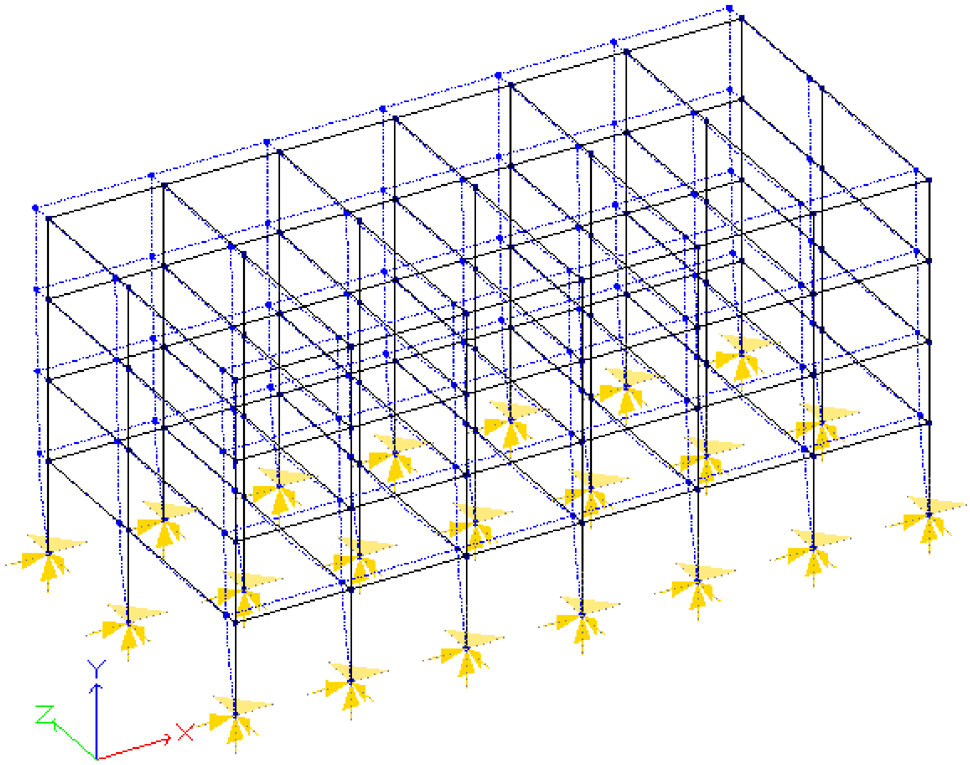


Fig 4.3 Deflection of unbraced frame under SLS load combination 1.0(dead + imposed + wind Z)

4.2 Frame with masonry infill panels in end frames

After analysing the unbraced frame it is now proposed to add masonry infill panels to the end walls in the smaller (6.000 m) spans.

It is proposed to use 215 mm thick solid dense concrete blockwork of 7.3 N/mm^2 unit strength laid `flat' in general purpose mortar of strength class M4. Using CADS Wallpanel MAX or BS EN 1996-1-1 itself the following properties are obtained:

Density as laid:		19.4	kN/m^3
Normalised unit strength	f_b	10.1	N/mm^2
Masonry characteristic compressive strength	f_k	3.83	N/mm^2
Modulus of elasticity (short term)	E_w	3.83	kN/mm^2
Characteristic initial shear strength	f_{kvo}	0.15	N/mm^2

First calculate the effective width of the equivalent struts following section 3.1:-

The calculation for the lowest storey panel is shown here. Upper storeys are similar.

E_w	=	modulus of elasticity of the wall masonry	3.83	kN/mm^2
E_c	=	modulus of elasticity of the column in C32/40 concrete	28.0	kN/mm^2
t	=	the thickness of the wall	215	mm

H	=	height of the panel	4000 - 500mm	3500	mm
L	=	length of the panel in mm	6000 – 300	5700	mm
θ	=	$\tan^{-1}(H/L)$		31.6	degrees
L_d	=	Diagonal length = $[L^2 + H^2]^{0.5}$		6689	mm
I_c	=	$300^4/12$		675×10^6	mm^4
I_b	=	$200.500^3/12$		3125×10^6	mm^4

Wall/column relative stiffness parameter:

λ_c	=	$[E_w . t . \sin 2\theta / (4 . E_c . I_c . H)]^{0.25}$		
	=	$[3.83 \times 215 \times 0.892/(4 \times 28 \times 675 \times 10^6 \times 3500]^{0.25}$	0.00129	mm^{-1}

Column contact length: w_c = $0.5.\pi/ \lambda_c$ — 1216 mm

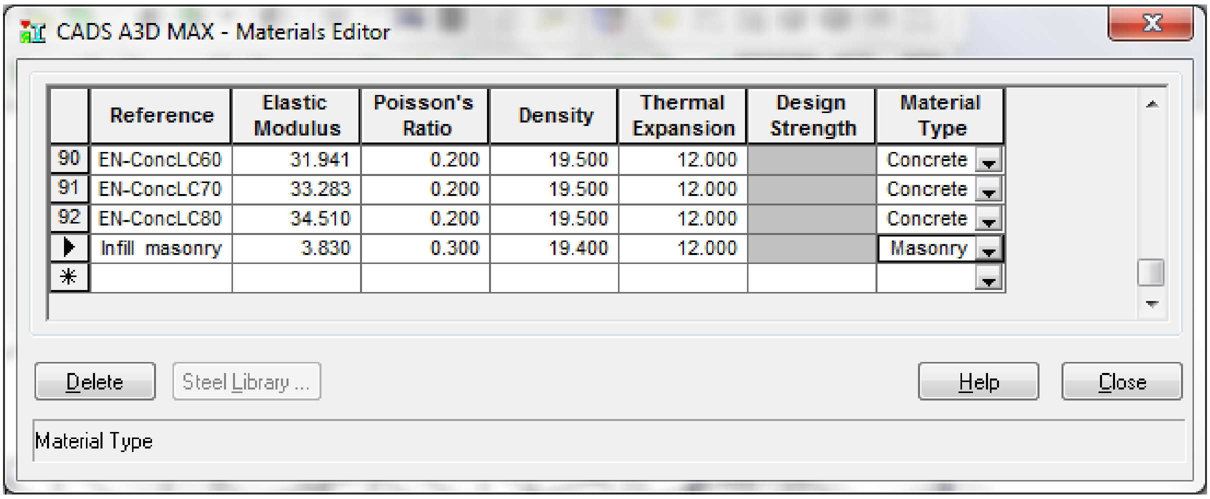
Wall/beam relative stiffness parameter:

λ_b	=	$[E_w . t . \sin 2\theta / (4 . E_b . I_b . L)]^{0.25}$		
	=	$[3.83 \times 215\text{x } 0.892/(4 \times 28 \times 3125 \times 10^6 \times 5700]^{0.25}$	0.000779	mm^{-1}

Beam contact length	w_b	=	$0.5.\pi/ \lambda_b$	2016	mm
Effective width of strut	w	=	$0.5. [w_c^2 + w_b^2]^{0.5}$	1177	mm
Magnitude check			$L_d/10$	669	mm

4.2.1 Analysis with single equivalent strut system

In the A3D MAX model a user defined material type was created:-



	Reference	Elastic Modulus	Poisson's Ratio	Density	Thermal Expansion	Design Strength	Material Type
90	EN-ConcLC60	31.941	0.200	19.500	12.000		Concrete
91	EN-ConcLC70	33.283	0.200	19.500	12.000		Concrete
92	EN-ConcLC80	34.510	0.200	19.500	12.000		Concrete
▶	Infill masonry	3.830	0.300	19.400	12.000		Masonry
*							

Fig 4.4 Materials editor showing user defined material type `Infill masonry'.

As noted in 2.2.1 above, the density is not actually used in compression-only members.

A member type was created for the equivalent strut as shown in fig 4.5.

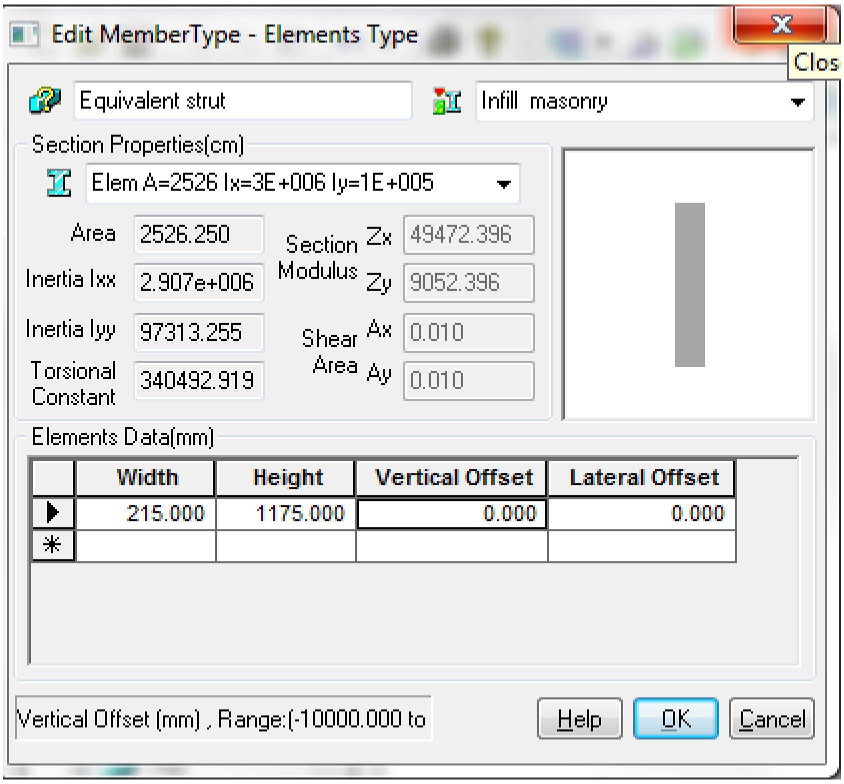


Fig 4.5 User inputs for equivalent strut member type using the `elements' option.

The equivalent strut member type was selected in *Quick member* with pinned ends specified as shown in fig 4.6.

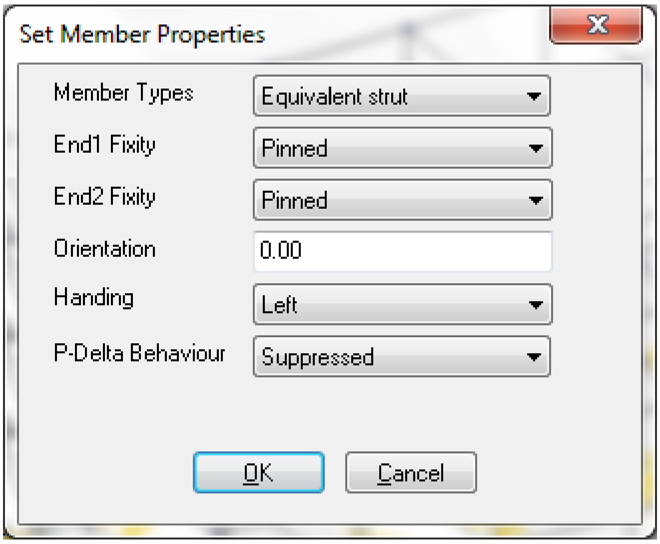


Fig 4.6 Quick member selection.

The equivalent strut cross braces were applied to the frame model to produce the arrangement shown in fig 4.7. All the bracing members were selected and defined as `compression-only' members using the *Member attributes > General* dialog.

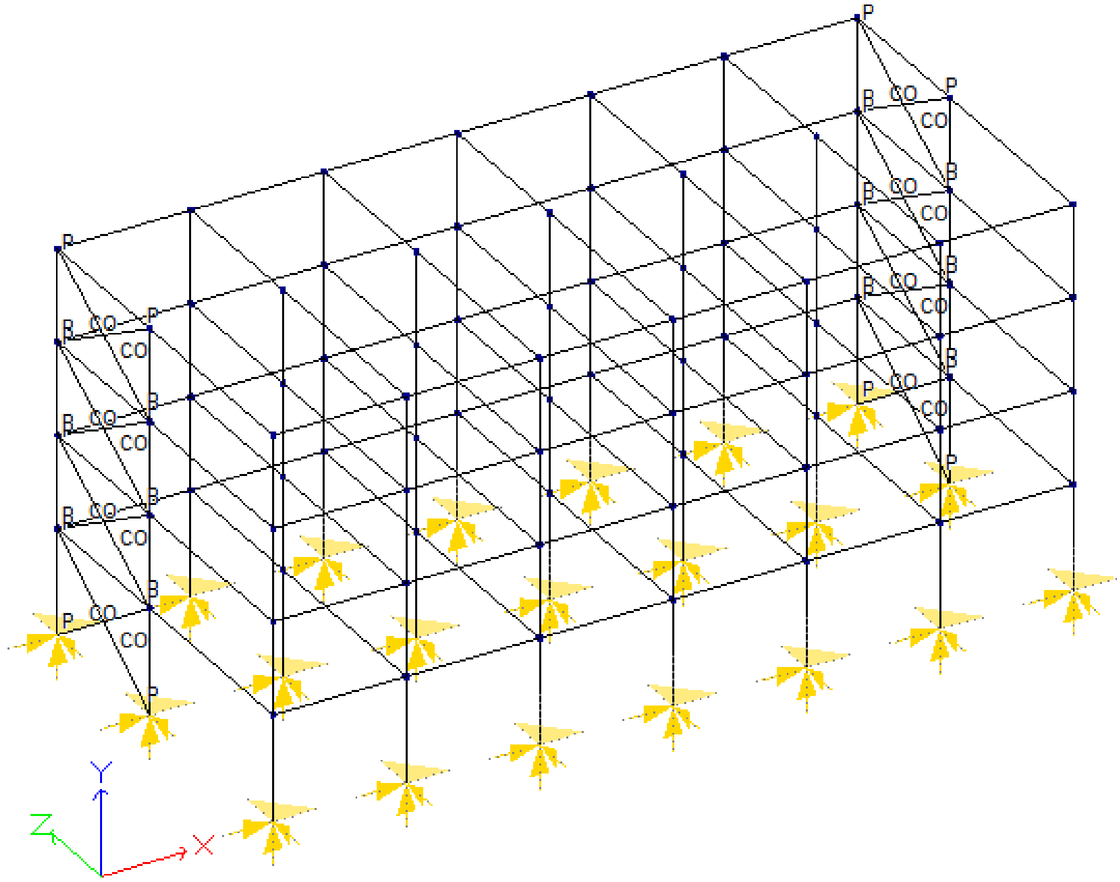


Fig 4.7 Stick model of frame after adding equivalent strut cross bracing.

Before analysis open the *Calculation options* dialog and check that *Tension/compression-only* behaviour is enabled.

Analysis results show horizontal deflection 3.5 mm at roof level due to SLS load combination 1.0(dead + imposed +Zwind). This may be compared with the corresponding value for the unbraced frame: 40 mm.

Fig 4.8 shows the axial force graphics for the ULS load combination 1.35 x dead + 1.05 x imposed + 1.5 x Zwind. Maximum force in the lowest storey diagonals is 258 kN.

Fig 4.9 shows the bending moment graphics for the ULS load combination 1.35 x dead + 1.05 x imposed + 1.5 x Zwind. Maximum moment in main beams at first floor is 286 kNm compared with 345 kNm for the unbraced frame. Maximum moment in internal columns of the lowest storey is 36 kNm compared with 108 kNm for the unbraced frame. Note that in contrast to the unbraced frame the moments in the windward (near) first storey columns are not reversed. In fact the wind loads in the Z direction are almost entirely carried by the infill masonry.

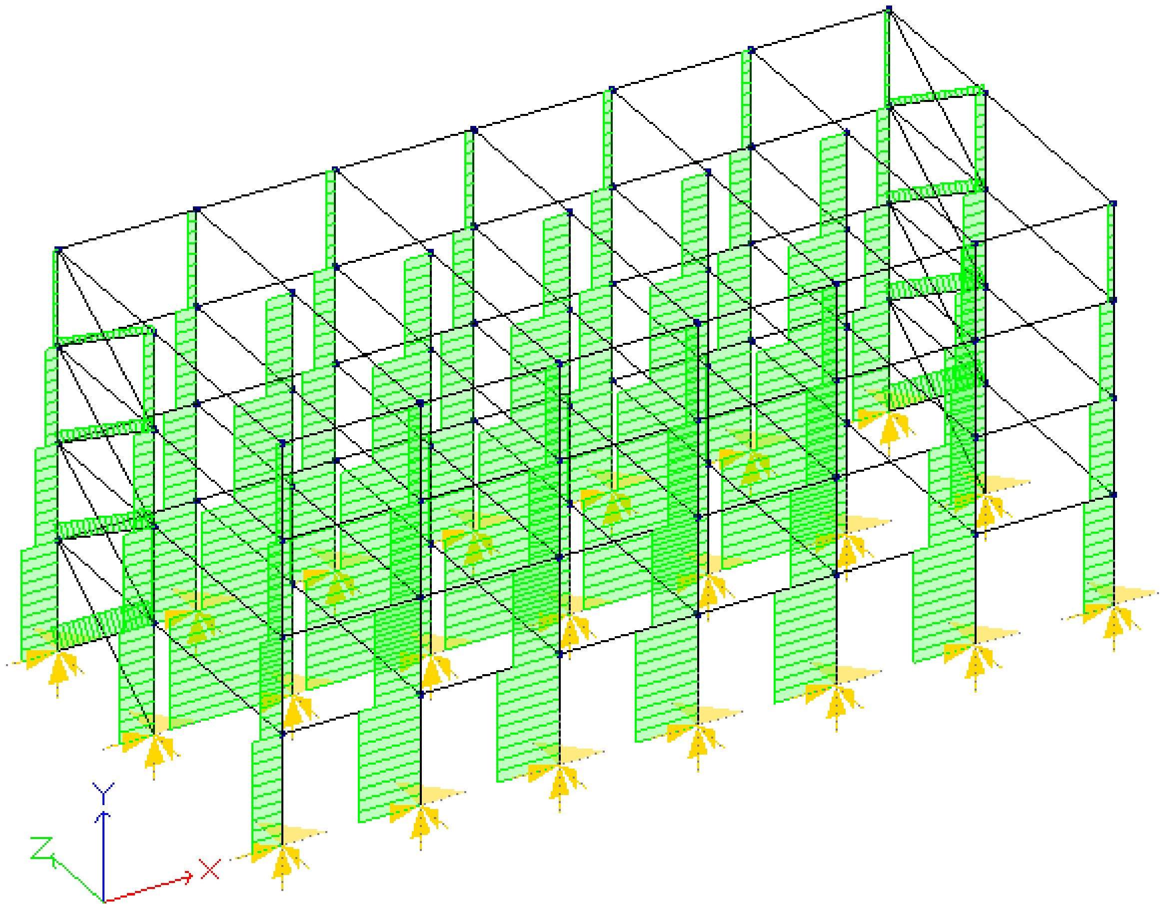


Fig 4.8 Stick model of frame showing axial force graphics for ULS load combination 1.35 x dead + 1.05 x imposed + 1.5 x Zwind.

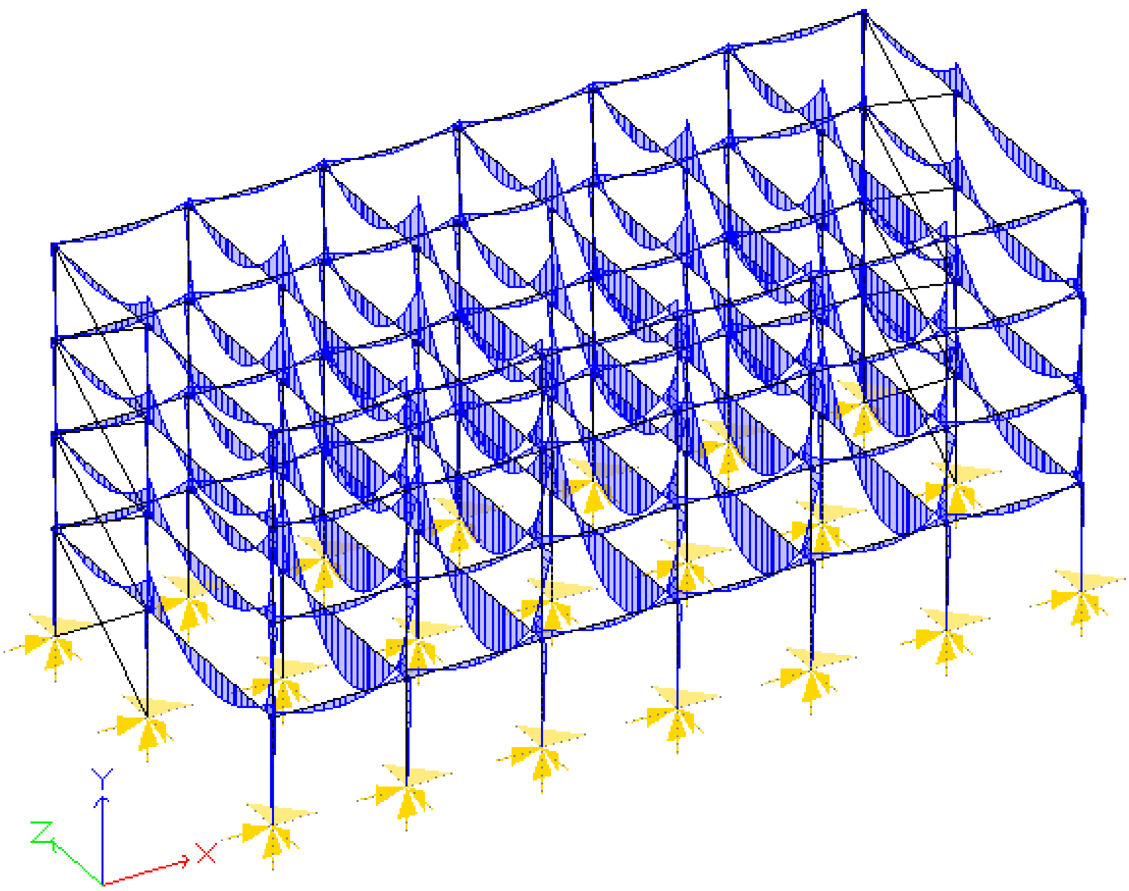


Fig 4.9 Stick model of frame showing moment graphics for ULS load combination 1.35 x dead + 1.05 x imposed + 1.5 x Zwind.

4.2.2 Analysis with twin equivalent strut system

As mentioned in section 3.3, the compression force in the equivalent strut is applied to the beams and columns over finite corner contact lengths. This induces shear forces and bending moments in the members. A reasonable method of modelling this is to divide each equivalent strut into two portions extending between nodes positioned at one-third the relevant contact lengths from the faces of the columns and beams at the corners of the panel. The section depths of the members representing the equivalent strut are half the values adopted for the single strut system. Fig 4.10 shows the stick model with the axial force graphics for ULS load combination 1.35 x dead + 1.05 x imposed + 1.5 x Zwind superimposed. The sum of the strut forces in the lower compression diagonals is 251.8 kN compared with 258 kN in the simpler single strut model. This is an insignificant difference but the effect on the moments and shears in the beams and columns is greater as illustrated in fig 4.11 and 4.12. The deflection at roof level is 4.2 mm compared with 3.5 mm for the single strut system.

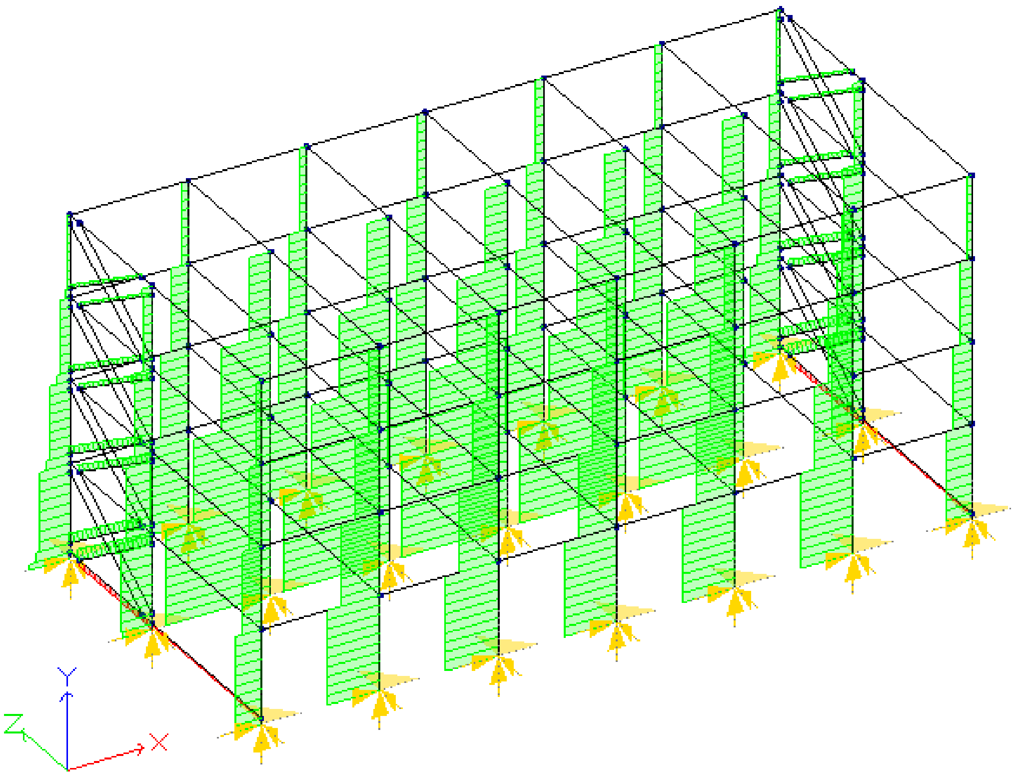


Fig 4.10 Frame with twin equivalent strut system with axial force diagram superimposed.

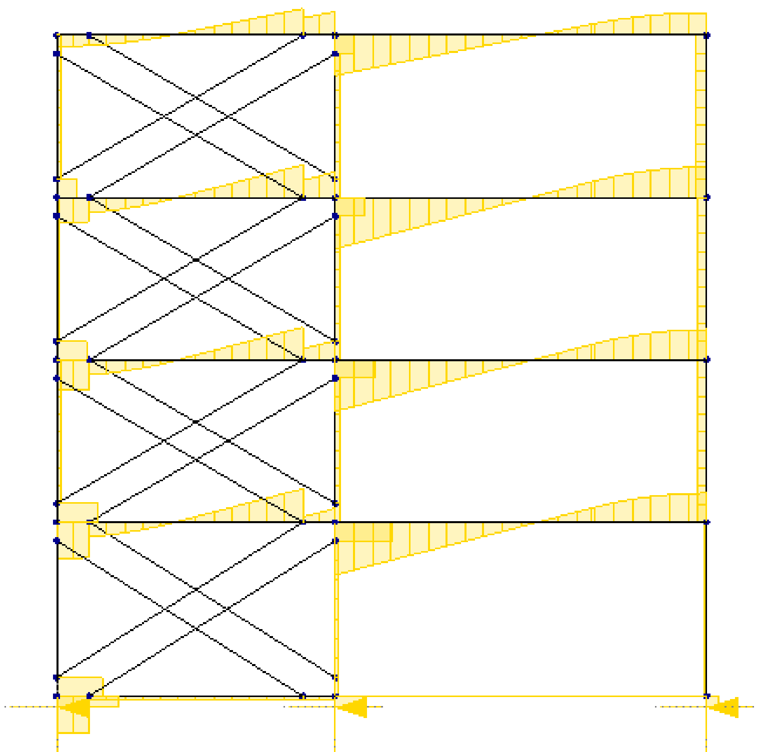


Fig 4.11 End frame shear force diagram

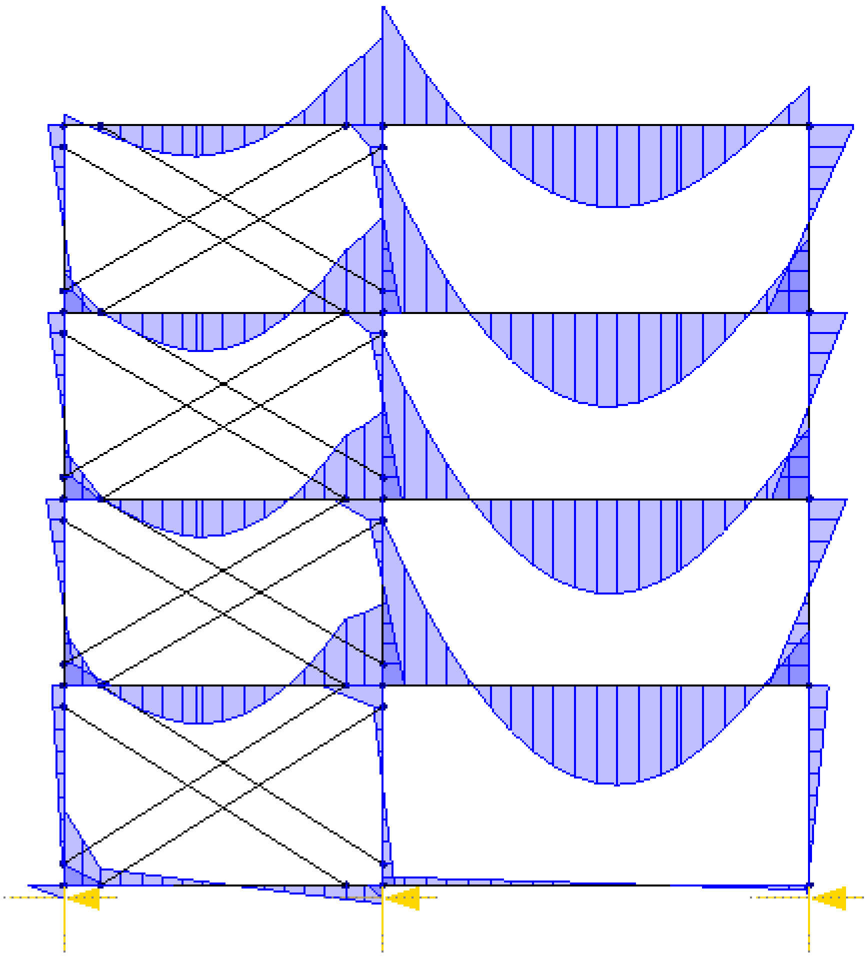


Fig 4.12 End frame moment diagram.

4.2.3 Verification of masonry panels

Having established the effect of the infilled panels on the behaviour of the frame it is necessary to check whether the masonry panels have adequate resistance to the applied forces following the procedure given in section 3.2:-

Force in the equivalent strut given by frame analysis at the ultimate limit state:

F_a = 258 kN

$\theta = \tan^{-1}(H/L) = 31.6$ degrees

Shear force on panel bed joints $F_h = F_a \cdot \cos\theta = 219.7$ kN

Normal force on panel bed joints $F_n = F_a \cdot \sin\theta = 135.2$ kN

Shear failure surfaces extend across the full length of the panel so find the length in compression:

Taking moments about leeward side of panel at mid-height:

Overturning moment of horizontal force component:

$M_o = F_h \cdot H/2 = 384475$ kNmm

Restoring moment of vertical force component

M_{rv} = $F_n . L$ = 770640 kNmm

Weight of wall panel W_w = 5.7 x 3.5 x 0.215 x 19.4 = 83.2 kN

Restoring moment of panel weight above mid height

M_{rw} = $W_w . L/4$ 118560 kNmm

Position of centroid of loads from leeward edge:

X = $(M_{rv} + M_{rw} – M_o)/(F_n + W_w/2)$ = 2855 mm

Length in compression L_c = Lesser 3X and L

= Lesser 8564 and 5700 5700 mm

The full length is in compression

Following BS EN 1996-1-1 clause 6.2 and UK NA for shear resistance at wall panel mid height:

Design compressive stress at mid height :

σ_d = $1000 . (F_n + W_w/2)/(L_c . t)$ = 0.144 N/mm^2

Characteristic shear strength f_{vk} = Lesser of: $f_{vk0} + 0.4\ \sigma_d$ and $0.065 f_b$ N/mm^2

= 0.15 + 0.4 x 0.144 and 0.065 x 10.1 N/mm^2

= lesser: 0.21 and 0.65 = 0.21 N/mm^2

Design shear resistance: V_{Rd} = $f_{vk} . L_c .t/(1000 . \gamma_M)$

= 0.21 x 5700 x 215/(1000 x 2.5)= 102.9 kN

Design shear force V_{Ed} = F_h = 219.7 kN

Shear resistance utilisation U_v = 219.7/102.9 = 2.14 -

Following BS EN 1996-1-1 clause 6.1.2.1 for compressive resistance of the equivalent diagonal strut:-

Section area of equivalent strut: A = $w.t/10^6$

= 1.175 x 0.215 = 0.252 m^2

small area reduction factor: k_a = lesser of: 1.0 and 0.7 + 3A

= 1.0 and 1.458 = 1.0 -

Eccentricity:

Assuming out-of-plane loading is negligible e_{hm} = 0 mm

No applied end moments e_{me} = 0 mm

Effective length of strut = panel diagonal h_{ef} = L_d = 6689 mm

Slenderness h_{ef}/t = 31.1 -

Eccentricity due to creep

Initial eccentricity e_{init} = $h_{ef}/450$ = 14.9 mm

Total eccentricity due to loads e_m = $e_{me} + e_{hm} + e_{init}$ = 14.9 mm

From UK NA table NA.7 creep coefficient φ_∞ = 1.5 -

Eccentricity due to creep e_k = $0.002. \varphi_\infty . (H_{ef}/t) . [t . e_m]^{0.5}$

	=	$0.002 \times 1.5 \times 31.1 \times [215 \times 14.9]^{0.5}$	=	5.3	mm
total eccentricity at mid height e_{mk}	=	Greater: $e_m + e_k$ and 0.05t			
	=	Greater: 20.2 and 10.75	=	20.2	mm

reduction factor for slenderness and eccentricity:

ϕ	=	$1 - 2e_{mk}/t$	=	0.812	-
Design compressive resistance N_{Rd}	=	$\varphi.k_a.w.t.f_k/(1000.\gamma_M)$			
	=	$0.812 \times 1.0 \times 1177 \times 215 \times 3.83/(1000 \times 2.7)$		291.5	kN
Design compression load N_{Ed}	=	F_a	=	258	kN
Compressive resistance utilisation U_c	=	258/291.5	=	0.89	-

Conclusion

The above masonry design checks indicate that the lowest storey equivalent struts have adequate compression resistance but the shear resistance of the masonry infill panel is inadequate by a factor of 2.14. Indications are that infilling both panels at each end of the building would provide sufficient shear resistance. This would need to be verified by adding the relevant diagonal members followed by re-analysis and masonry design checks.

Finally the beam and column members of the infilled frames need to be verified for the moments, shears and axial forces produced by the analysis as shown in figs 4.11 and 4.12 using the normal procedures for reinforced concrete or steelwork as appropriate. Attention should also be given to the columns and foundations below the level of the ground floor beams to ensure that these are not the `weakest links in the chain.'

As this analysis and design method for masonry infilled frames is not yet supported by Eurocodes or equivalent authoritative guidance it would be prudent not to pursue a `tight' design.

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5 References

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4	Stafford-Smith, B	*Lateral stiffness of infilled frames,* J Structural division ASCE vol 88 ST6 (1962) pp183-9.

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