Ecobrix Structural Design Guidance

The following design guidance has been produced to assist Qualified Structural Engineers in carrying out the design of structures where Ecobrix blocks are used as permanent structural formwork. This guidance is provided in good faith, but as ever, the project structural engineer remains responsible for ensuring that any structures are safe, stable, and durable. This document must not be used as a substitute for project-specific design advice provided by a Qualified Structural Engineer.

Description of the Structural Form of Ecobrix Walls

Ecobrix walls using D300 or D365 blocks comprise two skins of 40mm woodcrete, with a 120mm concrete core and a variable thickness of insulation – 100mm for D300 blocks and 165mm for D365. For the D250 and D170 blocks, there is no insulation. D250 blocks have two layers of 35mm woodcrete, surrounding a 180mm core of concrete, and with D170 blocks the woodcrete is 25mm thick, with 120mm of concrete core.

The format of Ecobrix blocks, when placed in a typical stretcher bond, with courses overlapped by half, results in continuous vertical concrete “columns” at 250mm horizontal centres, linked by 100x100mm concrete “rungs” at 250mm vertical centres. This mesh of vertical “columns” and horizontal “rungs” forms the basis of the structural design of Ecobrix walls. Conservatively, the contribution of the woodcrete is generally disregarded. This is illustrated in the following diagrams – these are based on D300 or D365 blocks, but the effect is similar with either D170 or D250 blocks.

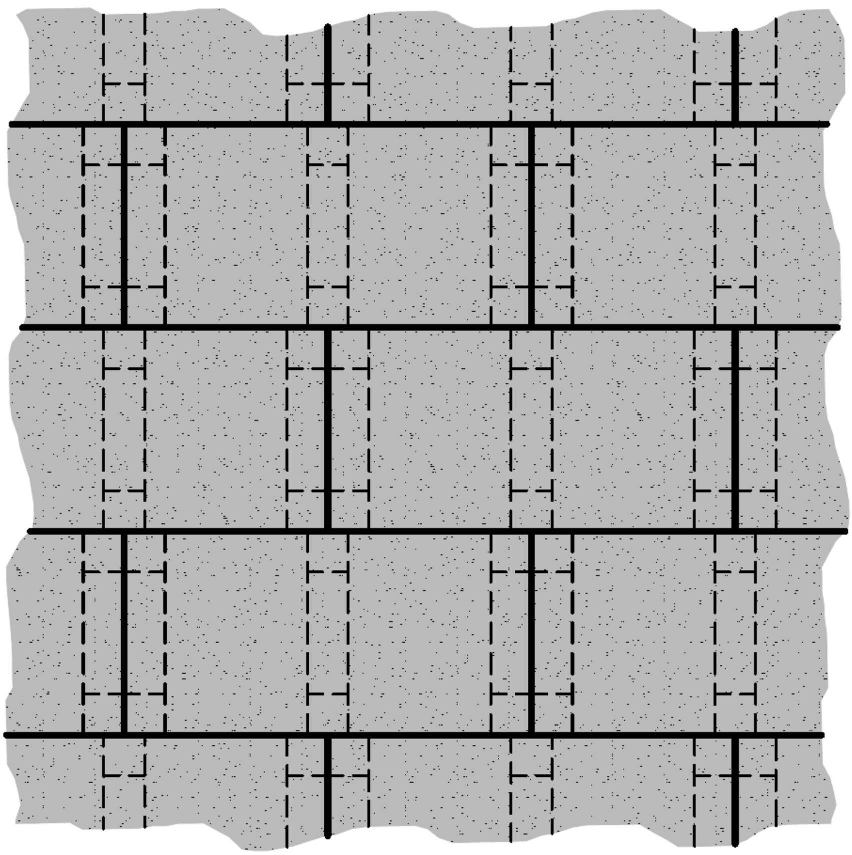

Figure 1. Typical Ecobrix wall with blocks laid in stretcher bond

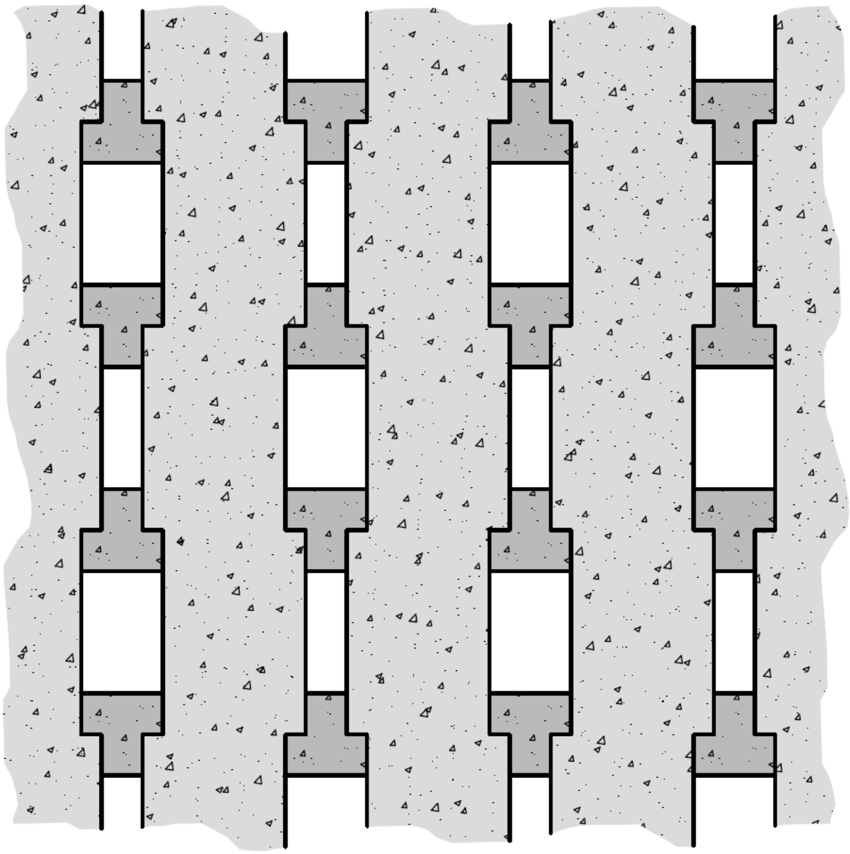

Figure 2. The same wall with woodcrete removed to show the in-situ concrete matrix

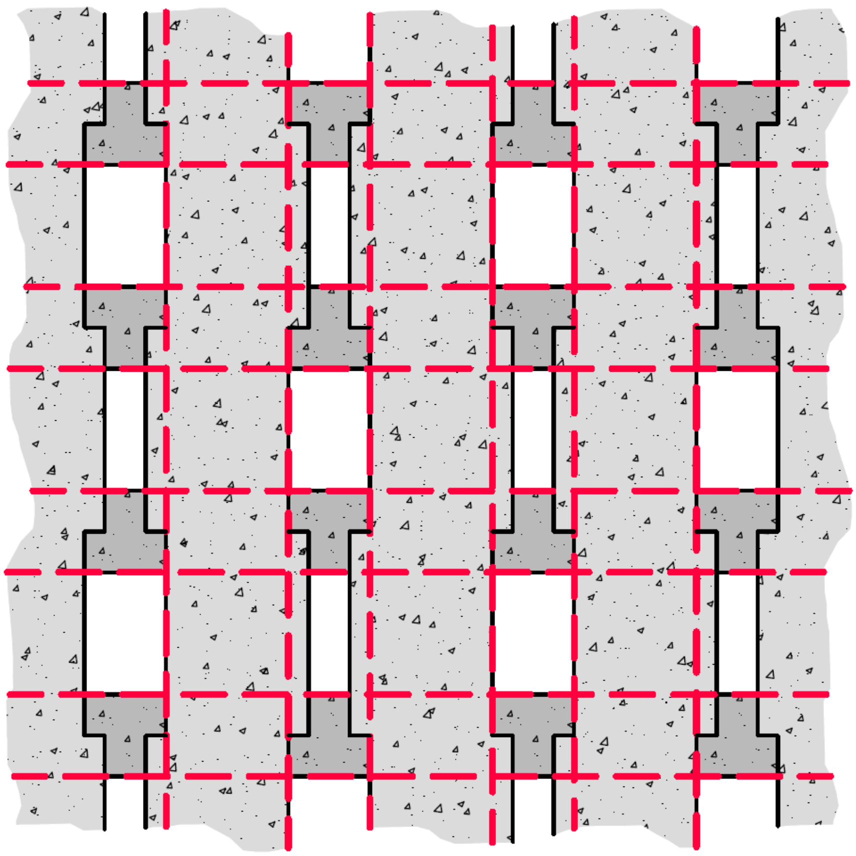

Figure 3. Notional design "Columns" and "Rungs" indicated

While the horizontal “rungs” are always 100x100mm, the dimensions of the continuous vertical columns vary, as shown below:

D300 and D365	150mm wide x 120mm deep	Equivalent to 60% solidity ratio
D170	160mm wide x 120mm deep	Equivalent to 64% solidity ratio
D250	190mm wide x 180mm deep	Equivalent to 76% solidity ratio

It is noted that the cores of vertically adjacent blocks are staggered when stacked in stretcher bond. Considering this, the above column dimensions take account only of the concrete that is aligned vertically, and so conservatively disregards some of the concrete infill (as illustrated in Figure 3).

To achieve the wall strengths described in this report, it is important to ensure that blocks are coursed correctly wherever possible, with blocks overlapped by half (250mm) between courses to achieve a true “stretcher bond”. If this is not carefully adhered to, the continuous vertical concrete columns assumed in much of the structural design will not be achieved, and the strength of the walls may be compromised.

One aspect which should be noted is that traditional masonry walls are typically significantly weaker in failure parallel to bed joints than perpendicular. This is often unhelpful in the structural design of masonry buildings, as wall panels in the majority of buildings tend to be wider than they are tall – i.e. they are “landscape”, rather than “portrait”. As such, they tend to be inclined to span vertically, which unfortunately coincides with the weaker direction of the material. As there are often window or door openings further reducing their ability to span horizontally, the design of external masonry wall panels is often problematic, leading to frequent requirements for wind posts or bed joint reinforcement.

Ecobrix walls do not exhibit this inherent problem, On the contrary, with Ecobrix blocks, the stronger span direction of the wall is vertically, which is consistent with the typical direction of span of masonry panels. In many cases, Ecobrix walls can be designed spanning solely in the vertical direction, neglecting the horizontal span direction.

It should be noted that, due to maximum pour height limits, it is inevitable that construction joints will occur, potentially around mid-height of wall panels. As the design method for Ecobrix walls as described here relies on the tensile strength of the concrete, this could be compromised by horizontal day joints. It is therefore important to ensure that the top surface of concrete is left as rough as possible, and that following pours proceed as soon as practicable, once the pour below has achieved sufficient strength to proceed.

In critical areas, where tensile stresses are expected to occur, reinforcement may be required at construction joints, in line with Eurocode 2 clause 12.9.2.

Simplified Structural Design

For low-rise domestic buildings (up to three storeys), and as a first step in the design of other buildings to be constructed using Ecobrix blocks, we recommend that walls are reviewed against Building Regulations Approved Document A "Structure", in England and Wales, "The Small Buildings Structural Guidance" in Scotland or "Technical Booklet D: 2012 Structure" in Northern Ireland. These documents include simplified rules, whereby in many cases, masonry walls can be assessed as "deemed to satisfy", with little or no further calculation necessary. Please note that the following sections refer to Approved Document A throughout, but this should be read as the appropriate document for projects in Scotland or Northern Ireland.

A cavity wall constructed from two skins of 90mm 7.3N/mm^2 blocks using M4 mortar would be deemed to satisfy the requirements of Approved Document A Table 3*, for external, compartment or separating walls up to 9m wide and 9m high, under conditions A, B and C, as shown on Diagram 9*, with openings and restraints in accordance with the document. For other internal load-bearing walls, Approved Document A requires the wall to be a minimum of 90mm thick, apart from those on the ground floor of a three-storey building, which should be a minimum of 140mm thick blockwork. Note that when referring to wall heights of up to 9m, this refers to the total height of the wall – the storey height is typically limited to 16 times the wall thickness for solid walls.

Calculations are attached which demonstrate that all Ecobrix walls, when constructed using any of the block sizes in stretcher bond, and filled with a minimum of C25/30 concrete, have at least comparable** horizontal and vertical flexural, axial and shear strength to the blockwork cavity wall described above, and often very much greater. It should be noted that these calculations rely solely on the concrete core, neglecting any contribution from the woodcrete, and so are conservative.

As it has been demonstrated that Ecobrix walls have at least comparable strength to the masonry cavity wall described, it is reasonable to conclude that Ecobrix walls can safely be checked on the same basis. This means reviewing all the Approved Document A requirements in relation to floors and roofs supported, panel dimensions, openings and buttresses. If all these requirements are satisfied, no further design checks are needed.

Based on the above, Ecobrix walls for most applications within low-rise domestic buildings can be justified with minimal further calculation required. As an example, an external wall under 2.7m high and 9m wide, with no more than 3 openings, none of which are more than 3m wide. comprising less than 2/3 of the total wall length, with at least 1m between openings, and 500mm piers at each end, should not require detailed calculations, and in most cases will require no vertical reinforcement, with only lintel reinforcement required.

For buildings over three storeys, or walls over 9m high or wide, more detailed design is likely to be required.

***** *Approved Document A Table 3 and Diagram 9 are equivalent to clauses 1.D.6 and 1.D.16 respectively in the "Small Buildings Structural Guidance" for Scotland, and to Table 4C.10 and Diagram 4C.7 in "Technical Booklet D: 2012 Structure" for Northern Ireland. The references are identical in Approved Document A for both England and Wales.*

*** It is noted that in one case, that of a 140mm thick blockwork wall, when checked using Eurocode 6, the flexural strength perpendicular to bed joints is 0.7% higher than the equivalent value for Ecobrix walls. However, as the calculations for Ecobrix walls are inherently conservative, ignoring the contribution of the woodcrete, and as the cavity wall calculation gives a lower value, it is considered reasonable to conclude that Ecobrix walls can be checked using Approved Document A.*

Detailed Structural Design

Detailed structural design of Ecobrix walls can be carried out using Eurocode 2 Section 12 "Plain and Lightly Reinforced Concrete Structures", although this is conservative, as it neglects any contribution from the woodcrete.

Guidance on this can be found in "Design and Construction using Insulating Concrete Formwork" published by The Concrete Society. It should be noted that this document was published in 2007 and does not reflect the changes in subsequent revisions of Eurocode 2. It should also be noted that the document is largely concerned with ICF products with a continuous concrete core, so allowances need to be made for the lack of continuity of the core when using Ecobrix.

Wall Panel Design under Out-of-Plane Lateral Loading

Wall panels under lateral loading can be checked as a grillage of vertical and horizontal plain concrete beams. It should be considered that horizontally, only the 100x100mm "rungs" provide continuity to the concrete, so the capacity of the walls to resist horizontal bending is limited. In most cases it will be simpler to ignore horizontal spanning and design for vertical spanning only.

If the checks noted above are not successful, usually the simplest solution will be to design windposts within the Ecobrix blocks, to break the wall down into panels which are compliant with Approved Document A.

The design of windposts involves reinforcing individual concrete "columns", based on the sizes noted in the table above, to act as beams spanning vertically between floors/roof. These can be designed as reinforced concrete beams to Eurocode 2.

Typically external cladding will provide waterproofing, so concrete cover can be kept to a minimum. Even taking account of this, the effective depth of reinforcement is typically only around 80mm (unless using D250 blocks). Given this, it is deemed impractical to incorporate shear links, as the maximum spacing of links would only be around 60mm. Because of the lattice nature of the wall, transverse redistribution of shear is considered possible, so shear reinforcement may be omitted provided that $V_{Ed} \geq V_{Rd,c}$. It may be the case that two or more adjacent columns will need to be reinforced to satisfy this requirement.

Of course, if including occasional windposts is still not satisfactory, reinforcement can be added at closer centres to enhance the capacity of a panel to span vertically or horizontally, designing as a series of parallel reinforced concrete beams, either 150x120mm (or as appropriate) vertically, or 100x100mm horizontally. As above, shear stress will need to be checked to avoid the need for shear links.

Wall Panel Design under Vertical Loading

As under lateral loads, Ecobrix walls can be checked under vertical loading as plain concrete walls, designed in accordance with Eurocode 2 Section 12. Account needs to be taken of the fact that the concrete is not fully continuous along the length of the wall, and this can be allowed for by reducing the load capacity derived from the calculation to reflect the percentage of vertically continuous concrete – e.g. 60% for D300 or D365 blocks (see table above). Depending on the wall height, loads in excess of 400kN/m can typically be supported, or 1000kN/m for D250 blocks.

Wall Panel Design under In-Plane Loading

Typically, if buttressing walls or piers satisfy the minimum dimensional requirements of Approved Document A, no further checks are required. If this is not the case, or for checks on global stability, walls can be designed as a series of vertical plain or reinforced columns, either cantilevering from foundations, or as part of a sway frame with beams or wall above. In this case, the columns are typically 150mm deep x 120mm wide. This conservatively disregards any composite action between the columns. It is important to ensure that these columns are connected to foundations, beams or other structures capable of resisting the end moments generated, and that reinforcement is detailed in a way to ensure that adequate anchorage is provided.

In extreme situations, it would also be possible to analyse walls as a grillage of vertical and horizontal beams under in-plane loads, either plain or reinforced, provided any transfer of moments between the vertical and horizontal beams can be justified.

Design of Lintels

Lintels are typically formed using face blocks on end, such that the bottom of the lintel has a continuous woodcrete face. For very small openings (e.g. service penetrations), it is possible to use the blocks unaltered, but in most cases, it is necessary to cut out one, two or three of the block webs, to create deeper continuous slots within the blocks. For the different blocks, the possible lintel sizes that can be created are as follows:

Block type	Lintel width	Lintel depth			
		Unaltered	Number of webs removed		
			1	2	3
D300 and D365	120mm	155mm	205mm	405mm	450mm
D170	120mm	160mm	210mm	410mm	455mm
D250	180mm	135mm	310mm	460mm	

Lintels are designed as reinforced concrete beams in accordance with Eurocode 2. In general we would not recommend determining lintel loading using (for example) BS 5977 “Lintels — Part 1: Method for assessment of load”, unless the designer is satisfied that the loading is sufficiently low to be distributed using the 100x100mm concrete “rungs” between the “columns” above.

For the shallower lintel depths, the small effective depth, and resulting maximum link spacing, means that it is not practical to include links. For these lintels, checks should be made to ensure that the applied shear stress is less than the design concrete shear strength, so that links can be omitted. Generally one or two reinforcing bars are provided in the bottom layer only.

In practice, installing a reinforcement cage would be difficult with any webs in place, so links are generally only used where all the webs are removed to create a lintel of 450 to 460mm deep (or less if the blocks are cut down, for example at eaves, or for coursing reasons). For 450 to 460mm lintels, 250mm link centres (i.e. one link per block) are generally practical and comply with the rules on maximum link spacing.

Apart from when using D250 blocks, the 120mm width of the lintel, taking account of reinforcement cover, means that sausage links (shape code 33 to BS 8666) are generally the most practical, using a maximum of 12mm bars as required, although shape code 51 links might be possible for links up to 8mm, subject to satisfying minimum cover requirements. When using shape code 33 links it is often simplest to have only a single bar in any layer in the lintels. Nominal top reinforcement is generally specified to keep the links securely in place.

From experience, lintels above approximately 1.5m clear span often require shear links, and so would then be designed as 450 to 460mm deep. For lintels above 2m span, Eurocode 2 requires that minimum links be installed, so these will always need to be of sufficient depth to include links.

For lintels with no shear links, it is generally acceptable to terminate the face blocks at the face of the opening, with the reinforcement projecting 250mm into the blocks on either side (subject to checks on anchorage). The blocks on either side will generally need to be adapted to allow the lintel reinforcement to extend over the supports.

For lintels requiring shear links, Eurocode 2 indicates that shear reinforcement should continue to the support, which in this case means the concrete core on either side of the opening. To achieve this, we would recommend continuing the vertical face blocks typically 500mm to either side of the opening, which should also allow sufficient projection of the main bars to satisfy anchorage requirements. In this case, holes will need to be made in the bottom face of the end blocks to ensure that concrete can flow into the blocks below. If it is not possible to extend the face blocks this far beyond the face of the opening, additional U or L-bars may be required to satisfy anchorage requirements.

Other beams can be incorporated into Ecobrix walls in a similar way, including cantilevers, with face blocks on end provided for the distance required to create the length of backspan necessary.

Steel Beams or Columns

On rare occasions it is necessary to incorporate steel beams or columns, either to resist wind sway loading, or if the load capacity of reinforced concrete lintels or walls is exceeded. In principle these can be built into face blocks, with webs removed, similar to incorporating reinforcement cages into lintels. In practice, however, as most steel UBs are at least 102mm wide, it would be difficult to ensure that they are fully encased in concrete where the concrete core is only 120mm wide.

Considering this, it is generally preferable to include the steel beam below the blocks. It is not necessary for the steel to support the full width of the blocks, provided that it supports the concrete core (the principal load-bearing element). However, if the full wall width is not supported by the beam, temporary support may be necessary due to the risk of dislodging blocks while pouring concrete. Any torsion induced by eccentric loading of the beam should, of course, be taken account of in the design.

Shear studs or welded rebar may be installed on top of steel beams to ensure the wall and beam are tied together for robustness, or to provide lateral restraint to the steel beam (assuming the wall or other structure can resist the horizontal forces so induced) but we would not recommend taking account of composite behaviour between the beam and concrete wall.

Bearing stresses from steel beams onto the concrete core can be checked based on the requirements for precast concrete bearings within Eurocode 2 Section 10.9.5 "Bearings". This is rarely likely to be a concern.

Robustness and Disproportionate Collapse

One of the very significant benefits of using Ecobrix over conventional masonry is the comparative ease with which horizontal and particularly vertical ties can be incorporated by the addition of reinforcement within the concrete core, to satisfy disproportionate collapse requirements. Reinforcing bars, designed to resist the necessary tie forces, can easily be provided vertically within the "columns" or horizontally within the 100x100mm "rungs", with adequate laps between bars.

Tie bars or straps can also be cast into the core to provide a robust connection to floor or roof structures where required.

For more detail, refer to Approved Document A Section 5 in England and Wales, Section 1.2 of the Domestic Technical Handbook in Scotland, Section 3 of Technical Booklet D:2012 Structure in Northern Ireland, or guidance such as the IStructE "Practical Guide to Structural Robustness and Disproportionate Collapse in Buildings".

Fire Resistance

Ecobrix walls have a four-hour fire rating to BS 476 – refer to the Ecobrix Technical Manual for further information.

Design of Connections of Floors and Roofs to Ecobrix Walls

Concrete floors will typically be connected to Ecobrix walls by simply extending the slab (precast or in situ) over the concrete core, with blocks cut to suit. It is recommended to leave a 20mm gap between the end of precast concrete floor units and the insulation, to allow for concrete to flow down to the voids below. In this situation, the eccentricity created by this will need to be allowed for in the design of the wall below, where appropriate.

The only checks required for the connection will be to ensure that bearing stresses are acceptable, and that slab edge reinforcement is suitably detailed for anchorage. Tying reinforcement can be incorporated if required for disproportionate collapse.

When connecting timber floors to Ecobrix, joists can be built into walls, but this is generally found to be complex and time consuming, and to place restrictions on programming, so in practice, floor joists are commonly supported using joist hangers fixed to wall plates connected to the side of the Ecobrix wall. This detail requires careful attention.

Fixing the wall plate with screws is unlikely to be adequate, as the loads achievable screwing into the woodcrete are generally not sufficient (refer to Ecobrix Manual Appendix 4 for allowable Fischer fixing loads). In general the wall plate should be fixed directly to the concrete core, using cast-in or post-fixed (mechanical, self-tapping or resin-anchored) fixings.

The offset distance between the concrete core and the outer face of the wall plate where the load is applied may be close to 100mm, and this needs to be allowed for, with the deflection of bolts considered. This can be offset by installing bolts in pairs, with one installed at a downwards angle to create a triangulated arrangement.

Restraint of external and internal walls also needs to be considered, and tension straps may need to be cast into Ecobrix walls. Reference to Building Regulations Approved Document A in England and Wales, "The Small Buildings Structural Guidance" in Scotland or "Technical Booklet D: 2012 Structure" in Northern Ireland, along with BSI PD 6697:2019 "Recommendations for the design of masonry structures to BS EN 1996-1-1 and BS EN 1996-2" is recommended.

When detailing roof connections, uplift may need to be considered. Typically, a timber wall plate will be fixed directly above the Ecobrix concrete core, and it may be possible to provide sufficient resistance to uplift by using resin anchors fixing the wall plate to the core. However, where uplift loads are high, the length of bolts needed to mobilise sufficient dead weight may make it impractical for them to be installed, and tie-down straps may be necessary. Fixing of the straps to the woodcrete will need to be considered, taking account of the fixing loads achievable (again refer to Ecobrix Manual Appendix 4).

Design of Connections to Cladding

Many types of cladding may be applied to Ecobrix blocks, including brick or stone slips, render, timber or other cladding, or a masonry outer leaf. For the majority of these, the only calculations necessary may be the fixings required to support the weight of cladding and resist pull-off wind forces. As before, permissible loads for Fischer fixings are included in the Ecobrix Manual Appendix 4.

If a separate masonry skin is to be used, we would recommend using independent lintels for the masonry – "timber-frame" type lintels, fixed back to the Ecobrix blocks for restraint, may be suitable, or for larger spans, rolled steel angles or channels. For wall tie fixings, we would recommend using ties slotted into channels fixed back to the Ecobrix blocks, again, using Fischer fixings.

Retaining Walls

Retaining walls can be designed using Ecobrix, using the concrete "columns" within the cores with or without reinforcement to resist lateral loading. D250 blocks, or D300 or D365 blocks with the insulation removed, are preferred for most instances, as the wider cores can resist significantly higher loads. It should be noted that, as the woodcrete only has a compressive strength of around $2N/mm^2$, it is not typically designed as composite with the concrete fill, in the way that concrete hollow blocks (with typical block strengths of 10 to $18N/mm^2$) would be.

When used within basements, it should be noted that the presence of woodcrete webs means that the wall cannot be relied upon to provide inherent Type B structural waterproofing, so applied waterproofing, drained cavities, etc. will be required, to the requirements of the Architect/Building Control.

Design Documents

The following design documents will most commonly be used for the structural design of Ecobrix:

Eurocodes:

BS EN 1991-1-1:2002	Eurocode 1: Actions on structures — Part 1-1: General actions — Densities, self-weight, imposed loads for buildings
NA to BS EN 1991-1-1:2002	UK National Annex to Eurocode 1: Actions on structures — Part 1-1: General actions — Densities, self-weight, imposed loads for buildings
BS EN 1991-1-3:2003	Eurocode 1: Actions on structures — Part 1-3: General actions — Snow loads
NA to BS EN 1991-1-3:2003	UK National Annex to Eurocode 1: Actions on structures — Part 1-3: General actions — Snow loads
BS EN 1991-1-4:2005	Eurocode 1: Actions on structures — Part 1-4: General actions — Wind actions
NA to BS EN 1991-1-4:2005	UK National Annex to Eurocode 1: Actions on structures — Part 1-4: General actions – Wind actions
BS EN 1992-1-1:2004	Eurocode 2: Design of concrete structures — Part 1-1: General rules and rules for buildings
NA to BS EN 1992-1-1:2004	UK National Annex to Eurocode 2: Design of concrete structures — Part 1-1: General rules and rules for buildings

Other Documents:

The Building Regulations 2010 – Structure: Approved Document A. For use in England

Domestic Technical Handbook – January 2025 edition. For use in Scotland

The Small Buildings Structural Guidance – July 2010. For use in Scotland

The Building Regulations 2010 – Structure: Approved Document A. For use in Wales

Building Regulations (Northern Ireland) 2012 Guidance: Technical Booklet D: October 2012 Structure. For use in Northern Ireland

The Concrete Society	Design and Construction using Insulating Concrete Formwork
The Institution of Structural Engineers	Practical Guide to Structural Robustness and Disproportionate Collapse in Buildings
PD 6697:2019	Recommendations for the design of masonry structures to BS EN 1996-1-1 and BS EN 1996-2

Structural Calculations

The following calculations have been prepared to demonstrate that Ecobrix walls are at least equivalent to walls deemed to satisfy the requirements of Building Regulations, in line with either Approved Document A "Structure", in England and Wales, "The Small Buildings Structural Guidance" in Scotland or "Technical Booklet D: 2012 Structure" in Northern Ireland. They are not intended to be worked examples.

It should be noted that, for the purposes of comparing Ecobrix and masonry walls on an equal basis, simplifying assumptions have been made, such as ignoring eccentricity of load when checking vertical load capacity or the beneficial effects of dead loading, when checking for bending capacity. The strength of masonry walls has been determined using commercial software, with nominal loading applied.

It should be noted that, although the "Second Generation" of Eurocodes have been issued, at time of writing no updated UK National Annexes have been issued to accompany them. As such, in line with current Institution of Structural Engineers guidelines, these calculations have been prepared based on the "First Generation" of Eurocodes.

Wall Loads on Elevation

		kg/m²	Dead (G_k) kN/m²	Live (Q_k) kN/m²
D170 Wall				
Dead Load	D170 Durisol blocks	52		
	Concrete infill	225		
	Total Dead Load	**277**	**2.72**	
D250 Wall				
Dead Load	D250 Durisol blocks	84		
	Concrete infill	352		
	Total Dead Load	**436**	**4.28**	
D300 Wall				
Dead Load	D300 Durisol blocks inc. insulation	100		
	Concrete infill	235		
	Total Dead Load	**335**	**3.29**	
D365 Wall				
Dead Load	D365 Durisol blocks inc. insulation	116		
	Concrete infill	235		
	Total Dead Load	**351**	**3.44**	

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140mm thick Solid Blockwork Wall

Thickness of leaf	140 mm
Block Strength	7.3 N/mm^2
Mortar strength class	M4
Category of manufacturing control	Category II
Class of execution control	Class 2
Assumed blockwork density	18 kN/m^3

Flexural Strength

Refer to TEDDS Calculation following

=> Design moment of resistance parallel to bed joints = 0.350 kNm/m

=> Design moment of resistance perpendicular to bed joints = 0.645 kNm/m

Compressive Strength

Refer to TEDDS Calculation following

=> Design vertical load resistance = 166 kN/m

Shear Strength

Characteristic shear strength, f_{vk} = $f_{vko} + 0.4\sigma_d \leq 0.065 f_b$ *(cl. 3.6.2 eq. 3.5)*

where f_{vko} = 0.15 N/mm^2 *(Table NA.5)*

σ_d = 0.05 N/mm^2

f_b = 9.49 N/mm^2 *(Refer to TEDDS calc)*

=> Characteristic shear strength, f_{vk} = 0.18 N/mm^2

Design shear strength = $\dfrac{f_{vk} \times A}{\gamma_{mv}}$

γ_{mv} = 2.5 *(Table NA.1)*

=> Design shear strength of wall = 10.03 kN/m

MASONRY WALL PANEL DESIGN

In accordance with EN1996-1-1:2005 + A1:2012 incorporating Corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.2.22

Summary table

	Allowable	Actual	Utilisation	
Height to thickness ratio	43.929;	19.286;	0.439;	PASS
Design moment to wall	0.645 kNm/m;	0.153 kNm/m;	0.237;	PASS

Masonry panel details

Solid 140thk wall - Unreinforced masonry wall without openings

Panel length L = 9000 mm

Panel height h = 2700 mm

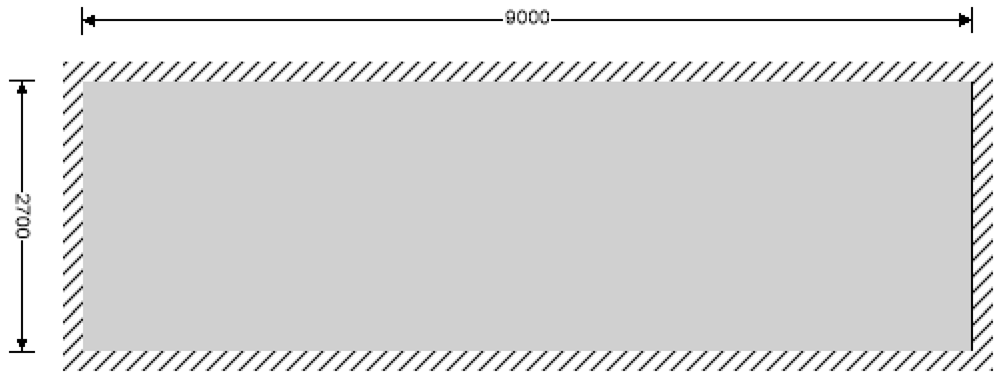
Panel support conditions

All edges supported

Effective height of masonry walls - Section 5.5.1.2

Reduction factor $\rho_2 = 1.000$

Effective height of wall - eq 5.2 $h_{ef} = \rho_2 \times h = 2700$ mm



Single-leaf wall construction details

Wall thickness t = 140 mm

Effective thickness of masonry walls - Section 5.5.1.3

Effective thickness $t_{ef} = t = 140$ mm

Masonry details

Masonry type — Aggregate concrete - Group 1

Compressive strength of masonry — $f_c = 7.3$ N/mm^2

Height of unit — $h_u = 215$ mm

Width of unit — $w_u = 140$ mm

Conditioning factor — $k = 1.0$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 — $d_{sf} = 1.3$

Norm. mean compressive strength of masonry $f_b = f_c \times k \times d_{sf} = 9.49$ N/mm^2

Density of masonry — $\gamma = 18$ kN/m^3

Mortar type — M4 - General purpose mortar

Compressive strength of masonry mortar — $f_m = 4$ N/mm^2

Compressive strength factor - Table NA.4 — $K = 0.75$

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 5.492$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

$f_{xk1} = 0.223$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

$f_{xk2} = 0.533$ N/mm^2

Lateral loading details

Characteristic wind load on panel — $W_k = 0.100$ kN/m^2

Partial factors for material strength

Category of manufacturing control — Category II

Class of execution control — Class 2

Partial factor for masonry in compressive flexure — $\gamma_{Mc} = 3.00$

Partial factor for masonry in tensile flexure — $\gamma_{Mt} = 2.70$

Partial factor for masonry in shear — $\gamma_{Mv} = 2.50$

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall — $N_{id} = \gamma_{fG} \times G_k + \gamma_{fQ} \times Q_k = 0$ kN/m

Moment at top of wall due to vertical load — $M_{id} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q = 0$ kNm/m

Initial eccentricity - cl.5.5.1.1 — $e_{init} = h_{ef} / 450 = 6$ mm

Moment at top of wall due to horizontal load — $M_{Eid} = 0$ kNm/m

Eccentricity at top of wall due to horizontal load — $e_h = 0$ mm

Eccentricity at top of wall - eq.6.5 — $e_i = \max(e_h + e_{init}, 0.05 \times t) = 7$ mm

Reduction factor at top of wall - eq.6.4 — $\Phi_i = \max(1 - 2 \times e_i / t, 0) = 0.9$

Vertical load at middle of wall — $N_{md} = \gamma_{fG} \times (G_k + \gamma \times t \times h / 2) + \gamma_{fQ} \times Q_k = 4.593$ kN/m

Moment at middle of wall due to vertical load — $M_{md} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q = 0$ kNm/m

Moment at middle of wall due to horizontal load — $M_{Emd} = 0.03$ kNm/m

Eccentricity at middle of wall due to horizontal load — $e_{hm} = M_{Emd} / N_{md} = 6.6$ mm

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Eccentricity at middle of wall due to loads - eq.6.7 $e_m = M_{md} / N_{md} + e_{hm} + e_{init} = 12.6$ mm

Eccentricity at middle of wall due to creep $e_k = 0$ mm

Eccentricity at middle of wall - eq.6.6 $e_{mk} = \max(e_m + e_k, 0.05 \times t) = 12.6$ mm

From eq.G.2 $A_1 = 1 - 2 \times e_{mk} / t = 0.821$

Short term secant modulus of elasticity factor $K_E = 1000$

Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k = 5492$ N/mm^2

Slenderness - eq.G.4 $\lambda = (h_{ef} / t_{ef}) \times \sqrt{(f_k / E)} = 0.61$

From eq.G.3 $u = (\lambda - 0.063) / (0.73 - 1.17 \times e_{mk} / t) = 0.875$

Reduction factor at middle of wall - eq.G.1 $\Phi_m = \max(A_1 \times e^{-(u \times u)/2}, 0) = 0.56$

Reduction factor for slenderness and eccentricity $\Phi = \min(\Phi_i, \Phi_m) = 0.56$

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

Partial safety factor for permanent load $\gamma_{fG} = 1$

Partial safety factor for variable imposed load $\gamma_{fQ} = 0$

Partial safety factor for variable wind load $\gamma_{fW} = 1.5$

Limiting height and length to thickness ratios for walls under the serviceability limit state - Annex F

Length to thickness ratio $L / t = 64.286$

Limiting height to thickness ratio - Figure F.1 43.929

Height to thickness ratio $h / t = 19.286$

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Self weight at middle of wall $S_{wt} = 0.5 \times h \times t \times \gamma = 3.402$ kN/m

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc} = 1.831$ N/mm^2

Design vertical compressive stress $\sigma_d = \min(\gamma_{fG} \times (G_k + S_{wt}) / t, 0.15 \times \Phi \times f_d) = 0.024$ N/mm^2

Design flexural strength of masonry parallel to bed joints

$f_{xd1} = f_{xk1} / \gamma_{Mt} = 0.083$ N/mm^2

Apparent design flexural strength of masonry parallel to bed joints

$f_{xd1,app} = f_{xd1} + \sigma_d = 0.107$ N/mm^2

Design flexural strength of masonry perpendicular to bed joints

$f_{xd2} = f_{xk2} / \gamma_{Mt} = 0.198$ N/mm^2

Elastic section modulus of wall $Z = t^2 / 6 = 3266667$ mm^3/m

Moment of resistance parallel to bed joints - eq.6.15

$M_{Rd1} = f_{xd1,app} \times Z = 0.35$ kNm/m

Moment of resistance perpendicular to bed joints - eq.6.15

$M_{Rd2} = f_{xd2} \times Z = 0.645$ kNm/m

Design moment in panels

Orthogonal strength ratio $\mu = f_{xd1,app} / f_{xd2} = 0.54$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha = 0.013$

Design moment in wall $M_{Ed} = \gamma_{fW} \times \alpha \times W_k \times L^2 = 0.153$ kNm/m

PASS - Resistance moment exceeds design moment

Project:	Ecobrix Manual				
Job no.:	25052	By:	A Dunford	Date:	25/11/2025
Subject:	140thk Solid Blockwork Wall Analysis and Design under Nominal Vertical Load				

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MASONRY WALL PANEL DESIGN

In accordance with EN1996-1-1:2005 + A1:2012 incorporating Corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.2.22

Summary table

	Allowable	Actual	Utilisation	
Slenderness ratio	27;	19.3;	0.714;	PASS
Vertical loading on wall	165.575 kN/m;	6.093 kN/m;	0.037;	PASS

Masonry panel details

Solid 140thk wall - Unreinforced masonry wall without openings

Panel length $L = 9000$ mm

Panel height $h = 2700$ mm

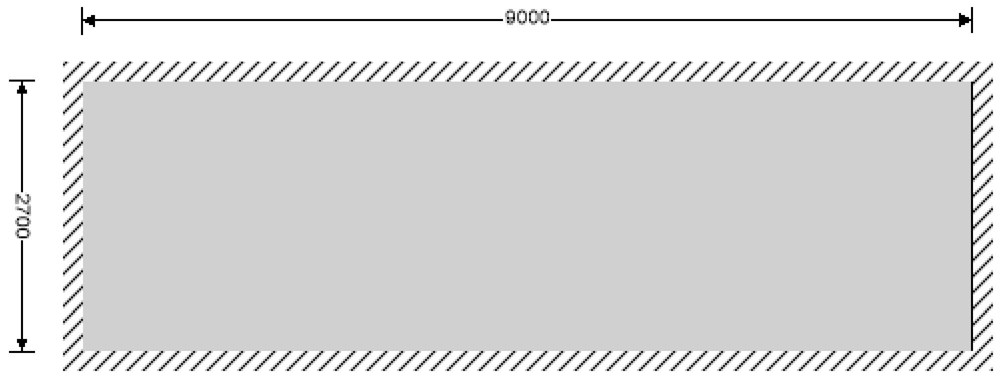
Panel support conditions

All edges supported

Effective height of masonry walls - Section 5.5.1.2

Reduction factor $\rho_2 = 1.000$

Effective height of wall - eq 5.2 $h_{ef} = \rho_2 \times h = 2700$ mm



Single-leaf wall construction details

Wall thickness $t = 140$ mm

Effective thickness of masonry walls - Section 5.5.1.3

Effective thickness $t_{ef} = t = 140$ mm

Masonry details

Masonry type — Aggregate concrete - Group 1

Compressive strength of masonry — $f_c = 7.3$ N/mm^2

Height of unit — $h_u = 215$ mm

Width of unit — $w_u = 140$ mm

Conditioning factor — $k = 1.0$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 — $d_{sf} = 1.3$

Norm. mean compressive strength of masonry — $f_b = f_c \times k \times d_{sf} = 9.49$ N/mm^2

Density of masonry — $\gamma = 18$ kN/m^3

Mortar type — M4 - General purpose mortar

Compressive strength of masonry mortar — $f_m = 4$ N/mm^2

Compressive strength factor - Table NA.4 — $K = 0.75$

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 5.492$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

$f_{xk1} = 0.223$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

$f_{xk2} = 0.533$ N/mm^2

Vertical loading details

Variable load on top of wall — $Q_k = 1$ kN/m

Partial factors for material strength

Category of manufacturing control — Category II

Class of execution control — Class 2

Partial factor for masonry in compressive flexure — $\gamma_{Mc} = 3.00$

Partial factor for masonry in tensile flexure — $\gamma_{Mt} = 2.70$

Partial factor for masonry in shear — $\gamma_{Mv} = 2.50$

Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio — $SR_{all} = 27$

Slenderness ratio — $SR = h_{ef} / t_{ef} = 19.3$

PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to mainly vertical loading - Section 6.1

Partial safety factors for design loads

Partial safety factor for permanent load — $\gamma_{fG} = 1.35$

Partial safety factor for variable imposed load — $\gamma_{fQ} = 1.5$

Partial safety factor for variable wind load — $\gamma_{fW} = 0.75$

Check vertical loads

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall — $N_{id} = \gamma_{fG} \times G_k + \gamma_{fQ} \times Q_k = 1.5$ kN/m

Moment at top of wall due to vertical load $M_{id} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q = 0$ kNm/m
Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{ef} / 450 = 6$ mm
Moment at top of wall due to horizontal load $M_{Eid} = 0$ kNm/m
Eccentricity at top of wall due to horizontal load $e_h = 0$ mm
Eccentricity at top of wall - eq.6.5 $e_i = \max(M_{id} / N_{id} + e_h + e_{init}, 0.05 \times t) = 7$ mm
Reduction factor at top of wall - eq.6.4 $\Phi_i = \max(1 - 2 \times e_i / t, 0) = 0.9$
Vertical load at middle of wall $N_{md} = \gamma_{fG} \times (G_k + \gamma \times t \times h / 2) + \gamma_{fQ} \times Q_k = 6.093$ kN/m
Moment at middle of wall due to vertical load $M_{md} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q = 0$ kNm/m
Moment at middle of wall due to horizontal load $M_{Emd} = 0$ kNm/m
Eccentricity at middle of wall due to horizontal load $e_{hm} = 0$ mm
Eccentricity at middle of wall due to loads - eq.6.7 $e_m = M_{md} / N_{md} + e_{hm} + e_{init} = 6$ mm
Eccentricity at middle of wall due to creep $e_k = 0$ mm
Eccentricity at middle of wall - eq.6.6 $e_{mk} = \max(e_m + e_k, 0.05 \times t) = 7$ mm
From eq.G.2 $A_1 = 1 - 2 \times e_{mk} / t = 0.9$
Short term secant modulus of elasticity factor $K_E = 1000$
Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k = 5492$ N/mm^2
Slenderness - eq.G.4 $\lambda = (h_{ef} / t_{ef}) \times \sqrt{(f_k / E)} = 0.61$
From eq.G.3 $u = (\lambda - 0.063) / (0.73 - 1.17 \times e_{mk} / t) = 0.814$
Reduction factor at middle of wall - eq.G.1 $\Phi_m = \max(A_1 \times e^{-(u \times u)/2}, 0) = 0.646$
Reduction factor for slenderness and eccentricity $\Phi = \min(\Phi_i, \Phi_m) = 0.646$

Verification of unreinforced masonry walls subjected to mainly vertical loading - Section 6.1.2
Design value of the vertical load $N_{Ed} = \max(N_{id}, N_{md}) = 6.093$ kN/m
Design compressive strength of masonry $f_d = f_k / \gamma_{Mc} = 1.831$ N/mm^2
Vertical resistance of wall - eq.6.2 $N_{Rd} = \Phi \times t \times f_d = 165.575$ kN/m

PASS - Design vertical resistance exceeds applied design vertical load

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Cavity Wall comprising 2 Skins of 90mm Blockwork

Thickness of leaf	90 mm
Effective thickness of leaf	120 mm
Block Strength	7.3 N/mm^2
Mortar strength class	M4
Category of manufacturing control	Category II
Class of execution control	Class 2
Assumed blockwork density	18 kN/m^3

Flexural Strength

Refer to TEDDS Calculation following

=> Design moment of resistance parallel to bed joints = 0.158 kNm/m per leaf

=> Design moment of resistance parallel to bed joints = 0.316 kNm/m total

=> Design moment of resistance perpendicular to bed joints = 0.300 kNm/m per leaf

=> Design moment of resistance perpendicular to bed joints = 0.600 kNm/m total

Compressive Strength

Refer to TEDDS Calculation following

=> Design vertical load resistance = 80.2 kN/m per leaf

=> Design vertical load resistance = 160.4 kN/m total

Shear Strength

Characteristic shear strength, f_{vk} = $f_{vko} + 0.4\sigma_d \le 0.065 f_b$ *(cl. 3.6.2 eq. 3.5)*

where f_{vko} = 0.15 N/mm^2 *(Table NA.5)*

σ_d = 0.05 N/mm^2

f_b = 10.22 N/mm^2 *(Refer to TEDDS calc)*

=> Characteristic shear strength, f_v = 0.18 N/mm^2

Design shear strength = $\dfrac{f_v \times A}{\gamma_{mv}}$

γ_{mv} = 2.5 *(Table NA.1)*

=> Design shear strength of wall = 6.45 kN/m per leaf

=> Design shear strength of wall = 12.90 kN/m total

MASONRY WALL PANEL DESIGN

In accordance with EN1996-1-1:2005 + A1:2012 incorporating Corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.2.22

Summary table

	Allowable	Actual	Utilisation	
Design moment to outer leaf	0.300 kNm/m;	0.078 kNm/m;	0.261;	PASS
Design moment to inner leaf	0.300 kNm/m;	0.078 kNm/m;	0.261;	PASS

Masonry panel details

90 + 90mm cavity wall - Unreinforced masonry wall without openings

Panel length $L = 9000$ mm

Panel height $h = 2700$ mm

Panel support conditions

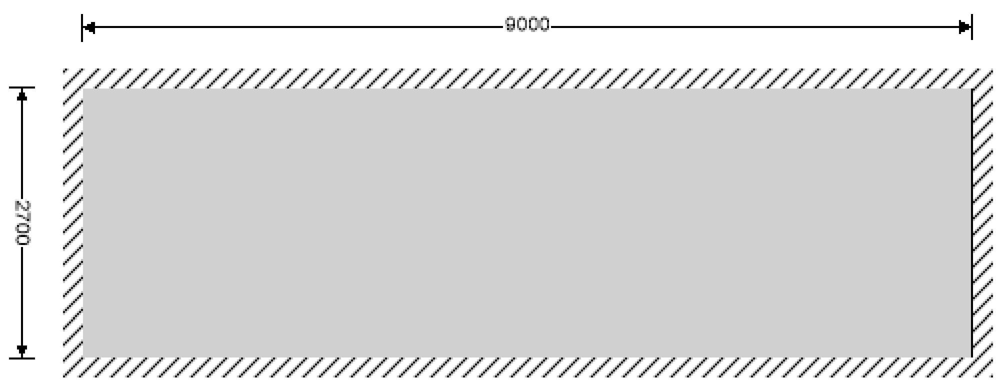
Outer leaf All edges supported

Inner leaf All edges supported

Effective height of masonry walls - Section 5.5.1.2

Reduction factor $\rho_2 = 1.000$

Effective height of wall - eq 5.2 $h_{ef} = \rho_2 \times h = 2700$ mm



Cavity wall construction details

Outer leaf thickness $t_1 = 90$ mm

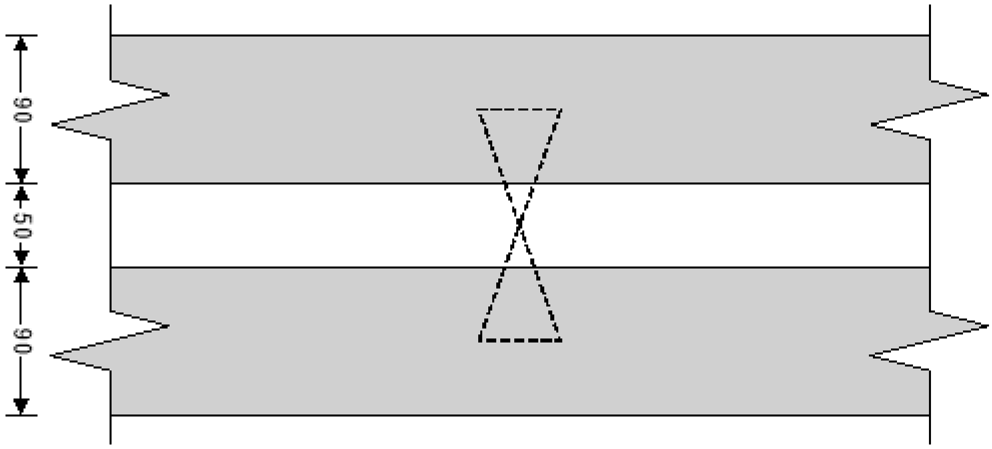
Cavity thickness $t_c = 50$ mm

Inner leaf thickness $t_2 = 90$ mm

Effective thickness of masonry walls - Section 5.5.1.3

Relative E factor $k_{tef} = 1.000$

Effective thickness - eq 5.11 $t_{ef} = (k_{tef} \times t_1^3 + t_2^3)^{1/3} = 113.4$ mm



Masonry outer leaf details

Masonry type Aggregate concrete - Group 2

Compressive strength of masonry f_{c1} = 7.3 N/mm^2

Height of unit h_{u1} = 215 mm

Width of unit w_{u1} = 90 mm

Conditioning factor k_1 = 1.0

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 d_{sf1} = 1.4

Norm. mean compressive strength of masonry $f_{b1} = f_{c1} \times k_1 \times d_{sf1}$ = 10.22 N/mm^2

Density of masonry γ_1 = 18 kN/m^3

Mortar type M4 - General purpose mortar

Compressive strength of masonry mortar f_{m1} = 4 N/mm^2

Compressive strength factor - Table NA.4 K = 0.70

Characteristic compressive strength of masonry - eq 3.1

$f_{k1} = K \times f_{b1}^{0.7} \times f_{m1}^{0.3}$ = 5.399 N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

f_{xk11} = 0.25 N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

f_{xk21} = 0.6 N/mm^2

Masonry inner leaf details

Masonry type Aggregate concrete - Group 2

Compressive strength of masonry f_{c2} = 7.3 N/mm^2

Height of unit h_{u2} = 215 mm

Width of unit w_{u2} = 90 mm

Conditioning factor k_2 = 1.0

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 d_{sf2} = 1.4

Norm. mean compressive strength of masonry $f_{b2} = f_{c2} \times k_2 \times d_{sf2}$ = 10.22 N/mm^2

Density of masonry $\gamma_2 = 18$ kN/m^3

Mortar type M4 - General purpose mortar

Compressive strength of masonry mortar $f_{m2} = 4$ N/mm^2

Compressive strength factor - Table NA.4 $K = 0.70$

Characteristic compressive strength of masonry - eq 3.1

$f_{k2} = K \times f_{b2}^{0.7} \times f_{m2}^{0.3} = 5.399$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

$f_{xk12} = 0.25$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

$f_{xk22} = 0.6$ N/mm^2

Lateral loading details

Characteristic wind load on panel $W_k = 0.100$ kN/m^2

Partial factors for material strength

Category of manufacturing control Category II

Class of execution control Class 2

Partial factor for masonry in compressive flexure $\gamma_{Mc} = 3.00$

Partial factor for masonry in tensile flexure $\gamma_{Mt} = 2.70$

Partial factor for masonry in shear $\gamma_{Mv} = 2.50$

Partial safety factors for design loads

Partial safety factor for permanent load $\gamma_{fG} = 1.35$

Partial safety factor for variable imposed load $\gamma_{fQ} = 1.5$

Partial safety factor for variable wind load $\gamma_{fW} = 0.75$

Considering outer leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall $N_{id1} = \gamma_{fG} \times G_{k1} + \gamma_{fQ} \times Q_{k1} = 0$ kN/m

Moment at top of wall due to vertical load $M_{id1} = \gamma_{fG} \times G_{k1} \times e_{G1} + \gamma_{fQ} \times Q_{k1} \times e_{Q1} = 0$ kNm/m

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{ef} / 450 = 6$ mm

Moment at top of wall due to horizontal load $M_{Eid1} = 0$ kNm/m

Eccentricity at top of wall due to horizontal load $e_{h1} = 0$ mm

Eccentricity at top of wall - eq.6.5 $e_{i1} = \max(e_{h1} + e_{init}, 0.05 \times t_1) = 6$ mm

Reduction factor at top of wall - eq.6.4 $\Phi_{i1} = \max(1 - 2 \times e_{i1} / t_1, 0) = 0.867$

Vertical load at middle of wall $N_{md1} = \gamma_{fG} \times (G_{k1} + \gamma_1 \times t_1 \times h / 2) + \gamma_{fQ} \times Q_{k1} = 2.952$ kN/m

Moment at middle of wall due to vertical load $M_{md1} = \gamma_{fG} \times G_{k1} \times e_{G1} + \gamma_{fQ} \times Q_{k1} \times e_{Q1} = 0$ kNm/m

Moment at middle of wall due to horizontal load $M_{Emd1} = 0.015$ kNm/m

Eccentricity at middle of wall due to horizontal load $e_{hm1} = M_{Emd1} / N_{md1} = 5.2$ mm

Eccentricity at middle of wall due to loads - eq.6.7 $e_{m1} = M_{md1} / N_{md1} + e_{hm1} + e_{init} =$ 11.2 mm

Eccentricity at middle of wall due to creep $e_{k1} = 0$ mm

Eccentricity at middle of wall - eq.6.6 $e_{mk1} = \max(e_{m1} + e_{k1}, 0.05 \times t_1) = 11.2$ mm

From eq.G.2 $A_{11} = 1 - 2 \times e_{mk1} / t_1 = 0.751$
Short term secant modulus of elasticity factor $K_E = 1000$
Modulus of elasticity - cl.3.7.2 $E_1 = K_E \times f_{k1} = 5399$ N/mm^2
Slenderness - eq.G.4 $\lambda_1 = (h_{ef} / t_{ef}) \times \sqrt{(f_{k1} / E_1)} = 0.753$
From eq.G.3 $u_1 = (\lambda_1 - 0.063) / (0.73 - 1.17 \times e_{mk1} / t_1) = 1.181$
Reduction factor at middle of wall - eq.G.1 $\Phi_{m1} = \max(A_{11} \times e^{-(u_1 \times u_1)/2}, 0) = 0.374$
Reduction factor for slenderness and eccentricity $\Phi_1 = \min(\Phi_{i1}, \Phi_{m1}) = 0.374$

Considering inner leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2
Vertical load at top of wall $N_{id2} = \gamma_{fG} \times G_{k2} + \gamma_{fQ} \times Q_{k2} = 0$ kN/m
Moment at top of wall due to vertical load $M_{id2} = \gamma_{fG} \times G_{k2} \times e_{G2} + \gamma_{fQ} \times Q_{k2} \times e_{Q2} = 0$ kNm/m
Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{ef} / 450 = 6$ mm
Moment at top of wall due to horizontal load $M_{Eid2} = 0$ kNm/m
Eccentricity at top of wall due to horizontal load $e_{h2} = 0$ mm
Eccentricity at top of wall - eq.6.5 $e_{i2} = \max(e_{h2} + e_{init}, 0.05 \times t_2) = 6$ mm
Reduction factor at top of wall - eq.6.4 $\Phi_{i2} = \max(1 - 2 \times e_{i2} / t_2, 0) = 0.867$
Vertical load at middle of wall $N_{md2} = \gamma_{fG} \times (G_{k2} + \gamma_2 \times t_2 \times h / 2) + \gamma_{fQ} \times Q_{k2} = 2.952$ kN/m
Moment at middle of wall due to vertical load $M_{md2} = \gamma_{fG} \times G_{k2} \times e_{G2} + \gamma_{fQ} \times Q_{k2} \times e_{Q2} = 0$ kNm/m
Moment at middle of wall due to horizontal load $M_{Emd2} = 0.015$ kNm/m
Eccentricity at middle of wall due to horizontal load $e_{hm2} = M_{Emd2} / N_{md2} = 5.2$ mm
Eccentricity at middle of wall due to loads - eq.6.7 $e_{m2} = M_{md2} / N_{md2} + e_{hm2} + e_{init} =$ 11.2 mm
Eccentricity at middle of wall due to creep $e_{k2} = 0$ mm
Eccentricity at middle of wall - eq.6.6 $e_{mk2} = \max(e_{m2} + e_{k2}, 0.05 \times t_2) = 11.2$ mm
From eq.G.2 $A_{12} = 1 - 2 \times e_{mk2} / t_2 = 0.751$
Short term secant modulus of elasticity factor $K_E = 1000$
Modulus of elasticity - cl.3.7.2 $E_2 = K_E \times f_{k2} = 5399$ N/mm^2
Slenderness - eq.G.4 $\lambda_2 = (h_{ef} / t_{ef}) \times \sqrt{(f_{k2} / E_2)} = 0.753$
From eq.G.3 $u_2 = (\lambda_2 - 0.063) / (0.73 - 1.17 \times e_{mk2} / t_2) = 1.181$
Reduction factor at middle of wall - eq.G.1 $\Phi_{m2} = \max(A_{12} \times e^{-(u_2 \times u_2)/2}, 0) = 0.374$
Reduction factor for slenderness and eccentricity $\Phi_2 = \min(\Phi_{i2}, \Phi_{m2}) = 0.374$

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads
Partial safety factor for permanent load $\gamma_{fG} = 1$
Partial safety factor for variable imposed load $\gamma_{fQ} = 0$
Partial safety factor for variable wind load $\gamma_{fW} = 1.5$

WARNING - Limiting height and length to thickness ratios for walls under the serviceability limit state were not checked as the design is beyond the scope of Annex F, these limits should be checked independently.

Design moments of resistance in panels

Considering outer leaf

Self weight at middle of wall $S_{wt1} = 0.5 \times h \times t_1 \times \gamma_1 = 2.187$ kN/m

Design compressive strength of masonry $f_{d1} = f_{k1} / \gamma_{Mc} = 1.800$ N/mm^2

Design vertical compressive stress $\sigma_{d1} = \min(\gamma_{fG} \times (G_{k1} + S_{wt1}) / t_1, 0.15 \times \Phi_1 \times f_{d1}) = 0.024$ N/mm^2

Design flexural strength of masonry parallel to bed joints

$f_{xd11} = f_{xk11} / \gamma_{Mt} = 0.093$ N/mm^2

Apparent design flexural strength of masonry parallel to bed joints

$f_{xd11,app} = f_{xd11} + \sigma_{d1} = 0.117$ N/mm^2

Design flexural strength of masonry perpendicular to bed joints

$f_{xd21} = f_{xk21} / \gamma_{Mt} = 0.222$ N/mm^2

Elastic section modulus of wall $Z_1 = t_1^2 / 6 = 1350000$ mm^3/m

Moment of resistance parallel to bed joints - eq.6.15

$M_{Rd11} = f_{xd11,app} \times Z_1 = 0.158$ kNm/m

Moment of resistance perpendicular to bed joints - eq.6.15

$M_{Rd21} = f_{xd21} \times Z_1 = 0.3$ kNm/m

Considering inner leaf

Self weight at middle of wall $S_{wt2} = 0.5 \times h \times t_2 \times \gamma_2 = 2.187$ kN/m

Design compressive strength of masonry $f_{d2} = f_{k2} / \gamma_{Mc} = 1.800$ N/mm^2

Design vertical compressive stress $\sigma_{d2} = \min(\gamma_{fG} \times (G_{k2} + S_{wt2}) / t_2, 0.15 \times \Phi_2 \times f_{d2}) = 0.024$ N/mm^2

Design flexural strength of masonry parallel to bed joints

$f_{xd12} = f_{xk12} / \gamma_{Mt} = 0.093$ N/mm^2

Apparent design flexural strength of masonry parallel to bed joints

$f_{xd12,app} = f_{xd12} + \sigma_{d2} = 0.117$ N/mm^2

Design flexural strength of masonry perpendicular to bed joints

$f_{xd22} = f_{xk22} / \gamma_{Mt} = 0.222$ N/mm^2

Elastic section modulus of wall $Z_2 = t_2^2 / 6 = 1350000$ mm^3/m

Moment of resistance parallel to bed joints - eq.6.15

$M_{Rd12} = f_{xd12,app} \times Z_2 = 0.158$ kNm/m

Moment of resistance perpendicular to bed joints - eq.6.15

$M_{Rd22} = f_{xd22} \times Z_2 = 0.3$ kNm/m

Calculate design wind load acting on each leaf

Outer leaf design wind load $W_{k1} = M_{Rd21} \times W_k / (M_{Rd21} + M_{Rd22}) = 0.050$ kN/m^2

Inner leaf design wind load $W_{k2} = M_{Rd22} \times W_k / (M_{Rd21} + M_{Rd22}) = 0.050$ kN/m^2

Design moment in panels

Considering outer leaf

Orthogonal strength ratio $\mu_1 = f_{xd11,app} / f_{xd21} = 0.53$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_1 = 0.013$

Design moment in wall $M_{Ed1} = \gamma_{fW} \times \alpha_1 \times W_{k1} \times L^2 = 0.078$ kNm/m

PASS - Resistance moment exceeds design moment

Considering inner leaf

Orthogonal strength ratio $\mu_2 = f_{xd12,app} / f_{xd22} = 0.53$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_2 = 0.013$

Design moment in wall $M_{Ed2} = \gamma_{fW} \times \alpha_2 \times W_{k2} \times L^2 = 0.078$ kNm/m

PASS - Resistance moment exceeds design moment

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MASONRY WALL PANEL DESIGN

In accordance with EN1996-1-1:2005 + A1:2012 incorporating Corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.2.22

Summary table

	Allowable	Actual	Utilisation	
Slenderness ratio	27;	23.8;	0.882;	PASS
Vertical loading on outer leaf	80.192 kN/m;	4.452 kN/m;	0.056;	PASS
Vertical loading on inner leaf	80.192 kN/m;	4.452 kN/m;	0.056;	PASS

Masonry panel details

Solid 140thk wall - Unreinforced masonry wall without openings

Panel length $L = 9000$ mm

Panel height $h = 2700$ mm

Panel support conditions

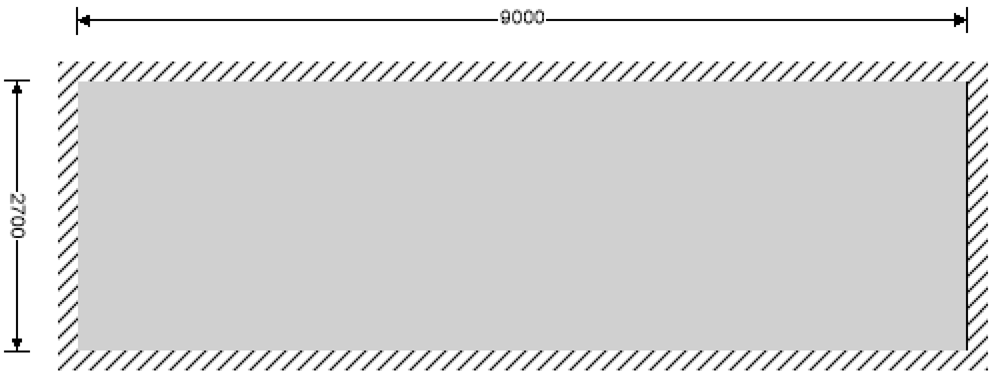
Outer leaf All edges supported

Inner leaf All edges supported

Effective height of masonry walls - Section 5.5.1.2

Reduction factor $\rho_2 = 1.000$

Effective height of wall - eq 5.2 $h_{ef} = \rho_2 \times h = 2700$ mm



Cavity wall construction details

Outer leaf thickness $t_1 = 90$ mm

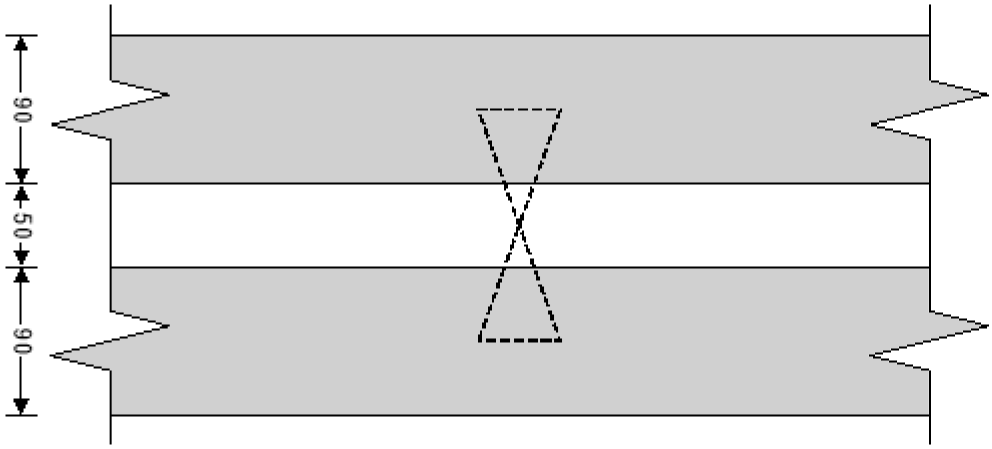
Cavity thickness $t_c = 50$ mm

Inner leaf thickness $t_2 = 90$ mm

Effective thickness of masonry walls - Section 5.5.1.3

Relative E factor $k_{tef} = 1.000$

Effective thickness - eq 5.11 $t_{ef} = (k_{tef} \times t_1^3 + t_2^3)^{1/3} = 113.4$ mm



Masonry outer leaf details

Masonry type	Aggregate concrete - Group 2
Compressive strength of masonry	$f_{c1} = 7.3\ N/mm^2$
Height of unit	$h_{u1} = 215\ mm$
Width of unit	$w_{u1} = 90\ mm$
Conditioning factor	$k_1 = 1.0$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1	$d_{sf1} = 1.4$
Norm. mean compressive strength of masonry	$f_{b1} = f_{c1} \times k_1 \times d_{sf1} = 10.22\ N/mm^2$
Density of masonry	$\gamma_1 = 18\ kN/m^3$
Mortar type	M4 - General purpose mortar
Compressive strength of masonry mortar	$f_{m1} = 4\ N/mm^2$
Compressive strength factor - Table NA.4	$K = 0.70$

Characteristic compressive strength of masonry - eq 3.1

$$f_{k1} = K \times f_{b1}^{0.7} \times f_{m1}^{0.3} = 5.399\ N/mm^2$$

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

$$f_{xk11} = 0.25\ N/mm^2$$

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

$$f_{xk21} = 0.6\ N/mm^2$$

Masonry inner leaf details

Masonry type	Aggregate concrete - Group 2
Compressive strength of masonry	$f_{c2} = 7.3\ N/mm^2$
Height of unit	$h_{u2} = 215\ mm$
Width of unit	$w_{u2} = 90\ mm$
Conditioning factor	$k_2 = 1.0$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1	$d_{sf2} = 1.4$
Norm. mean compressive strength of masonry	$f_{b2} = f_{c2} \times k_2 \times d_{sf2} = 10.22\ N/mm^2$

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Density of masonry γ_2 = 18 kN/m^3
Mortar type M4 - General purpose mortar
Compressive strength of masonry mortar f_{m2} = 4 N/mm^2
Compressive strength factor - Table NA.4 K = 0.70
Characteristic compressive strength of masonry - eq 3.1
$f_{k2} = K \times f_{b2}^{0.7} \times f_{m2}^{0.3}$ = 5.399 N/mm^2
Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6
f_{xk12} = 0.25 N/mm^2
Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6
f_{xk22} = 0.6 N/mm^2

Vertical loading details
Variable load on top of outer leaf Q_{k1} = 1 kN/m
Variable load on top of inner leaf Q_{k2} = 1 kN/m

Partial factors for material strength
Category of manufacturing control Category II
Class of execution control Class 2
Partial factor for masonry in compressive flexure γ_{Mc} = 3.00
Partial factor for masonry in tensile flexure γ_{Mt} = 2.70
Partial factor for masonry in shear γ_{Mv} = 2.50

Slenderness ratio of masonry walls - Section 5.5.1.4
Allowable slenderness ratio SR_{all} = 27
Slenderness ratio $SR = h_{ef} / t_{ef}$ = 23.8
PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to mainly vertical loading - Section 6.1

Partial safety factors for design loads
Partial safety factor for permanent load γ_{fG} = 1.35
Partial safety factor for variable imposed load γ_{fQ} = 1.5
Partial safety factor for variable wind load γ_{fW} = 0.75

Considering outer leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2
Vertical load at top of wall $N_{id1} = \gamma_{fG} \times G_{k1} + \gamma_{fQ} \times Q_{k1}$ = 1.5 kN/m
Moment at top of wall due to vertical load $M_{id1} = \gamma_{fG} \times G_{k1} \times e_{G1} + \gamma_{fQ} \times Q_{k1} \times e_{Q1}$ = 0 kNm/m
Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{ef} / 450$ = 6 mm
Moment at top of wall due to horizontal load M_{Eid1} = 0 kNm/m
Eccentricity at top of wall due to horizontal load e_{h1} = 0 mm
Eccentricity at top of wall - eq.6.5 $e_{i1} = \max(M_{id1} / N_{id1} + e_{h1} + e_{init}, 0.05 \times t_1)$ = 6 mm
Reduction factor at top of wall - eq.6.4 $\Phi_{i1} = \max(1 - 2 \times e_{i1} / t_1, 0)$ = 0.867
Vertical load at middle of wall $N_{md1} = \gamma_{fG} \times (G_{k1} + \gamma_1 \times t_1 \times h / 2) + \gamma_{fQ} \times Q_{k1}$ = 4.452 kN/m
Moment at middle of wall due to vertical load $M_{md1} = \gamma_{fG} \times G_{k1} \times e_{G1} + \gamma_{fQ} \times Q_{k1} \times e_{Q1}$ = 0 kNm/m

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Moment at middle of wall due to horizontal load $M_{Emd1} = 0$ kNm/m
Eccentricity at middle of wall due to horizontal load $e_{hm1} = 0$ mm
Eccentricity at middle of wall due to loads - eq.6.7 $e_{m1} = M_{md1} / N_{md1} + e_{hm1} + e_{init} =$ 6 mm
Eccentricity at middle of wall due to creep $e_{k1} = 0$ mm
Eccentricity at middle of wall - eq.6.6 $e_{mk1} = \max(e_{m1} + e_{k1}, 0.05 \times t_1) = 6$ mm
From eq.G.2 $A_{11} = 1 - 2 \times e_{mk1} / t_1 = 0.867$
Short term secant modulus of elasticity factor $K_E = 1000$
Modulus of elasticity - cl.3.7.2 $E_1 = K_E \times f_{k1} = 5399$ N/mm^2
Slenderness - eq.G.4 $\lambda_1 = (h_{ef} / t_{ef}) \times \sqrt{(f_{k1} / E_1)} = 0.753$
From eq.G.3 $u_1 = (\lambda_1 - 0.063) / (0.73 - 1.17 \times e_{mk1} / t_1) = 1.058$
Reduction factor at middle of wall - eq.G.1 $\Phi_{m1} = \max(A_{11} \times e^{-(u_1 \times u_1)/2}, 0) = 0.495$
Reduction factor for slenderness and eccentricity $\Phi_1 = \min(\Phi_{i1}, \Phi_{m1}) = 0.495$

Verification of unreinforced masonry walls subjected to mainly vertical loading - Section 6.1.2
Design value of the vertical load $N_{Ed1} = \max(N_{id1}, N_{md1}) = 4.452$ kN/m
Design compressive strength of masonry $f_{d1} = f_{k1} / \gamma_{Mc} = 1.800$ N/mm^2
Vertical resistance of wall - eq.6.2 $N_{Rd1} = \Phi_1 \times t_1 \times f_{d1} = 80.192$ kN/m
PASS - Design vertical resistance exceeds applied design vertical load

Considering inner leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2
Vertical load at top of wall $N_{id2} = \gamma_{fG} \times G_{k2} + \gamma_{fQ} \times Q_{k2} = 1.5$ kN/m
Moment at top of wall due to vertical load $M_{id2} = \gamma_{fG} \times G_{k2} \times e_{G2} + \gamma_{fQ} \times Q_{k2} \times e_{Q2} = 0$ kNm/m
Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{ef} / 450 = 6$ mm
Moment at top of wall due to horizontal load $M_{Eid2} = 0$ kNm/m
Eccentricity at top of wall due to horizontal load $e_{h2} = 0$ mm
Eccentricity at top of wall - eq.6.5 $e_{i2} = \max(M_{id2} / N_{id2} + e_{h2} + e_{init}, 0.05 \times t_2) = 6$ mm
Reduction factor at top of wall - eq.6.4 $\Phi_{i2} = \max(1 - 2 \times e_{i2} / t_2, 0) = 0.867$
Vertical load at middle of wall $N_{md2} = \gamma_{fG} \times (G_{k2} + \gamma_2 \times t_2 \times h / 2) + \gamma_{fQ} \times Q_{k2} = 4.452$ kN/m
Moment at middle of wall due to vertical load $M_{md2} = \gamma_{fG} \times G_{k2} \times e_{G2} + \gamma_{fQ} \times Q_{k2} \times e_{Q2} = 0$ kNm/m
Moment at middle of wall due to horizontal load $M_{Emd2} = 0$ kNm/m
Eccentricity at middle of wall due to horizontal load $e_{hm2} = 0$ mm
Eccentricity at middle of wall due to loads - eq.6.7 $e_{m2} = M_{md2} / N_{md2} + e_{hm2} + e_{init} =$ 6 mm
Eccentricity at middle of wall due to creep $e_{k2} = 0$ mm
Eccentricity at middle of wall - eq.6.6 $e_{mk2} = \max(e_{m2} + e_{k2}, 0.05 \times t_2) = 6$ mm
From eq.G.2 $A_{12} = 1 - 2 \times e_{mk2} / t_2 = 0.867$
Short term secant modulus of elasticity factor $K_E = 1000$
Modulus of elasticity - cl.3.7.2 $E_2 = K_E \times f_{k2} = 5399$ N/mm^2
Slenderness - eq.G.4 $\lambda_2 = (h_{ef} / t_{ef}) \times \sqrt{(f_{k2} / E_2)} = 0.753$

From eq.G.3 $u_2 = (\lambda_2 - 0.063) / (0.73 - 1.17 \times e_{mk2} / t_2) = 1.058$

Reduction factor at middle of wall - eq.G.1 $\Phi_{m2} = \max(A_{12} \times e^{-(u_2 \times u_2)/2}, 0) = 0.495$

Reduction factor for slenderness and eccentricity $\Phi_2 = \min(\Phi_{i2}, \Phi_{m2}) = 0.495$

Verification of unreinforced masonry walls subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load $N_{Ed2} = \max(N_{id2}, N_{md2}) = 4.452$ kN/m

Design compressive strength of masonry $f_{d2} = f_{k2} / \gamma_{Mc} = 1.800$ N/mm^2

Vertical resistance of wall - eq.6.2 $N_{Rd2} = \Phi_2 \times t_2 \times f_{d2} = 80.192$ kN/m

PASS - Design vertical resistance exceeds applied design vertical load

D170 Ecobrix Wall

Overall thickness of wall	170 mm
Thickness of concrete infill	120 mm
Width of cont's concrete columns	160 mm @ 250mm centres
% of continuous vertical concrete	64 %
Concrete grade	C25/30
Overall wall density	16.0 kN/m^3

Flexural Strength

Design moment of resistance = $f_{ctd,pl}Z$

where $f_{ctd,pl} = \alpha_{ct,pl} f_{ctk,0.05} / \gamma_c$ *(cl. 12.3.1 eq. 12.1)*

$\alpha_{ct,pl}$ =	0.8	*(NA 12.3.1 (1))*
$f_{ctk,0.05}$ =	1.8 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{ctd,pl}$ = 0.96 N/mm^2

Plane of failure parallel to bed joints

$$Z = \frac{160 \times 120^2}{6} \times 4 = 1.54\text{E}+06 \text{ mm}^3/\text{m}$$

=> Design moment of resistance parallel to bed joints = 1.47 kNm/m

Plane of failure perpendicular to bed joints

$$Z = \frac{100 \times 100^2}{6} \times 4 = 6.67\text{E}+05 \text{ mm}^3/\text{m}$$

=> Design moment of resistance perpendicular to bed joints = 0.64 kNm/m

Compressive Strength

Design vertical load resistance $N_{Rd} = \mathrm{b} \times h_w \times f_{cd,pl} \times \Phi$ *(cl. 12.6.5.2 eq. 12.10)*

where $f_{cd,pl} = \alpha_{cc,pl} f_{ck} / \gamma_c$ *(cl. 3.16 eq. 3.15)*

$\alpha_{cc,pl}$ =	0.6	*(NA 12.3.1 (1))*
f_{ck} =	25.0 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{cd,pl}$ = 10.0 N/mm^2

and $\Phi = 1.14 \times (1 - 2e_{tot}/h_w) - 0.02 \times l_o/h_w \leq (1 - 2e_{tot}/h_w)$ *(cl. 12.6.5.2 eq. 12.11)*

where $l_o = \beta\, l_w$ *(cl. 12.6.5.1 eq. 12.9)*

l_w = 2700 mm

b = 9000 mm

=> β = 0.92 *(Table 12.1)*

=> l_o = 2477 mm

$e_{tot} = e_o + e_i$ *(cl. 12.6.5.2 eq. 12.12)*

e_o = 0.0 mm

$e_i = l_o/400$ = 6.19 mm *(cl. 5.2(9))*

=> e_{tot} = 6.19 mm

=> Φ = 0.609

=> Design vertical load resistance, N_{Rd} = 731 kN/m

However, this assumes a continuous wall, so factoring for the % of continuous vertical concrete:

=> Design vertical load resistance, N_{Rd} = 468 kN/m

Shear Strength

Characteristic shear strength, f_{cvd} is calculated as follows:

For vertical self weight loading, σ_{cp} = 0.096 N/mm^2 *(cl. 12.6.3 eq. 12.3)*

$$\sigma_{c,lim} = f_{cd,pl} - 2\sqrt{f_{ctd,pl}(f_{ctd,pl} + f_{cd,pl})}$$ *(cl. 12.6.3 eq. 12.7)*

$f_{cd,pl}$ = 10.00 N/mm^2 *(See above)*

$f_{ctd,pl}$ = 0.96 N/mm^2 *(See above)*

=> $\sigma_{c,lim}$ = 3.51 N/mm^2

=> $\sigma_{cp} < \sigma_{c,lim}$ => $f_{cvd} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} f_{ctd,pl}}$ *(cl. 12.6.3 eq. 12.5)*

=> f_{cvd} = 10.00 N/mm^2

k = 1.5 *(NA 12.6.3 (2))*

=> Design shear strength of wall in horizontal direction = 512 kN/m

For vertical plane, σ_{cp} = 0 N/mm^2

=> f_{cvd} = 10.00 N/mm^2

k = 1.5

=> Design shear strength of wall in vertical direction = 267 kN/m

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D250 Ecobrix Wall

Overall thickness of wall	250 mm
Thickness of concrete infill	180 mm
Width of cont's concrete columns	190 mm @ 250mm centres
% of continuous vertical concrete	76 %
Concrete grade	C25/30
Overall wall density	17.1 kN/m^3

Flexural Strength

Design moment of resistance = $f_{ctd,pl}\mathrm{Z}$

where $f_{ctd,pl} = \alpha_{ct,pl} f_{ctk,0.05}/\gamma_c$ *(cl. 12.3.1 eq. 12.1)*

$\alpha_{ct,pl}$ =	0.8	*(NA 12.3.1 (1))*
$f_{ctk,0.05}$ =	1.8 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{ctd,pl}$ = 0.96 N/mm^2

Plane of failure parallel to bed joints

$$Z = \frac{160\times120^2}{6}\times 4 = 4.10\mathrm{E}{+}06\ mm^3/m$$

=> Design moment of resistance parallel to bed joints = 3.94 kNm/m

Plane of failure perpendicular to bed joints

$$Z = \frac{100\times100^2}{6}\times 4 = 6.67\mathrm{E}{+}05\ mm^3/m$$

=> Design moment of resistance perpendicular to bed joints = 0.64 kNm/m

Compressive Strength

Design vertical load resistance $N_{Rd} = \mathrm{b} \times h_w \times f_{cd,pl} \times \Phi$ *(cl. 12.6.5.2 eq. 12.10)*

where $f_{cd,pl} = \alpha_{cc,pl} f_{ck}/\gamma_c$ *(cl. 3.16 eq. 3.15)*

$\alpha_{cc,pl}$ =	0.6	*(NA 12.3.1 (1))*
f_{ck} =	25.0 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{cd,pl}$ = 10.0 N/mm^2

and $\Phi = 1.14 \times (1 - 2e_{tot}/h_w) - 0.02 \times l_o/h_w \le (1 - 2e_{tot}/h_w)$ *(cl. 12.6.5.2 eq. 12.11)*

where $l_o = \beta\, l_w$ *(cl. 12.6.5.1 eq. 12.9)*

l_w = 2700 mm

b = 9000 mm

=> β = 0.92 *(Table 12.1)*

=> l_o = 2477 mm

$e_{tot} = e_o + e_i$ *(cl. 12.6.5.2 eq. 12.12)*

e_o = 0.0 mm

$e_i = l_o/400$ = 6.19 mm *(cl. 5.2(9))*

=> e_{tot} = 6.19 mm

=> Φ = 0.786

=> Design vertical load resistance, N_{Rd} = 1415 kN/m

However, this assumes a continuous wall, so factoring for the % of continuous vertical concrete:

=> Design vertical load resistance, N_{Rd} = 1076 kN/m

Shear Strength

Characteristic shear strength, f_{cvd} is calculated as follows:

For vertical self weight loading, σ_{cp} = 0.084 N/mm^2 *(cl. 12.6.3 eq. 12.3)*

$$\sigma_{c,lim} = f_{cd,pl} - 2\sqrt{f_{ctd,pl}(f_{ctd,pl} + f_{cd,pl})}$$ *(cl. 12.6.3 eq. 12.7)*

$f_{cd,pl}$ = 10.00 N/mm^2 *(See above)*

$f_{ctd,pl}$ = 0.96 N/mm^2 *(See above)*

=> $\sigma_{c,lim}$ = 3.51 N/mm^2

=> $\sigma_{cp} < \sigma_{c,lim}$ => $f_{cvd} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} f_{ctd,pl}}$ *(cl. 12.6.3 eq. 12.5)*

=> f_{cvd} = 10.00 N/mm^2

k = 1.5 *(NA 12.6.3 (2))*

=> Design shear strength of wall in horizontal direction = 912 kN/m

For vertical plane, σ_{cp} = 0 N/mm^2

=> f_{cvd} = 10.00 N/mm^2

k = 1.5

=> Design shear strength of wall in vertical direction = 267 kN/m

D300 Ecobrix Wall

Overall thickness of wall	300 mm
Thickness of concrete infill	120 mm
Width of cont's concrete columns	150 mm @ 250mm centres
% of continuous vertical concrete	60 %
Concrete grade	C25/30
Overall wall density	11.0 kN/m^3

Flexural Strength

Design moment of resistance = $f_{ctd,pl} \mathrm{Z}$

where $f_{ctd,pl} = \alpha_{ct,pl} f_{ctk,0.05}/\gamma_c$ *(cl. 12.3.1 eq. 12.1)*

$\alpha_{ct,pl}$ =	0.8	*(NA 12.3.1 (1))*
$f_{ctk,0.05}$ =	1.8 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{ctd,pl}$ = 0.96 N/mm^2

Plane of failure parallel to bed joints

$Z = \frac{160 \times 120^2}{6} \times 4$ = 1.44E+06 mm^3/m

=> Design moment of resistance parallel to bed joints = 1.38 kNm/m

Plane of failure perpendicular to bed joints

$Z = \frac{100 \times 100^2}{6} \times 4$ = 6.67E+05 mm^3/m

=> Design moment of resistance perpendicular to bed joints = 0.64 kNm/m

Compressive Strength

Design vertical load resistance $N_{Rd} = \mathrm{b} \times h_w \times f_{cd,pl} \times \Phi$ *(cl. 12.6.5.2 eq. 12.10)*

where $f_{cd,pl} = \alpha_{cc,pl} f_{ck}/\gamma_c$ *(cl. 3.16 eq. 3.15)*

$\alpha_{cc,pl}$ =	0.6	*(NA 12.3.1 (1))*
f_{ck} =	25.0 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{cd,pl}$ = 10.0 N/mm^2

and $\Phi = 1.14 \times (1 - 2e_{tot}/h_w) - 0.02 \times l_o/h_w \leq (1 - 2e_{tot}/h_w)$ *(cl. 12.6.5.2 eq. 12.11)*

where $l_o = \beta\, l_w$ *(cl. 12.6.5.1 eq. 12.9)*

l_w =	2700 mm	
b =	9000 mm	
=> β =	0.92	*(Table 12.1)*
=> l_o =	2477 mm	
$e_{tot} = e_o + e_i$		*(cl. 12.6.5.2 eq. 12.12)*
e_o =	0.0 mm	
$e_i = l_o/400$ =	6.19 mm	*(cl. 5.2(9))*
=> e_{tot} =	6.19 mm	

=> Φ = 0.609

=> Design vertical load resistance, N_{Rd} = 731 kN/m

However, this assumes a continuous wall, so factoring for the % of continuous vertical concrete:

=> Design vertical load resistance, N_{Rd} = 439 kN/m

Shear Strength

Characteristic shear strength, f_{cvd} is calculated as follows:

For vertical self weight loading, σ_{cp} = 0.123 N/mm² *(cl. 12.6.3 eq. 12.3)*

$$\sigma_{c,lim} = f_{cd,pl} - 2\sqrt{f_{ctd,pl}(f_{ctd,pl} + f_{cd,pl})}$$ *(cl. 12.6.3 eq. 12.7)*

$f_{cd,pl}$ = 10.00 N/mm² *(See above)*

$f_{ctd,pl}$ = 0.96 N/mm² *(See above)*

=> $\sigma_{c,lim}$ = 3.51 N/mm²

=> $\sigma_{cp} < \sigma_{c,lim}$ => $f_{cvd} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} f_{ctd,pl}}$ *(cl. 12.6.3 eq. 12.5)*

=> f_{cvd} = 10.01 N/mm²

k = 1.5 *(NA 12.6.3 (2))*

=> Design shear strength of wall in horizontal direction = 480 kN/m

For vertical plane, σ_{cp} = 0 N/mm²

=> f_{cvd} = 10.00 N/mm²

k = 1.5

=> Design shear strength of wall in vertical direction = 267 kN/m

D365 Ecobrix Wall

Overall thickness of wall	365 mm
Thickness of concrete infill	120 mm
Width of cont's concrete columns	150 mm @ 250mm centres
% of continuous vertical concrete	60 %
Concrete grade	C25/30
Overall wall density	9.4 kN/m^3

Flexural Strength

Design moment of resistance = $f_{ctd,pl}Z$

where $f_{ctd,pl} = \alpha_{ct,pl} f_{ctk,0.05}/\gamma_c$ *(cl. 12.3.1 eq. 12.1)*

$\alpha_{ct,pl}$ =	0.8	*(NA 12.3.1 (1))*
$f_{ctk,0.05}$ =	1.8 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{ctd,pl}$ = 0.96 N/mm^2

Plane of failure parallel to bed joints

$Z = \frac{160 \times 120^2}{6} \times 4$ = 1.44E+06 mm^3/m

=> Design moment of resistance parallel to bed joints = 1.38 kNm/m

Plane of failure perpendicular to bed joints

$Z = \frac{100 \times 100^2}{6} \times 4$ = 6.67E+05 mm^3/m

=> Design moment of resistance perpendicular to bed joints = 0.64 kNm/m

Compressive Strength

Design vertical load resistance $N_{Rd} = \mathrm{b} \times h_w \times f_{cd,pl} \times \Phi$ *(cl. 12.6.5.2 eq. 12.10)*

where $f_{cd,pl} = \alpha_{cc,pl} f_{ck}/\gamma_c$ *(cl. 3.16 eq. 3.15)*

$\alpha_{cc,pl}$ =	0.6	*(NA 12.3.1 (1))*
f_{ck} =	25.0 N/mm^2	*(Table 3.1)*
γ_c =	1.5	*(Table 2.1N)*

=> $f_{cd,pl}$ = 10.0 N/mm^2

and $\Phi = 1.14 \times (1 - 2e_{tot}/h_w) - 0.02 \times l_o/h_w \leq (1 - 2e_{tot}/h_w)$ *(cl. 12.6.5.2 eq. 12.11)*

where $l_o = \beta \, l_w$ *(cl. 12.6.5.1 eq. 12.9)*

l_w = 2700 mm

b = 9000 mm

=> β = 0.92 *(Table 12.1)*

=> l_o = 2477 mm

$e_{tot} = e_o + e_i$ *(cl. 12.6.5.2 eq. 12.12)*

e_o = 0.0 mm

$e_i = l_o/400$ = 6.19 mm *(cl. 5.2(9))*

=> e_{tot} = 6.19 mm

=> Φ = 0.609

=> Design vertical load resistance, N_{Rd} = 731 kN/m

However, this assumes a continuous wall, so factoring for the % of continuous vertical concrete:

=> Design vertical load resistance, N_{Rd} = 439 kN/m

Shear Strength

Characteristic shear strength, f_{cvd} is calculated as follows:

For vertical self weight loading, σ_{cp} = 0.129 N/mm^2 *(cl. 12.6.3 eq. 12.3)*

$$\sigma_{c,lim} = f_{cd,pl} - 2\sqrt{f_{ctd,pl}(f_{ctd,pl} + f_{cd,pl})}$$ *(cl. 12.6.3 eq. 12.7)*

$f_{cd,pl}$ = 10.00 N/mm^2 *(See above)*

$f_{ctd,pl}$ = 0.96 N/mm^2 *(See above)*

=> $\sigma_{c,lim}$ = 3.51 N/mm^2

=> $\sigma_{cp} < \sigma_{c,lim}$ => $f_{cvd} = \sqrt{f_{ctd,pl}^2 + \sigma_{cp} f_{ctd,pl}}$ *(cl. 12.6.3 eq. 12.5)*

=> f_{cvd} = 10.01 N/mm^2

k = 1.5 *(NA 12.6.3 (2))*

=> Design shear strength of wall in horizontal direction = 480 kN/m

For vertical plane, σ_{cp} = 0 N/mm^2

=> f_{cvd} = 10.00 N/mm^2

k = 1.5

=> Design shear strength of wall in vertical direction = 267 kN/m

Wall capacities are as follows:

Wall Type	Design Code	Flexural Strength		Comp. Strength	Shear Strength	
		Parallel kNm/m	Perp. kNm/m	Vertical kN/m	Horiz. kN/m	Vertical kN/m
Solid masonry wall	EC 6	0.350	0.645	166	10.03	10.03
Masonry Cavity wall	EC 6	0.316	0.600	160	12.90	12.90
D170	EC 2	1.475	0.640	468	512	267
D250	EC 2	3.940	0.640	1076	912	267
D300	EC 2	1.382	0.640	439	480	267
D365	EC 2	1.382	0.640	439	480	267