A Limit Analysis of Unreinforced Masonry Failing Under Flood Water Pressures

by

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ABSTRACT

This paper defines, and offers a solution to, the general problem of failure in unreinforced masonry walls from combined hydrostatic and hydrodynamic lateral pressures. This issue is important for assessing the height to which homeowners should seal their properties against flood waters. Yield line analysis theory is refined and used.

With static flood water, many walls would fail when the flood depth differential between the inside and outside of a dwelling is approximately 1.0-1.5m. With flood velocities which could arise in realistic situations, this depth differential for collapse can drop below 0.5m. The most effective means of strengthening walls is to increase the density or thickness of the masonry units. Other concerns may preclude such action. Despite uncertainties in the calculations, the method is validated by comparison with the literature and supports advice that sealing should not be completed above 0.9-1.0m above ground level.

NOTATION

A B C D E F G H I J K L nodes on a wall panel

A	residence plan area [m^2]
avg	(as a subscript) average or typical value
C_f	compression depth factor, i.e. the percentage by which the effective thickness of the panel is reduced due to crushing at the edge of the masonry when rotation occurs [no units]
f_{diff}	maximum flood depth differential between the inside and outside of a residence [m]
g	acceleration due to gravity [9.80665m s^{-2}] [1]
H	height of wall panel [m]
j	number of storeys or levels in a residence [no units]
m	ultimate resisting moment per unit length [N]
max	(as a subscript) maximum
min	(as a subscript) minimum
ΔP	pressure difference [Pa]
$\Delta P_{y=y1}$	pressure difference at the point y = y1 [Pa]
v	velocity [m s^{-1}]
T	thickness of a masonry leaf [m]
$T_{blockwork}$	thickness of blockwork [m]
$T_{brickwork}$	thickness of brickwork [m]
W	width of wall panel [m]
x	horizontal coordinate direction
y	vertical coordinate direction
γ	mass supported by wall panel [kg]
δ	deflection of masonry slabs before failure [m]
ζ	factor in yield line analysis [no units]
η	factor in yield line analysis; η = H/W [no units]
$\theta\ \theta_{x1}\ \theta_{x2}\ \theta_y$	angles of rotations in slabs for yield line analysis [radians]
λ	fraction of residence's dead load supported by a wall panel; may have subscript "blockwork" or "brickwork" [no units]
ρ_b	density of a masonry leaf [kg m^{-3}]
$\rho_{blockwork}$	density of blockwork [kg m^{-3}]
$\rho_{brickwork}$	density of brickwork [kg m^{-3}]
ρ_w	density of water [kg m^{-3}]
$\sum$	sum
φ	factor in yield line analysis [no units]
ψ	friction coefficient to determine the ultimate resisting moment perpendicular to the bed joint in yield line analysis [no units]

1. INTRODUCTION

This paper examines unreinforced masonry wall collapse due to a combined hydrostatic and hydrodynamic pressure from flood water. Recent flooding in the U.K. has led to advice being published on flood damage prevention for homeowners. Various products are marketed for sealing dwellings in order to prevent floodwater entry, but such sealing will permit the water to rise outside the residence without a counterbalancing pressure inside. EA/CIRIA [2] advise that no sealing should take place "more than one metre above the level of the ground surrounding the property". DTLR [3] suggests:

For most properties the external walls should have sufficient strength to retain flood depths up to 900mm (3 feet) above external ground level, with no water inside the building. However, the actual water level that can be safely retained by the external walls will depend upon the construction, age and condition of the walls.

This paper provides evidence for assessing the appropriateness of this recommendation while extending current theory in unreinforced masonry failure under lateral pressures.

Cavity walls are used in the majority of unreinforced masonry dwellings currently being constructed in the U.K., hence these walls are considered for the analysis. These walls comprise a brickwork outer leaf attached to a blockwork inner leaf by metal ties and are neither pre-compressed, nor pre-stressed, nor reinforced. External walls on residences are partitioned vertically by supported timber floors and horizontally by internal partition walls, so the failure analysis should not consider an entire external wall. Instead, wall panels are considered.

2. PAST STUDIES AND EMPIRICAL EVIDENCE

Past studies rarely detail non-uniform loading on unreinforced masonry walls. The case of blast loading has been studied, for example ALEXANDER and TAYLOR [4], but peak pressures last a fraction of a second. This load duration is unhelpful for studying flood pressures which generally last longer, at least several seconds and possibly minutes or hours.

USACE [5] subjected unreinforced masonry walls to hydrostatic loading. Their conclusion was that the walls "can safely withstand about [0.9m] of waterhead" [5]. This conclusion perhaps indicates the origin of USACE's [6] suggestion that most buildings would collapse for $f_{diff} > 0.9$m as well as the advice quoted above from EA/CIRIA [2] and DTLR [3]. The 0.9-1.0m maximum for unreinforced masonry walls withstanding lateral hydrostatic pressure translates into $\Delta P_{avg} \approx 5 kN/m^2$.

Table 1 displays a sample of empirical evidence for uniform lateral loading. From these results, wall panels under a uniform lateral pressure would be expected to fail in the approximate range 1-10kN/m^2, with the lower end of this range more common than the upper end. Translating the uniform load results into a hydrostatic and hydrodynamic combined pressure is not

Table 1
Examples of empirical research into wall failure under uniform, lateral loading

Reference	Wall Type	H	W	Notes	Failure Pressure
CAVANAGH *et al.* [7]	Cavity walls with two brickwork leaves in an old building.	2.2m	3.6m	-damp-proof course -plaster on the inside -bricked-up opening	12.5kN/m^2
		2.2m	4.57m	-damp-proof course -plaster on the inside -boarded-up opening	12.5kN/m^2
DUARTE [8]	Sixteen single-leaf brickwork walls (laboratory testing).	1.2m	1.2m to 1.8m	-each wall with a 0.4m × 0.4m opening, but the opening's position varied	lowest 2.6kN/m^2 (W = 1.8m) highest 10.2kN/m^2 (W = 1.2m)
GAIRNS and SCRIVENER [9]	Five single-leaf blockwork panels (laboratory testing).	2.58m	2.35m to 4.75m		lowest 2.64kN/m^2 (W = 4.75m) highest 5.16kN/m^2 (W = 2.35m)
	Two single-leaf brickwork panels (laboratory testing).	2.58m	4.75m		3.63kN/m^2 3.82kN/m^2
SINHA [10]	Two single-leaf brickwork panels (laboratory testing).	-H and W are not given, but "all test panels…were built in 1/3rd scale bricks" (p. 59) -each panel had a small central opening			8.31kN/m^2 10.34kN/m^2
SINHA *et al.* [11]	Seven single-leaf brickwork panels (laboratory testing).	-H and W are not given, but "all the test walls were built with bricks one third of the normal size" (p. 192). -H/W ranged from 0.54 to 2.14.			least 1.37kN/m^2 (H/W = 2.14) most 10.2kN/m^2 (H/W = 0.54)
SOUTHCOMBE *et al.* [12]	Ten single-leaf brickwork panels (laboratory testing).	2.475m	2.915m or 5.615m	-some with openings, but the opening's position varied	lowest 2.2kN/m^2 highest 7.5kN/m^2
SOUTHCOMBE and TAPP [13]	Five single-leaf brickwork panels (laboratory testing).	2.475m	4.840m	-each wall had an opening, two similar to a window and three similar to a door	lowest 0.60kN/m^2 highest 2.00kN/m^2
TEMPLETON *et al.* [14]	A two-storey cavity wall house was built in a laboratory.	ground floor 3.150m first floor 2.400m	2 panels, 4.222m each	-loading was applied starting at 0kN/m^2 in 0.25kN/m^2 increments	cracking started at 1.25kN/m^2 but the house was still taking load when the testing was stopped at 3kN/m^2
WEST *et al.* [15]	91 Single brickwork leaves (laboratory testing)	mainly 2.6m; some 1.3m or 3.6m	1.52m to 5.50m	-some with a damp-proof course	lowest 1.72kN/m^2 (W = 5.50m) highest 15.51kN/m^2 (W = 1.52m)
	Nineteen cavity walls with one brickwork leaf and one leaf of either blockwork or brickwork (laboratory testing).	2.6m	3.6m to 5.5m	-some with solid ties and some with butterfly ties, but no significant difference in failure pressure noted	lowest 2.41kN/m^2 (W = 5.5m with butterfly ties) highest 6.4kN/m^2 (W = 5.5m with quadruple solid ties)

straightforward, but a rough estimate would be that failure would occur when the linear load's ΔP_{avg} approximately equals the uniform ΔP for failure of 1-10kN/m^2.

The research in Table 1 has weaknesses, particularly the laboratory results. The experiments do not test walls with doors and windows of the sizes and positions observed in real residences. Nonetheless, the experiments are indicative of the range of failure ΔP expected and walls without openings are sometimes used in dwellings. The wall edge conditions in the laboratory are most frequently simply supported, which would be weaker than the continuous supports found for most wall panels in residences. Hence, walls in residences may be expected to be stronger than the literature suggests, although the expected effect of openings is not clear (see Section 9).

Ties between the cavity wall leaves must be considered. WEST and HODGKINSON [16] inferred that solid ties transfer the load between leaves whereas butterfly ties do not. Their conclusion was based on one wall with each tie type and is refuted by results from WEST *et al.* [15] where no pattern emerges with respect to ties. Instead, cavity walls appear to behave more as single-leaf walls with the two leaves failing independently. Other observations in Table 1 of cavity wall failure pressure compared to single-leaf failure pressure suggest that a reasonable, conservative assumption is that, irrespective of how a leaf or wall fails, load transfer between the leaves through the ties is negligible. This worst-case assumption means that the leaves fail independently—or, at least, they fail more independently than not—an assumption validated by observations. Possible failure mechanisms for cavity wall failure are not discussed in the literature, so Section 3 proposes possibilities for the leaves failing independently.

Damp proof courses appear to have minimal impact. WEST and HODGKINSON's [16] conclusion that damp proof courses have a significant effect resulted from considering an unrealistic scenario in which the damp proof course is placed between the wall and the steel frame holding the wall. WEST *et al.* [15] support WEST and HODGKINSON's [16] conclusions but admit they arise from an impractical situation. WEST *et al.* [15] also tested one wall each with a blue brick damp proof course and a permagrip damp proof course, stating that "the enhanced lateral resistance is marked" compared to walls without a damp proof course. This conclusion is not solidly based on the data considering that (a) the sample size was one each for walls with damp proof courses and (b) larger pressure variations were observed between two similar walls than between walls with and without a damp proof course.

The other references in Table 1 rarely mention that damp proof courses may be important and the values reported do not indicate any significant influence from them. SOUTHCOMBE and TAPP [13] suggest that for yield line analysis (see Section 3) "a nominal degree of fixity at the dpc [damp proof course], derived from the self weight of the masonry corresponding to 10% of the mid-span moment" would be appropriate.

The empirical research provides the failure pressures expected in order to validate analysis of failure under flood water pressures. To start such analysis, a failure mechanism must be postulated and an analysis technique is needed. For failure mechanism, BS 5628-1 [17] - based on the work of, amongst others, WEST *et al.* [15], HENDRY [18], and HENDRY *et al.* [19] - determines brickwork

strength with tests which break through the bricks. This approach is surprising considering that the tensile strength of mortar is in most cases likely to be weaker than the brick for most modern U.K. residences.

Empirical evidence does indicate that failure under lateral pressures in brickwork and blockwork is characterised by failure along the mortar joints rather than through bricks. Laboratory work [5,10,14,20], in situ tests [7,21], and observed failure incidents ([22] and Figures 1 and 2) illustrate the tendency for failure along mortar joints with masonry units occasionally breaking. Selecting an analysis technique which incorporates failure along mortar joints is thus appropriate.

Figure 1–Brickwork wall hit by a car in Cambridge, U.K., December 2001

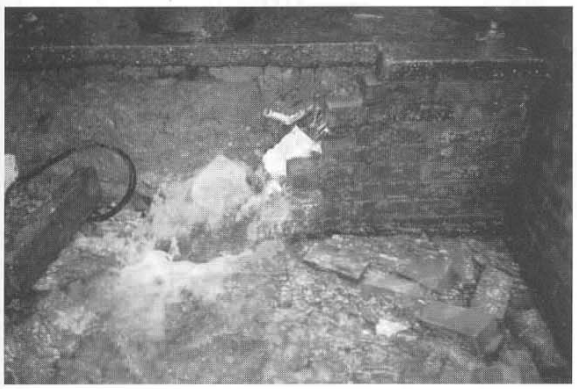


Figure 2–Brickwork wall broken by flood water, Malton, U.K., November 2000

3. YIELD LINE ANALYSIS AND FAILURE MECHANISM

Yield line analysis can be adapted to incorporate failure along mortar joints and is well-established in the literature. Yield line analysis was originally developed for structural concrete and therefore assumed plasticity. The theory proposes that a panel under a load develops a plastic hinge in a region of high moment. This hinge resists the hinging moment, transferring any increase in the load to other regions which then yield and become part of the hinge. This region of the hinge is a yield line. When yield lines have formed across the panel dividing it into slabs, these slabs rotate plastically under the applied load and the panel collapses.

In criticising the use of yield line analysis for brickwork, HASELTINE *et al.* [22] note:

Clearly a brick wall could not be expected to reach a constant moment and hold it since brickwork is a brittle material which will fail as soon as its resistance moment is reached; no moment, or very little, will be carried across the resultant crack.

GAIRNS and SCRIVENER [9], MARTINI [23], and SINHA [24] support this view. SINHA [24] develops a modified yield line analysis technique termed "fracture line analysis" which avoids the assumption of brickwork plasticity. SINHA's [24] assumptions are:

All deformations take place along the fracture lines only, and the individual parts of the slab rotate as rigid bodies. The load distribution is in accordance with the stiffness in the respective directions. The fracture lines develop only when the relevant strengths are reached in the two directions…Every portion of the panel into which it is divided by the fracture lines is in equilibrium under the action of external forces and reactions along the fracture lines and supports.

SINHA [24], however, obtains brickwork properties through experiments which break through bricks rather than the observed fracturing along mortar joints. He also places his test specimens horizontally rather than the true vertical position of wall panels, thereby ignoring the panel's self-weight.

LOVEGROVE [25] adapts the theoretical explanation of yield line analysis to be appropriate for unreinforced masonry. He considers the energy needed to produce a crack in order to develop the same yield line analysis equations, thereby bypassing assumptions of plastic hinges or elasticity. MARTINI [23] also proposes an equivalent of a plastic moment. For a horizontal crack, he uses the overturning resisting moment due to vertical loads. For a vertical crack, he suggests that "moment transfer takes place through friction between horizontal surfaces at the bed joints between courses". Implementing this approach requires the fundamental principle of static equilibrium of the unreinforced masonry panel in the collapse limit state under the external load. In this paper, the work method is adopted to define the equilibrium equations by analysing a small, arbitrary displacement or deflection δ of the slabs in the direction of the applied load.

The main design advantage of yield line analysis is that it provides an upper bound for failure ΔP. One failure mechanism is identified from which a failure ΔP is determined, but another failure mechanism could exist which would provide a lower failure ΔP. Yield line analysis thus indicates the upper limit at which failure would definitely occur, subject to the uncertainties in the parameters used. The designer knows that if the failure ΔP calculated by yield line analysis were applied to the wall, failure must occur irrespective of the failure mechanism. This bound is useful for knowing the limit beyond which the wall cannot survive. Conversely, yield line analysis does not suggest a lower bound for failure. The lowest ΔP for which the wall could fail remains unknown.

The discussion so far has described wall failure up to the point where pressure breaks one leaf of the cavity wall. The most appropriate approach, based on empirical data from Table 1, would be to assume that the leaves fail independently. The result is a conservative estimate for failure load. This assumption is addressed in Sections 8 and 9 regarding the choice of leaf thickness. Although this approach is acceptable, a plausible failure mechanism should be postulated to ensure that a potential explanation exists for this choice.

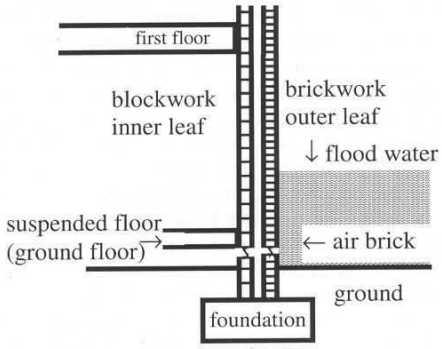
The key is to demonstrate that an explanation exists for the ties transferring negligible load between the two leaves under the relatively large loads imposed by flood pressures. The ties are set in the mortar of each leaf which may provide this explanation. Under large compressive loads, the ties may buckle or they may be pushed through the mortar in the leaf opposite the load. Under large tensile loads, the ties may be pulled out of the mortar.

Therefore, ties potentially transfer negligible load under either compression or tension.

Two possible failure mechanisms for cavity walls must be explored (Figure 3). In Figure 3a, water enters the cavity through the air bricks which are low and numerous where used. The water rises rapidly in the cavity inducing ΔP across the inner leaf. The ties experience tensile loading and the inner leaf breaks. The water in the cavity rushes into the house leaving ΔP across the outer leaf. The outer leaf might or might not collapse.

In Figure 3b, water rises too quickly outside the residence for significant infiltration to occur through the air bricks or air bricks may not exist. ΔP exists across the outer leaf. The ties experience compressive loading and the outer leaf breaks. The outer leaf may collapse completely leaving ΔP exerted on the inner leaf which might or might not break. Alternatively, the cavity's narrow width may not permit the outer leaf to collapse completely. Instead, the load would be transferred to the inner leaf which might or might not collapse.

Figure 3 displays the common situation of the first floor, and by extension other floors above if they exist, connected to only the inner leaf. Previous discussion suggested that the ties transfer negligible load between leaves. Thus, it seems likely that the outer leaf would fail as a single panel over all storeys of the residence. When the outer leaf fails and the ties are under compression, they may buckle or they may be pushed through the inner leaf's mortar. If the ties buckle, then the outer leaf is liable to fail as a single panel. In such a case, some of the slabs would span two storeys or more and it is unclear whether or not such a large slab would rotate as a rigid body, particularly when the pressure is experienced only at the bottom of the slab. If the ties are pushed through the mortar in the leaf opposite the load, then they would reach the first floor and would likely be stopped (Figure 4). The ties may buckle or, at this stage, the panel below the first floor may start failing with a yield line forming near the first floor since an effectively continuous support exists there.



- The ground floor may be partially or fully embedded in the inner leaf.
- The ground floor may be solid rather than suspended.
- The foundation may be a raft or may extend farther than depicted.
- Some air bricks may be directly connected between the inner and outer leaves. Air bricks may be absent from Figure 3b.
- In one-storey residences, the "first floor" would be the roof and the leaves would not extend above the roof.

Figure 3- Legend

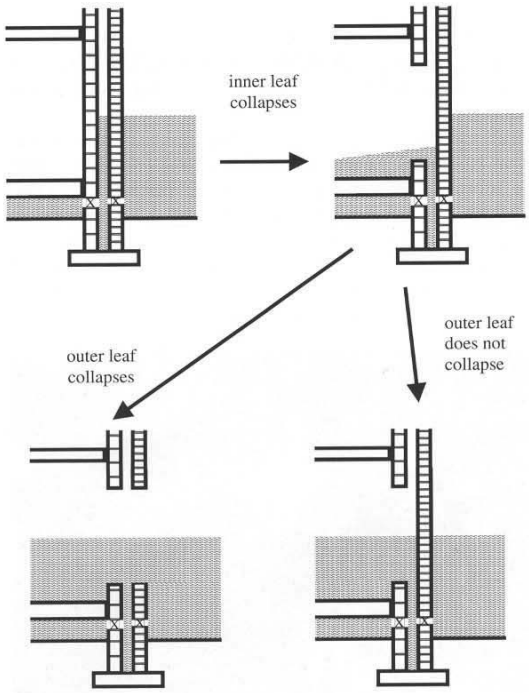


Figure 3a–Schematic for wall failure where the inner leaf fails first (ties under tension)

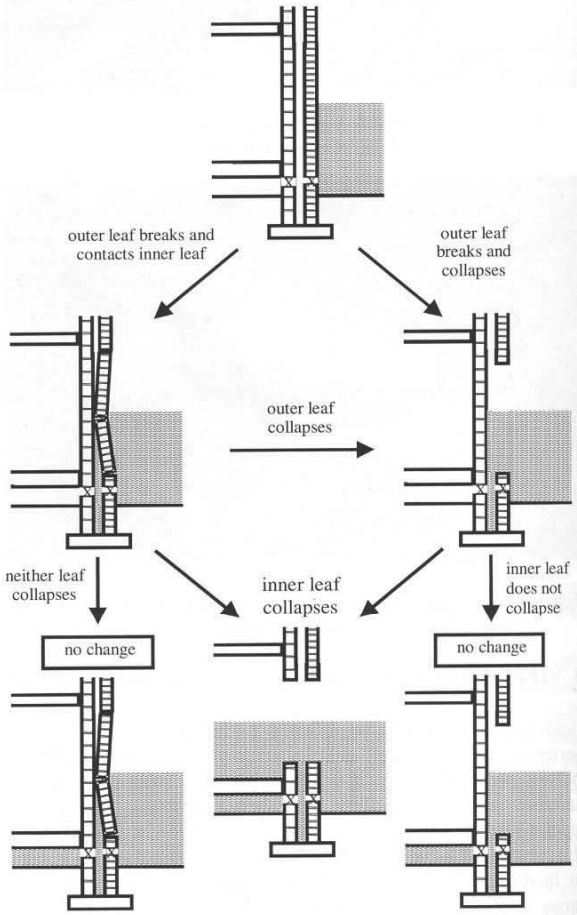


Figure 3b–Schematic for wall failure where the outer leaf fails first (ties under compression)

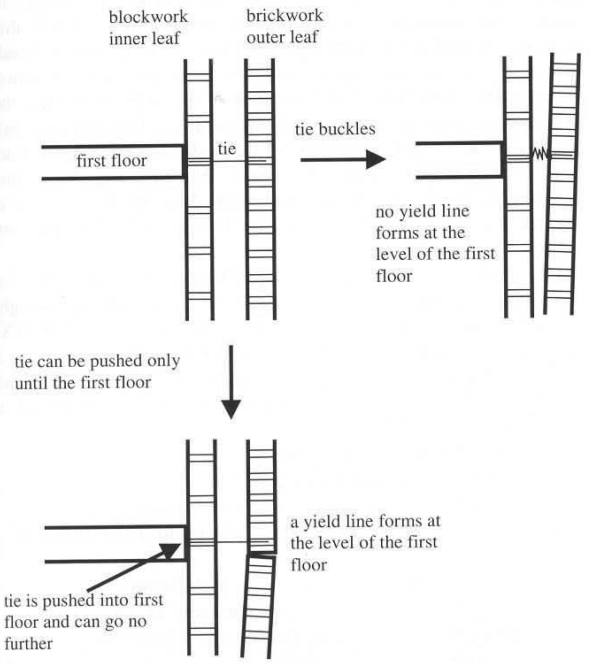


Figure 4–Potential failure mechanisms of the brickwork outer leaf

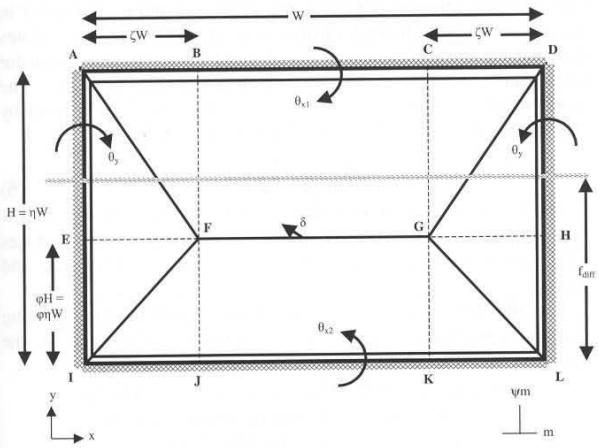


Figure 5–Problem definition for wall panel failure

The situation is ambiguous. The literature, having considered neither non-uniform loads nor the cavity wall's mechanism of failure, does not discuss the failure mechanism to this level of detail. Therefore, for these calculations, the arbitrary assumption is made that the outer leaf fails with a yield line forming at the level of the first floor, as the inner leaf must. Section 9 notes that this assumption influences the results negligibly. Further research, most usefully laboratory work, would be needed to establish exactly how multi-storey, cavity wall, outer leaves fail and how ties influence the failure mechanism.

The mechanisms presented here are speculative but realistic. To verify these proposed failure mechanisms, laboratory work which examines each step of the failure process would be needed. Field observations of collapsed cavity walls may not leave clues enabling reconstruction of the pathway to failure. Despite the lack of either supporting or contrary evidence, the assumptions made here are, at minimum, feasible. Thus, the calculations may proceed, but with caution warranted by the uncertainties. Further comment is provided in Section 9.

The failure mechanism is now firmly established: the two leaves fail independently and either leaf, or both leaves in either order, may fail. Expected results for failure are that the average pressure on the panel would be in the range 1-10kN/m^2 and most likely towards the lower end. These results indicate only the lowest ΔP at which the panel would definitely fail, not the highest ΔP at which the panel would definitely survive.

4. CALCULATIONS

The assumptions which must be made are:

- the failure mechanism described by slightly reinterpreted yield line analysis is correct.
- openings are not considered.
- brickwork and blockwork are neither fully elastic nor fully plastic, but they are brittle.
- the yield line pattern which forms is shown in Figure 5 with idealised yield lines. Section 9 addresses this assumption.
- the slabs formed by the yield lines are rigid, i.e. they do not deform within themselves.

Additionally, an assumption illustrated in Figure 5 is that $H > f_{diff} > \varphi H$ at failure. The calculations include this situation plus the two other possibilities of $\varphi H > f_{diff} > 0$ and $f_{diff} > H$.

Figure 6 illustrates the basic pressure equation, from which the formulae are derived:

$$\Delta P = \rho_w g(f_{diff} - y) + \tfrac{1}{2}\rho_w v^2$$
$$= \Delta P_{y=0} - \rho_w g y \text{ for } 0 \le y \le f_{diff} \text{ and } \Delta P = 0 \text{ for } y > f_{diff} \quad (1)$$

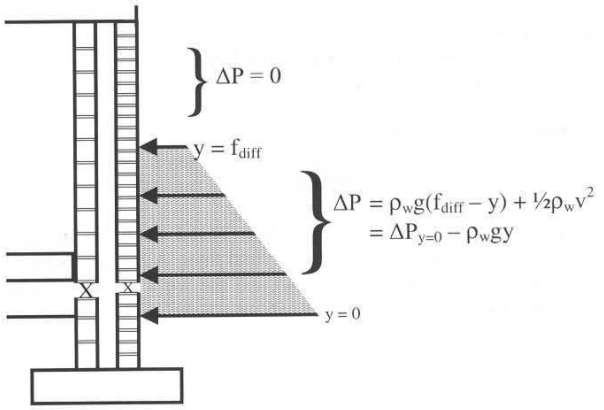


Figure 6–Pressure function on a wall

In Figure 5, if point **I** at the bottom-left is set to (0,0), then the coordinates of the other points, the equations of each segment, and the segment lengths may be easily determined.

The variables sought are f_{diff}, v, ζ and φ. f_{diff} and v are determined via ΔP. By definition, $f_{diff}, v \ge 0$; $0 \le \zeta \le \tfrac{1}{2}$; and $0 \le \varphi \le 1$. m is a function of tension in the mortar and the weight supported by the wall panel above the yield line. Calculating m is described later in this Section. ψ is a material property and is also interpreted later. H, W and η are properties of the residences. ρ_w and g are physical constants.

The edge conditions for **AI**, **DL**, and **IL** are set as continuous as the most realistic representation of residence walls. **AD** is also depicted as continuous in Figure 5, but both leaves would be simply supported in one-storey residences. The calculations assume that a yield line forms along **AD**, but where **AD** is simply supported and no yield line would form, the appropriate ultimate resisting moment term is set to zero. The yield lines are thus **AD**, **AF**, **AI**, **DG**, **DL**, **FG**, **FI**, **GL** and **IL**. Lines around which the slabs rotate are:

- **AD** (slab **ADGF** rotates θ_{x1});
- **AI** (slab **AFI** rotates θ_y);
- **DL** (slab **DGL** rotates θ_y); and
- **IL** (slab **FGLI** rotates θ_{x2}).

Due to failure along the mortar joints rather than through the masonry units, the vertical and diagonal yield lines actually comprise a sequence of horizontal and vertical segments (Figures 7 and 8).

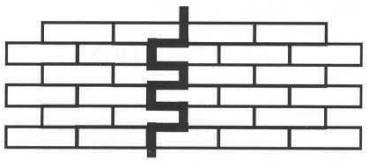


Figure 7–Vertical yield line in brickwork and blockwork

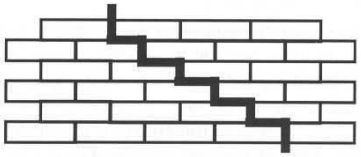


Figure 8–Diagonal yield line in brickwork and blockwork

Maximum deflection δ occurs along **FG** including the points F and G. Assuming that $\tan\theta \approx \theta$ for small θ and, since length cannot change in the x-y plane, a gap appears between slabs (presumably due to the masonry's thickness):

$$\delta = \mathbf{BF}\,\theta_{x1} = \mathbf{FJ}\,\theta_{x2} = \mathbf{EF}\,\theta_y$$
$$\Rightarrow \quad \eta W(1-\varphi)\,\theta_{x1} = \varphi\eta W\,\theta_{x2} = \zeta W\,\theta_y \qquad (2)$$

Therefore:

$$\theta_y = \eta(1-\varphi)\theta_{x1} / \zeta$$
$$\theta_{x2} = (1-\varphi)\,\theta_{x1} / \varphi$$
$$\delta = \eta W(1-\varphi)\,\theta_{x1} \qquad (3)$$

Each slab **AFI**, **ADGF**, **DGL** and **FGLI** must be in static equilibrium during the formation of yield lines. A balance between the work done by the external load and the work used internally, or expended by the yield lines, is maintained. This balance is expressed by the work equation:

$$\Sigma_{\text{over all yield lines}} m \times (\text{length of yield line experiencing m}) \times \theta\ (\text{slab's angle of rotation})$$
$$= \Sigma_{\text{over all slabs}} \int_{\text{over slab's area}} (\text{load experienced by a slab}) \times (\text{slab's deflection})\ dx\ dy \qquad (4)$$

m × (length of yield line experiencing m) is the ultimate resisting moment of a yield line which, at failure, equals the actual bending moment along that line. The resisting moment is a function of tension in the mortar and the weight supported by the slab above the yield line. For the slabs to rotate resulting in collapse of the panel, all yield lines would have formed, i.e. all mortar joints along yield lines would have broken and no tension would exist in the mortar along these lines.

For yield line analysis, SPENCE and COBURN [21] assume that "The mortar used, whether of soil, cement or lime, is assumed to have zero tensile strength, but high compressive strength". Similarly, in developing a failure criterion for laterally loaded masonry walls, GUGGISBERG and THÜRLIMANN [26] write "the tensile strength of the mortar joints will be neglected". The assumption is thus made that this failure process of yield lines forming may be modelled by stating that all tension in the mortar joints is overcome to form the yield lines before the slabs begin to rotate.

Throughout this process, ΔP has not changed. f_{diff} and v are assumed to reach their values instantaneously and to be invariable during the time required for the yield lines to develop, the slabs to rotate, and the panel to fail—in the instances in which this sequence would occur. Although work is obviously done to break the mortar joints, this work does not enter the work equation because the slabs have not rotated. For the slabs to rotate, the resisting moment of m × (length of yield line experiencing m), based on the weight supported by the slab above the yield line, must be overcome. Thus, the work equation compares this resisting moment with the moment applied by ΔP. If the latter is greater than the former, failure occurs. Otherwise, failure does not occur.

Calculating m × (length of yield line experiencing m) for horizontal components of yield lines involves only the weight above (in the y-direction) each yield line (see also MARTINI [23]). Consequently, m × (length of yield line experiencing m) is a function of y incorporating the self-weight of the wall panel and the dead and live loads of the structure which are transferred to this panel:

$$\begin{aligned} &m \times (\text{length of yield line experiencing m}) \\ &= [\ (\text{panel's weight above yield line}) \\ &\quad + (\text{dead and live weight above yield line})\ ] \\ &\quad \times (\text{distance of the weight's force vector from its line of rotation}) \\ &= [\ \rho_b g \times (\text{panel's volume above yield line})] \\ &\quad + \gamma g W_{\text{yield line}} / W_{\text{panel}} \times (\text{approximately } TC_f/2) \\ &= [\ \rho_b g T \times (\text{panel's area directly above yield line}) \\ &\quad + \gamma g W_{\text{yield line}} / W_{\text{panel}}\ ] \times TC_f/2 \end{aligned} \qquad (5)$$

Calculating m × (length of yield line experiencing m) for vertical components of yield lines involves a different mechanism. A yield line is theoretically a line with negligible thickness, hence the weight above a purely vertical yield line must be zero. Solely vertical yield lines do not exist (see Figures 7 and 8). To factor in these complicated changes of direction, MARTINI [23] postulates that, for the vertical components of yield lines, "moment transfer takes place through friction between horizontal surfaces". The work which must be expended to overcome the friction from by the slab rotating around the yield line is calculated by:

$$\text{work} = \psi(m_{\text{top of yield line}} + m_{\text{bottom of yield line}}) / 2 \times (\text{vertical length of yield line}) \times \theta \qquad (6)$$

Using the mean of m is necessary since m changes linearly with y. ψ is interpreted as the friction coefficient between the two surfaces which rotate relative to each other, in this case masonry units and mortar.

For the left-hand side of the work equation, the resisting moment's work at failure in the yield lines is illustrated for the diagonal segment **AF**:

$$\begin{aligned} &m_{y=\eta W(\varphi+1)/2} \times \mathbf{AB} \times \theta_{x1} + \psi m_{y=\eta W(\varphi+1)/2} \times \mathbf{AE} \times \theta_y \\ &= [\ \rho_b g T \times (\text{slab area directly above yield line}) \\ &\quad + \gamma g W_{\text{yield line}} / W_{\text{panel}}\ ] \times TC_f / 2 \times \theta_{x1} \\ &\quad + \psi(m_{\text{top of yield line}} + m_{\text{bottom of yield line}}) / 2 \\ &\quad \times (\text{vertical length of yield line}) \times \theta_y \\ &= [\ \rho_b g T \times \mathbf{AB} \times \mathbf{BF} / 2 + \gamma g \zeta W / W\ ] \times TC_f/2 \times \theta_{x1} \\ &\quad + \psi(m_{\mathbf{AD}} + m_{\mathbf{FG}}) / 2 \times \eta W(1-\varphi) \times \eta(1-\varphi)\theta_{x1} / \zeta \\ &= [\ \rho_b g T \times W^2 \zeta\eta(1-\varphi) / 2 + g\gamma\zeta\ ] \times TC_f/2 \times 0_{x1} \\ &\quad + \eta^2\theta_{x1}(1-\varphi)^2\psi / (2\zeta) \times [\ \rho_b g T \eta W^2(1-\varphi) + 2\gamma g\ ] \\ &\quad \times TC_f / 2 \end{aligned} \qquad (7)$$

where:

$$\begin{aligned} m_{\mathbf{AD}} &= [\ \rho_b g T \times 0 + \gamma g W / W\ ] \times TC_f / 2 / \mathbf{AD} \\ &= \gamma g TC_f / (2W) \end{aligned} \qquad (8)$$

$$\begin{aligned} m_{\mathbf{FG}} &= [\ \rho_b g T \times \mathbf{BF} \times \mathbf{BC} + \gamma g W(1 - 2\zeta) / W\ ] \times TC_f / 2 / \mathbf{FG} \\ &= [\ \rho_b g T \times \eta W^2(1-\varphi) + \gamma g\ ] \times TC_f / (2W) \end{aligned} \qquad (9)$$

For the right-hand side of the work equation, the total work done in moving a slab is:

$$\int \Delta P \times \text{deflection}\ dA = \iint \Delta P \times \text{deflection}\ dx\ dy = \iint \Delta P \times \text{deflection}\ dy\ dx \qquad (10)$$

where both ΔP and deflection are functions of x and y and deflection occurs perpendicular to the x-y plane. Each slab is broken down into subcomponents for the analysis.

ΔP is from Equation (1) and the deflection equations are:

slab **FGLI**: deflection = $y\delta/(W\eta\varphi)$
slab **AFI**: deflection = $x\delta/(W\zeta)$
slab **DGL**: deflection = $-x\delta/(W\zeta) + \delta/\zeta$
slab **ADGF**: deflection = $-y\delta/(W\eta(1-\varphi)) + \delta/(1-\varphi)$

δ may then be substituted as a function of θ_{x1}.

For $W\eta > f_{diff} > W\eta\varphi$, the calculations are illustrated for **ABF**:

ABF for $W\eta > f_{diff} > W\eta\varphi$

$$\int_{EF}^{y=f_{diff}} \int_{AF}^{BF} \Delta P \times \text{deflection} dx\, dy$$

$$= \int_{y=W\eta\varphi}^{y=f_{diff}} \int_{x=\zeta(W\eta-y)/[\eta(1-\varphi)]}^{x=W\zeta} (\Delta P_{y=0} - \rho_w g y) \times (-y\delta\delta/(W\eta(\quad -\varphi) + \delta/(1\ -\varphi)) \quad dx\, dy$$

$$= \theta_{x1}\zeta/(\eta(1-\varphi)) \times [\ \Delta P_{y=0} \times (f_{diff}^2 W\eta(1+\varphi)/2 - f_{diff}^3/3 - f_{diff} W^2\eta^2\varphi + W^3\eta^3\varphi^2/2 - W^3\eta^3\varphi^3/6) + g\rho_w \times (f_{diff}^4/4 - f_{diff}^3 W\eta(1+\varphi)/3 + f_{diff}^2 W^2\eta^2\varphi/2 + W^4\eta^4\varphi^4/12 - W^4\eta^4\varphi^3/6)\ ] \quad (11)$$

For $f_{diff} > W\eta$, the calculations follow a similar pattern. For $0 < f_{diff} < W\eta\varphi$, **ABF**, **AEF**, **BCGF**, **CDG** and **DGH** contribute zero work done because pressure is not imposed on these slabs. Otherwise, the calculations follow a similar pattern.

Summing and equating the right-hand and left-hand sides yields a function of $\Delta P_{y=0}$ and f_{diff} (or of f_{diff} and v). For minimum collapse pressure, $\Delta P_{y=0}$ is differentiated with respect to ζ and φ and these equations are set to 0:

$$\partial \Delta P_{y=0} / \partial \zeta = 0 \qquad \partial \Delta P_{y=0} / \partial \varphi = 0 \quad (12)$$

Solving for ζ and φ provides $\Delta P_{y=0\ min}$. The extra f_{diff} terms complicate the differentiation, so solving numerically is most appropriate.

5. PARAMETERS

$A = 38m^2$, $55m^2$, $84m^2$ will be taken as representative of small, medium and large dwellings in the U.K. These values were obtained through surveys of cavity wall dwellings in case study sites and represent a reasonable range for A [27].

H for a wall panel equals the height of a ground floor storey which will be taken as 2.4m. This value was also determined through field surveys of cavity wall dwellings [27]. H in this instance refers to the distance from the bottom to the top of the ground floor. For cavity wall failure, this distance strictly refers to only the blockwork inner leaf which has a continuous support where it meets the floor at the bottom of the ground floor. The brickwork outer leaf extends down to the ground (Figure 3) so that H for the outer leaf is normally 0.2-0.3m greater than H for the inner leaf [27]. If SOUTHCOMBE and TAPP [13] are correct that damp proof courses provide a nominal degree of fixity (Section 2), then this difference in H would be slightly less. Furthermore, the leaves of some cavity walls are physically attached by connecting air bricks below floor level which would add a degree of fixity to the outer leaf, reducing its effective H. Finally, the same argument from Figure 4 applies to this case. Therefore, for these calculations, H = 2.4m will be used, but the sensitivities of the results to a 0.3m variation in H will be examined in Section 7.

C_f will be taken as 0.95 and then varied to check sensitivity. $\rho_w = 1{,}026kg/m^3$ [1] and $g = 9.80665m/s^2$ [1]. $j \in \{1, 2, 3, 4\}$ represents most unreinforced masonry dwellings in the U.K. Calculations were completed for all four values of j [27], but results for only j = 1 and j = 2 are presented in this paper. Panel widths of $W \in \{1.0m, 2.0m, 3.0m, 4.0m, 5.0m, 6.0m\}$ were inferred from house plans obtained from estate agents along with books on architectural design of U.K. residences [28,29,30,31]. W = 1.0 and W = 6.0m are relatively rare but represent extremes. ψ will be taken as 0.3 to provide a conservative estimate and then varied to check sensitivity.

γ is between 500kg and 16,000kg. In order to calculate γ, three parameters are needed. First, A which has already been provided. Second, the loading from upper storeys, which varies depending on j and the panel's location within the residence. Only the dead load (the permanent mass) of the residence is considered for a conservative estimate.

The blockwork inner leaf supports the mass of the blockwork above the inner leaf of the ground floor plus the residence's dead load which is predominantly floors ($\approx 50kg/m^2$ per storey) and the roof ($\approx 70kg/m^2$). Each panel will take some fraction, λ, of this dead load. Thus, the total mass on the inner leaf is:

$$\gamma = \rho_{blockwork} HWT_{blockwork} \times (j-1) + [\ (\text{floor kg/m}^2) \times (j-1) + (\text{roof kg/m}^2)\ ] \times A\lambda_{blockwork} \quad (13)$$

For the brickwork outer leaf, the dead load has the same formula as for blockwork except that $\lambda_{brickwork} = 0$.

Third, $\lambda_{blockwork}$ is needed. This factor accounts for distribution of the non-wall dead load amongst the residence's wall panels, both internal and external. For minimum dead load $\lambda_{blockwork}$ is taken as 0.0, for average dead load $\lambda_{blockwork}$ is taken as 0.2, and for maximum dead load $\lambda_{blockwork}$ is taken as 0.4.

Other typical values used are $\rho_{brickwork} = 1{,}900kg/m^3$, $\rho_{blockwork} = 1{,}000kg/m^3$, $T_{brickwork} = 0.1025m$, and $T_{blockwork} = 0.1000m$ although variations in these choices are examined in Section 7.

6. RESULTS

All f_{diff} values presented have 0.2m added to reflect the minimum height by which most dwelling ground floors are raised. If a residence is raised more than 0.2m, then f_{diff} may easily be adjusted accordingly. Figures 9 and 10 depict f_{diff} at which a wall panel would fail as a function of W assuming that v = 0m/s. These failure f_{diff} values are not directly translatable into failure $\Delta P_{y=0}$ because the failure $\Delta P_{y=0\ min}$ changes as f_{diff} changes.

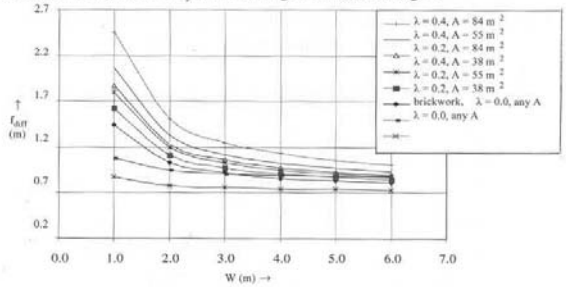


Figure 9–Failure f_{diff} vs W for v = 0m/s and a one-storey residence (curves are for blockwork unless indicated as brickwork)

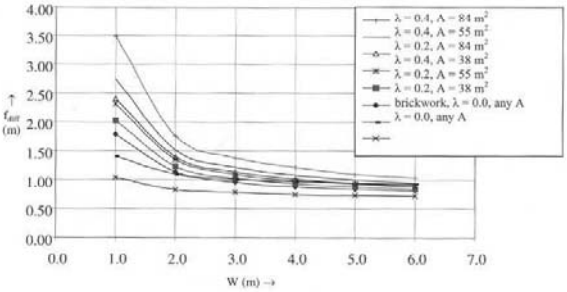
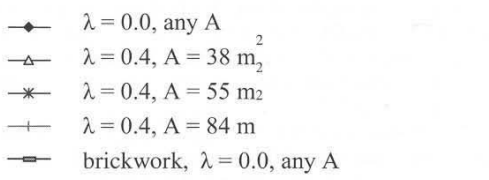


Figure 10–Failure f_{diff} vs W for v = 0m/s and a two-storey residence (curves are for blockwork unless indicated as brickwork)

In Figures 11-14, one curve has been produced for each A-j-W-λ combination where A $\in$ ($38m^2$, $55m^2$, $84m^2$), j $\in$ (1,2), W $\in$ (2.0m, 5.0m) representing narrow and wide wall panels respectively, and $\lambda \in$ (0.0,0.4) representing minimum and maximum possible failure scenarios respectively.



Legend for Figures 11-14

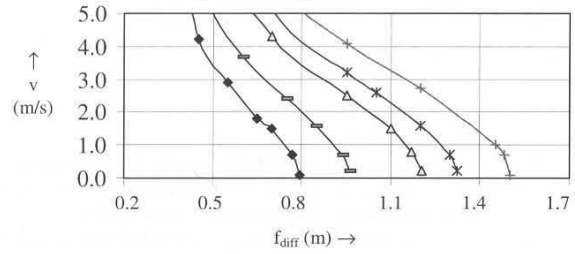


Figure 11–f_{diff}-v failure curves for a one-storey residence with narrow wall panels (W = 2.0m)

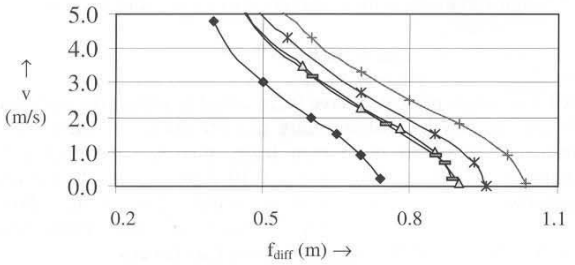


Figure 12–f_{diff}-v failure curves for a one-storey residence with wide wall panels (W = 5.0m)

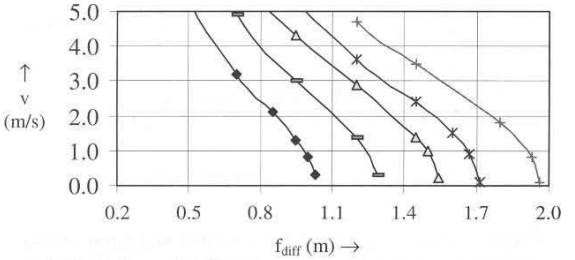


Figure 13–f_{diff}-v failure curves for a two-storey residence with narrow wall panels (W = 2.0m)

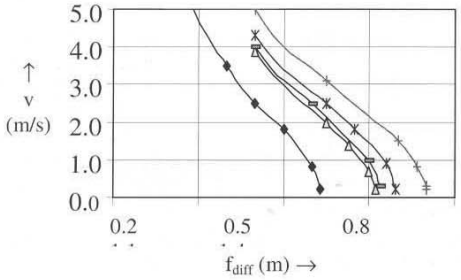


Figure 14–f_{diff}-v failure curves for a two-storey residence with wide wall panels (W = 5.0m)

The region to the right (and above) each curve is the zone of failure whereas the region to the left (and below) each curve is the zone of non-failure. The curves for a specific W may be paired according to high $\lambda_{blockwork}$ = 0.4 and low $\lambda_{blockwork}$ = 0.0. Only one curve for $\lambda_{blockwork}$ = 0.0 is provided for all A because the curves are the same for any A. The region in between the two curves of $\lambda_{blockwork}$ = 0.0 and $\lambda_{blockwork}$ = 0.4 is where failure may or may not occur, depending how much dead load the blockwork panel supports. Brickwork is provided in each graph for reference.

7. SENSITIVITY ANALYSIS

To determine the influence on the results of the parameters deemed to be fixed, variations were examined by taking for comparison a typical blockwork wall with A = $55m^2$ and j = 2. In Table 2, variables are altered individually with others being held at their typical values, except for the two dead loads which were varied +25% simultaneously and –25% simultaneously. Varying $\rho_{blockwork}$ with λ = 0.0 effectively compares brickwork and blockwork since $T_{blockwork} \approx T_{blockwork}$.

Table 2 illustrates that varying C_f and H produces negligible differences. Therefore, any potential difference in H between the blockwork and brickwork leaves (up to 0.3m, the maximum expected) has negligible effect on the results. Varying ψ and the dead loads yields a potentially significant difference, depending on other parameters. This difference is not critical because even wide variations could lead to a maximum difference in f_{diff} of only 0.3m. Nonetheless, further examination is warranted. Table 3 considers the four parameters discussed so far in the extreme cases of each parameter at its maximum value or each parameter at its minimum value.

The only case for concern is (W,λ) = (2.0m, 0.4) which displays a large difference. Since all parameters are unlikely to coincide to produce such an extreme situation, the values chosen for these parameters are acceptable for a first-order analysis.

In Table 2, varying $\rho_{blockwork}$ and $T_{blockwork}$ produces differences significant enough to warrant further examination. These two parameters are likely to be the most controllable and easily changed out of the parameters in Table 2. KELMAN [27] provides graphs of $\rho_{blockwork}$ and $T_{blockwork}$ varying simultaneously which are not reproduced here for brevity. $\rho_{blockwork}$ varying from $500kg/m^3$ to $2{,}000kg/m^3$ changed failure f_{diff} up to 0.5m for the range of $T_{blockwork}$ considered. $T_{blockwork}$ varying from 0.075m to 0.200m can double failure f_{diff} for the range of $\rho_{blockwork}$ considered. While the choices of the original values selected are supported by the literature, altering the blockwork used in residence construction has significant consequences, discussed in Section 8.

8. DISCUSSION

The range of $\Delta P_{y=0}$ under which panels start to fail runs from $8kN/m^2$ for the weakest panel to nearly $40kN/m^2$ for the strongest panel with the majority between $10kN/m^2$ and $20kN/m^2$. The range of ΔP_{avg} is generally from just over $1kN/m^2$ to approximately $20kN/m^2$ with the most common being between just over $1kN/m^2$ and $10kN/m^2$.

This range for ΔP_{avg} conforms with the range found in experimental studies of uniform loading (Sections 2 and 3), somewhat validating the method and assumptions chosen here. One outcome from these calculations is that defining a specific $\Delta P_{y=0}$ or ΔP_{avg} for a panel's failure is not possible as with uniform loading. The combination of hydrostatic and hydrodynamic pressures means that ΔP_{min} for failure shifts according to the f_{diff}-v combination. Either f_{diff} or v must be defined before ΔP_{min} for failure may be calculated.

For v = 0m/s, wall panels start failing at approximately f_{diff} = 0.8m. Most wall panels would have failed by f_{diff} = 2.0m. This

Table 2
f_{diff} for v = 0m/s varying parameters individually

Variable	(W,λ) = (2.0m, 0.0)	(W,λ) = (5.0m, 0.0)	(W,λ) = (2.0m, 0.4)	(W,λ) = (5.0m, 0.4)
$C_f = 0.85$	1.00m	0.90m	1.64m	1.16m
$C_f = 0.95$	1.03m	0.93m	1.71m	1.20m
$C_f = 1.00$	1.05m	0.94m	1.74m	1.22m
H = 2.1m	0.99m	0.90m	1.70m	1.19m
H = 2.4m	1.03m	0.93m	1.71m	1.20m
H = 2.7m	1.08m	0.96m	1.74m	1.21m
$\rho_{blockwork} = 500kg/m^3$	0.84m	0.77m	1.64m	1.11m
$\rho_{blockwork} = 1{,}000kg/m^3$	1.03m	0.93m	1.71m	1.20m
$\rho_{blockwork} = 1{,}500kg/m^3$	1.18m	1.05m	1.79m	1.28m
$\rho_{blockwork} = 2{,}000kg/m^3$	1.29m	1.15m	1.85m	1.35m
$\psi = 0.1$	0.96m	0.91m	1.54m	1.16m
$\psi = 0.3$	1.03m	0.93m	1.71m	1.20m
$\psi = 0.5$	1.09m	0.95m	1.85m	1.23m
$T_{blockwork} = 0.10m$	1.03m	0.93m	1.71m	1.20m
$T_{blockwork} = 0.15m$	1.35m	1.19m	2.10m	1.46m
floor dead load is $25kg/m^2$ roof dead load is $35kg/m^2$	1.03m	0.93m	1.43m	1.08m
floor dead load is $35kg/m^2$ roof dead load is $50kg/m^2$	1.03m	0.93m	1.56m	1.13m
floor dead load is $50kg/m^2$ roof dead load is $70kg/m^2$	1.03m	0.93m	1.71m	1.20m
floor dead load is $65kg/m^2$ roof dead load is $90kg/m^2$	1.03m	0.93m	1.85m	1.26m
floor dead load is $100kg/m^2$ roof dead load is $140kg/m^2$	1.03m	0.93m	2.16m	1.39m

Table 3
f_{diff} for v = 0m/s varying C_f, H, ψ and the dead loads simultaneously

Variable	(W,λ) = (2.0m, 0.0)	(W,λ) = (5.0m, 0.0)	(W,λ) = (2.0m, 0.4)	(W,λ) = (5.0m, 0.4)
$C_f = 0.85$ H = 2.1m $\psi = 0.1$ floor dead load is $35kg/m^2$ roof dead load is $50kg/m^2$	0.89m	0.85m	1.35m	1.04m
$C_f = 1.00$ H = 2.7m $\psi = 0.5$ floor dead load is $65kg/m^2$ roof dead load is $90kg/m^2$	1.16m	1.00	2.08m	1.33m

1.2m range is relatively narrow, indicating a fairly rapid jump across a threshold from non-structural failure to structural failure. If a residence were completely sealed and f_{diff} = 1.0m, the swell from non-breaking waves or even from a boat or vehicle travelling through the water could raise f_{diff} over the threshold and precipitate structural failure.

In considering failure of the cavity wall by the two leaves failing independently, as discussed in Section 3, the results show that the two leaves would fail at close to the same ΔP in most cases. In extreme cases, f_{diff} up to 0.5m higher may be required for the blockwork panel to fail. Because the failure pressures are so close, situations would be rare in which only one leaf fails. These pathways in Figure 3 are unlikely. An acceptable interpretation would be a binary system in which the wall panel either does not fail or fails entirely.

As noted in Section 7, the most effective means of strengthening a wall panel is increasing the density or thickness of the materials used. Bricks may be difficult to change because their size and material are ingrained in current practice [32]. Conversely, blocks are available in many sizes and densities [33]. Alternatively, two courses of each material could be used for constructing a cavity wall. Increasing thickness without altering density has the advantage of increased stability along with the advantage of increased self-weight, perhaps reflected in thickness' significant ability to change a wall panel's strength.

Caution must exercised in recommending a specific solution tailored to flood management without considering the wider context. Increasing either density or thickness increases the amount of material used which increases economic and environmental cost. Furthermore, changing only one leaf would not necessarily help in preventing the other leaf from collapsing given the uncertainties in the failure mechanism.

Another option is to increase the effective thickness of the wall panel by using ties which completely and unambiguously transfer load between panels. The issue of effective thickness of a cavity wall is tricky - FOSTER [34], HENDRY *et al.* [19], and WEST *et al.* [15] each suggest a different algorithm for calculating it - leading to the conservative assumption here that the ties do not transfer load. A feasible option is to mandate the use of ties which would definitely produce an effective thickness equal to the sum of the thicknesses of the two leaves. Care would be required to ensure that the ties are designed and installed without exacerbating other problems, such as condensation wetting the masonry.

In any case, the open cavity wall is already uncommon, with the norm being cavities partially or completely filled with insulation. Whether or not this insulation could transfer loads or increase the leaf's effective thickness depends on the material and type and amount used.

Decreasing wall panels' effective height would strengthen them. A strong, rigid connection between the panels half-way up each

storey could act as a continuous or simply supported edge. Considering a continuous edge, reducing H from 2.4m to 2.1m had inconsequential effect on the wall panel's strength (Tables 2 and 3), but using H = 1.2m may increase failure f_{diff} by up to 2.0m depending on the other parameters. Achieving this connection without sacrificing the function of the cavity or the position and size of openings would be a design challenge.

The most effective means of strengthening a wall panel would be to increase density or thickness of the materials and the most effective means of weakening a wall panel would be to decrease these parameters' values. Despite the environmental and economic costs of overusing material, wall panels are already relatively weak under lateral pressures from floods. Any movement towards reducing density or thickness for wall panels in residences should be avoided.

9. LIMITATIONS

This analysis assumed the yield line pattern shown in Figure 5. Although similar patterns are observed empirically in blockwork and brickwork (e.g. TEMPLETON *et al.* [14]), other patterns are observed and postulated. An inverted-Y (Figure 15a) and MARTINI's [23] pattern (Figure 15b) are possibilities. SINHA [24] also considers the pattern in Figure 15c, but the hydrostatic pressure component in floods may preclude such a pattern. HENDRY [18] provides different patterns for different support conditions under uniform loads.

Figure 156a Figure 15b Figure 15c

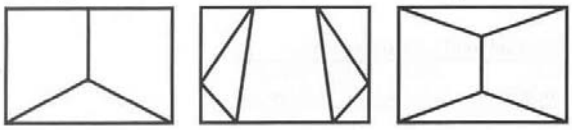


Figure 15–Alternative patterns for yield line analysis

The hydrostatic pressure component influences the pattern seen. For example, in Figure 5, φ is introduced rather than the assumption of vertical symmetry which is used for uniform load analysis. Although the yield line pattern selected for this study is an assumption, a choice must be made and the choice made is reasonable for the conditions encountered and the problem defined. For the first-order approximation required for these calculations, the choice is adequate.

The second major limitation of these results is that the wall panels considered do not have openings. Some wall panels lack openings, hence these results are applicable to some real cases. Openings may weaken a panel by creating a hole through a solid mass and by providing points for failure to begin. Alternatively, door and window frames bridge the cavity, joining the two leaves in a manner suggested in Section 8. Front doors are generally 2.0m high in a 2.4m high wall panel. The door's effect could be to split the panel, reducing the panel's width and increasing the panel's strength. Similarly, large windows may produce several smaller, stronger panels.

Finally, the failure mechanism postulated in Section 3 may be incorrect. If the ties generally do transfer load, then walls would be stronger than suggested here. Calculations, however, showed that whether or not a yield line forms at the first floor in the outer leaf influences the results negligibly. Increased weakness due to increased panel height is offset by the extra weight above the yield line which prevents the slabs from rotating.

10. CONCLUSIONS

Many residence walls would collapse at f_{diff} = 1.0-1.5m with v = 0.0m/s. With v > 0.0m/s, but at realistic values, f_{diff} for collapse can drop below 0.5m for some wall panels. These values indicate the lowest ΔP at which the panel would definitely fail, not the highest ΔP at which the panel would definitely survive.

The most effective means of strengthening walls is to increase density or thickness for both brickwork and blockwork. Nonetheless, reasons unrelated to flooding must be considered for formulating policy on strengthening residence walls.

Much research into unreinforced masonry collapse under lateral pressures has yet to be completed and caution is suggested in interpreting some of the results presented. Despite the assumptions made and the uncertainties, the method used is validated by comparison with the literature and produces results based on a plausible failure mechanism. First-order results were achieved and uncertainties were analysed.

The outcome is that the recommendations from DTLR [3] and EA/CIRIA [2] have been somewhat validated. Homeowners should be cautious about sealing their properties to a level above 0.9-1.0m above ground level. Furthermore, a preliminary understanding of the strength of residence walls, and of strengthening them, has been achieved which could be of value to homeowners, insurers, and regulatory authorities.

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