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Analysis of crack damage in three Georgian facades in the City of Bath, England, with a simple approach to the repair problem

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ABSTRACT

The field observation and analysis of four long facades demonstrates that the major component of deformation is due to shear not bending. Much of this occurs in the spandrel wall element between windows. Cracking however usually starts through tensile bending strains at the window corners; the tolerance to diagonal shear cracking being greater. A simple analysis of the spandrel shows that, if a technique for in-situ reinforcement can suppress the first bending cracks, then it could offer an extended life to the masonry without the traditional recourse to underpinning. Some preliminary laboratory tests on half-scale model walls serves to support the foregoing.

HISTORICAL BACKGROUND

In historical terms the tolerance of masonry structures to deformation has received comparatively little attention from Structural Engineers. More often the problem has been addressed by the Geotechnical engineering community, during the post-analysis stage of fault finding, in in-service buildings [1,2,3] Inevitably their specialist interest focused more on the sub-soil behaviour, which brought about some unacceptable structural movement, than the mechanics of deformation of the brick/masonry itself. Therefore they diagnosed 'cause', and merely observed a crack or no-crack situation. Through this of course, we now have at our disposal a valuable framework between, - simple empirical settlement limits to cause damage in various types of structures, and origins of damage. This offers usually safe guide-lines for foundation design of new Works. Unfortunately, it says less about what to do in those cases which are found later, to fall outside tolerable settlement limits.

The single exception in the literature, to the 'foundation cause' approach, is Burland and Wroth [4] They developed earlier work by Polshin and Tokar[5] in which they compared the masonry wall deformation to the action of a simply loaded deep beam. It highlighted the points that:

(i) the determinant for cracking of masonry is better related to strain rather than stress.

(ii) the deep beam model is able to correlate the separate thresholds of damage due to tensile strain in shear and bending, and this differentiates between normally vertical and/or diagonal crack patterns

(iii) the crack damage arising from sagging and hogging have somewhat different mechanisms; the consequence of which is that damage is worse when a foundation 'heaves', or the end or corner of the building sinks.

The current approach to crack damage repair

For buildings of some age it is normally the case for crack damage to be the result of foundation movement. Basic defects in materials, or faulty construction, come to light in the early life of the structure. Up to the present there has been a preference for repairing crack damage by isolation of its root cause, through varying degrees of improvement to the foundations. This then permits a simple cosmetic treatment of the crack damage without having to address the traditional weaknesses of the masonry itself. Commonsense though this may seem it is analogous to a medical situation where there is surgery but no medicine. The solution must be open-heart surgery without an alternative drug, special diet or pace-maker!

In this there is something of a paradox. The complaint of damage comes about most often from the owners' observation of cracks; the geotechnical literature classifies damage according to crack size. Yet it is interesting to observe that the settlement, tilt or deformation which caused the cracks is actually of secondary concern to the owner,and might otherwise have been tolerated.

RESEARCH PROGRAMME

The study reported here came about through the rising costs of maintenance and refurbishment; and its object was to gather a better understanding of the relation between loading, deformation and crack patterns in brick and masonry walling. This in turn should confirm or otherwise establish better methods of appraisal of older in-service structures at their point of maintenance or rehabilitation.

FIGURE 1:
Georgian Facades
Pulteney Street, Bath
L.H.S: Nos.42 - 52
R.H.S: Nos.66 - 77

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With masonry the appraisal process usually starts at the point of cracking and the foregoing was carried out through:

(i) A series of field studies of settlement crack damage to relate the patterns of cracking to the mode of deformation.

(ii) An investigation as to how existing masonry cracks might be repaired, in a way which would heal crack scars so they did not re-appear if there were further structural movement. In this respect a programme of laboratory tests were undertaken to examine the behaviour of the spandrel wall detail

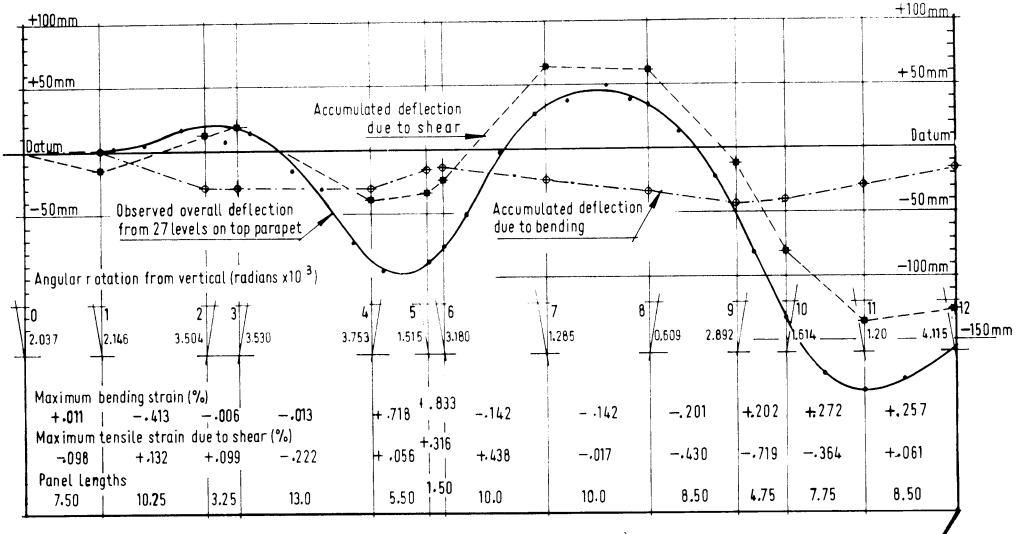
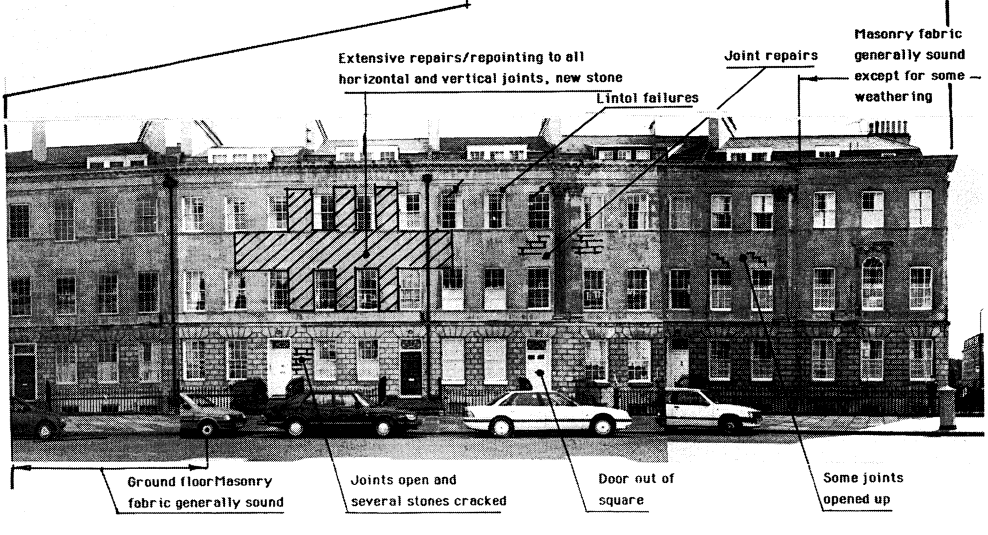


FIGURE 2A: Settlement Profiles, 42-52, Great Pulteney Street,Bath



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FIELD STUDIES OF THE SETTLEMENT OF LONG FACADES

For investigations into the analysis of damage, a long facade length offers certain advantages over shorter structures. They have a regular pattern of doors, windows, etc., which provide a useful common denominator when comparisons are to be made between different settlement modes, crack patterns and damage generally.

On these grounds four facades were selected, three were Georgian terraces in the City of Bath, and the fourth a 1960's flat block development in Grangemouth, Scotland. The Georgian character of Pulteney Street can be appreciated from fig.1. Historic settlement draws attention to itself by the waving corbel lines. They were built with deep basements on a causeway above river flood level.

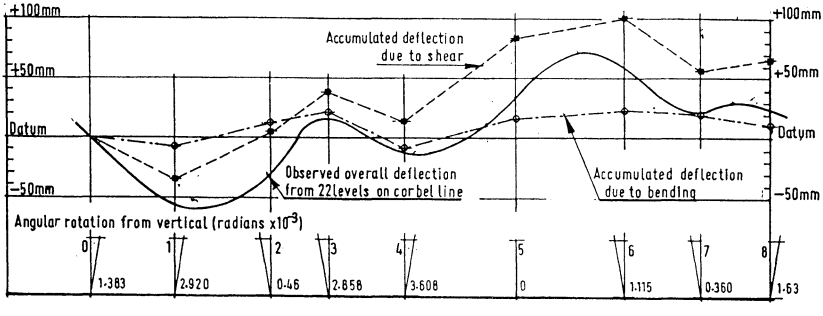

FIGURE 3: Settlement Profiles.. 29 - 40 Great Pulteney Street, Bath

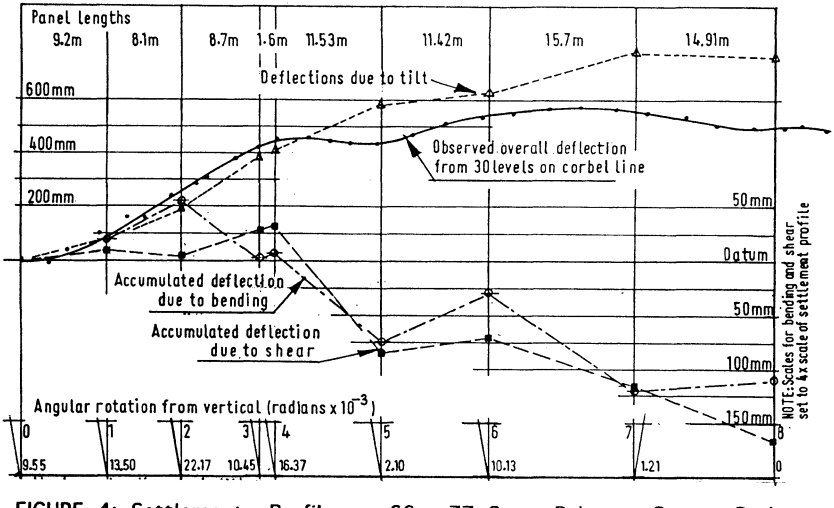

FIGURE 4: Settlement Profiles... 66 - 77 Great Pulteney Street, Bath

(i) Overall and separated deflections: Total= Shear + Bending + Tilt
(ii) Angular rotation of points on Facade (given in Radians x 10^{-3})

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The flats at Grangemouth had been the subject of earlier investigations by Green, Macleod and Stark(6) and by Macleod and Abu-el Magid(7), because significant settlement had been anticipated at the construction stage.

In all cases the structures revealed quite large differential settlements of 60mm to 550mm in their extremes, and the deflection profile showed several reverses of curvature. The entire data collection and a fuller analysis is given in reference (8). Examples of the observations and derivations of shear and bending strains derived from it, are shown for the Bath Terraces in Pulteney Street, as follows:

Figs.2 and 2A: Nos. 42 - 52 Fig. 3: Nos. 29 - 40 Fig. 4: Nos. 66 - 77

The procedure was to take about 30 level measurements along each 90m facade length. In addition, a theodolite was used to estimate the degree of vertical misalignment or rotation of the structure, by setting up square to the gable ends and at 8 to 12 intermediate positions. At some points features in the facade offered good reference lines or otherwise an average line was taken through the centres of a column of 3 or 4 windows. The criterion was to choose features which had the minimum disturbance to the original construction.

Separation of components of bending, tilt and shear

The degree of vertical mis-alignment of the structure, in addition to the settlement profile, is a parameter usually neglected by Geotechnical and masonry engineers in their normal investigations. However its inclusion completes what is necessary to fulfill a simple beam deformation analysis. Taking sequential sections in turn the relative contributions due to Tilt, Bending and Shear can be isolated as shown in fig. 5. If the adjacent sides of a panel unit are tilted but remain parallel then the deformation can only be due to body rotation and/or shear. A bending deflection is present only if there is a change in vertical mis-alignment; this being equal to twice the relative rotation or angular distortion from the earlier definitions. If Tilt is taken as the rotation of the left side of the panel unit, then:

Shear deformation = [Total level change] - [Bending] - [Tilt].....eqn(1)

Using a consistent sign notation it is then possible to progressively work out the accumulated deformations and strains from bending and shear with the body rotation (Tilt) removed. An example is given diagrammatically in fig. 6. It should be noted the sign convention is given positive if the right hand side of the panel unit lifts upwards with respect to the left (denoted as sagging). In the reverse it is denoted as hogging, so hogging is the condition for tensile bending at the roof level as previously defined by Burland. The three examples of the Pulteney St. surveys (figs. 2, 3 and 4) set out:

(i) The differential settlement profile from a left-hand end datum. It is smoothed in as a full line. (ii) The angular rotation in radians (ω) (iii) The maximum calculated bending and shear strains (e_b, e_s), and also,(iv) The panel lengths.

The heights of the Georgian facades were 15.2m. (to basement)

Using the method described, the deflections due to tilt, bending and shear within each panel unit were derived and the accumulated deflections for bending and shear then plotted alongside the differential settlements.

The prime conclusion from the fieldwork analysis was that multi-windowed facades have very little stiffness in shear. In simplest form it is seen by the fact that

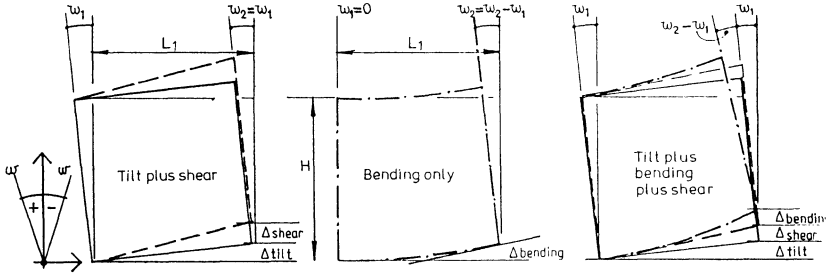

FIGURE 5:
Modified Definitions for separation of components of Bending, Tilt and Shear

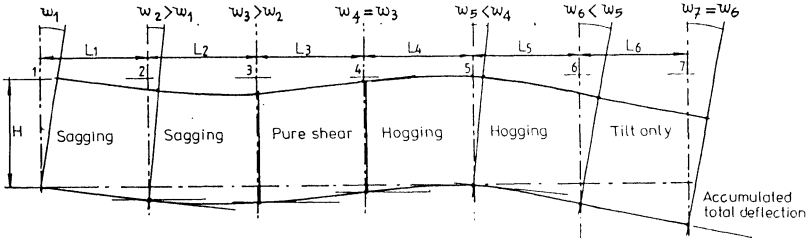

FIGURE 6: Example of Survey Plot

Note on sign convention:
If..... $w_{n+1} > w_n$, then the deflection mode is Sagging (plus Tilt and/or shear)
If..... $w_{n+1} < w_n$, then the deflection mode is Hogging (plus Tilt and/or shear)
If..... $w_{n+1} = w_n$, then the deflection mode is Tilt and/or shear only
Shear deformation = [Total measured level change] - [Bending] -]Tilt]

in two cases the accumulated deflections due to shear are materially greater than the bending, and in the third they are equal. It may be of little surprise that Burland had earlier recognised this, and tentatively suggested the feature might be compensated by an imaginary relationship between shear and bending stiffness (E/G) of 12.5 (vis-a-vis 2.6 for a solid isotropic wall). In Cook's[8] evaluation of the same, a comparison was made with the shear/bending deflection ratios, at points along an isotropic beam of similar proportions to the facades (H/L = 0.69), with varying load/reaction distributions which also gave reverses of curvature similar to Pulteney Street. Although the numbers in all instances were rather scattered, the latter were seen to come in to soft focus if the beam were assigned an imaginery stiffness ratio of, E/G = 30.

ANALYSIS OF THE SPANDREL WALL ELEMENT

The surveys showed the accumulated shear deflections to be 2 to 4 times greater than the bending. Much of this must occur within the spandrel wall elements above and below windows, which are most susceptible to damage, particularly at the upper floor levels where there is less gravity weight (see fig. 2A). Therefore the actual

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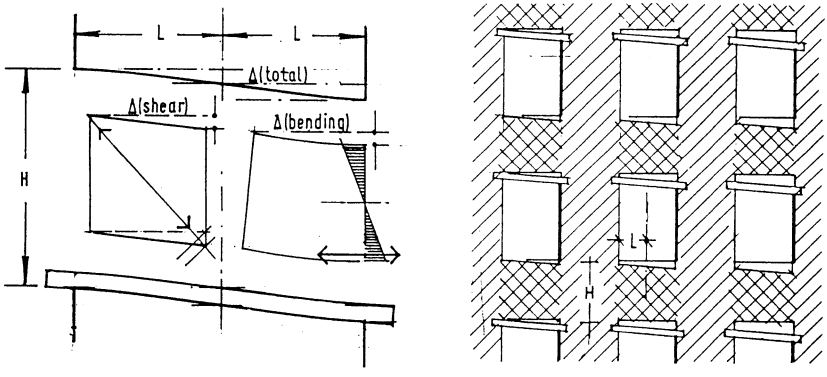


FIGURE 7:
Deformation of the spandrel wall element in a masonry facade

deflection profile might be regarded as a series of short steps between window and door openings, shown diagrammatically in fig.7.

From fig. 7, it is possible top make a simple analysis of the spandrel if it is assumed it deflects in shear as a fixed ended beam; and it is sufficiently accurate to assume plane sections remain plane. By symmetry, the half span acts as a simple deep cantilever, and the ratio of shear to bending deflection is:-

$$\frac{\Delta\,(\text{shear})}{\Delta\,(\text{bending})} = \frac{3.E}{8.G}\left[\frac{H}{L}\right]^2 \qquad \text{eqn (2)}$$

Assuming that before cracking the ratio, E/G = 2.5(approximately), then for a normal structural beam the shear effect is negligible. But at, $^H/_L = 1$ the two are equal; and at

$^H/_L = 1.8$ the shear deflection is 3 times the bending, which is the in the range of the geometry of a spandrel wall.

Now the shear stress/strain remains constant along the length whilst the bending strain varies linearly with the moment. The ratio of

$$\frac{\text{Bending strain}}{\text{Shear strain}} = 8\left[\frac{G}{E}\right]\left[\frac{L}{H}\right] \qquad \text{eqn (3)}$$

This is shown diagrammatically in fig. 8. It points out that unless the cantilever half span is very deep ($^H/_L = 3.2$), then the element will always reach a tensile strain limit in bending first, which is before the shear limit. Therefore the first cracks are in a sub-vertical direction at the window corners. One such instance of this is at, Grangemouth, Scotland, shown in figure 9. It is interesting to observe here also, how the lintol beam serves to act as a reinforcing element as it serves to divert the crack direction.

The relevance to masonry repair work.

Fig. 10 gives the graph of deflection ratios, Δ/L versus $^H/_L$, for shear and bending, for a limiting tensile strain of 0.075%. It is similar to the method of presentation by Burland (4). The point of this graph is to show that if the spandrels could be reinforced against the bending strain, so that any continued movement is

suppressed and/or dispersed (point A), there still now remains a quantifiable measure of residual tolerance to shear (point B), before the onset of diagonal shear cracking.

In this respect the last two decades have seen the successful introduction of reinforcement into new masonry structures, similar to reinforced concrete, and its use is now commonplace. Given a reliable theoretical basis and the techniques for effecting in-situ masonry reinforcement, then in many cases some option on the principle of fig.10 could offer a viable and cheaper alternative to underpinning.

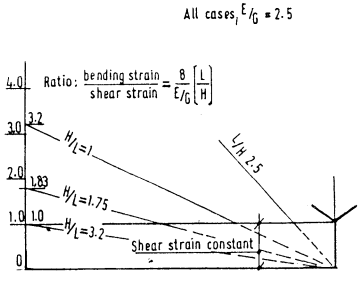


FIGURE 8:
Ratio: Bending and shear strains in a cantilever as a function of, H/L

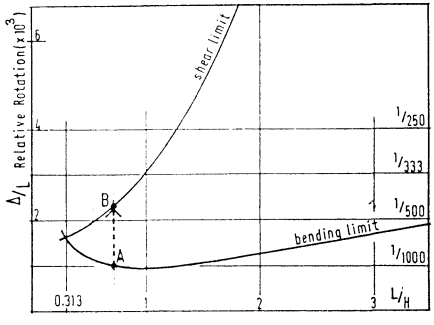
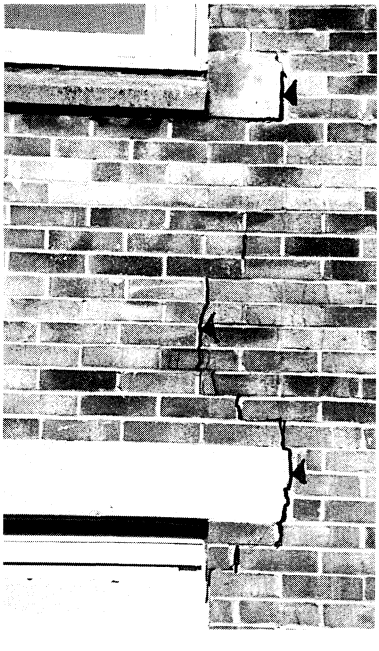


FIGURE 10:
Relative rotation Δ/L versus H/L, limiting tensile strain of, 0.075%



Flat Block at Garry Place, Grangemouth.
Fig.9A (above):
Settlement along south face of block.
Fig.9B (left):
Shear cracks from window corners

However at the present, it is unfortunately the case that the ordinary practising engineer dealing with building settlement, is very reluctant to prescribe reinforcement of the fabric, as an alternative to underpinning. Through inexperience, lack of precedent, or the anxiety of litigation, he/she has not got the confidence it will work well enough to stop future cracking. In consequence there still seem to be few credible middle-ground options between a major underpinning operation on a cracked structure, and simple cosmetic repair. The medical analogy has already been referred to whereby there was surgery but no medecine. A heart attack must be cured by a new heart; there would be no options for drugs or a pace-maker.

LABORATORY TESTS

The next step in the study was to observe whether the foregoing could be verified in some simple manner. A half-scale model brick/masonry wall was therefore designed to an H-shape, chosen to represent the head and sill element of a masonry facade. The principles are given in fig.11A, and two typical assembled wall units are shown in fig. 11B. It was built on the the University's Laboratory strong-floor between two specially fabricated channel frames providing resistance to rotation. It was loaded simultaneously from bottom-right and top-left to give a deflection mode shown in fig.11A. The side panels to the model window openings (bottom left and top right) were pre-loaded to a constant pressure to simulate the gravity effect of 2 or three floors of brickwork. It was achieved through a pair of reinforced pressure bags connected to a calibrated pressure line. The load was applied under strain control through load cells, to which a screw-jack system had been attached.

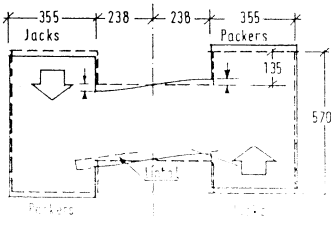


FIGURE 11A: Diagrammatic Layout of the Laboratory spandrel wall test

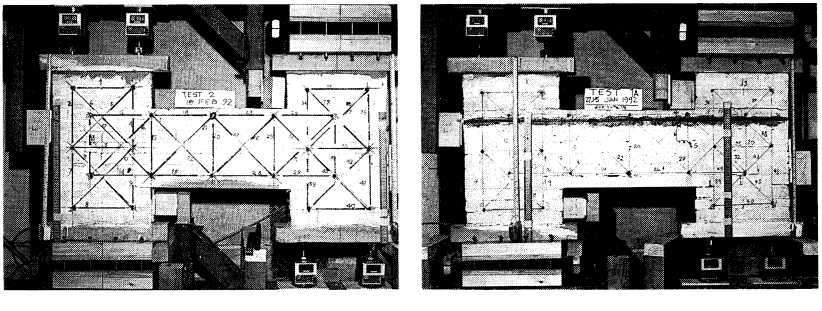


FIGURE 11B: Assembled model wall showing pattern of demec studs for strain measurement, set out on rectangular grid. Measurements included sides and diagonals to define the distorted shape of the central section of wall (9)
LEFT: Typical wall before test. RIGHT: Reinforcement in place after first test

In all cases deformations were measured externally by means of a 'Wild' precision level; but more accurately through the computer logging of the strain grid measurements across the wall face, which enabled the deformed geometry of the wall itself to be defined. The trace of the strain grid and the diagonals can be seen in the photographs, figs.11B, 11C. The laboratory measurement technique had been developed earlier(9), and through this the software package developed by the second author quantified the deformed shape and, importantly, the principal internal strains in the central section. These could then be observed at any stage of a test in the manner shown by figs. 12A and 12B.

7 model walls were fabricated, each being tested twice. In the first stage the wall was unreinforced and taken to the point at which the first visible cracks became evident at the window corner. The load was then relaxed and the model restored to its original position. A 4mm dia mild steel bar was then resin bonded below the window cill, fig.11C; this being the natural point of first cracking. If cracks were evident at the diagonally opposite corner (by the lintol beam), then a second bar was placed across this point before the re-test..

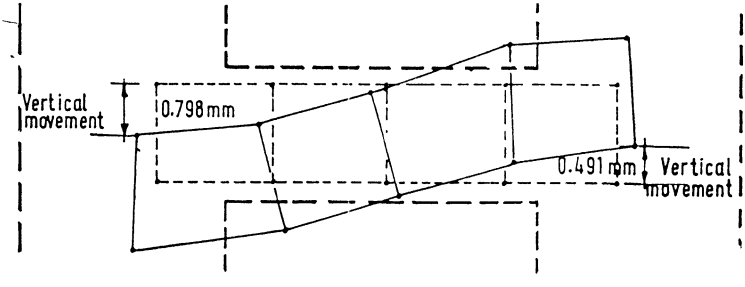


FIGURE 12A: Spandrel wall test 1A (trace of computer display)
Typical deformed shape of spandrel from strain grid measurements

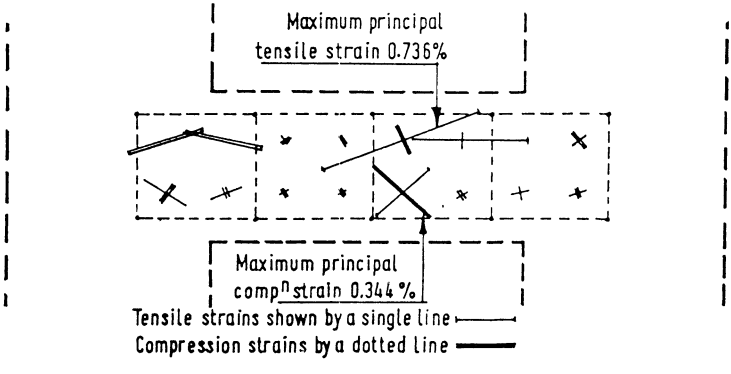


FIGURE 12B: Spandrel wall test 7B (trace of computer display)
Graph plots of principal tensile and compressive strains around node points

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Results

In general terms initial cracking was clearly visible at a semi-displacement of 1.5 to 2mm, or a deflection ratio Δ/L of 1/160 to 1/120. The cracking was at the corner(s) and due to bending not shear. There is a problem comparing these figures with a prototype wall size because of the scale effect of using half-size bricks, which distorts the true stiffness, and was earlier reported in ref.(10). The figures themselves must therefore be regarded as somewhat arbitrary. However, some comparisons are useful. Table I below gives the actual principal tensile strains at the top right (spandrel position) or bottom left (Lintol Position) at which the models first showed clear cracks, and then the strain at which a new crack developed - or the old re-opened - after installing the reinforcement. In all but one case, there was a measurable increased tolerance to movement.

Test No.	Location of onset of cracking Initial test (unreinforced)	strain(%)	Re-test (reinforced)	Strain (%)
2	lintol, .L.H.S.	0.25	spandrel, R.H.S.	0.43
4	spandrel, R.H.S.	0.13	lintol, L.H.S.	0.50
5	spandrel, R.H.S.	0.22	spandrel, R.H.S.	0.16
6	spandrel, R.H.S.	0.18	spandrel, R.H.S.	0.38
7	spandrel, R.H.S.	0.26	spandrel, R.H.S.	0.61

Table I:
Summary of principal tensile strains at which the models first showed clear cracks,

The foregoing results are currently under more detailed examination and will form the subject of a later publication. However the preliminary findings serve to confirm the substance of the argument that there is a case for addressing the problem of crack repair, in a way that will tolerate small further movements by suppressing a direct re-opening of the old wound.

CONCLUSIONS

The literature relating to the tolerance of masonry to cracking has left a wealth of valuable guidelines for new construction, but less about the way in which the inherent weaknesses of the material can be compensated once the damage has appeared.

In the field studies of long facades reported here, a development of Burland and Wroth's simple deep beam model has enabled a separation to be made between the components of tilt, bending and shear, within the total settlement profile. Shear deformation is seen to provide an abnormally high contribution because of the reduced shear stiffness brought about by the presence of window openings.

However, an analysis of the spandrel wall element shows that cracks from tensile bending strains are likely to occur first. Some preliminary laboratory tests indicate that, by insertion of suitable reinforcement, these can be suppressed so that further deformation does not cause an immediate re-opening of the old wound.

/continued

The relevance of the foregoing rests on the prospect of a viable alternative to underpinning a structure, in some cases, where it may not be necessary to cure cause. At the present time the ordinary practising engineer is reluctant to do this. Through inexperience, lack of precedent or the anxiety of litigation he/she has not got the confidence it will work well enough to stop future cracking. Presently there are few middle-ground options between simple cosmetic repair and a total foundation rebuild.

ACKNOWLEDGEMENTS

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