DISCUSSION

SEISMIC BEHAVIOUR OF UNREINFORCED MASONRY WALLS

M. J. N. Priestley

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INTRODUCTION

The important subject of strength assessment of existing masonry structures and the formulation of procedures for securing against risk has drawn strong and growing interest in recent years. Dr Priestley's article, then, will have attracted the attention of many engineers involved in this area of earthquake engineering.

Suitably conservative rules which may be appropriate for the design of new structures may not be appropriate for review of existing ones. In the former case application of a reasonable degree of conservatism may impose only minor economic penalty. But in the case of existing buildings a similar degree of conservatism may lead to encroachment across the threshold between "need not strengthen" and "need to strengthen" and result in undue cost. Procedures which reflect due caution yet recognise stabilising mechanisms not normally accounted for are therefore to be encouraged.

Such procedures should preferably be formulated through easily understood and familiar concepts in which the essential ingredients of behaviour and response are described in physically appealing ways. A formulation based on the principle of conservation of energy as employed by Dr Priestley is certainly direct, and employs concepts which are familiar and easily understood. However, care must be exercised to ensure that the correct interpretations are applied, and that, to assist in understanding, recognisable units of energy are preserved throughout the argument. These objectives do not appear to have been met in Dr Priestley's article, and, as a consequence, the results are both erroneous and misleading. It is with a view to examining the general method proposed by Dr Priestley and the validity of that method that this review is entered.

SCOPE OF THE DISCUSSION

Dr Priestley's paper is rather too wide ranging to be successfully addressed in a discussion of this nature. So, to allow a specific issue to be addressed, the response of face-loaded walls only will be examined, using the same fifth storey wall panel illustrated in Dr Priestley's numerical example for reference.

Many engineers will be surprised by the results of that example which indicate that a rather thin (220 mm) and tall (5.00 m) simply supported unreinforced masonry wall in the fifth storey of a building of relatively long fundamental period (0.635 s) will survive an earthquake with about one-half the spectral acceleration of the N-S component of the 1940 El Centro earthquake, particularly in view of the poor past performance of such elements.

The boundary conditions employed in the example will be adopted in this discussion, but it should be recognised that this simplification is not a realistic modelling of the actual wall of which the panel is a component. Even as Dr Priestley's examples show, the example panel will fail at a much lower level of response than lower panels. Such failure cannot occur without the development of significant moment and attendant rocking at the fourth floor level. Indeed, the entire wall should be treated as one entity, even recognising the possibility of out-of-phase response through adjacent stories. The modelling is adopted, however, to allow of direct comparisons of numerical results, and might be viewed as representing a panel supported on a flexurally stiff but torsionally flexible beam at the fourth floor level. For similar reasons, the effect of damping will not be included in this review, but such effects can quite simply be included within the modelling suggested.

One exclusion from Dr Priestley's assumptions concerns vertical ground accelerations, here ignored. Dr Priestley's approach in essence assumes a constantly applied vertical acceleration. Since vertical ground acceleration effects will be propagated through the structure at frequencies close to the predominant frequency of the ground motion, his assumption would appear to be too conservative to be realistic, and it has been abandoned in this discussion.

NOTATION

With minor exception, the notation employed in this discussion follows that used by Dr Priestley. Additional notation will be described as it is introduced. Where Dr Priestley's equations are quoted they are referred to thus: Eq (11); equations introduced in this review are prefixed with "A", thus: Eq (A3). Figures are similarly referenced.

ACCELERATION - DEFLECTION DIAGRAM

The method used in the production of the acceleration/deflection diagram shown in Fig 17 is based on familiar concepts, but the result is presented in an unfamiliar way. Since the graph is derived directly from moment/rotation

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considerations, the area may be viewed as the potential energy of the system at maximum displacement, albeit in unfortunate units. This potential energy is composed of strain energy, S, due to induced stresses, and load potential, L, due to raising of the weights above their stable equilibrium position.

Unfortunately there is an error in the treatment of the P-delta effects in the paper. The self-weight of the wall, W, being eccentric to the supports because of wall displacement, requires horizontal reactions at the floor levels to maintain equilibrium of the wall. The moment at the mid-height of the wall due to P-delta effects, is therefore

$$M(\text{P-delta}) = (P + \frac{W}{2})\Delta \qquad (A1)$$

and not

$$M(\text{P-delta}) = (P + \frac{W}{4})\Delta \qquad (A2)$$

as is inferred from Eqs (11) and (12), a result apparently erroneously drawn by neglecting the horizontal reactions induced by W.

It can therefore be shown that the maximum deflection at the mid-height of the wall, Δ_u, is given by

$$\Delta_u = \frac{t - a}{2} \qquad (A3)$$

and not, as according to Dr Priestley, by

$$\Delta_u = \frac{t - a}{2} + \Delta' \qquad (11)$$

Indeed the formulation using R, y_R and Δ' complicates the problem unnecessarily. The result of this error in the treatment of P-delta effects is that the potential energy is overestimated. This is a fairly serious overestimate even in this case, and would be very serious in the absence of P, the load at the top of the wall.

Fig A1 shows the corrected curve, plotted as M-θ, so that the area is directly representative of energy without further conversion. Note that the curve does not pass through the theoretical maximum rotation, an outcome directly attributable to the approximation employed in determining the M-θ relationship, which is that the mid-height deflection is directly proportional to the mid-height curvature, as in Dr Priestley's article, and because it is possible for stationary potential to exist before the attainment of the maximum deflection, especially where Young's modulus is small. Also plotted in Fig A1 are additional graphs for greater values of Young's modulus, E. A value of 1 GPa used by Dr Priestley is abnormally low, especially since most deformation occurs through the mortar

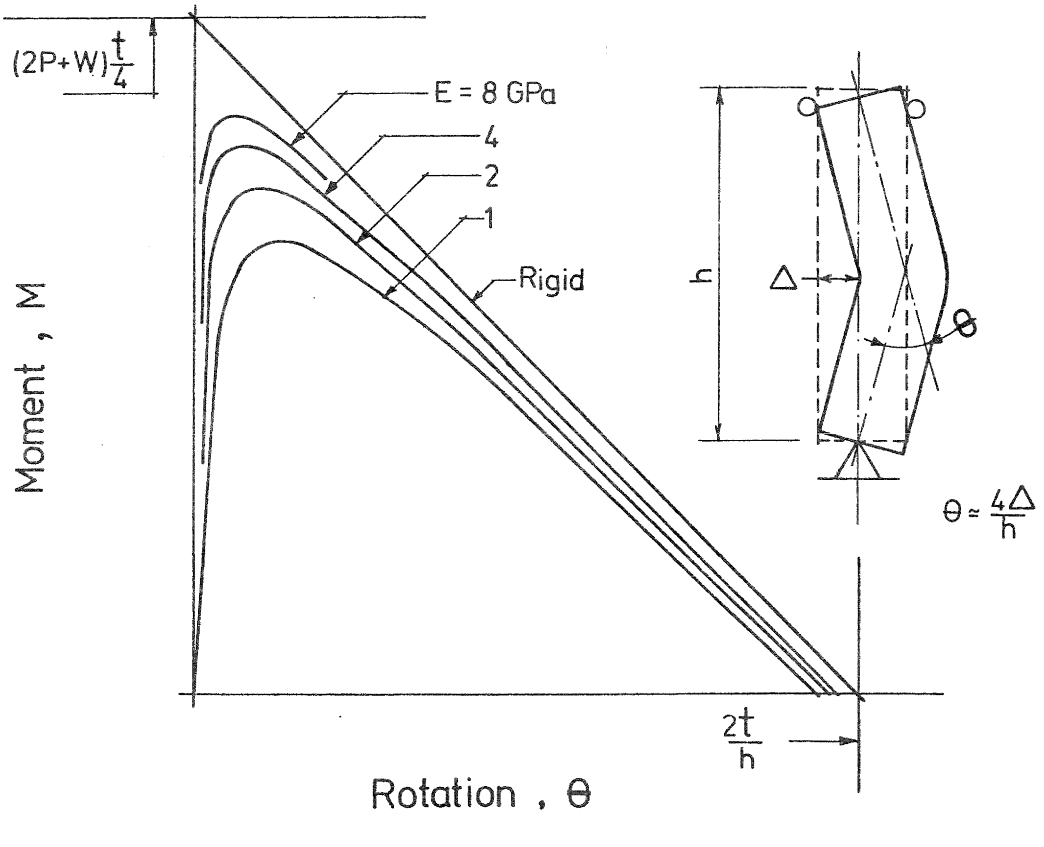

Figure A1: Moment-rotation curve for face-loaded wall which represents as little as 10 to 15 percent of the total height of the wall. Perhaps a more realistic value would be 10 GPa or greater; in any event the graphs quickly approach the rigid case. The approximations employed in constructing these graphs leads to an under-estimate of the total potential energy of the system, indicating that analytical results based on the assumption of rigidity might be acceptable. With that assumption the total potential energy of the system, T_p, is composed entirely of the load potential, L, since strain energy S is zero and T_p is given by

$$T_p = L = (2P + W)\frac{t^2}{4h} \qquad (A4)$$

where T_p and L are both derived at the maximum displacement of the wall. Eq (A4) uses half the thickness of the wall, t/2, as an approximation to the internal level arm. This simplifying assumption is used to demonstrate without undue algebraic complication that T_p given by Eq (A4) can be as simply derived as the work exerted against the loads P and W in moving them into the deformed position of the wall shown in Fig (A3b). Assuming small slopes of the blocks:

Point 1 moves down $\frac{1}{2}\left(\frac{t^2}{4h}\right)$

Point 2 moves up $\frac{5}{2}\left(\frac{t^2}{4h}\right)$

Point 3 moves up $2\left(\frac{t^2}{4h}\right)$

The potential energy in this configuration is therefore

$$T_p = \frac{t^2}{4h}\left[2P + \frac{5}{2} - \frac{1}{2}\frac{W}{2}\right]$$

$$= (2P + W)\frac{t^2}{4h}$$

as given by Eq (A4).

As stated above, the method of determining the graphs in Fig A1 underestimates the potential energy. The cumulative area under the 1 GPa graph is shown in Fig A2 along with graphs produced without the assumptions employed for Fig A1. Fig A2 is a plot of total potential energy, T_p, against the mid-height deflection, Δ, with T_p shown separated into strain energy and load potential components, both referred to the stable (at-rest) equilibrium position as datum. Invoking the principle of stationarity of potential, the graphs confirm that the maximum tolerable displacement is one-half the wall thickness, and since T_p is a maximum, the equilibrium at that displacement is unstable, as expected.

Fig A2 shows that strain energy is a small component of the total potential energy, and confirms that assumptions of rigidity will be acceptable.

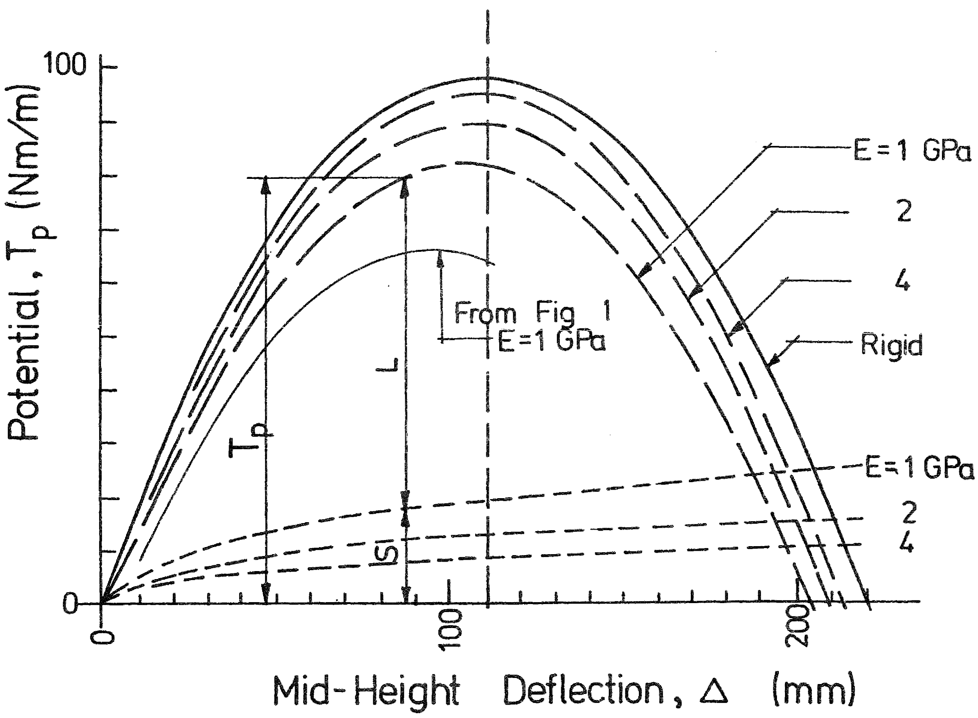

Figure A2: Potential energy vs deflection for face-loaded wall

The reason for this alternative formulation is to emphasise the physical nature of the quantities involved in a more familiar way. It is also a simpler method to use when slopes are not small, it draws attention to the vertical and rotational deformations and associated velocities and accelerations induced by wall deflection, and it introduces the functional relationships which are useful in the development of more general methods.

EQUIVALENT LINEAR ELASTIC SYSTEM

Through Eq 25 Dr Priestley claims to derive an equivalent linear elastic response which will induce failure in the non-linear elastic wall. This proposition is offered without explanation, so it is as well to examine what is in fact derived.

Firstly we should recognise that Eq 25 "appears" wrong, because of the relationship to k, and therefore to E. As was found above, potential energy is not strongly related to E.

In the present form of Eq 25, k is directly related to the initial stiffness of the wall, and A_2 may be interpreted, as for A_1, as potential energy.

An undamped system, in free vibration, possessing at maximum displacement the potential given by Eq A4, will possess this same energy as kinetic energy as it passes through the origin. This applies to all such conservative systems, including linear elastic ones, although the maximum accelerations will depend on the force-displacement characteristics of the system. Ignoring, for the present, that A_1 was derived assuming a uniform distribution of inertial loading, rather than an inertial loading which is distributed in proportion to the displacement as shown in Fig A4(b), we can answer, through Eq 25, the question: "What maximum acceleration will a linear system experience when released from a position of known potential energy, and allowed to vibrate freely?" This is not, however, the question which should be addressed. We are concerned with forced vibrations. Dr Priestley cogently describes the support motions elsewhere in his paper, pointing out that the frequency of the structural response will dominate the excitation at the level of the supports. But the logic of applying that excitation appears to be lost. And in the introduction to the paper, quoting others, he points to the trend of results showing the importance of spectral velocities. That, too, seems to have been ignored.

It is possible to validly interpret k, otherwise than as above, as a pseudo-stiffness, k^*, given by

$$k^* = \omega^2 W/g \qquad \text{(A5)}$$

where ω is the first mode frequency of vibration of the structure. k^* is thus that value of stiffness which would allow the wall as a single degree of freedom system to vibrate in simple harmonic motion at frequency ω. With obvious meaning attached to A_2^*, putting A_1^* equal to T_p in Eq A4, and equating A_1^* to A_2^* we have

$$\frac{(\ddot{a}_e W)^2}{2k^*} = A_2^* = A_1^* = (2P + W)\frac{t^2}{4h} \qquad \text{(A6)}$$

Substituting for k^* from Eq A5 and solving:

$$\ddot{a}_e = \frac{\omega t}{\sqrt{2gh}}\left[\frac{2P + W}{W}\right]^{\frac{1}{2}} \qquad \text{(A7)}$$

This equation clearly gives lower values of $\ddot{a}_e$ than those calculated by Dr Priestley, but, in that Eq A7 captures the essentials of structural frequency and wall characteristics, the values of $\ddot{a}_e$ calculated from it are more meaningful.

What we have not, thus far, discussed is the effect of amplification of support accelerations within the wall. Essentially we have solved the system shown in Fig A3(c), with no amplification. But amplification of accelerations will exist to some degree for the true collapse configurations shown in Fig A3(e), or, more simply, approximated in Fig A3(d).

It is not appropriate to assess the response on the basis of wall period calculated from maximum displacement, at which collapse will be incipient. Indeed, at that displacement the "period" is infinite. Since displacement of one-half the maximum tolerable will indicate significant response, with failure probable in the next half-cycle, period based on that displacement is more reasonable. For a displacement of one-half the maximum tolerable, the period of the wall panel will be about 1.0 s, not greatly removed from the structural period of 0.635 s. Significant amplification is therefore to be expected in this case.

To determine the effect of amplification, it is simpler to revise the method of calculating response, equating potential energy to kinetic energy directly, rather than through an equivalent linear elastic system as was undertaken above to preserve coherence in the argument thus far. This we shall now do.

REVISED ASSESSMENT OF MAXIMUM SUSTAINABLE RESPONSE

The kinetic energy T_k in the wall just before it passes through the origin may be written as

$$T_k = \frac{1}{2}\left(\frac{W}{g}\right) f(\dot{u})^2 \qquad \text{(A8)}$$

where $\dot{u}$ is the mean velocity of the supports, and f is a function of the velocity profile.

For the conditions described in Dr Priestley's paper, an acceptable approximation for the mean support velocity $\dot{u}$ is given by

$$\dot{u} = \frac{\lambda S_a^*}{\omega} \qquad (A9)$$

where ω is the first mode frequency of the structure, S_a^* is the maximum sustainable spectral acceleration, and λ is the amplification of accelerations at the supports. Ignoring energy losses on impact through the origin, T_k given by Eq A8 will also exist just past the origin.

Substituting Eq A9 into Eq A8 and equating the result to Eq A4 yields

$$\frac{\lambda S_a^*}{g} = \frac{\omega t}{\sqrt{(2gh)}} \left[\frac{2P + W}{Wf}\right]^{\frac{1}{2}} \qquad (A10)$$

This Eq A10 is identical to Eq A7 derived earlier, with f = 1.0, and $\ddot{a}_e = \lambda S_a^*/g$, as is to be expected.

Using the same values as in Dr Priestley's paper:

P = 10 000 N

W = 20 500 N

b = 0.22 m

h = 5.0 m

ω = 9.89 radians/s

λ = 1.17/0.44 = 2.66

we have

$$\frac{\lambda S_a^*}{g} = 0.31/\sqrt{f} \qquad (A11)$$

The probability of amplification of support velocities within the wall is high. Computer simulations undertaken by the writer in the past indicate that the velocity profile at zero displacement tends to the displacement profile at maximum displacement. Some typical displacements in the example wall are indicated in Fig A3. The support displacement, where shown, is equal to the amplified structural spectral displacement, and the displacement at the mid-height of the wall relative to the average support displacement will also nearly equal this same structural spectral displacement in this case (due to numerical accident: this will not always be so). Fig A3(e) shows the configuration for this example panel at collapse. Using these approximations and that the velocity profile is as for the displacement profile, the values of f shown in Fig A3 are derived. Note that f is replaced by f' in Fig A3(e), f' accounting for the reduced potential for that particular configuration. Using these values of f, values for S_a^*/g shown in Table A1 are found.

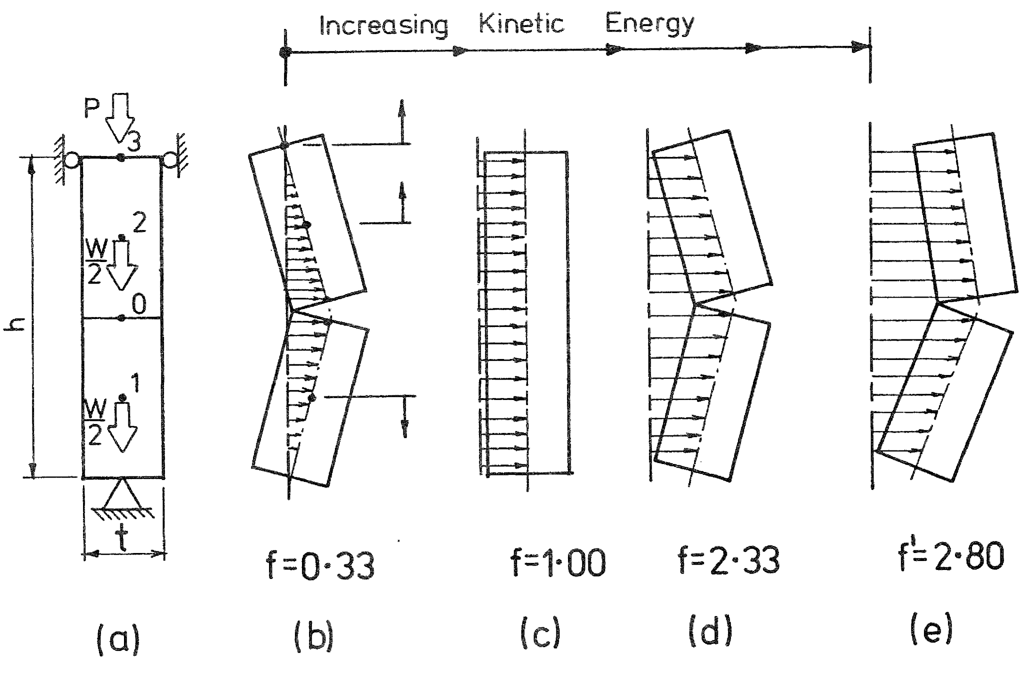

Figure A3: Possible displacement profiles at failure for example wall

Table A1: Maximum sustainable spectral accelerations for various velocity profiles

Configuration	f	S_a^*/g	$\lambda S_a^*/g$ (Eq A11)
Fig A3 (b)	0.33	0.21	0.56
Fig A3 (c)	1.00	0.12	0.31
Fig A3 (d)	2.33	0.08	0.20
Fig A3 (e)	2.80	0.07	0.18

As expected, the configuration shown in Fig A3(b), which is consistent with free vibration, yields a value of about one-half of the required spectral acceleration, close to that calculated by Dr Priestley because of his misconstruction of the problem. The configuration shown in Fig A3(c), which involves no amplification and which is consistent with the assumptions of mean support motions used by Dr Priestley, is the best that can be expected. However, the configuration of Fig A3(e), which is suggested as more likely, yields values of sustainable spectral acceleration considerably less. This rather alarming reduction from Dr Priestley's results requires that further assessment of likely response be undertaken.

COMPUTER SIMULATION

To assess the validity of the foregoing, several analyses were performed on the wall panel modelled as described above. Input motions consisted of random wave forms superimposed over a sinusoidal wave of frequency equal to the first mode frequency of the structure. The maximum sustainable accelerations were then found for each ensemble form. Results are plotted in the histogram shown in Fig A4. The correspondence between these values and those calculated earlier is striking, and leads support to the view, intuitively natural, that considerable amplification of support motions is likely.

CONCLUDING REMARKS

Since "yielding" of any panel in the wall will affect the response of other panels, the analysis cannot be viewed as having determined the true response of the entire wall. To state that panels will respond out-of-phase and therefore that each panel may be analysed in isolation with free boundary constraints does not recognise the effects of accelerations induced in all panels by deformations of any one. Similar considerations lead to reservations about inelastic dynamic analyses that do not properly account for accelerations induced by plastic extensions through yielding regions, especially in shear wall structures. Nevertheless, subject to the boundary constraints assumed, the analysis does indicate that considerable enhancement of sustainable accelerations over those based on a purely statical approach is achieved, if not to the same degree as found by Dr Priestley.

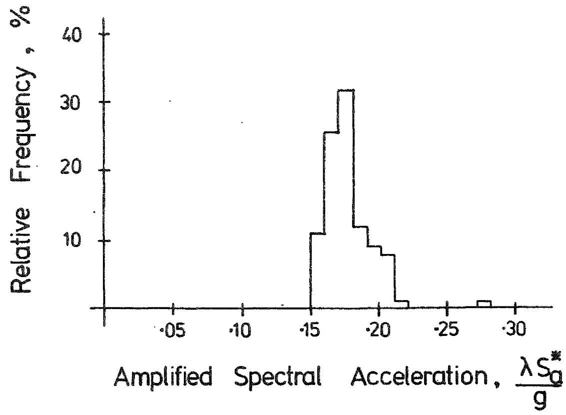

Figure A4: Response of wall to random wave forms superimposed on sinusoidal first-mode response

Dr Priestley is to be congratulated on his initiating this subject in the Bulletin, and this writer encourages him to proceed with refinement of the analysis method through further research and testing. Ingredients thought essential in this ongoing programme include:

1. The ability of such walls to remain intact and not degrade through impact or shear dislocation.

2. The effect of damping, including equivalent viscous damping through energy loss on impact.

3. The degree to which the non-linear response of significant masses affect the overall structural response.

4. Mutual interference between adjacent panels, and the study of higher modes of response.

5. The true effect of vertical ground motion, ignored in this discussion.

AUTHOR'S REPLY

I would like to thank Mr Robinson for his very extensive discussion presenting an alternative viewpoint on the use of energy considerations to assess the seismic performance of unreinforced masonry walls. Such a detailed discussion deserves a considered response, particularly as it appears to me that Mr Robinson's claim of 'erroneous and misleading results' may be levelled at some of the methodology and many of the statements in Mr Robinson's discussion.

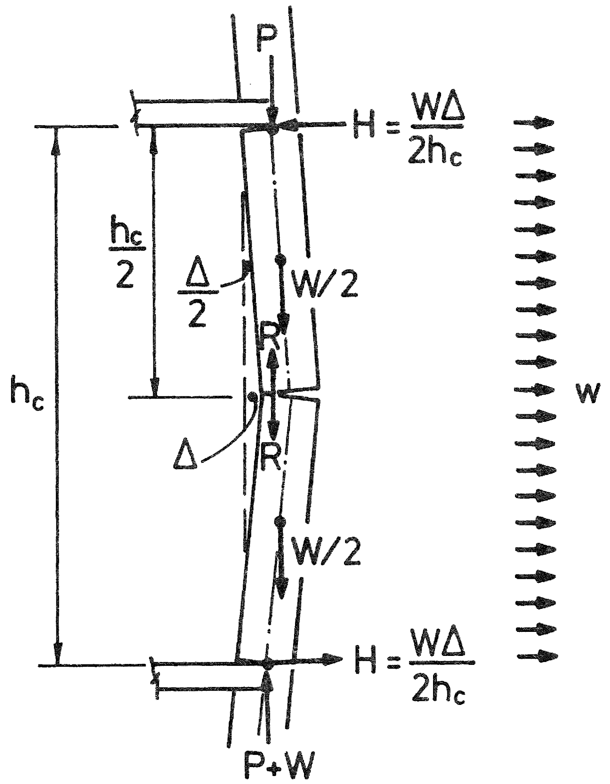

Figure A5: Forces on face-loaded wall including lateral reactions

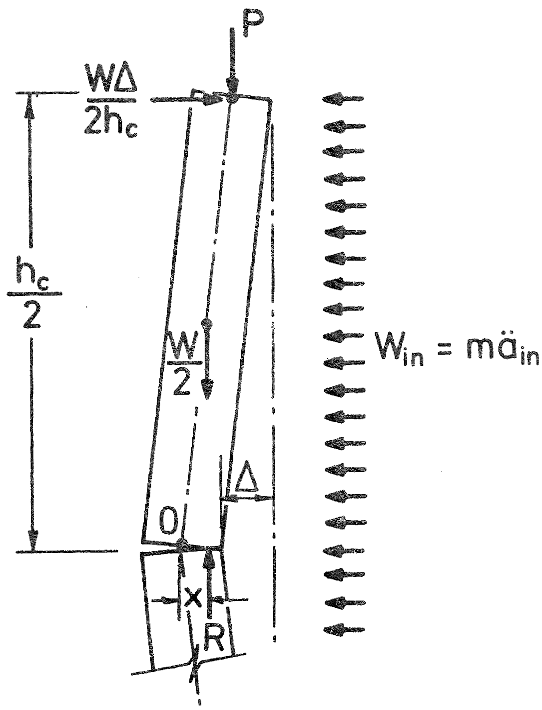

Figure A6: Moment equilibrium for face-loaded wall (modified from Fig 10)

First, though, I am indebted to Mr Robinson for pointing out the omission of the horizontal reactions needed for stability of the displaced wall. Fig A5 shows the full set of forces involved. Taking moments about the base reaction (P + W) yields

$$H.h_c = W\frac{\Delta}{2}$$

ie $$H = \frac{W\Delta}{2h_c}, \qquad (A12)$$

and hence Eq A1 and A3 as noted by Mr Robinson. Consequent changes to the paper need to be made to the moment equilibrium of Fig 10 and Equations 22 to 24. From Fig A6, taking moments about 0,

$$R.x = \frac{w_{in} h_c^2}{8} + \frac{W}{2}\frac{\Delta}{2} + P\Delta + \frac{W\Delta}{2h_c}\cdot\frac{h_c}{2}$$

$$= \frac{w_{in} h_c^2}{8} + R\Delta \qquad (A13)$$

where R = P + W/2, as before. Consequently the response acceleration required to develop a displacement Δ is given by

$$\ddot{a}_{in} = \frac{w_{in}}{m} = \frac{8}{mh_c^2} R(x - \Delta) \qquad (A14)$$

The example given in the Appendix to the paper needs corrections based on the formulation above. The moment-curvature relationships of Fig 16 are correct, but the acceleration-displacement plots of Fig 17 require modification based on the use of Eq A14 rather than Eq 24.

Figure A7 shows the corrected form of Fig 17, with the original acceleration-displacement curves of Fig 17 shown as dashed lines for comparison. As expected, the corrections are more significant for level 5 than for level 3 or 1.

Revised areas under the three acceleration-displacement curves are:

Level 5: A_5 = 5.27 (mm x g units)
Level 3: A_3 = 12.8 (mm x g units)
Level 1: A_1 = 14.1 (mm x g units)

The equivalent elastic response accelerations to induce failure, and the percentage change from the results given in the original paper are thus

Level 5: $\ddot{a}_e$ = 0.52 g (-14.7%)
Level 3: $\ddot{a}_e$ = 0.81 g (-7.4%)
Level 1: $\ddot{a}_e$ = 0.85 g (-2.9%)

Turning now to other comments made by Mr Robinson.

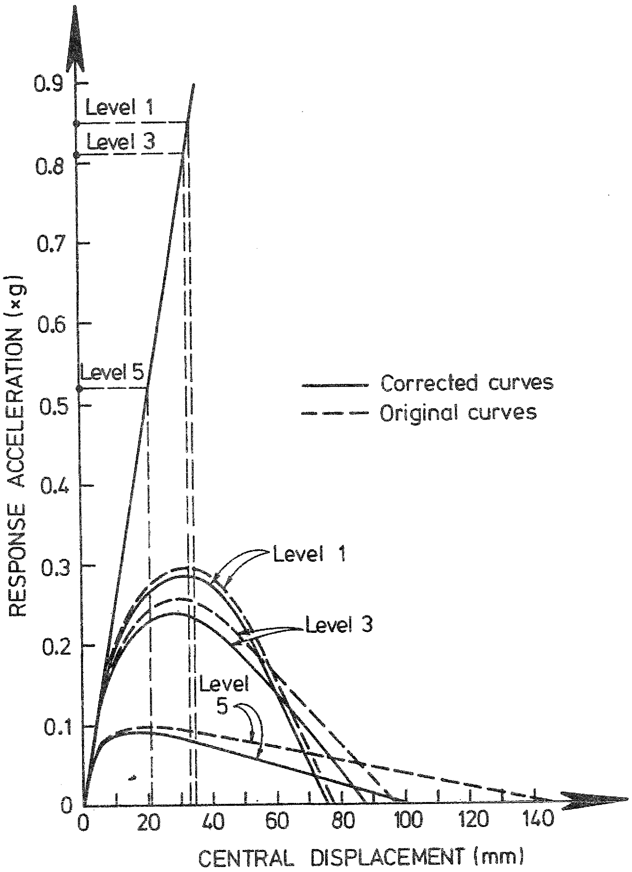

Figure A7: **Corrected equivalent elastic response accelerations at different levels for example building**

1. Vertical Accelerations

Mr Robinson asserts that it is not necessary to consider vertical acceleration effects 'since these will be propagated through the structure at frequencies close to the predominant frequency of the ground motion'. Except in unusual soil conditions there is no predominant ground frequency, but a frequency range which may contain significant energy between 0.03 Hz and (say) 2 Hz or higher. Dynamic analyses have shown that very substantial dynamic amplification of vertical ground accelerations can occur in vertical members when either (1) vertical members have low modulus of elasticity, or (2) floor masses are excited, inducing dynamic reactions in the vertical members. Both are possible in the type of buildings under consideration. The effects are most significant in the uppermost stories.

The level of 0.2 g for vertical accelerations adopted in the study was intended to provide some protection against vertical response. The somewhat arbitrary level chosen was felt to be substantially lower than peak vertical response to take into account the low probability of coincidence of peak vertical and horizontal response.

2. Modulus of Elasticity

Mr Robinson asserts that a value of 1 GPa for old masonry in compression is abnormally low, and that a more realistic value 'would be 10 GPa or greater', but does not substantiate this with test results. I would refer him to recent American studies[A1] which indicated in-situ modulii of elasticity consistently below 1 GPa, and often as low as 0.4 GPa, and results by Gurley and Nicholls[A2] where 1-2 GPa was measured for panels cut from the Auckland Ferry Building.

3. Energy Methods

The most important disagreement between Mr Robinson and myself relates to the use of energy considerations to estimate peak response. The approach adopted in the paper is based on the rather widely utilised 'equal energy' observation whereas Mr Robinson prefers to equate kinetic and potential energy to obtain an estimate of response. Neither approach is, of course, correct - they are both approximations.

The 'equal energy' observation is often elevated, quite unrealistically, to the state of a 'principle'. Many inelastic time-history studies have demonstrated that it gives a reasonable estimate of peak displacement response for simple systems with rather short periods. Fig A8 illustrates the range of applicability. For very short period structures ($T \rightarrow 0$), the equal acceleration principle is valid (assuming elasto-plastic response). for $T <$ (say) 0.25 s, the 'equal energy' approach is non conservative, while for $T >$ (say) 0.5 s the approach is conservative, and

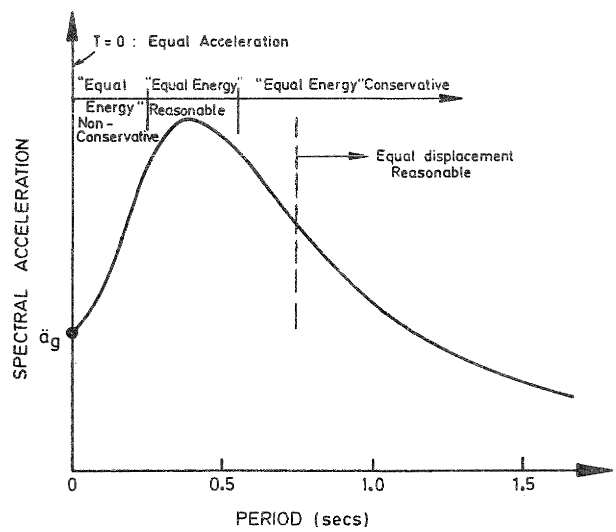

Figure A8: **Validity of 'equal-energy' approach related to period**

the 'equal displacement' approximation is more reasonable. The actual range of applicability depends on the characteristics of the response spectrum. However, the approach has the merit that it has been demonstrated to give reasonable results for periods around 0.3-0.5 s for many earthquake records.

The approach outlined by Mr Robinson was originally developed by Housner(A3) as a means for assessing the response of rocking rigid bodies. Equating potential energy at maximum displacement with kinetic energy at zero displacement provided Housner with a sound method for developing the equations of motion, but has not, to my knowledge been proved to provide accurate predictions of peak displacements. The problem is that the method assumes a stationary stochastic process, with zero energy input over the critical time period. Earthquakes manifestly do not conform to these assumptions.

The poor agreement between the methods proposed by Mr Robinson and myself is very largely a result of the values for f given in Fig A3. These reflect relative amounts of kinetic energy assumed to apply for different possible displacement profiles. It appears that Mr Robinson has made two serious errors in this application. First, the shape of the displacement profile varies in a non-linear fashion as the displacement amplitude increases. This will make Mr Robinson's assumption that the peak velocity profile (ie at zero displacement) will approximate the peak displacement profile suspect, at best.

Second, and much more important, Mr Robinson has assumed that the peak displacements of floor and wall response will be in phase, as shown by the profiles of Fig A3(d) and A3(e). In fact, as noted by Mr Robinson, the effective natural frequency of the wall response decreases to a very small value as the displacement approaches instability, and will be very much smaller than the forcing frequency corresponding to the floor excitation. As the frequency of the wall goes through resonance with the excitation frequency, the phase angle between response and excitation will change from in phase to 180° out of phase, as shown in Fig A9. As a consequence, the more probable displacement profile of Fig A10 should be considered by Mr Robinson. It is clear that the kinetic energy associated with this profile will be small.

However, as mentioned above, it is not felt that this approach has relevence because of the non-linear nature of the displacement, and the variable nature of phase angle as displacement increases. Any approach based on an assumed equivalent elastic sinusoidal response must be suspect, and hence the supporting evidence given by Mr Robinson in Fig A4 can not be given much credence.

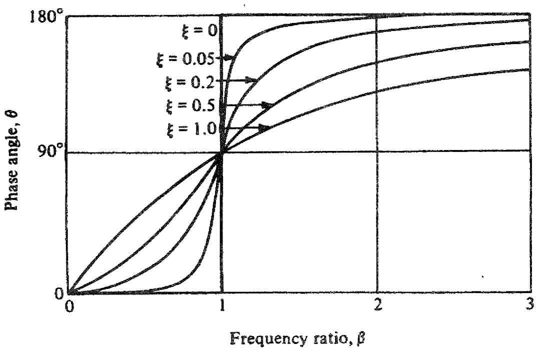

Figure A9: Variation of phase angle with ratio of forcing frequency to response

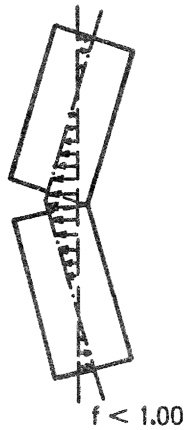

Figure A10: Probable out-of-phase displacement profile at failure (compare with Fig A3)

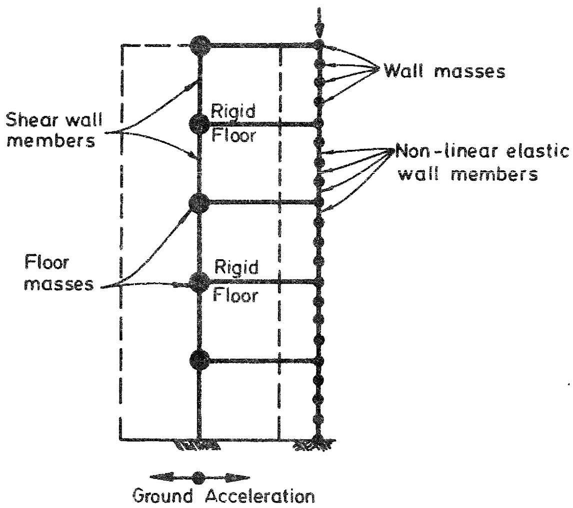

Figure A11: Simulation of example building for dynamic inelastic time history analysis with P-Δ effect

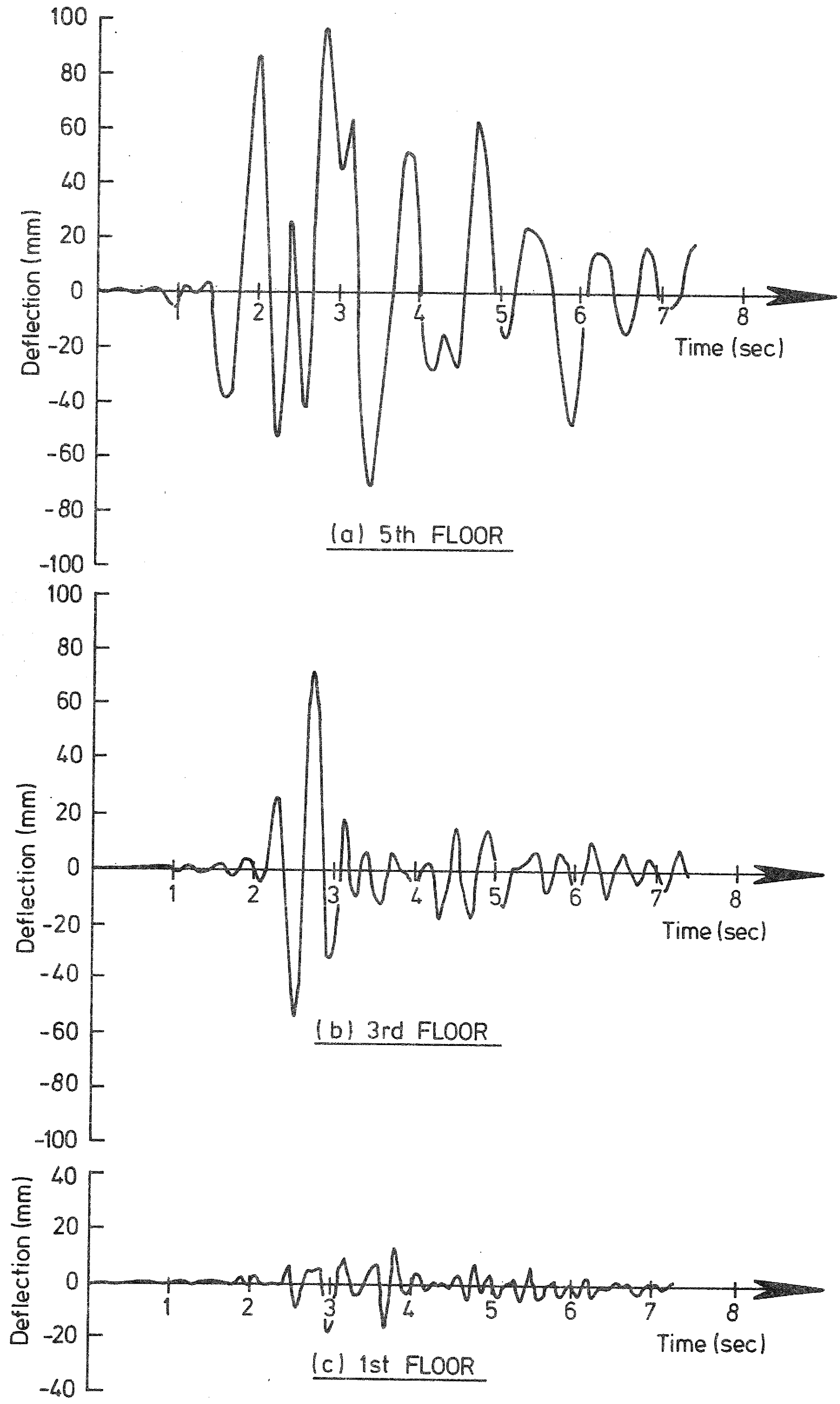

Figure A12: Time history analysis of wall relative displacements under El Centro 1940 NS Accelerogram using example building with rigid floors (P-Δ effect included)

Recently, a study has been carried out at the University of Canterbury into the dynamic response of unreinforced masonry structures by Zoutenbier(A4). The study has involved the development of appropriate member deformation characteristics to model the non-linear elastic moment curvature characteristics of an unreinforced wall, as given, for example, in Fig 16. These were used in conjunction with an existing program, Ruaumoko(A5) which has the capability of modelling the P- effects resulting from wall deformation.

Because of the rather arbitrary response spectrum of Fig 14, and the even more arbitrary factor of two incorporated in the results to account for the effects of floor flexibility, it was not possible to provide a direct comparison between the results given in the paper and those predicted by time history analysis. However, the structure given in the appendix to the paper was subjected to the N-S trace of El Centro 1940, with the assumption of rigid floor and roof diaphragms. Fig A11 shows the simulation used. The shear wall was modelled by vertical elements of appropriate stiffness to produce the structural period of 0.635 s. The face loaded wall consisted of a large number of small vertical elements with different moment-curvature characteristics representing the variation in axial load with height.

Fig A12 shows response to the first seven seconds (all that where significant in this case) of the accelerogram, in the form of displacements of the midheight of each wall relative to average displacement of the floors above and below. The peak displacements may be compared with the values causing instability, given in Fig A7. It will be seen that level 5 just reaches a potential failure situation, while level 3 and 1 have greatly reduced response.

A modified analysis, using the method developed in the paper, and the 5 percent damping elastic response spectrum for the 1940 El Centro trace, indicated that failure would be expected, at the fifth floor at about 85 percent of El Centro, but that floors 3 and 1 would not fail (unless as a consequence of the failure of the upper floor). The agreement with the dynamic analysis is rather close, conservative and much better than would be expected on the basis of Mr Robinson's argument.

It is significant, however, to note that using a higher value of E = 4 GPa caused the predictions by the hand method given in the paper to be significantly non-conservative, as would be expected from the consequent period shift associated with the increased stiffness.

A further point of interest resulting from Zountenbier's analyses was that the influence of floor diaphragm flexibility was substantial, and inconsistent in effect. It was concluded that where significant floor flexibility occurred, the only realistic means for assessing response would be by time-history analysis.

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