lateral load coefficients.

The following comments are offered:

1. Mr. Glogau asserts that the panels were unrealistically 'sound' due to small panel size, despite attempts to use no more than normal care. It should be noted that the walls were laid, not by a qualified block-layer, but by a general tradesman. It should further be noted that since the walls were nominal 150 mm rather than the more common 200 mm widths the extremely high steel percentages adopted made grouting much more difficult than would normally be the case. General construction practice deliberately did not conform to the high standards required by the MWD. Despite these factors, as pointed out in the paper, all walls substantially exceeded the theoretical flexural capacity, based on measured material properties. If the recommended under-capacity factor of 0.85 was adopted, the minimum excess of actual strength over design strength would have been 53%. These results indicate a certain amount of leeway to take account of any artificial strength due to unforseen 'soundness' of the test panels.

2. The author is not aware of any experimental evidence to support Mr. Glogau's statement that "... the present undercapacity factors (do not) .. realistically represent the effects of the variability of the completed construction". It appears that Mr. Glogau is advocating a reduction in the flexural undercapacity factor. It is, however, accepted that the quality of masonry constrution is very variable, and the author would not advocate the use of $\phi = 0.85$ for inadequately supervised, or unsupervised construction. A current masonry test programme at the University of Canterbury is investigating, amongst other variables, the influence of workmanship on strength and ductility, and it is intended to offer the results, in the form of a paper, to the Bulletin in the near future.

3. As mentioned in the paper, and agreed by Mr. Glogau, the critical region for shear is the base-joint area. Shear within the wall can be adequately controlled by properly designed horizontal steel. It is important to realise that during inelastic cycling, a wide crack will occur along the base course, regardless of the extent of joint preparation, and base slip will be resisted solely by dowel action until sufficient lateral load has been developed to cause the compression end of the base crack to close. This mechanism will not be improved by limiting the shear force. Consider two geometrically identical walls, height H and base length ℓ, the second designed to carry twice the shear load of the first. The amount of distributed vertical steel required is as follows:

Wall 1: Shear $V_1 = V$, Moment $M = V.H$

$$\text{Vert. steel } As_1 \doteq \frac{2VH}{\ell . f_y} \qquad (1a)$$

Wall 2: Shear $V_2 = 2V$, $As_2 \doteq \dfrac{4VH}{\ell . f_y}$ (1b)

Dowel Resistance will be proportional to the number of vertical bars times the section modulus of the vertical bars

$$\text{i.e. } F_{D1} \propto N_1 \, d_{b1}^{\;3} \qquad (2a)$$

$$F_{D2} \propto N_2 \, d_{b2}^{\;3} \qquad (2b)$$

where d_{b1}, d_{b2} are the diameter of the vertical steel.

There are two extremes for providing the extra vertical steel required for wall 2.

(a) Use the same size bars, i.e. $d_{b1} = d_{b2}$.

Then $N_2 = 2N_1$

Clearly from equations (1) and (2),

$$\frac{F_{D1}}{V_1} = \frac{F_{D2}}{V_2}$$

The ratio of dowel strength to flexural strength is constant, and it seems reasonable to expect the two walls to perform equally well (or equally poorly, as the case may be) in limiting base slip.

(b) Use the same number of bars, i.e. $N_1 = N_2$

Then $d_{b2} = \sqrt{2}\, d_{b1}$

From equations (1) and (2)

$$\frac{F_{D2}}{V_2} = \sqrt{2}\,\frac{F_{D1}}{V_1}$$

In this case relatively better resistance to base slip is developed by the more heavily loaded wall.

4. It is suggested that as engineers we expect, and frequently get, poorer quality workmanship for masonry structures than would be tolerated for (say) reinforced concrete construction. The doubts expressed by Mr. Glogau must remain unless the average standards of construction can be improved. It is unfortunate, however, that the high quality work required by some designers and obtained regularly from some competent contractors should be subject to limitations based on sub-standard construction by other contractors.

"SLENDERNESS EFFECTS IN EARTHQUAKE RESISTING FRAMES" - A. L. Andrews

Bulletin of N.Z. National Society for Earthquake Engineering, Vol. 10, No. 3, September, 1977.

H. M. IRVINE*

Mr. Andrews has presented a novel means by which energy principles may be used to assess the degree to which deflection control limits P-δ effects in frames. Mr. Andrews concludes that, although zone A is satisfactory, some tightening of control is needed for zones B and C. However, it is not clear that zone A is satisfactory because these conclusions are based on his equation (15) which is itself based on a "top-heavy" triangular distribution

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of earthquake loading. If, for example, this distribution is such that $n = 2 n_g$ (there is some evidence for this[6]), and K_2 remains equal to 2 Cd, equation (17) would become

$C > 0.115$,

which points to tighter control of interstorey deflections (including control in zone A0 than Mr. Andrews advocates.

Reference

6. H. M. Irvine, "The Centre of Earthquake Loading on Tall Buildings", Bulletin of the N.Z. National Society for Earthquake Engineering, Vol. 9, No. 4, December, 1976.

D. G. ELMS*

Mr. Andrews has made a most worthwhile attempt at defining the conditions under which slenderness or P-delta effects can be ignored for earthquake-resistant frames, in the context of the design requirements of the N.Z. Loadings Code, NZS 4203:1976.

However, though the analysis is sound, its two main assumptions deserve further consideration. The first assumption is that a frame structure as a whole behaves in the elastic/perfectly plastic manner shown by Figure 3.

A more reasonable assumption for the lateral load-deflection characteristics of a frame is given in Figure A. Taking $\lambda\delta_e = V_{code}$, then $V_{max} \simeq 1.25/0.9 V_{code} = 1.4\lambda\delta_e$.

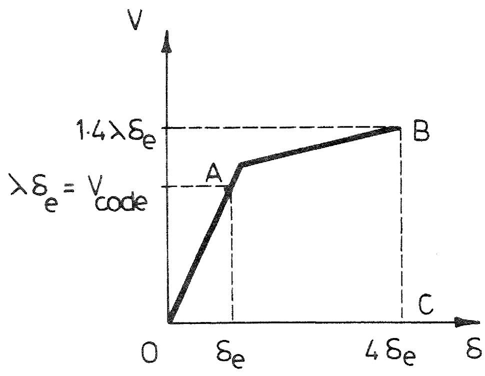

FIGURE A

With a ductility factor of 4, and assuming the elastic part of the diagram climbs a little above V_{code}, then the work U_o done by the lateral forces is the area OABC, or

$$U_o \simeq 4.5\lambda\delta_e^{\,2} \quad \ldots \text{(A)}$$

Putting $\mu = 4$ in Eq. (4) of the original paper gives the equivalent expression

$$U_o = 3.5\lambda\delta_e^{\,2} \quad \ldots \text{(B)}$$

Adoption of the assumption of Figure A thus reduces the limitation on C given in the original paper by 3.5/4.5 so that Eq. (17) may be replaced by the constraint

$$C > 0.067/I \quad \ldots \text{(C)}$$

However, this is not a very large difference. Much more important is Mr. Andrews' adoption of an energy ratio figure of 10% as the basis of the criterion and recommendations given in his paper. This figure is quite arbitrary. It could easily have been taken as 15%, in which case the recommendation that the code drift limitations should be tightened up for zones B and C would not have been required. If the paper had shown that the code drift limitations were totally inadequate to limit P-delta effects, it would have been another matter; but as it is, the analysis shows that within the limits of the reasonableness of its assumptions, P-delta effects will not be serious for buildings designed to NZS 4203: 1976, and that therefore the code need not be altered.

There is in any case a more fundamental reason why the code drift limitations should not be changed. Mr. Andrews' analysis derives a criterion for, essentially, a stiffness limitation on a structure below which P-delta effects become significant and should be taken into account in the analysis of the structure. It would be wrong, to my mind, to use this criterion, designed to show the limitations of an analytic technique, as a reason for limiting the physical characteristics of the structure.

A better approach might be to invert the criterion of Eq. (17). If we assume the P-delta decrement to be α% of the work represented by Eq. (10), rather than 10%, then modification of inequality (C) leads to

$$\alpha = \frac{0.67}{CI} \quad \ldots \text{(D)}$$

as an estimate of the percentage error (in energy terms) introduced by neglecting the P-delta effect for a code-designed building. The largest possible value of α would be for the case where $C = 0.05$ and $I = 1$, giving $\alpha = 13\%$. Of course, because of the assumptions involved, this figure can only be regarded as indicating a rough order of magnitude. Nevertheless the very important result of Mr. Andrews' analysis, that P-delta effects are not important for code-designed buildings, is borne out.

I would like to acknowledge some helpful suggestions from Professor Paulay which have been incorporated in this discussion.

B. W. BUCHANAN**

Mr. Andrews' paper is most helpful in drawing attention to the P-delta problem, and to the considerable differences between structures in an elastic condition (as considered in ACI 318-71) and structures in an inelastic mode, as occurs in seismic conditions and which must therefore be provided for in seismic design codes.

For an arbitrary limit of 10% reduction in work capacity due to P-Δ effects, Mr. Andrews shows that this limit is exceeded for structures in Zone B with periods exceeding 0.9 seconds, and those in Zone C with periods exceeding 0.65 seconds. It might be useful to evaluate the extent of energy loss for all zones and periods.

Defining L as the percentage loss of

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work (into potential energy) at any particular value of C_d, we can restate the inequalities (7) et seq as equations, to obtain

$$\frac{K_1}{C_d} = \frac{2(2\mu - 1) \cdot L}{150\ \mu^2} \quad \ldots (15A)$$

$$\text{giving } C = \frac{150\ \mu^2}{400\ (2\mu - 1)\ I.L} \quad \ldots (16A)$$

With $\mu = 4$ and $I = 1$, we obtain

$$L = \frac{0.857}{C} \quad \ldots (17A)$$

This result is plotted in Fig. B against T, where the T-C relationship is that of Figure 3 of NZS 4203.

From (17A), clearly L will only be less than 10% of the work intended to be expended when C is greater than .0857. While the author suggests new zone-dependent stiffness controls to keep the P-Δ losses to less than 10%, the losses are C-dependent rather than zone-dependent. It would be more appropriate for the code stiffness control clause to maintain the present absolute limit of 0.01.h for its damage control purpose, and to introduce a further limit of 0.1 C.h for control of P-Δ losses. This would have the effect of restricting P-Δ work loss to a consistent 8.57%, independent of zone or the particular code's pseudo-spectrum.

Mr. Andrews' paper is on a static basis - as is the Code's primary approach to seismic design. It is well recognised that this is a poor representation of the real dynamic behaviour. It is clear from Fig. 4 of the paper that a deflected structure must rise to recover its original position, which immediately suggests that there will be a tendency for flexible or ductile structures to deflect further rather than to recover the energy lost. Does the author anticipate dynamic effects being significantly different from those covered by his static analysis?

A. L. ANDREWS

I agree with Dr. Irvine that a flexibility restriction of the kind the paper recommends for controlling P-delta effects is linked to the load distribution specification and would have to be reconsidered if any reconsideration were given to that specification. However, if the control proposed for use with the Code as it stands were to be applied to proportion a frame, using for the purpose the distribution currently specified, the result would be satisfactory for stiffness when checked out against Dr. Irvine's complete proposal.

One can think of the recommendation in rough terms in this way: there is a lateral stiffness which, when provided for any frame supporting a gravity load and a lateral load which is a simple function of the gravity load, will ensure that P-delta effects are limited to a predeterminable fraction of lateral load effects. The stiffness is a property of the frame, but it is measured by the size of the response to lateral load, so the control specification must account for both the magnitude and for the distribution of lateral load.

I am not willing to engage in serious debate with Professor Elms over the second limb of the bilinear force-displacement idealisation, whether it ought to show a positive strain hardening slope or whether a flat top corresponding with perfect plasticity is an acceptable representation for our purposes. Were I to need justification for my choice I might have found it in the general endorsement given to practical ideas which rely on the relevance of the flat top for their validity. Among these are the collapse assessment procedures of the plastic method for steel frames devised by Baker's Cambridge team[7,8] and the procedure earthquake engineers use to reduce elastic response forces to design forces by division by the numerical equivalent of a displacement ductility factor[9]. It might have been unwise to infer from the success of these and of other widely used "flat top" dependent ideas that it is generally reasonable to ignore the strain hardening slope which sometimes appears, although not always as noticeably as in Professor Elms' illustration; but the discusser's conclusion that there is nothing very significant to be had from this refinement makes it unnecessary to pursue the matter.

Professor Elms' willingness to tolerate potential error arising from P-delta neglect is conspicuously inconsistent with what can be inferred to be his attitude to precision of lateral load determination. As one of the authors of NZS 4203, he shares responsibility for specifying load qualifiers which differ from unity by as little as 0.1. Examples occur in Table 7 and Figure 3 of the Code, accounting, respectively, for numbers of people accommodated in assembly buildings and for the effect of flexible subsoils on response in Zone A. He is, therefore, anticipating that force assessment errors will be substantially smaller than 10%. The P-delta load decrement at displacement ductility factor 4 for the most flexible of permitted structures can be shown to be

$$\beta\ (\%) = \frac{1.5}{CI}$$

(flat top bilinear spring behaviour assumed). With CI at its minimum value of 0.05 and P-delta ignored this represents a force error of 30%!

Mr. Buchanan proposes a modification to the paper's flexibility limiting proposal; but one which is more complex and which, on that account, will probably find less favour with designers. We have now complexity enough to contend with. What I see to be needed is a control, the simpler the better, which will halt dangerous pursuit of the specious advantage of slenderness by designers of buildings for seismic zones B and C.

I had considered dynamic effects; but did not report this in the paper because the work was (and still is) incomplete. We can forecast the probable result from a dynamic study. Figure C (ii) shows a completed behaviour limit diagram for a single degree of freedom structural system with the kind of idealised material behaviour previously considered. Within the limits represented by lines sloping negatively at $\frac{\lambda P}{P_{cr}}$, the system has Masing type behaviour of the kind customarily assumed for all earthquake resisting structures. These limit lines cross the zero displacement axis at lateral resistance $\pm \lambda\delta_e$. The slope of elastic behaviour lines is $\lambda(1 - \frac{P}{P_{cr}})$. Limit lines intersect the zero lateral resistance axis at two points $\pm \frac{P_{cr}}{P} \delta_e$. These points are static collapse or catastrophe points.

Figure C (iii) shows how the work required to cause a displacement from the original undisplaced position is less than that needed to restore the structure.

Deformation of the structure is therefore biased in the direction of first significant yield. Every framed structure subject to excitation causing responses which exceed elastic limits is certain to collapse from this cause if the excitation is maintained for a long enough time. Thus, it may well be that P-delta considerations are more significant than the simple static consideration shows.

The static consideration was proposed as part of a submission I made to those responsible for drafting the new standard 3101P. It was published because standards are now under review, and these things ought not be ignored.

It is, perhaps, because of the way we develop engineering, applying intense effort at any one time to a single idea (like the capacity design idea) without a reasonable overview, that we have so often in the past overlooked much more serious shortcomings than our efforts were designed to correct. As has been abundantly demonstrated, it is at least possible that all our refinement of the capacity procedure will be in vain for so long as we continue to ignore P-delta. It is indisputable that P-delta action is the ultimate cause of every collapse and that it is entirely neglected in most engineering assessments.

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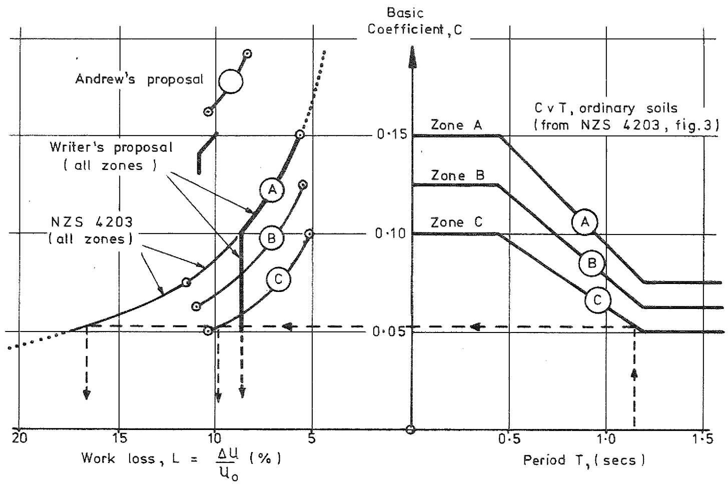

FIGURE B: LOSS OF WORK CAPACITY FOR I = 1, μ = 4

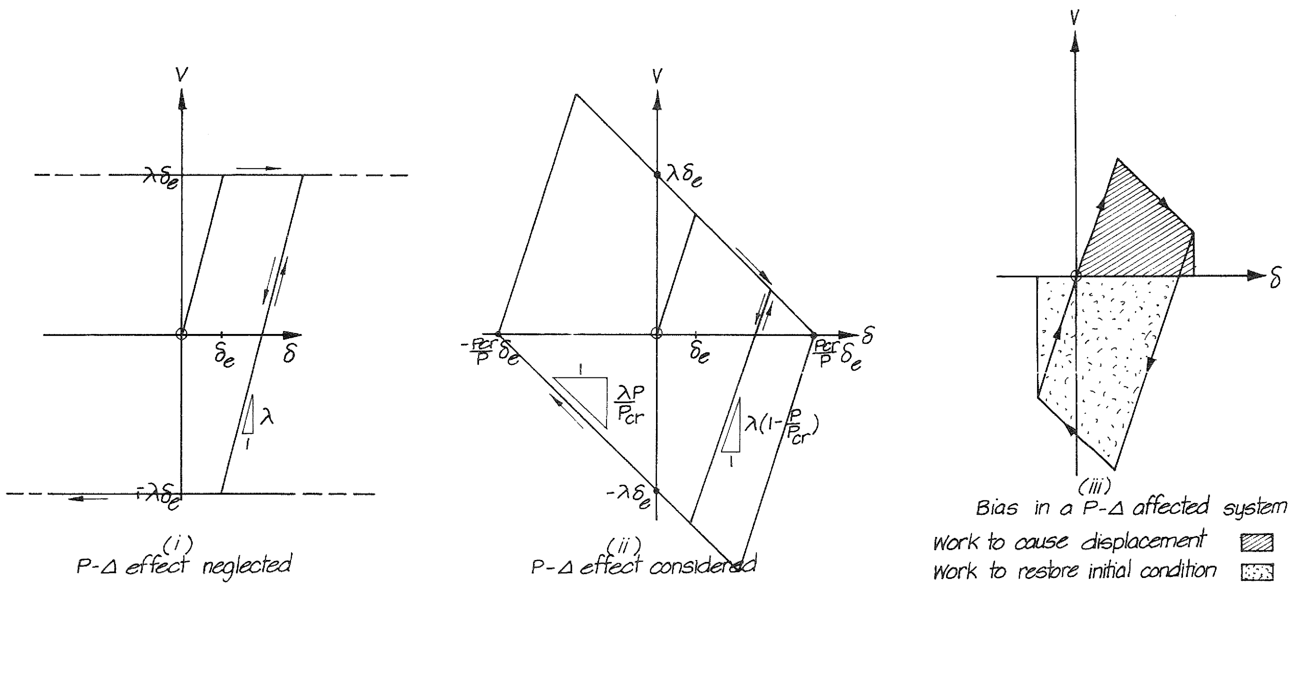

FIGURE C: SINGLE DEGREE OF FREEDOM SYSTEM