SECTION G

LOW RISE REINFORCED CONCRETE BUILDINGS OF LIMITED DUCTILITY

Some lessons from recent Earthquake Damage

O.A. Glogau*

ABSTRACT:

The New Zealand Loading Code sets earthquake design levels which are intended, amongst other factors, to reflect the manner in which seismic energy will be dissipated by a structure.

In this paper the load levels prescribed for reinforced concrete structures unlikely to behave in a flexural ductile manner, are compared with the strength and behaviour of structures of similar type affected by 3 recent Japanese earthquakes.

The adequacy of some of the provisions in the current loading and concrete code and modifications proposed for the design of these so called structures of limited ductility in a companion paper by L.M. Robinson[1] are examined.

INTRODUCTION:

In the New Zealand Code of practice for "General Structural Design and Design Loadings for Buildings, NZS 4203:1976" the Total Horizontal Seismic Design Force on a structure is determined from a multi-term expression

$$V = C_d W_t$$

where C_d = CISMR

The meaning of the symbols is given in the code. The term S is called the structural type factor and varies from 0.8 to 6. It is intended principally to reflect the manner in which a structure dissipates seismic energy.

Structures assumed to dissipate seismic energy in modes other than by flexural ductile yielding, including by shear or sliding must be designed for seismic forces derived from "S" factors not less than 1.6 to 2.4, the applicable value depending on a number of other criteria.

In terms of 5% damped peak elastic response to a "code earthquake" a design force level derived from S= 1.6 to 2.4 represents approximately 30 to 45% of this response.

Obviously a significant amount of seismic energy must be "dissipated" - to use the term favoured by designers. Following earthquake, owners of buildings refer to the same process usually as "damage".

For practical reasons the objectives of codes cannot preclude the occurrance of some structural damage in events that occur very rarely, but risk to life should be very low. Thus total collapse of more than an insignificant fraction of buildings must be avoided.

To determine whether this objective, (numerically illdefined though it may be at present), can be met, an attempt is made in the following to calibrate the New Zealand code provision for S = 1.6 to 2.4 reinforced concrete structures, by reviewing the behaviour of some Japanese buildings affected by 3 recent strong earthquakes.

RECENT EARTHQUAKE DAMAGE EVALUATIONS IN JAPAN:

The affected buildings were designed using working stress design methods and min. code loadings of 18-20% g, but the actual strengths were often much greater.

Detailing standard was comparable to NZS 1900, Ch. 9.3 1964 and concrete quality generally was similar to that in N.Z. buildings.

The poor performance of buildings of this type, some of which failed in a catastrophic manner, came in Japan to engineers, as it would in this country, as a considerable shock. Several investigations were carried out to determine the expected risk represented by the many remaining buildings of this type.

It is important to note that the conclusions were not derived from the results of a single, perhaps unusual earthquake affecting a few buildings. Quite to the contrary, the sample comprised several hundred buildings subjected to earthquakes of widely varying characteristics, namely:

(1) 1968 Tokachi-oki, M 7.9, some 170 km from the affected sites at Hachinohe City where the measured ground acceleration was (only) 0.23 g,

*Chief Structural Engineer, NZ Ministry of Works and Development

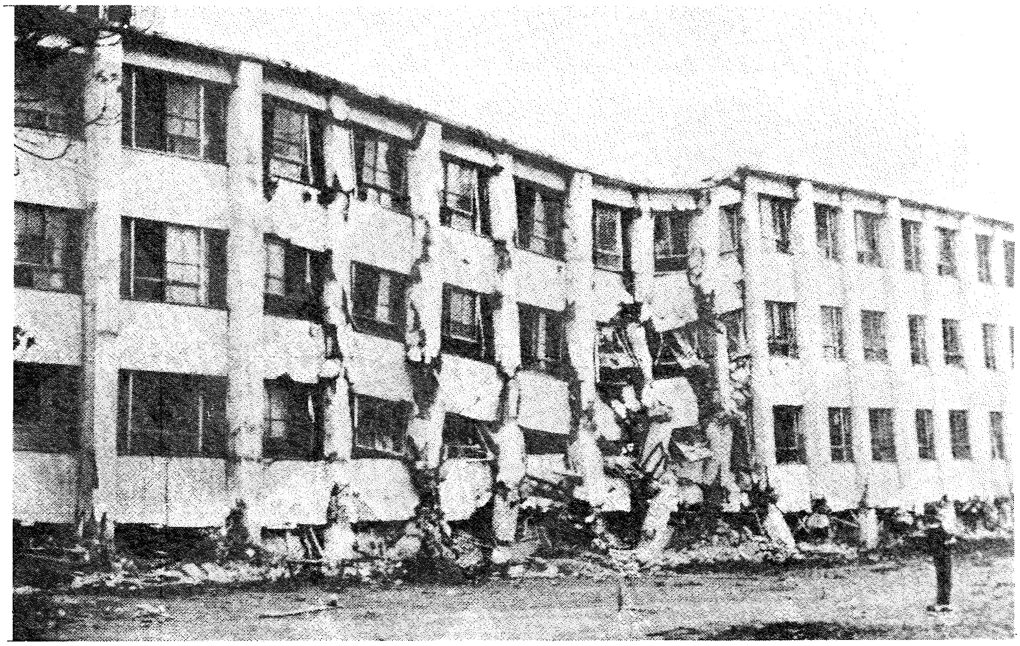
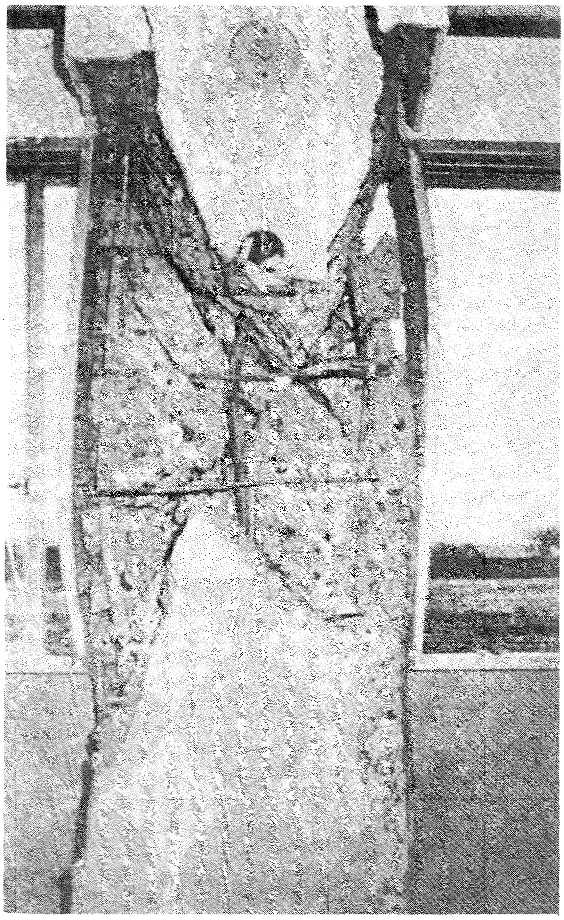


Fig. 1 - Hakodate College Tokachi Oki E.Q. 1968 (2)



Heavily damaged short column ($^{h}0/D \doteqdot 2$)

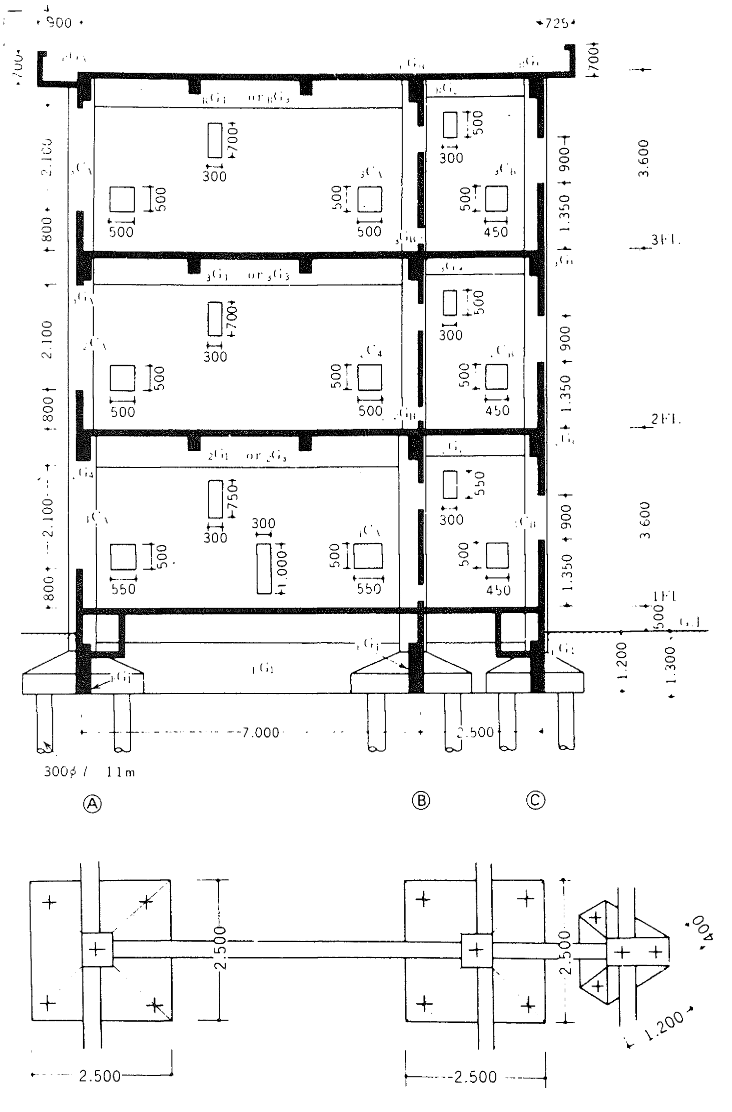
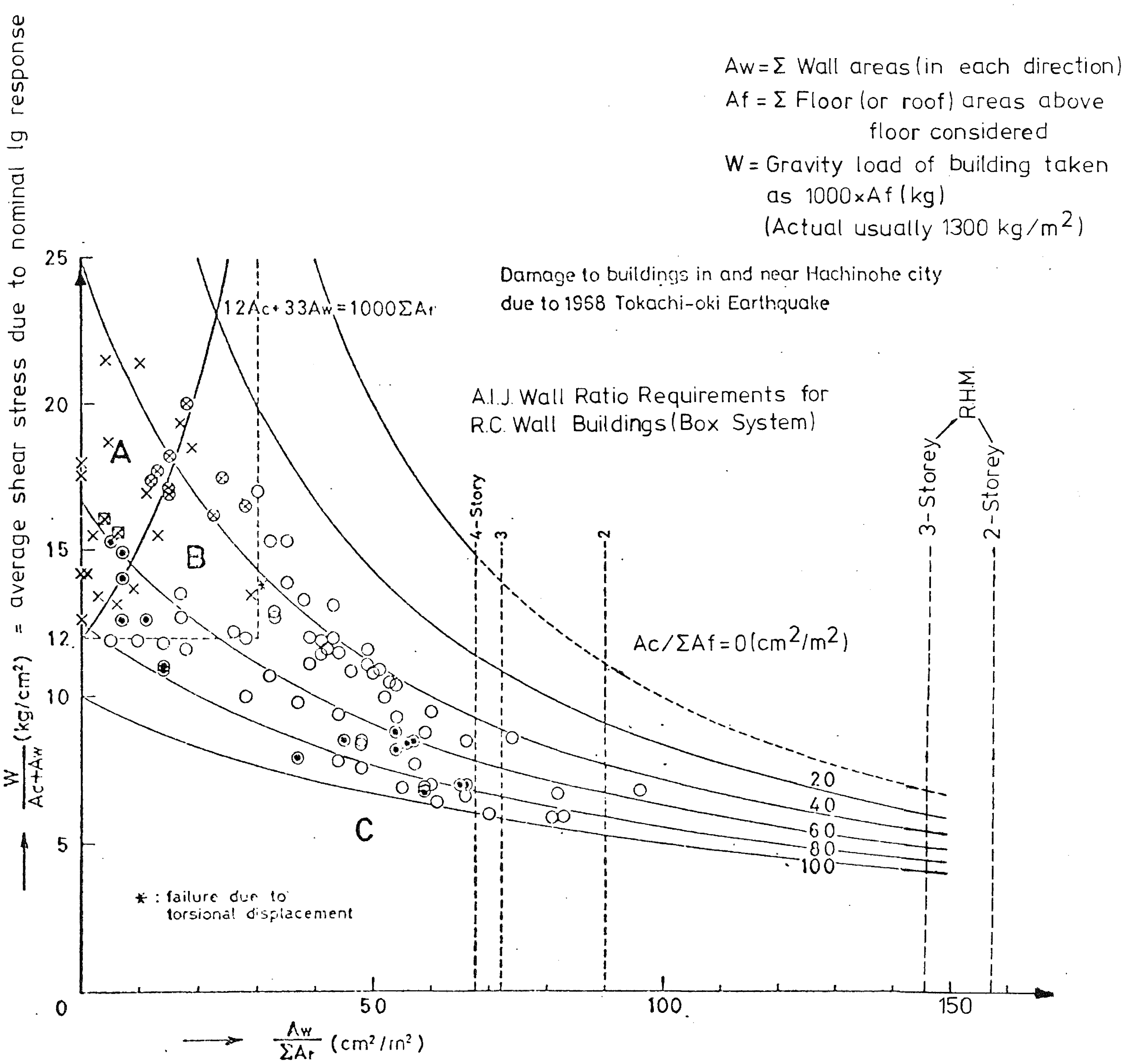


Fig. 2 - Hachinohe Technical College (2)



○ : Small or no damage in columns and shear walls

(◉ = C type school building)

△ : Collapse

× : Shear failure in most 1st-story columns

⊠ : Bending failure in about a half of 1st-story columns, shear failure in some 1st-story columns and slight shear cracks in shear walls

⊗ : Shear cracks in most 1st-story shear walls and slight damage in columns

Wall-Area Index, Column-Area Index and Average Shear Stress in Walls and Columns.

Fig. 3

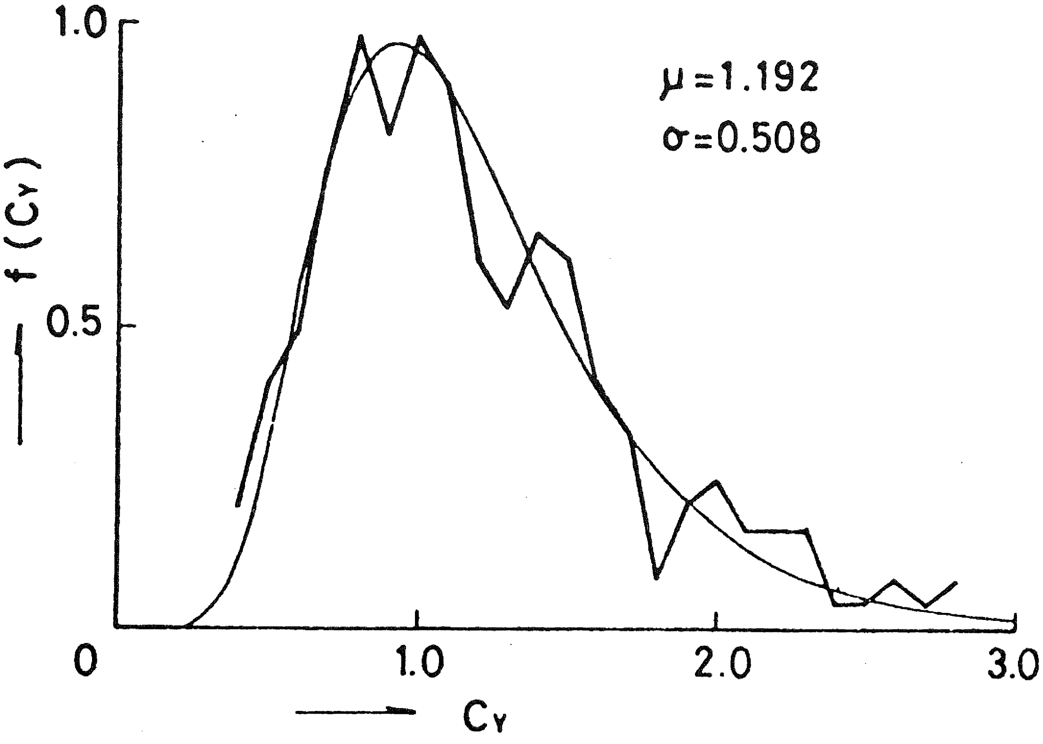


Fig. 4 Distribution of Earthquake Resistant Capacity

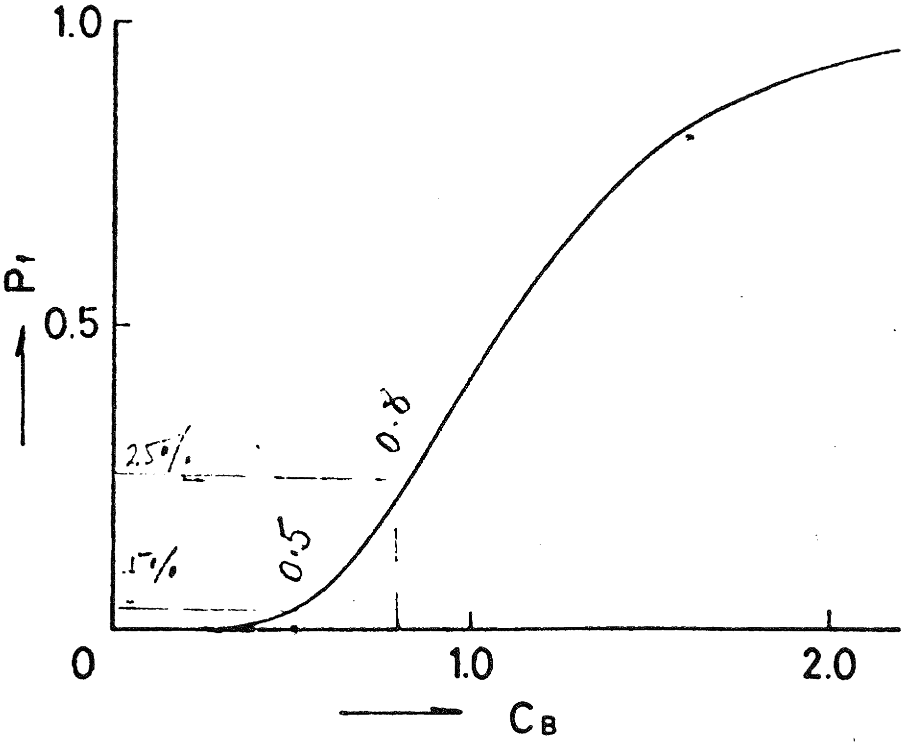


Fig. 5 Probability of Failure of Buildings

with a sharp peak in the power spectrum at 0.4 sec. There were no foundation failures.

(2) 1975 Ooita earthquake, M 6.4, very shallow and only some 7 km from the affected sites at Lake Kuju. Ground accelerations were estimated to be 0.4 g. Soil effects were not considered to have been significant.

(3) 1978 Miyagiken-oki earthquakes, February 20 and June 12, 1978, M 6.8 and 7.4 respectively. Recorded peak ground accelerations were 0.2 g on hard ground on which however buildings were only slightly affected. Serious damage was suffered by buildings on soft ground.

The two typical buildings shown in figures (1) and (2), seriously damaged due to the Tokachi-oki earthquake,[2] are similar to many NZ R.C. buildings built between 1935-1976 and some of this type and detailing are still being erected. It is important too, to note, that few reinforced hollow masonry structures, old or recent, would have the inherent capacity of the R.C. structures shown.

For buildings of the type considered, a few very simple parameters were found to be of critical importance. The most elementary of these is the'wall area ratio', simply the ratio of wall areas (in each direction) to floor area of building above. Since most buildings of a type have comparable unit weights, the wall area ratio is virtually an expression giving the maximum allowable average shear stress in walls due to a standardised earthquake load.

Figure 3[3] is an attempt to relate damage to a building's capacity, and while perhaps carried a little too far, it is a good indication of the trends. 245 buildings of 1-5 storeys were analysed. The stresses in the figure were derived from a 1 g response, and a nominal building weight of 1 t/m^2. The actual reinforced concrete buildings weighed approximately 1.3 t/m^2 and therefore the shear stresses in the figure are identical with those due to a 0.7 to 0.8 g response, which also happens to be the value obtained from response analysis of the recorded ground motions. It was concluded from the damage pattern that undamaged buildings have:

- A wall area index $A_W/A_f > 30\ cm^2/m^2$ (0.3%)
- An average shear stress in short columns

  $\frac{W}{A_C} < 1.2\ N/mm^2$ (12 kg/cm^2)
- An average nominal ultimate shear stress in walls of

  less than 3.3 N/mm^2 (33 kg/cm^2)

Note: Only walls longer than 60 cm were included. The notation is that in figure 3.

To evaluate an existing building of this type its capacity is taken as:

$$1.2\ A_c + 3.3\ A_W\ (N)$$

where A_c (col. areas) and A_W (wall areas) are in mm^2, and compared with the seismic load due to a 0.8 g response (or for convenience with a 1 g response and an assumed nominal gravity load of 1000 kg/m^2.

As mentioned previously, the real capacity of the buildings at Hachinohe was far greater than the code minimum of 18-20% g. In fact, as shown in figure 4, a large proportion of buildings had capacities exceeding $C_y = 1$, (Representing elastic response).

For a given distribution such as that in figure 4 it is possible to evaluate the probability of failure[3]. This has been expressed in figure 5. C_B represents the response shear coefficient for various earthquake intensities. For buildings of reasonably similar characteristics, such as those at Hachinohe, C_B may be taken as constant for each level of earthquake intensity. It is seen that probability theory tends to support the high failure and serious damage rate at Hachinohe even though a large proportion of buildings had high capacities (but limited ductility). For $C_B = 0.5$, P_f is only 5% but increases to 25% when $C_B \doteq 0.8$ (ie, 80% g).

Figure 6 shows some of the buildings subjected to the 1975 Ooita earthquake[4].

The most severe damage was sustained by Block C (K), which collapsed due to brittle failure of the first storey supporting the upper three storeys. Some permanent deformations were also suffered by Block A consisting of translations NW and anticlockwise rotation above the first floor.

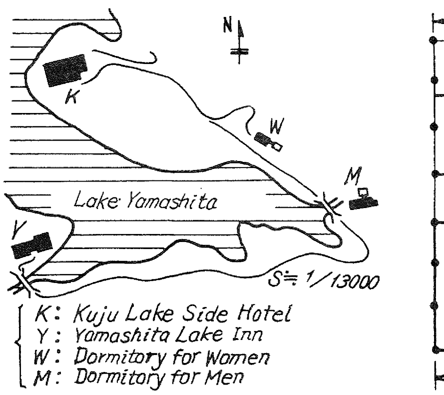
The column and wall ratios of several of the buildings are shown in figure 7 (from reference 9). The seismic capacity of Block C and A of the KL Hotel (Y direction) was evaluated[4] as around 0.7 g but subject to brittle failure for displacements over 2 cm. Block B was estimated to have a capacity of 3 g, the Yamashita Lake Inn (Y direction) 2½ g and the dormitory for women (X direction) 5 g, presumably on the assumptions that rocking was prevented.

Damage to Block B and all the other buildings was slight or nil.

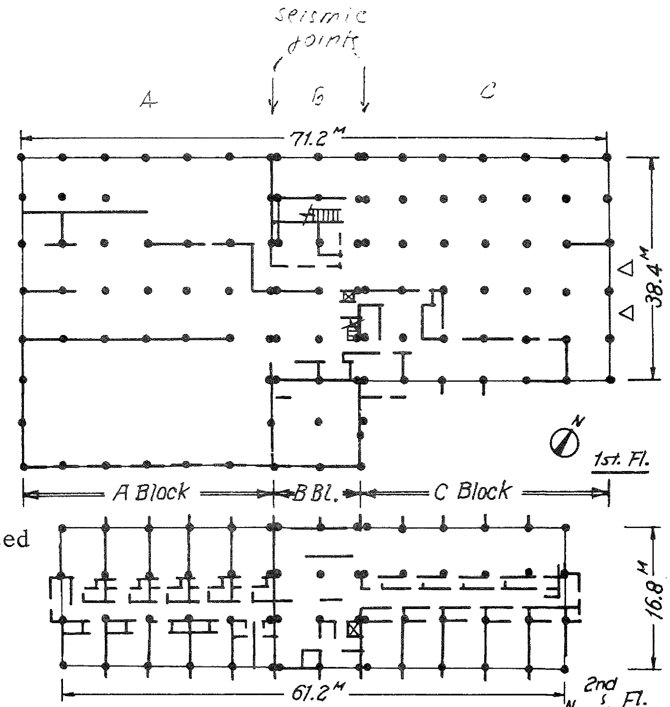
While individual buildings no doubt had particular features that contributed to their poor performance, the generally consistent pattern of damage in relation to wall ratios, has convinced Japanese investigators that for this type of structure, wall ratios were an important guide to performance in strong earthquakes.

A JAPANESE EVALUATION PROCEDURE:

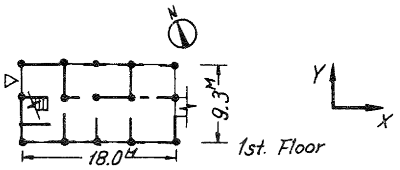
A committee set up by the Japanese Government under Chairmanship of Prof.



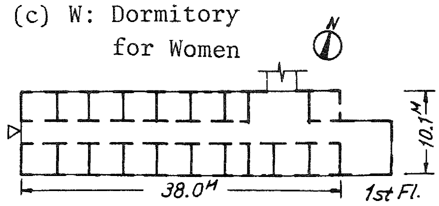
Disposition of Reinforced Concrete Buildings



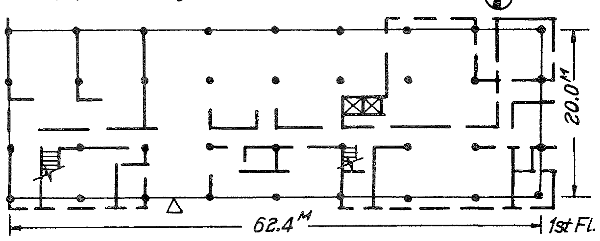
(a) K: Kuju Lake Side Hotel



(c) W: Dormitory for Women



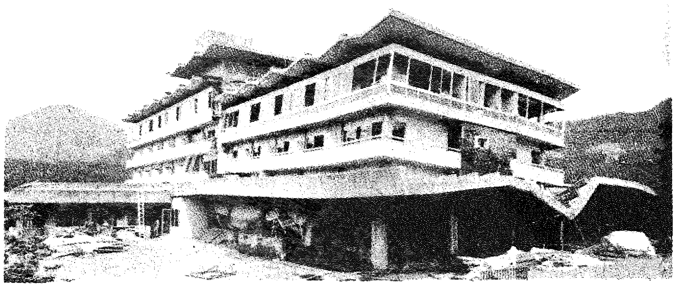
(d) M: Dormitory for Men



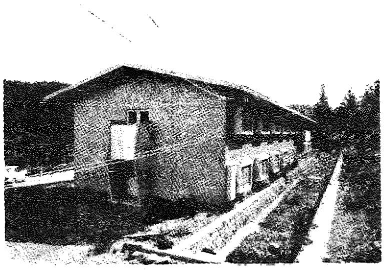
(b) Y: Yamashita Lake Inn

Key Plans of RC Buildings around Lake Yamashita (S=1/800)

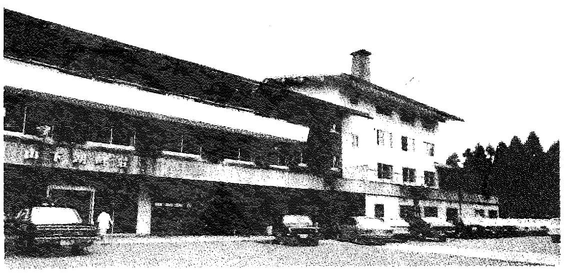
(c) W; Dormitory for Women



(a) K; Kuju Lake Side Hotel



(d) M; Dormitory for Men



(b) Y; Yamashita Lake Inn

Fig. 6 Photoghaphs of RC Buildings taken after the Ooita-Earthquake

Higashi (Tokyo Met. Univ.) and including Prof. Aoyama (Univ. of Tokyo) recommended the following[6] as a simple and practical method of evaluating the earthquake resistance properties of existing low rise buildings of the type described above, provided they are located on stable soils.

The evaluation proceeds as follows:

(1) Buildings of irregular plan are divided into pseudo rectangular parts.

(2) Evaluations are made separately in each principal direction.

(3) Except where there are significant changes between floor plans, elevations and arrangements of shear walls, evaluation of the lowest storey suffices.

(4) I_o, the standard horizontal force seismic coefficient is taken as 1 except where the predominant period of the ground T_G has been measured, in which case $I_o = (1.25 - 0.5\ T_G)$.

(5) Excepting where measured or available from contract documents the concrete strength is taken as not more than 17 MPa (2500 psi).*

(6) Buildings are considered to be safe if they have wall rates in each direction of not less than:

$2n$ cm/m^2 where walls are 15 cm thick, and for walls of thickness t cm, not less than:

$$2n\ \frac{15}{t}\ \mathrm{cm/m^2}$$

n = number of storeys above level concerned but not to be taken as less than 2.

The wall rate is computed as the sum of clear length of walls (cm) surrounded by rigid frame members, taken in each principal direction separately. The widths of columns are deducted. Expressed as a % of floor area:

$$\frac{2n \times 15}{10^4} \times 10^2 = \frac{30n}{100} = 0.3n\%$$

(with a min, of 0.6%)

For comparison with the damage plot in figure 3, let us evaluate this recommendation for the case of a 3 storey building:

Weight of building above 1st floor, taken as 1300 kg/m^2.

$W = 2 \times 1300 = 2600$ kg/m^2

Seismic load at 0.77 g response : $2600 \times 0.77 \times 9.81 = 20{,}000$ N/m^2.

*(Yield strength of reinforcing bars for older NZ buildings should not be taken as more than 33,000 psi(230MPa) except for overstrength considerations.)

If it has the min, safe wall rate : $2 \times 2 \times 15 = 60$ cm^2/m^2.

Therefore the nominal shear stress

$$= \frac{20000}{6000} = 3.3\ \mathrm{N/mm^2}.$$

This is the same value as that recommended in ref. 3 and found to be safe at Tokachi-oki.

If the building examined does not meet the requirements of this procedure step 6, further, slightly more detailed investigations may be carried out:

A shear resistance force coefficient is determined as follows:

$$S_B = \frac{Q}{W_g}\ \alpha$$

α is a coefficient judged from the general condition of the building R_c and the distribution of stiffness R_e in the building both in plan and between floors. However α varies only 30% i.e.,

$$0.7 < \alpha = \frac{R_c}{R_e} < 1.0$$

$$0.7 < R_c < 1$$

$$0.8 < R_e < 1$$

For uniform distribution of stiffness $R_e = 1$

$$Q = \Sigma\ Q_c + \Sigma\ Q\ \text{walls}$$

$$= 0.5\ (1 + \frac{A_c}{A_c + A_{w1} + A_{w2}})\ A_c + 3\ A_{w1} + 2\ A_{w2}\ \text{(Newtons)}$$

where A_c = sum column areas on a floor (mm^2)

A_w = sum of wall areas in one principal direction on a floor (mm^2)

A_{w1} = as above but where wall has columns at each end

A_{w2} = as above but without columns at both ends

The expression for Q, except for the very simple adjustment for compatability for the contribution of cols (coefficient of term A_c), is similar in nature to that in the preceeding procedures.

Thus for walls a nominal shear stress of from 2 to 3 N/mm^2 (20 to 30 kg/cm^2) is allowed.

Columns are assessed as contributing from 0.5 to 1.0 N/mm^2 (.5 to 10 kg/cm^2). The building is safe when:

$$S_B > I_o \text{ (refer step 4)}$$

(7) Should the building be judged not to be safe by the procedure of step 6 its capacity can be evaluated by a number of methods of varying complexity. The following is the simplest and was stated to have given good results:

The shear force coefficient of a storey S'''_B is obtained by combining the contribution of columns of flexural and shear failing behaviour with that of walls in the following manner.

$$S_{B1} = \frac{(A_{C1} \cdot V_{C1})}{W_g} \quad \text{bending failing type col.}$$

$$S_{B2} = \frac{(A_{C2} \cdot V_{C2})}{W_g} \quad \text{shear failing type col.}$$

$$S_{B3} = \frac{3\, A_{w1} + 2\, A_{w2}}{W_g}$$

V_{c1} and V_{c2} in N/mm^2 $(\tau_{1,2})$ are obtained from figure 8 ref. [6] depending on the clear height to depth ratio $R_o = h_o/D$. The lesser value of V_{c1} and V_{c2} should be used and is seen to vary from about 4 N/mm^2 for squat columns to about 0.8 N/mm^2, for slender ones.

The regions where shear or bending apply are obvious from the figure. R_o = 3.3 is the limiting case.

$$S'''_B = \frac{S_{B1}}{S_{B1} + S_{B2} + S_{B3}} + S_{B2} + S_{B3}$$

The expected performance of the structures is evaluated as follows:

I	$S'''_B \geqslant 1.0$	Good
II	$1 > S'''_B \geqslant 0.6$	Sufficient
III	$0.6 > S'''_B \geqslant 0.4$	Poor
IV	$0.4 > S'''_B$	Dangerous

Unless more detailed investigations discover redeeming features, strengthening or reduced life must be considered for the last two cases.

The method outlined above was applied to 14 buildings at Tokachi-oki[6]. The results are shown in figure 9 and indicate that the method gave good results.

EXPECTED PERFORMANCE AT DESIGN LOAD LEVELS OF NZS 4203[7]:

Design procedures and detailing have a decisive influence on performance. As stated previously, because of the similarity in design procedures, detailing and construction standard, the experience from Japanese damage is of great relevance in reviewing older NZ code provisions.

As we move away through from these, to more stringent detailing e.g., closer spacings, greater quantities of shear reinforcing etc. the more difficult it becomes to forecast performance from damage experience to date. This is particularly so since some damage is acceptable in severe earthquakes, though life risk should be low.

NZS 4203 includes a provision for a "strength" design only. Except for the very restrictive elastic response procedure, strength design is associated with design load levels derived from S = 1.6 or S = 2.4.

The commentary to NZS 4203 gives reasons for permitting strength design. Some of these, it is necessary to recall, no longer apply, (eg, at the time ductile frames had to have width to depth ratios of 0.25 - 0.3 which virtually excluded wall frames). Other valid reasons remain, including the fact that some walls, particularly the squatter types cannot reliably be made ductile. The code presumed that these, for strength only designed structures would fail in shear, though hopefully not suddenly and - given a sufficiently high load level - not too early in an earthquake. Damage must also be avoided in the more frequent, moderate earthquakes.

Concern that significant damage might initiate total collapse was the reason (refer to NZS 4203, C3.4.8.3) for requiring some members to have substantial minimum sizes e.g., 800 mm for the wall piers of S = 2.4 structures and 1500 mm for S = 1.6 strength designed cantilevers with aspect ratios greater than 2. These criteria are obviously crude and evidently not likely to be equally effective in all applications, but at least they are simple.

The option to use strength as an alternative to capacity design in cases where the latter could reasonably easily be applied, caused some concern and this is evident from the comment in NZS 4203 on Table 5, item 6.

Let us presume that a designer used the "strength" method in accordance with the code for a system for which either S = 1.6 or S = 2.4 was appropriate, together with conventional RC design and detailing eg, less rigorous than DZ 3101[8]. For the resulting structure evaluation for performance using the Japanese earthquake damage experience with the storey shearforce coefficient S'''_B of section 3 as the indicator would be appropriate. Some adjustments must be made for the more conservative shear stresses the hypothetical designer must have used to comply with NZ codes. Design criteria in all countries of thw world are more conservative than collapse or serious damage evaluation procedures. It is therefore not surprising that even recent draft codes such as DZ 3101 allow, on nominally reinforced sections, less shear than the Japanese review procedure. Many factors enter, but to establish the order of quantities consider a conventional (pre "Robinson") design comparable to the lightly reinforced Japanese buildings. Assume that the designer used $v_c = 0.166\sqrt{f'_c}$ and 0.25%

wall reinforcing (v_s = 0.66 MPa). For f'_c = 17 MPa v_c = 0.68. It follows that $v_i = v_c + v_s \cong 0.68 + 0.66 = 1.34$. The designer probably used d = 0.85 ℓw and a ϕ factor of say 0.9. The nett result is that he would determine his wall areas using:

$$v \text{ code} < v_i \phi b_w d < 1.34 \times 0.85 \times 0.9\ b_w\ \ell_w$$

$$< 1.0\ A_{wall}$$

Reference to figure 8 however indicates that the following levels of v could be sustained:

h_w/ℓ_w	0.5	1.0	2.5
v	3	2	1

Depending on the aspect ratio, the designer has therefore used a from 1 to 3 times "overconservative" design stress. Expressed in other terms, the designer has used a shear amplification factor.

On the other hand the designer will almost certainly have used C_d values much lower than those shown to be necessary for the survival of a structure failing in shear, namely S''_B greater 0.6 (refer Section 3.0 - "A Japanese Evaluation Procedure". If the designer has used NZS 4203 he will have chosen S = 1.6 or S = 2.4 and hence in zone A have obtained C_d = 0.15 S (if I, M and R = 1.0) i.e. C_d = 0.24 or 0.36. We may equate the conservative shear design stresses to proportionally more effective C_d values. Thus, if we term the ratios between the values for shear stresses from figure 8 (reproduced in the table above), to the (effective) current New Zealand code shear design stress of say 1.0 (MPa) "de facto" shear amplifiers α_v:

h_w/ℓ_w	0.5	1.0	2.5
"de facto" α_v	3/1=3	2/1 = 2	1/1 = 1
eff. $C_d = \alpha_v C_d$ for			
S = 1.6	0.72	0.48	0.24
S = 2.4	0.108	0.72	0.36

It will be noted by comparison of the effective C_d values in the table with the S''_B shear resistance coefficients that the designs with squat walls (h_w/ℓ_w < 0.5) would be classified as "good" for S $\cong$ 2.4 and sufficient for S = 1.6. Designs containing more "slender" piers would pass only for S = 2.4 and $h_w/\ell_w \leqslant 1$. At first sight this would appear to be a contradiction with the Japanese findings of poor behaviour of squat piers, particularly $h_o/_D < 2$ ($\simeq h_w/\ell_w < 1$). It is important to remember however that slender piers have good earthquake performance only when they fail in a ductile manner. If shear is the failure mode, then squater piers are able to take higher stresses (ref. figure 8)

To correct the problem Robinson[1] proposes the following:

- Amplification of the seismic component of shear:

$$v_i^e > \frac{3.2.V_e}{S\ \phi} \qquad \phi = 0.85$$

- A maximum design shear stress not exceeding that given in Chapter 7 of the concrete draft code (February 1980).

$$v_1 \text{ (max)} = \frac{V_1}{b_w d} < 0.83 \sqrt{f'_c} \quad \text{(or}$$

$0.8 \sqrt{f'_c}$ in later versions)

- In the end regions, a reduction of the contribution of the concrete to 0.5 of that allowed by the concrete draft code (February 1980 version) under "special provisions for walls", namely the lesser of:

$$v_c \text{ (max)} = 0.5\ (0.27 \sqrt{f'_c} + \frac{Nu}{4Ag})$$

$$v_c \text{ (max)} = 0.5\ (0.05\ f'_c + \frac{0.1 \sqrt{f'_c} + \frac{Nu}{5Ag}}{\frac{Mu}{V_u \ell_w} - 0.5})$$

Since $\frac{Mu}{V_u I_w} = h_w/\ell_w$, v_c (max) reduces with increasing slenderness ratio.

(refer figure 6, ref. (1)).

- v_c (min) = 0.5 (0.17 $\sqrt{f'_c}$) rising for values of $\frac{Nu}{Ag}$ > 0.1 f'_c

To evaluate to what extent Robinson's proposals remedy the shortcomings in the current NZ code consider the following:

The proposed amplifaction for the seismic component of shear is:

$$\alpha_v^e = \frac{3.2}{S}$$

The maximum design shear stress v_1 = 0.83 $\sqrt{f'_c}$, or at non amplified loading, i.e. loading derived from S,

$$v \text{ (code)} = \frac{v_1 \text{ (max)}}{\alpha} = \frac{0.83}{3.2} S\sqrt{f'_c} = 0.26\ S\sqrt{f'_c}$$

The maximum (earthquake component) of the code shear force that can be carried by a cross section is thus:

$$V_u^e = \frac{v_1}{\alpha_v^e} \times b_w \times d \times \phi$$

for ϕ = 0.85, d = 0.8ℓ_w f'_c = 17 (MPa), $b_w \ell_w = A_{wall}$

$$V_u^e \leqslant 0.726\ S\ A_{wall}$$

$\frac{V_u^e}{A_{wall}} \leqslant 0.726$ S and may be regarded as the effective shear stress determining minimum wall areas. It is seen that the

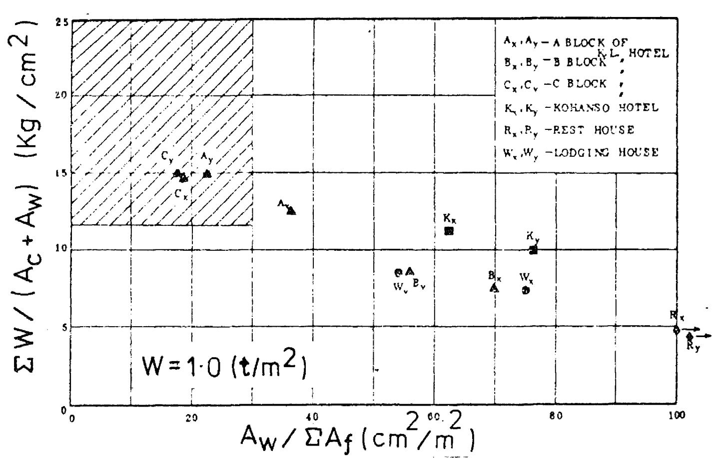


Fig. 7 - Damage and relationship between Ratio of shear wall and average shear stress.

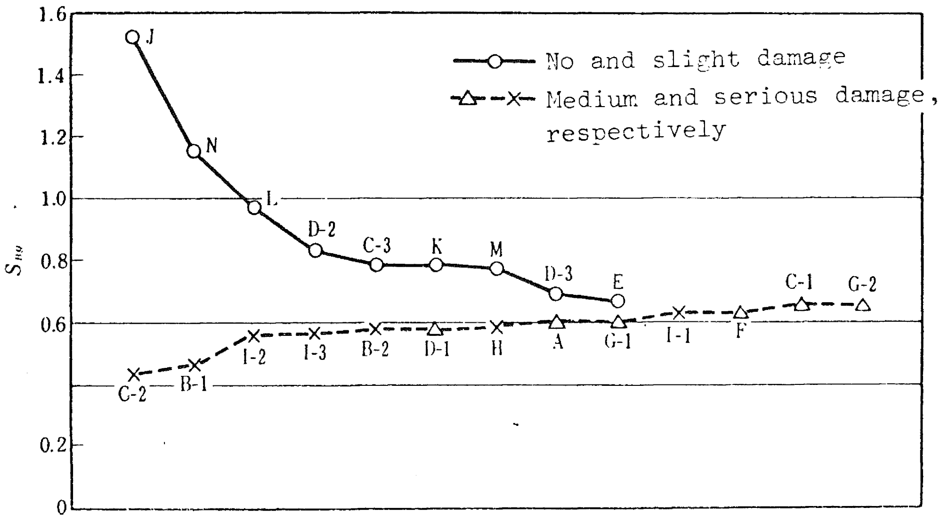


Fig. 9 Comparison between results by C method of 3rd step and degree of damages due to 1968 Tokachi-oki Earthquake. Note: Symbol marks show the investigated building. Abscissa has no special meaning.

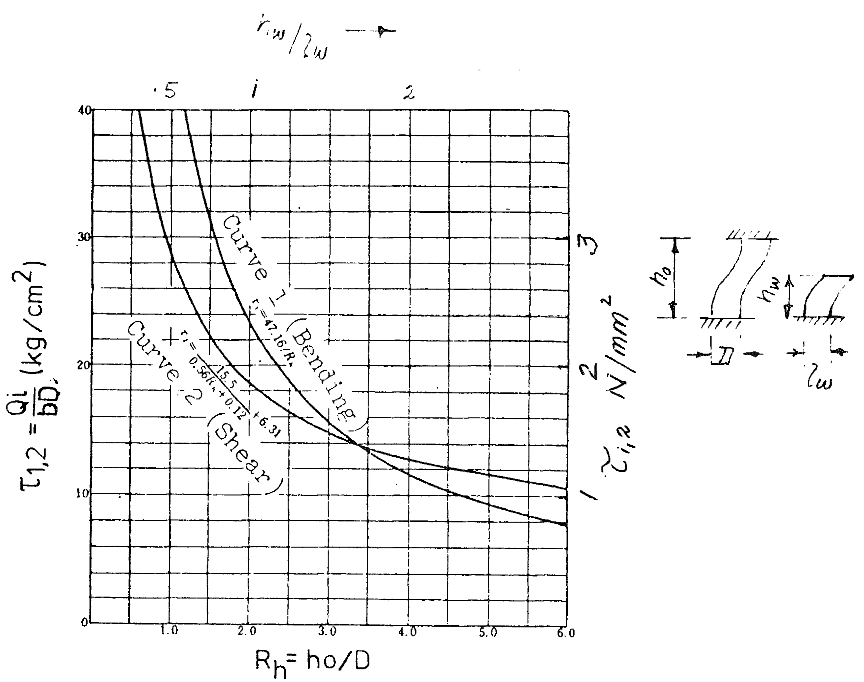


Fig. 8 - Calculation diagram of $\bar{\tau}_1$ and $\bar{\tau}_2$ **B method of 3rd step.**

values of the effective shear stress are 1.16 (MPa) for S = 1.6 and 1.74 (MPa) for S = 2.4. A comparison can now be made with the values given in figure 8 and a set of "effective" C_d values determined by multiplying the values from NZS 4203 by $\tau/0.726$ S.

Thus for the case of I = 1, M = 1, R = 1 and C = 0.15.

C_d = CSIMR = 0.15 S

The effective C_d value = $C_d \times \tau/.726$ S

= 0.15 S τ/.726 S = 0.207 τ

h_w/ℓ_w	0.5	1.0	2.5
τ	3	2	1
effective C_d = 0.207 τ	0.62	0.41	0.21

By reference to Section 3.0 "A Japanese Evaluation Procedure" of this paper a value of C_d = 0.6 when used with the stresses in figure 8 is the minimum required for systems failing in shear.

The lowering of the value of the maximum design shear stress through the amplificating of the seismic component of shear must therefore be supplemented by measures that either avoid shear failure or minimise the inelastic deformation demands should shear failure take place. The effect of lowering v_c to 0.5 of the values allowed by the concrete code for other members ensures that most of the amplified shear must be taken by shear reinforcing when υ_1 is at a maximum. With reference to the equations given above or figure 6, ref. (1):

$S = 1.6,\ f_c' = 20$ (MPa), $\frac{Nu}{Ag} = 0,\ 0.38 < v_c > 0.6$

The effect of the shear amplification factor is to increase the design shear force by

$V_e \left(\frac{3.2}{1.6 \times 0.85} - 1\right) = 1.35\ V_e$ If v_i is at max. its value is $0.83\sqrt{f_c'} = 3.71$ (MPa)

Thus $V_s = V_i \text{ (code)} - V_c = \frac{3.2}{S\ \phi} V_i^e - V_c$

$$= 2.35\ v_1\ b_w d - v_c\ b_w d$$

$$\frac{V_s}{b_w d} = 2.35\ v_1 - v_c \quad (S - 1.6)$$

If the shear had not been amplied the first term of the above expression would have been v_1. Consequently the additional quantity of shear reinforcing in the horizontal direction per unit wall area is proportional to 1.35 v_1.

Compared to that value, the contribution of v_e is small when high design shear stresses are used e.g.

$S = 1.6,\ f_c' = 20$ (MPa) $\frac{Nu}{Ag} = 0$ From the equations for v_c (max) or Figure 6, ref. (1)

$v_1 = 3.7,\ 0.38 > v_c > 0.6$ depending on h_w/ℓ_w

It is seen that $\frac{0.6}{1.35 \times 3.7} \times 100\% = 12\%$

For $\frac{Nu}{Ag} = 2$ (MPa) $\quad 0.38 > v_c > 0.85$

$$\frac{0.85}{1.35 \times 3.7} \times 100\% = 17\%$$

For higher values of $\frac{Nu}{Ag}$ the contribution of the concrete increases but axial compressive stresses tend to be low in shear walls.

Obviously the contribution of concrete increases as the value of v_1 reduces from the maximum allowed. A low v_1 value means that greater than minimum wall areas are available and this in turn (from Section 3) would be reflected in high shear resistance coefficients i.e. S_B or C_d (effective) values for the particular structures.

A final comment should perhaps be made with regard to the much higher concrete shear stresses allowed by the Japanese review procedures. Since many of the walls were nominally reinforced only i.e. say 0.25% of wall area, a significant proportion of the seismic force must have been taken by the concrete (30% for h_w/ℓ_w = 2.5 to 80% for h_w/ℓ_w = 0.5). The explanation could be that walls which survived without damage had capacities corresponding to S = 4 to 5. Since the failure mode tended to be shear type, capacities of the walls (except perhaps where very squat) must have been higher in flexure as well. It is not surprising therefore that the contribution of the concrete was significant.

The case is different for the NZ proposals. Flexural capacities in Zone A will be much lower, C_d = 0.24 IMR (for S = 1.6 and C = 0.15) or C_d = 0.36 IMR (for S = 2.4 and C = 0.15). This together with the uncertainty with regard to the effects of vertical earthquake would not make it prudent to allow a greater contribution for the concrete than is proposed in the Robinson proposals.

Review of the damage sustained by Japanese buildings does not allow extrapolation of the lesson too far. Thus while the Robinson proposals appear to be reasonable and of the right order, calibration of all the values used, against E.Q. damage is not at present possible. In particular it should be remembered that Japanese evaluations were made on the assumption that all the affected buildings, subjected to the same intensity of motion, would respond to the same level. This was for the buildings reviewed, of reasonably similar, short period and similar mode of failure, an acceptable assumption. But assumptions adequate for review of safety against collapse are not necessarily adequate for detailed formulation of new design rules.

CONCLUSIONS:

Investigations carried out in Japan into the damaging effects of a number of strong earthquake motions on several hundred modern "ordinary" low RC buildings allow the following conclusions to be drawn with respect to systems dissipating energy in shear:

(1) The earthquake design load level of NZS 4203 is by no means overconservative.

Quite to the contrary, conventional design procedure of reinforced concrete walls with aspect ration greater than about three-quaters using a design force level derived from S = 1.6 will lead to extensive damage in severe earthquakes.

The present limitation in the code on member size, intended to avoid total collapse, may be only of limited effectiveness but it should not be removed until other, hopefully better, design measures take its place.

(2) While the damage reviews allow certain conclusions to be drawn care must be taken not to extrapolate the lessons too far for purposes of design procedures because:

(a) Changes in detailing will alter performance.

(b) The reviews did not consider reinforcing content.

(3) With respect to the Robinson[1] proposals the reviews seem to indicate that the suggested shear modifier related to design force level is a move in the right direction.

REFERENCES:

1. Robinson, L.M.,"Shear Walls of Limited Ductility", Bulletin of the NZNSEE, Vol. 13, No.2, 1980

2. General Report on the Tokachi-oki Earthquake of 1968.

3. Shiga, T.,"Earthquake Damage and Amount of Walls in Reinforced Concrete Buildings", 6WCEE, New Delhi, 1977.

4. Yamada, M. and Kawamura, H., "Damaged and Undamaged Reinforced Concrete Buildings at the Ooita Earthquake", 6WCEE, Delhi 1977.

5. Tomii, M and Yoshimura, K. "Damage to a Reinforced Concrete Hotel due to the Ooita Earthquake" 6WCEE, Delhi, 1977.

6. Higashi et al, "An evaluation method of E.Q. resistant properties of existing R.C. school buildings", US Japan co-operative school programme, Hawaii 1975.

7. NZS 4203 : 1976, "Code of Practice for General Structural Design and Design Loadings for Buildings", Standards Association of New Zealand

8. DZ 3101, "Draft New Zealand Standard, COP for The Design of Concrete Structures", Standards Association of New Zealand, October 1978

9. Tsuchiya H. Haito T. et al, "Damage to R.C. Buildings due to the Ooita Earthquake of April 21, 1975." 6WCEE, Delhi 1977.

NOTATION:

The use of reproduced reference material from a number of sources made some variations on notations unavoidable. However with the conversions given no difficulties should be experienced.

For NZ Code requirements[7,8] the notations are those used by these documents.