

OPTIMUM DESIGN OF REINFORCED CONCRETE SHEAR WALLS

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Presented at the Third South Pacific Regional Conference on
Earthquake Engineering, Wellington, May 1983.

ABSTRACT

The recently published New Zealand Code of Practice for the Design of Concrete Structures (NZS 3101:1982) and the newly amended Code of Practice for General Structural Design and Design Loadings for Buildings (NZS 4203) permit a variety of possible design approaches for reinforced concrete shear wall structures. A series of wall designs for dimensionally similar four-storey and eight-storey buildings has been carried out and a comparison of construction cost estimates obtained together with an assessment of the relative design effort required for the different design options.

INTRODUCTION

In June 1980, the "Discussion Group on Seismic Design of Reinforced Concrete Walls and Diaphragms" of the New Zealand National Society for Earthquake Engineering reported the results of its deliberations in the Society's quarterly Bulletin. The work of this group was subsequently reflected in the New Zealand Standard for Design of Reinforced Concrete⁽¹⁾, published two years later.

Two broad classes of shear walls are defined, namely, "ductile shear walls" and "shear walls of limited ductility". The distinction is made on the basis of overall height to depth ratio, with walls having a value of this ratio of less than 1.0 being classified as walls of limited ductility⁽²⁾. Ductile shear walls have an aspect ratio 1.0 or more, and may have the form of cantilevers or of "coupled walls". In the latter case, two or more ductile cantilever walls are connected by "a number of appropriately reinforced ductile coupling beams that are capable of dissipating a significant proportion of the seismic energy"⁽¹⁾.

The procedure for design of walls of limited ductility is less complicated particularly because explicit capacity design for shear is not required. Instead, the dependable shear strength (ϕV_u) must be able to resist twice the value⁽²⁾ of shear induced by code-prescribed⁽²⁾ seismic loading together with shear resulting from the appropriately factored gravity loading⁽³⁾ (clause 14.4.2.1). This procedure is used for shear wall systems where the overall height to depth ratio ("aspect ratio") is small. However, walls of greater aspect ratio may, at the discretion of the designer, be designed as walls of limited ductility⁽²⁾ (clause 3.3.6.1) with increased loadings⁽²⁾ (Table 5, item 4).

The designer may also choose to design walls to respond elastically to earthquake loading through application of an equivalent

static seismic load which is two-and-a-half times as great as for limited ductility design. Elastically responding walls are subject neither to requirements for capacity design nor to the need for confining reinforcement.

There are likely to be attractions for designers to design walls to a higher level of seismic loading and corresponding lesser ductility demand. In some low-wall situations, shrinkage reinforcement alone may provide sufficient strength to ensure elastic response. Additional vertical reinforcement may enable the reduction, or elimination, of expensive confining ties. The increased simplicity of design approach utilising a reduced ductility demand may be sufficient attraction in itself.

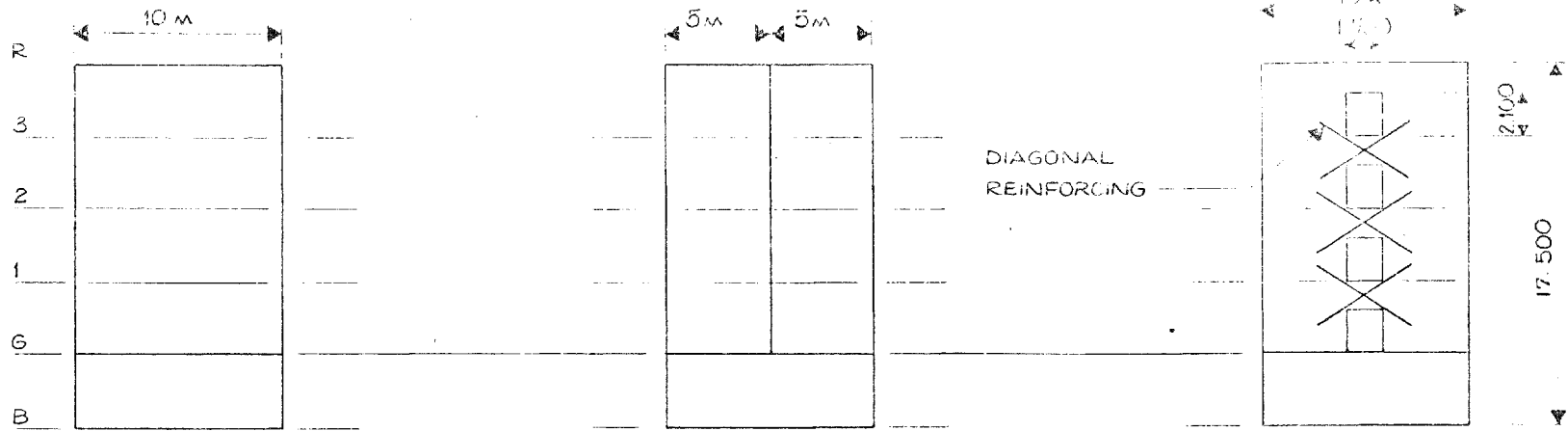
In this study, a variety of shear walls has been designed for four and eight storey buildings. Both ductile walls and walls of limited ductility have been designed, all systems having an overall length of 10 metres. The efficiency of each solution is provided in terms of estimated cost and design effort. Table 1 summarises the eight walls.

DESCRIPTION OF WALLS

An architectural constraint consisting of a 10 metre length was retained for all walls. The floor area for each building height was chosen such that the full dependable strength of each wall in the base region was mobilised when designed according to references (2) and (3). As a result, wall thickness varied at the base between wall types but the tributary floor area was kept constant for each building height. The wall outlines are shown in Figure 1.

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4 STOREY BUILDING

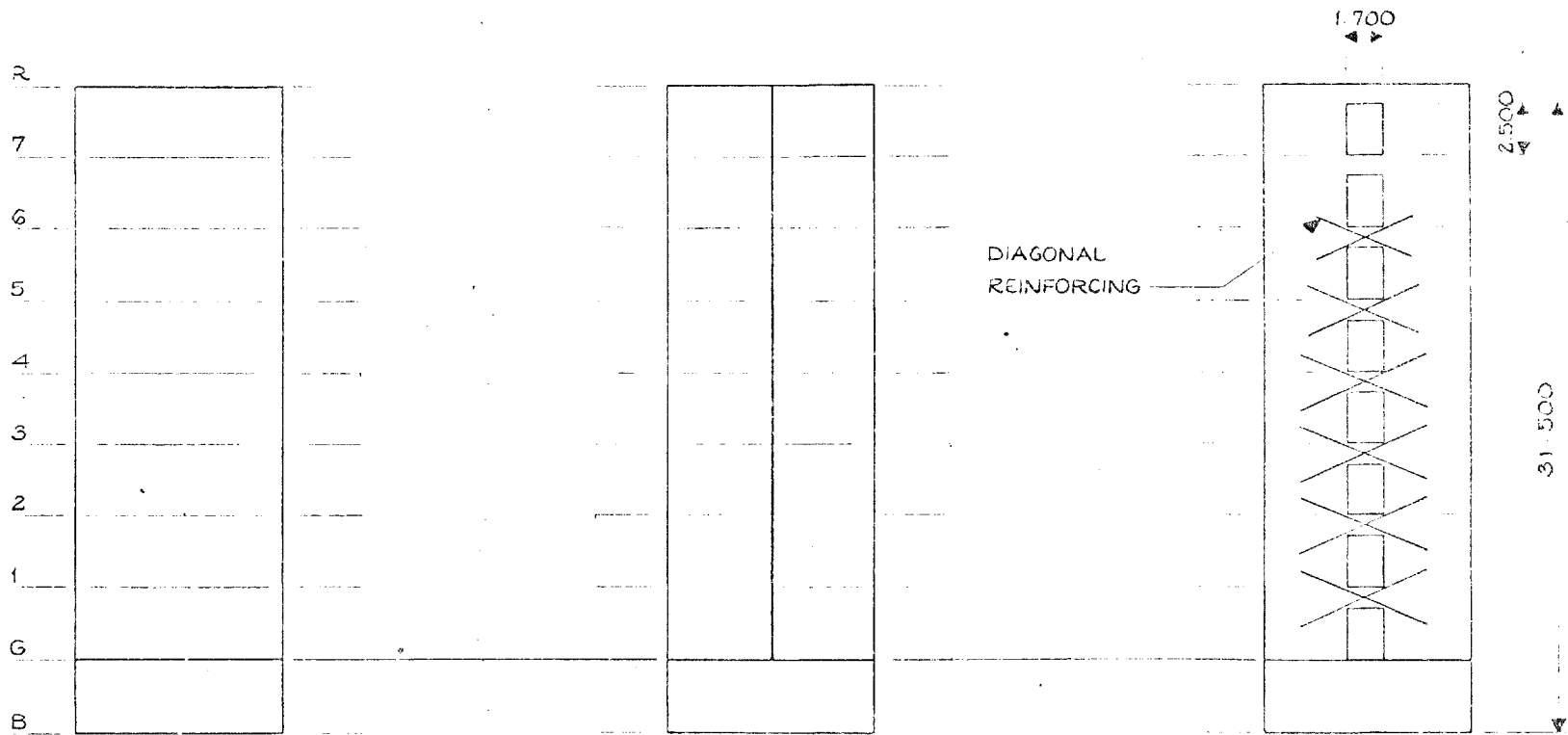


A. DUCTILE CANTILEVER WALL
B. LIMITED DUCTILITY WALL

C. TWIN CANTILEVER WALL

D. COUPLED WALL

8 STOREY BUILDING



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FIG 1. WALL CONFIGURATIONS STUDIED

The buildings were assumed to be situated in seismic zone A, and the risk factor taken as unity. In the third amendment to reference (2), a materials factor of 0.8 is proposed for reinforced concrete. Thus the seismic base shear is

$$\begin{aligned} V &= C_d W_t \\ &= C(T) S M R W_t \\ &= C(T) 0.8 S W_t \end{aligned} \quad (1)$$

where $C(T)$ has dimensions of gravity acceleration and is a function of first mode period T of the structure, and W_t is the seismic weight of the building (usually dead plus one-third of live load).

Three types of wall were considered within the 10 metre length constraint, namely

- a continuous 10 metre wall
- two 5 metre walls with abutting ends
- a coupled shear wall of overall dimension 10 metres.

All of the three systems could be made outwardly identical through lining and thus the same architectural finish retained for all.

APPLIED SEISMIC LOADS

Equation 1 states that the level of seismic loading to be considered for the reinforced concrete structures depends not only on the building weight but also on two other factors, C and S .

Coefficient C has a constant value of 0.15 g for buildings in zone A of natural period less than 0.45 seconds, and reduces linearly to 0.075 g as the period increases to 1.2 seconds. For the structures studied, the natural periods for the assumed cracked wall sections were determined during the course of computer analysis using the ICES STRUDL package. Hence seismic loadings could be adjusted if necessary and the computer output scaled accordingly.

The other variable, the structural type factor S , reflects the amount of ductility required of the shear wall during response to the design earthquake and the higher S , the more nearly elastic or less ductile the response. The lowest value for S , for frames and some coupled shear walls, is 0.8 while an elastically responding reinforced concrete structure is required to be designed for an S of 5.0 (2).

Values of the S -factor were determined for the different walls studied as follows:

1) 10 Metre Walls

According to the Third Amendment to the New Zealand Loadings Code, the structural type factor - S - for a building containing more than one cantilever shear wall in the principal direction being considered depends on the ratio of wall height to length, thus

$$2.0Z \leq S \leq 2.0 \quad (2)$$

where $1.0 \leq Z = 2.5 - 0.5h_w/l_w \leq 2.0$

h_w is overall wall height

l_w is overall wall length

Thus, for the four storey building an S -factor of 1.8 results and 1.1 for the eight storey building. An aspect ratio of less than 1.0 results in a value of Z equal or greater than 2.0 and this implies a shear wall of limited ductility for which a constant value of $S = 2.0$ is to be used.

When an earthquake is resisted by a single wall, a 20 percent increase in strength is required (2) because of the reduced redundancy. Hence

$$1.2Z \leq 2.0 \quad (3)$$

and both sides are equal at an aspect ratio of 1.67. If only a single 10 metre wall resisted earthquake in the case of the four storey building, then

$$\begin{aligned} 1.2Z &= (2.5 - 0.5(4 \times 3.5)/10) \\ &= 2.16 > 2 \end{aligned}$$

and the appropriate design approach is that of "limited ductility" with an S -factor of 2.0.

The designs considered here are for the case where more than one 10 metre wall gives seismic resistance in each direction. Both the four storey and eight storey example buildings can be designed as ductile walls in this situation, with S -factors as derived above from equation 2. However, in order to compare design approaches, both sets of 10 metre walls were also designed using the limited ductility procedure and an S -factor of 2.0.

2) Twin 5 Metre Cantilever Walls

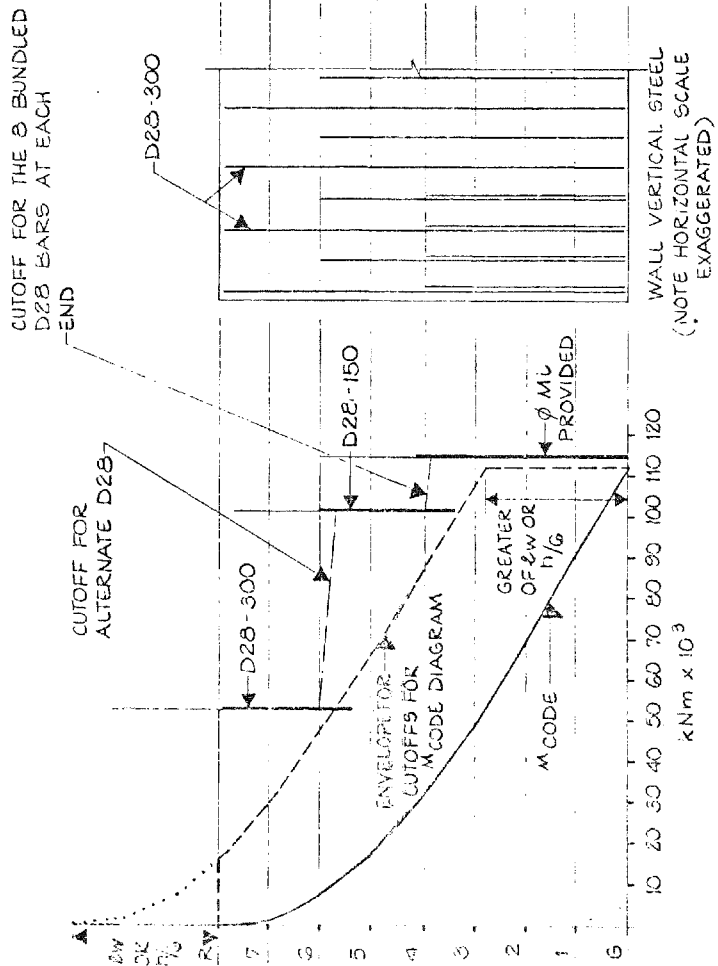
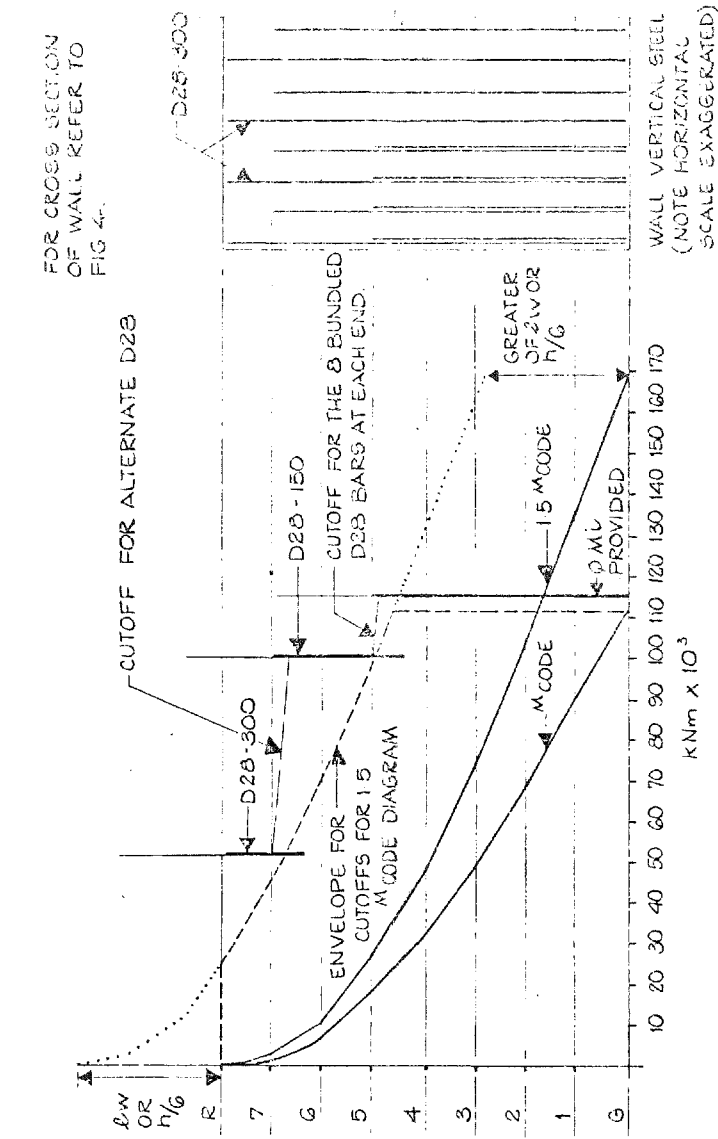
For both buildings, these walls are ductile with an S -factor determined from equation 2 as 1.1 for the four storey building and 1.0 for the eight storey building.

3) Coupled Walls

In determining the geometry of this system the main constraint imposed was that the overall wall length be 10 metres. The size of the opening beneath the coupling beam was set at 2.1 metres high by 1.7 metres wide so that it could serve as a doorway.

The Third Amendment to the Loadings Code (2) prescribes the S -factor for a coupled shear wall as follows:

- a) $A \geq 0.67$, then $S = 0.8$
- b) $A < 0.33$, then $S = 1.0Z \leq 2$
- c) $0.33 \leq A < 0.67$, interpolate between (a) and (b),



A. PERMITTED PROCEDURE

- NOTES: 1. VERTICAL BARS CAN BE TERMINATED SLIGHTLY EARLIER THAN FOR PROCEDURE B
2. DOES NOT SATISFY MAGNIFICATION REQUIREMENTS OF REF (3) CL 14.4.2.2. THEREFORE FULL HEIGHT OF WALL MUST BE DESIGNED AS FOR END REGIONS I.E. TIES REQUIRED THROUGHOUT (CL 14.6.2.).

B. SUGGESTED PROCEDURE

- NOTES: 1. VERTICAL BARS MUST CONTINUE SLIGHTLY FURTHER BEFORE CURTAILING, THAN FOR PROCEDURE A.
2. SATISFIES MAGNIFICATION REQUIREMENTS OF REF (3) CL 14.4.2.2, 14.6.1 OUTSIDE END ZONE, THEREFORE CONFINING TIES NOT NECESSARY ABOVE END REGION (I.E. BEYOND 10M ABOVE GROUND FLOOR).

FIG.2. EIGHT-STOREY LIMITED DUCTILITY WALL FLEXURAL STEEL CURTAILMENT (REF(6))

FOR CROSS SECTION OF WALL REFER TO FIG 4-

where A is the proportion of total overturning moment resisted by all beams, and Z is as defined previously. The larger A, the more slender the walls for constant coupling beam geometry, and the more frame-like the response of the system. A value of S of 0.86 was determined for the four storey building. A lower S could have been achieved by widening the openings. However, if the openings were made wide enough to achieve S = 0.80, the structure would become too flexible and exceed the limitations on interstorey drift imposed in the loadings code⁽²⁾. In the case of the eight storey structure, an S = 0.80 was obtained with 2.1 x 1.7 metre openings while still satisfying drift limitations.

DETERMINATION OF WALL THICKNESS

1) Analytical Model

It was assumed that both buildings contained a basement. In such a situation, a large seismic shear is reacted at the ground floor level through the floor slab which acts as a "transfer diaphragm"⁽⁴⁾ and sheds load to the perimeter retaining walls. A shear of reversed direction exists in the shear wall between basement and ground floor level. The value of this force may be very high and it is very sensitive to the model - in particular, to whether the ground floor diaphragm has finite or infinite stiffness and to whether the base is fully fixed against rotation, pinned, or modelled on springs⁽⁵⁾. Further, particularly when the wall is longer than the interstorey height, it is important to explicitly model the shear stiffness as well as flexural stiffness rather than treat the wall total stiffness as that of an equivalent flexure-only cantilever. The difference is shown for one example in Figure 5.

2) Stability of Wall Edge

For walls designed to the "ductile" requirements, the thickness of that part of the compression zone within the end region where reinforcement is yielding may not exceed one-tenth of the distance between effective lateral supports (usually the storey height)⁽³⁾ (clause 10.5.2.1). This applies only when the length of yielding zone exceeds twice the wall thickness. On the other hand, the general (non-seismic) limitation in width (1/25 of unsupported distance) applies to walls designed⁽³⁾ using the limited ductility approach (clause 14.3.1 and clause 10.3.2.1).

In this study, the thicknesses within end regions were generally determined by the upper limit on v_1 , the total shear stress, except for the four storey twin 5 metre walls. The stability requirement will often be met in practice by adjoining walls or may be achieved by a local thickening at free edges of walls.

3) End Region

The "end region" of a shear wall is the

region in which plastic hinges may be expected to form under severe seismic loading. For both ductile walls and walls of limited ductility, the end region is generally of height equal to the length of the wall or one-sixth of its total height, whichever is the greater⁽³⁾ (clause 10.5.5.3 and 14.5.2). This distance is to be measured up from the point of maximum moment, and thus, for the walls considered in this study, from the level of the ground floor.

The end region for the 10 metre walls, from the above criteria, extended to just under three storey heights above ground.

Even though there is a basement, the "end region" is correctly measured upwards from the level of the ground floor for the prescribed⁽³⁾ distance. This is the level where maximum curvature occurs in a cantilever shear wall and hence where the potential for initiation of a plastic hinge exists. However, the plastic hinge may also spread downwards from the ground floor and for this reason as well as the need to maintain the same degree of protection against shear failure as exists immediately above the ground floor, a magnified value of applied shear should be used for design of wall between ground floor and foundation level.

4) Limit on Total Shear Stress

It is important for the designer to appreciate the effect of shear strength/shear stress requirements at the stage of preliminary sizing of walls. In order to minimise the likelihood of shear failure, the seismic code loading must be magnified by a minimum factor 2 in the case of walls of limited ductility. The degree of magnification is not spelled out for ductile walls, nevertheless "appropriately modified capacity design procedures shall be used to ensure that the ideal shear strength of walls is in excess of the shear force⁽³⁾ when flexural overstrength is reached"⁽³⁾ (clause 3.5.7.3). A multiplier, w_{ϕ} , is recommended (refer⁽³⁾ clause 3.5.7.3) for ductile walls, which has a minimum value of about 2.1. On the other hand, while (for 30 MPa concrete) the total shear stress can approach 6.0 MPa at any section of walls of limited ductility (refer⁽³⁾ clause 7.3.14.3), maximum total shear stress permitted in the end region of ductile walls is a function of structural type factor (S) thus:

$$V_1 = (0.3\phi_0 S + 0.16)\sqrt{f'_C} \quad (4)$$

(refer. (3), clause 7.5.5.2)

where ϕ_0 the overstrength factor, can only be finally calculated once detailing of flexural steel is completed (a value of 1.4 is a reasonable initial assumption), and f'_C is the specified compressive strength of the concrete.

The capacity reduction factor ϕ may be taken equal to unity when designing a section for shear forces obtained from the overstrength of adjacent members or sections. However, when the limited ductility design procedure is followed, the

Design	4 storey and 8 storey building
Ductile	10 m walls
Limited ductility	10 m walls
Ductile	Twin 5 m walls
Ductile	Coupled walls

TABLE 1

Summary of Wall Types Designed

$\frac{h}{e}$	Height of End Zone Wall height	Ductile				Limited Ductility		
		S	K	Within End Zone v_i	Outside End Zone K/v_i	K/v_i $\frac{2.0 \times 2.0}{6}$		
1.0	1.0	2.0	4.20	5.48	0.77	6.0	0.70	0.67
1.5	0.67	1.75	3.67	4.91	0.75	6.0	0.61	0.67
2.0	0.50	1.5	3.15	4.33	0.73	6.0	0.53	0.67
2.5	0.40	1.25	2.63	3.75	0.70	6.0	0.44	0.67
3.0	0.33	1.0	2.10	3.18	0.66	6.0	0.35	0.67
3.5	0.29	1.0	2.10	3.18	0.66	6.0	0.35	0.67

TABLE 2

Comparison of Load/Strength with different design approach and wall aspect ratio

WALL	TIE SIZE		
	Bar Diameter	Total Length of Steel per Tie	TOTAL NUMBER OF TIES
Ductile:			
Single 10 m: 8-storey	12 mm	1.10 m	600
: 4-storey	-	-	-
Limited Ductility:			
Single 10 m: 8-storey	10 mm	0.90 m	560
: 4-storey	10 mm	0.90 m	420
Ductile:			
Twin 5 m: 8-storey	16 mm	1.85 m	680
: 4-storey	16 mm	1.25 m	400
Ductile:			
Coupled : 8-storey	12 mm	1.45 m	154
: 4-storey	10 mm	1.55 m	248

TABLE 3

Comparison of Extent of Confining Ties
with Different Design Approach

usual ϕ for shear must be used.

The recent amendment to the Loadings Code (2) (clause 3.3.6.1) permits slender walls (that is, in which flexural effects dominate due to their large total height to length ratio) to be designed using the approach for walls of limited ductility, should the designer so choose. It is thus instructive to compare benefits between the two approaches from the point of view of minimising wall cross-section.

Let the base applied seismic shear force, $L = KP$, where P is the load at $S = 1$ and K is the multiplier necessary to obtain design applied shear, L at wall base.

From the preceding, for a wall of limited ductility

$$K = 2 \times S = 4$$

while for ductile walls

$$K = w\phi_0 S \\ = 2.1S.$$

Let wall length, l_w , and thickness, b_w , be fixed. Hence, a change in S results from a change in total height, h_w , of wall through equation (2).

Table 2 obtains the factor K/v_i for both design approaches and for a range of aspect ratios (h_w/l_w), assuming two or more walls make up the resisting system. These factors may be regarded as indicating the relative amounts of wall cross-sectional area needed, the higher value implying a greater area (3) is required to satisfy code requirements.

In the case of the "ductile" design approach, the maximum total shear stress (v_i) is allowed to increase substantially outside the end zone (up to nearly twice the value and depending on wall aspect ratio - see Table 2). The commentary to the Concrete Design Code (3) recommends that use of a magnification factor ($w\phi$) for applied shear be retained outside the end zone and this has been followed in preparing Table 2.

There is little difference between values of wall cross-sectional areas required within the end zone for either design approach, but the advantage of a "ductile" design approach becomes rapidly apparent outside of the end zone with increasing aspect ratio. A greater rate of reduction of wall area with height is therefore possible for the "ductile" design.

In the case of all the four storey wall designs, the basement to ground floor wall thickness was increased to 500 mm to accommodate shear strength requirements. A thickness of 600 mm was needed in this area for the 10 metre eight storey wall designed to either design approach. The aspect ratio for these walls is 2.8 and Table 2 indicates that similar shear area would be required in the end region for the two approaches. However, a 500 mm thickness at base was sufficient for the more slender twin 5 metre wall as well

as the coupled shear wall for the eight storey building.

For the design examples, the wall cross-section dimensions were not altered above ground floor level. Clearly, there is greater scope for reduction here in the case of a ductile design approach but economies here would be partly offset by the need for more flexural and shear reinforcement.

The Concrete Design Code does not deal with the method of analyses of coupled shear walls, but refers to reference (1). A crucial equation in this reference involves the distribution of seismic maximum shear force. This is considered to be proportional to the value of flexural overstrength at that level (the base of the walls). The enhancement of flexural strength due to the effect of seismic-induced axial load in a compression wall is considerable compared to the flexural strength of the "tension" wall, and hence the distribution of shears is uneven in terms of

$$V_i = w_v \phi_0 \left[\frac{M_i^O}{M_1^O + M_2^O} \right] V_{\text{code}} ; i = 1, 2$$

where w_v = dynamic shear magnification factor

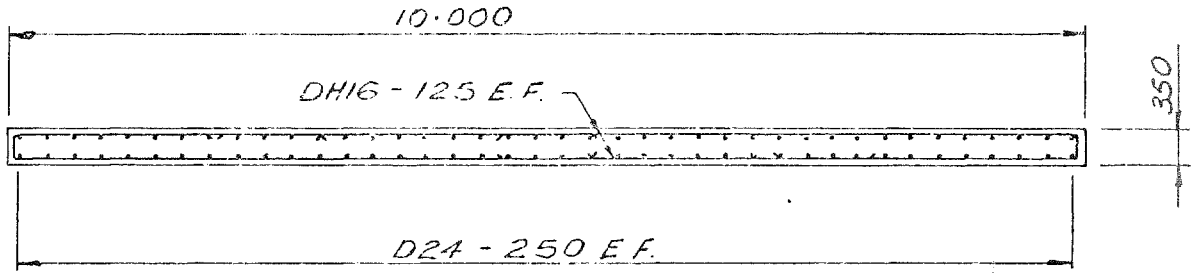
ϕ_0 = overstrength factor

M_i^O = overstrength moment at base of wall i .

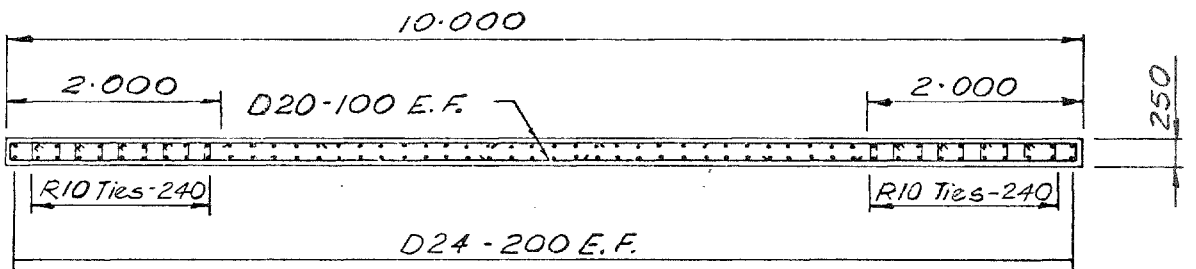
A more even division of shear can be obtained only by increasing the flexural strength of the tension wall without at the same time making it stronger when in compression. This was thought desirable, and accordingly, longitudinal reinforcement was concentrated in the outer ends of the wall set as shown in Figure 3 for the four storey building. The effect was to reduce the ratio of total shear stress between the two walls from 5.7:1 to 4.3:1. The end thickenings were terminated at the extent of the end zone (third floor level).

The additional overturning effect in the eight storey building is not compensated for by a greater gravity load on the walls because most of the gravity load is taken by columns. However, all of the seismic load is assumed to be taken by shear walls and the difference between seismic and gravity-induced axial load increases with the number of stories. Hence, a concentration of reinforcement at the outer edge would still not result in a significant value of flexural strength of the tension wall when compared to that of the compression wall. In addition, Paulay and Williams (1) recommend an even distribution of vertical reinforcement at the wall base to help to prevent the situation of a few large cracks arising and forming a potential plane of sliding.

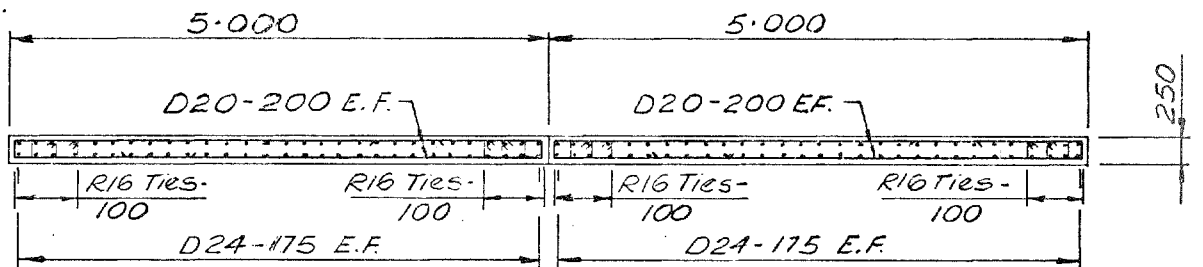
Thus, retaining an even pattern of vertical reinforcement for the eight storey



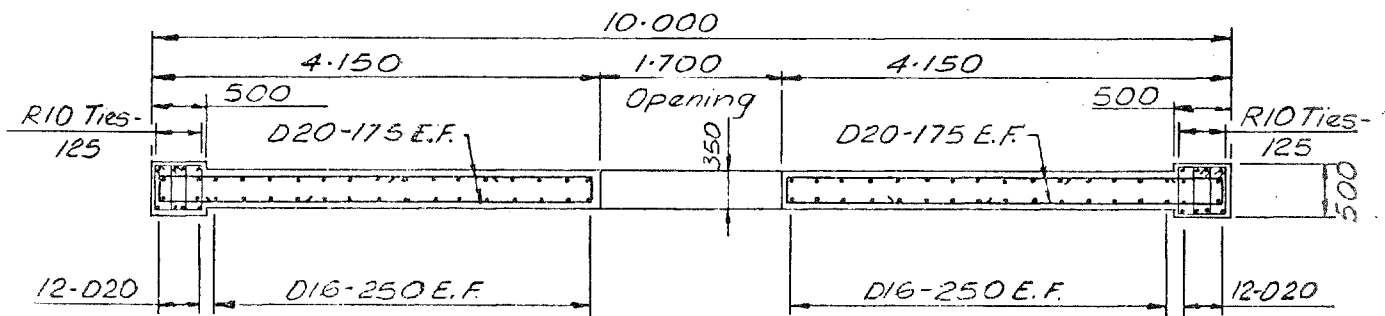
A. 10m. long cantilever wall designed to ductile provisions of Code.⁽³⁾



B. 10m. long cantilever wall designed to limited ductility provisions of Code.⁽³⁾



C. Twin 5m. long cantilever walls.



D. Coupled shear walls.

Fig.3 4 STOREY BUILDING - Wall cross sections at ground level.

wall set resulted in the situation where nearly 90 percent of the overstrength shear was resisted by the compression wall. A 500 mm thickness at base level was sufficient to keep total shear below the maximum value.

"END REGIONS" AND CURTAILMENT OF FLEXURAL REINFORCEMENT

To prevent the plastic hinges being more extensive, the commentary section of the Code for Design of Concrete Structures⁽³⁾ recommends that, for ductile cantilever walls, vertical reinforcement be terminated so that "the wall's ideal moment of resistance reduces linearly from the end of the potential plastic hinge zone to the value of the design moment at the top of the structure"⁽³⁾ (clause 3.5.7.3). (See also reference (1) page 121.)

The very similar S factors in the four storey 10 metre walls (1.8 and 2.0) resulted in nearly identical quantities of vertical reinforcement for these walls. For the 5 metre walls, the governing extent of end region is 5 metres for both the four and eight storey walls, half that of the 10 metre walls and so curtailment of vertical reinforcement can begin earlier for the 5 metre walls.

For all splices of principal vertical reinforcement in ductile walls, ties spaced at not more than ten times the main bar diameter must surround the splice⁽³⁾ (clause 10.5.8.2). However, in the end regions of ductile walls, only one-third of the longitudinal reinforcement can be spliced at a particular level⁽³⁾ (clause 10.5.8.1). Coupled with the requirement (reference (3), clause 10.5.8.2) that "stagger between splices shall be not less than twice the splice length", the situation arises for a ductile wall in excess of about 11 metres in length, and requiring D28 vertical bars, that there is nearly a three storey interval between successive splices on a bar. This could impose a considerable demand on the cost of steel fixing.

The end region of a coupled shear wall would normally be related to the length of one of the walls coupled rather than the length of the assemblage. The exception is when the coupling beam is so stiff that the deflected shape of the assemblage approaches that of a single cantilever wall of overall length equal to that of the assemblage.

CONFINING REINFORCEMENT TIES AND COMPRESSION EDGE REQUIREMENTS

Transverse reinforcement is required to:

- restrain longitudinal compression reinforcement against buckling
- give protection against failure to compression zones of wall concrete when this zone is sufficiently deep.

In order to avoid buckling of the free compression edge (no lateral walls giving

support) when the neutral axis is relatively distant, the "ductile design" approach requires that the thickness of wall with strains greater than 0.0015 not exceed one-tenth of the vertical clear distance between floors (in the case of the designed walls, $0.1 \times 0.1 \times 3.300$ metres = 330 mm). No such limitation on thickness is made in the "limited ductility" design approach. In that case, confinement reinforcement in the end region is always required over 20 percent of the wall length from either end when the percentage of longitudinal reinforcement in this region exceeds about 1 percent. A greater quantity of confinement reinforcement must be provided here when the estimated compressive stress in this portion of wall length exceeds $0.2 \phi f'_c$, in order to prevent compression failure of the concrete.

In the case of walls of limited ductility, the need for special transverse reinforcement outside the end region is cancelled provided that the dependable flexural strength outside of the end zone is 50 percent greater than that required by the Loadings Code⁽²⁾ (see reference (3) clause 14.4.2.2). Figure 2 shows the two major options available: Procedure B is more attractive than prolonging the extent of the labourwise costly confining ties.

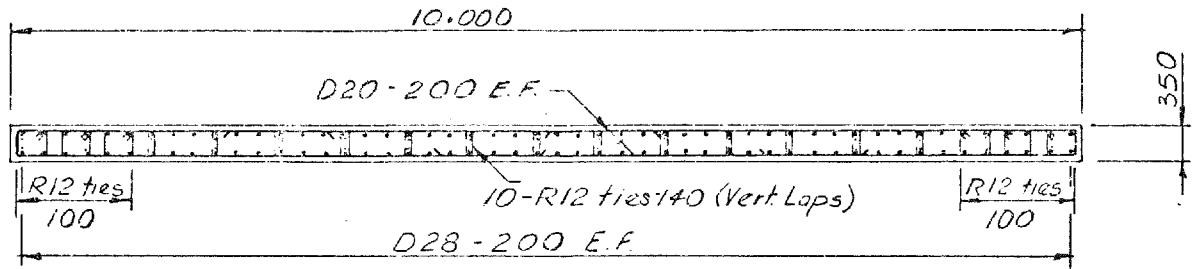
Table 3 indicates that the total weight of confining ties for the eight storey 10 metre wall is reduced by about 50 percent when the "limited ductility" design approach is used (Procedure B - see Figure 2) instead of the "ductile" approach. On the other hand, no ties were required in the case of the four storey 10 metre wall designed to the "ductile" provisions, and thus the most attractive of the two approaches changes with wall height. By far the smallest number of ties was required for the coupled shear walls.

Cantilever walls of a lesser number of stories may, on the other hand, require fewer confining ties if designed by the "ductile" approach. Owing to the significant reduction in bending at the base of the ductile coupled walls, fewer confining ties were required here than for the pure cantilever walls.

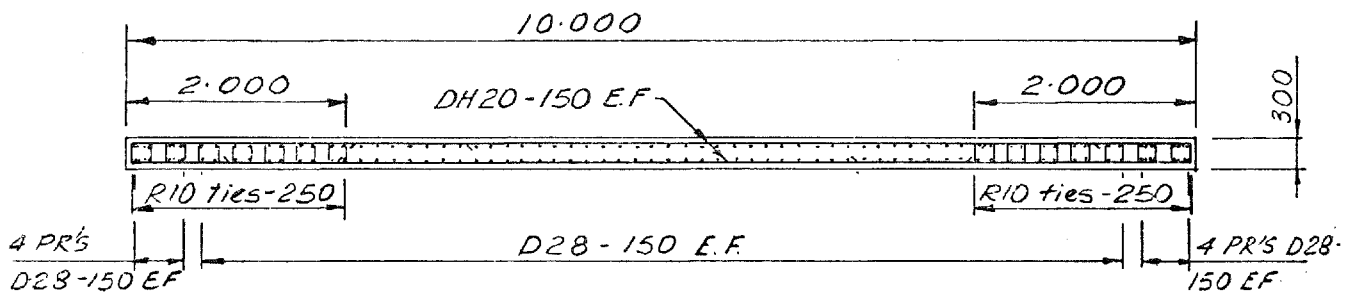
The ductile twin 5 metre walls have a lesser total height of ties, but the number tabulated allows for the four ends of the wall set.

A reduction in number of ties should facilitate wall construction and hence, logically, the cost. However, the total weight of ties is very small compared to the remainder of reinforcing steel and if the same unit cost (dollars/kilogram) for "supply and place" of ties is used as for the remainder of the reinforcement, then the reduction in difficulty of construction will not be completely reflected in the cost difference between the design approaches.

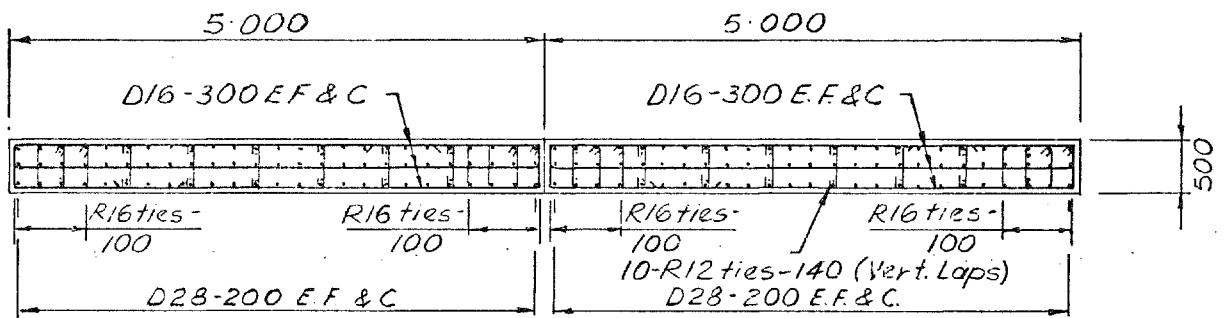
No differentiation in unit rate for various types of reinforcement was made in the quantity survey for the different walls designed in this study.



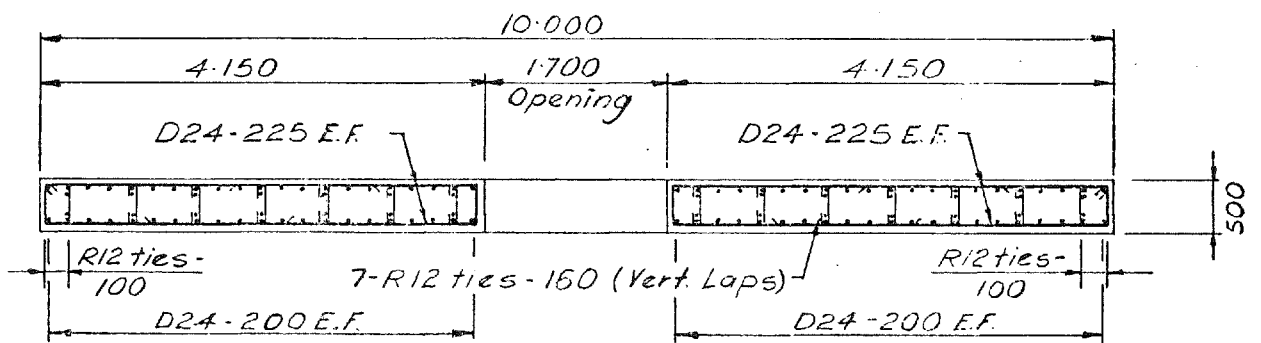
A. 10m. long cantilever wall designed to ductile provisions of Code.⁽³⁾



B. 10m. long cantilever wall designed to limited ductility provisions of Code.⁽³⁾



C. Twin 5m. long cantilever walls.



D. Coupled shear walls.

Fig. 4 8 STOREY BUILDING - Wall cross sections at ground level.

ESTIMATED COSTS OF DIFFERENT WALL DESIGNS

Table 4 compares cost, including that of wall foundation.

These costs were estimated from detailed drawings by the Quantity Survey of the MWD. In determining the percentage of total structural cost, it was assumed that the building consisted of two sets of the 10 metre overall walls in each horizontal direction. The figures for the eight storey building are more than double those of the four storey building. This is because not only is twice the area of wall required to support essentially the same total floor area when it is contained in twice the number of stories, but considerably more vertical reinforcement is needed. The coupled wall looks economically attractive in both cases. The costs do not merely reflect the volume of concrete used for, in fact, the coupled wall had the second highest quantity of concrete, but was the cheapest in the case of the eight storey building.

The use of high yield (Grade 380) reinforcement leads to a significant saving although its use should be restricted in the main to areas where significant ductility demand on the steel is not possible because of the reduced ultimate strain compared to Grade 275 reinforcement. For example, a saving of 7 percent of the total wall cost is involved for the four storey ductile example when Grade 380 steel is used as horizontal reinforcement.

The saving between the options studied is little more than 1 percent of the total structural cost. Hence there is not a case established in favour of a particular design from the economic point of view for the structures studied.

However, if a separation of non-structural elements could be avoided by designing one of the stiffer, possibly "limited ductility" options available, then the saving here would probably more than offset any additional structural cost. In the case of the walls designed, however, inter-storey deflections were all in excess of $0.0006 \times$ interstorey height, the maximum deflection at which non-separation is permitted in the code⁽²⁾. (A review of deflections for the 10 metre four storey wall with zero deformation permitted in ground floor slab resulted in deflections being more than halved. However, the reduction was insufficient to avoid the need for separations, because of the high forces being resisted by the wall.)

COMPARISON OF DESIGN EFFORT REQUIRED

At the time of this study, use of the 1982 "Code of Practice for the Design of Concrete Structures"⁽³⁾ is still in its beginnings, and designers will not be familiar with the detail of the document for several months to come.

Our experience was that requirements for both the "ductile" and "limited ductility" design approaches took some effort to assimilate the first time round. However,

the latter approach does contain sufficient simplifications to make it worthy of consideration for design of even moderately slender walls (overall height to length ratio of up to about 3) notwithstanding the additional flexural reinforcement that is to be expected.

With experience, a designer will not be daunted by either design approach, and he will be wise to give first consideration to the "ductile" approach for slender shear walls. A coupled shear wall would normally be designed by the "ductile" approach. However, for this wall type it is particularly difficult to determine earliest points of curtailment of flexural reinforcement, for two reasons, viz:

- The bending moment diagrams are considerably different between "tension" and "compression" wall because different moments of inertia are used for each wall⁽¹⁾.
- The flexural strength of the walls varies greatly with height because of the effect of changing axial load which is transferred in large increments at each floor level from shear in the coupling beams. Thus, for a given quantity of vertical reinforcement, the tension wall gains strength with height and the compression wall becomes weaker. A further detraction in the design procedure for ductile coupled shear walls is the calculation of the overstrength factor ϕ , the effort for which is considerably greater than in the case of a cantilever wall.

For the more common low- and moderate-rise shear wall structures, the "limited ductility" approach is more attractive.

Table 5 shows the relative number of calculation pages required for the eight storey walls. In the situation of designing a set of walls, the results of analysis for the "ductile" example were scaled directly for the "limited ductility" wall. The relative value for the latter example would only rise to a figure of about "0.67" (Table 5) had the benefit of the previous analysis not been taken advantage of. Hence, the "limited ductility" approach is attractive.

The procedure for "elastically responding structures" should appeal to designers of low-rise, low slenderness ratio shear wall structures. However, a considerable increase in the quantity of vertical reinforcement may be required. For example, nearly three times the amount is required at the base of the four storey example wall compared to the "ductile" design. Nevertheless, the situation will often arise in practice where a wall such as this may require little more than nominal reinforcement to respond elastically to the design earthquake, in which case the "elastically responding" design procedure is the most attractive.

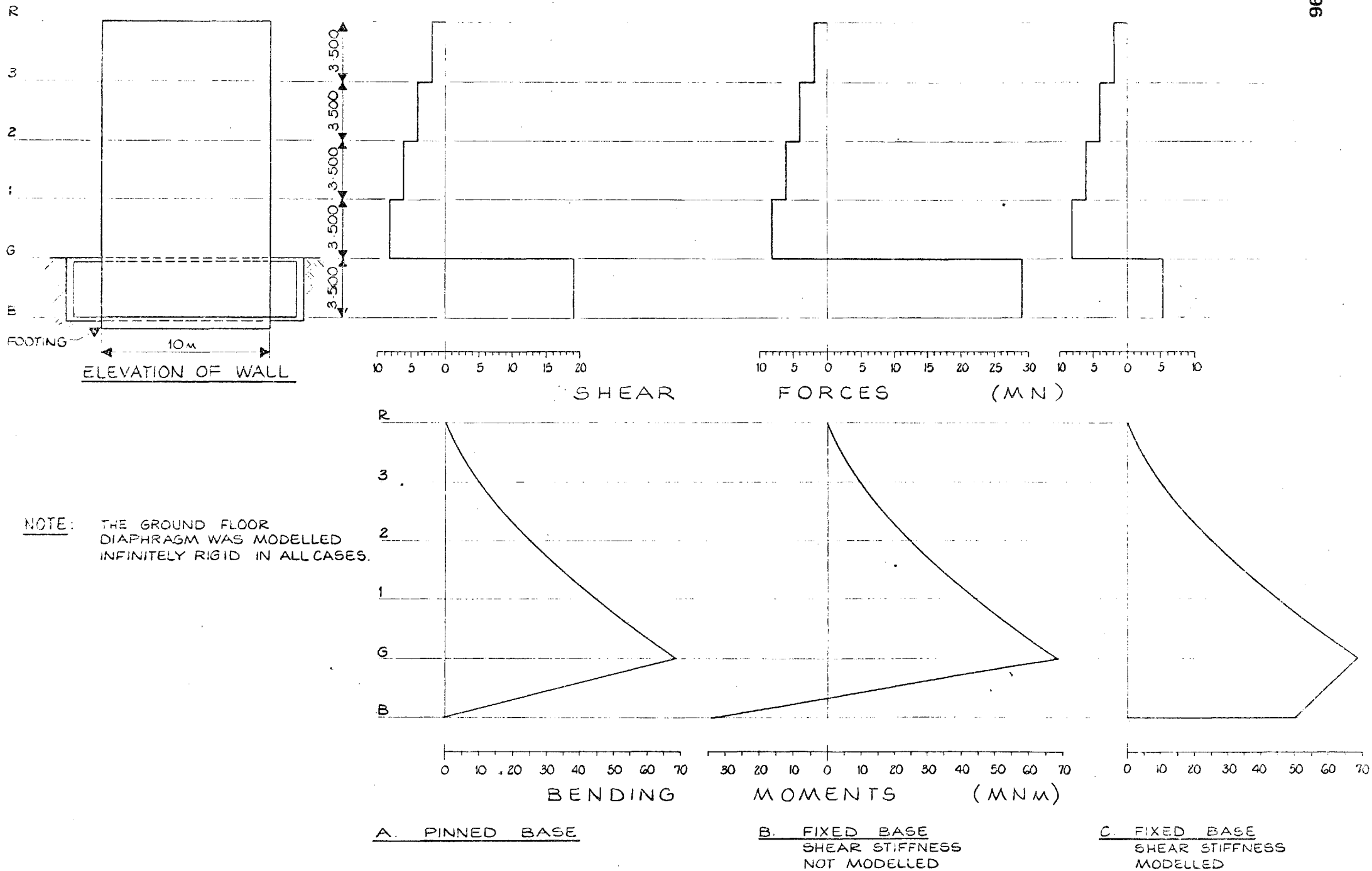


FIG.5. VARIATION OF SHEAR FORCE AND BENDING MOMENT WITH COMPUTER MODELLING ASSUMPTIONS

CONCLUSIONS

The study has considered the approaches available in the recently published "Code for Design of Concrete Structures" by applying them to a selection of highly stressed walls.

Although the difference in cost of the various designs can be quite significant, this is not great when expressed as a proportion of total building structural cost.

For walls of height to length ratio of up to about 3, a designer will do well to consider the "limited ductility" approach as a first design option, because of the reduced design effort required.

ACKNOWLEDGEMENTS

The permission of the Commissioner of Works to publish this study is gratefully acknowledged. The helpful comments of Mr G H F McKenzie and Mr G R McKay are appreciated.

GLOSSARY OF TERMS

\emptyset	strength reduction factor
V_i	ideal shear strength of a section (Newtons)
v_i	total shear stress (MPa)
C_d	seismic design coefficient (g)
$C(T)$	basic seismic coefficient, a function of natural period (T)
S	structural type factor
M	structural material factor
R	risk factor
W_t	total reduced gravity load above the level of imposed lateral ground restraint
\emptyset_o	ratio of overstrength moment of resistance to moment resulting from code specified loading, where both moments refer to the base section of wall
f'_c	specified compression strength of concrete
w	dynamic magnification factor

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a Four Storey Building

WALL		COST	COST AS PERCENTAGE OF TOTAL STRUCTURE
Ductile:	Single 10 m	\$ 50,043	3.4
Limited Ductility:	Single 10 m	\$ 54,873	3.7
Ductile:	Twin 5 m	\$ 65,200	4.4
Ductile:	Coupled	\$ 57,100	3.9

b Eight-Storey Building

WALL		COST	COST AS PERCENTAGE OF TOTAL STRUCTURE
Ductile:	Single 10 m	\$131,582	9.0
Limited Ductility:	Single 10 m	\$146,431	10.1
Ductile:	Twin 5 m	\$145,593	10.0
Ductile:	Coupled	\$126,071	8.7

TABLE 4

Costs of Designed Walls

EIGHT-STOREY WALL	DESIGN EFFORT
10 m Ductile	1.0
10 m Limited Ductility	0.5 (0.67)
2 x 5 m Ductile	1.0
Coupled	1.33

TABLE 5

Relative Design Effort for Eight-Storey Walls