EQC

Earthquake And War Damage Commission

EARTHQUAKE RISK ASSOCIATED WITH 1935 - 1975 REINFORCED CONCRETE BUILDINGS IN NEW ZEALAND

E.L. Blaikie D.D. Spurr

WORKS Consultancy Services

A DIVISION OF Works and Development Services Corporation (NZ) Ltd

CONTENTS

PAGE

SECTION 3 : STRUCTURAL WALLS YIELDING IN A COMBINED SHEAR AND FLEXURAL MODE

INTRODUCTION AND OVERVIEW

This report gives the results of an investigation into the earthquake risk associated with New Zealand's 1935-1975 reinforced concrete building stock.

Current New Zealand legislation on earthquake risk buildings (Local Government Act 1974, Section 624) covers only unreinforced masonry and unreinforced concrete buildings. This type of construction was not permitted in New Zealand for main structural elements after the introduction of the 1935 building code.

However buildings constructed using reinforced concrete have collapsed in previous earthquakes. This was highlighted recently by the collapse of the Cypress Street Viaduct which killed 42 people during the Loma Prieta earthquake. The viaduct was a two level, elevated freeway structure. It was designed in 1951 and completed in 1957 and collapsed due to brittle behaviour of the columns supporting the upper level of the highway. This demonstrated once again the potential seismic vulnerability of early reinforced concrete structures.

The objective of this research project was to obtain an assessment of the type and extent of problems associated with reinforced concrete buildings of the 1935 to 1975 era and hence to obtain a more realistic assessment of their vulnerability to earthquake damage.

It is expected that the results of this investigation will assist with a more rational planning approach to the use of these buildings and a more accurate assessment of their seismic risk for insurance purposes.

The report is presented in three sections:

> **Section 1** details the results of a survey of the buildings constructed within the non residential part of Wellington City between 1935 and 1975. Wellington was chosen for the survey because its associated seismic risk is greater than that for any other New Zealand city.
>
> The characteristics of the buildings were identified in the survey as the first step that is required to establish the size and nature of the earthquake risk associated with this group of buildings.
>
> The survey indicated that the floor area of 1935 to 1975 reinforced concrete buildings in the surveyed area constitutes approximately 10% of the total floor area of all buildings in Wellington City.
>
> The survey also indicated that 78% of the floor area of the 1935 to 1975 reinforced concrete buildings is concentrated in buildings with four or more storeys and that 86% of this floor area was constructed during the 1960s and early 70s.

Section 2 of the report gives the results of a preliminary evaluation of a range of typical 1935 to 1975 Wellington buildings. The evaluation identifies common potential deficiencies in the buildings.

An assessment was then made of the risk of damage or collapse associated with the potential deficiencies. This assessment was based on a review of the performance of reinforced concrete buildings in previous earthquakes. It was concluded that many of the potential deficiencies identified would result in strength and stiffness degradation. This aspect of the buildings likely behaviour was then examined by a review of previous research on the effects of strength and stiffness degradation.

Extensive research indicates that the effects of stiffness degradation are not as significant as they were once thought to be. However more research is required to clarify the effects that strength degradation has on the response of structures.

A tentative method for evaluating structures exhibiting strength and stiffness degradation is proposed for future development.

Section 3 of the report details an investigation of two reinforced concrete shear wall buildings built in the late 1950 and early 1960s. The walls were modelled so that they could yield in both shear and flexure at the base of the wall and analysed using inelastic dynamic analysis.

Current New Zealand design standards are intended to preclude significant shear yielding.

The analysis indicated that earthquakes could impose large inelastic shear displacement demands on walls if they are not designed to these standards.

A tentative procedure for evaluating the seismic performance of walls that are likely to be subjected to significant inelastic shear displacement demand is proposed for future development.

Each of the three sections of the report concludes with a summary and conclusion.

SECTION 1

WELLINGTON BUILDING SURVEY : 1935 TO 1975

REINFORCED CONCRETE BUILDINGS

1.1 OBJECTIVE OF SURVEY

The survey was carried out to determine the number and types of reinforced concrete buildings built between 1935 and 1975 in Wellington City. This was considered to be the first step required to establish the size and nature of the earthquake risk associated with this group of buildings. Wellington was chosen for the survey because its associated seismic risk (loss x frequency) is far greater than that of any other New Zealand city.

1.2 SURVEY METHODOLOGY

The survey area of Wellington selected for study is shown in Figure 1.1. Two additional areas covering Wellington Hospital in Newtown and the high rise residential area in Oriental Parade were also included.

The initial data base for the survey area was obtained from Valuation New Zealand and the boundaries of the area correspond to Valuation NZ roll area boundaries.

The total floor area for each of the main use categories was calculated for this total survey area from the data base (see Figure 1.2).

Buildings with less than two storeys and those built outside the 1935 to 1975 period of interest or known to be constructed of materials other than reinforced concrete were then culled from the survey area data base. Valuation NZ data does not define the number of storeys a building has. Also, the data contains a large number of buildings without a defined date of construction. These are principally buildings built prior to 1930 and buildings or building complexes built in stages. Therefore, to obtain a "1935 to 1975 reinforced concrete building" data base it was necessary to upgrade the Valuation NZ data using a combination of Wellington City Council's "Earthquake Risk Buildings List" and "Wellington City Scope"[28]. WORKS "Design Features Reports" and "Building Survey Data For Public Buildings" were also used.

Wellington City Council's "Earthquake Risk Building List" provides data on age of construction, number of storeys, size and some information on construction materials. This data helped to cull buildings built prior to 1935 and those not constructed using reinforced concrete from the "1935 - 1975 reinforced concrete building" data base.

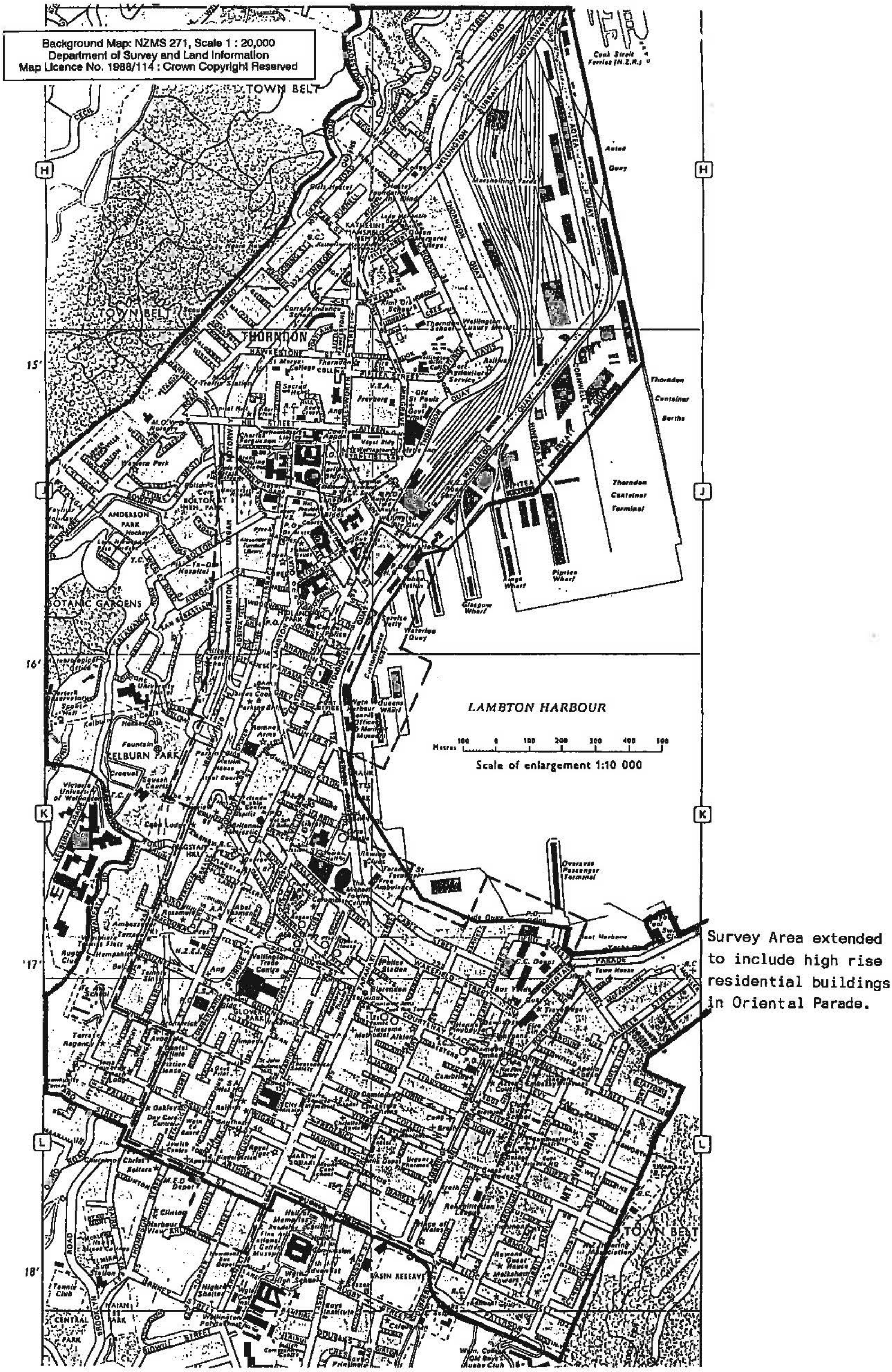


Fig.1.1 **Boundaries of Survey Area** (City Scope Boundaries shown dotted)

*includes approx. 5% by Number & 1.5% by floor area, 1930's and 1970's buildings built outside age range 1935-75 or buildings with steel frames.

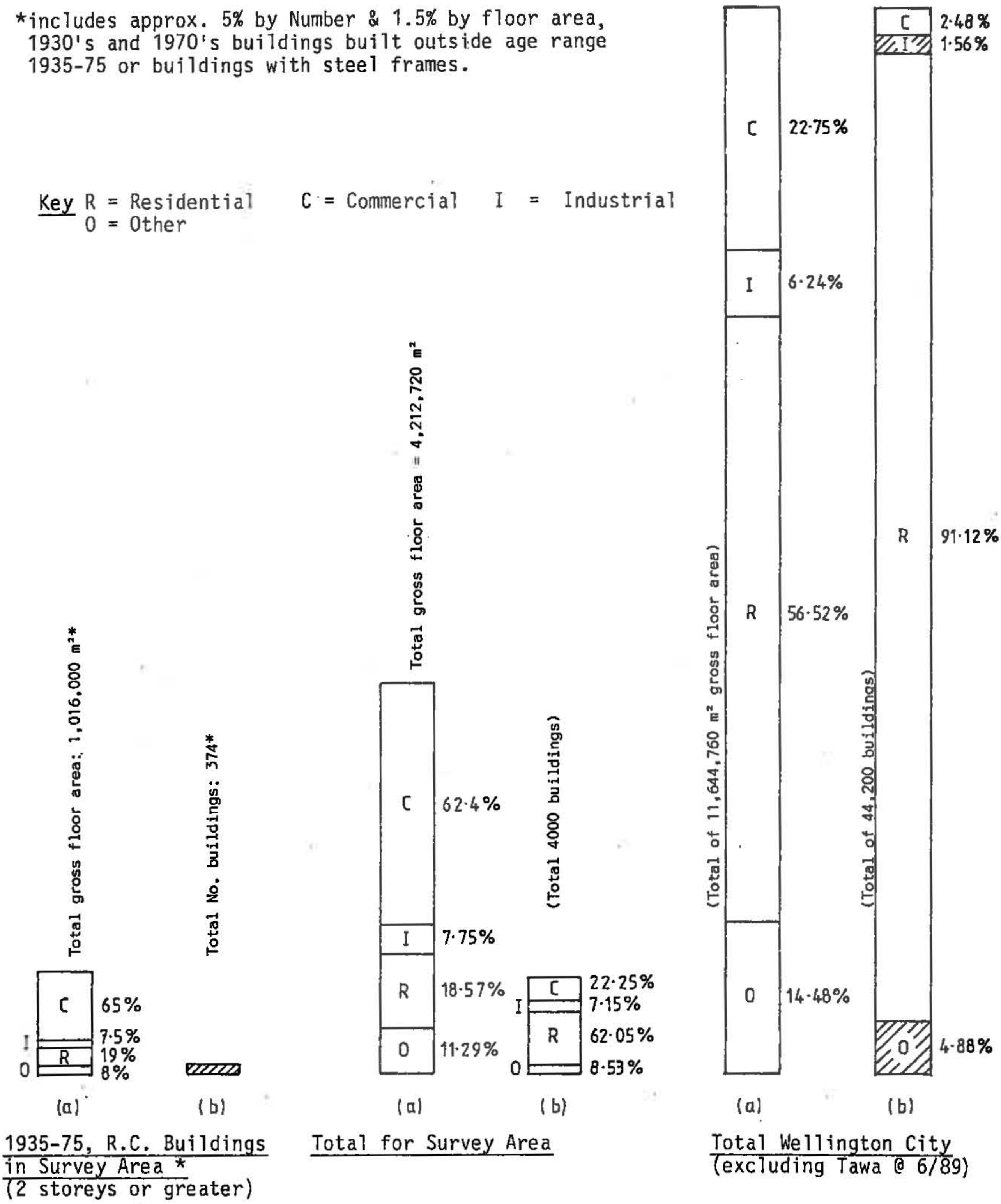


Fig.1.2 Comparison between Buidings in "Total Wellington City", the "Total Survey area" and "Target 1935-75 buildings" (a) proportion by area (b) by number of buildings.

"Wellington City Scope" contains a brief description of the history and characteristics of most buildings in the central part of Wellington City. Boundaries of the area covered by "Wellington City Scope" are shown in Figure 1.1. The City Scope data was particularly useful as a means of identifying the number of storeys and the age of buildings built in the 1930s and 1970s that were outside the 1935 to 1975 period of interest.

Data for buildings within the survey area but outside the "City Scope" area, was obtained by a street survey or, in the case of Wellington Hospital, from information supplied by the Hospital Board.

For buildings designed by WORKS, "Design Features Reports" prepared by designers prior to construction provided comprehensive information on all aspects of the buildings. Information on the age, floor area and type of construction of most other government leased or owned buildings was available from a computer file "Building Survey Data for Public Buildings" held by WORKS.

Where the number of storeys was not identified from these other sources the ratio of gross floor area to site coverage, as given by Valuation NZ data, was used to indicate the number of storeys for each building. This ratio only gives the correct number of storeys when all floors have the same floor area.

To ensure that effort was concentrated on more significant buildings, only those with two or more storeys were considered. It was assumed that buildings with a ratio of gross floor area to site coverage less than 1.6 were essentially one storey buildings. For buildings with four or more storeys the number of storeys was generally confirmed from other sources. Where the number of storeys was not known from other sources, ratios of gross floor area to site coverage of 3.5, 6.5 and 9.5 were assumed as values at which the number of storeys changed from 3 to 4, 6 to 7 and 9 to 10 respectively.

Buildings with four or more storeys were placed in subgroups of similar age, type of use and number of storeys as indicated in Appendix A.1. The structural type was identified for a sample of approximately 1/3 of these buildings. The proportion varied from 1/3 for individual subgroups depending on the number of buildings in the subgroup. Structural type results for the sample buildings in each subgroup were weighted to reflect the floor area of the sample buildings as a proportion of the total floor area in the subgroup.

As age, occupation (use) and number of storeys are likely to have the most influence on structural type, this method of sampling helped to ensure that all subgroups were adequately sampled.

It also allowed all buildings in a limited number of subgroups with only one or two buildings to be surveyed for structural type without distorting the overall statistical results.

For post 1960's buildings with less than six buildings in any age/use subgroup at least 80% of the floor area was included in the sample.

Structural type information was difficult and time consuming to obtain. Most was obtained from Wellington City Council permit files. This source was supplemented by WORKS Design Features Reports, and other structural data held by WORKS, street survey data and an interview with the Wellington City Council's Director of Buildings and Structural Branch (Mr K Mullholland).

1.3 GENERAL COMMENTS ON 1935-75 REINFORCED CONCRETE BUILDING DATA BASE

During the building survey it was found that a surprising number of what had been thought to be reinforced concrete framed buildings had concrete encased steel frames. It would appear that prior to the late 1960's this was a very common form of construction in Wellington. Where possible these steel framed buildings were culled from the data base but some, especially in the 2-3 storey category, will still be included in the data base.

Gross floor area was used as a measure of the relative significance of the buildings in the analysis of the data instead of the dollar value of "improvements" as given by Valuation NZ. The most recent valuations for Wellington buildings were made during a recent speculative property boom. This resulted in many buildings having very low or no value placed on improvements. Even if current values were available it is likely that the market conditions would be quite different after a major earthquake and change the value of improvements yet again.

1.4 QUALITY OF SAMPLE

There were six buildings for which construction completion dates could not be found. These include the St Mary's and Sacred Heart Schools complex which contain some two to four storey buildings in the 1935-75 age group.

The remaining five buildings are two and three storey buildings, some of which were probably built between 1935 and 1975.

There are also two other groups of buildings, one group built in the 1930's the other built in the 1970's for which accurate construction completion dates could not be established. These were mainly two and three storey buildings. Given the influence of the economic depression in the early 1930s and the building boom of the early 1970s, over half these two groups of buildings can be expected to lie within the target group of 1935 to 1975 buildings.

For buildings over four storeys there were only four buildings of 4-6 storeys built in the 1970s that had unidentified ages of construction. These represented 0.5% of the total floor area of buildings with four or more storeys.

As stated earlier a significant number of large buildings known to have steel frames were deleted from the data base before sampling to determine structural type was undertaken. Closer examination of the sample taken to determine structural type revealed two more steel buildings (approximately 3% of sample). These two buildings contained 1.27% of the total floor area of the sample.

Therefore for buildings over four storeys the total floor area determined is likely to be about 1% higher than it should be due to these various factors.

For two and three storey buildings the likely error is larger and more uncertain. In this group of buildings, those identified as being built at some time in the 1970's and 1930's, including some which may be post 1975 or pre 1935, make up 30% of the total gross floor area. Inclusion of these buildings but exclusion of the buildings with totally unknown completion dates suggests an over-estimate of perhaps 15% of the total floor area if an allowance is also made for some blockwork and steel frame buildings. However this possible over-estimate of floor area corresponds to only 1.6% of the total floor area of the 1935-75 reinforced concrete buildings surveyed.

1.5 DISCUSSION OF SURVEY RESULTS

1.5.1 Significance of Survey Area

Figure 1.2 shows a comparison between buildings in the entire Wellington City area (excluding Tawa), the buildings in the survey area and the reinforced concrete buildings in the survey area that were identified as being completed between 1935 and 1975.

Two comparisons are made, one on the basis of number of buildings, Figure 1.2(b), the other on the basis of gross floor area, Figure 1.2(a).

As small residential units make up the vast majority of buildings in the city the total number of 1935-75 reinforced concrete buildings is not large compared with the total number of buildings in the city. Less than 1%. However, in terms of gross floor area, the 1935-75 reinforced concrete buildings represent nearly 10% of the total floor area.

This comparison still probably under-estimates the relative importance of the 1935-75 buildings as the replacement cost/m² of commercial buildings, which make up over 2/3's the 1935-75 buildings is two to three times the cost/m² of small residential units which make up over half the total floor area for Wellington city as a whole.

As expected, a comparison (Figure 1.2(a)) between the floor areas in the survey area and the totals for Wellington city indicates that the survey area is dominated by commercial buildings.

In fact, the survey area includes almost the entire commercial floor area, as indicated in Figure 1.3, and includes a significant proportion of the smaller quantities of "Industrial" and "other" types of floor area. However it only includes a small proportion of the total residential floor area as would be expected. Nevertheless the survey area does include a significant proportion of the multi-storey - multi-unit residential buildings in the city although there are a number of significant multi-storey residential buildings in the Newtown, Berhampore and Brooklyn areas that are excluded.

In the "other" building category the most significant concentrations of buildings omitted from the survey is Victoria University and various school buildings.

1.5.2 1935-75 Buildings

Two and three storey buildings make up 52% of the total number of 1935-75 reinforced concrete buildings in the survey area as shown in Figure 1.4(b) but make up only 22% of the total floor area as shown in Figure 1.4(a).

The floor area of each group of buildings, as shown in Figure 1.4(a), is shown again in Figure 1.5, subdivided according to building use. Commercial use dominates but there are significant floor areas being used for residential, "other" and, for buildings with less than six storeys, industrial uses.

As two to three storey buildings make up only 22% of the total floor area and include a large number of buildings for which data is difficult to obtain, these buildings were not considered further.

Figure 1.6 shows the floor area constructed in each of the decades between 1935 and 1975 for buildings with four or more storeys.

For each decade the floor area is subdivided to show the floor area in each use category.

The most striking feature of Figure 1.6 is that it shows that 86% of the floor area in the survey group with four or more storeys was constructed in the 1960's and early seventies. Also the relatively high proportion of residential floor area is perhaps unexpected.

Prior to the 1960's relatively few buildings above seven storeys were constructed as shown in Figure 1.7. However in the 1960's and early 70's the trend to high rise is obvious.

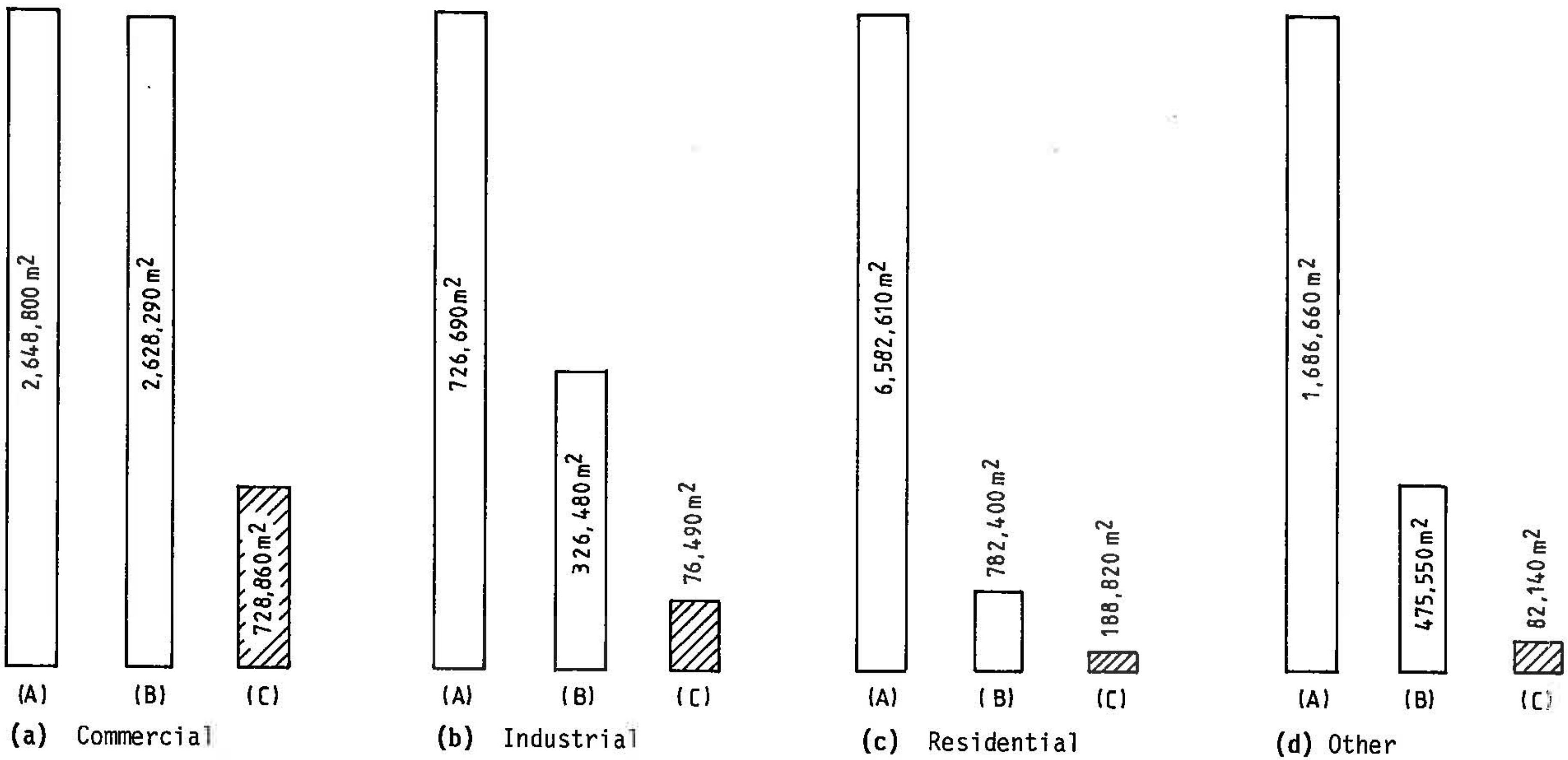


Fig. 1.3 Comparison between the gross floor area of buildings in : (A) Total Wellington City (B) Total Survey Area and (C) 1935 to 1975 R.C. Buildings in Survey Area* when buildings classified according to "USE"

(*Includes approx. 1.5% that is either steel frame construction or built in 1930's or 1970's outside range 1935-75.)

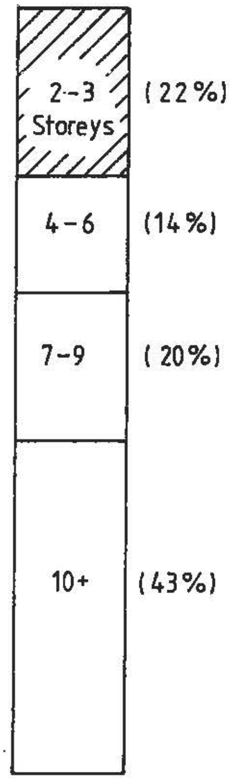


Fig.1.4a 1935-75 R.C.Buildings: Proportion of floor area in each group where buildings grouped by No. of storeys. (Brackets indicate % of total, = 1,015,980m² gross.)

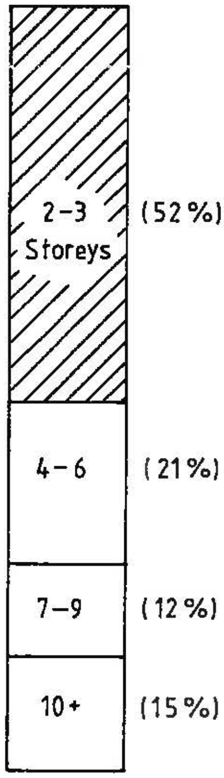


Fig.1.4b 1935-75 R.C.Buildings : Proportion of total number buildings in group when buildings grouped by No. of storeys. (Brackets indicate % of total, = 374 buildings)

("USE" : **C** = Commercial, **I** = Industrial, **R** = Residential,
O = Other
(**XX%**) indicates % of group, [X] indicates number of buildings in use category)

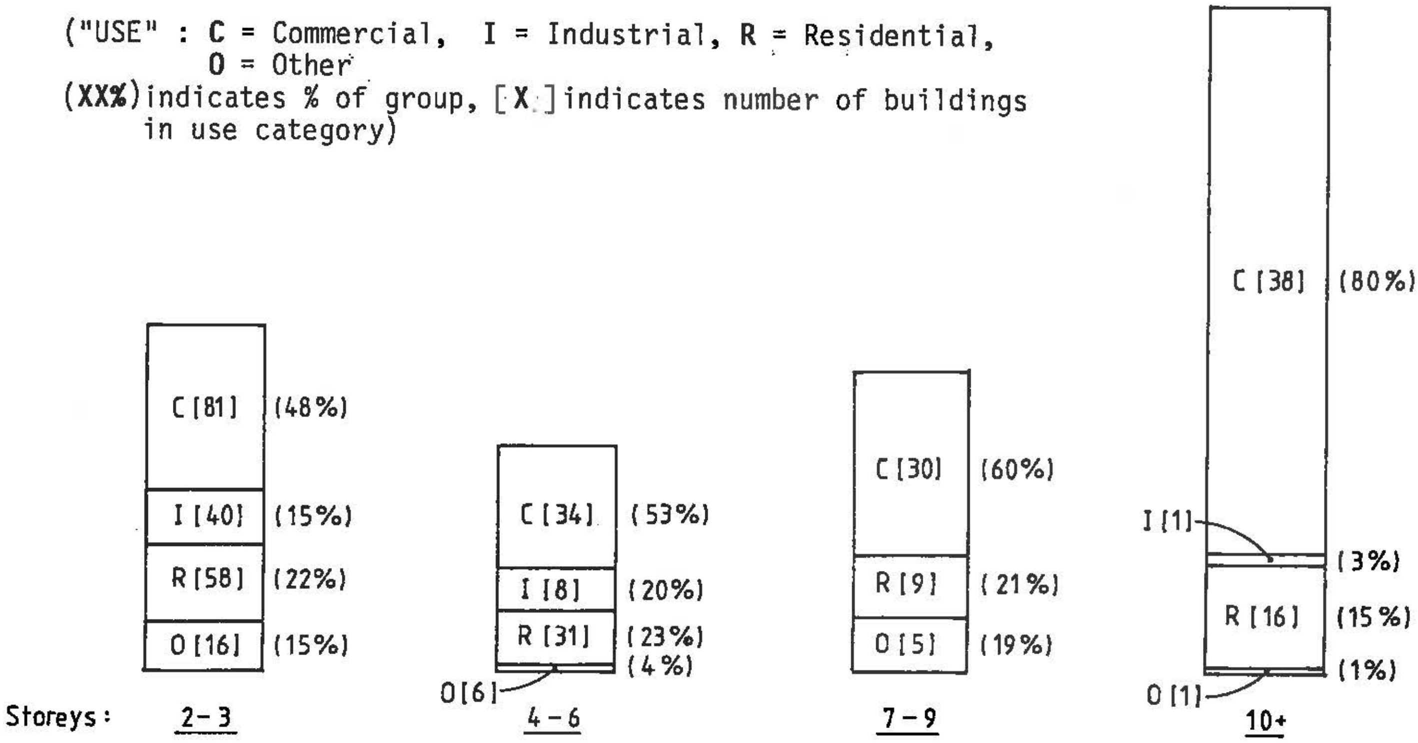


Fig.1.5: 1935-75 R.C.Buildings : Proportion of floor area in group for each "USE" category when buildings grouped by number of storeys.

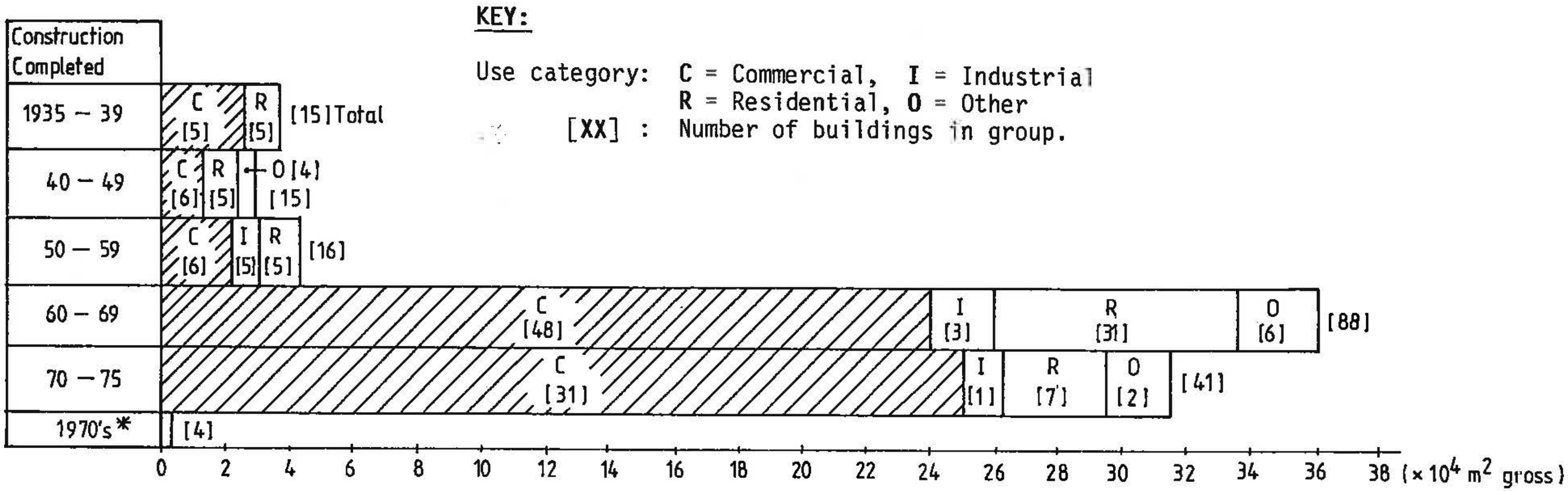


Fig.1.6 : 1935-75 R.C. Buildings four or more storys : Floor areaconstructed in each decade subdivided to show area in each "Use" category

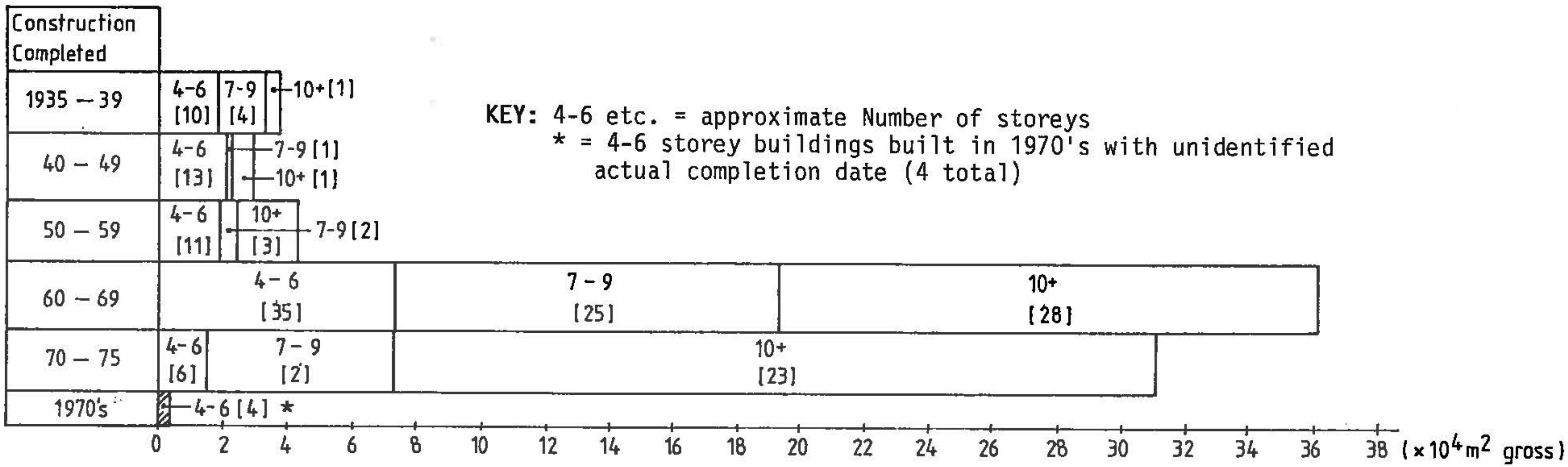


Fig.1.7 : 1935-75 R.C. Buildings with 4 or more storeys : Floor area constructed in each decade subdivided to show area in each "number of storeys" category.

1.5.3 Structural Types Used for 1935-1975 Buildings

Figure 1.8 shows the floor area constructed in each decade subdivided to indicate the relative importance of various types of structure. Only the dominant structural type in each direction is considered and for buildings with mixed systems in their two orthogonal directions, half the floor area was allocated to each of the two directions. Figure 1.8 indicates that prior to 1950 the dominant type of construction used to resist seismic loads was perforated walls.

Walls were considered to be "perforated" when they had sufficient openings and the openings were arranged in such a way that frame type action under seismic loads would be at least as important as wall type action.

Frame construction can be seen to have become important in the 1950's but not to have become dominant until the early 1970's.

The survey results shown in Figure 1.8 for the 1960's and early 70's are shown reanalysed in Figure 1.9. Here the structural type of construction with frame in one orthogonal direction and wall in the other (F/W), is treated as a separate category and the results are further subdivided according to number of building storeys.

The results in this figure need to be treated with caution as the number of sample buildings selected to determine structural type for some of the subgroups was small especially where the number of buildings in the subgroup was also small.

For example, it is unlikely that there were not any 7 to 9 storey buildings built in the 1960's with frames in one direction and walls in the other as the sample indicated. Also, for the subgroup of 1970-75 buildings of more than 10 storeys, one very large frame building made up nearly half the sample floor area and the sample for this subgroup did not include any Government Centre buildings which are known to be predominately wall type structures. The three Government Centre buildings in this subgroup have a total of 46,500 m² of floor area which is more than the 30,000 m² of wall type buildings indicated for this subgroup by the sampling technique used. Therefore, allowing for the difference between the Government Centre and the remainder of Wellington city, the proportion of wall type buildings in this subgroup should, perhaps, be approximately doubled as suggested by the dotted line in Figure 1.9. However, Figure 1.9 still indicates fairly conclusively that wall and frame/wall combinations were the dominant structural form in the 1960's and that frame buildings only became the dominant type in the early 70's and then only for buildings with 10 or more storeys.

Key:

W = Wall, F = Frame, PW = perforated wall - dominant structure type in each direction considered only.

SF* = steel frame buildings not known and therefore eliminated prior to taking structural sample.

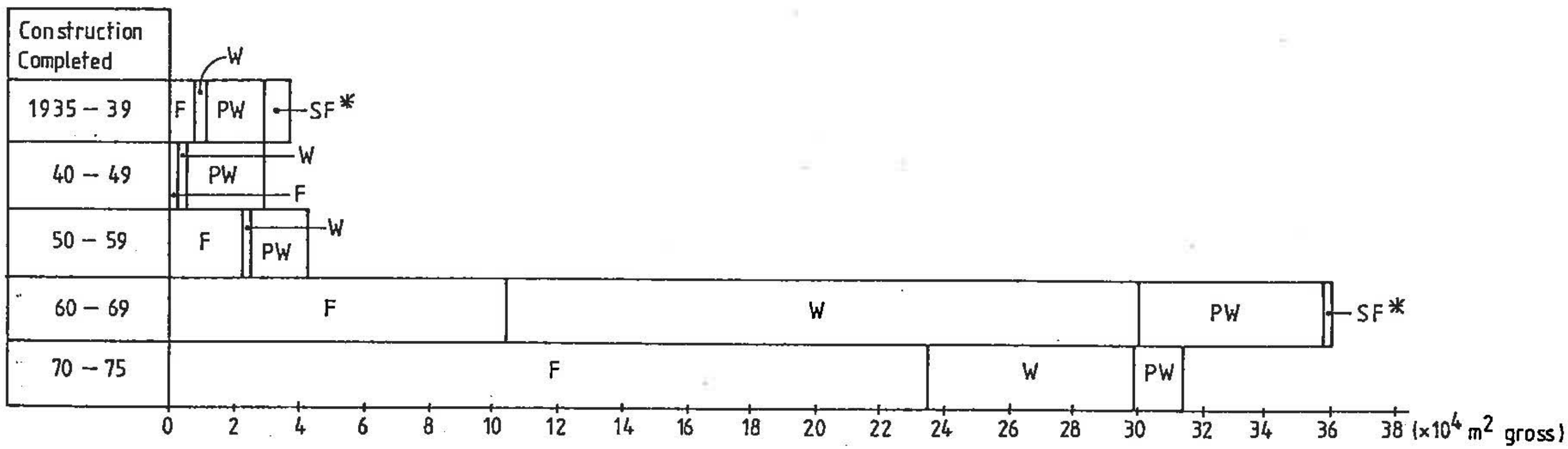


Fig. 1.8 : 1935-75 R.C.Buildings with 4 or more storeys : Floor area constructed in each decade subdivided to show floor area in each "structural type" category.

Key to Structural Type :

F = Frame, **W** = Wall, **PW** = Perforated Wall.
FW = Frame in one direction wall in other - only dominant structural type in each direction considered.

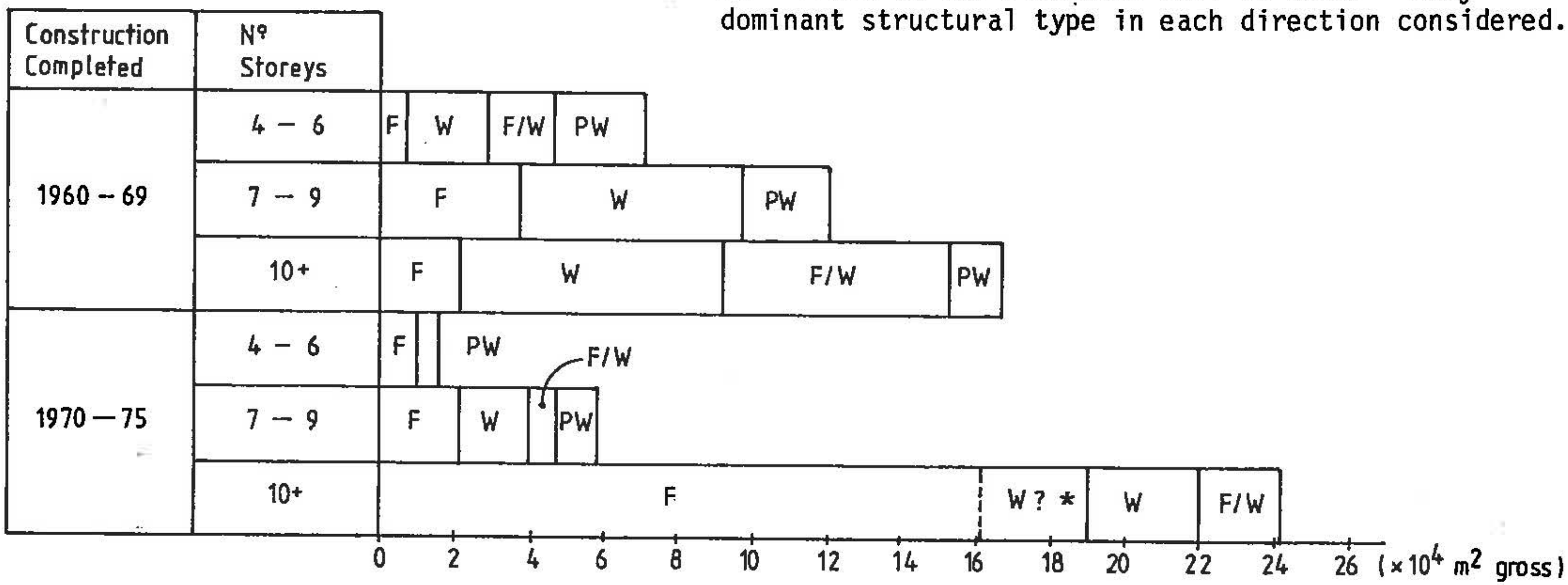


Fig. 1.9 : 1935-75 R.C. Buildings more than 4 storeys: Floor area of buildings completed in 1960's and early 70's showing relationship between number of storeys and structural type.

* see explanation in section 1.5.3

1.6 SUMMARY AND CONCLUSIONS OF SURVEY

Although the 1935-75 reinforced concrete buildings in the survey area with two or more storeys only make up a small proportion of the total number of buildings in Wellington city (less than 1%) they do constitute a significant proportion of total gross floor area (approximately 10%).

This comparison, on the basis of floor area only, under-estimates the relative importance of the 1935-75 buildings as the survey area includes almost the entire commercial floor area which has a replacement value two to three times that of residential floor area.

Most of the 1935-75 floor area is concentrated in buildings with four or more storeys (78%) and 86% of this floor area was constructed during the 1960's and early 70's. Hence the survey results indicate that when evaluating the potential seismic risk associated with the 1935-75 building stock in Wellington, effort should be concentrated on buildings constructed in the 1960's and early 70s. This conclusion would also apply to New Zealand as a whole if the age distribution of buildings in the remainder of New Zealand is similar to that in Wellington. Further research is required to establish whether or not this is the case.

For buildings built in the 1960's, wall and frame/wall combinations are the most common type. However the next section of this report establishes that the frame type buildings constitute the greatest seismic risk. For buildings built in the early 70's, frame and frame/wall buildings with more than seven storey constitute most of the floor area potentially at risk.

SECTION 2

POTENTIAL SEISMIC PERFORMANCE OF NEW ZEALAND BUILDINGS BUILT BETWEEN 1935 AND 1975

2.1 INTRODUCTION

This section of the investigation of New Zealand's 1935 to 1975 building stock has three inter-related parts. Initially the performance of reinforced concrete buildings in previous earthquakes overseas is reviewed. Emphasis is placed on buildings of similar vintage to those targeted for this study as similar structural details are more likely to have been used in their construction.

In the second part of this section, a range of typical Wellington buildings, built between 1935 and 1975, is examined to identify common potential structural deficiencies.

A quantitative assessment is then made of the risk of damage or collapse associated with the potential deficiencies identified in the typical Wellington buildings. The assessment is based on the performance of similar buildings in previous earthquakes.

It is concluded that many of the structural deficiencies will not necessarily lead to collapse. However, they may result in a building's lateral load resisting system having the characteristics that are associated with strength and/or stiffness degradation.

In the final part of this section of the investigation, existing research relating to strength and stiffness degradation is reviewed and a tentative procedure for evaluating buildings with strength and stiffness degrading structural systems is proposed.

2.2 PERFORMANCE OF RC BUILDING IN PREVIOUS EARTHQUAKES

2.2.1 General Features of Review

Previous post-earthquake evaluations of RC building performance were reviewed. Examples and features of damage caused to frames and walls in RC buildings are given in Tables 2.1 and 2.2 respectively. Other deficiencies that have resulted in damage to both wall and frame buildings are outlined in Table 2.3.

Only structural causes of damage are considered. For example, damage caused by ground or foundation failures is not addressed.

Where an example given in the tables is illustrated by a photograph, a figure reference is given in the second last column of the tables. It is assumed that the photographs and tables will be read together.

Where the buildings used as examples in the tables collapsed or are known to have been demolished this is also noted.

2.2.2 Behaviour of RC Frames in Previous Earthquakes

As indicated in Table 2.1 it is not difficult to find examples of frame buildings that have collapsed in previous earthquakes. Of the 210 buildings that collapsed in the Mexico City earthquake in 1985, 82% were RC frame buildings [51]. However a little over half of these had waffle slabs providing the "beams" of the frames. It was noted [51] that the "vast majority of failures in reinforced concrete frame buildings were due to column failures in eccentric compression, diagonal tension or a combination of both". When examining columns located in collapsed or badly damaged frame buildings it is usually very difficult to isolate the relative contribution that flexure, shear and axial load have made to the failures. Dynamic testing [27] of columns with only low volumetric ratios of spiral confining steel under axial load has shown that failure was usually on a single diagonal shear plane. It is, therefore, likely that many of the failures attributed to shear in the past were in fact primarily compression failures. It is also likely that when a column "fails" in shear it usually requires a significant axial load to convert the failure into a collapse. Only two examples [35], were found where one or two storey frame buildings had collapsed. However this could be because damage to small buildings is not reported as often as damage to more significant buildings, rather than a reflection of the influence of column axial load levels.

In most of the examples of brittle column failure listed in Table 2.1 the columns only had light tie reinforcement. These were typically R6 to R10 ties spaced at .7 to 1.0 h (where h = minimum column dimension). In some cases "failure" did not lead to collapse even where a complete air gap developed in the column. This illustrates the important roll that axial load and alternative load paths (redundancy) play in the collapse mechanism of frames.

It can be seen from Table 2.1 that the high shear/moment ratios that occur in "short" columns has often contributed to brittle column behaviour. The short columns are often part of the basic structure or they may be generated unintentionally by non structural elements such as infill panels located below window cill level.

There are cases where failure of external beam/column joints has lead to partial collapse. In all the cases identified, the axial load was significant. However no case was found where failure of an internal beam/column joint could be identified as the principal cause of collapse. It is known from laboratory tests that joint failure of lightly or unreinforced beam-column joints results in degrading strength and stiffness of frame buildings. This is likely to increase the lateral displacement of frames during earthquakes and therefore increase the amount of non structural damage. Collapse of a frame solely due to joint failure with the column remaining intact would be expected to result in large lateral displacements of the floor slabs in the collapsed structure. This type of collapse is relatively rare. However, it is possible that joint failures switch to a brittle column failure mechanism at some lateral displacement level when the column is weak in shear.

Flexible frames with slender members tend to be associated with various forms of unreinforced masonry (URM) infill panels. Damage is often concentrated where infill panels are missing or begin to fail first. This concentration of ductility demand often leads to collapse.

No cases could be found where beam shear failures were identified as the principal cause of a buildings collapse, or even where a floor had collapsed locally due to a beam shear failure. However it is known from laboratory testing that beam shear failures will contribute to degrading strength and stiffness of a buildings frame. This will increase non structural damage and may contribute to a building's final collapse.

TABLE 2.1 - EXAMPLES OF RC FRAME COLLAPSE AND DAMAGE IN PREVIOUS EARTHQUAKES

EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE — Building Collapsed = C, Building Partly Collapsed = PC, Building Demolished = D		PHOTO REF (FIGURE NO.)	SOURCE REF.
	<u>BRITTLE COLUMN FAILURES</u>			
Mexico 1985	• The photo indicates a circular column which has sheared and/or crushed under axial load - Tie's are close spaced (≈ d/2 - d/3) but are very light (≈ 6 - 10 mm ϕ).	C	2.1(a)	32
	• As above but column is large and square.	PC	2.1(b)	32
Chile 1960	• Short column effect generated by "non structural" elements. Note small column dimension and widely spaces ties evident. Low axial load probably avoided local collapse.	-	2.1(c)	45
	• A "circular" one storey "market building" with RC frame and brick infills. Columns had only light ties (≈ 6ϕ @ 180) and weak concrete (8.0 MPa in places). Columns suffered diagonal tension failures and sliding occurred on column construction joints.	-		45
	• 1942 Brewery Building with RC frames. Concrete block infill panels generated short column effect. Columns failed in shear but did not collapse. Ties light (≈R6ϕ @ 350 cnrs).			45
	(Note: Chilean code only required one tie at 12 main bar diameter spacing or least column dimension - "but instances of column failure were few". Concrete strengths were typically low.)			41
Romania 1977	• Bucharest Computing Centre. Columns in 1st floor of three storey flat slab building failed in shear. Reinforcement in ground floor columns was high (p = .03, Fy = 510 MPa) but ½ this was terminated at 3/4 height of storey.	C	2.1(d)	49
	• 32 pre World War II buildings between seven and 14 storeys collapsed. Of these 15 inspected and found to have at least four of following defects: (1) f'c low (i.e. down to 12 MPa) (2) low column reinforcement (p < .005 plain mill steel) (3) wide spacing of ties (s ≥ 250 mm) and ties anchored with short 90° hooks (4) Columns designed for axial stress only (5) no shear reinforcement in beam/column joints (6) Short laps in column bars (< 20 bar diameter).	C		49

TABLE 2.1 cont'd

EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
San Fernando 1971	• Holy Cross Hospital. Seven storey shear wall and frame building with three storey wings, that was built in 1963. Walls designed to carry lateral loads but columns unable to "follow" inelastic wall deformations. Column had light (R6) widely spaced ties (see photo). Also general damage to walls and diaphragms especially in lower four storeys.	(prob D)	2.1(e)	35
	• Olive View Hospital. Large stocky columns had large flexural bars but only light widely spaced ties. Adjacent columns with closely spaced spiral ties carried axial load and prevented total collapse of "soft" 1st storey.	PC D	2.1(f) and (g)	35
Miyagiken-Oki, Japan 1978	• Several examples of 2-4 storey buildings with squat columns - some collapses and partial collapses of ground floors (soft storeys).	PC, C		37
	• Obisan Building. Three storey building with squat columns - appears to have had close spaced perimeter ties (see photo).	C	2.1(h)	38
Lima, 1974	• Police School. Three storey RC building with shallow ground floor. Columns failed in shear due to light widely spaced ties.	PC	2.1(i)	39
Anchorage, 1964	• Mt McKinley Building. A 14 storey RC building with coupled shear walls. Walls replaced by columns at ground floor which failed in shear and/or compression - ties light (R6 @ ≈h/3).	-	2.1(j), (k)	42
Caracas, 1967	• Charaima Building. A 10 storey building that suffered partial collapse at 7th floor level due to column failure. Failure occurred where column main bars reduced (from 14 x 1" bars to 12 x ¾" bars). Peripheral ties only but were not light (R10 @ 250 cnrs).	PC D	2.1(l)	43
	• Macuto Sheraton. 10 storey RL building with pairs of large columns supporting walls at 3rd floor level. Columns failed in shear/ compression in spite of moderate quantity of spiral ties.	-	2.1(m)	43
	• Los Paloes Grandes Area. Four 10 to 12 storey RC buildings collapsed with little lateral movement of first few floors indicating brittle shear/compression column failures were probably responsible.			43

TABLE 2.1 cont'd

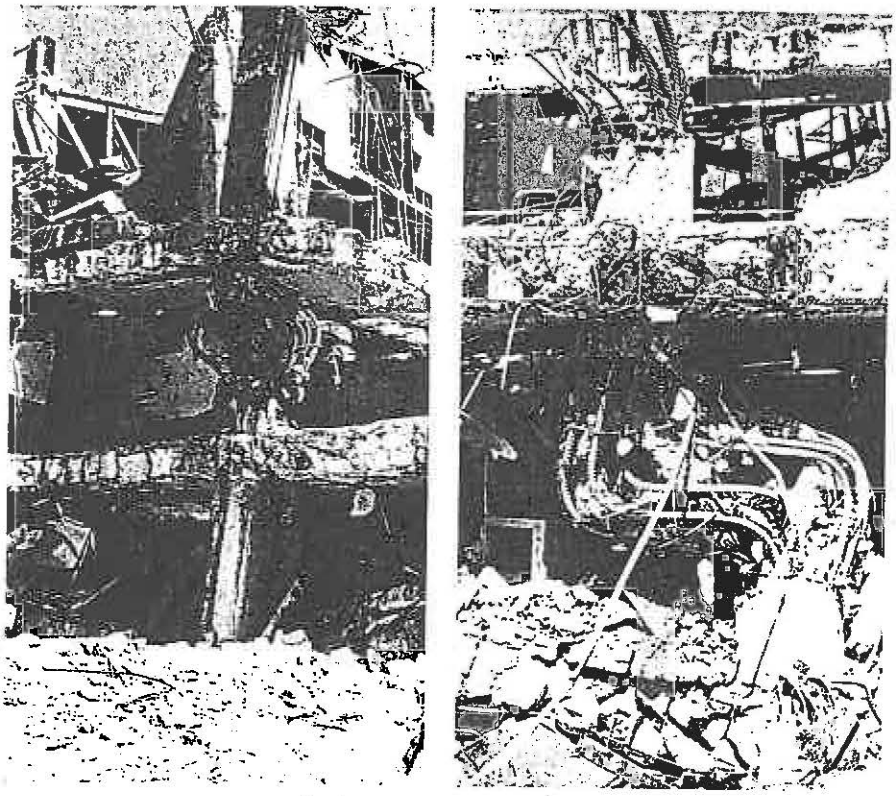
EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
	• Caromay Building. 18 storey RC frame building with brick infill panels above 1st floor level. Brick infilled frame acted as shear wall and generated high column compression which crushed the 1st storey columns at midheight. Ties were at close centres (d/4 to d/3) and had cross ties but were light and only had 90° hooks which opened after spalling.	-	2.1(n)	43
San Salvador 1986	• Two storey laboratory building with light roof. Frame at ground floor was infilled with "non structural" spandrel panels to window cill height creating a short column effect. Note heavier confinement in hinge zone shown in photo. Damage surprising given low axial load and moderate quantity of ties at ≈ h/4.	-	2.1(o)	46
	• Three four to seven storey RC frame buildings. Two of the buildings had unreinforced walls supplementing the frames. All lost one or two storeys due to soft storeys developing.	C		46
	• Benjamin Bloom Childrens Hospital. Collapse of three storey RC frame building due to "short column" effect caused by "non structural" masonry infills - ties light relative to size and number of flexural bars. Many other examples of short column effect causing failure given even where moderate quantities of ties had been used.	C	2.1(p)	47
Loma Prieta - California	• I280 Elevated Motorway. Where the upper and lower lanes were not aligned, the columns supporting one side of the upper lane were supported off the lower deck. The short columns so formed, failed in shear. Note short "pins ended column" that formed in the compression zone (see photo). Ties with short 90° hooks were ineffective.		2.1(r)	50
	<u>PΔ COLUMN FAILURE</u>			
Friuli Italy, 1976	• A three storey frame building with brick infill panels in the upper two storeys. Building developed significant drift in the main event (see photo) and collapsed in an aftershock.	C	2.1(q)	36

TABLE 2.1 cont'd

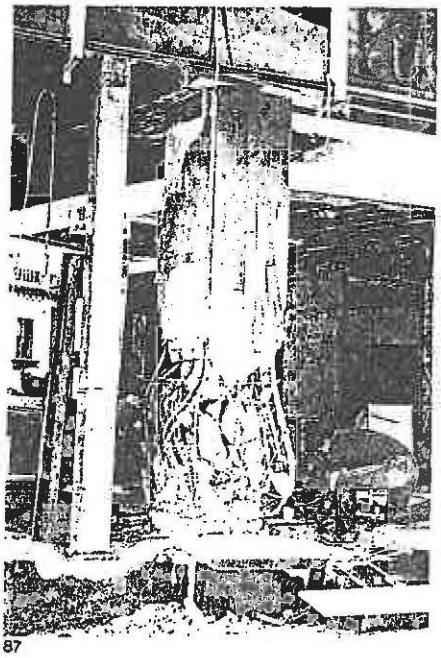
EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
	BEAM/COLUMN JOINT FAILURES			
Armenia 1988	• A large number of frame/precast panel buildings collapsed. Joint failures could be identified in debris. Extent that joint failure contributed to collapse not clear (could have resulted from collapse process or demolition and rescue effort).	C, D		34
Thessaloniki Greece, 1978	• Ippodromion square apartment building. This eight storey RC frame building collapsed completely. Failed external beam/column joint can be seen in photo. It is unclear how much joint failures contributed to collapse of this relatively slender member frame.	C	2.1(s)	40
Mexico, 1985	• There were numerous examples of beam/column joint failures in flexible frames. It was difficult to determine the extent that joint failures contributed to collapses. High axial load combined with joint failure did cause at least partial collapse of some external and corner columns (see photo). Buildings with failed joints generally had extensive structural and non structural damage indicating flexibility.	PC	2.1(t)	32, 48
Loma Prieta, 1989	• Embarcado motorway - some external joints in this two storey elevated motorway exhibited severe shear cracking.	-	2.1(u)	50
	FLEXIBLE FRAMES			
Armenia, 1988	• 133 nine storey apartment buildings in Leninakan - All collapsed or were demolished. Difficult to relate type of precast frame/panel construction to NZ present or past practice: - column dimensions typically 400 mm square - eccentric welded column bar splices used extensively - floor diaphragms very weak and failed completely - column ties widely spaced with 90° hooks - poor quality concrete common	C (54%) D (46%)	2.1(v) 2.1(w)	34

TABLE 2.1 cont'd

EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
El Asnam 1980	• Use of slender columns and hollow tile infills resulted in heavy non structural damage. Those that remainded standing were 100% insurance losses.	many C		36
Mexico City 1985	• Many examples of slender frames that were infilled with masonry and suffered at least severe structural and non structural damage especially to masonry infills and partitions.	some C	2.1(x)	32
	<u>BEAM SHEAR FAILURES</u>			
Armenia, 1988	• Some of precast frame/panel buildings in Leninakan had extensive beam shear failures in spite of moderate size and spacing of stirrups.	D	2.1(y)	34
Dannevirke NZ, 1990	• Margrethe Plaza Building. A RC frame building with URM. Probably built prior to 1940. Exhibited an isolated beam shear failure.	-	2.1(z)	WORKS
Caracas, 1967	• Laguna Beach Building. A 14 storey RC frame building with hollow masonry infill panels. The 1st floor beams suffered shear failures just outside the normal hinge zone. Beam torsion may have contributed to the cracks.	-	2.1(z1)	43
Mexico City, 1957	• Two Storey School Building. Some spectacular beam shear failures occurred without causing the buildings to collapse. The major diagonal crack shown in Figure 2.1(z3) passes through a section where the main steel is cut off.		2.1(z2), (z3),(z4)	30
Chile, 1960	• Elevated Water Tank Support Frame. Photo shows beams with very little residual strength after failing in shear. However the tank did not collapse.		2.1(z5)	30

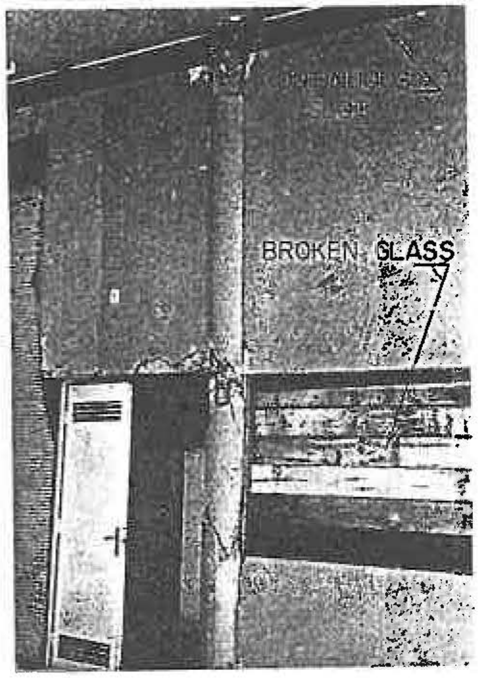


(a) Two details of the building section which collapsed to the ground: The extremely severe building oscillations crushed the concrete on the load-bearing columns, depriving them of their load-bearing capacity despite the strong steel reinforcement



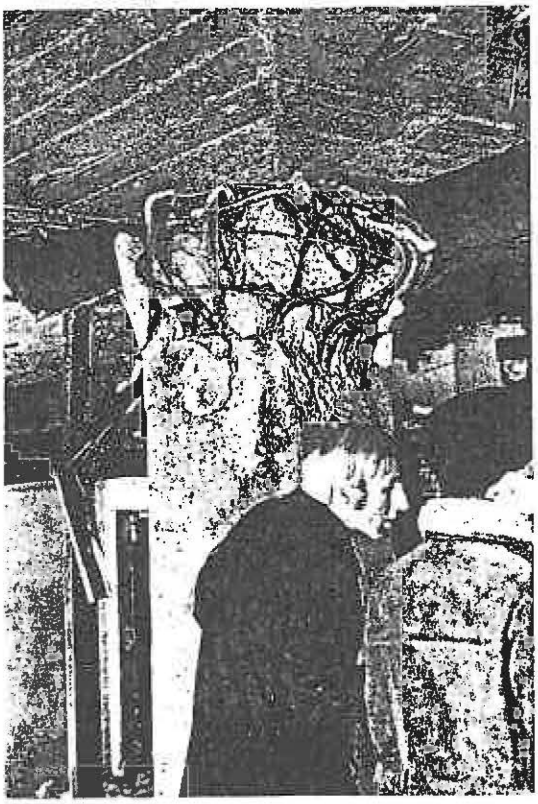
This column in the ground floor was bent by the huge forces of the building swaying to and fro. The entire building was therefore on the verge of collapsing

(b)



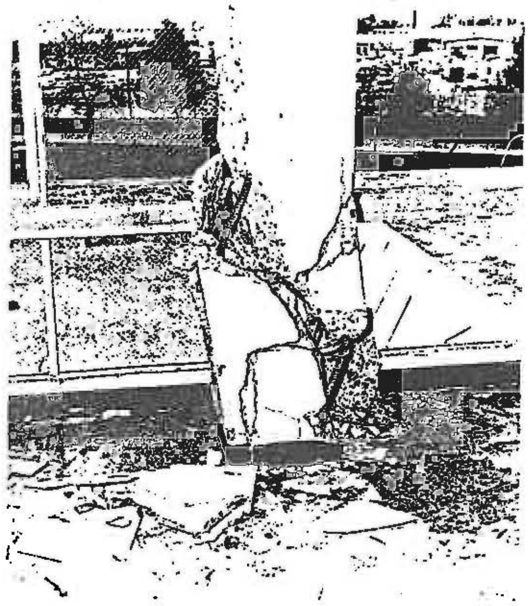
(c)

Fig. 2.1 Brittle Column Failures (see Table 2.1 for Details)



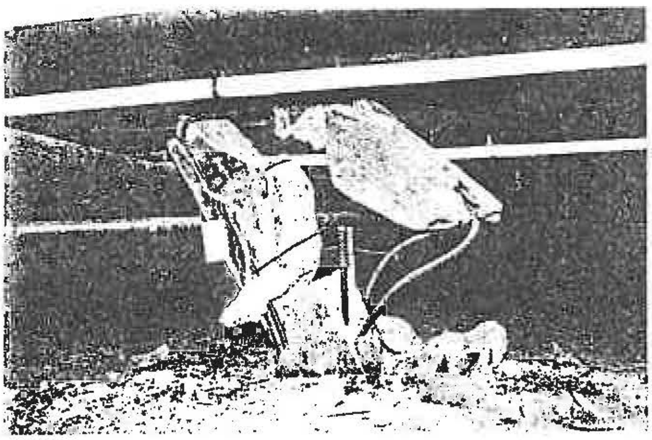
Computing center
column failure in ground story

(d)



—Holy Cross Hospital. Column failure at E-11 (third floor).

(e)



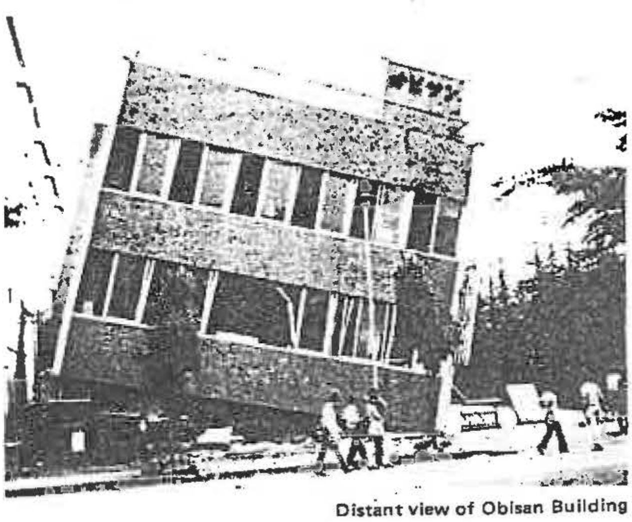
—Olive View Hospital, medical treatment and care unit. Complete collapse of tied column located in pipe space area near north wall.

(f)

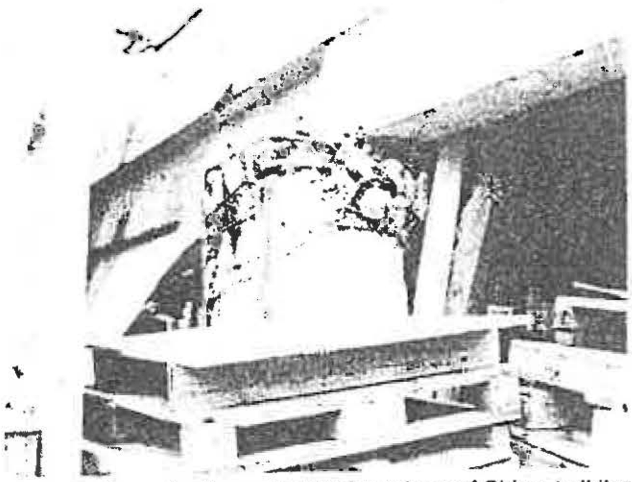
—Olive View Hospital, medical treatment and care unit. Collapse of corner column at first-floor level.

(g)

Fib. 2.1 (Cont'd) Brittle Column Failures (see Table 2.1 for Details)

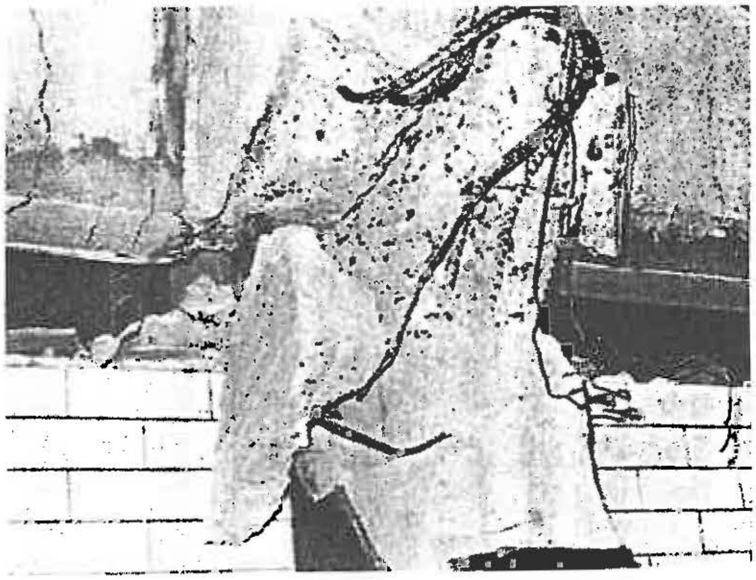


Distant view of Obisan Building



State of collapse of a left side column of Obisan building

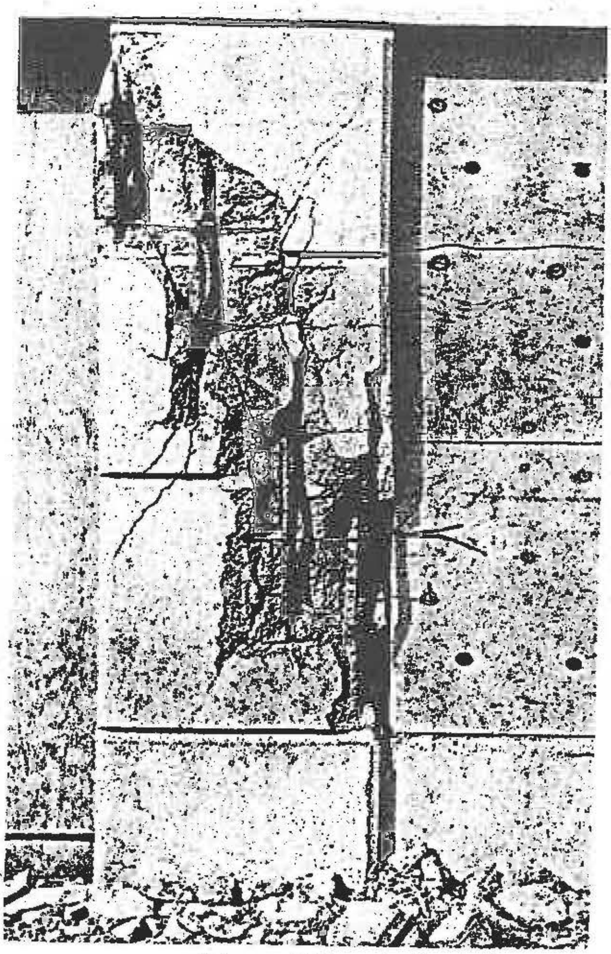
(h)



(i)

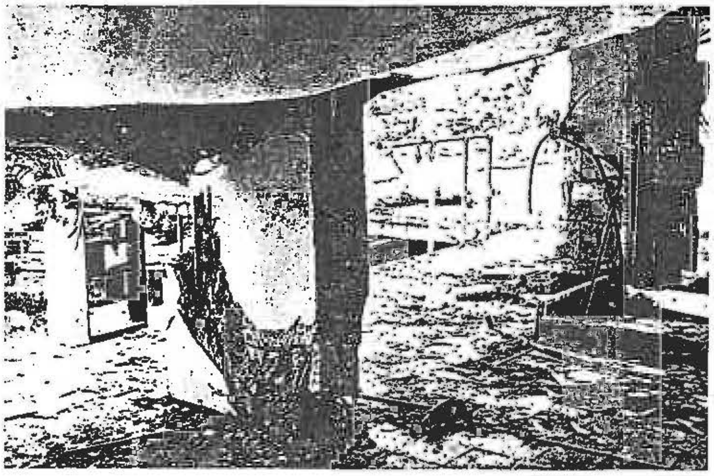
Mt. McKinley Building, Looking NNW

Fig. 2.1 (cont'd) Brittle Column Failures (see Table 2.1 for Details)



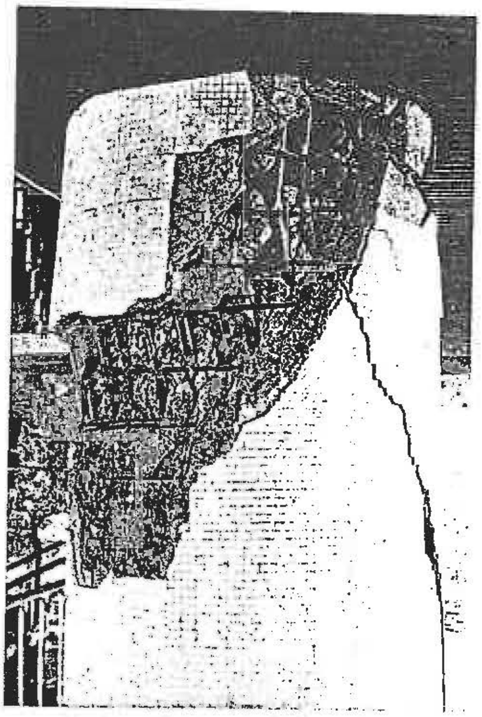
Column Failure in South Face

(k)



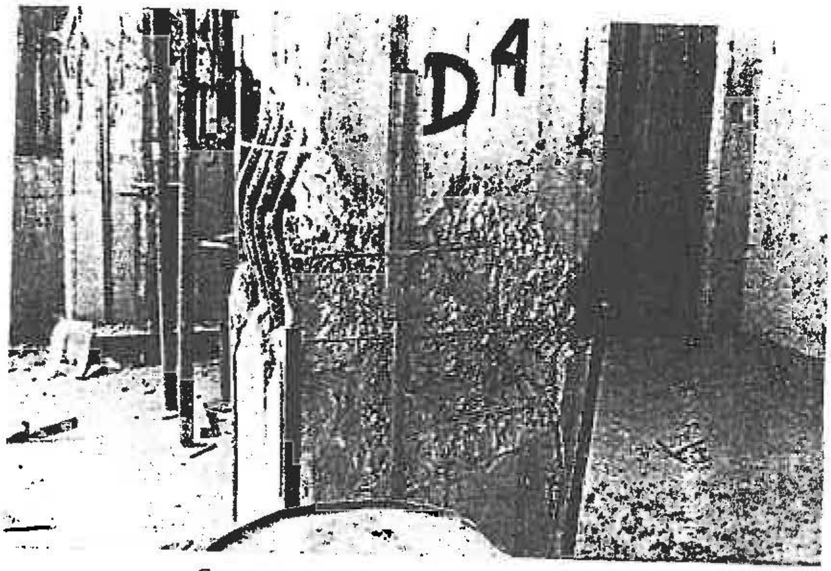
Charaima, looking south-west at the seventh storey. The crushed column is in the third row from the south and just west of the mid length of the building. The column has crushed down about 2 ft.

(1)



(m)

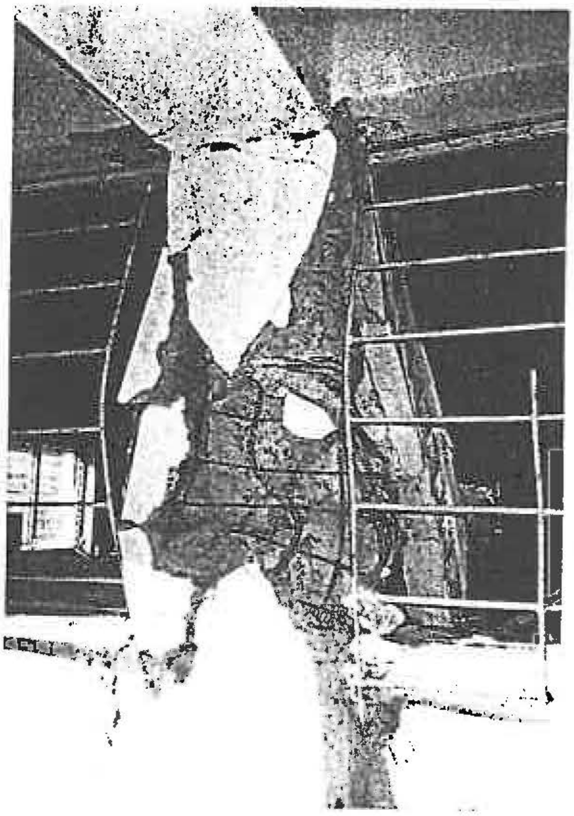
. Macuto Sheraton, looking west at the third storey. Detail of the column, second from the north and fourth from the west.



Caromay, typical crushed columns of the first storey.

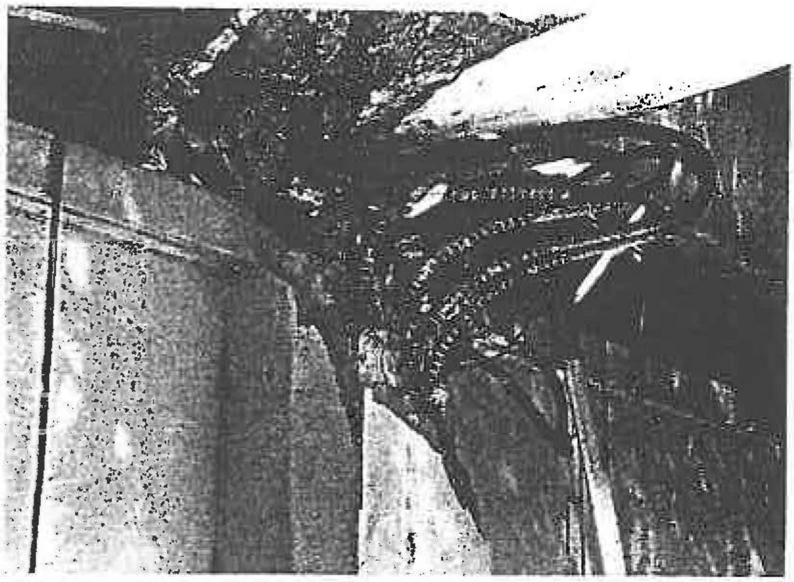
(n)

Fig. 2.1 (cont'd) Brittle Column Failures (see table 2.1 for details)



– Typical failed column along south wall of Laboratory Building at Ricaldone Tecnico Institute.

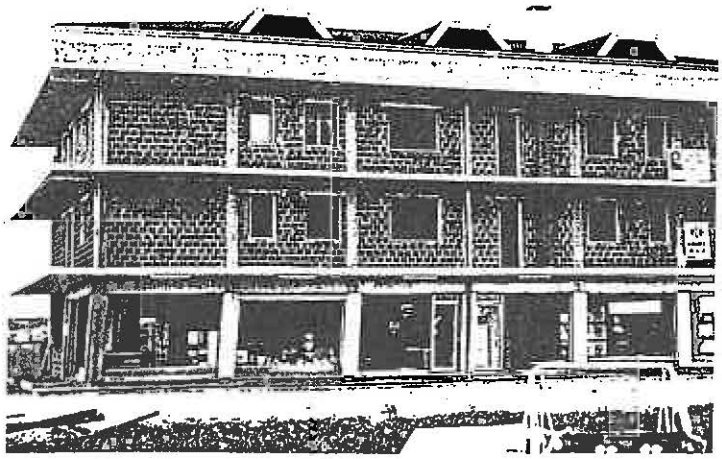
(o)



- Short-column failure at the Benjamin Bloom Children's Hospital.

(p)

Unsuitable top heavy design with little resistance of the ground floor columns.



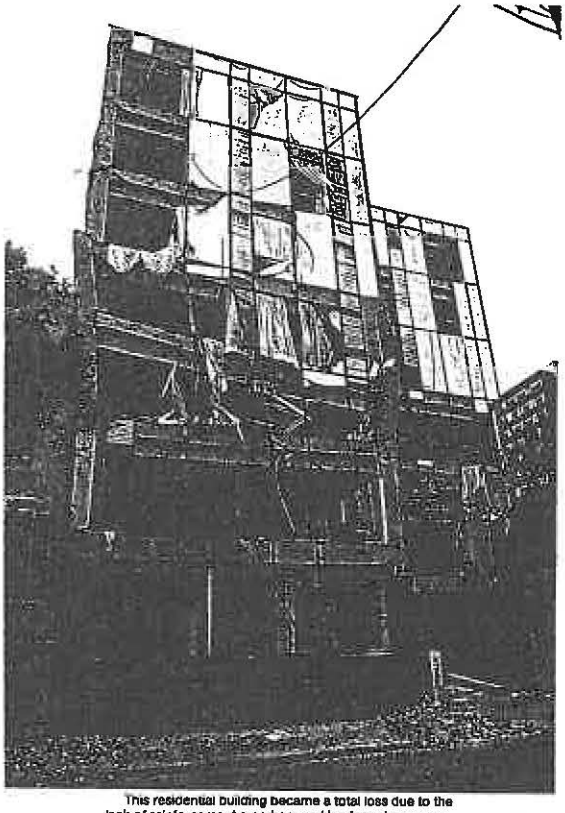
(q)

I 280 : NEAR ARMY STREET. TYPICAL SHORT COLUMN SHEAR FAILURE.

(r)

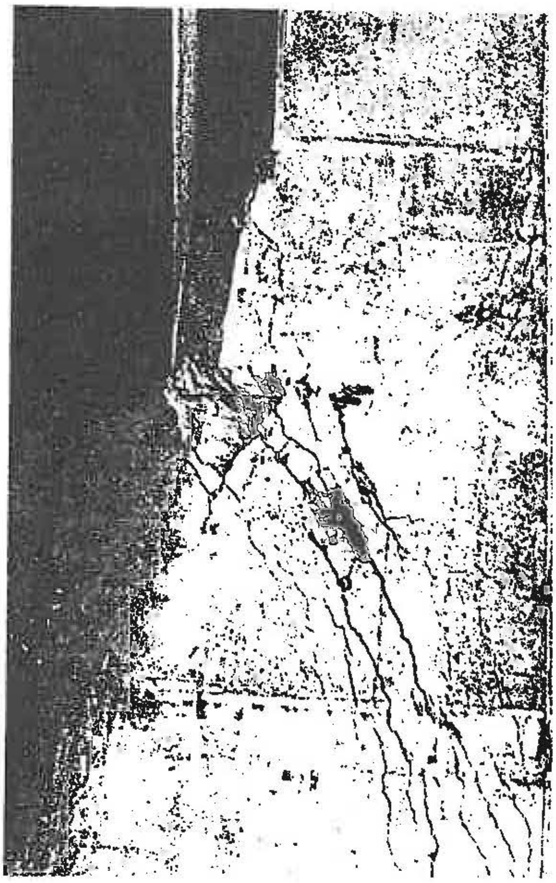
Fig. 2.1 (cont'd) Brittle Column Failures (see Table 2.1 for Details)

(s)

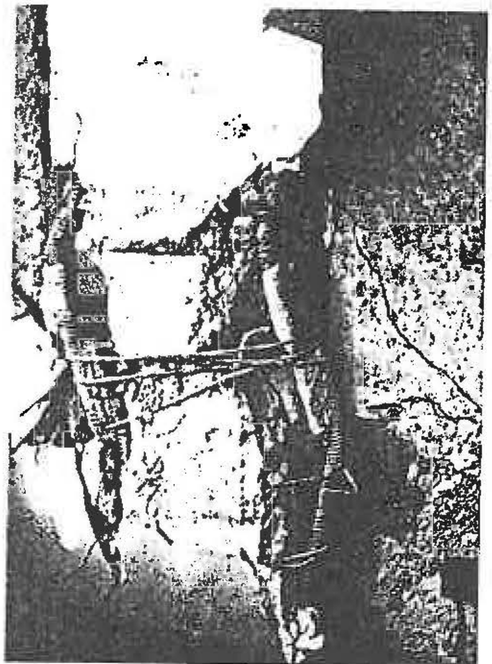


This residential building became a total loss due to the lack of reinforcement members and inadequate connections between the columns and ceiling slabs. Presumably the rigid neighbouring building (left) also contributed to the collapse of the middle floors

(t)



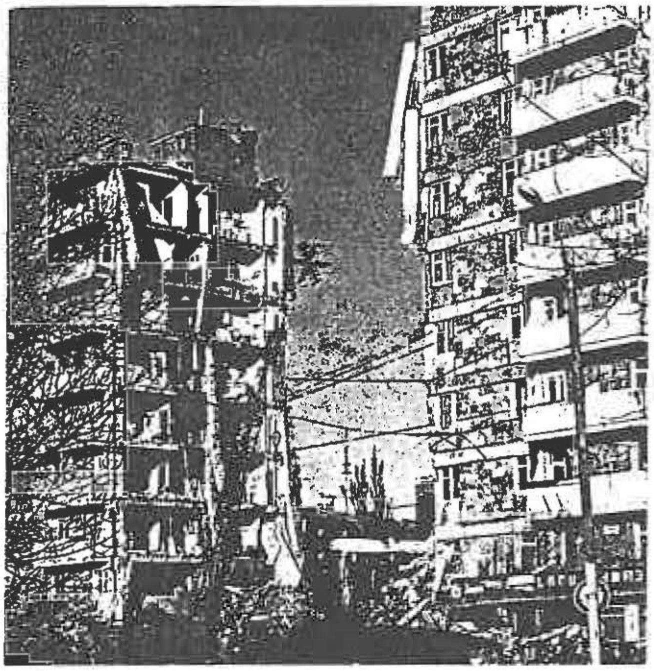
(u)



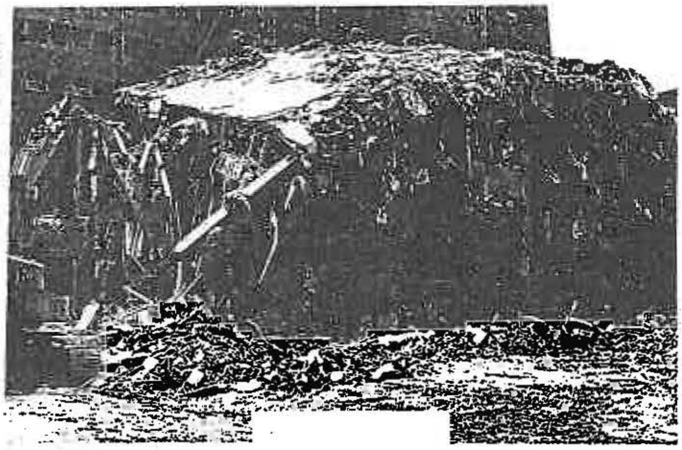
Failed column-splice
in first story of
nine-floor
precast-frame building.
Note eccentricities
in bar-splices.

(v)

Fig. 2.1 (cont'd) Beam-Column Joint Failures and a Welded Splice Failure.

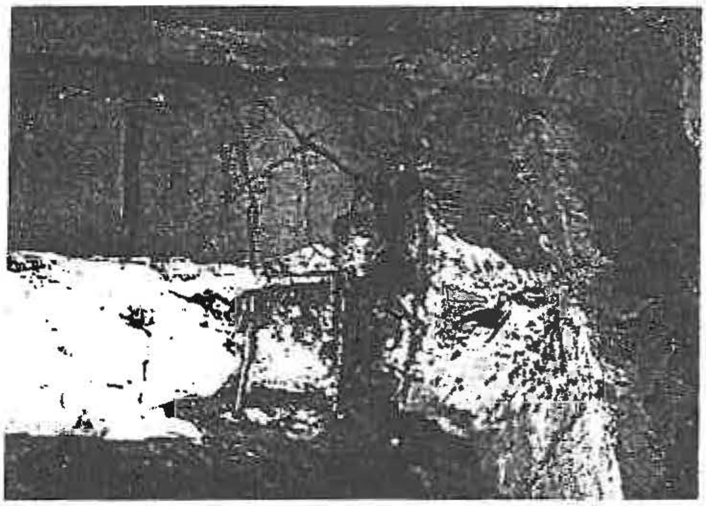


(w)

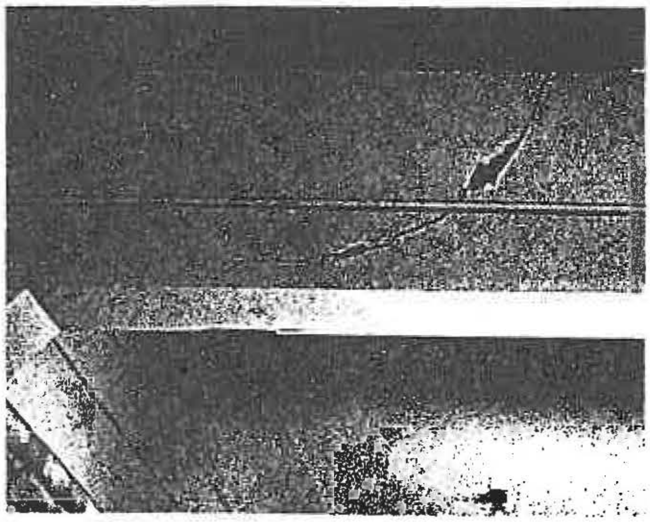


Another typical example: The top floors of this building have totally collapsed

(x)

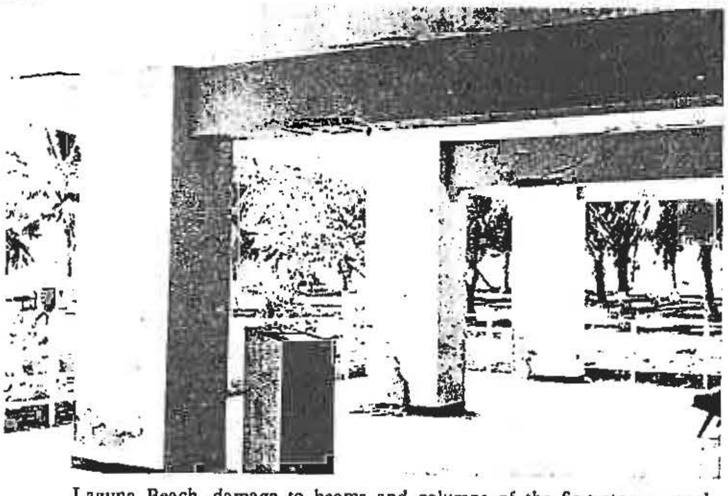


(y)



(z)

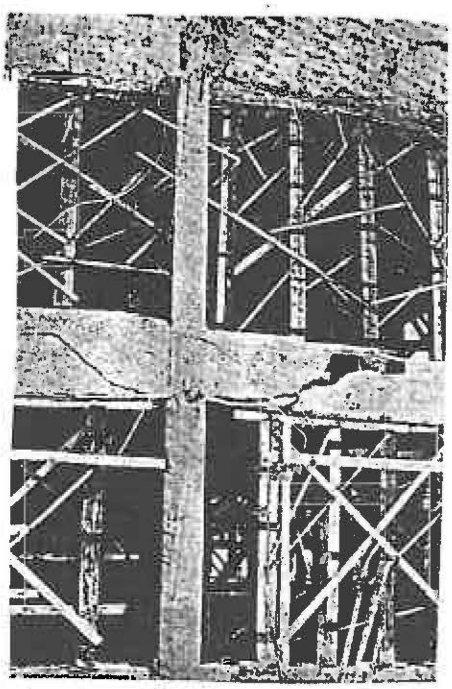
(z1)



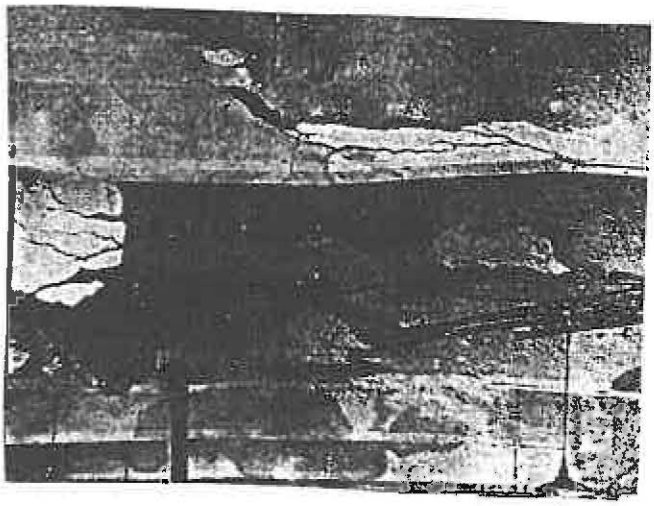
Laguna Beach, damage to beams and columns of the first storey, north-west corner.

Fig. 2.1 (cont'd) (w) and (x) Examples of Flexible Frame Collapse.
(y), (z) and (z1) Examples of Beam Shear Failures (see table 2.1 for details)

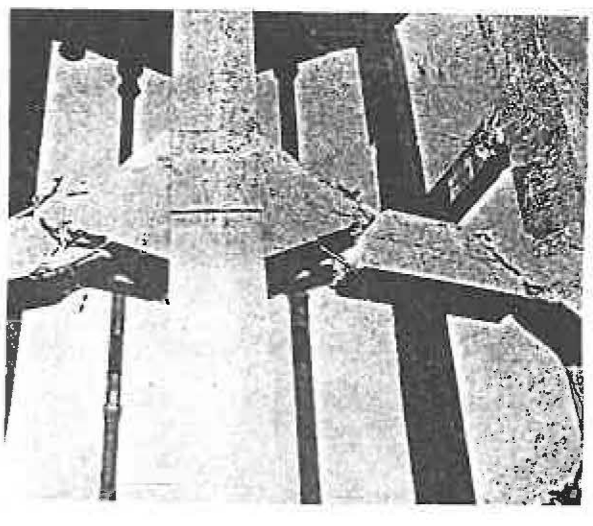
(z2)



(z3)



(z4)



(z5)

Fig. 2.1 (cont'd) Beam Shear Failures (See Table 2.1 for details)

2.2.3 Behaviour of RC Walls in Previous Earthquakes

It was noted after the Chile earthquake of 1960 [45] that "as is customary with reinforced concrete shear wall buildings [in earthquakes] extensive fracturing of shear walls did not bring about total collapse".

In earthquakes after 1960 only three cases of building collapse were identified by the review where the building was primarily dependent on walls for its lateral resistance. These are detailed in Table 2.2.

Most of the seriously damaged walls identified seem to have been very lightly reinforced compared with, for example, current New Zealand practice. Because of the light reinforcement there is a tendency for only one flexural crack (often at a construction joint) or two diagonal shear crack (type X) to form in walls and coupling beams. Damage then tends to be concentrated on the line of these cracks.

Damage to wall construction joints is often very extensive and is surprisingly common. Lack of joint preparation and extension of lightweight floors through walls appear to be common contributing factors. The use of light wall reinforcement may have also been a factor as it may have concentrated damage at the "weak link" formed by the construction joint instead of at diagonal shear cracks. Displacement capacity of construction joints may have acted as a "fuse" and prevented the development of shear forces sufficient to cause diagonal shear cracking in the remainder of the wall. When significant displacement takes place on construction joints vertical bond cracks tend to develop especially near the ends of the wall. This may initiate crushing/spalling failures due to flexure and/or axial loads.

TABLE 2.2 - EXAMPLES OF RC WALL COLLAPSE AND DAMAGE IN PREVIOUS EARTHQUAKES

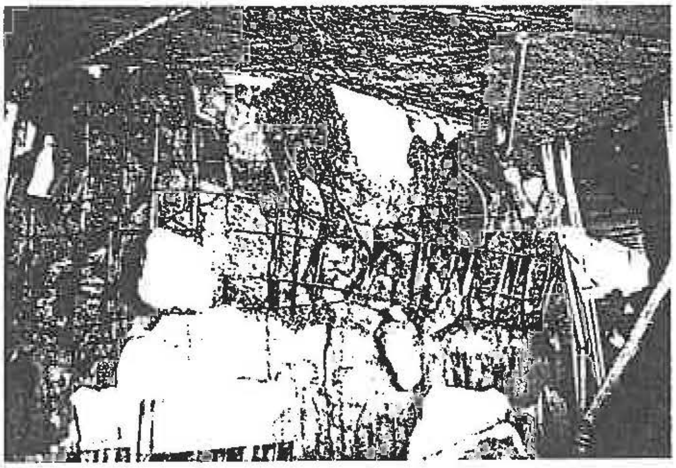
EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
	<u>WALL FAILURES</u>			
Chile, 1960	• Valdinvia Orthopedic Hospital. A six storey RC "bearing" wall building with extensive RC walls and non structural brick partitions that was nearly completed at time of earthquake. Walls suffered extensive diagonal cracking and "serious" movement on construction joints (CJ's) throughout the six storeys.	-		45
Armenia, 1988	• 16 storey lift slab building with nearly circular central shear wall core. The core was almost entirely crushed at 1st floor level due to shear, axial load and torsion with extensive cracking and non structural damage elsewhere. Horizontal ties in wall were very light (R6 @ more than 300 cnrs) and concrete quality appears to have been poor - see photo.	D	2.2(a)	34
	• A nearby 10 storey lift slab building with twin circular cores collapsed so completely that the cause could not be investigated. The circular cores were designed to resist the entire lateral loads.	C		
Anchorage,	• Mount McKinley and 1200 L Buildings. Two 14 storey buildings with coupled shear walls and perforated walls built in early 1950s. Coupled walls exhibited crushing failures (see photo 2.2(b)), extensive movement on construction joints and heavy damage to coupling beams. Walls were only lightly reinforced and did not have additional heavy edge members or reinforcement. Perforated walls had extensive shear cracking (X type) to beam and column elements. Tendency for only one significant flexural or shear crack to form suggests low ratios of reinforcement throughout members. Availability of alternative loads paths (redundancy) for gravity loads appears to have saved this building from collapse.	-	2.2(b) also 2.1(j) and (k)	42
	• Four Seasons Apartment Building. A six storey lift slab building with twin RC cores providing lateral resistance and with steel columns supporting the floors. Building was structurally complete but not occupied at time of earthquake. Because of complete nature of collapse, sequence and cause of collapse not known. However cores were noted as being "fractured in the first storey". Given detailing of other buildings in Anchorage the cores were probably only lightly reinforced.	C	2.2(c) and (d)	42

TABLE 2.2 cont'd

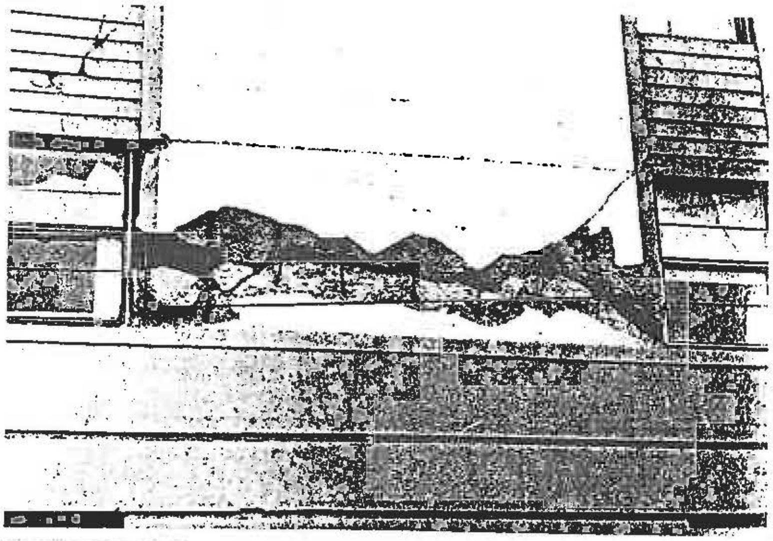
EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
	<u>WALL CONSTRUCTION JOINT DAMAGE</u>			
Chile, 1960	• Regional Hospital in Valdivia. Sliding occurred at CJ in fins that provided lateral support for a large water tank. Distortion of flexural bars and "dowel" cracks has resulted in spalling of compression zone.	-	2.2(e)	45
	• Valdivia Orthopedic Hospital. "Serious" movement or working occurred at most construction joints of this six storey RC shear wall building.			45
San Fernando 1971	• Indian Hills Medical Centre. This was a seven storey RC wall and frame building that was located in the "Epicentral" region of the EQ. Walls were lightly reinforced (D16 @ 450 cnrs BW's) and lightweight floors were extended through the walls. Sliding on the construction joint appears to have initiated crushing of the wall boundary column in the splice zone above the joint. Column ties appear to be quite close but not more than R6.	-	2.2(f)	35
	• Museum for Antique Cars. A five storey RC shear wall building with light internal frame and located in "epicentral" region. 200 mm walls were lightly reinforced and lightweight floor concrete continued through walls. Given the solid box type of structure (see photo) the movement on CJs (up to 35 mm) would not have been anticipated. Movement was sufficient to fracture bars (see photo - note that bars spliced above CJ).	-	2.2(g) and (h)	35
	• Pacoima Memorial Lutheran Hospital (Nursing Unit). A four storey RC wall and frame building located in the "epicentral" region. Extensive movement occurred on both horizontal and vertical CJs of walls.	Part D		35
Anchorage, 1964	• JC Penney Building. A five storey RC building with walls on three sides. Construction joints had slots to receive precast panel brackets and the remainder of the joints were not prepared. Sliding on CJs (partly due to torsional response) had a large enough component perpendicular to the face of some walls to cause the upper part of the wall to move over the edge of the lower part of the wall and collapse.	PC and D	2.2(i)	42

TABLE 2.2 cont'd

EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
	SPANDREL AND COUPLING BEAM FAILURES			
Chile, 1960	• Valdivia Regional Hospital (Medical Services Tower). An eight storey RC wall and deep member frame building built in 1935. Wide shear cracks (type X) formed in spandrel beams adjacent to walls. Concrete was weak (≈ 12 MPa).	-	2.2(j)	45
Armenia, 1988	• Leninakin nine storey apartments. Buildings had coupled walls providing lateral resistance in one direction and frames in other. Coupling beams had little or no shear reinforcement and shattered. Given defects in frames and lack of effective floor diaphragms the contribution of the wall defects to collapses could not be defined.	some C	2.2(k)	34
Anchorage	• Mount McKinley and 1200 L Apartment Buildings. Both buildings suffered extensive shear cracking (type X) in coupling beams and spandrel beams.	-	2.2(l)	42

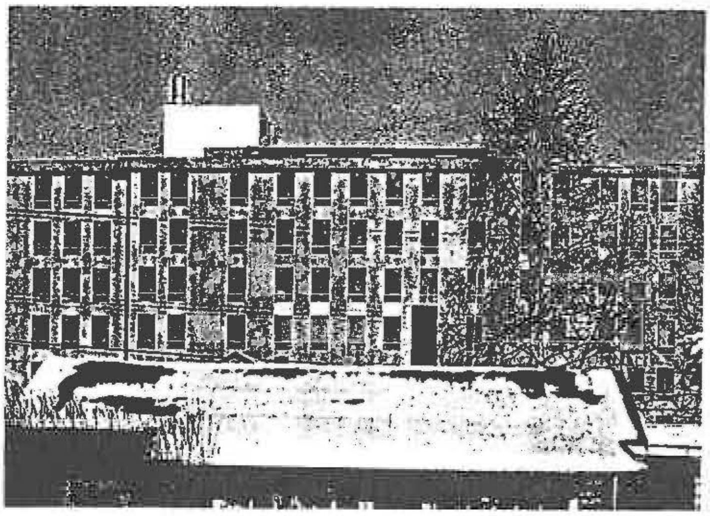


(a)



Pier Failure in South Face

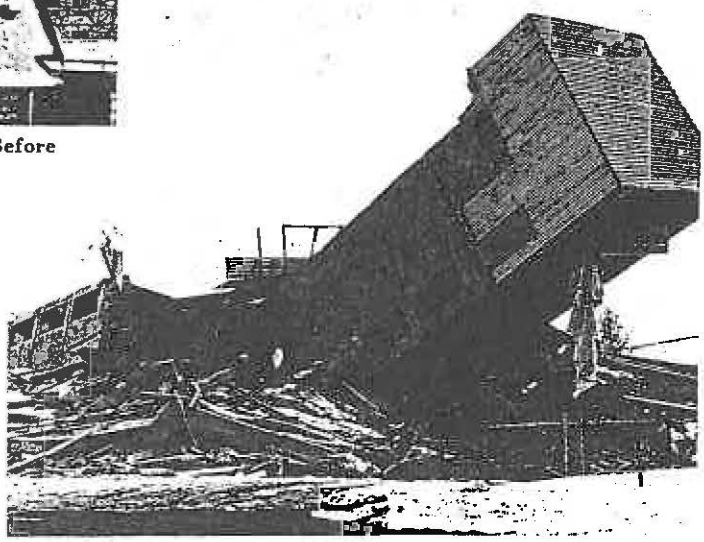
(b)



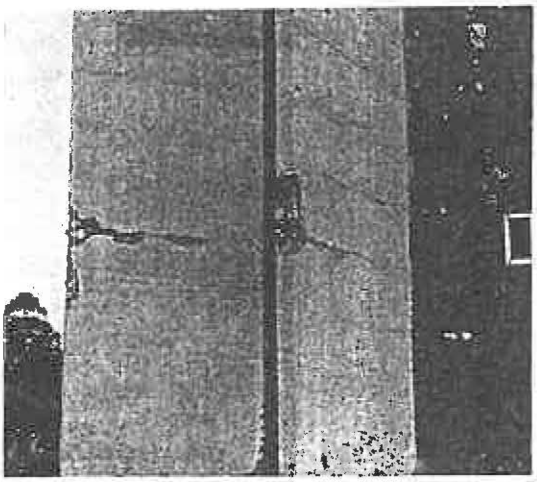
Four Seasons Apartment Building — Before Earthquake

(c)

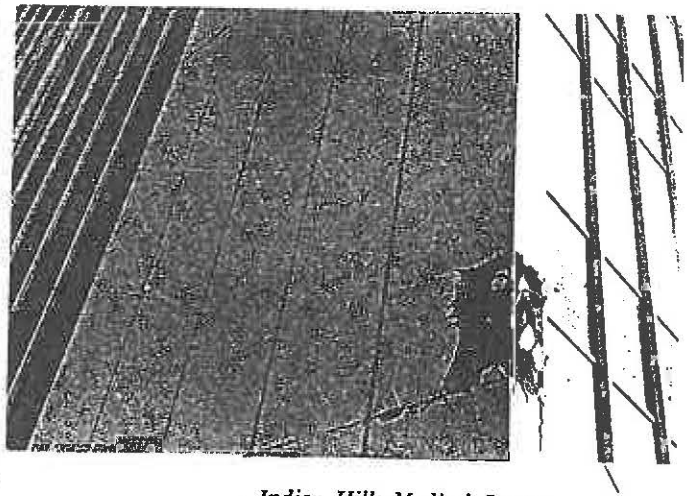
Fig. 2.2 Examples of Wall Failures
(see table 2.2 for Details)



(d)

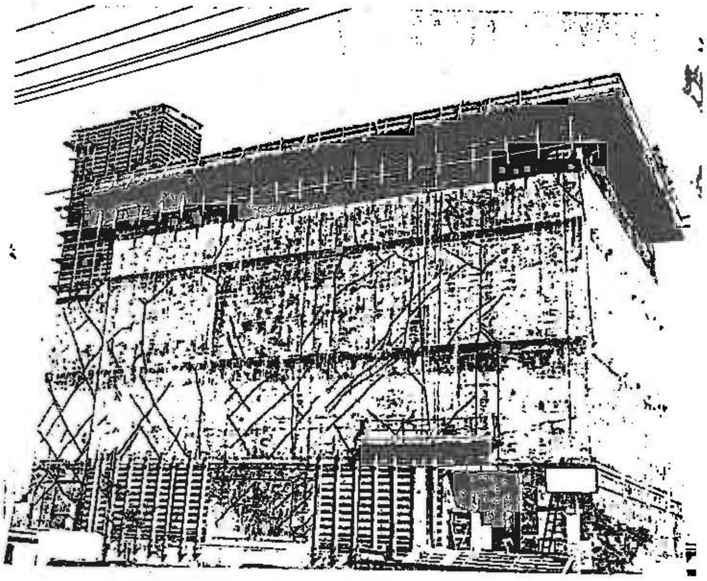


(e)



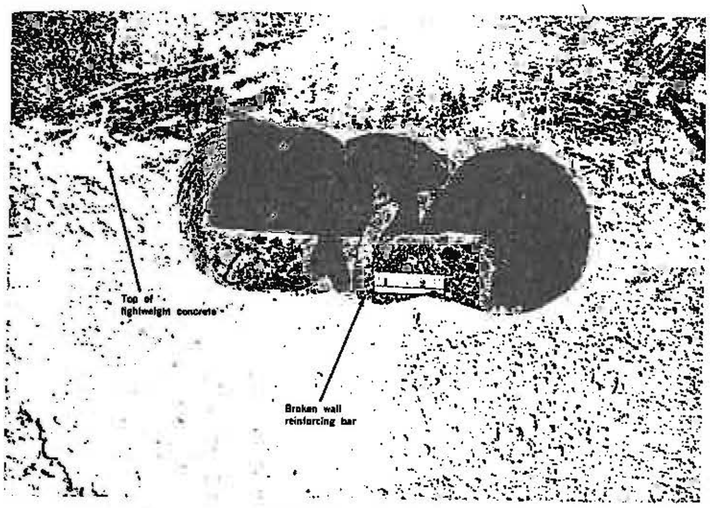
—Indian Hills Medical Center. North-side shear wall, west end.

(f)



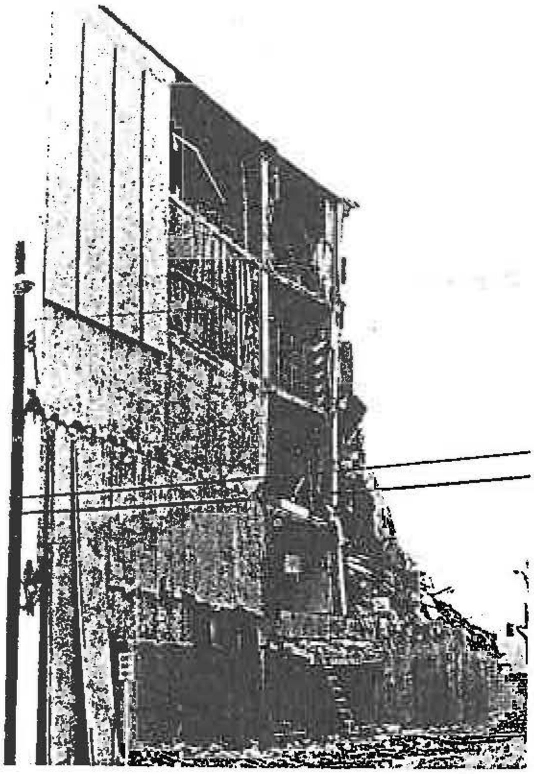
—Museum for Antique Cars. Building north wall, with repair of wall cracks in progress.

(g)

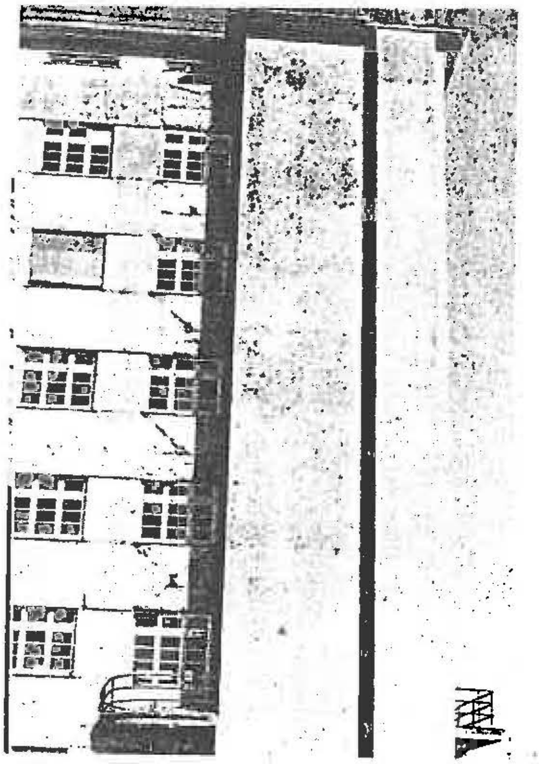


—Museum for Antique Cars. Typical wall reinforcing bars at first floor in north wall. Note offset of upper portion of bar to right of lower portion.

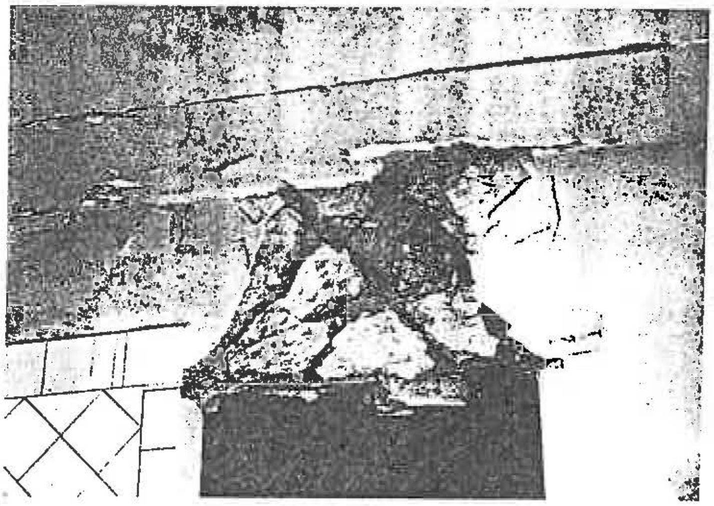
Fig. 2.2(cont'd) Examples of Damage to Wall Construction Joints (See Table 2.2 for Details)



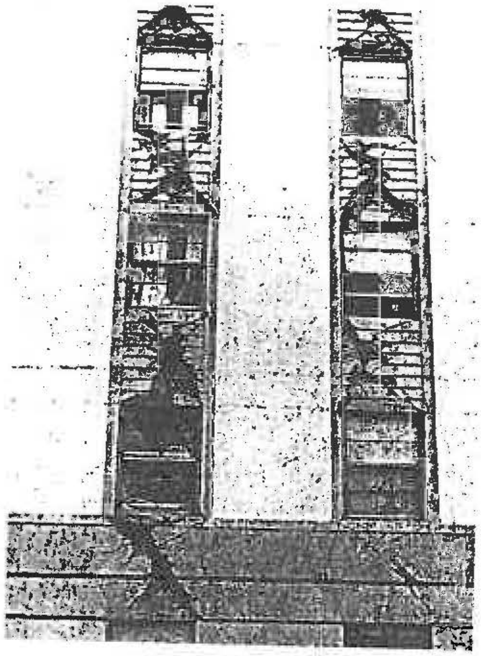
(i)



(j)



(k)



(l)

Fig. 2.2 (cont'd) (i) Collapse due to Construction Joint Failure
(j), (k) and (l) Spandrel and Coupling Beam Damage (see Table 2.2 for Details)

2.2.4 Other Deficiencies in Wall and Frame Buildings

Table 2.3 lists examples of other deficiencies in wall and frame buildings exposed by previous earthquakes.

Apart from the Mexico City earthquake (1985) where a number of partial collapses were attributed to pounding, no other cases of pounding causing collapse were found. However damage attributed to pounding is common.

Some examples of waffle slab collapse are given in Table 2.1. Use of waffle slabs is not common in New Zealand and it is unlikely that they will have been used as the "beams" of frames. However, the examples given in Table 2.2 suggest that flat slabs are susceptible to progressive collapse due to punching shear especially where there is no bottom slab steel passing through the columns.

There are several other factors relating to seismic damage which are not addressed in Table 2.3.

It has been noted [36] that buildings that are irregular in plan or elevation have mean damage ratios 3 to 6 times higher than those for regular buildings. Also, 42% of the buildings in Mexico City that failed in the 1985 earthquake were corner buildings [51]. Most of these had interior walls that were stiff and strong relative to the structure used on the buildings street boundaries.

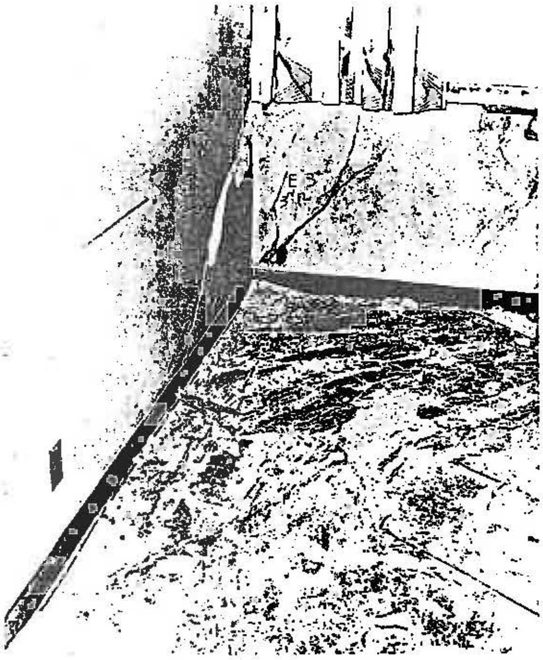
It should also be kept in mind that, generally, more than 80% of the cost of earthquake damage is the result of damage to infill walls, partitions, ceilings, plumbing, windows services and other non structural items.

TABLE 2.3 - EXAMPLES OF OTHER DEFICIENCIES IN WALL AND FRAME BUILDINGS EXPOSED BY PREVIOUS EARTHQUAKES

EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
	<u>DIAPHRAGM DAMAGE</u>			
San Fernando 1971	• Holy Cross Hospital. A seven storey shear wall building constructed in early 1960s. There was considerable damage to floor diaphragms especially where walls were not continuous to foundation level and diaphragms were forced to act as shear transfer elements.	-	2.3(a)	35
Armenia, 1988	• Nine storey apartment buildings in Leninakan. Buildings had precast hollow core floor units that did not have a topping and were not interconnected except for a small amount of insitu concrete at each end of the supporting beams. Lack of an effective diaphragm is believed to have significantly contributed to the collapse of many of these buildings.	many C	see 2.1(w)	34
	<u>POUNDING</u>			
Mexico, 1985	• De Carlo Hotel. One of many examples of where a flexible frame buildings pounded against an adjacent building. Damage was most pronounced where adjacent buildings had different heights and the floor slabs of the two buildings were at different levels. Dynamic loads generated by impact and damage to columns caused by impact of slabs in adjacent buildings lead to many partial collapses.	PC	2.3(b)	32
Chile, 1960	• Valdivia Regional Hospital. Built in approximately 1935 as six wings that were not structurally tied together. Wings were RC wall and deep member frame structures. Pounding caused extensive damage to walls and caused cracking of floor slabs. It was noted that differences in floor slab levels appeared to contribute to the amount of damage.			45

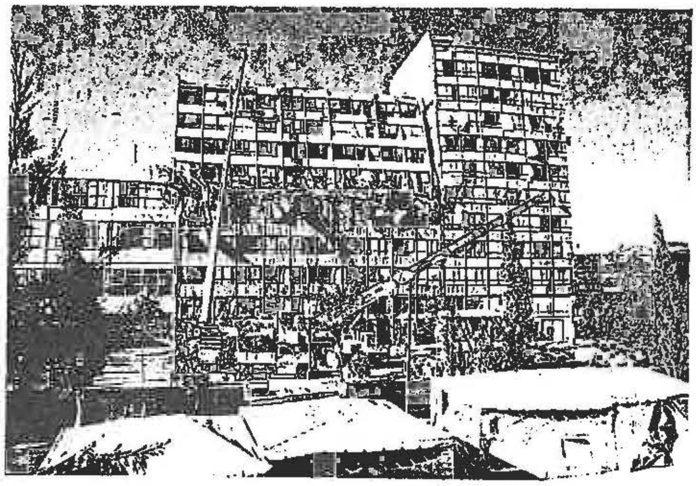
TABLE 2.3 cont'd

EARTHQUAKE	EXAMPLES OF FEATURES OF DAMAGE	Building Collapsed = C Building Partly Collapsed = PC Building Demolished = D	PHOTO REF (FIGURE NO.)	SOURCE REF.
Loma Preta - California, 1939	• Building South of Market Area - San Francisco. Impact damage was frequent in this earthquake. Photo shows large diagonal cracks in column cladding panels just above roof level of adjacent building. Note that crack pattern is consistent with large inertia forces being generated in the building above the impact point and acting to the right.	-	2.3(c)	50
	<u>PUNCHING SHEAR FAILURE OF SLABS</u>			
Newcastle, 1989	• Newcastle Workers Club. A heavy brick retaining wall collapsed onto an upper waffle floor slab causing a punching shear failure around the columns. The collapsed waffle slab then caused a punching shear failure of a lower waffle slab. As can be seen from the photos, top slab bars were ineffective as shear reinforcement as they simply "pealed" out of the top surface of the slab. Lack of slab bottom steel at columns, poor quality concrete and lack of internal walls contributed to the collapse.	C	2.3(d) 2.3(e)	31
Anchorage, 1964	• JC Penny Building. Several columns punched through the 250 thick RC flat floor slab. The failures seem to have been initiated by partial wall collapse and may not have contributed to initiation of the partial collapse.	PC		42
Mexico, 1985	• Waffle Slab Buildings. Many of these buildings were dependent on the waffle slabs to provide the beams for frame action. When buildings "pancaked" (see photo) punching failures were evident but the sequence of collapse was lost in the rubble.	C	2.3(f)	32



—Holy Cross Hospital. Diaphragm failure at west wall.

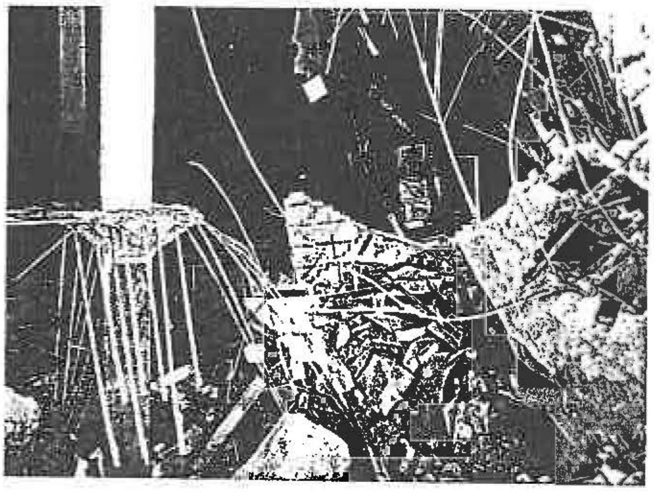
(a)



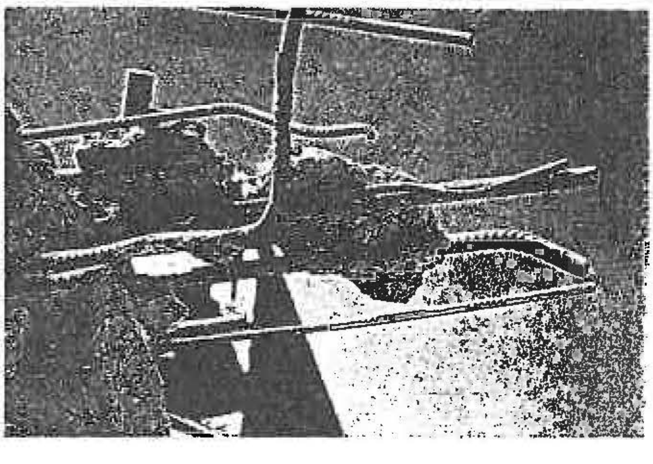
(b)

IMPACT DAMAGE. SEVERE DAMAGE TO
WINDOWS AND COLUMN FACINGS AT THE
LEVEL OF THE ADJACENT ROOF.

(c)

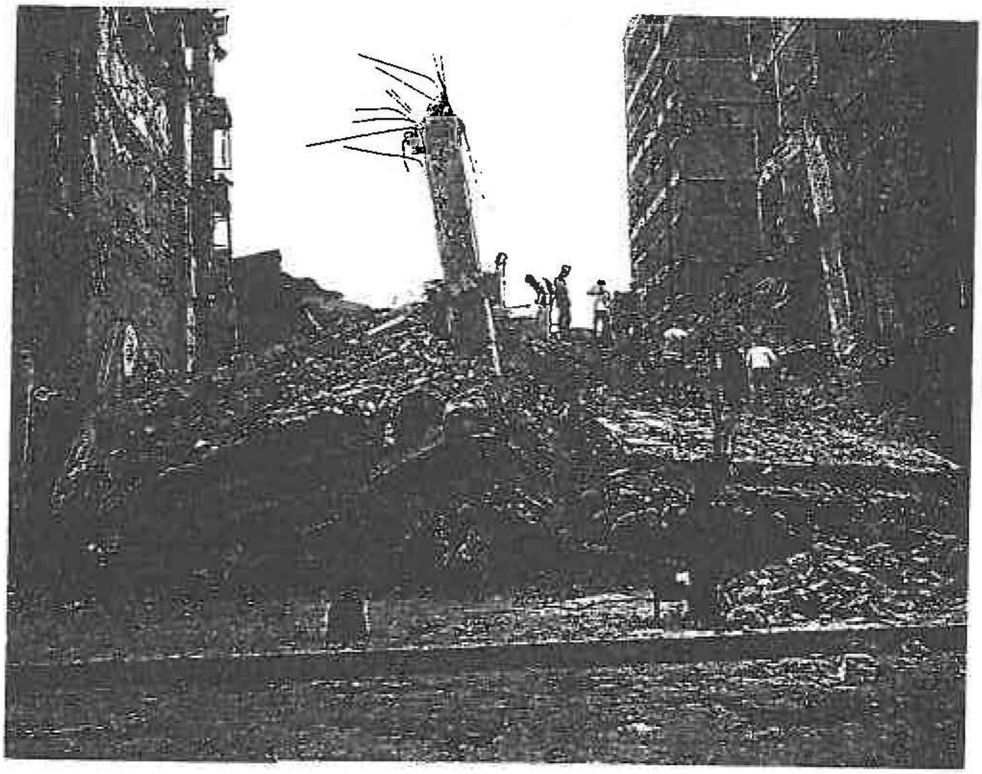


(d)



(e)

Fig. 2.3 (a) Diaphragm Damage (b) and (c) Impact Damage
(d) and (e) Punching Shear Failure of a slab (see Table 2.3 for Details)



In this building as in many others, the load-bearing column was forced like a punch through the concrete ceilings which collapsed like a sandwich

(f)

Fig. 2.3 (cont'd) Pancake Collapse of Waffle Slab building with Evidence of Punching Shear Failures Around Columns (See Table 2.3 for Details).

2.3 EXAMPLES OF TYPICAL WELLINGTON BUILDING : POTENTIAL DEFICIENCIES

A range of typical buildings were evaluated to identify common potential deficiencies in the 1935-75 building stock of Wellington City.

Details of the findings are given in Appendix A2. It is emphasised that the term "failure" in the detailed findings is used to identify the weakest links in the structural system and does not necessarily imply the element or building will collapse.

A summary of the findings is given in Table 2.4 along with the type and extent of damage that can be expected for structures with the potential deficiencies identified. The extent of damage and risk of collapse given in the table was evaluated using, as a guide, the seismic performance of buildings with similar deficiencies that was detailed in the previous section.

The term "high risk of collapse" used in the table is difficult to quantify. However based on the performance of buildings in previous major earthquakes it is likely to be less than 10% in the epicentral region.

The study of potential deficiencies in the Wellington buildings established that deficiencies that have resulted in poor performance of buildings in overseas earthquakes are also common in Wellington's 1935 to mid 1960s building stock. Most frame buildings, built in the late 1960s and early 1970s, had closer tie spacing than the majority of badly damaged and collapsed columns that were identified in the previous section. Some of the frames had overall detailing that was comparable to current New Zealand practice. Unfortunately, buildings that perform well in earthquakes are not studied as closely as those that fail in some manner. Therefore, the correlation between modern detailing practices and good seismic performance will remain uncertain until modern detailing is more extensively tested in major earthquakes.

The structural consequences of the Potential Deficiencies observed in the frames and walls of the case studies are summarised in Table 2.5. This table can be read in conjunction with Tables 2.1 to 2.3 in the previous section.

Apart from brittle column failures and external beam/column joint failures the deficiencies identified in the Wellington buildings do not normally lead directly to collapse. However, they will result in degrading strength and stiffness of the buildings structural system during a major earthquake. This may increase non structural damage and may indirectly contribute to a buildings collapse.

In the next section previous theoretical research on the effects of degrading strength and stiffness is reviewed.

TABLE 2.4 SUMMARY OF CASE STUDIES EXAMINING POTENTIAL DEFICIENCIES IN 1935-75 WELLINGTON BUILDINGS

Case (See Appendix A2)	Approximate Construction Date (No. Storeys)	Structural Type**	Potential Deficiencies (weak links)	Damage Expected - Major Earthquake
A	*L60's (7)	F	Beam/column joints only lightly reinforced. Beam steel cut off close to columns.	Low stiffness - damage to precast cladding and other non structural damage. Shear cracking beams and columns. - medium risk of collapse
B	L60's (10)	F	Beam/column joints unreinforced. Beam steel cut off close to columns. Light shear steel in beams Block infilling of some frames.	Low stiffness leading to high non structural damage. Extensive cracking of beams. Some flexural or shear cracking of columns. - medium risk of collapse
C	L30's (5)	PW/F	Inadequate shear and confining steel in beams, columns and joints.	Extensive shear cracking of beam and column elements. - risk of at least partial collapse is high
D	*M60's (7)	W/F,W	Shear strength and confinement of columns lacking, potential for highly torsional response if light diaphragm fails.	Shear failure in columns and diaphragm expected. - high risk of partial collapse

* L = Late M = Mid ** See Figure 1.9 for definition

TABLE 2.4 cont'd

Case (See Appendix A2)	Approximate Construction Date (No. Storeys)	Structural Type	Potential Deficiencies (weak links)	Damage Expected - Major Earthquake
E	E60's (8)	W/F	Columns, beam/column joints and wall weak in shear. Columns weaker than beams.	Damage likely to be localised in one storey of frames and bottom storey of shear walls. - high risk of collapse
F	M60's (9)	W/F,W	Walls, beams, columns and beam/column joints weak in shear.	Extensive shear cracking expected throughout. - medium risk of collapse
G	M60's (12)	W/F	Well detailed but shear still weak link in most elements such as the walls and columns.	Extensive shear and flexural cracking throughout. - low risk of collapse
H	L60's (14)	W/W,F	Only nominal steel in elements such as coupling beams.	Extensive shear cracking expected throughout. - low risk of collapse
I	L50's (5)	PW, W, F	Non-ductile detailing, elements likely to fail in shear. Anchorage failures likely due to plain bars.	Extensive shear and flexural cracking throughout. - medium risk of collapse
J	L60's (15)	W	Walls and diaphragms weak in shear.	Extensive shear cracking of podium diaphragms and walls expected. - low risk of collapse

TABLE 2.4 cont'd

Case (See Appendix A2)	Approximate Construction Date (No. Storeys)	Structural Type	Potential Deficiencies (weak links)	Damage Expected - Major Earthquake
K	L50's E60's (14)	W	Walls expected to fail in shear.	Extensive damage near base of shear walls. - low risk of collapse
L	E70's (10)	W	Architectural fin/columns slender and not tied to floor slabs well. Fins and transverse walls weak in shear.	Extensive cracking at base of transverse shear walls - failure of fin/columns in shear or at junction with floors likely. - high risk of at least partial collapse
M	L60's E70's (12)	W/F,W	Only detailed for limited ductility.	Walls should help distribute damage throughout frame - localised damage in shear walls and piles due to shear cracks expected. - medium risk of at least partial collapse
N	E50's (10)	F	Columns in exterior frames weaker than beams. Weakest link still likely to be shear in columns and beams. Infill panels will concentrate ductility demand.	Extensive wide shear cracking of elements. - medium risk of at least partial collapse

TABLE 2.4 cont'd

Case (See Appendix A2)	Approximate Construction Date (No. Storeys)	Structural Type	Potential Deficiencies (weak links)	Damage Expected - Major Earthquake
O	E60's (10)	W,F	Corner building with high torsion.	Severe cracking to columns on street frontages. - high risk of at least partial collapse
P	E70's (14)	F	Working stress design approach without ductile detailing.	Extensive wide cracks. - high risk of collapse
Q	L50's E60's (6)	F,W	No beam/column ties, ties widely spaced, short relatively weak columns in external frames.	Extensive wide cracks expected especially in columns of external frames. - high risk of partial collapse
R	L50's (6)	W	Corner building with high torsion. Non-ductile columns and flat slab connections.	Extensive wide shear cracks in exterior frame. Risk of collapse of floors due to shear failure of columns and/or slab connections high.
S	E60's (4)	W,F	Corner building with high torsion. Columns weak in flexure relative to beams. Will fail in shear in the columns and/or beam column joints.	Extensive wide shear cracks in columns and beam/column joints. - high risk of collapse

TABLE 2.4 cont'd

Case (See Appendix A2)	Approximate Construction Date (No. Storeys)	Structural Type	Potential Deficiencies (weak links)	Damage Expected - Major Earthquake
T	L60's E70's (16)	W	Walls likely to fail in shear rather than flexure.	Extensive wide shear/flexural cracks in walls particularly near base of wall. - low risk of collapse
U	E70's	F	Working stress type design and detailing. Connections between frame and boundary walls unlikely to cope with expected relative movements. Frames on street frontage have blockwork infills below window sill level.	Blockwork boundary walls likely to separate from frame and collapse. Extensive wide shear cracks in street frontage columns expected. - high risk of at least partial collapse
V	L60's (16)	F	Advanced design including provision of overstrength in columns and ductile detailing - no known weak links.	Repairable flexural cracking expected to beams throughout. - very low risk of collapse
W	E60's (10)	W/ W,F,PW	Detailing and dimensions of shear walls don't meet current limited ductility provisions. Shear walls weaker in shear than flexure. Transfer diaphragm not designed for this action.	Intensive damage to transverse shear walls above 3rd floor level. Wide shear cracking of perforated wall elements, diaphragms at 3rd floor level and transverse walls below 3rd level. - low risk of collapse

TABLE 2.4 cont'd

Case (See Appendix A2)	Approximate Construction Date (No. Storeys)	Structural Type	Potential Deficiencies (weak links)	Damage Expected - Major Earthquake
X	M60's (9)	W,PW	Shear walls offset at 1st floor and 1st floor diaphragm not designed as transfer diaphragm. External perforated walls and internal frames not detailed for ductility.	Localised but severe cracking of 1st floor diaphragm. Extensive cracking of column, and joint zones of perforated walls. - low to medium risk of collapse

TABLE 2.5 STRUCTURAL CONSEQUENCES OF POTENTIAL DEFICIENCIES NOTED IN FRAMES AND WALLS OF CASE STUDIES

Structural Type	Potential Deficiency (weak links)	Structural Consequences in Major Earthquake	Cases Identified - see Table 2.4
F	Brittle Column Failure: - inadequate shear or confinement reinforcement - high axial load - pounding - short column effect	Most common causes of partial or total collapse of buildings. Often difficult to isolate flexural, shear and axial load contribution to damage. Pounding against adjacent building may cause short column type shear failure or shear failures due to high impact shears above the point of impact.	A,C,D,E,F,G,I,L, O,P,Q,R,S,U
	Soft Storey: - columns weaker than beams - walls discontinuous at a storey - inadequately separated infill panels	May be initiated due to infill panel or wall failure at one level. Concentrates ductility demand at one level - any consequential loss of strength accentuates concentration of demand. PΔ effects increased and concentrated in one storey - may result in collapse.	B, E, I, N
	Beam/Column Joint Detailing: - light reinforcement - no reinforcement	Frame flexibility increased resulting in greater displacement demand and larger PΔ effects - may lead to collapse. Joints may protect adjacent beams and column from shear or flexural failures by providing a fuse or weak link. Failure of external joints may result in partial collapse especially if axial load is high.	A,B,C,E,F,Q,S,X

TABLE 2.5 cont'd

Structural Type	Potential Deficiency (weak links)	Structural Consequences in Major Earthquake	Cases Identified - see Table 2.4
F	Beam Shear Failure: - beam steel cut off too close to columns - inadequate shear or confining reinforcement	Beam shear failures result in slumping of floors and loss of strength and stiffness for frame. Providing bottom beam steel is adequately anchored into support even local collapses are rare.	A, B, C, F
	High Torsional Eccentricity: - corner buildings with walls on boundaries remote from street	Can generate additional displacement demand and concentrate it in the weakest elements - may contribute significantly to collapse.	O, Q, S, U
W	Inadequate Shear Strength: - wall weaker in shear than flexure - weak construction joints	Diagonal crushing and fracture of wall ties leads to degrading strength and stiffness. Localised damage may be severe but examples of collapse rare.	E,F,G,I,J,K,L, M,T,W
	Inadequate Wall Ductility: - lack of confinement in compression zone - excessive slenderness	Spalling of concrete in compression zones and buckling of compression reinforcement leads to degrading strength and stiffness. Examples of collapse rare.	W, many of above cases would be included here if closely examined.
	Diaphragm Capacity: - Inadequate	Crushing and cracking may be severe but examples where diaphragms have been identified as contributing to collapse are rare.	D, W, Y

TABLE 2.5 cont'd

Structural Type	Potential Deficiency (weak links)	Structural Consequences in Major Earthquake	Cases Identified - see Table 2.4
W	Spandrel and Coupling Beam Detailing:	Cracking and crushing damage often severe - leads to degrading strength and stiffness. Normally does not lead to collapse.	H
	Wall Rocking: - inadequate foundation strength	Wall uplift may damage beams and slabs fixed to wall extremities.	D, I, J, L
	Pile Strength: - inadequate shear strength and/or ductility	May result in slumping and lateral displacement at foundation level - does not normally result in collapse.	M

2.4 EFFECTS OF STIFFNESS AND STRENGTH DEGRADATION

2.4.1 Introduction

The preceeding two parts of this section of the report indicated that many of the structural deficiencies identified in Wellington 1935 to 1975 building will result in strength and stiffness degradation.

Frame buildings usually collapse because the columns fail brittlely and lose their axial load carrying capacity. This type of collapse, which does not involve a significant lateral translation of the building, is not considered in this part of the report.

However, brittle failure of some of the columns in a frame that results only in an overall loss of stiffness and strength in a buildings lateral load resisting structural system is considered.

2.4.2 Stiffness Degradation

The inelastic behaviour of structures is often modelled as ideal elastoplastic behaviour. In practice bond slip, yielding of shear reinforcement and other irreversible effects give rise to stiffness deterioration and even "pinching" of the structures hysteretic load/deformation response.

There have been numerous studies that examine the effects that different types of stiffness deterioration have on a structures response to various types of earthquake motions.

These have included earthquake motions from Europe [2, 7], Taiwan [29], California [8, 2, 7, 12] including motions representative of near fault effects [4, 7, 8, 12] and the long period motion recorded in Mexico City's Lake Bed Zone [6].

Types of hysteretic models studied have included the effects of shear slip [2, 29], stiffness degradation [2, 6, 8, 7, 12, 29], pinching [7, 12] and both moderate [2, 7, 12, 29] and high rates [12] of post yield strain hardening.

The conclusion that can be drawn from these studies is that, on average, the difference between the maximum displacement demand, Δ_u (or displacement ductility demand Δ_u/Δ_y, where Δ_y is the yield displacement) for stiffness degrading systems and ideal elastoplastic systems is not large [8]. However for some ground motions and some period ranges the maximum displacement demand may differ significantly from those obtained for an elastoplastic system [2, 7, 12, 29]. This difference can often, although not always, be explained by the reduced post yield stiffness of the degrading system which increases the effective period of the structure. As the elastic displacement response of structures to earthquakes generally increases with increasing period this "period shift" usually increases the building peak displacement response. Figure 2.4, obtained from reference [2], illustrates this period shift, "ΔT", for a shear slip model relative to a bilinear model (i.e. Elastoplastic model with 10% strain hardening) when both models are used to examine the response of a SDOF structure to the ground motion recorded during the 1979 Montenegro earthquake.

2.4.2.1 Effect of Initial Slackness

There does appear to be one case, however, where the type of hysteretic model effects the peak displacement demand consistently. This is the extreme case where the structural system is modelled as having zero strength for an initial displacement as would be caused by "initial slackness" in the "tension only" cross bracing of a frame. In this case the maximum displacement demand is increased by approximately the same amount as the initial slackness and for short period structures and some earthquake records the displacement demand may increase by more than the initial slackness [11].

The shear slip model appears to be an intermediate case between the more moderate amounts of stiffness degradation experienced by R.C. structures and structural systems with initial slackness. After the first yield cycle the shear slip model develops "slackness" equal to the yield displacement. A study using this model and 60 Tiawanese rock site earthquake records [29] showed that, for structures with elastic periods less than 2.5 secs, peak displacements were, on average, 2 to 4 times greater than those obtained using an elastoplastic model.

This can be compared with an average increase of less than 50% obtained using other degrading stiffness models (Takeda, modified Clough and Q) within the same period range. These results are similar to those obtained for the Montenegro Earthquake [2], and shown in Figure 2.4.

2.4.2.2 Importance of Hysteretic Damping

Until relatively recently it was felt that stiffness degradation would increase maximum displacement demand significantly because it would reduce the effective damping of a structure's response. It has now been concluded [7], that variations in the shape of the hysteresis loop for a structure will not have a major influence on its dynamic inelastic displacement demand. The explanation given for this observation is that an initial increment of hysteretic damping will have a marked effect on a structures response while further increments have a rapidly diminishing effect. However there may not be a significant difference between the hysteretic damping of stiffness degrading and elastoplastic systems.

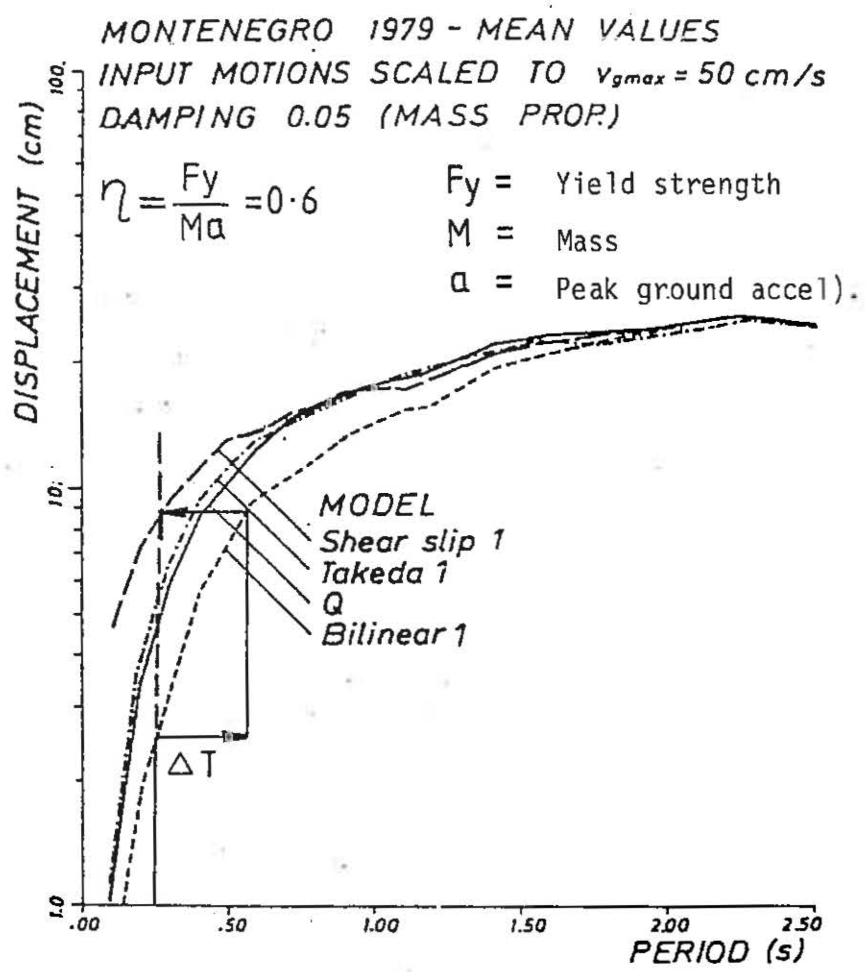


Fig.2.4 Effect of hysteretic model on displacement demand.

(obtained from ref.[2])

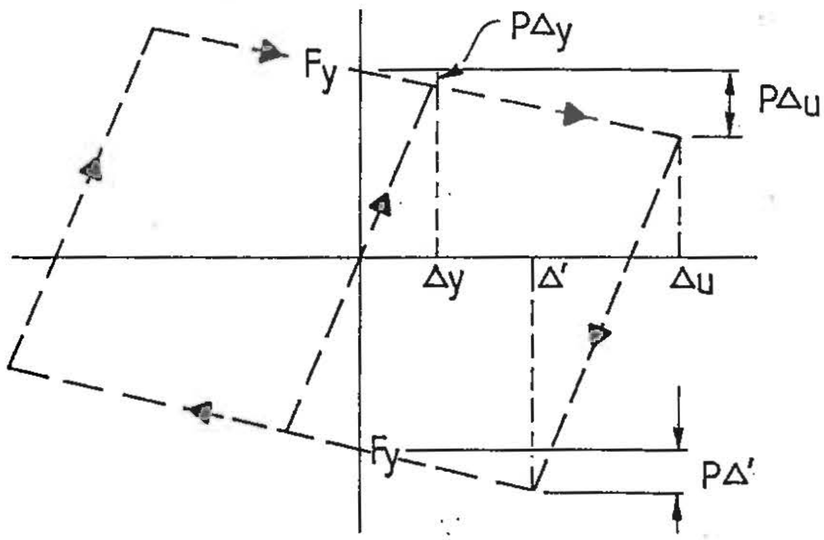


Fig.2.5 Net strength after allowing for PΔ Effect

A surprising difference [8] between the behaviour of degrading stiffness and elastoplastic systems is that stiffness degrading structures experience far fewer cycles in which the structure reaches its peak strength (i.e. yield excursions). In spite of this, at least for a range of Californian earthquake records [8], the total hysteretic energy absorption for the two systems is similar due to significant energy absorption during the small displacement cycles of stiffness degrading systems. This suggests that structural damage and effective damping will be similar for both types of system. Some recent research [29] using 60 Tiawanese rock site records has indicated that degrading stiffness systems (excluding the shear slip model), on average, consistently absorb twice the hysteretic energy that elastoplastic models do across a wide period range (0.5 to 5.0 secs). This suggests that, for these earthquake records, the degrading stiffness models may have greater effective damping. However, as energy absorption is likely to be related to structural damage, it may also indicate that structures with degrading stiffness will be subjected to greater structural damage.

2.4.3 Strength Degrading Systems

At present the effects of degrading strength are far more difficult to predict with confidence than the effects of degrading stiffness. Much of the work on strength degrading structures has been carried out using a bilinear model like that shown in Figure 2.5 which models, for example, the effective residual strength of a structure after allowing for PΔ effects.

The displacement demand of elastoplastic systems, may be amplified significantly by PΔ effects [19] and at a particular strength level for a given earthquake the amplification may result in collapse as indicated by Figure 2.6. This effect has been found to be particularly severe for the Pacoima Dam Earthquake Record [8, 9] which has the type of long acceleration pulse which is often associated with near fault earthquake records.

The amplification of displacement demand due to PΔ effects takes place because the structure develops an incrementally increasing drift in one direction as indicated in Figure 2.7. The amplification is therefore, quite sensitive to the duration of the earthquake or to the influence of aftershocks.

There are three possible factors that might contribute to the amplification of displacement demand that occurs with this type of strength degradation. These are:

(i) unequal "strength" when load reverses;

(ii) release of elastic stored energy;

(iii) loss of "strength".

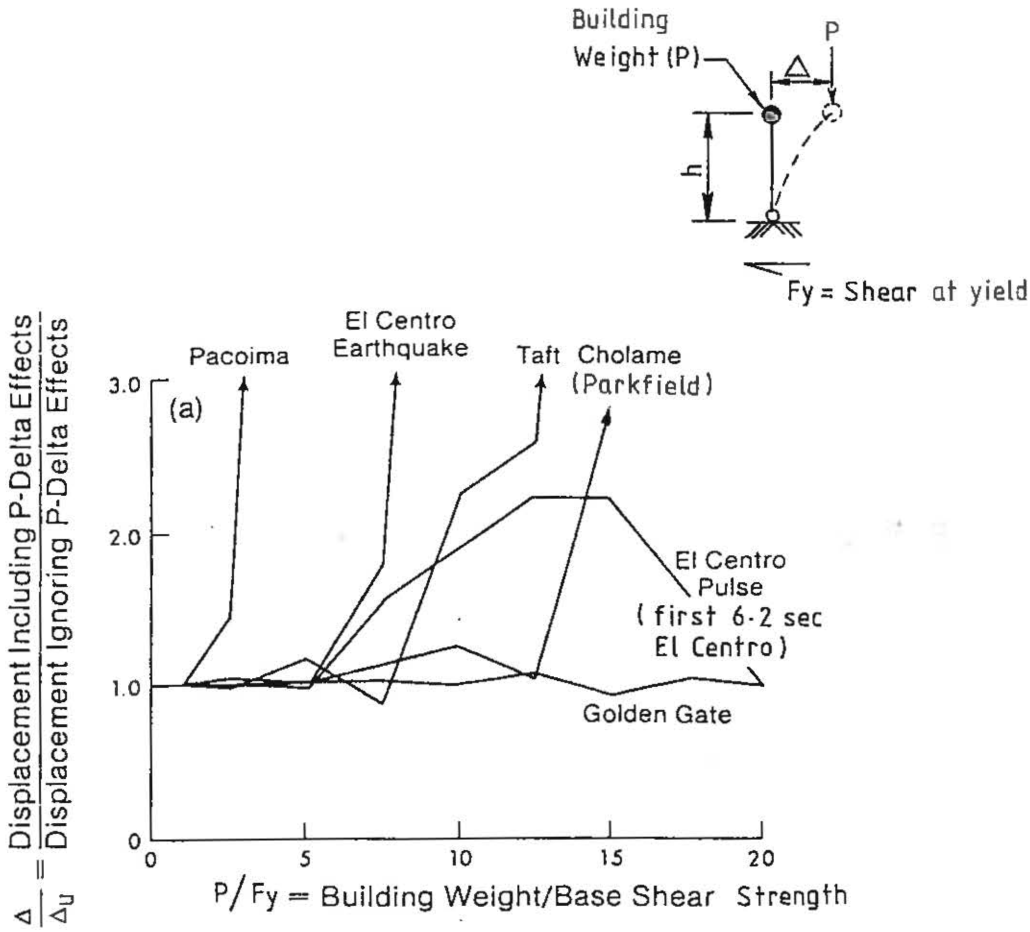


Fig 2.6 Amplification of maximum displacement demand due to PΔ effects (for 1 storey building with Δ at yield = Δy = 0.005h)

(figure obtained from [9])

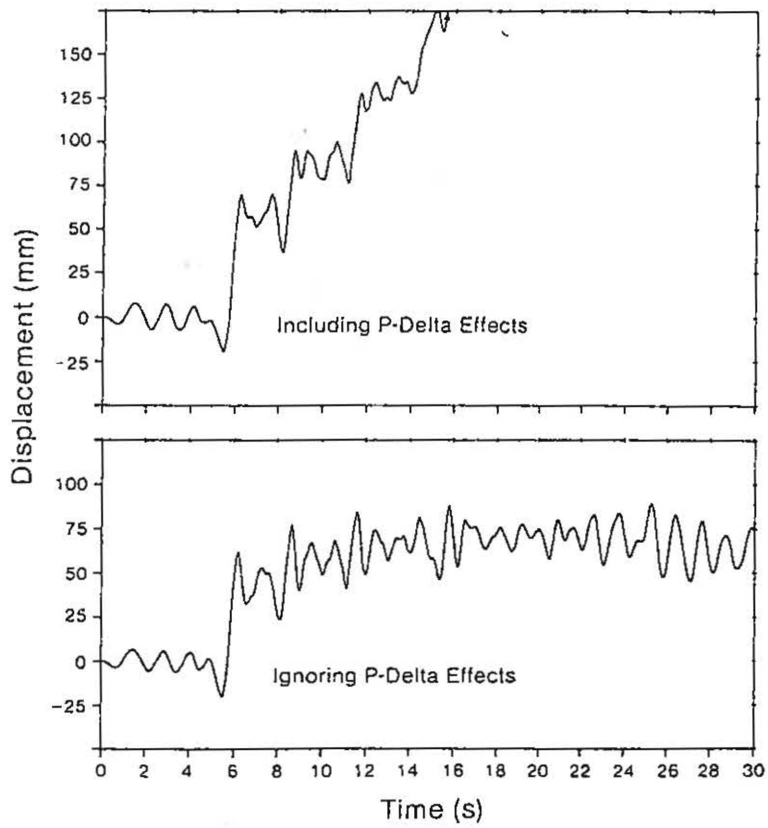


Fig 2.7 Time-history response, Taft earthquake : effect of PΔ (obtained from [9])

2.4.3.1 Unequal Strength for Load Reversal

Figure 2.5 indicates that once an inelastic displacement has occurred in one direction a strength or energy "hump" must be overcome for the structure to return to zero displacement. This aspect of PΔ type strength degradation is similar to that modelled in Figure 2.8 where the structural system has unequal strength under load reversal (i.e. $F_y \pm P\Delta$).

The response of a system like that shown in Figure 2.8 has been compared with that of an elastoplastic system with yield strength F_y in each direction [5]. The ground motion used for the comparison was the E-W component recorded at the Ministry of Communications and Transportation in Mexico City during the 1985 earthquake. Although this is not a typical earthquake motion the study indicated that a strength differential, PΔ, equal to 20% of F_y could amplify maximum displacement demand by a factor of 6 for a SDOF structure with an elastic period of two seconds. The same structure modelled as shown in Figure 2.5 with $P\Delta_y$ equal to 10% F_y and subjected to the same earthquake record required nearly three times the yield strength that an ordinary elastoplastic system required for both structures to have the same maximum displacement demand [6].

Therefore, at least a large part of the amplification of maximum displacement demand that occurs with PΔ type strength degrading structures can be explained by considering the effects of unequal strength under load reversal.

Systems with unequal strength under load reversal also develop incremental progressive drift similar to that indicated in Figure 2.7 for a PΔ strength degrading system and therefore, the unequal strength factor probably also explains this phenomena as well.

It is perhaps ironic that stiffness degradation may eliminate the strength/energy hump indicated in Figure 2.5 so that stiffness degrading systems may not be as susceptible to PΔ amplification of displacement demand to the extent that elastoplastic systems are.

2.4.3.2 Release of Elastic Stored Energy

Figure 2.9 shows two alternative load deformation paths for a structure between points "a" and "d". One is via "b" the other via "c". During unloading between "b" and "d" the elastic energy stored in the system represented by the area 'abc' is released and the energy represented by the area 'bdfe' is absorbed hysteretically. If the two areas 'abc' and 'bdc' are equal as drawn, the total net energy absorbed when moving along the path 'bd' is represented by the area 'cdfe'. This is the same energy that would be absorbed if the load path had been directly from 'c' to 'd'. Therefore, providing the falling branch of the loading curve is not too steep, the release of elastic energy is not likely to significantly effect the dynamic response of a structure.

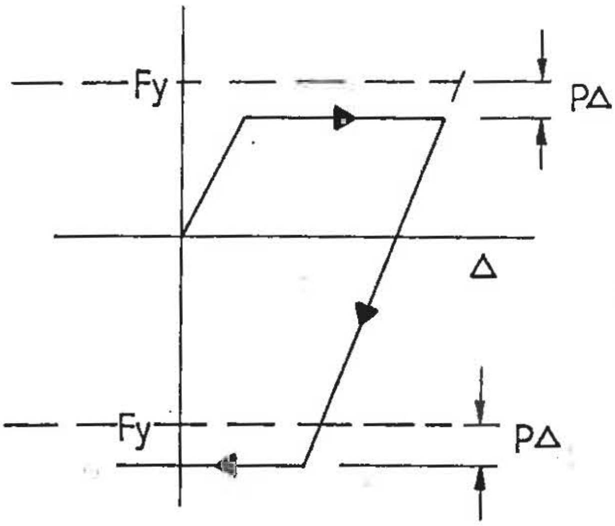


Fig. 2.8 Structure with unequal strength in each direction

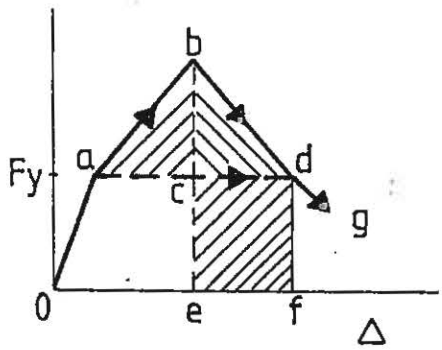


Fig.2.9 Degrading strength falling branch effect

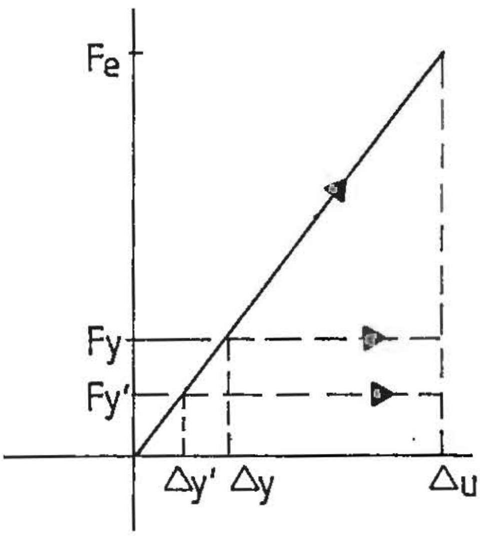


Fig. 2.10 Equal displacement principal - Loss of strength effect

Reference [11] reports the results of inelastic dynamic analysis using degrading stiffness models with spine curves similar to those indicated by 'oabdg' and 'oacdg' in Figure 2.9. Although not explicit, it would appear that the maximum displacement demand was not effected by the choice of spine curve. It would, therefore, appear that the release of elastic energy associated with the falling branch of an hystersis loop is not an important consideration when examining strength degrading systems.

2.4.3.3 Loss of Strength

Figure 2.10 illustrates the equal displacement rule for structures responding to earthquake motions. This rule states that the maximum displacement demand, Δ_u, is the same for an inelastic structure as it is for an elastic one with the same initial stiffness. This rule suggests that a reduction in yield strength from F_y to F_y' will not change the maximum displacement demand Δ_u as indicated in Figure 2.10.

Therefore degrading strength alone should not increase the maximum displacement demand imposed on a structure by an earthquake.

Several researchers have evaluated the validity of the equal displacement rule. Figure 2.11 was obtained from reference [2] and shows the response spectra for a SDOF structure with various strength levels, $\eta = F_y/Ma$, where M = structures mass and a, is the peak ground acceleration in the 1979 Monenegro earthquake record that was used in the analysis. The curve corresponding to $\eta = 5$ is the elastic response and the remaining curves are inelastic response spectra.

For structures with short initial periods (computed using the stiffness before yielding) there is clear evidence of a 'period shift' in Figure 2.11 similar to that indicated in Figure 2.4. The "period shift" means that structures with inelastic displacement demand have a peak displacement demand, Δ_u, corresponding to that expected for an elastic structure with a longer initial period, $T + \Delta T$. This "period shift" corresponds to the reduced effective stiffness of the inelastic system and can be seen to increase with the inelastic displacement associated with low yield strength levels. Other researchers [8, 12, 7, 11, 29] present their results in a manner that tends to disguise the effects of period shift. However if the results given in references [7] and [12] for example, are reinterpreted using the appropriate displacement response spectra obtained from reference [7] the results indicate that increased displacement demand of ductile structures can generally be explained by considering the effect of a period shift.

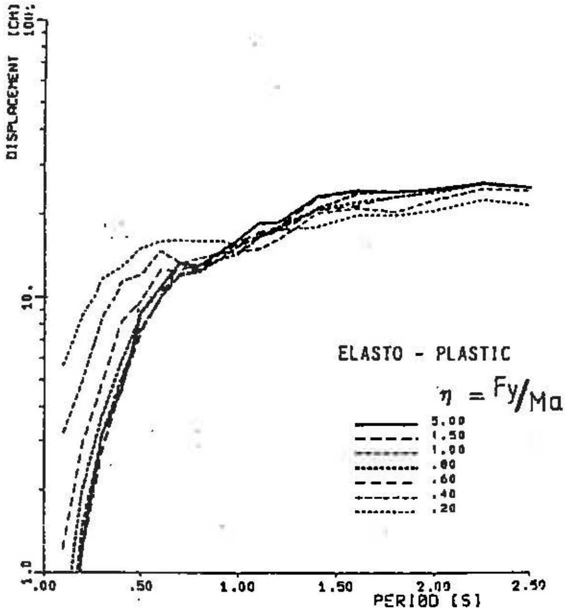


Fig. 2.11 Effect of yield strength on displacement demand
(obtained from ref [2])

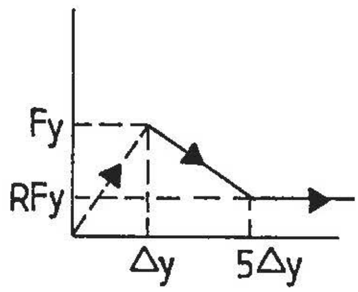


Fig.2.12 Yield Strength Multiplier [7]

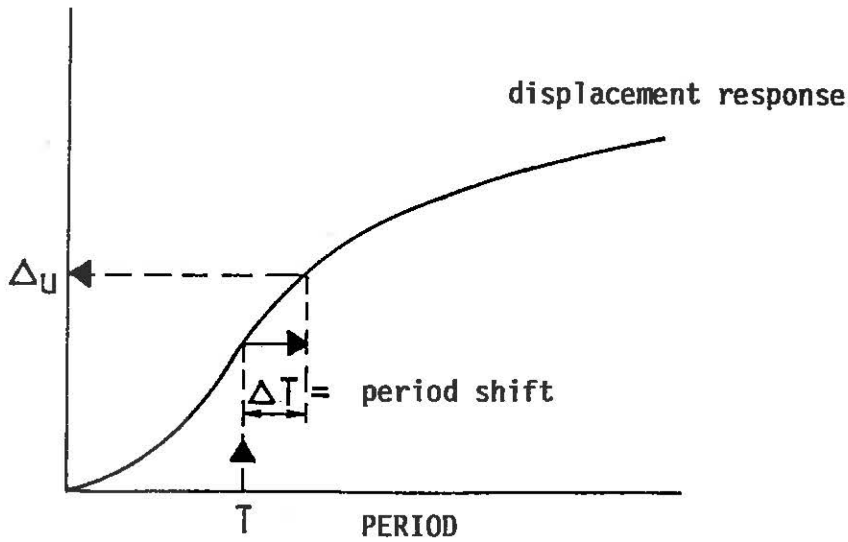


Fig. 2.13 Estimation of maximum displacement demand

However, there is at least one earthquake motion for which the equal displacement rule does not apply. This is the SCT ground motion recorded in the Lake bed zone of Mexico City during the 1985 earthquake. For this earthquake motion [6] the maximum displacement demand, Δ_u, is only a ¼ of that predicted by the equal displacement rule for a SDOF structure with an initial period of 2.5 seconds and responding with a ductility factor, $\Delta_u/\Delta_y = 4.0$. Although the recorded motion was unusual, being almost synosoidal and of long duration, it illustrates that the equal displacement rules applicability can vary with the type of earthquake motion. However in general, if an allowance for period shift is made, the equal displacement rule suggests that a loss of strength considered in isolation should not increase the maximum displacement demand imposed on a structure.

There appear to have been only a limited number of studies of strength degrading structures which do not involve the "energy hump" associated with PΔ type strength degradation.

Fajfar [2] examined the behaviour of a strength and stiffness degrading SDOF system that had a range of yield strengths and natural periods. For each yield strength level a critical natural period was reached that suddenly caused displacements to increase rapidly to "collapse". The form of the results suggests numerical instability in the analysis may be responsible for this apparent behaviour and that this may not be a real phenomena.

Moss [7] modified an elastoplastic hysterisis model by reducing the yield strength whenever previous cycles had exceeded the initial yield displacement, Δ_y. For subsequent cycles the yield strength was factored by a multiplier as indicated in Figure 2.12 It would appear that this method of modelling strength degradation did not involve a falling branch in the load path and therefore the release of elastic energy. The results of dynamic analysis indicate that large increases in maximum displacement demand occurred under some conditions. These results tend to conflict with the results of a more extensive study by Dean et al [11] who used a number of hysteresis models with combined degrading stiffness and strength and concluded that the choice of hysteresis model (i.e. load path) generally didn't effect the maximum displacement demand.

Obviously more work is required to clarify the effects of strength degradation. Even so it may be tentatively concluded that if an allowance is made for "period shift", loss of strength alone will not, in general, increase maximum displacement demand. However loss of strength will make a structure more susceptible to amplification of maximum displacement demand by PΔ effects. Montgomery [9] concluded that if the overturning moment for a SDOF structure due to PΔ effects, $P \times \Delta_u$ is less than 10% of the overturning moment at yield, $F_y h$ (see inset Figure 2.6) a significant amplification of displacement demand due to PΔ effects would not occur.

This suggests that if residual strength, F_R, at maximum displacement demand, Δ_u, is considered instead of initial yield strength F_y, $P\Delta$ effects should not be significant for strength degrading structures if $P\Delta_u < 0.1\ F_R h$. This rule may be modified for multi-storey structures as suggested by reference [9], but may be too conservative for stiffness degrading systems which do not develop a significant "energy hump". However it would appear [9], that the rule may be unconservative for the Pacoima Dam earthquake record with its near fault characteristics. Some structures, such as buildings with long walls that carry most of the gravity loads, will not be significantly effected by $P\Delta$ effects. It may be tentatively concluded that the displacement demand imposed on these structures by earthquakes will not be significantly effected by strength loss.

2.4.4 Proposed Method of Analysis for Strength and Stiffness Degrading Structures

The current NZ loadings code [16] limits the maximum displacement capacity of a structure by permitting no more than a 20% strength loss at peak displacement. Most building collapses are due to brittle column failures which result in the columns being unable to carry their axial loads. However the damage sustained by other buildings after earthquakes suggests that structures can sustain a loss of more than 20% of the peak strength of their lateral load resisting systems without collapse. This suggests that there is a significant margin between "code failure" and "collapse". Although the size of this margin will depend on structural type, the use of code criteria which are based on a "failure" criteria of 20% strength loss often results in an unrealistic assessment of the risk of collapse for existing structures.

There is therefore, the need to develop a more realistic analysis method to evaluate the risk of collapse of existing structures during earthquakes.

Although research to date, as outlined above, is inconclusive it would appear that the maximum displacement demand imposed on a structure by an earthquake is relatively insensitive to stiffness and strength degradation of its lateral load resisting system. However, this assumes that the structure retains enough strength to resist amplification of the displacement demand due to $P\Delta$ effects which could lead to progressive incremental collapse.

This suggests that the following analytical procedure would be the appropriate when evaluating the risk of collapse of existing structures.

1. Obtain the structures initial elastic fundamental period, T.

2. Estimate an appropriate period shift, ΔT, to allow for the increase in structural period due to loss of

effective stiffness. This step would need to be iterative as the period shift will depend on the size of the inelastic component of the displacement demand.

3. Use an elastic displacement response spectra to estimate the maximum displacement demand, Δ_u, as indicated in Figure 2.13 (note: a pseudo displacement response spectra, S_d, could be obtained from a "design" acceleration response spectra, S_a, using the relationship $S_d = \left[\frac{T}{2\pi}\right]^2 S_a$).

4. Evaluate, using appropriate test results, the structure residual strength, F_R, at the maximum displacement demand, Δ_u.

5. Check that the structure has adequate residual strength to resist $P\Delta$ effects.

For SDOF structures an appropriate $P\Delta$ check might be $P\Delta_u < .1F_Rh$. For multi-storey frame structures this could be modified as given in [9]. An appropriate criteria for checking shear wall structures needs to be developed. A tentative procedure for evaluating shear walls is given in Section 3.7 of this report. The procedure does not consider $P\Delta$ effects or strength degradation but does indicate the need to also consider effective damping as well as period shift when estimating peak displacement demand.

2.5 SUMMARY AND CONCLUSIONS : SECTION 2

Most buildings that have collapsed in previous earthquakes have been dependant on frames for their seismic resistance. Relatively few RC buildings that are primarily dependant on walls for their seismic resistance have collapsed.

The vast majority of frame collapses have been attributed to brittle column failure. Where column shear failures have lead to collapse, axial load has tended to play a significant if not dominate roll. It is very difficult to find examples of building collapses where the investigators have attributed the collapse to beam shear or beam/column joint failure.

In walls severely damaged in previous earthquakes there has been a tendency for damage to be concentrated at wall construction joints or at a single pair of diagonal cracks in walls, coupling beams or in the "beams and column" elements of perforated walls. This type of damage may be characteristic of the type of damage that can be expected in lightly reinforced walls.

The structural forms and detailing of a range of 1935 to 1975 Wellington buildings was examined. The examination indicated that the types of potential deficiencies that have lead to severe damage or collapse in previous overseas earthquakes are common in Wellington's 1935 to 1975 building stock.

Many of the potential deficiencies in these buildings will result in degrading strength and stiffness of the buildings structural system during a major earthquake and not necessarily lead to collapse.

Existing theoretical research on stiffness degradation indicates that these aspects of structural behaviour are not as important as they were once thought to be. When various stiffness degrading structural models are compared with an ideal elastoplastic model the energy absorption and therefore effective damping is found to be similar or greater for the degrading stiffness systems. However, when compared with a structure deforming elastoplastically, structures with degrading stiffness may experience a "period shift" and an increase in maximum displacement demand as a consequence.

More theoretical work is required to clarify the effects of strength degradation on the response of structures. It was tentatively concluded that if an allowance was made for "period shift", loss of strength alone would not increase the peak displacement imposed on a structure by an earthquake. However a loss of strength would make a structure more susceptible to amplification of its peak seismic displacement due to the influence of $P\Delta$ effects.

A tentative method for evaluating structures exhibiting strength and stiffness degradation is proposed for future development. It is unfortunate that most laboratory testing of structural components is terminated before the characteristics of the components strength degradation is established. This will hinder the development of the proposed methodology.

In the first section of this report on 1935 to 1975 RC buildings, wall and wall/frame combinations were identified as the dominant structural form used for this vintage of building in Wellington.

In this section of the report the lack of documented cases of shear wall buildings collapsing in previous earthquakes was highlighted. This is in spite of a lack of ductile detailing and capacity design to ensure that the walls did not fail in shear. In the final section of this report the behaviour of walls that will fail at least partially in a shear mode is examined.

SECTION 3

STRUCTURAL WALLS YIELDING IN A COMBINED SHEAR AND FLEXURAL MODE

3.1 INTRODUCTION

The New Zealand concrete design code [13] requires structural walls to be capacity designed to ensure that they do not fail in a shear mode if advantage is to be taken of ductile flexural yielding to substantially reduce the seismic design loads.

To comply with the capacity design requirements, the wall shear force calculated assuming the inverted triangular distribution of loads must be factored up to allow for both dynamic magnification and probable overstrength of the plastic hinge moment capacity. The overstrength factor (minimum 1.39) allows for the actual detailed reinforcement content (cf min. required), probable yield strengths of the reinforcement (cf code characteristic values) and strain hardening of the flexural reinforcement. The dynamic magnification factor required to be applied (up to 1.8 for buildings 15 storeys or higher) principally allows for lowering of the effective height of the dynamic load centroid due to higher mode effects.

The effect of higher modes on the moment/shear ratio at the base of an elastically responding wall is illustrated in Figure 3.1. The figure shows the two alternative ways that the dynamic loads corresponding to the first and second modes of a building's response can be combined. It can be seen that the level of the centroid of the dynamic load for the wall responding in its first mode only, h, is increased or reduced to h_1 or h_2 by the 2nd mode load.

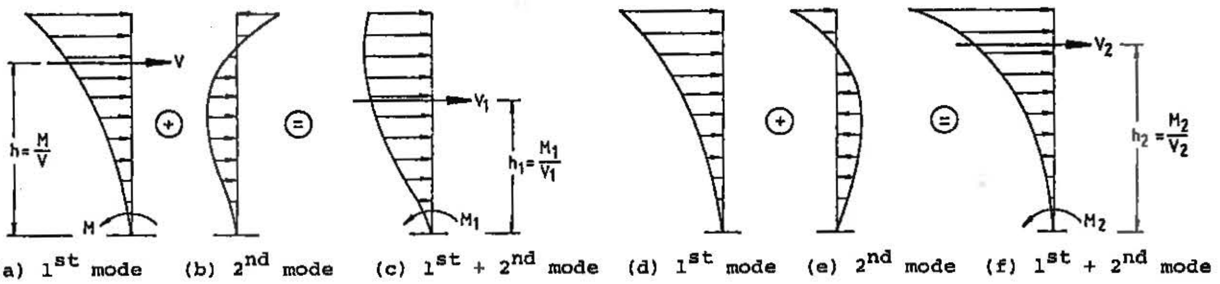


Fig. 3.1 Effect of higher modes on moment/shear ratio at base of wall

Most buildings have traditionally been designed for a triangular distribution of equivalent static loads which have a similar moment/shear ratio at the base of the wall as that given by the walls first mode response. The amplification of wall shears given by a triangular distribution of load by a dynamic magnification factor, ω, allows for the increase in shear/moment ratio that can be generated by higher modes. This higher shear/moment ratio effect corresponds to the low moment/shear ratio case indicated by Figure 3.1(c). By allowing for flexural overstrength at the base of the wall, ϕ_o, and dynamic magnification of shears, the capacity design procedure given in the NZ design code aims to ensure failure in a flexural rather than shear mode.

Most walls in buildings designed in New Zealand prior to 1976 do not meet these capacity design requirements and are therefore expected to fail, at least partially, in a shear mode.

In order to evaluate the inelastic shear displacement demand that could be imposed on such walls by earthquakes, two shear wall buildings designed and built in the late 50s and early 60s were selected for study by computer modelling and inelastic dynamic analysis.

Before starting the study it was postulated that the inelastic shear displacement demand generated by higher modes might be quite small. Table 3.1 indicates that, although the dynamic loads (accelerations) generated by the 2nd mode of an elastically responding wall are large relative to those generated by the 1st mode, displacements generated by the 2nd mode are relatively small. Therefore, if a wall had just sufficient shear and flexural strength to respond elastically in its first mode, the inelastic displacements generated by higher modes could be expected to be quite modest. In this case structural and non structural damage resulting from an earthquake would not be as great as would be expected from a comparison between a building shear strength and that required by current design requirements.

This is still thought to be true for walls with sufficient strength to respond elastically in their first mode. However, the study established that it is not true when walls have a strength level that results in significant inelastic response. In this case earthquake motions are capable of generating a large shear displacement demand in the wall if higher mode amplification of shear forces has not been allowed for in the design of the wall.

Table 3.1 : Relative 1st and 2nd Mode Accelerations and Displacements at Roof Level of Walls Given by Elastic Modal Analysis

Number Storeys of Wall	Acceleration (%g)				Displacements (mm)		
	1st Mode Period (sec)	1st Mode	2nd Mode	1st Mode / 2nd Mode	1st Mode	2nd Mode	1st Mode / 2nd Mode
12	.9	0.78	.644	1.21	156	4.1	38
24	3.41	0.23	.547	.42	658	42.6	15.4
30	5.28	0.145	.436	.33	1000	80	12.5

Notes:
1. Sample walls obtained from reference [14].
2. Modal analysis used the response spectrum given in DZ4203 for $\mu = R = Z = 1.0$ and normal soils [15].

3.2 FIRST SHEAR WALL BUILDING

3.2.1 Computer Model Used for Inelastic Dynamic Analysis

Figure 3.2(a) shows an elevation of the first shear wall building selected for computer modelling. Elements of the gravity load carrying system are not shown for clarity.

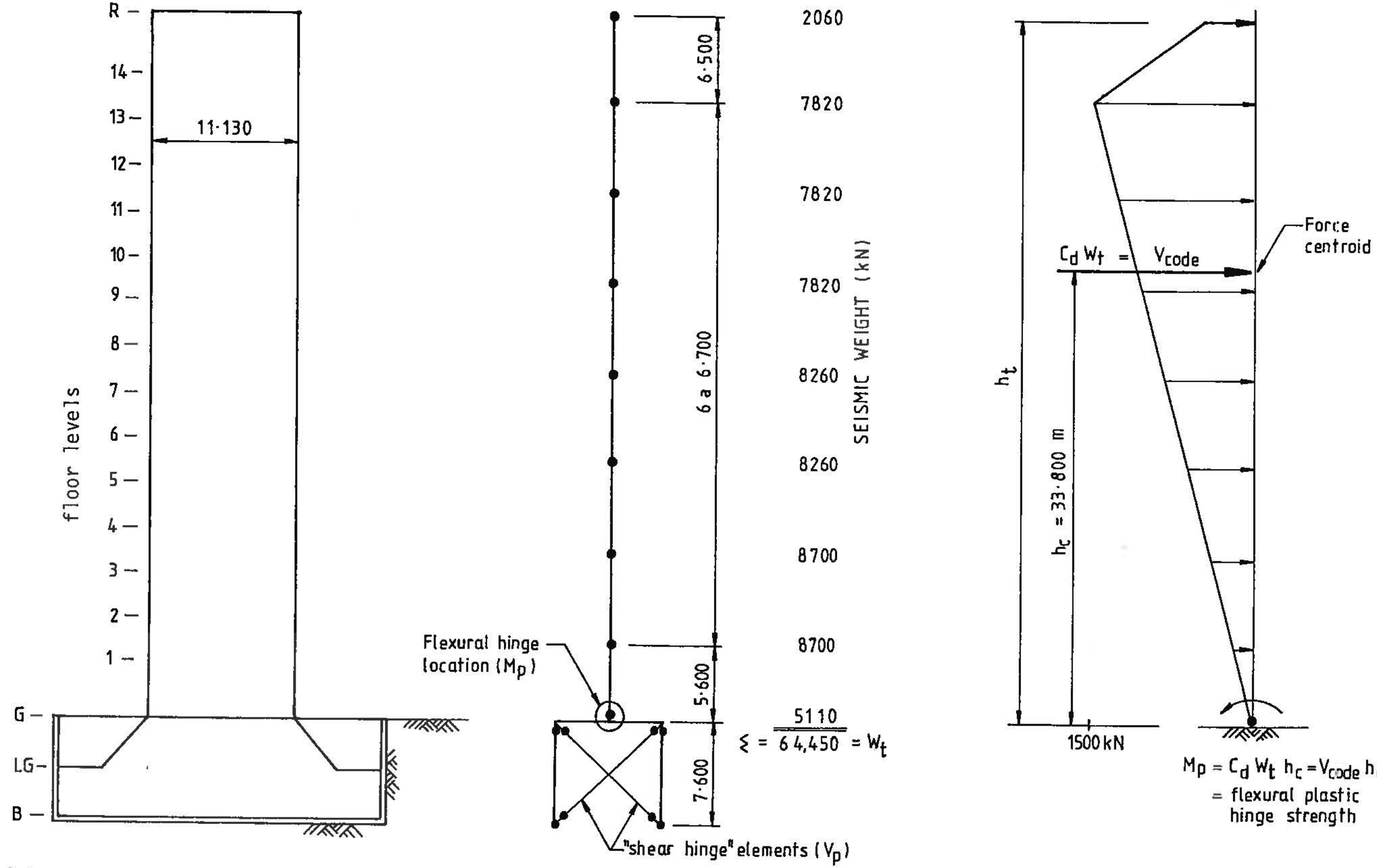
A lumped mass computer model for the wall is shown in Figure 3.2(b) and indicates the locations near the base of the wall where any flexural or shear plastic "hinging" is assumed to occur.

In the real structure shear and flexural plastic hinging is expected to occur between the ground and 1st floor levels. Positioning the shear plastic "hinge" below ground floor level instead of above it will only alter the dynamic forces generated in the ground floor lumped mass when significant shear yielding occurs. It is not, therefore, expected to have had a marked effect on the overall results of the inelastic dynamic analysis.

The principal variables chosen for study were the earthquake ground motion, flexural plastic hinge strength, M_p, and the ratio of shear plastic hinge strength, V_p, to flexural plastic hinge strength, M_p.

The first 10 seconds of three recorded earthquake motions where selected as the first principal variable to be used for the inelastic dynamic analysis of the wall. The El Centro N-S 1940 motion was selected because it has a similar spectral intensity to the elastic design spectra proposed for Wellington by the draft New Zealand loading code [15]. The other two earthquake motions, Pacoima Dam S16E 1971 and Imperial Valley (I.V.) College N230, 1979 were selected because they both have a long acceleration pulse. This type of damaging motion is a characteristic of the ground shaking recorded close to earthquake faults when the rupture of the fault propagates along the fault towards the observation site. Although a rupture of the Wellington fault locally could well produce stronger shaking in Wellington than that recorded at Pacoima Dam (1971) and I.V. College (1979), the motions were used without scaling. They may not, therefore, represent the maximum probable earthquake that can be expected in Wellington.

The second major variable examined was the strength of the flexural plastic hinge that was assumed to be located at ground floor level. Figure 3.2(c) shows the "triangular" distribution of load that was used for the original design of the building. The total lateral load, $V_{code} = C_d \times W_t$, has a centroid located h_c above ground floor level so that it generates a plastic hinge moment, $M_p = C_d\ W_t\ h_c = V_{code} h_c$ (notation is defined in the notes to Figure 3.2). Hence, the flexural plastic hinge strength, M_p, can be varied by varying the seismic design coefficient, C_d.



(a) Elevation of shear wall

(b) Computer model

(c) Code Distribution of Load

Fig. 3.2 : Model of wall able to plastic "hinge" in both flexure and shear

Notes on Fig. 3.2:

WALL PROPERTIES USED FOR COMPUTER MODELLING OF 1ST WALL.

- Elastic modulus = $25x10^3$ MPa

- Effective 2nd moment of inertia = $150m^4$
  (allows for cracking and varying wall thickness)

- Shear Area = $2.0m^2$
  (allows for cracking and varying wall thickness)

- Shear modules = $10x10^3$ MPa

- Flexural hinge strength :

$$M_p = C_d x\, W_c\, h_c$$

where : C_d = seismic design coefficient (varied)
W_c = total building seismic weight (including mass at ground floor level)
h_c = height to the centroid of the seismic load for code distribution of loading.

- "Shear Hinge" Properties

  The diagonal elements provide a shear yield strength, V_p, where : (except where noted otherwise)

  (a) 30% of V_p is provided by elastoplastic elements yielding in tension and compression.

  (b) 70% of V_p is provided by elements that buckle in compression and yield elastoplastically in tension.

  (c) The yield deflection at ground floor level is 4mm at first yield of the "shear hinge".

- Damping (except where noted otherwise)
  5% for period of vibration of .4 or 1.0 sec
  7.8% for period of vibration of .2 or 2.0 sec

- Initial fundamental period of vibration:- 1.0 secs (approximately)
  Initial 2nd mode period of vibration:- .2 secs (estimated)

In order to evaluate the influence of the ratio of the shear and moment plastic "hinge" strengths the variable $V_p h_c/M_p$ was selected as the third major variable. For the code distribution of load shown in Figure 3.2(c), simultaneous flexural and shear yielding will occur when $V_p h_c/M_p = 1.0$. If M_p is held constant and the shear plastic hinge strength, V_p, is increased (i.e. $V_p h_c/M_p > 1.0$), yielding in shear will only occur when the centroid of the dynamic load is lowered by higher modes as indicated in Figure 3.1(c). Similarly, if the shear plastic hinge strength, V_p, is reduced (i.e. $V_p h_c/M_p < 1.0$) yielding in flexure will only occur when the centroid of the dynamic loads is raised by higher modes as indicated in Figure 3.1(d).

This behaviour may be compared with the behaviour of a Single Degree of Freedom (SDOF) structure. In this case all the yielding would be in the shear mode when the shear/moment strength ratio $V_p h_c/M_p$ is less than 1.0 and all the yielding would be in the flexural mode when the ratio is more than 1.0.

Secondary variables examined were the level of viscous damping assumed for the dynamic analysis and the way in which the shear plastic hinge was modelled. Initially the diagonal members of the shear hinge (Figure 3.2(b)) were modelled as elastoplastic "truss" elements and the displacement at ground floor level at the initiation of yield, Δ_y, was varied with the yield strength of the shear hinge (i.e. $\Delta_y = 1.25 \times V_p h_c/M_p$ mm).

However most of the analyses were carried out using a model for the shear plastic hinge that had only 30% of the shear strength provided by elastoplastic elements and the remaining 70% provided by diagonal element that yielded elastoplastically in tension but buckled in compression. Therefore, the buckling elements behaved like yielding cross bracing rods in a frame and were intended to model the behaviour of yielding horizontal reinforcement in the wall.

Initially it was found that with buckling elements, a small and therefore uneconomic time step was required to produce stable results from the dynamic inelastic analysis computer program used (DRAIN 2D). However by increasing the yield displacement of the shear plastic hinge, Δ_y, to 4.0 mm the results become less sensitive to the time step selected and an economic time step of .01 seconds could be used to produce stable results. The 4 mm shear yield displacement was estimated by considering the likely average strains in the wall horizontal steel at onset of yield.

3.2.2 Results of Analysis of the First Wall : El Centro Earthquake Motion

Figure 3.3 shows the results of the initial analyses using only elastoplastic shear yielding elements in the shear plastic "hinge". To obtain the results the flexural plastic hinge strength, M_p, was held constant (i.e. $C_d = .1$) and the shear

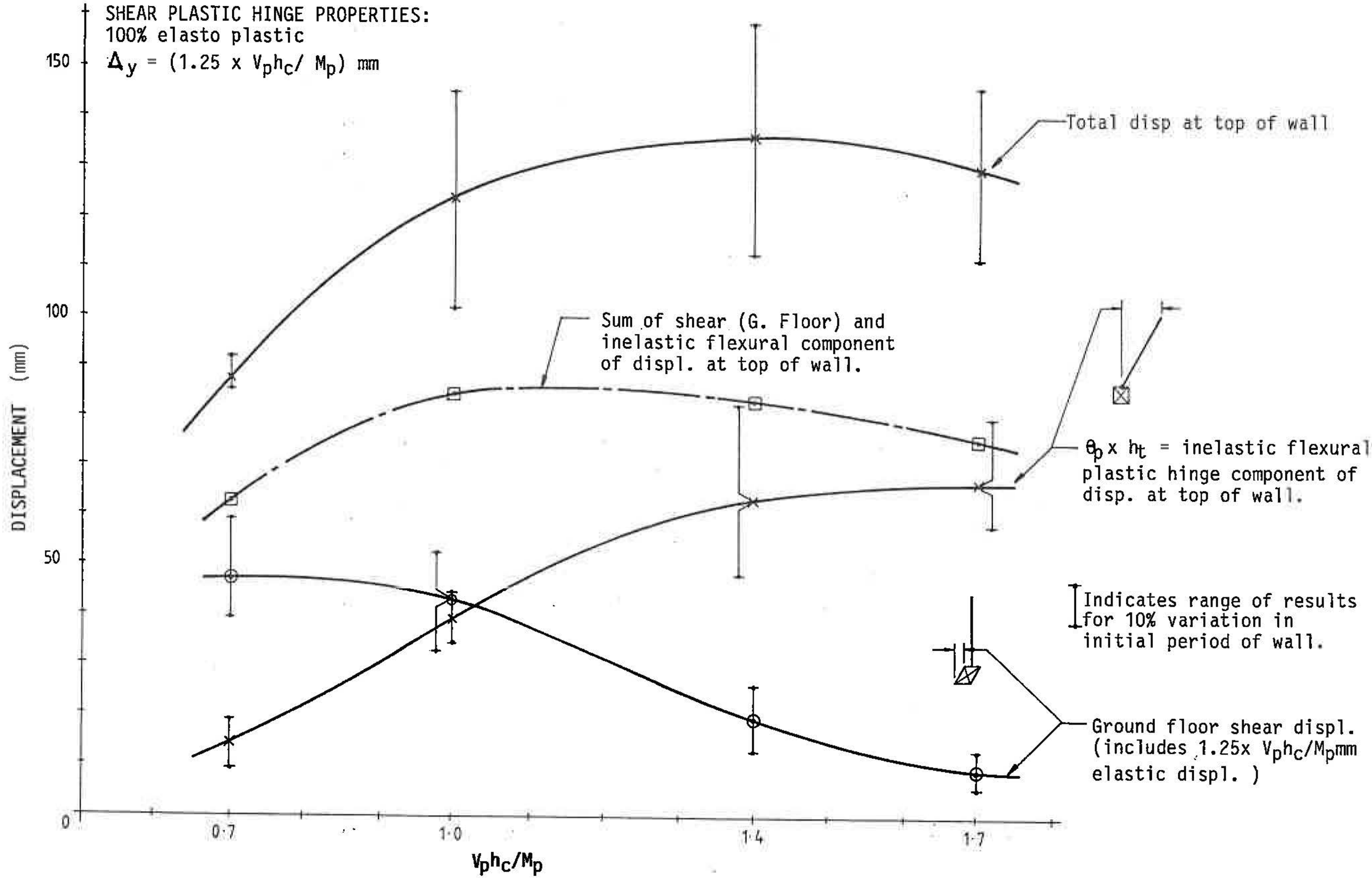


Fig 3.3. Peak Displacements of wall with a constant flexural Plastic Hinge Strength, M_p, computed for $C_d = .1$ and Variable Shear "Plastic Hinge" strength, V_p, responding to El Centro NS 1940 EQ motion with 5% damping.

plastic hinge strength, V_p, was varied so that the ratio $V_p h_c/M_p$ had a value of .7, 1.0, 1.4 or 1.7. Preliminary analysis established that the results were not sensitive to a small change in the ratio $V_p h_c/M_p$ but they were found to be sensitive to small changes in the initial elastic stiffness assumed for the wall.

Therefore, for each value of $V_p h_c/M_p$ considered, the analysis was repeated with the wall stiffness varied so that it's initial elastic period of vibration varied by $\pm$ 10%. The results plotted are the average values obtained from the three analyses. The three results that were averaged, were the peak displacement values and these did not necessarily occur at the same time during the earthquake record and, in some cases, did not even have the same sign. The range of the three results is also indicated in Figure 3.3.

The results from these analyses show that the distribution of inelastic deformation between the shear and flexural modes changes relatively "slowly" with changes in the ratio $V_p h_c/M_p$. When this ratio is less than about 0.7 almost all inelastic deformation is provided by shear yielding (i.e. 50 mm for these runs using El Centro NS 1940). The plotted results also indicate that significant inelastic shear displacements (e.g. 20 mm or more) could still be expected for ratios of $V_p h_c$ to M_p up to 1.5.

In these particular analyses the inelastic displacement demand was met approximately equally by shear and flexural yielding when $V_p h_c/M_p$ = 1.0, which is the case when shear and flexural plastic hinge strengths are proportioned according to the inverted triangular load distribution shown in Figure 3.2(c). In subsequent analyses, when the alternative shear hinge model with buckling elements was used, equal inelastic shear and flexural displacements tended to occur when the ratio $V_p h_c/M_p$ was greater than 1.0.

Figure 3.4 shows the time history of the moment and shear forces at the shear and flexural plastic hinge locations for the computer analysis of the wall when the ratio $V_p h_c/M_p$ = 1.0. The value of the shear force is factored by h_c so that the moment and shear plots would have been identical if the distribution of the dynamic load retained the same inverted triangular shape assumed in design and shown in Figure 3.2(c).

The plots indicate that the shear is more strongly influenced by the higher modes than the moment and that the higher modes tend to cause the moment and shear plots to be out of phase. Consequently, there is very little simultaneous yielding in both shear and flexure as can be seen by examining the flattened peaks of the plots.

This suggests that the higher modes act like a randomly fluctuating gate that distributes the total inelastic displacement demand between shear and flexural yielding just like a blind man drafting sheep. This explains why the results are

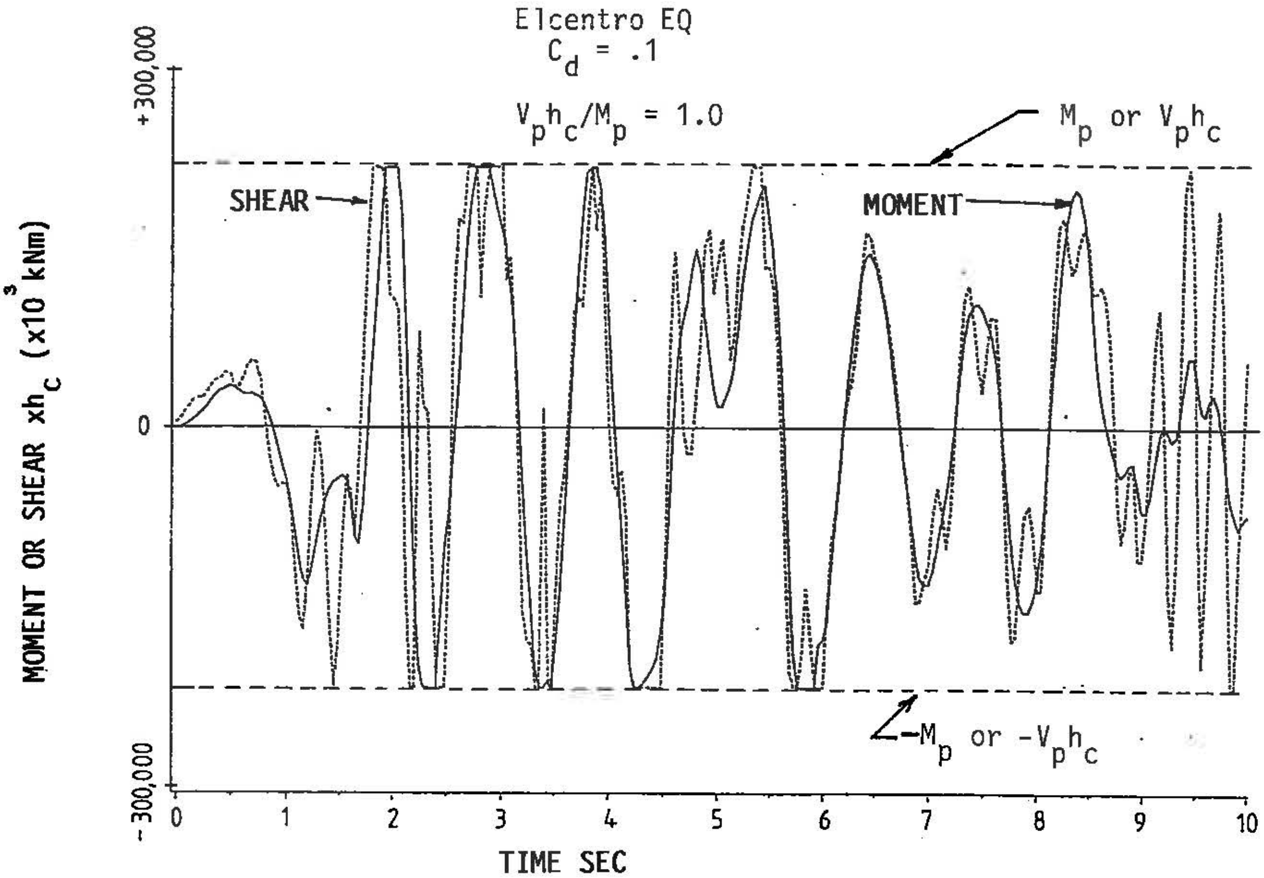


Fig. 3.4 **Time History of moment and shear in flexural and shear Plastic Hinges**

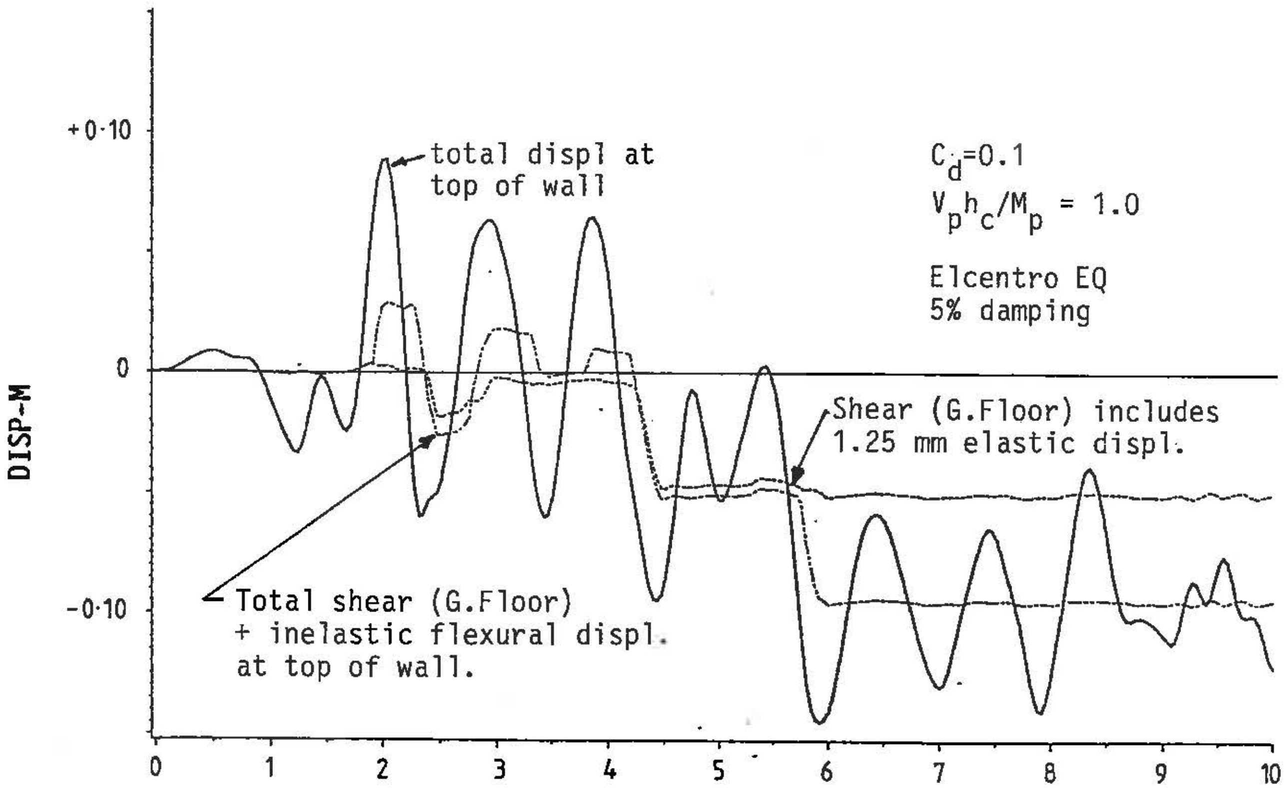


Fig. 3.5 **Displacement Time History at top of wall and components due to shear and flexural yielding.**

sensitive to small changes in the natural period of the wall. When the strength of the shear hinge is increased ($V_p h_c/M_p > 1.0$) the gate is given a bias and more of the inelastic demand is allocated to flexural yield although the total inelastic demand remains relatively constant (see 2nd top curve in Figure 3.3).

Figure 3.5 shows the displacement time history at the top of the wall and its components due to shear and flexural yielding. The shear displacement at ground floor level is almost entirely inelastic as the yield displacement of the shear hinge is only 1.25 mm for this plot. The inelastic flexural displacement at the top of the wall was computed by multiplying the plastic hinge rotation at ground floor level, θ_p, by the height of the wall, h_t, (Figure 3.2(c)). The difference between the combined shear and inelastic flexural displacement curve and the curve for the total displacement at the top of the wall is a measure of the elastic displacement of the wall. However because peak shear, flexural and elastic displacements do not necessarily take place at the same time, it is not equal to the elastic displacement.

It can be seen from Figure 3.5 that most of the shear yielding takes place between 4.0 and 5.0 seconds from the start of the earthquake record and most of the flexural yielding takes place just before six seconds. It can also be seen that this yielding corresponds to the long flattened peaks in Figure 3.4.

It is important to note that the inelastic shear displacement is not directly caused by higher modes. The period of the motion shown in Figure 3.4 indicates that the inelastic yielding is principally the result of the first mode response with the higher modes acting principally as a "gating" mechanism to allocate the inelastic demand between the shear and flexural yielding options.

The effect of changing the model used for the shear plastic hinge, so that 70% of the strength was provided by buckling elements, can be seen by comparing Figures 3.3 and 3.6. To make the comparison easier, the shear displacement plot at ground floor level in Figure 3.3 is also shown dotted in Figure 3.6. Only a small part of the difference between the shear displacement plots can be explained by the increase in elastic yield displacement of the shear plastic hinge to 4.0 mm. Note that the analyses using the model with buckling elements predicts that shear displacements up to 25 mm may still occur even with the ratio $V_p h_c/M_p$ as high as 1.7.

The two sets of curves in Figure 3.3 and 3.6 have a similar form for $V_p h_c/M_p > 1.0$ but have a quite different form for $V_p h_c/M_p < 1.0$. The difference in form between the two sets of curves is not thought to be due to the change in shear hinge modelling alone as all the sets of curves plotted during the study had one of these two characteristic forms.

A close examination of all the time history plots produced in the study for $V_p h_c/M_p = .7$, like those shown in Figures 3.4 and 3.5, failed to find a consistent explanation for the two types of behaviour.

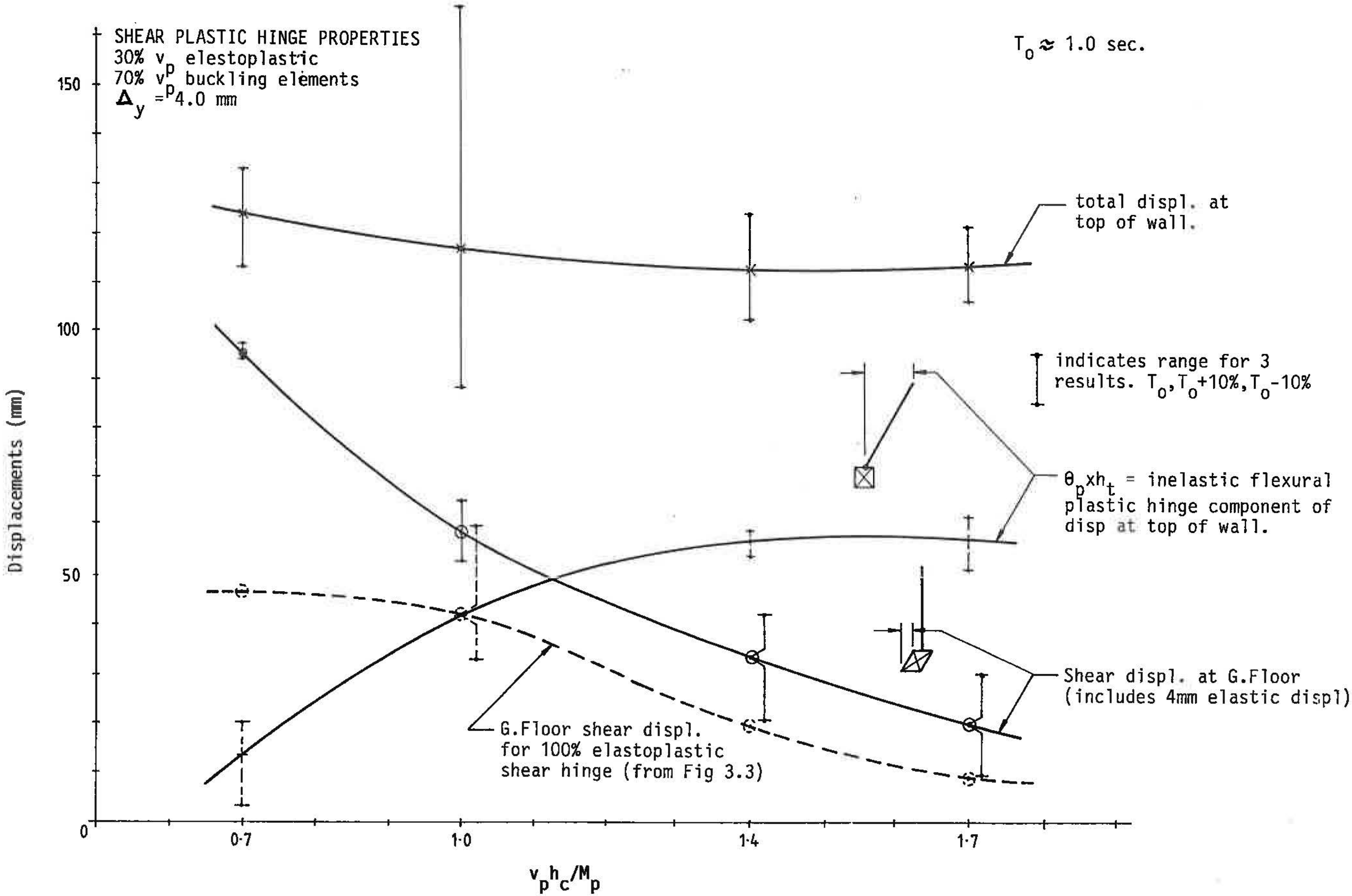


Fig 3.6 Peak displacements of wall with a Constant Flexural Plastic Hinge Strength, M_p Computed for C_d = .1 and Variable Shear Hinge Strength, v_p, Responding to Elcentro N.S. 1940 EQ motion with 5% damping

However, in most cases, it was noted that when the shear displacement was relatively large, so that the curves had a form like that show in Figure 3.6, the elastic deflection of the wall tended to be small at the end of the shear yielding episode that produced the maximum shear displacement. The behaviour is consistent with the elastic energy stored in the wall being converted into shear displacement. As can be seen from Figure 3.6, the inclusion of pinching into the shear displacement model, but not in the flexural model, has preferentially increased the inelastic shear displacements. As a result the inelastic shear displacements exceed the inelastic flexural displacements up to a value of $V_p h_c / M_p$ of approximately 1.1. This bias towards inelastic shear displacements was noted in all subsequent analyses in which the "pinched" shear model was used.

The effect of doubling the flexural and shear plastic hinge strengths of the wall (i.e. increasing C_d to .2) can be seen by comparing Figures 3.6 and 3.7. Again, to make it easier to compare the sets of curves, the ground floor shear displacement plot from Figure 3.6 is reproduced in Figure 3.7. As the elastic stiffness of the shear hinge was not increased when the strength was doubled the yield displacement, Δ_y, was also doubled to 8.0 mm. After producing Figure 3.7 it was decided to standardise Δ_y at 4.0 mm for the remainder of the study.

As expected doubling the wall strength can be seen to significantly reduce the inelastic displacement demand for both shear and flexural yielding.

3.2.3 Results for Pacoima Dam and I.V. College Earthquake Records

The results of the inelastic dynamic analysis using the Pacoima S16E earthquake record are shown in Figure 3.8.

The form of the results is similar to that shown in Figure 3.6 for the El Centro earthquake record but the change in scale of the vertical (displacement) axis should be noted. The relatively high displacement ductility demand induced by the Pacoima record can be seen by comparing the combined flexural and shear yield displacement with the total displacement at the top of the wall.

As all but 4 mm of the ground floor shear displacement is inelastic, the difference between the upper two curves shown in Figure 3.8 gives an approximate measure of the walls elastic displacement.

Figure 3.8 also shows the results obtained by repeating the inelastic analysis of the wall using 13% damping (at the initial period of one second). The results of the analysis were scaled up by a factor of 1.33 before plotting. It can be seen that the increased damping only changes the magnitude of the displacements and does not change the form of the curves significantly.

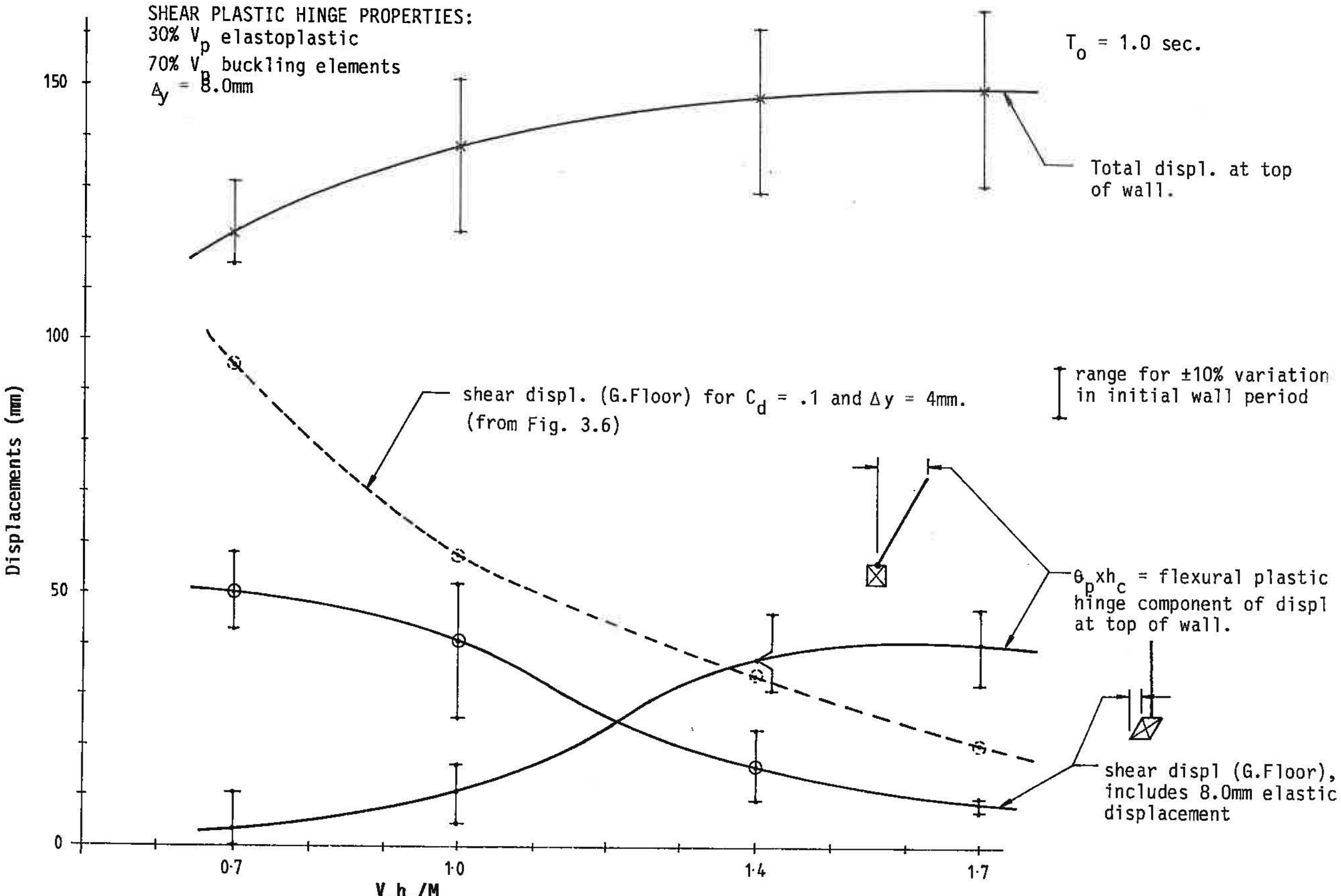


Fig. 3.7 Peak Displacements of wall with a Constant Flexural Plastic Hinge strength, M_p computed for C_d = .2 and Variable Shear Plastic Hinge Strength, V_p, Responding to Elcentro N.S.1940 EQ motion with 5% damping.

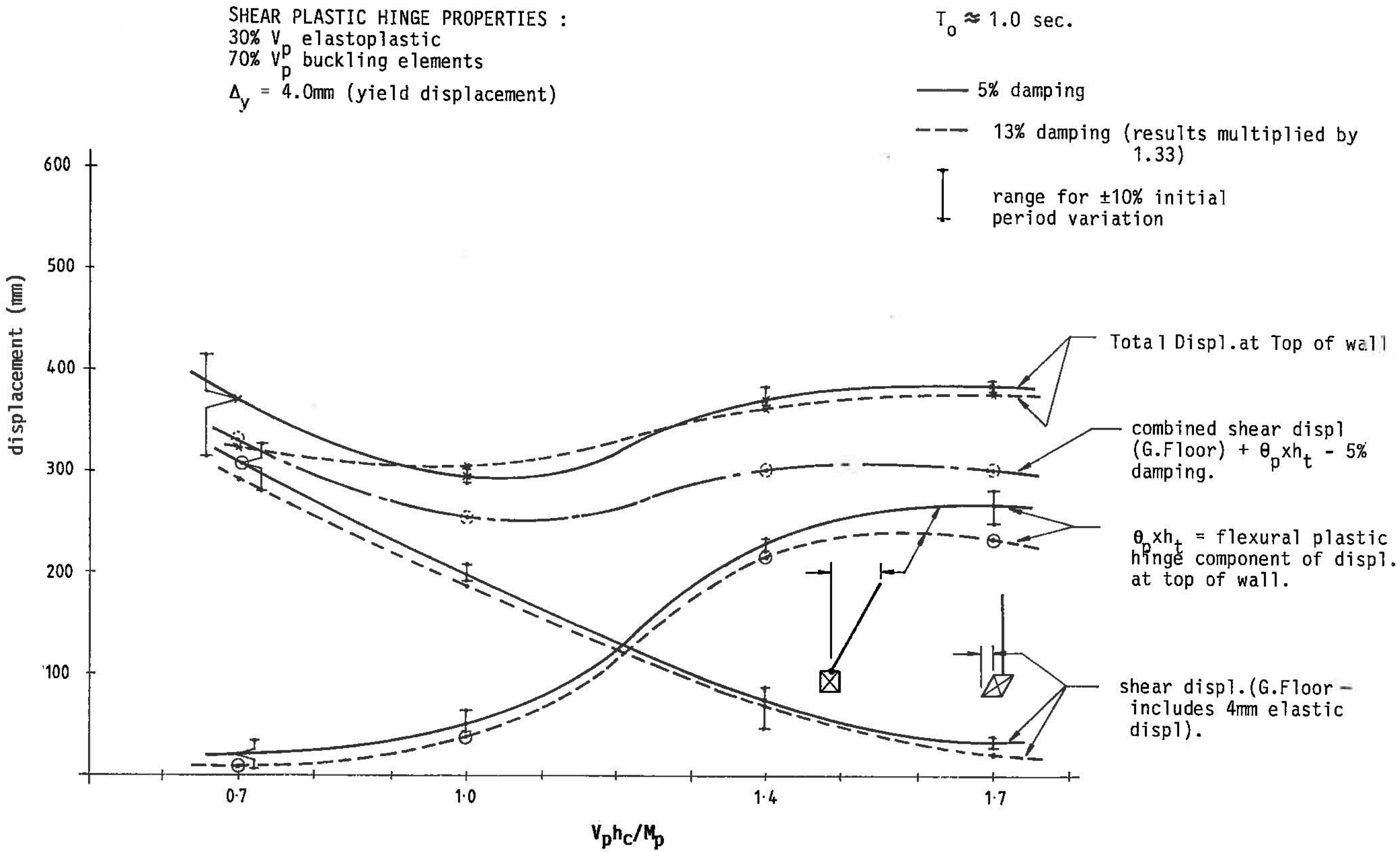


Fig.3.8 Peak Displacements for wall with Constant Flexural Plastic Hinge Strength ,M_p, computed for Cd = .2 and Responding to Pacioma S16E EQ Record.

The analysis with 13% damping was also repeated with the yield displacement of the shear plastic hinge, Δ_y, increased from 4 to 8 mm. It was found that the results only changed marginally with, for example, the ground floor shear displacement increasing by 1.0 and 6.0 mm for $V_p h_c/M_p$ values of 1.4 and 1.7 respectively.

The results obtained when the shear and flexural plastic hinge strengths were doubled (i.e. C_d increased to .4) and the Pacoima S16E earthquake record was used in the analysis are shown in Figure 3.9. The effect of doubling the wall strength when using the Pacoima record is similar to that previously described for the El Centro earthquake record. This can be seen by comparing Figure 3.6 with 3.7 and then comparing Figure 3.8 with 3.9.

The results obtained when using the I.V. College earthquake record for the inelastic analysis of the wall are shown in Figure 3.10. The results are similar to those obtained from the analysis using the Pacoima S16E record for the same wall shear and flexural plastic hinge strength (see Figure 3.8).

However a comparison of Figures 3.8 and 3.10 indicates that the fall off in shear yield displacement with increasing shear plastic hinge strength, V_p, is more rapid for the I.V. College earthquake motion.

3.2.4 Shear Yielding Response of Wall

To examine the shear yielding behaviour of the wall in detail, the wall's response to the Pacoima S16E earthquake motion was selected for further study. In particular the wall analysis with a flexural plastic hinge strength, M_p, computed using C_d = .2 and with a shear to moment plastic hinge strength ratio, $V_p h_c/M_p$, of 0.7 was chosen for detailed examination.

The displacement time history of the wall for the analysis is presented in Figure 3.11 and the time history of the bending moment and shear force in the walls plastic hinge zones is shown in Figure 3.12. Most of the walls shear displacement in the positive direction can be seen to occur between 2.74 and 3.18 seconds after the start of the earthquake record. This period during the motion corresponds to part of the long acceleration pulse (i.e. "near fault fling") shown plotted in Figure 3.13. It is interesting to note that the peak ground acceleration during this part of the motion is only 0.548g. This is less than half the peak ground acceleration of 1.17g that occurs at 7.72 seconds from the start of the motion. However this high acceleration only lasts for a very short period (i.e. is a spike) so that its effect on the building response is hardly discernible in Figures 3.11 and 3.12.

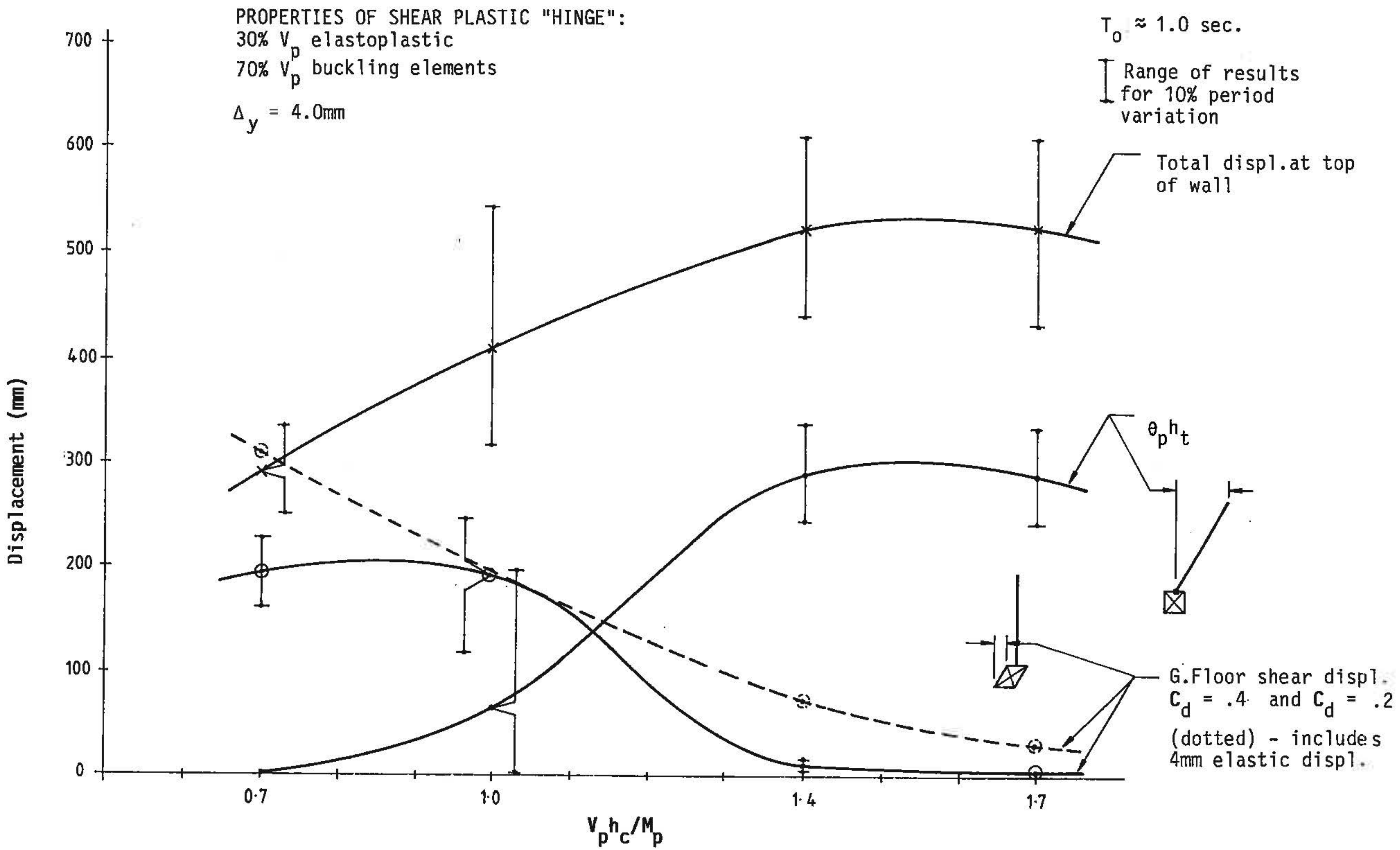


Fig. 3.9 Peak Displacements for Wall with constant Flexural Plastic Hinge Strength Computed for C_d = .4 and Responding to Pacoima S16E EQ Record with 5% damping

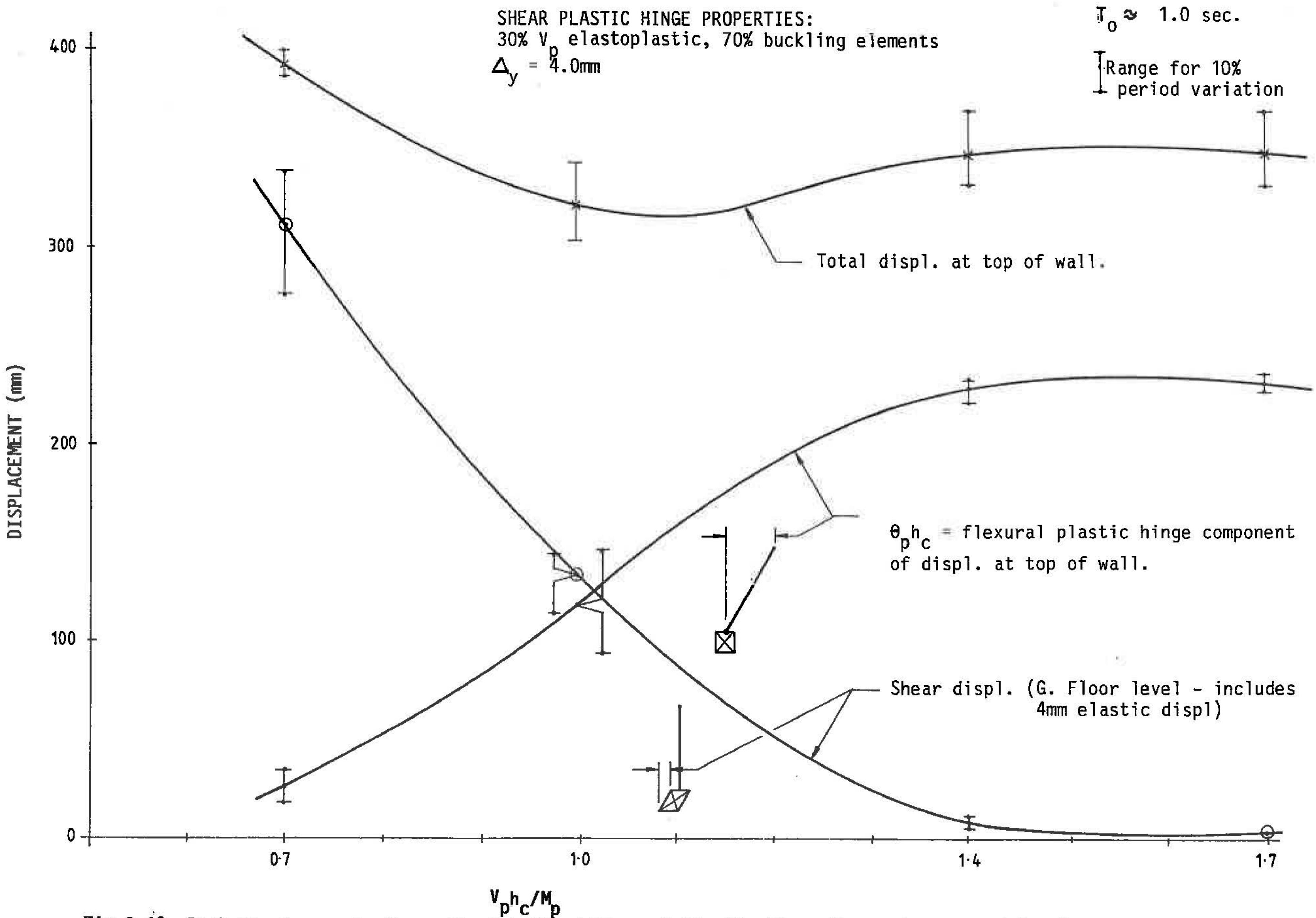


Fig.3.10 Peak Displacements for wall with Fixed Flexural Plastic Hinge Strength computed for C_d = .2 and responding to I.V. College EQ motion with 5% damping.

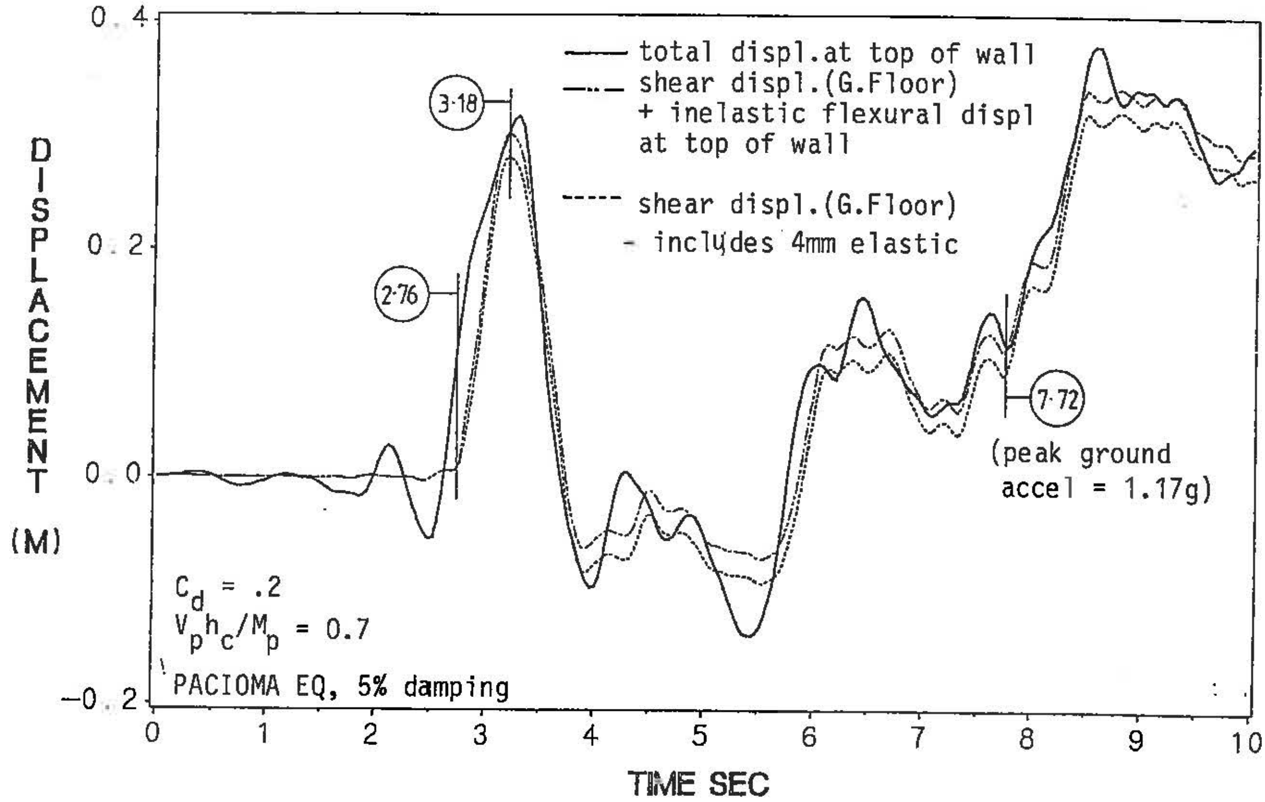


Fig. 3.11 Displacement Time History at Top of Wall and Components Due to Shear and Flexural Yielding

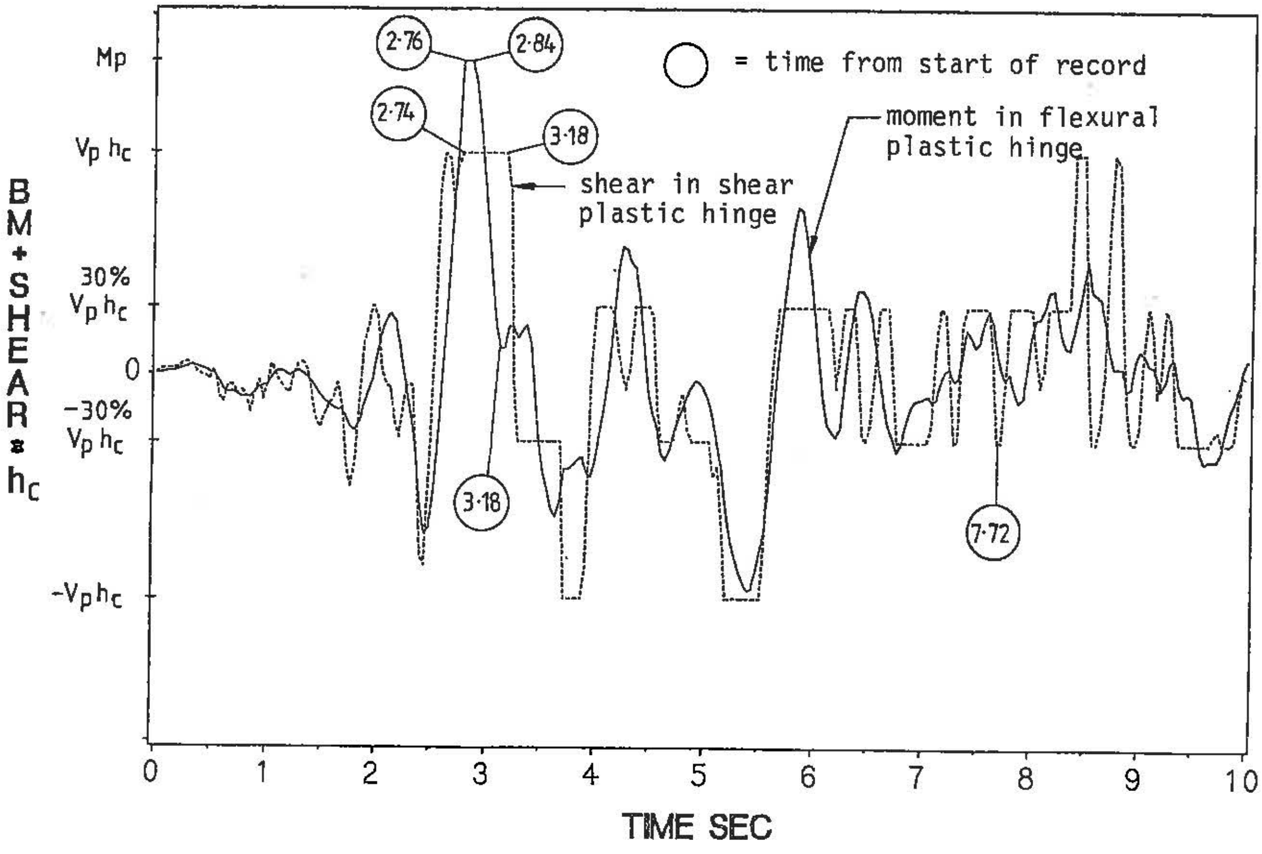


Fig. 3.12 Time History of Moment of shear in Flexural and shear Plastic Hinges (same analysis as Fig.3.11)

"Snap shots" of the walls response during the shear yielding period between 2.76 and 3.18 seconds are shown in Figure 3.14. The first snap shot is at 2.76 seconds. As can be seen from Figures 3.12 and 3.14(a) this is just after the start of yielding in shear which occurs at 2.74 seconds and is at the onset of flexural yielding.

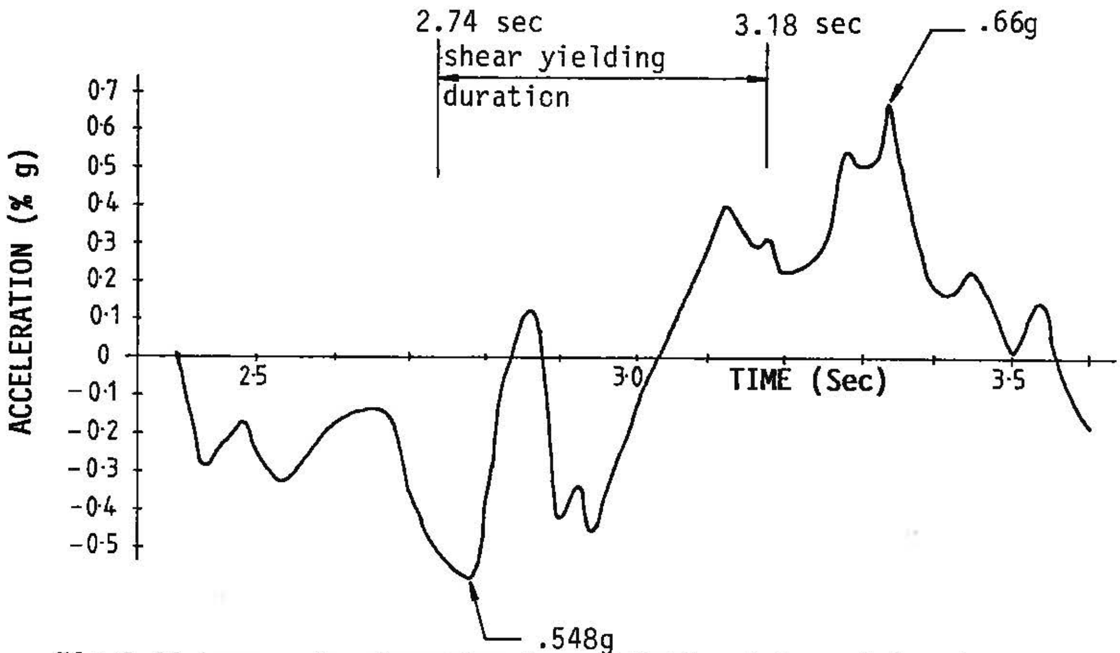


Fig.3.13 Long Acceleration Ground Motion Pulse of Pacoima S16E EQ Record.

Figure 3.14(d) shows the dynamic loads acting on the wall. These were derived by dividing the difference between adjacent interstorey shears (i.e. dynamic forces) by the seismic weight assumed to act at each floor. They are therefore expressed in terms of acceleration units. At ground level the wall acceleration is the same as the ground acceleration. The value plotted in Figure 3.14(d) was obtained from Figure 3.13.

Shear yielding commenced at 2.76 seconds and the displacement, shear, bending moment and acceleration profiles over the height of the wall at this time are shown in Figure 3.14.

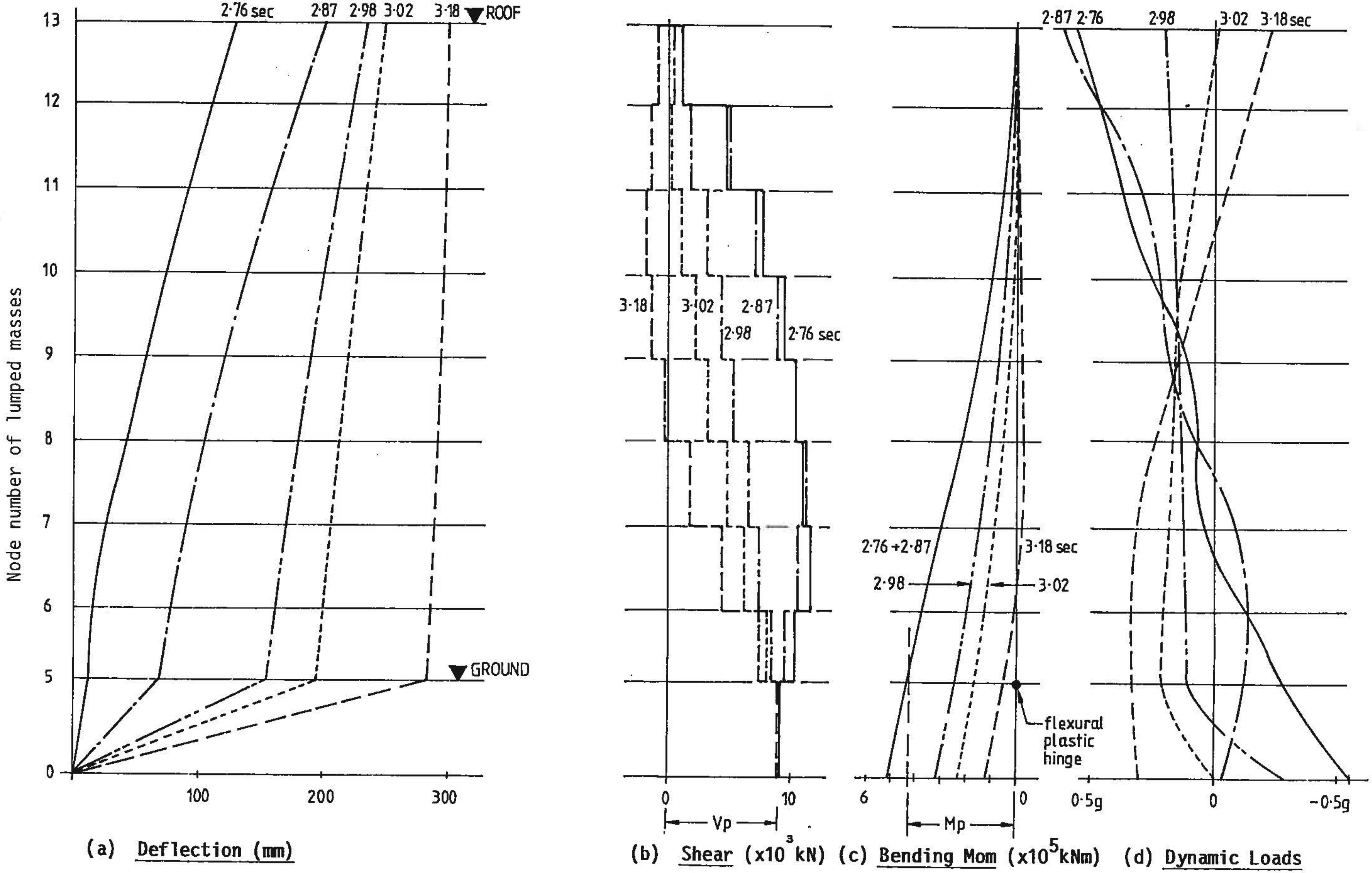


Fig 3.14 Shear yielding response of Wall to Pacoima S16E Pulse Between 2.76 secs and 3.18 secs - 5% damping

At 2.87 seconds the flexural yielding has just finished (flexural yielding stopped at 2.84 seconds - See Figure 3.12) but the bending moment at the plastic hinge has not fallen significantly below yield (see Figure 3.14(c)). During the time interval between 2.76 to 2.87 seconds the shear yield displacement is 59 mm while the displacement at the top of the wall generated by flexural yielding was 21 mm. During this time interval the dynamic loads do not change significantly except at the base of the wall where the ground acceleration falls to nearly zero (see Figure 3.14(d)). Consequently there is very little change in the bending moment and shear force distribution over the height of the wall as indicated by Figures 3.14(c) and (d). It is interesting to note that the shear force remains above the base shear yield force, V_p, over most of the wall height during this time interval. In the real wall where shear yielding is not confined to the base of the wall and shear strength declines over the height of the wall, due to a fall off in axial load and shear reinforcement, shear yielding would be expected in the upper parts of the wall. This would probably reduce the shear yielding displacement demand at the base of the wall.

It is also interesting to note that the peak acceleration reached at the top of the wall is 0.6g. This is three times the lateral load coefficient, C_d = .2, required to cause flexural yielding and over four times the coefficient C_d = .14 required to cause shear yielding for a triangular code distribution of load. In other analyses, ratios of peak acceleration to C_d (x g) up to 9 were noted at the top of the walls for both shear and flexurally yielding walls. The NZ Loadings Code [16] assumes this ratio is approximately two when computing seismic loads for parts and portions located at the top of buildings.

As noted previously, shear yielding continued for approximately 0.4 seconds. A wall with a 1.0 second first mode period can be expected to have an elastic 2nd mode period of approximately 0.2 seconds. If the postulated mechanism of higher modes allocating inelastic demand between shear and flexural yielding was to hold during the .4 second yielding period, the flexural hinge bending moment would be expected to reach two peaks during the .4 seconds. However, Figures 3.12 and 3.14(c) indicate that the flexural moment at the plastic hinge location declined throughout the shear yielding period.

The reason for the almost uniform decline in the wall moment is that the long period of shear yielding isolated the wall above ground floor level. As a result there was no excitation of the higher modes. This behaviour can be seen from the acceleration profiles shown in Figure 3.14(d). At the start of shear yielding (T = 2.76 sec), the effect of the higher modes can be clearly seen in the wall acceleration profile. This is still apparent at T = 2.87 sec, but from 2.98 sec to 3.18 sec the slope of the acceleration profile up the wall is almost constant above ground level indicating that the higher mode part of the response has largely been damped out. The ratio of $V_p h_c$ to M_p for this analysis was only 0.7. The inelastic demand would therefore be accommodated by shear rather than flexural yielding, even for a purely first mode response.

Another aspect of the behaviour apparent from Figure 3.14(d) is that the ground acceleration itself can significantly influenced the shape of the acceleration profile and hence the height of the shear centroid, particularly during long acceleration pulses such as illustrated in this example.

Clearly the roll of higher modes in the yielding mechanism, postulated earlier, is a useful but over simplified explanation of the walls yielding behaviour.

At the end of the shear yielding period (3.18 seconds) the walls elastic deflection between ground and roof level was only 1.0 mm. The visual impression given by Figure 3.14(a) is that the energy stored elastically in the wall at the start of shear yielding (at 2.76 seconds) has been converted to shear yielding distortion by the end of the shear yielding period (at 3.18 seconds). It was noticed that for $V_p h_c / M_p = .7$ relatively large shear yielding and total displacements at the top of the wall occurred when the bending moment in the flexural plastic hinge fell to near zero at the end of the principal shear yielding period.

Conversely, when the bending moment rose during the principal shear yielding period the shear yielding displacement tended to be relatively smaller. This may explain the two distinct forms that the sets of curves in Figures 3.6 to 3.10 exhibit.

After the initial long acceleration pulse the time history response shown in Figure 3.12 indicates that the pinched shear displacement historesis loops had the effect of limiting the forces developed in the wall. It is also noticeable that once the "initial slackness" (i.e. pinching) in the shear response had developed, all subsequent inelastic deformations occurred by shear "yielding". Even after the slackness had been taken up, the moments at the base of the wall did not even reach 0.7 times M_p. This means that even if the flexural strength had been equivalent to the shear capacity (i.e. $V_p h_c = M_p$), there would not have been any further flexural yielding.

3.3 SECOND SHEAR WALL BUILDING

3.3.1 Building Characteristics and Computer Modelling

The second building selected for study was constructed in reinforced concrete and has its seismic lateral resistance provided by two external shear walls located symmetrically about the building's centre of mass. The building is eight storeys high and was constructed in the late 1960s.

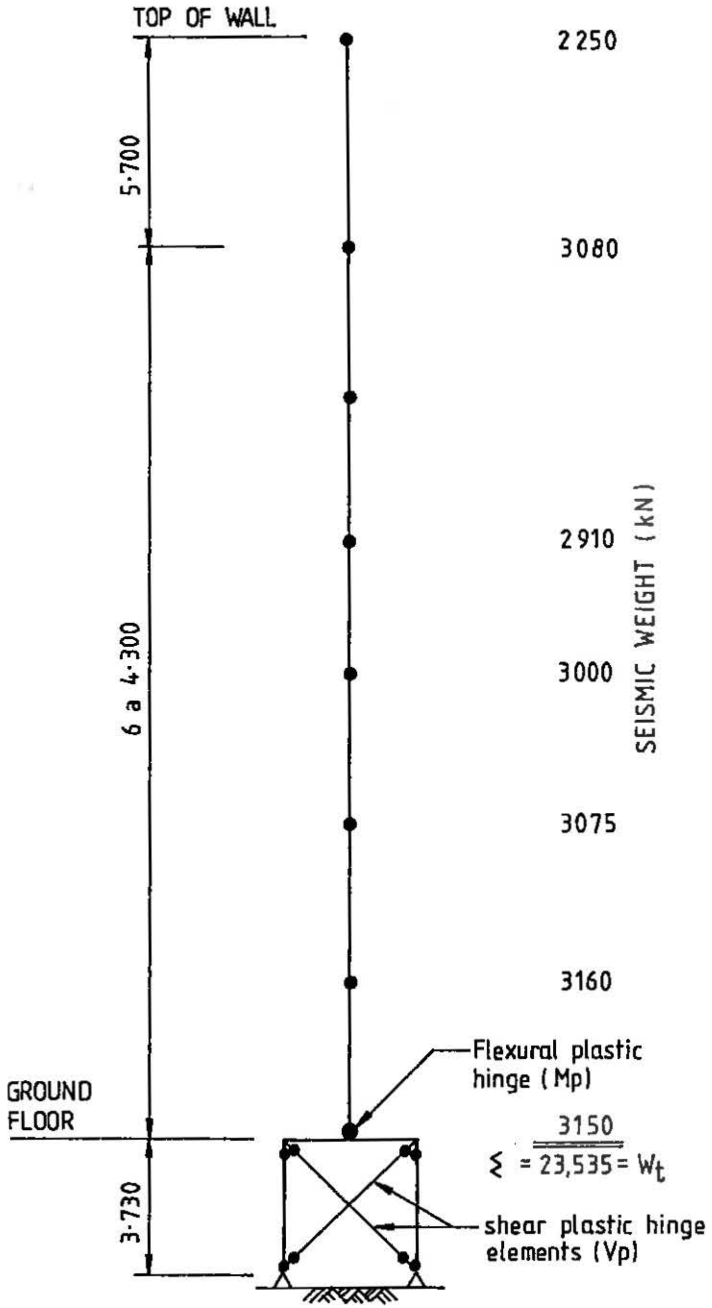


Fig. 3.15 Computer Model for Second Wall Selected for Study.

WALL PROPERTIES

. Effective 2nd moment of inertia = $65m^4$

. Damping - 5% for period of vibration of .2 and .5 secs.

. Initial fundamental period of vibration = .5 secs approximately.

. h_c = 21.27m (height of centroid of seismic load above G.Floor level for code triangular distribution)

Note: Other wall properties are as given for the 1st wall selected for study (see notes to Fig. 3.2) except that this model has one less lumped mass).

The computer model used for the inelastic analysis of the building's walls is shown in Figure 3.15 and, except as noted, the model is similar to that used for the first building selected for study. The wall's initial elastic period was estimated by assuming the wall's mass was uniformly distributed and treating the wall as a uniform cantilever beam. After making an allowance for elastic shear displacements the period was estimated to be 0.5 seconds.

This is half the 1.0 second period that was estimated for the first building using the same method.

As the hinge zone is modelled with a high flexural stiffness the method used to estimate the elastic periods of the walls will result in a small over estimate.

The reduced initial elastic period of the second wall is the most important difference between the two walls selected for study.

3.3.2 Results of Analysis

The results of the inelastic dynamic analysis of the second wall is shown in Figures 3.16 and 3.17 for the Pacoima and El Centro earthquake motions respectively. In each plot the results for two flexural plastic hinge strengths, M_p, corresponding to C_d = .2 and .4 are shown.

The form of the sets of curves for each C_d value are similar to that obtained for the wall with an initial period T_o, of 1.0 seconds. The exception is the part of the curve in Figure 3.17 that corresponds to the analysis results obtained when C_d = .2 and when the shear/moment plastic hinge strength ratio, $V_p h_c/M_p$, is equal to 1.4.

The results of this analysis indicate that the peak displacement at the top of the wall is less than the peak shear displacement at ground floor level. This, somewhat anomalous result, comes about because at the time of peak shear displacement in the shear plastic hinge, the displacement component at the top of the wall due to flexural plastic hinging had the opposite sign to the inelastic shear displacement. If this result is ignored (i.e. curves shown dotted are assumed) the form of the set of curves is similar to that obtained for all the other analysis but they appear to be shifted to the right. This has necessitated an extra analysis to obtain results corresponding to a ratio of $V_p h_c/M_p$ = 2.0 so that the now familiar form of curves can be completed.

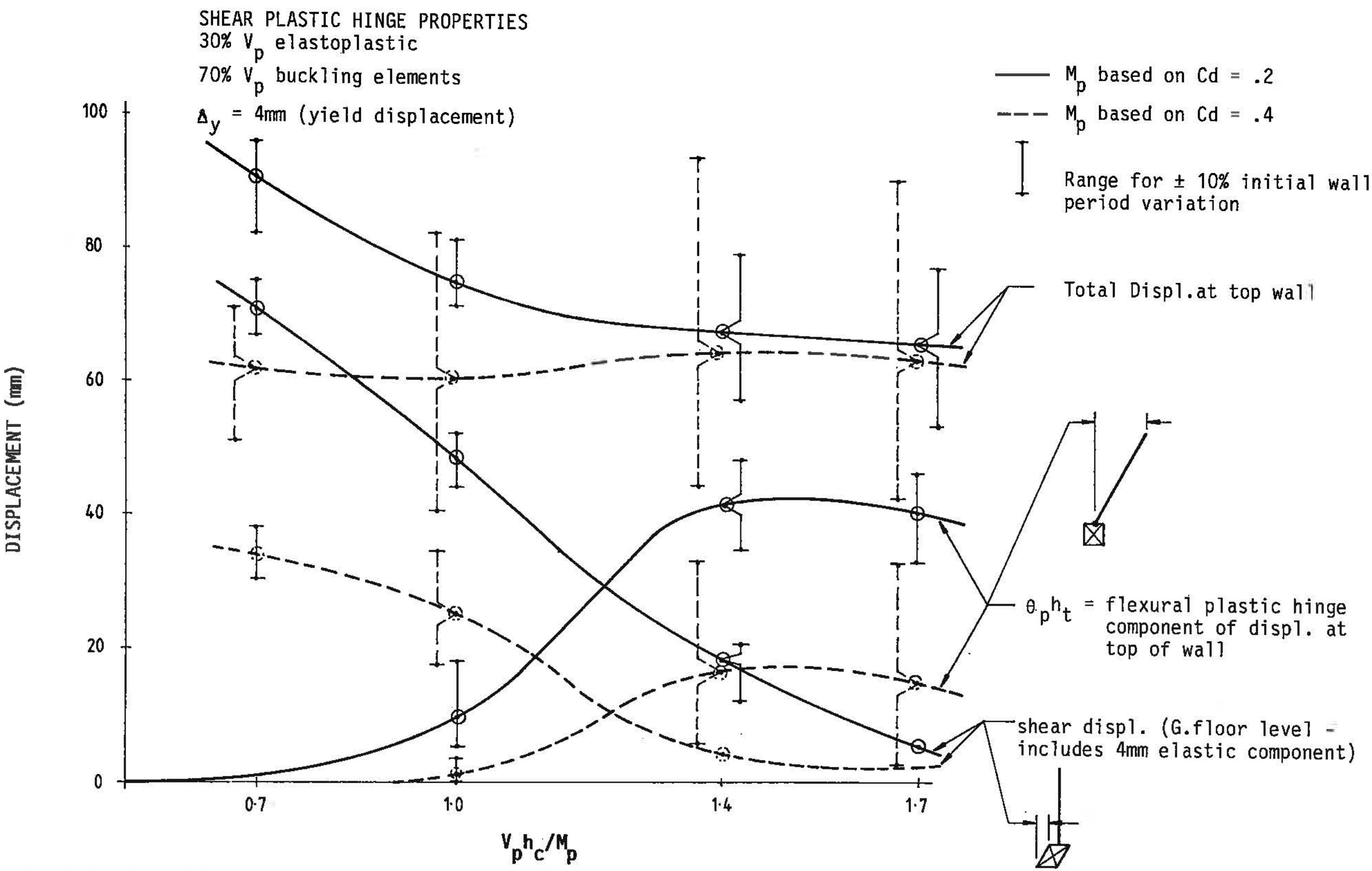


Fig. 3.16 Peak Displacements of wall with initial period T_o = 0.5 secs and Flexural Plastic Hinge Strength, M_p, computed for C_d = .2 or .4 . Wall Responding to ELCENTRO NS 1940 EQ Motion with 5% damping. Strength of Shear Plastic Hinge Strength, V_p, varied.

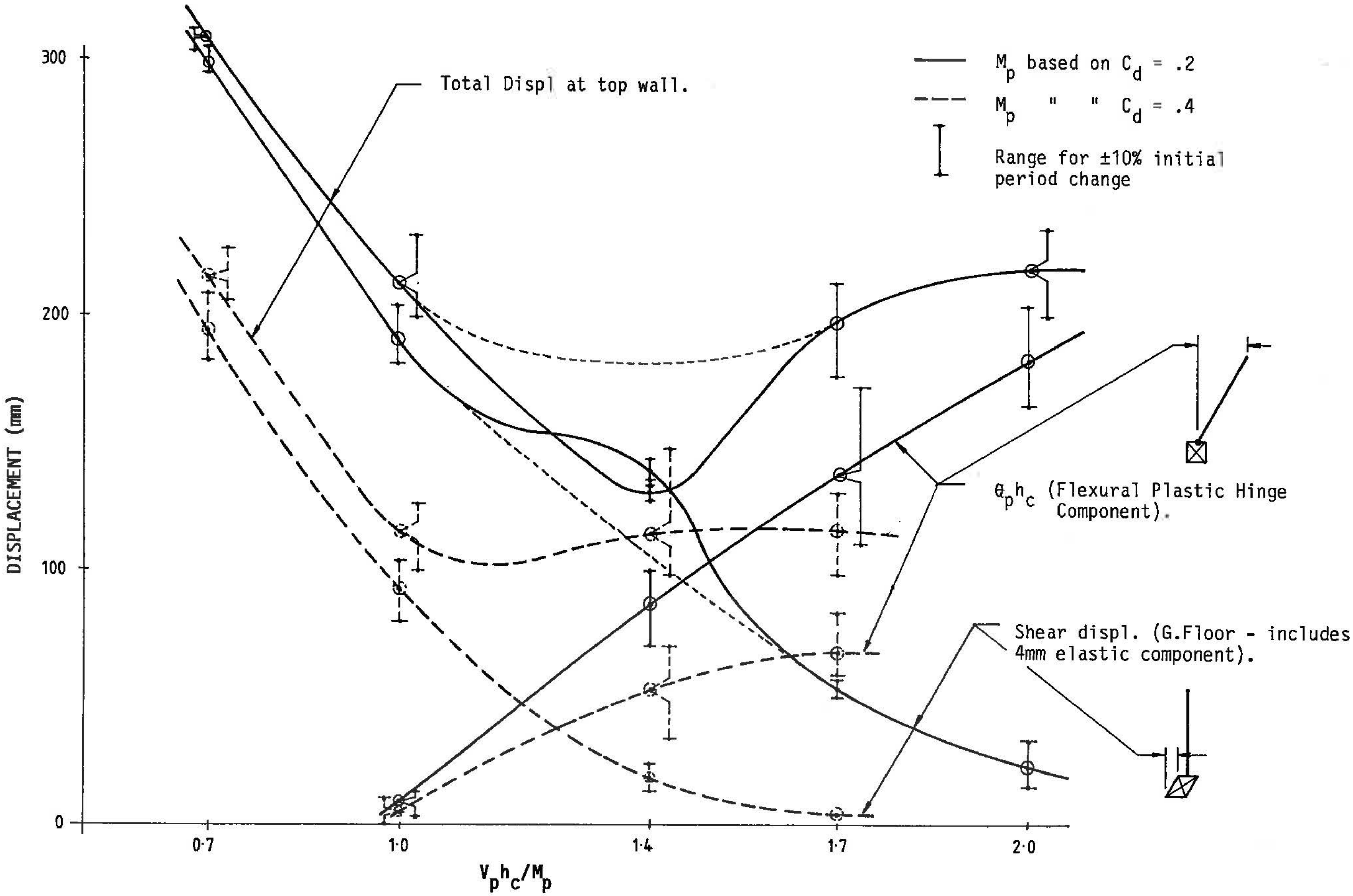


Fig. 3.17 Peak Displacements of wall with Initial Period 0.5 secs and Flexural Plastic Hinge Strength, M_p, computed for C_d = .2 or .4 and Responding to PACOIMA S16E EQ motion with 5% damping.

3.4 PREDICTABILITY OF FIRST AND SECOND SHEAR WALL RESULTS

3.4.1 Effect of Initial Elastic Period on Displacement Demand

The results of the analysis of the two walls with initial periods of 0.5 and 1.0 seconds, that were previously shown in Figures 3.7 and 3.16, are reproduced in Figure 3.18 to a common scale to facilitate comparison. The plots are for the El Centro earthquake motion using a flexural plastic hinge strength M_p, corresponding to C_d = .2.

It can be seen that the principal difference between the two sets of curves is the smaller total displacement at the top of the wall for the wall with a 0.5 second initial period. This is mainly due to the walls reduced elastic displacement as its shorter period is a consequence of it being much stiffer. It can be seen that the inelastic shear displacement demand is generally larger for the wall with the shorter initial period and the flexural plastic hinge component of the displacement at the top of the wall is similar for both walls. However, when the difference between wall heights, h_t, is taken into account the stiffer and shorter wall can be seen to require a larger flexural plastic hinge rotation, θ_p.

The similar inelastic displacement demand for the two walls with strength based on a C_d factor of .2 is an interesting result. However, because of the shape of most design spectra used to determine seismic loading, shorter period walls would normally be designed for a higher C_d factor and this would reduce the inelastic demand imposed on them.

Similar results for the Pacoima earthquake motion are reproduced in Figure 3.19 from Figures 3.8 and 3.17.

The anomalous result obtained for the analysis of the 0.5 second period wall with $V_p h_c / M_p$ = 1.4 has been ignored.

The results are similar to those obtained for the El Centro earthquake motion except that the component of the displacement at the top of the wall due to flexural plastic hinge rotation, $\theta_p h_t$, is smaller for the stiffer wall. However once the difference in wall height, h_t, has been allowed for there is not much difference between the flexural plastic hinge rotations, θ_p, generated in the two walls.

3.4.2 Use of Elastic Response Spectra to Predict Inelastic Results

Figure 3.20 shows the elastic displacement response spectra for the first 10 seconds of the Pacoima and El Centro earthquake motions used for the inelastic analysis of the walls.

It can be seen that the spectral displacements tend to increase with period. This is the common trend for earthquake motions.

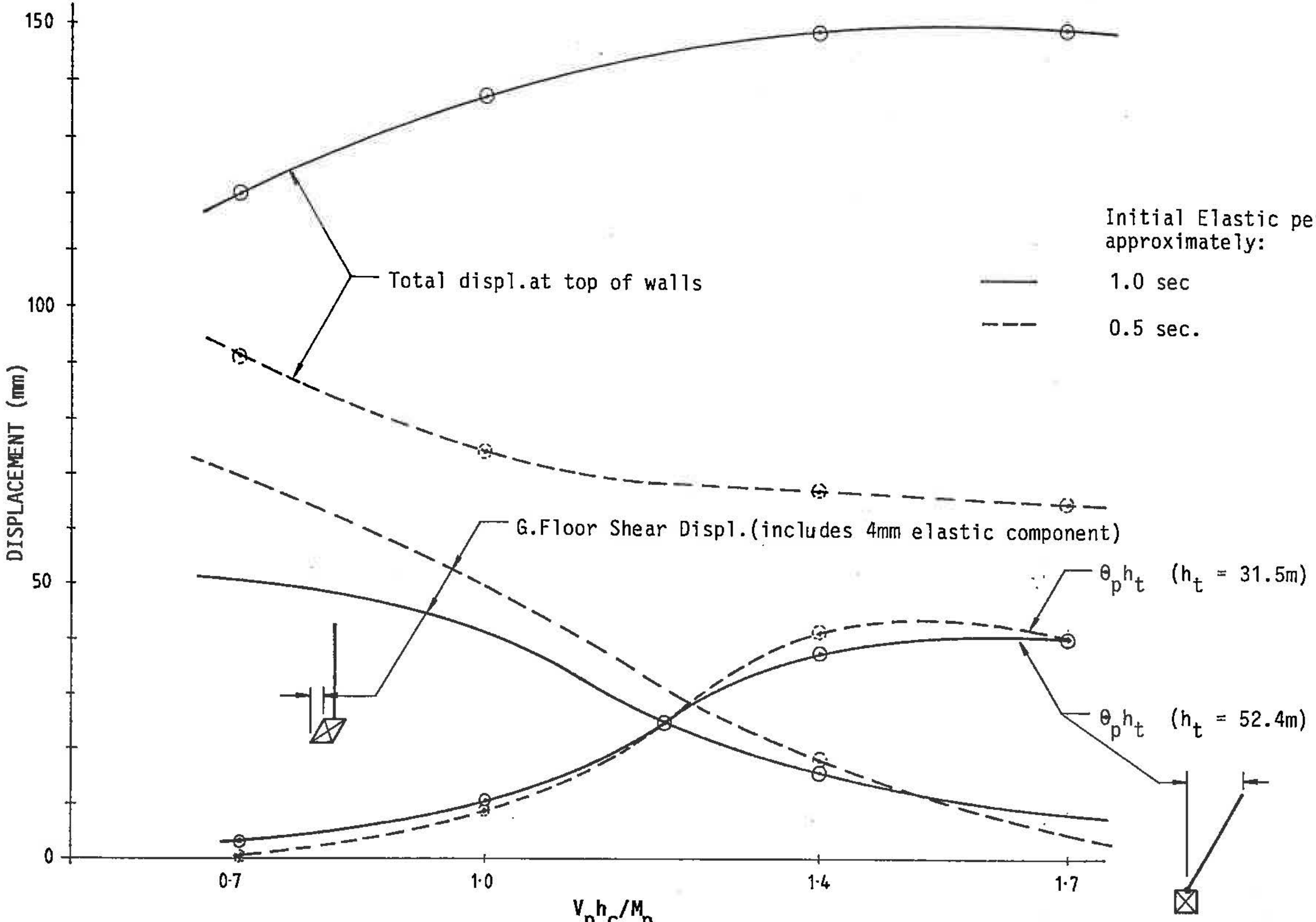


Fig. 3.18 Comparison of Peak Displacements of Walls with Initial Elastic Periods of 1.0 and 0.5 secs - walls responding to ELCENTRO EQ, C_d = .2 .

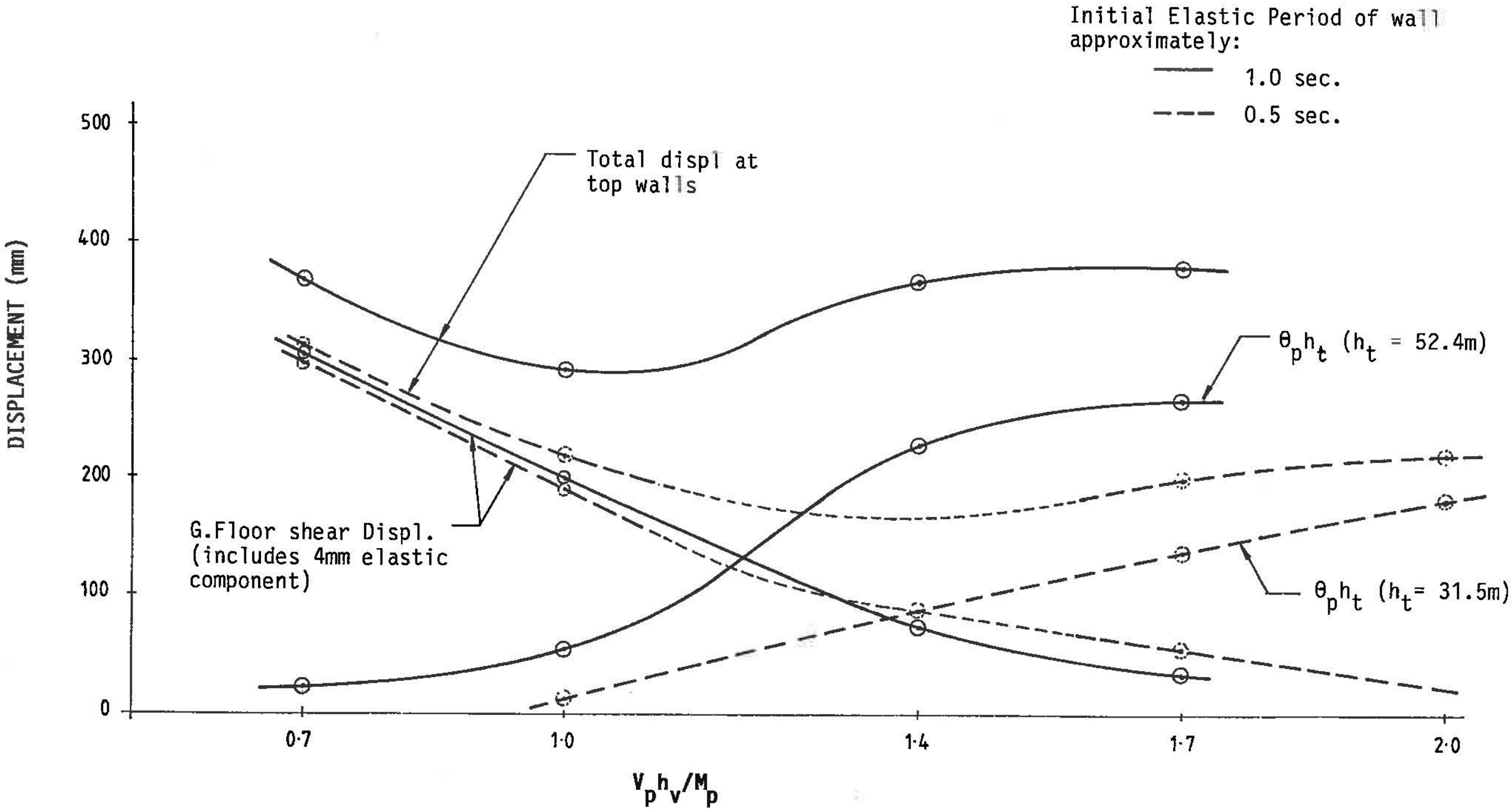


Fig. 3.19 Comparison of Peak Displacements of wall with Initial Elastic Periods of 1.0 and 0.5 secs. Walls Responding to PACOIMA EQ Motion, C_d = .2 .

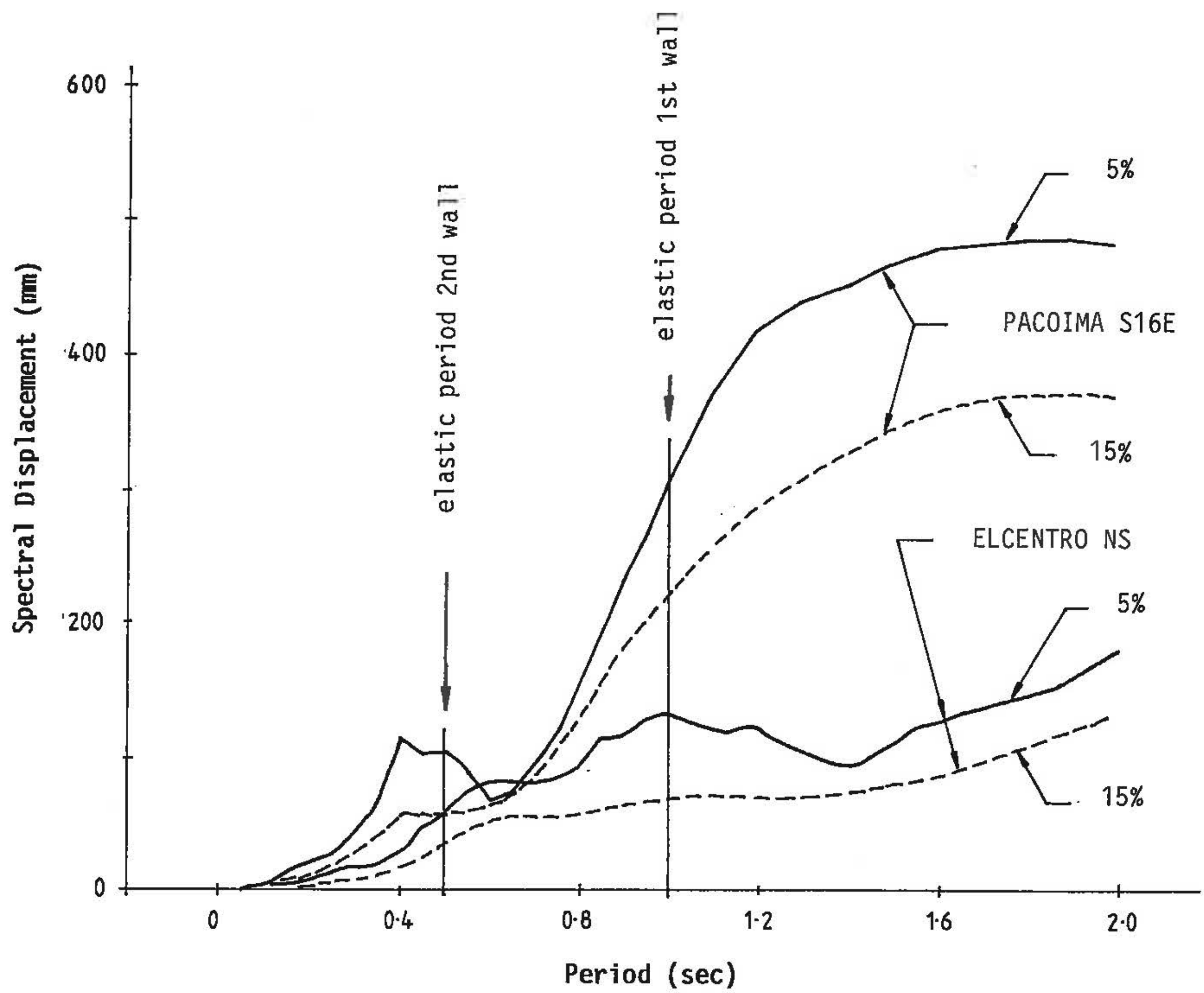


Fig.3.20 Elastic Displacement Response Spectra for first 10 secs of PACOIMA and ELCENTRO EQ motions with 5% and 15% Damping. Used to indicate effect of Period Shift on Displacement Demand.

It was therefore expected that inelastic shear and flexural displacement demand would also increase with wall period. It can be seen from Figures 3.18 and 3.19 that this did not occur. However, the actual behaviour of the walls can generally be explained by considering the detailed shape of the elastic displacement response spectra, the effects of period shift and the effect of increased effective damping. The influence that these factors have on the wall response will now be examined.

The time history shown in Figure 3.21 is the result of an inelastic analysis of the wall with the parameters given in columns (1) to (4) in the first line of Table 3.2. The period of the inelastic response can be seen in Figure 3.21 to be about 0.46 seconds. This is close to the 0.5 second initial elastic period that was estimated for this wall. This was to be expected as the analysis used relatively high strength parameters (C_d = .4, $V_p h_c/M_p$ = 1.4) so that the inelastic demand imposed on the wall was low. The low inelastic demand (i.e. ductility) can be observed in Figures 3.16 and 3.21.

The peak displacements at the top of a wall responding elastically in a flexural mode to earthquake motions is approximately 1.5 times that of a single degree of freedom (SDOF) oscilator [14]. However, if a wall is responding elastically with only a large shear displacement at its base and very little elastic displacement over the wall height, this factor would be closer to 1.0. To allow for this effect, a Multi Degree of Freedom multiplier, F, has been used to estimate the inelastic displacement demand from the elastic SDOF spectra results.

This factor is given in column (6) of the table and varies between a value of 1.5 when the peak shear displacement at ground floor level is zero and a value of 1.0 when the peak shear displacement at ground floor level is equal to the peak displacement at the top of the wall. For example, to obtain the value of F = 1.48 given in the first line of the table, linear interpolation was used. The peak ground floor shear displacements and peak total displacement at the top of the wall were obtained from Figure 3.16 for C_d = .2 and $V_p h_c/M_p$ = 1.4. The ratio of the ground floor shear displacement to the total displacement was then used to linearly interpolate between the values of F = 1.5 and 1.0, and obtain a value of 1.48.

The displacements at the top of the wall given in column (7) of the table were obtained from the spectral displacements shown in Figure 3.20 for the inelastic period shown in column (5). To allow for Multi-degree of Freedom Effects the spectral displacements were then multiplied by the MDOF multiplier F, shown in column (6).

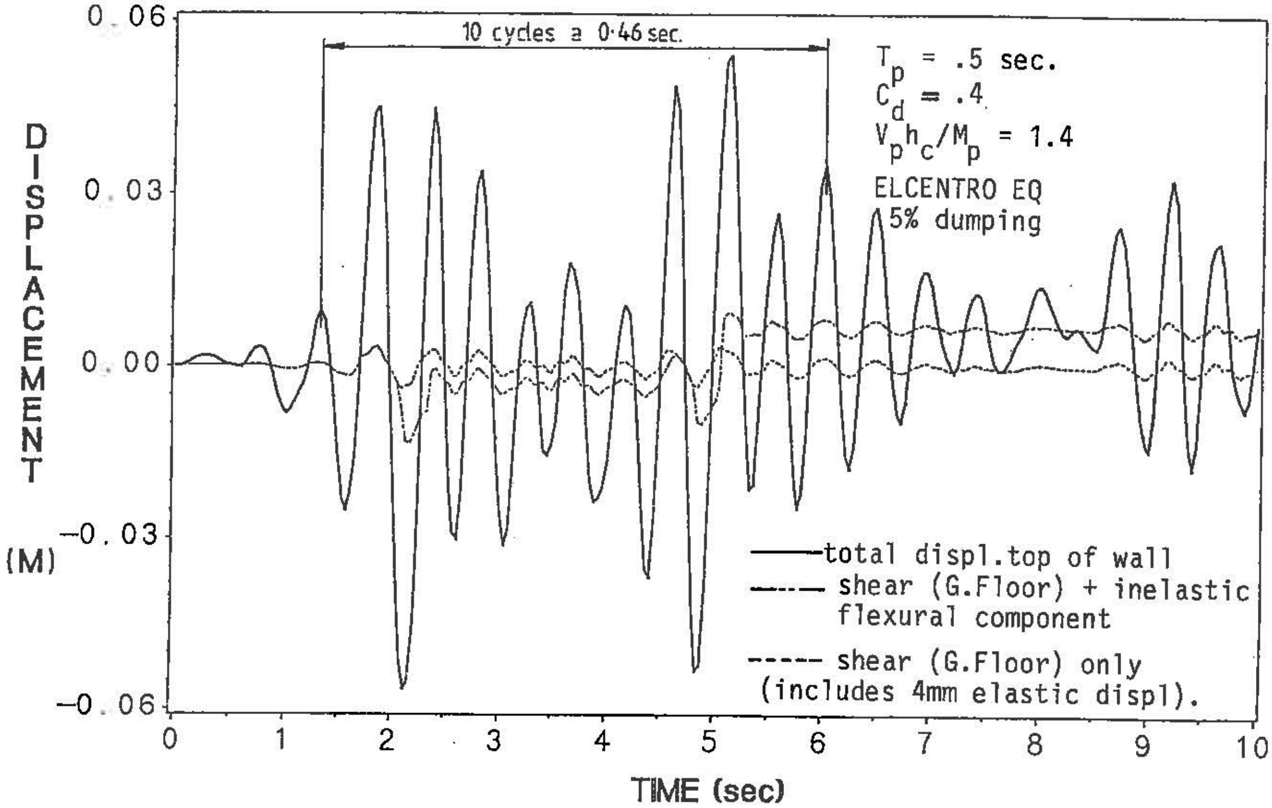


Fig. 3.21(a) Displacement time History at top of wall and components due to shear and flexural yielding for wall with low Inelastic Demand.

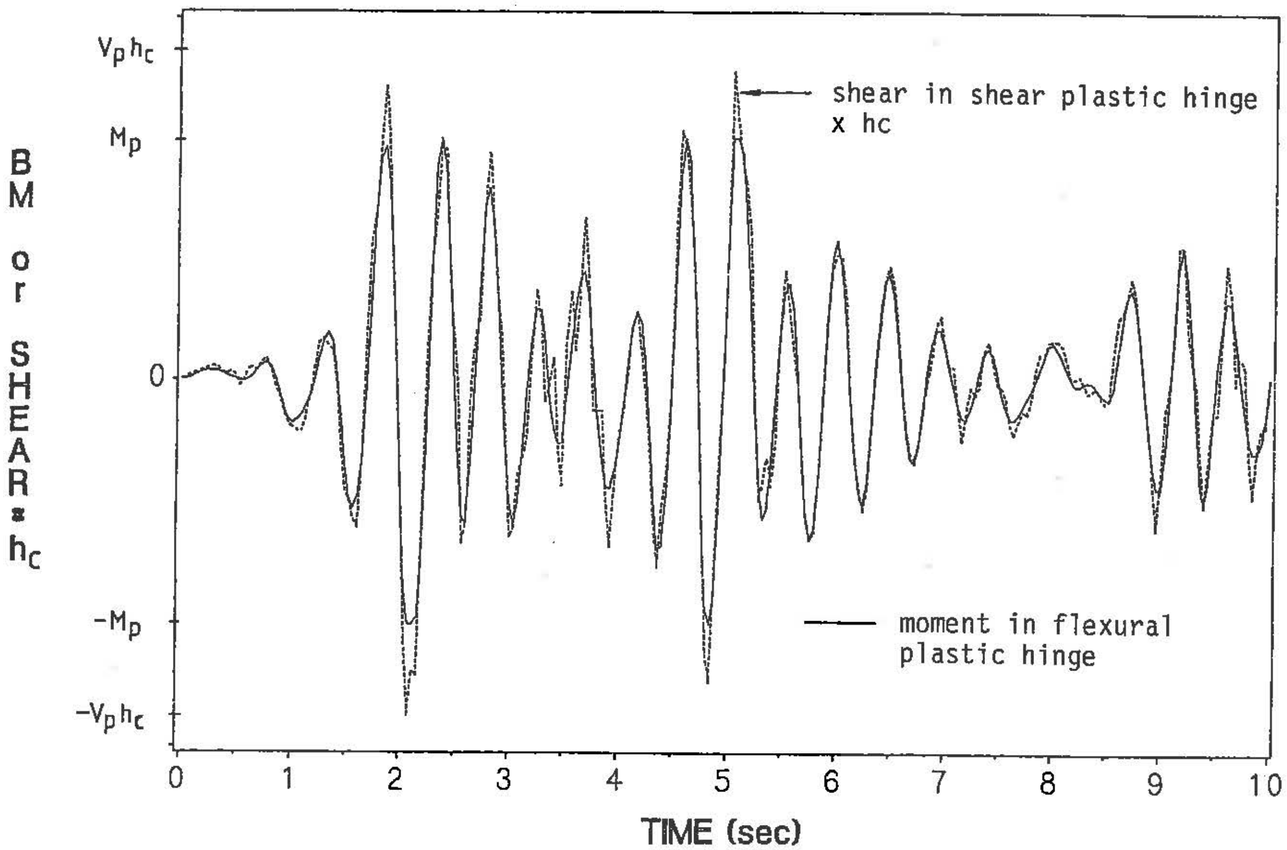


Fig. 3.21(b) Time History of Moment and Shear in Flexural and Shear Plastic Hinges (same analysis as Fig. 3.2(a))

TABLE 3.2 USE OF ELASTIC SPECTRA TO PREDICT INELASTIC RESPONSE

(1) Initial Elastic Period $= T_o$ (sec)	(2) EQ Record	(3) C_d	(4) $\frac{V_p h_c}{M_p}$	(5) Inelastic Period = T_i (sec) (analysis)	(6) MDOF multiplier = F	(7) Displ. of top of wall = (Spectral Disp for T_i) x F (mm) for damping of: 5%	(7) 15%	(8) Displ. top of wall (analysis) (mm)	(9) Ductility ratio : ((8)/(11))	(10) Approx yield displ. at top of wall - triangular load dist. (by calculation)	(11) Elastic displ. at top of wall from inelastic analysis	(12) Ratio (11)/(10) (* = valve for $V_p h_c/M_p$ = 0.7)
.5	El Centro	.4	1.4	.46	1.48	64	39	64	1.3	56	48	.86
		.2	.7	.94	1.11	133	71	85	7.1	18	12	.66*
			1.0	.92	1.17	132	76	74	3.5	26	21	.81
		"	1.4	.61	1.36	103	73	67	4.8	26	14	.54
			1.7	.66	1.48	115	82	65	2.6	26	25	.96
.5	El Centro	.4	.7	.85	1.23	130	75	62	1.6	37	31	.84*
			1.0	.55	1.3	88	60	60	1.9	52	31	.60
		"	1.7	.46	1.47	62	38	63	1.3	52	48	.92
.5	Pacoima	.2	.7	1.5/.54	1.01	468/90	347/45	308	30.8	18	10	.56*
			1.0	1.6/.62	1.05	500/70	374/57	212	8.8	26	24	.92
		"	1.4	1.5/.54	-	-	-	-	-		-	-
			1.7	1.26/.54	1.36	588/121	451/61	198	16.5	26	12	.46
.5	Pacoima	.4	.7	1.42/.59	1.04	468/73	338/62	215	10.7	37	20	.54*
			1.0	1.6/.52	1.1	523/100	391/61	115	5.5	52	21	.40
		"	1.4	1.1/.44	1.41	425/143	306/79	113	2.5	52	45	.86
			1.7	1.1/.44	1.5	480/154	325/84	115	2.4	52	47	.90
1.0	El Centro	.2	.7	1.2	1.29	152	91	120	1.9	80	62	.77*
			1.0	1.05	1.35	163	93	138	1.6	115	88	.76
		"	1.4	.9	1.44	154	93	147	1.6	115	94	.81
			1.7	.9	1.47	157	94	149	1.4	115	109	.94
		.1	1.0	1.07	1.4	170	98	117	5.8	57	20	.35
1.0	Pacoima	.2	.7	1.4	1.08	483	350	370	8.4	80	44	.55*
			1.0	1.45	1.16	522	383	295	6.5	115	45	.39
		"	1.4	1.1	1.4	517	354	370	5.3	115	70	.61
			1.7	1.1	1.46	537	368	385	4.1	115	93	.80
1.0	Pacoima	.4	.7	1.2	1.17	486	327	295	2.9	161	103	.64*
			1.0	1.25	1.26	536	365	410	2.6	230	155	.67
		"	1.4	.95	1.49	390	298	525	2.3	230	230	1.0
			1.7	1.03	1.5	430	330	525	2.2	230	235	1.0

The resulting displacements at the top of the wall shown in column (7) can be compared with those obtained from the inelastic analysis results that are shown in column (8). These were obtained from the various plotted results of the inelastic analysis of the two walls. Comparing the displacement values given in the first line of the table suggests that the effective inelastic damping was close to the elastic damping of 5% assumed for the inelastic dynamic analysis. As the ratio of the combined shear and flexural inelastic displacement to total displacement at the top of the wall is relatively low (see Figure 3.16) the ductility demand is also low. This is consistent with the low effective damping of 5%.

Figures 3.22(a) and (b) show the time history results of the inelastic wall analysis corresponding to the second line of the table. In this case the inelastic period of .94 seconds (see Figure 3.22(a)) is significantly greater than the initial elastic period of 0.5 seconds, indicating a significant period shift. Also the ductility demand is high as indicated in Figures 3.16 and 3.22(b).

In this case, comparing the displacement values in columns (7) and (8) of the table indicates that the effective damping is closer to 15% than 5% as would be expected when ductility demand is high.

Similar comparisons for the remainder of the table between columns (7) and (8), taking into account the ductility demand indicated in column (9), shows reasonable agreement between the displacement at the top of the wall predicted from the elastic spectra and that obtained from the inelastic analysis of the walls.

The exception is the results obtained for the Pacoima earthquake record and the wall with an initial period of 0.5 seconds. Figure 3.23(a) and (b) shows the time history results for this wall when $C_d = .2$ and $V_p h_c/M_p = 1.7$. It can be seen that the large displacements corresponding to the time interval of the pulse in the Pacoima record (see Figure 2.13) has a period of approximately 1.26 seconds while the complete strong motion part of the response has an average period of only .54 seconds. As can be observed from examining the table, the effective period, T_i, that would be required to give the same displacements at the top of the wall from the elastic spectra (column (7)) and the inelastic analysis (column (8)) would lie between these two limits.

Using the same earthquake record the agreement between columns (7) and (8) is better when the wall has an initial period of 1.0 seconds. This is probably because, in this case, the period of the pulse and the wall are closer together.

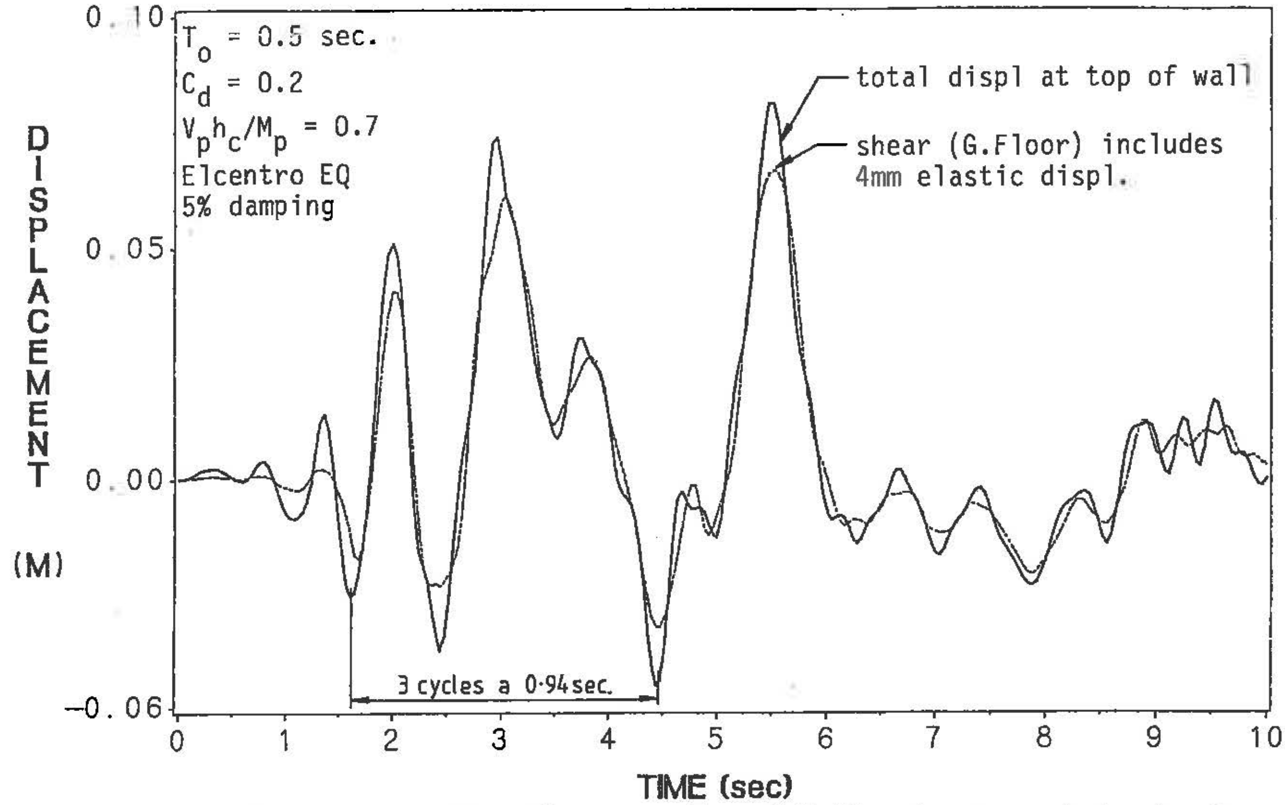


Fig. 3.22(a) **Displacement Time History at Top of Wall and component due to shear yielding (no flexural yielding and high inelastic demand).**

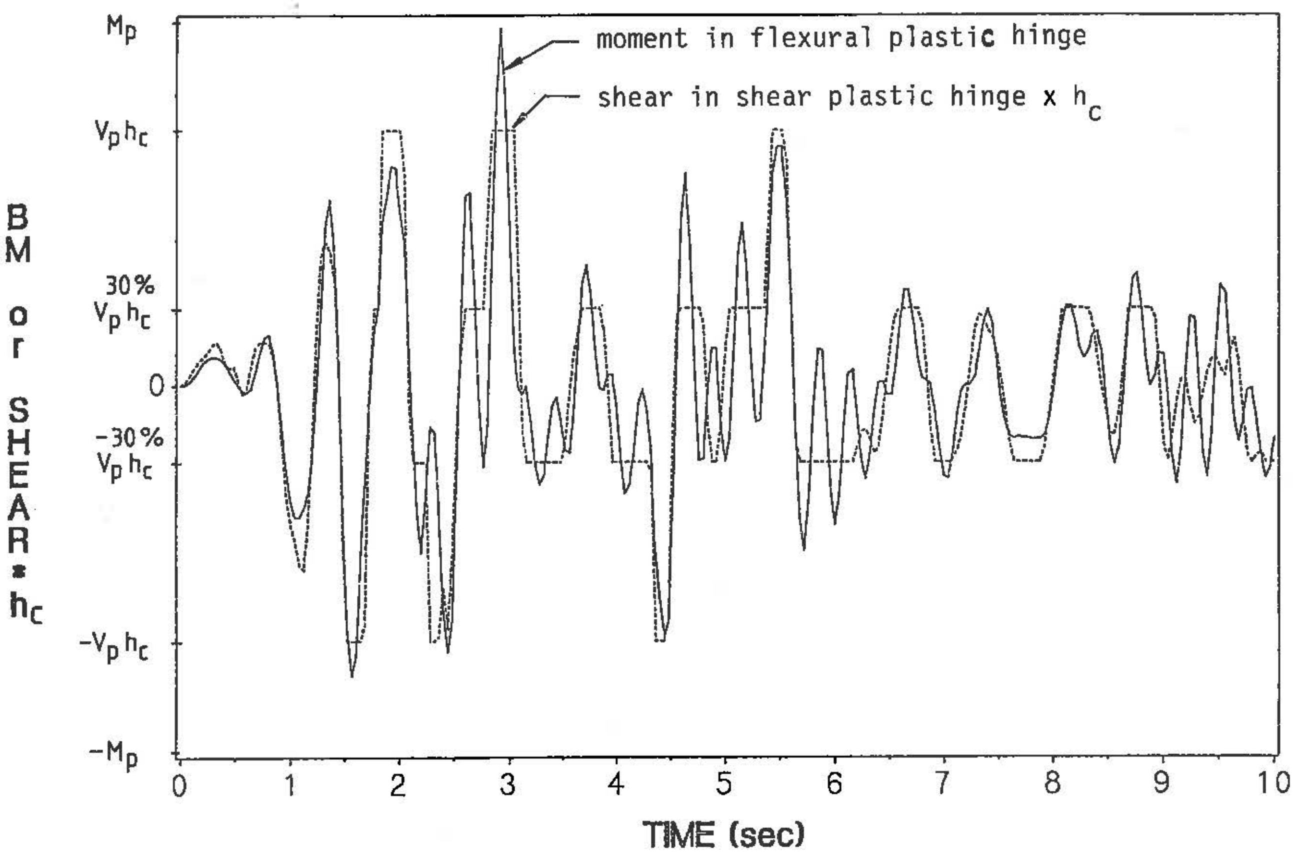


Fig.3.22(b) **Time History of Moment and Shear in Flexural and Shear Plastic Hinges** (same analysis as 3.22(a)).

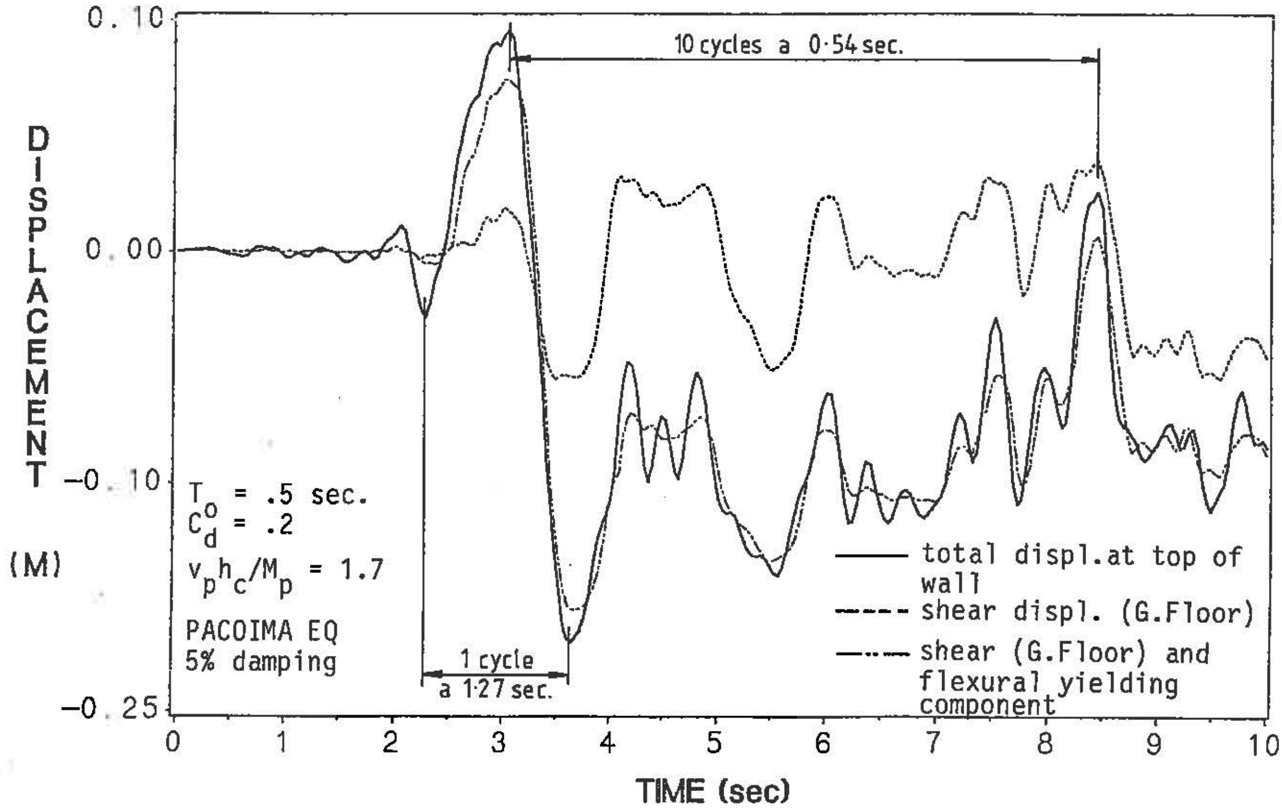


Fig. 23(a) Displacement Time History at Top of Wall and Components Due to Shear and Flexural Yielding.

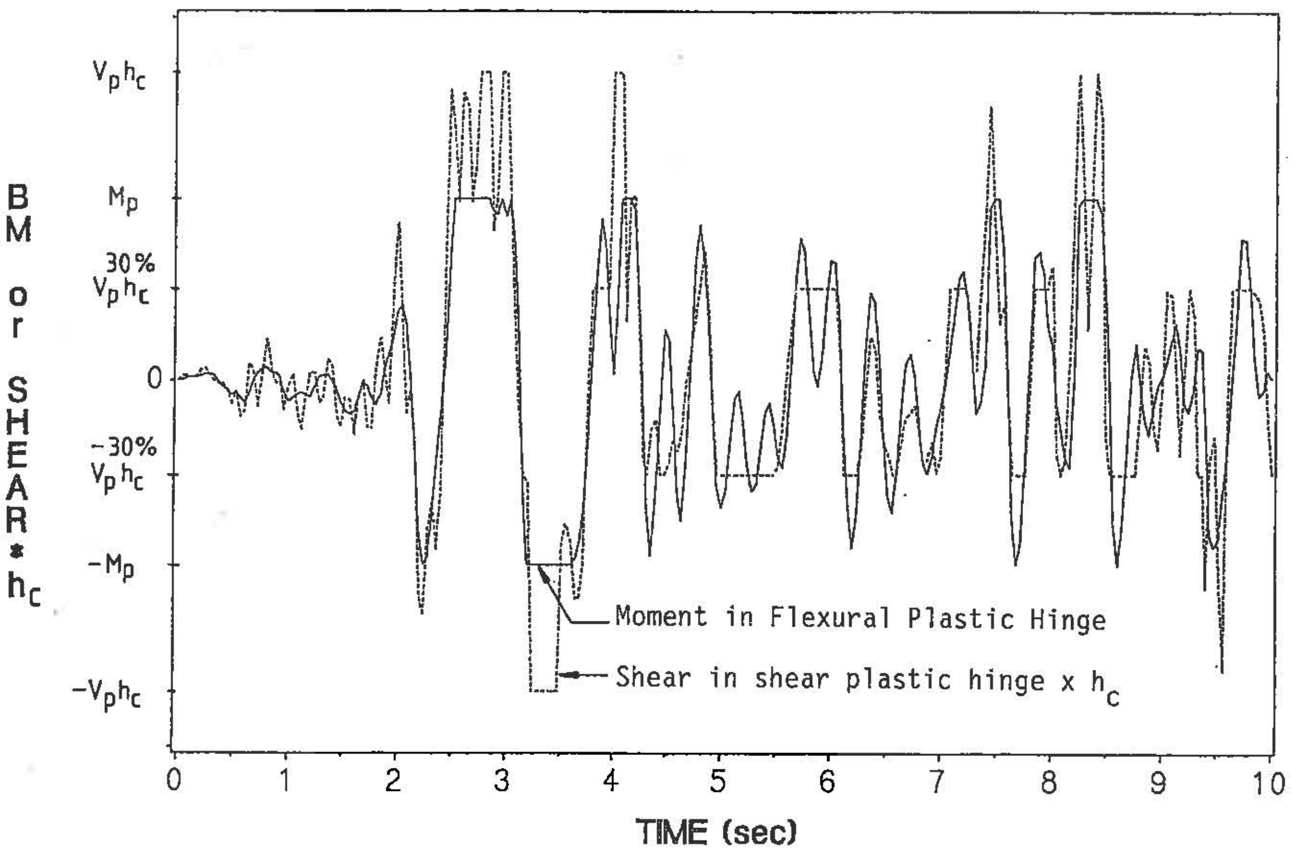


Fig 23(b) Time History of Moment and Shear in Flexural and Shear Plastic Hinges (same analysis as Fig 23 (a))

Column (10) of the table indicates the approximate elastic yield displacement of the wall. This was calculated assuming the walls mass and stiffness were uniformly distributed and that the load required to cause shear or flexural yielding had a triangular distribution above the flexural plastic hinge level. This yield displacement may be compared with the "elastic" displacement that can be obtained from the inelastic analysis of the walls by subtracting the combined peak inelastic shear and flexural displacement from the peak total displacement at the top of the wall. These values of "elastic" wall displacement were read from the various plotted results and are shown in column (11) of the table.

The ratio of the two displacements is shown in column (12) and indicates that the "elastic" component of the wall's displacement when the wall reaches its peak displacement is, on average, only 61% of its elastic yield displacement.

It can also be observed that the average ratio of the two displacements tends to increase from 65% to 85% as the shear/moment strength ratio; $V_p h_c/M_p$, increases from 0.7 to 1.7 (ratios corresponding to 0.7 are indicated by an *).

For a SDOF system yielding in shear or flexure this ratio would always be 1.0 as the elastic displacement when the oscilator reaches its peak total displacement must be the yield displacement.

However this is not true for a MDOF system. This was illustrated in subsection 3.2.4 for a wall yielding predominantly in a shear mode where the wall had practically zero elastic displacement at the top of the wall when the wall reached its peak shear displacement.

Table 3.2 indicates that a SDOF elastic displacement response spectra could be used to estimate the inelastic displacement demand in a wall that is yielding in a combined flexure and shear mode if an appropriate allowance is made for period shift, MDOF amplification, effective damping, pulse effects and the elastic displacement that the wall is likely to have when it reaches its peak total displacement.

This suggests that it should be possible to develop a procedure for evaluating the adequacy of walls yielding in a combined shear and flexural mode based on the use of an elastic response spectra.

3.5 TENTATIVE PROCEDURE FOR EVALUATING WALLS YIELDING IN A COMBINED SHEAR AND FLEXURAL MODE

Development of a complete analysis procedure for evaluating shear walls is beyond the scope of this study. However the following steps could form the basis of such a procedure:

(1) Obtain an appropriate set of displacement response spectra for 5 to 15% damping. These could be pseudo displacement spectra obtained from the acceleration spectra that are normally used in design.

(2) Using the spectra and the initial elastic period of the wall estimate the total displacement at the top of the wall making an appropriate allowance for MDOF amplification effects.

(3) Estimate the total inelastic displacement demand of the wall by subtracting an appropriate allowance for the elastic displacement of the wall.

(4) Use the total inelastic demand to estimate the likely effective period of the wall T_i and its effective damping. Use these values to reevaluate the total displacement demand as in step 1 and iterate steps 1 to 4 as required.

(5) When the total inelastic displacement demand has been estimated it can be split between the shear and flexural modes. As the various plotted results of the inelastic analysis indicate, the proportion for each mode will depend on the shear/moment strength ratio $V_p h_c / M_p$.

(6) Use modified compression field theory to evaluate the capacity of the wall to develop the required shear strength, V_p, simultaneously with the required inelastic shear and flexural strains. This part of the tentative procedure is outlined more fully in subsection 3.6.3.

3.6 POTENTIAL FOR STRUCTURAL DAMAGE TO R.C. WALLS

3.6.1 Inelastic Shear Displacement Demand for First Wall

If the minimum specified strengths of the wall reinforcement and concrete are used to compute the ideal strength of the 1st wall selected for study, the wall's shear/moment plastic hinge strength ratio, $V_p h_c/M_p$, is approximately 1.0. Also, a lateral load coefficient of C_d = .2 is required to develop the walls flexural plastic hinge strength, M_p, assuming the seismic load has a code type triangular distribution. The corresponding inelastic dynamic analysis results given in Figures 3.7, 3.8 and 3.10 indicate an inelastic shear displacement demand of 32 mm, 195 mm and 140 mm would be imposed on the wall by the El Centro, Pacoima and I.V. College earthquake motions respectively.

The analysis results also indicate that any overstrength in the flexural plastic hinge would increase the inelastic shear displacement demand. For example, if an overstrength factor of 1.25 was assumed, M_p (and C_d) would be increased by a factor of 1.25 and the shear/moment strength ratio, $V_p h_c/M_p$, would be reduced from 1.0 to 0.8. Interpolating between the ground floor shear displacements for C_d = .2 and .4 in Figures 3.9 for $V_p h_c/M_p$ = 0.8, indicates that increasing the flexural plastic hinge strength by 25% increases the inelastic shear displacement demand.

However a 25% overstrength factor is obviously too large in this case given the corresponding small amount of flexural yielding that the curve for $\theta_p \times h_t$ implies is generated in the flexural plastic hinge when $V_p h_c/M_p$ = .8.

The large, and probably unsustainable inelastic shear displacement demands given above for the "near fault" Pacoima and I.V. College earthquake records were unexpected given the good performance of shear wall buildings in previous earthquakes.

Part of the explanation for this is likely to be the influence that floor slabs and beams have on the shear strength of walls as the beneficial influence of floor elements is normally ignored in both the analysis and design of walls.

3.6.2 Influence of Floor Slabs on Wall Shear Strength

Floor slabs can act like horizontal wall ties and form part of a shear resisting truss mechanism.

If the wall has concentrated flexural reinforcement in wall boundary columns or flanges, these will act as the "cords" of the truss mechanism. In this case the floor slabs must extend well beyond the boundaries of the wall to be fully effective.

The floor slab reinforcement can then be anchored beyond the "cords" and the floor can then supply a compressive reaction to balance the forces developed in the diagonal compression struts that must form in the web of the wall as part of the truss mechanism.

In the early 1970s Barda [17] tested some squat shear walls that had flanges and used a "floor slab" element to introduce the load into the tops of the walls. The flanges of the walls were heavily reinforced to ensure that the walls failed in shear rather than flexure.

Most of the walls had a height (h_w) to length (l_w) ratio of ½ but one of the walls had a h_w/l_w ratio of 1.0.

As the squat walls did not have flexural moments and axial loads acting at their top boundaries their behaviour is likely to differ from that of the hinge zone of a multi-storey shear wall. However they do illustrate the likely influence that floor slabs can have on the shear strength of shear walls and the displacement capacity of walls that fail in diagonal compression without significant yielding of vertical and/or horizontal reinforcement.

Figure 3.24 shows the hysterisis loops for the wall with $h_w/l_w = 1.0$ up to a displacement of 30.4 mm (1.2 inches) and Figures 3.25(a) and (b) show the wall at peak load and after cycling to $\pm$ 75 mm respectively. Beyond a displacement of about 38 mm (1.5 inches) the web was ineffective and the strength was due almost entirely to frame action of the flanges.

The peak shear stress when the web started to fail by crushing was approximately $1.0\sqrt{f'_c}$ (or .22 f'_c, where f'_c was the measured concrete compressive strength) and as the horizontal web reinforcement accounted for approximately half this strength the concrete component corresponded to approximately $.48\sqrt{f'_c}$ (.09 f'_c). Most of this would have been provided by arching action with large flange and "floor element" forces being resisted by a diagonal strut in the web of the wall.

For walls with h_w/l_w = ½ the tests showed that this diagonal strut mechanism could develop shear stresses of $.7\sqrt{f'_c}$ (.14 f'_c) before the wall failed by diagonal crushing even without any vertical reinforcement in the web of the wall. The provision of vertical web steel increased the failure shear stress to more than .2f'c before diagonal crushing occurred probably because it reduced the stress concentration on the web diagonal and controlled web cracking. These squat wall tests suggest that if the floor slabs and flanges of the shear wall have sufficient reinforcement the peak shear strength of a wall will correspond to a diagonal crushing shear strength of the web of the wall. The NZ design code for concrete [13] implies a design value of $.2f'_c$ for the shear stress corresponding to diagonal crushing. This is close to the .22 f'_c shear stress at which Barda's wall with $h_w/l_w = 1.0$ failed.

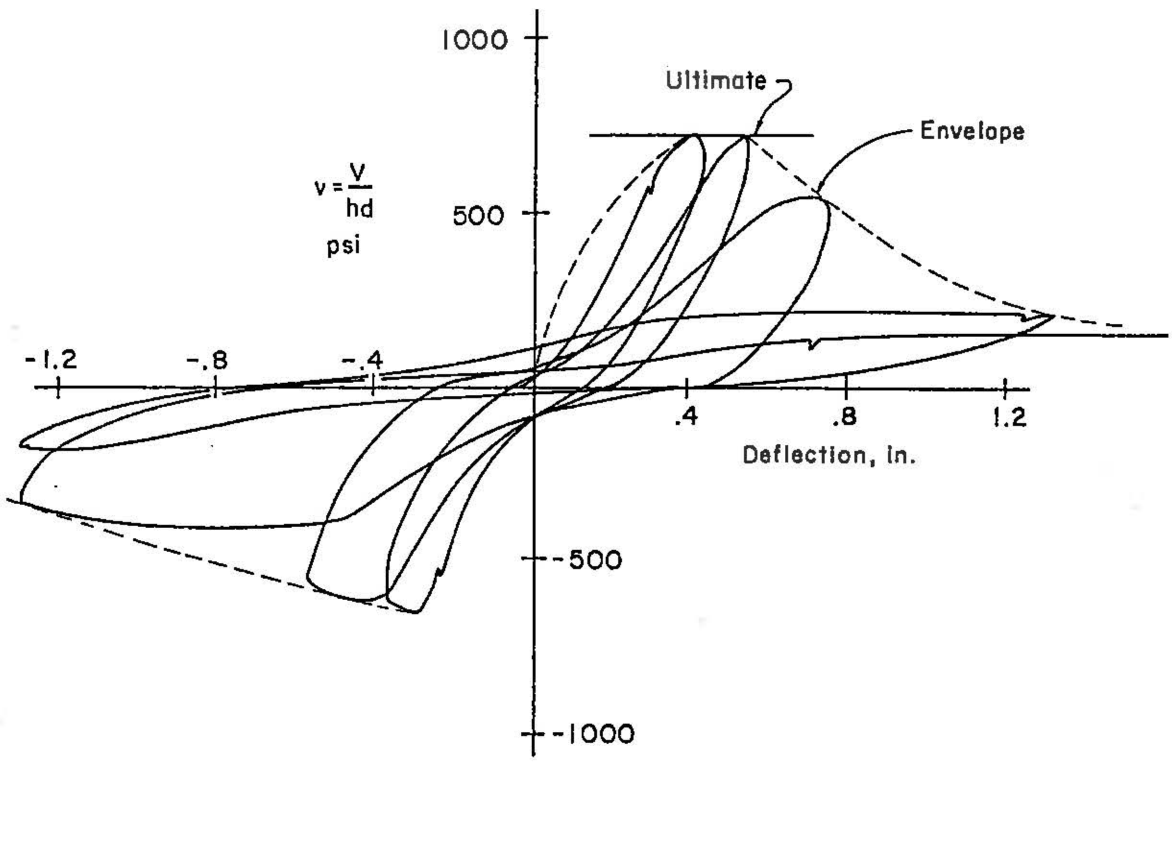
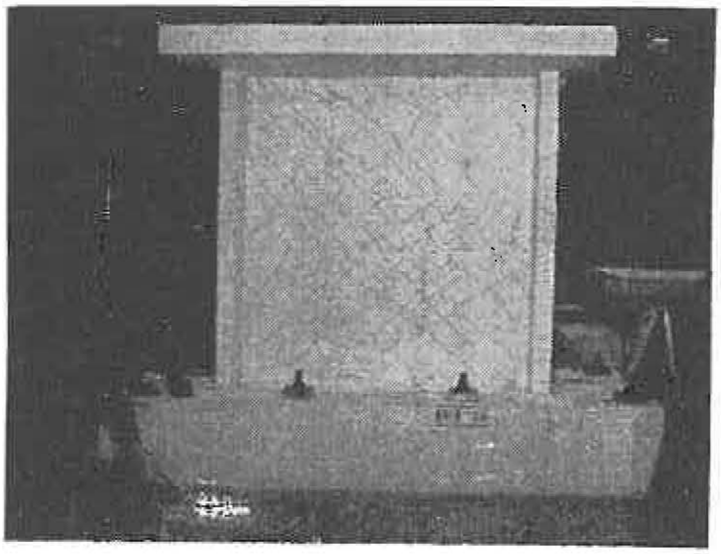
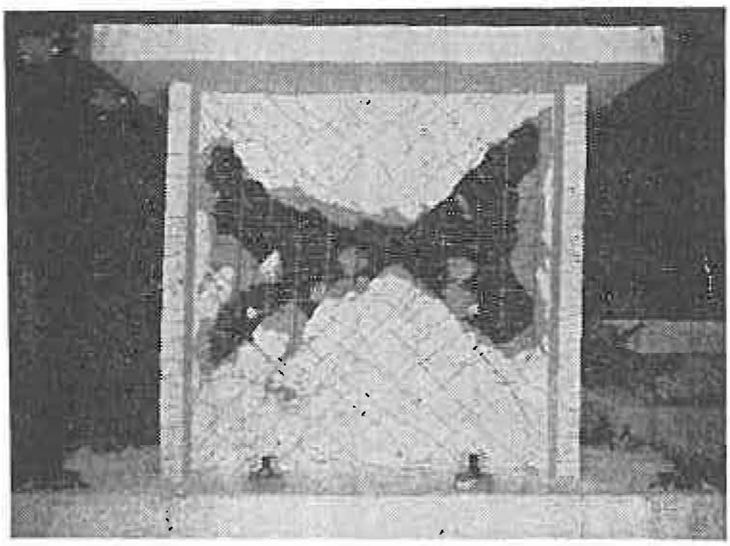


Fig. 2.24 Hysteresis Loop for squat wall with h_w/l_w = 1.0 - Intermediate cycles to ultimate and ± 1.2 inch displ. only (obtained from ref. [17].



(a)



(b)

Fig.3.25 squat wall with h_w/l_w = 1.0 (a) at peak load
(b) after cycling to ± 75mm displacement

Figure 3.24 illustrates that even though this type of failure mechanism is relatively brittle, peak load can still be sustained at a displacement of approximately 15 mm (.5 inches). Given that the wall is only 1.9 m long and is only 1/3 scale (for a squat wall) the shear displacement capacity of the hinge zone of multi-storey shear walls is likely to be significantly greater than 15 mm before there is significant strength loss.

However, the effects of flexural ductility also need to be considered.

A similar wall element to those tested by Barda formed the flexural plastic hinge zone of a full scale seven storey reinforced concrete building that was tested in Japan [20, 21 and 22]. The wall's flexural plastic hinge zone sustained significant flexural ductility and in spite of this, it appears to have sustained a much higher shear stress than the NZ concrete design code would predict.

The test structure had a central shear wall with column boundary elements and was tied to surrounding frames by floor slabs.

The pattern of flexural/shear cracks in the walls [22] suggests that the floor slabs were acting as effective horizontal shear reinforcement for the walls.

The wall finally failed in shear after some of the main reinforcement in the wall boundary columns fractured and the wall had reached an average inter-storey displacement of 1.33% over its height. The shear failure was accompanied by concrete crushing over the full depth of the wall [21].

Using measured material properties and assuming that points of inflection in the columns of the frames were located 2 m above foundation level, the writer has estimated that, at failure, the wall carried at least 68% of the total shear force applied to the structure. This means that the wall was able to resist a shear stress of at least $.19f'_c$ in spite of the high flexural ductility imposed on its plastic hinge zone. This is almost twice the stress that the NZ concrete design code [13] predicts the wall could withstand ($0.115\ f'_c$) if the contribution of the floor slabs is ignored. If half the reinforcement in the floor slab and beams of the frame are assumed to be effective as wall shear reinforcement the code approach would predict that the wall could develop the NZ Code diagonal crushing stress of $0.2\ f'_c$.

This suggests that if adjacent floor slabs and beams have adequate effective reinforcement, shear walls could develop their diagonal crushing strength without significant yielding of horizontal wall ties. The walls will then develop a higher shear strength but will fail in a more brittle fashion than would be expected from a normal analysis ignoring the floor slabs and assuming some yielding of the wall ties.

The first wall selected for this study would develop approximately twice the shear strength derived using a conventional code analysis if the floor slabs could develop a diagonal crushing stress of 0.2 f'_c in the wall. As this would increase the shear/moment strength ratio, $V_p h_c/M_p$, from 1.0 to 2.0, the wall would easily cope with the small shear deformations that the three earthquake motions examined would impose on the walls. As the wall is 11 metres long, three floors will cross any potential 45° failure plane. However the floor slab has large service openings in the core on one side of the wall and as the remaining floor slab is a waffle slab it has a relatively small amount of bottom steel in its ribs.

If a 45° shear failure plane is assumed and an allowance is made for the bottom steel required to carry gravity loads, the floor slabs would increase the shear strength of the wall by only 40%. The shear/moment strength ratio, $V_p h_c/M_p$, would then equal 1.4 and for the three earthquake motions considered, some shear yielding would still be required. However yielding of the slab and horizontal wall reinforcement may be able to provide the required shear yielding displacement demand.

As companion test specimens to the squat shear walls tested by Barda, six walls without floor slabs and with rectangular cross sections were tested by Cardenas [23]. Two of the walls, one with a h_w/l_w of 3.3 the other with h_w/l_w = 1.9 failed in a "flexural-shear" mode as shown in Figure 3.26.

At failure, diagonal crushing occurred at the base of the wall and some stirrups fractured suggesting significant shear yielding. Although shear displacements at the base of the wall were measured they were, unfortunately, not reported.

3.6.3 Modified Compression Field Theory

Modified compression field theory shows great promise as a means of predicting the shear yield displacement capacity and strength of walls that fail in shear.

The Canadian Concrete Design Code permits a simplified version of modified compression field theory to be used in design [24]. It is practical to use this simplified procedure when evaluating a particular building in a design office setting.

The traditional method of evaluating the shear strength of walls is to use a truss analogy with concrete struts forming at 45° to the main flexural reinforcement. The shear strength is then considered to be made up of two components, one due to the shear reinforcement the other due to a "concrete component".

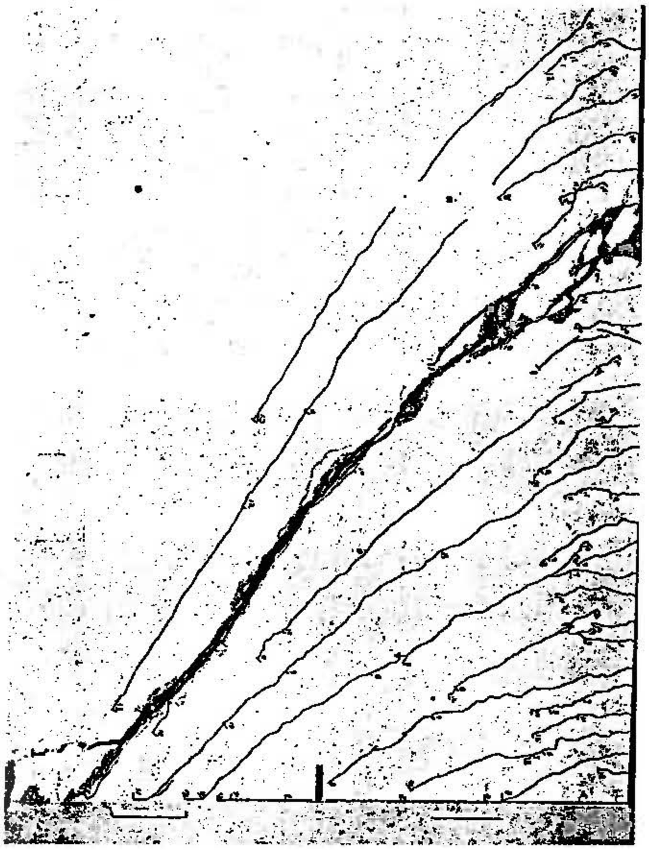


Fig 3.26 Flexural shear failure of a rectangular shear wall

(obtained from reference [23])

Modified compression field theory abandons this approach. A separate "concrete component" is not considered and the angle of the compression struts is permitted to vary from the 45° assumed in the truss model. As the shear failure plane will be parallel to the compression struts a larger amount of shear reinforcement will cross the failure plane if the compression struts form at a shallow angle to the flexural reinforcement. The theory predicts that the angle at which the struts form is a function of the shear stress intensity, the strain in the flexural reinforcement (flexural ductility) and the strain in the shear reinforcement (including shear yielding).

The theory assumes that all shear failures, whether preceded by yielding of the shear reinforcement or not, are ultimately by crushing of the concrete diagonal struts. The greater the strains in the flexural or shear reinforcement (i.e. the greater the flexural or shear yielding) the lower the stress at which the diagonal struts crush.

This means that a shear wall does not have a single value for it's "shear strength". If any two of the three principal variables; shear stress, strain in the flexural or strain in the shear reinforcement are fixed the theory allows the third variable to be calculated. The theory can therefore be used to check the ability of a wall to withstand the inelastic displacement demand likely to be imposed by an earthquake. For example, if a wall is assumed to have a $V_p h_c/M_p$ ratio of 1.4 this assumes a fixed shear strength, V_p, for the wall. This in turn fixes the angle of the failure plane as it determines the amount of wall reinforcement (or floor slabs) which will cross the failure plane at the onset of diagonal crushing. Once the angle of the failure plane is fixed the average stress in the diagonal struts can be calculated as the struts will be parallel to the failure plane. If, for example, a wall with $V_p h_c/M_p = 1.4$ was subjected to the Pacoima earthquake motion, Figure 3.8 could be used to estimate the shear and flexural strain demand imposed on the wall. By assuming two of the three variables, say shear stress and flexural reinforcement strain, modified compression field theory would allow the third variable of shear strain capacity at the onset of diagonal crushing to be calculated. The shear displacement capacity could then be calculated and compared with the demand. The check procedure could then be repeated for other assumed values of $V_p h_c/M_p$ (i.e. the shear strength, V_p).

Such an approach, using curves like those in Figure 3.8, would tend to be conservative as the peak displacement values shown do not necessarily occur simultaneously.

3.6.4 Potential for Structural Damage to Walls Selected for Study

The type of analysis outlined above was carried out for the first shear wall selected for study. With a ratio of $V_p h_c/M_p = 1.4$, the simplified theory predicts that the struts will form at 40°

to the flexural reinforcement in order to develop the required shear strength.

Using the peak flexural plastic hinge rotations (Figure 3.8) and assuming a hinge length, the average flexural strains at the mid-depth of the wall was estimated at 0.25%. The theory then predicts that diagonal crushing of the compression struts will start when the average strain in the stirrups reaches 0.3%. When an allowance is made for shear deflection resulting from crushing strains in the struts (0.2%), the total shear displacement capacity of the hinge zone is estimated to be 70 mm at the onset of concrete crushing. This is less than the 80 mm shear displacement demand that Figure 3.8 predicts the Pacoima earthquake motion would impose on the wall. The resultant diagonal concrete crushing would cause a fall off in the walls shear strength. The consequences of this for the walls behaviour are beyond the scope of this part of the study but were addressed in Section 2.4.2.

It is interesting to note that, by inspection, the wall would easily cope with the inelastic demands imposed by the El Centro or I.V. College earthquake motions corresponding to a $V_p h_c/M_p$ ratio of 1.4 (see Figures 3.7 and 3.10).

If the New Zealand concrete code procedures using minimum specified wall reinforcement and concrete strengths are used to compute the ideal strengths of the 2nd wall selected for study the wall's shear/moment plastic hinge strength ratio, $V_p h_c/M_p$ is approximately 2.4. Also, a lateral load coefficient of C_d = .24 is required to develop the wall's flexural plastic hinge strength, M_p, assuming the seismic load has a code type triangular distribution. The corresponding inelastic dynamic analysis results for the wall given in Figures 3.16 and 3.17 indicate that the inelastic shear displacement demand for the wall would be negligible for the two earthquake records used in the analysis. In this case there is little point in applying modified compression field theory to evaluate the walls ability to withstand the small inelastic shear displacement demand.

3.6.5 Application of Modified Compression Field Theory to Previous Wall Test Results

An attempt was made to apply the simplified modified compression field theory to the rectangular walls tested by Cardenas [23]. A value for the "material resistance factor", ϕ_c, of 1.0 rather than the value of 0.6 suggested for use in design was used for the evaluation [24]. For the wall with a h_w/l_w = 3.3 the failure plane would need to be at approximately 30° to the flexural bars to engage sufficient wall ties to develop the measured shear strength. Although it is not clear which of the two shear failing walls the photograph in Figure 3.26 applies, the steepest cracks in the photograph are close to this 30° angle. However, given the relatively low shear stress at failure in the wall (.06 f'_c at the wall base), modified compression field theory would have predicted much steeper cracking and therefore a greater shear strength. The presence of high strains in the stirrups, implied by the large diagonal cracks and fracture of some stirrups, would also result in modified compression field theory predicting more steeply inclined cracks at failure.

If shear stresses were uniformly distributed across the squat walls tested by Barda, modified compression field theory would predict much greater shear strengths than those measured given that the strains measured in the flexural and shear reinforcement were generally less than yield. However, the presence of floor slab and flange elements concentrates stress in the diagonal strut that forms in the web of the walls tested.

A lower average diagonal crushing stress is therefore to be expected from the simplified theory. In fact modified compression field theory could be used to compute an effective width of the diagonal struts that formed in these walls.

The plastic hinge zone of the seven storey full scale building tested in Japan was similar to the squat walls tested by Barda except that they had significant flexural ductility imposed on them. Modified compression field theory would predict a reduction in diagonal crushing strength due to the presence of large strains in the flexural reinforcement.

The actual reduction from .22 f'_c crushing stress, for Barda's wall with $h_w/l_w = 1.0$, to something in excess of .19 f'_c for the seven storey building is less than would be expected from the theory.

However, in spite of its limitations, modified compression field theory shows promise as a means of predicting the shear displacement capacity of a wall when the shear force and flexural ductility are taken as fixed variables.

It is of concern that simplified compression field theory predicts that the single value of .2 f'_c given by the NZ design code [13] for shear stresses corresponding to diagonal compression failures may be unconservative where the strains in the flexural or shear reinforcement are high and/or there are stress concentrations in diagonal concrete struts within the web of a wall.

3.7 POTENTIAL FOR NON-STRUCTURAL DAMAGE

The effect that the shear displacements derived from the inelastic analysis would have on non-structural elements can be estimated from Table 3.3. In Table 3.3 the interstorey drift, corresponding to the damage indicated for the listed non-structural elements, is given as a ratio of the storey height. It is also given as a displacement for a 3 m storey height so that a direct comparison can be made with the shear displacements shown in the plotted results of the inelastic analysis of the walls. When the wall has a length significantly longer than the interstorey height it is probably too conservative to consider all the shear displacement taking place in one storey. The strain in the diagonal struts is likely to be approximately 2 mm/m [24] at the onset of diagonal crushing. This corresponds to a shear displacement component of 2 to 3 mm per metre of interstorey height. At least this component could be considered as being spread over the depth of the shear plastic hinge zone.

A survey of 162 buildings [25] of more than five storeys that were damaged by the San Fernando earthquake (California 1971) indicated that 80% of damage was non-structural. Partition repair costs made up the largest component (23%) while replacing glass contributed only 1% to the total cost. A higher level of damage to glass perhaps would have been expected from the figures given in the table depending on the types of window frames used.

TABLE 3.3 : POTENTIAL FOR NON STRUCTURAL DAMAGE DUE TO INTER STOREY DRIFT

NON STRUCTURAL ELEMENT	DAMAGE LEVEL	REF NO.	INTER STOREY DRIFT BY STOREY HEIGHT	INTER STOREY DRIFT FOR 3 m STOREY (mm)
PARTITIONS:				
Gypsum board on wood studs, stuco on metal lath and studs, Gypsum plaster on timber lath and studs - no separations.	• Cracks around door openings	1*	1/1000	3
	• Doors jamb and cracking extends into walls		1/500	6
	• Mortar and plaster on lath begins to fall off - door jambs separate from partitions		1/250	12
	• Separation of Gypsum board from frame		1/125	24
PARTITIONS:				
Gypsum wallboard on metal studs.	• Cracking and popping sounds - onset of damage	2	1/1500-1/400	2 - 7.5
	• First permanent damage		1/250	12
PARTITIONS AND OTHER ARCHITECTURAL ELEMENTS	• No cost for repairs	2**	1/1000	3
	• Damage ratio as proportion of original cost = 10%		1/200	15
	• Damage ratio as proportion of original cost = 30%		1/100	30
	• Damage ratio as proportion of original cost = 100%		1/50	60
WINDOW AND FRAMES	• No costs of repair		1/1000	3
	• Damage ratio as proportion of original cost = 30%	2**	1/200	15
	• Damage ratio as proportion of original cost = 80%		1/100	30
	• Damage ratio as proportion of original cost = 100%		1/50	60
WINDOWS:				
Aluminium sash windows above R.C. spandrels.	• Cracks in windows with hardening putty	1*	1/500	6
	• Cracks in windows with elastic sealant		1/125-1/75	24 - 40
	• No breakage of windows in sliding frames or loss of glass from frames where glass wired or coated with polyester adhesive film		1/75	40

* in reference 1 drifts are given as average for a seven storey building

** figures relate to a type of three storey steel framed building used by US Navy - cost of repair estimated at 1.5 times the % of original cost given for windows and 1.25 times for partitions and architectural elements to allow for demolition etc.

3.8 SUMMARY AND CONCLUSIONS : SECTION 3

Most shear wall buildings built in New Zealand prior to 1976 do not meet the capacity design requirements that are necessary to ensure that any inelastic displacement demand is satisfied principally by flexural yielding. Many of the walls in these buildings can, therefore, be expected to yield or possibly fail in shear unless the floor slabs are effective in contributing adequate additional shear strength.

In order to evaluate the inelastic shear displacement demand that could be imposed on such walls by earthquakes, two shear wall buildings designed and built in the late 50s and early 60s were selected for computer modelling and inelastic dynamic analysis. However the results of the analysis are approximately applicable to all walls with an initial elastic period of 1.0 or 0.5 seconds providing they have relatively uniform mass and stiffness distribution and the hysteretic model used for the flexural and shear plastic "hinges" is appropriate.

The computer model used to analyse the walls permitted shear or flexural yielding or a combination of the two yielding modes to take place near the base of the walls.

The results of the inelastic dynamic analyses indicate that the total displacement at the top of the wall and its inelastic component are not particularly sensitive to whether the yielding takes place in the shear or flexural mode. However the proportion of the total inelastic demand that takes place in the shear or flexural mode is sensitive to the shear/moment plastic "hinge" strength ratio, $V_p h_c/M_p$. When this ratio is less than about 0.7 almost all of the inelastic demand is in the shear mode and when the ratio is greater than 1.7 most is in the flexural mode.

The inelastic dynamic analysis results also indicate that the shear or flexural yielding is principally the result of the 1st mode response of the wall. The primary roll of the higher modes is to allocate the total inelastic demand between the shear and flexural yielding modes. Although this is a useful conceptual framework within which to view the role of higher modes, a detailed examination of the shear yielding behaviour of one of the walls indicated that it is an over simplification. During relatively long periods of shear yielding, the dynamic lateral loads acting on the wall are generated by a complex interaction of ground accelerations and the modal responses of the wall which are in turn, modified by the wall's inelastic response.

It was observed that when the wall was predominately deforming with shear yielding at the base of the wall, shear forces higher than those developed at the base of the walls were present above the mid height of the wall. There were also relatively high floor accelerations generated at the top of the walls for both shear and flexurally yielding structures.

The "form" of the results expressed as a characteristic shape of the plotted curves of peak wall displacements was insensitive to a number of variables examined. However there were local departures. Increasing the viscous damping reduced the magnitude of the peak displacements without changing the "form" of the results. Increasing the walls flexural and shear plastic hinge strength by the same amount increased the "elastic" component of the wall's peak displacement and reduced its inelastic component as would be expected. Increasing the intensity of the earthquake motion increased the total inelastic demand without changing the general form of the results. Reducing the wall's elastic period by increasing its stiffness principally reduced the "elastic" component of the walls displacement. However it did not reduce the inelastic displacement demand as expected. Generally this could be explained by considering the detailed shape of the elastic displacement response spectra corresponding to the earthquake motions used for the inelastic analysis.

It was concluded that an approximate estimate of the inelastic displacement demand in a wall that is yielding in a combined flexural and shear mode could be obtained from a SDOF elastic response spectra. This required an appropriate allowance to be made for period shift, MDOF amplification, effective damping, pulse effects and the elastic displacement that the wall is likely to have when it reaches its peak total displacement.

Initially the shear and flexural strengths of the 1st wall selected for study were evaluated using a conventional NZ concrete design code approach. Based on these strengths it was concluded that the inelastic shear displacement demand that the analysis predicted would be imposed on the wall by two of the earthquake motions used in the study would probably not be sustainable. On the basis of observed damage to walls in previous earthquakes this was unexpected. However, a review of research on shear failing walls suggests that floor slabs can form the horizontal ties of a truss mechanism and therefore enhance the shear strength of walls.

After allowing for the influence of floor slabs Modified Compression Field Theory was used to predict the shear yielding displacement capacity of the wall. It was concluded that the wall could not quite sustain the inelastic shear displacement demand that would be imposed by the Pacoima Dam earthquake motions without diagonal crushing of the concrete at the base of the wall. At the onset of crushing the wall could be expected to lose shear strength rapidly with increased shear displacement demand. However the performance of walls in previous earthquakes suggests that the wall may still have a reasonable margin of displacement capacity before it would collapse.

A tentative procedure for evaluating the seismic performance of walls that are likely to be subjected to significant inelastic shear displacement demand is proposed for future development.

The procedure suggests the use of a SDOF elastic response spectra to estimate inelastic shear and flexural displacement demand and the use of simplified modified compression field theory to evaluate the walls capacity to withstand the inelastic shear displacement demand without a loss of shear strength.

The procedure needs to be extended to allow for the effects of strength degradation and PΔ effects so that it can be used to evaluate the risk of collapse.

The risk of building collapse obviously correlates strongly with the risk to life. However, up to 80% or more of the cost of earthquake damage may be due to non structural damage. As the cost of building collapse makes up only part of the remaining 20% due to structural damage, the total cost of earthquake damage probably correlates poorly with the risk of building collapse.

Current design and detailing practices place a heavy emphasis on preventing building collapse rather than preventing damage. The influence that these current design and detailing practices will have on the cost of earthquake damage deserves further research but will ultimately be determined by the performance of modern buildings in future earthquakes.

REFERENCES

1. R. Park, "Evaluation of Ductility of Structures and Structural Assemblages from Laboratory Testing", Bulletin NZNSEE Vol. 22 No. 3, September 1989.

2. P. Fajfar, M. Fischinger, "Parametric Study of Inelastic Seismic Response Spectra", Pacific Conference on Earthquake Engineering NZ, August 1987, Vol. 3, p.237.

3. G Sanjayon, P.A. Darvall, "Dynamic Response of Softening Structures", ASCE Journal at Structural Engineering, Vol. 113, No. 6, June 1987.

4. S.A. Mahin, "Effects of Duration of Aftershocks on Inelastic Design EQ's", Proc. of 7th World Conference on EQ Engineering, Turkish National Committee, Vol. 5, September 1980, p.677.

5. S.E. Ruir, E. Rosenblueth, R. Diederich, "The Mexico Earthquake of September 19, 1965 - Seismic Response of Asymmetrically Yielding Structures", Earthquake Spectra, Vol. 5, No. 1, 1989.

6. R. Meli, J.A. Avila, "The Mexico Earthquake of September 19, 1985 - Analysis of Building Response", Earthquake Spectra, Vol. 5, No. 1, 1989.

7. P.J. Moss, A.J. Carr, A.H. Buchanan, "Seismic Response of Low Rise Buildings", Bulletin of NZNSEE, Vol. 19, No. 3, September 1986.

8. S.A. Marhin, V.V. Bertero, "An Evaluation of Inelastic Seismic Design Spectra", Journal of Structural Division, ASCE, Vol. 107, No. ST9, September 1981.

9. C.J. Montgomery, "Influence of P-Delta. Effects on Seismic Design", Canadian Journal of Civil Engineering, March 1981.

10. J.A. Dean, W.G. Stewart, A.J. Carr, "The Seismic Design of Sheathed Timber Frame Shear Walls", Pacific Conference on Earthquake Engineering, Wairakei NZ, August 1987, Proceedings Vol. 2.

11. J.A. Dean, A.H. Buchanan, "Seismic Design Loadings for Timber Structures", Pacific Conference on Earthquake Engineering, Wairakei NZ, August 1987, Proceedings Vol. 2.

12. J.A. Dean, W.G. Stewart, A.J. Carr, "The Seismic Behaviour of Plywood Sheathed Shearwalls", NZNSEE Bulletin Vol. 19, No. 1, March 1986.

13. SANZ, NZS 3101, Part 1: "Code of Practice for the Design of Concrete Structures" and Part 2: "Commentary on the Design of Concrete Structures". Standards Assocation of New Zealand, Wellington, New Zealand, 1982.

14. R.C. Fenwick, B.J. Davidson, "Dynamic Behaviour of Multi-Storey Buildings", Report No. 463, University of Auckland, School of Engineering, 1989. (Note : Participation factors and mode shapes were obtained from the authors in private correspondence).

15. Standards Association of NZ, "2nd Draft Code of Practice for General Structural Design and Design Loadings for Buildings", DZ 4203:1986.

16. Standards Association of NZ, "Code of Practice for General Structural Design and Design Loadings for Buildings", NZS 4203:1986.

17. F. Barda, "Shear Strength of Low Rise Walls With Boundary Elements", PhD Thesis Lehigh University, Pennyslvania, 1972.

18. F. Barda, J.M. Hanson and G. Lorley, "Shear Strength of Low Rise Walls With Boundary Elements", ACI Special Publication No. 53, 1977.

19. O. Hernandez, M.E. Zermeno de L, "Strength and Behaviour of Structural Walls with Shear Failure", 7th World Conference on Earthquake Engineering, Istanbul, Turkey, 1980.

20. J.K. Wright, "Earthquake Design Compared to Measured Response", Journal of Structural Engineering, Vol. 112, No. 1, January, 1986.

21. United States/Japan Joint Technical Coordinating Committee. "Interim Summary Report on Test of Seven Storey R.C. Building". Journal of Structural Engineering, Vol. 110, No. 10, October 1984.

22. U.S. Members of Joint Technical Coordinating Committee, "US-Japan Research : Seismic Design Implications", Journal of Structural Engineering, ASCE, Vol. 114, No. 9, September 1988.

23. A.E. Cardenas, D.D. Magura, "Strength of High Rise Shear Walls - Rectangular Cross Sections", Response of Multi-Storey Structures to Lateral Forces, ACI Special Publication SP36, Detroit, Michigan, 1973, p.119-150.

24. M.P. Collins and D. Mitchell, "A Rational Approach to Shear Design - The 1984 Canadian Code Provisions", ACI Journal, Nov-Dec, 1986.

25. C. Arnold, D. Hopkins, E. Elsesser, "Design and Detailing of Architectural Elements for Seismic Damage Control". Building Systems Development Inc., KRTA Ltd, Forell/Elsesser Engineers Inc., March 1987.

26. J.M. Ferrito, "Economics of Seismic Design for New Buildings", Journal of Structural Engineering, ASCE, December 1984.

27. J.B. Mander, J.M. Priestley, R. Park, "Observed Stress Strain Behaviour of Confined Concrete". Journal of Structural Engineering, ASCE Vol. 114, No. 8, August 1988.

28. Denis Wederell, "Wellington City Scope". Chaunter Publications Ltd, June 1990.

29. C. Loh, R. Ho, "Seismic Damage Assessment Based on Difference Hysteretic Rules". Journal of Earthquake Engineering and Structural Dynamics, Vol. 19, 1990.

30. R.D. De Cossio, E. Rosenblueth, "Reinforced Concrete Failures During Earthquakes". Journal of American Concrete Institute, November 1961.

31. Swiss Re, "Newcastle The Writing on The Wall", Swiss Reinsurance Company, Switzerland, 1990.

32. Munich Re, "Earthquake Mexico '85" - Published by Munich Reinsurance Company, 1986.

33. Swiss Re, "Small Earthquake Small Exposure?", Swiss Reinsurance Company, Switzerland, 1987.

34. EERI, "Armenia Earthquake Reconnaissance Report", Special Supplement to "Earthquake Spectra" - Journal of Earthquake Engineering Research Institute, August 1989.

35. US Department of Commerce, "San Fernando, California Earthquake of February 9, 1971", US Government Printing Office, Washington DC, Vol. 1, Part A, 1973.

36. Swiss Re, "A Short Guideline to Earthquake Risk Assessment - Swiss Reinsurance Company, 1982.

37. K. Saito, et al, "The Damage to Buildings due to Miyagiken-Oki Earthquakes of February 20 and June 12, 1978", Takenaka Technical Report, No. 21, April 1979.

38. "Damage to Various Structures Caused in 1978 by Earthquake off Miyagi Perfecture", TECHNOCRAT Vol. 2, No. 9, September 1978.

39. R. Husid et al, "The Lima Earthquake of October 3, 1974 : Damage Distribution", Bulletin of Seismological Society of America, Vol. 67, No. 5, October 1977.

40. J.A. Blume, M.H. Stauduhar, "Thesaloniki, Greece Earthquake, June 1978", EERI Reconnaissance Report, June 1979.

41. PCA, "The Behaviour of Reinforced Concrete Buildings Subjected to the Chilian Earthquakes of May 1960", Advanced Engineering Bulletin No. 6, Portland Cement Association, 1963.

42. G.V. Berg, J.L. Stratta, "Anchorage and Alaska Earthquake of March 27, 1964". American Iron and Steel Institute, New York, 1964.

43. R.I. Skinner, "Engineering Study of Caracas Earthquake, Venezuela, 29 July 1967". NZ Department of Scientific and Industrial Research Bulletin 191, 1968.

44. R. Diaz Pe Cossio, E. Rosenblueth, "Reinforced Concrete Failures During Earthquakes", ACI Journal, November 1961.

45. K.V. Steinbrugge, R. Flores, "The Chilean Earthquake of May 1960 : A Structural Engineering Viewpoint", Bulletin of Seismological Society of America, Vol. 53, No. 2, pp225, February 1963.

46. R.W. Anderson, "The San Salvador Earthquake of October 10, 1986 - Review of Building Damage", Earthquake Spectra - Journal of EERI Vol. 3, No. 3, August 1987.

47. F.M. Franz Sauter, "The San Salvador EQ of October 10, 1986 - Structural Aspects of Damage", Earthquake Spectra - Journal of EERI, Vol. 3, No. 3.

48. "The September 1985 Mexico Earthquakes - Final Report of the New Zealand Reconnaissance Team", NZNSEE Bulletin, Vol. 21, No. 1, March 1988.

49. EERI Newsletter, "Earthquake in Romania, March 4 1977", Newsletter EERI Vol. 2, No. 3B May 1977 (Also untitled draft report by MA Sozen dated 20 March 1977 held by WORKS).

50. R.B. Shephard et al, "The Loma Prieta, California, Earthquake of October 17, 1989 - Report of the NZNSEE Reconnaissance Team", Bulletin NZNSEE Vol. 23, No. 1, March 1990.

51. E. Rosenblueth, R. Meli, "The 1985 Earthquake : Causes and Effects in Mexico City". Report by Subcommittee on Norms and Construction Procedures of Committee for Mexico City's Metropolitan Area, National Reconstruction Commission, Concrete International, May 1986.

WPST1-3

Appendix A1:
List of Surveyed Wellington Buildings with 4 or More Storeys

STREET	NUMBER	OCCUPIER / OWNER / BLDG NAME	BUILT 19--	FLOOR AREA (*10 sq.m)	NUMBER OF STOREYS	ROLL & ASS/BAR		USE	CONSTRUCTION	IMPROVEMENTS	Number
** Date of Building Construction: 1935 to 1939											
* Type of Use: Commercial											
HERD ST	0	POST OFFICE (5TH FLOOR 1941)	37	750	4 to 6	17261	15100	8	CC	BLDGS (2) OI	1.00
LAMBTON QUAY	326	SOUTH BRITISH	36	160	4 to 6	17260	14800	84	CC	OFFICE RETAIL BLDG	1.00
MANNERS ST	11	INGRAM BLDING	37	108	4 to 6	17270	22600	80	CC	RETAIL OFFICE BLDG	1.00
MANNERS ST	125	TROJAN HOUSE	37	194	4 to 6	17270	24400	80	CM		1.00
THE TERRACE	136	THC FLATS	38	91	4 to 6	17260	29700	84	CC	OFFICE BLDG OI	1.00
WILLIS ST	161	INVINCIBLE HOUSE	35	102	4 to 6	17270	7900	80	BM	OFFICE BLDG OB OI	1.00
DIXON ST	64	DIXON BLDING	38	212	7 to 9	17270	19400	8	CC	WAREHOUSE BLDG	1.00
HUNTER QUAY	33	MLC HOUSE	39	410	7 to 9	17260	16500	84	XC	OFFICE BLDG	1.00
LAMBTON QUAY	330	THE COMMERCIAL BANK	35	190	7 to 9	17260	14900	84	CC	OFFICE BLDG OI	1.00
FEATHERSTON ST	131	FEATHERSTON HOUSE	37	380	10 & above	17260	19300	80	CH	OFFICE BLDG	1.00
* Subsubtotal *				2597							10.00
* Type of Use: Residential											
ABEL SMITH ST	152	RAHANA FLATS	37	124	4 to 6	17240	75600	92	CC	FLATS (9) OB OI	1.00
ORIENTAL PDE	212	ANSCOME FLATS LTD	37	95	4 to 6	17300	12700	92	CM	FLATS (5) OI	1.00
ORIENTAL PDE	280	WILKINSON ESTATE	37	90	4 to 6	17300	1600	92	CC	FLATS 8 OI	1.00
THE TERRACE	222		39	77	4 to 6	17240	32100	93	XI	MOTELS(15 UNITS) O/I	1.00
WATERLOO QUAY	29	WATERLOO HOTEL	36	720	7 to 9	17260	8800	94	CC	HOTEL OI	1.00
* Subsubtotal *				1106							5.00
** Subtotal **				3703							15.00
** Date of Building Construction: 1940 to 1949											
* Type of Use: Commercial											
GHUZNEE ST	11	THOMAS BULINGER BLDG	44	160	4 to 6	17270	41702	8	XX	WAREHOUSE/FACTORY BLDG	1.00
GHUZNEE ST	22	ATLAS HOUSE	44	270	4 to 6	17270	40700	80	CC	OFFICE WAREHOUSE BLDG	1.00
MOLESWORTH ST	127	WEST HAVEN	4	101	4 to 6	17230	11800	8	CC	OFFICES FLATS OI	1.00
PLIMMERS STEPS	5		4	105	4 to 6	17270	1400	80	CC	RETAIL / OFFICE BLDG	1.00
TORY ST	58	MCCARTHY G -EST	40	408	4 to 6	17280	11100	84	BM	OFFICE BLDG OI	1.00
WATERLOO QUAY	99	NZ RAILWAY BLDING	4	262	4 to 6	17260	1100	84	CX	ROUNDHOUSE	1.00
* Subsubtotal *				1306							6.00
* Type of Use: Residential											
BOULCOTT ST	84	A A INSURANCE LTD	4	127	4 to 6	17270	1300	92	CC	FLATS (16) OB OI	1.00
BROUGHAM ST	17	OWD TRAFFORD FLATS LTD	4	116	4 to 6	17310	6500	92	CH	FLATS (12) O/I	1.00
ORIENTAL PDE	118	SAVILLE	44	33	4 to 6	17300	21300	92	CC	FLATS 3 OI	1.00
ORIENTAL PDE	262	SUNHAVEN COURT - WGTN LTD	4	140	4 to 6	17300	1200	92	CC	11 FLATS OB OI	1.00
DIXON ST	134	DIXON ST FLATS	40	733	10 & above	17240	49400	92	CC	FLATS 117 OB OI	1.00

Appendix A1 (cont'd)

STREET	NUMBER	OCCUPIER / OWNER / BLDG NAME	BUILT 19--	FLOOR AREA (*10 sq.m)	NUMBER OF STOREYS	ROLL	& ASS/BAR	USE	CONSTRUCTION	IMPROVEMENTS	Number
* Subsubtotal *											
				1149							5.00
* Type of Use: Other											
HOSPITAL RD	0	STAFF RESID NO.3	44	60	4 to 6	17330	1200	42	CC	HOSPITAL BLDG O/B O/I	1.00
PIPITEA ST	4	WELLINGTON GIRLS COLL.	45	200	4 to 6	17720	37100	41	CC	SCHOOL BLDINGS	1.00
THE TERRACE	324	WINDEMERE	40	94	4 to 6	17240	51100	40	CC	FLATS (9) OB OI	1.00
HOSPITAL RD	0	STAFF RESID NO.2	44	74	7 to 9	17330	1200	42	CC	HOSPITAL BLDG O/B O/I	1.00
* Subsubtotal *											
				428							4.00
** Subtotal **											
				2883							15.00
** Date of Building Construction: 1950 to 1959											
* Type of Use: Commercial											
GHUZNEE ST	39	FREEMASONS BLDING	58	117	4 to 6	17270	42300	84	XX	RETAIL OFFICE BLDG	1.00
GILMER TCE	8	DB HOUSE	59	220	4 to 6	17260	16900	80	CC	OFFICE BLDING	1.00
DIXON ST	84	CASTROL HOUSE	59	246	7 to 9	17270	19900	8	CC	OFFICE/WAREHOUSE BLDG	1.00
FEATHERSTON ST	139	WOOL HOUSE	50	243	7 to 9	17260	19500	80	CC	RETAIL OFFICE BLDG	1.00
FEATHERSTON ST	187	AMP CHAMBERS	5	489	10 & above	17260	20202	80	CC	OFFICE BLDG	1.00
LAMBTON QUAY	126	MASSEY HOUSE (BUILD IN STAGES)	58	877	10 & above	17260	5400	8	XX	OFFICE BLDG	1.00
* Subsubtotal *											
				2192							6.00
* Type of Use: Industrial											
FREDERICK ST	11	D.N.WILSON & CO LTD	58	80	4 to 6	17290	37501	70	CC	WAREHOUSE OI	1.00
HAINING ST	16		57	55	4 to 6	17290	38300	7	GG	WAREHOUSE/ CLUBROOMS	1.00
TARANAKI ST	135	J. DICKENSON	56	46	4 to 6	17290	30000	77	CF	WAREHOUSE/OFFICE BLDGS	1.00
TORY ST	148	TRUSTEES GEORGE LEMMON TRUST	50	188	4 to 6	17290	31200	70	CF	WAREHOUSE O/I	1.00
WALTER ST	3	VARIOUS	50	513	4 to 6	17290	4600	70	CC	WAREHOUSE OFFICE BLDG OI	1.00
* Subsubtotal *											
				882							5.00
* Type of Use: Residential											
AUSTIN ST	10	KINGSGATE FLATS LTD	5	156	4 to 6	17310	9900	92	XX	FLATS 21 OB OI	1.00
CLAREMONT GR	4	VARIOUS	5	195	4 to 6	17310	17900	92	CA	FLATS (25) GARAGES (15)	1.00
MAARAMA CRES	20		5	65	4 to 6	17250	58300	92	CC	FLATS 20 OI	1.00
TARANAKI ST	152	MURRY SQUIRES MEMORIAL TRUST	50	271	4 to 6	17290	9800	95	CM	BLDGS OI	1.00
ORIENTAL PDE	276	WHARENUI APARTMENTS LTD	57	533	10 & above	17300	1500	92	CM	FLATS 40 OB OI	1.00
* Subsubtotal *											
				1220							5.00
** Subtotal **											
				4294							16.00

Appendix A1 (cont'd)

** Date of Building Construction: 1960 to 1969

* Type of Use: Commercial

STREET	NUMBER	OCCUPIER / OWNER / BLDG NAME	BUILT 19--	FLOOR AREA (*10 sq.m)	NUMBER OF STOREYS	ROLL & ASS/BAR		USE	CONSTRUCTION	IMPROVEMENTS	Number
BOWEN ST	0	CHARLES FERGUSSON WEST BLOCK	66	500	4 to 6	17240	43500	84	CC	OFFICE BLDING	1.00
BOWEN ST	84	BROADCASTING HSE	60	670	4 to 6	17240	43400	0			1.00
COURTENAY PL	38	INVINCIBLE HOUSE	65	155	4 to 6	17280	5500	80	CI	RETAIL OFFICE BLDG	1.00
DIXON ST	31	SCHAFLINE HOUSE	60	110	4 to 6	17270	39101	80	BC	COMMERCIAL BLDG OI	1.00
DIXON ST	25	MUTUAL BLDGS WGTN LTD	60	105	4 to 6	17270	39100	80	CC	BDLG OI	1.00
GHUZNEE ST	85	PROPERTY SECURITIES LTD	63	214	4 to 6	17270	43300	8	XX	OFFICE/WAREHOUSE OI	1.00
KENT TCE	80	N Z MASTER BUILDERS FED INC	60	261	4 to 6	17310	44700	80	CC	OFFICE BLDG O/I	1.00
LAMBTON QUAY	138	MACATHY TRUST BLDG	68	46	4 to 6	17260	5600	80	CC	OFFICE BLDG OI	1.00
TARANAKI ST	84	WINSTONE LTD	6	316	4 to 6	17270	38001	80	CA	OFFICE BLDG	1.00
THORNDON QUAY	181	RANKINE & HILL LTD	6	170	4 to 6	17220	33000	8	CI	WAREHOUSE OFFICES OB OI	1.00
THORNDON QUAY	125	ONGLEY BLDING	63	267	4 to 6	17220	31100	83	CC	WAREHOUSE OFFICE BLDG OI	1.00
THORNDON QUAY	218	VARIOUS	6	374	4 to 6	17220	30600	8	CC	W/HOUSE OFFICE BLDGS	1.00
VICTORIA ST	140	PPTA BLDING (1/2 - 1928)	63	112	4 to 6	17270	10600	80	XX	BLDGS (2) OI	1.00
WATERLOO QUAY	0	PORT OF WELLINGTON LTD	6	201	4 to 6	17261	54000	82	CI	OFFICE W/HOUSE BLDG	1.00
WILLIS ST	204	ARNOLD & WRIGHT LTD (ARURITE H)	60	252	4 to 6	17240	63900	80	CC	OFFICE WAREHOUSE BLDG OI	1.00
CUBA ST	108	WGTN. TRADE CENTRE (WTC5)	65	1590	7 to 9	17270	31100	80	XX	CARPARK & OFFICES	1.00
BRANDON ST	26	CENTRAL HOUSE LTD	62	235	7 to 9	17260	24100	80	XC	OFFICE RETAIL BLDG	1.00
CUSTOMHOUSE QUAY	111	VARIOUS	63	495	7 to 9	17260	21600	84	CC	BLDG	1.00
FEATHERSTON ST	166	ROYAL INSURANCE BLDING	63	303	7 to 9	17260	18500	84	XC	OFFICE BLDG	1.00
KENT TCE	16	CUBE W & N BLDING	65	280	7 to 9	17310	401	84	CC	OFFCIE BLDG O/I	1.00
KENT TCE	32	MOTOR TRADE OFFICES LTD	60	300	7 to 9	17310	1500	84	CC	OFFICE BLDG O/I	1.00
LAMBTON QUAY	182	NAT MUTUAL LIFE ASSOC	61	334	7 to 9	17260	6300	80	CC	OFFICE BLDG	1.00
LAMBTON QUAY	298	TAB	65	632	7 to 9	17260	14500	80	CC	OFFICE BLDG OI	1.00
MOLESWORTH	101	MOLESWORTH HOUSE	65	1000	7 to 9	17220	49900	84	CM	OFFICE BLDING OI	1.00
MOLESWORTH ST	55	ICI HOUSE	66	565	7 to 9	17230	35900	80	CC	OFFICE BLDG OI	1.00
THE TERRACE	104	UOC TOWER	66	327	7 to 9	17260	28900	84	CC	OFFICE BLDG O/I	1.00
THE TERRACE	145	JAMES C. CAR PARK	62	400	7 to 9	17260	10500	8	CC	PARKING BLDG	1.00
VICTORIA ST	81	CONFERENCE CHAMBERS	6	180	7 to 9	17270	16400	80	CC	RETAIL OFFICE BLDG	1.00
VIVIAN ST	130	ROSTREVOR HOUSE	60	245	7 to 9	17270	45700	80	XC	WAREHOUSE/OFFICE BLDG OI	1.00
WAKEFIELD ST	138	ANVIL HOUSE	60	632	7 to 9	17270	17300	84	CX	OFFICE BLDG	1.00
WARING TAYLOR ST	26	LAW SOCIETY BLDG	66.	254	7 to 9	17260	9901	84	CM	OFFICE BLDG	1.00
WILLIS ST	181	WESTBROOK HOUSE	66	287	7 to 9	17270	24900	80	CC	OFFICE BLDG O/I	1.00
BOWEN ST	0	BOWEN STATE	61	2000	10 & above	17240	43500	86	CC	OFFICE BLDING	1.00
CUBA ST	108	WGTN. TRADE CENTRE (WTC6)	69	510	10 & above	17270	31100	84	XX	OFFICE BLDG	1.00
FEATHERSTON ST	149	SUN ALLIANCE INSURANCE LIMITED	6	335	10 & above	17260	19700	84	CC	OFFICE BLDG	1.00
FEATHERSTON ST	153	NATIONALS MUTUAL	64	835	10 & above	17260	20000	80	CC	OFFICE RETAIL BLDG	1.00
FEATHERSTON ST	170	NATIONAL BANK	69	1360	10 & above	17260	18800	80	CC	RETAIL/OFFICE BLDG	1.00
LAMBTON QUAY	116	LOCAL GOV BLDING	67	432	10 & above	17260	5200	80	XC	OFFICE BLDG	1.00
LAMBTON QUAY	120	MAINCHESTER UNITY	64	404	10 & above	17260	5300	80	CX	OFFICE BLDG	1.00
MOLESWORTH ST	95	FEDERATION HOUSE	64	275	10 & above	17230	10700	80	CC	OFFICE BDLG O/I	1.00
MULGRAVE ST	9	VOGEL BLDING	6	1600	10 & above	17220	53400	84	CC	OFFICE BLDG OI	1.00
THE TERRACE	70	NAT MUTUAL LIFE ASSOC	69	1000	10 & above	17260	28400	84	CC	OFFICE BLDG O/I	1.00
THE TERRACE	81	BORTHWICK HSE	69	644	10 & above	17260	3100	84	CC	OFFICE BLDG	1.00

Appendix A1 (cont'd)

STREET	NUMBER	OCCUPIER / OWNER / BLDG NAME	BUILT 19--	FLOOR AREA (*10 sq.m)	NUMBER OF STOREYS	ROLL & ASS/BAR		USE	CONST RUCTI ON	IMPROVEMENTS	Number
THE TERRACE	106	CABLE PRICE DOWNER LTD (UDC)	63	557	10 & above	17260	29000	84	CC	OFFICE BLDG O/I	1.00
THE TERRACE	120	TERRACE CHAMBERS	69	509	10 & above	17260	29200	84	CC	OFFICE BLDG OI	1.00
WAKEFIELD ST	126	VARIOUS	6	348	10 & above	17270	17100	84	CC	OFFICE BLDG	1.00
WARING TAYLOR ST	38	GEN ACCIDENT FIRE & LIFE CORP	66	631	10 & above	17260	10100	84	CF	OFFICE BLDG	1.00
WHITMORE ST	17	INVESTMENT HOUSE	67	758	10 & above	17260	9301	80	CC	COMMERCIAL OFFICE BLDG	1.00
* Subsubtotal *											
				24010							48.00
* Type of Use: Industrial											
GARRETT ST	22	SECURITY EXPRESS HSE	64	203	4 to 6	17270	44500	70	CC	WAREHOUSE BLDG OI	1.00
THORNDON QUAY	13	GOV. PRINT	67	1720	4 to 6	17220	45800	77	CC	W/HSE OFFICE BLDG OI	1.00
VICTORIA ST	215	APPAREL HOUSE	60	126	4 to 6	17290	3500	70	CF	WAREHOUSE OFFICE BLDG OI	1.00
* Subsubtotal *											
				2049							3.00
* Type of Use: Residential											
AUSTIN ST	123	LANDSCAPE APARTMENTS LTD	6	136	4 to 6	17310	67100	92	CC	FLATS (10) OI	1.00
BROUGHAM ST	72		6	126	4 to 6	17310	5700	92	CC	FLATS (14) O/B O/I	1.00
LEVY ST	5		6	117	4 to 6	17310	23600	92	CC	FLATS (14) O/B O/I	1.00
LEVY ST	20		6	91	4 to 6	17310	4500	92	CC	FLATS (10) O/I	1.00
MAJORIBANKS ST	38		6	83	4 to 6	17310	16400	92	CC	FLATS 16 OI	1.00
MONCRIEFF ST	2		6	44	4 to 6	17310	3300	92	CI	FLATS (9)	1.00
ORIENTAL PDE	116	ROCKHAVEN APARTMENTS LTD	60	35	4 to 6	17300	21200	92	CC	FLATS 5 OI	1.00
PATANGA CRES	24		6	64	4 to 6	17230	63101	92	CC	FLATS (9) O/I	1.00
SALAMANCA RD	88	GREENMANTLE FLATS	68	117	4 to 6	17240	28901	92	CI	FLATS 10 OI	1.00
THE TERRACE	179	ADELPHI APARTMENTS LTD	6	84	4 to 6	17240	34800	92	CI	FLATS 14 OI	1.00
THE TERRACE	217	AVON APARTMENTS LTD	60	32	4 to 6	17240	36000	92	CI	FLATS 16 OI	1.00
THE TERRACE	257	WAIKITE FLATS	60	158	4 to 6	17240	37900	92	CX	FLATS (20) OB OI	1.00
TINAKORI RD	340,	DORAE PROPERTIES LTD	68	59	4 to 6	17230	48100	92	CC	FLATS 8 OI	1.00
TINAKORI RD	374		6	98	4 to 6	17230	49600	92	CC	FLATS (12) O/I	1.00
ORIENTAL PDE	178	VARIOUS	6	84	7 to 9	17300	11500	92	CI	FLATS 5 OI	1.00
ORIENTAL PDE	202	CLIFTON	62	123	7 to 9	17300	12500	92	CC	FLATS 9 OI	1.00
ORIENTAL PDE	236	KENSINGTON	6	255	7 to 9	17300	13900	9	CX	FLATS 22 SHOPS 3 OI OB	1.00
THE TERRACE	152	HUME INDUSTRIES -N Z- LTD	66	400	7 to 9	17260	30100	94	CC	OFFICE BLDG OI	1.00
THE TERRACE	314	GORDON WILLIAMS FLATS	63	712	7 to 9	17240	50800	92	CC	FLATS (115) OB OI	1.00
WILLIS ST	355	QUALITY INN WILLIS ST	64	699	7 to 9	17290	2500	94	CX	CONFERENCE CENTRE	1.00
ABEL SMITH ST	131	ASTON TOWERS LTD	62	471	10 & above	17250	49500	92	CC	FLATS 50 OB OI	1.00
COTTLEVILLE TCE	19	GROSVENOR FLATS	6	327	10 & above	17220	1300	92	CI	FLATS (40) OB OI	1.00
GRANT RD	121	BIRCHINGTON COURT LTD	6	258	10 & above	17230	900	92	CC	FLATS (31) O/B O/I	1.00
ORIENTAL PDE	40	BAY PLAZA HOTEL	6	353	10 & above	17300	18700	94	CC	MOTOR HOTEL O/B O/I	1.00
ORIENTAL PDE	144	DORCHESTER	69	139	10 & above	17300	22500	92	CI	FLATS 10 OB OI	1.00
ORIENTAL PDE	214	BROADWATER APARTMENTS LTD	64	242	10 & above	17300	12800	92	CC	FLATS 8 OI	1.00
ORIENTAL PDE	248	ORIANA	63	147	10 & above	17300	500	92	CI	FLATS (10) O/B O/I	1.00
ORIENTAL TCE	20	JERNINGHAM APPARTMENTS LTD	6	819	10 & above	17300	15200	92	CC	69 FLATS OB OI	1.00
THE TERRACE	186	HERBERT GARDENS	65	735	10 & above	17240	30800	92	CI	FLATS (55) OB OI	1.00
THE TERRACE	191	JELLICOE	66	244	10 & above	17240	35300	92	CC	FLATS 18 OI	1.00

Appendix A1 (cont'd)

STREET	NUMBER	OCCUPIER / OWNER / BLDG NAME	BUILT 19--	FLOOR AREA (*10 sq.m)	NUMBER OF STOREYS	ROLL & ASS/BAR		USE	CONSTRUCTION	IMPROVEMENTS	Number
TINAKORI RD	16	NEWMAN COURT LTD	6	299	10 & above	17220	6800	92	CC	FLATS (32) O/B O/I	1.00
* Subsubtotal *											
				7551							31.00
* Type of Use: Other											
FREDERICK ST	7	LICHFIELD HOUSE	60	90	4 to 6	17290	37500	32	CC	CARPARK OI	1.00
HOSPITAL RD	0	SEDDEN ANNEX	62	45	4 to 6	17330	1200	42	CC	HOSPITAL BLDG O/B O/I	1.00
HOSPTIAL RD	0	SEDDON BLOCK	62	90	4 to 6	17330	1200	42	CC	HOSPITAL BLDG O/B O/I	1.00
GILMER TCE	2	WILLIAMS BLDING C. PARK	67	1000	7 to 9	17260	16800	32	CC	CAR PARKING	1.00
PIPITEA ST	4	WELLINGTON GIRL'S COLLEGE	68	726	7 to 9	17220	37100	41	CC	SCHOOL BLDGS OB OI	1.00
THORNDON QUAY	161	DALFO HOUSE LTD	62	533	10 & above	17220	32400	41	CC	WAREHOUSE OFFICES OI	1.00
* Subsubtotal *											
				2484							6.00
** Subtotal **											
				36094							88.00
** Date of Building Construction: 1970 to 1975											
* Type of Use: Commercial											
DIXON ST	99	EXIM ASSOCIATES LTD	73	561	4 to 6	17270	40200	8	CI	BLDGS (2) O/I	1.00
GARRETT ST	17	MARITIME HUNTS HSE	72	115	4 to 6	17270	45000	84	CI	OFFICE BLDG OI	1.00
GHUZNEE ST	35	GEORGE JEFFERY & CO LTD	74	175	4 to 6	17270	42200	84	CI	RETAIL OFFICE BLDG	1.00
LAMBTON QUAY	354	AUCKLAND BLG SOCIETY HSE	71	147	4 to 6	17270	2000	80	CI	OFFICE/RETAIL BLDG	1.00
BOULCOTT ST	93	NEWSPAPER HOUSE	74	405	7 to 9	17240	39600	8	CI	OFFICE BLDG OI	1.00
GHUZNEE ST	75	GENERAL PROPERTIES HOUSE	75	400	7 to 9	17270	43100	84	II	OFFICE BLDG OI	1.00
MANNERS ST	49	REGENT TAVERN	71	231	7 to 9	17270	23300	80	CI	RETAIL OFFICE BLDG	1.00
MANNERS ST	141	NATIONAL MUTUAL LIFE ASS LTD	74	348	7 to 9	17270	24600	80	CC	OFFICE BLDG	1.00
MOLESWORTH ST	123	ROSS MORE HOUSE	71	356	7 to 9	17230	11700	84	CC	OFFICE BLDG OI	1.00
PANAMA ST	22	VARIOUS	72	173	7 to 9	17260	24200	80	CC	OFFICE RETAIL BLDG	1.00
WILLIS ST	164	PROPERTY TRADING CO LTD	71	260	7 to 9	17240	41100	84	CC	OFFICE BLDG	1.00
WILLIS ST	219	PEARSE HSE	73	511	7 to 9	17270	25900	84	CM	OFFICE BLDG O/I	1.00
BOULCOTT ST	69	ANSETT HOUSE	75	240	10 & above	17240	39200	81	CX	OFFICE BLDG	1.00
BOWEN ST	0	CHARLES FERGUSSON	75	1500	10 & above	17240	43500	84	CC	OFFICE BLDING	1.00
BUNNY ST	20	RUTHERFORD HOUSE	73	1500	10 & above	17260	6400	80	CA	OFFICE BLDG	1.00
CUSTOMHOUSE QUAY	20	B P HOUSE	70	1450	10 & above	17260	20400	84	CC	OFFICE BLDG	1.00
FEATHERSTON ST	142	COMMERCIAL UNION H.	72	562	10 & above	17260	18100	84	CC	OFFICE BLDG O/I	1.00
FEATHERSTON ST	109	AMP SOCIETY	72	1600	10 & above	17260	8600	80	CC	OFFICE BLDG	1.00
GILMER TCE	2	WILLIAMS BUILDING	75	2400	10 & above	17260	16700	80	CC	RETAIL OFFICE	1.00
LAMBTON QUAY	140	MCCARTHY TRUST BLDING	70	800	10 & above	17260	5700	80	CC	BLDGS 2 OI	1.00
LAMBTON QUAY	318	WESTPAC BANKING CORPORATION	74	1340	10 & above	17260	14700	80	CC	OFFICE BLDG OI	1.00
MOLESWORTH ST	85	AORANGI HOUSE	73	681	10 & above	17230	10400	80	CC	OFFICE BLDG OI	1.00
MULGRAVE ST	51	FRYBERG BLDGS	74	1400	10 & above	17220	50200	84	CM	OFFICE BUILDING	1.00
MURPHY ST	15	DOMINION BREWERY	71	1650	10 & above	17220	20000	8	XX	BLDGS OI	1.00
THE TERRACE	126	ICI HOUSE	72	467	10 & above	17260	29300	84	CC	OFFICE BLDG OI	1.00
THE TERRACE	114	DALMUIR HOUSE	70	670	10 & above	17260	29100	84	CC	OFFICE BLDG OI	1.00
THE TERRACE	163	NATIONAL MUTUAL LIFE ASSOC	75	374	10 & above	17260	11400	84	CC	OFFICE BLDG O/I	1.00

Appendix A1 (cont'd)

STREET	NUMBER	OCCUPIER / OWNER / BLDG NAME	BUILT 19--	FLOOR AREA (*10 sq.m)	NUMBER OF STOREYS	ROLL & ASS/BAR		USE	CONSTRUCTION	IMPROVEMENTS	Number
THE TERRACE	171	DATA BANK HOUSE	74	1200	10 & above	17260	11600	84	CI	OFFICE BLDG OI	1.00
THE TERRACE	155	IBM BLDING	70	2160	10 & above	17260	11000	84	CI	OFFICE BLDG OI	1.00
VICTORIA ST	154	FELTEX N Z LTD	72	561	10 & above	17270	27300	80	CI	RETAIL OFFICE BLDG OB OI	1.00
WILLIS ST	178	EDUCATION HOUSE LTD	75	900	10 & above	17240	62900	84	CC	OFFICE BLDGS (2) OI	1.00
* Subsubtotal *											
				25137							31.00
* Type of Use: Industrial											
WILLIS ST	237	CUMBERLAND HOUSE	71	1200	10 & above	17270	26400	70	CI	OFFICE WAREHOUSE BLDG	1.00
* Subsubtotal *											
				1200							1.00
* Type of Use: Residential											
EVERTON TCE	10	EVERTON HALL (3 & 4 STOREY)	75	266	4 to 6	17240	20900	92	BI	FLATS (22) OB OI	1.00
MAURICE TCE	18	VICTORIA HOUSE	73	256	4 to 6	17240	67501	95	CI	HOSTEL OI	1.00
ORIENTAL PDE	92	QUALITY INN (PART 1973)	71	666	7 to 9	17300	20400	94	CI	HOTEL OI	1.00
WILLIS ST	169	TAS HOTEL	74	460	7 to 9	17270	8000	94	CC	HOTEL OFFICE OI	1.00
BROUGHAM ST	131	MELKSHAM TOWERS	73	262	10 & above	17310	57200	92	CC	FLATS (36) O/I	1.00
GRANT RD	1	MANSFEILD TOWERS	72	455	10 & above	17220	1000	92	CI	FLATS 46 OI	1.00
HOBSON ST	70	HOBSON COURT	75	800	10 & above	17220	33100	92	CC	FLATS (83) O/I	1.00
* Subsubtotal *											
				3165							7.00
* Type of Use: Other											
BOND ST	28	LOMBARD CARPARK	70	1350	7 to 9	17270	18800	32	CC	PARKING BLDG	1.00
FEATHERSTON ST	70	CENTRAL TX (STRENGTHENED)	72	630	7 to 9	17260	7900	61	CI	TELEPHONE EXCHANGE	1.00
* Subsubtotal *											
				1980							2.00
** Subtotal **											
				31482							41.00
** Date of Building Construction: 1970s											
* Type of Use: Commercial											
LORNE ST	22	HUTCHWILCO LTD	7	198	4 to 6	17280	23800	84	CI	OFFICE W/HOUSE BLDG OI	1.00
* Subsubtotal *											
				198							1.00
* Type of Use: Residential											
DERBY ST	4	VARIOUS	7	65	4 to 6	17310	74300	92	CI	FLATS (11) OI	1.00
SCARBOROUGH TCE	4	VARIOUS	7	50	4 to 6	17310	73800	92	CI	FLATS (8) OI	1.00
THE TERRACE	367	MIET ANNETTE MARIE	7	28	4 to 6	17240	56400	92	CM	FLATS 11 OB OI	1.00
* Subsubtotal *											
				143							3.00

Appendix A1 (cont'd)

STREET	NUMBER	OCCUPIER / OWNER / BLDG NAME	BUILT 19--	FLOOR AREA (*10 sq.m)	NUMBER OF STOREYS	ROLL & ASS/BAR	USE	CONSTRUCTION	IMPROVEMENTS	Number
** Subtotal **				341						4.00
*** Total ***				78797						179.00

APPENDIX A.2

TYPICAL BUILDING EXAMPLES : POTENTIAL DEFICIENCIES

A range of typical examples of Wellington's 1935 to 1975 building stock were evaluated to identify common potential deficiencies. The results of these case studies are discussed and summarised in Section 2.3 of the report. Detailed findings from the case studies are as follows:

Case A

Building is seven storey frame structure completed in late 1960s.

Through light by modern standards stirrups and ties in beams and columns are closely spaced providing reasonable confinement. Main potential defects appears to be the provision of only light steel in beam column joints and cut off of beam flexural steel close to columns.

Case B

Building is a 10 storey frame structure completed in late 1960s.

Main potential defect in frames is the lack of shear strength in the beams. Cut off beam flexural steel close to the columns may force plastic hinging into the zone where beam stirrups are widely spaced.

Beam column joints are almost completely unreinforced and presence of concrete block infill panels will concentrate ductility demand in some parts of the frames.

Case C

Building is a five storey haunched beam frame and perforated wall (deep member frame) structure completed in the late 1930s.

Potential defects include the lack of adequate shear or confining steel in the beams and columns. Failure is likely to be by shear in columns of both the haunched beam and deep member frames. Failure may also occur in the beam/column joints due to lack of joint steel or by shear in the beams due to wide spacing of the stirrups.

Case D

This is a seven storey building with boundary shear walls providing lateral resistance in the longitudinal direction and a shear wall and frames providing resistance in the transverse direction. Construction of the building was completed in the mid 1960s.

In the transverse direction the shear wall is expected to rock on its foundation while plastic hinges or shear failures develop in the columns of the frames. Stresses in the stahlton floor slab topping may cause diaphragm failure leading to a highly torsional response and collapse of the frames remote from the transverse shear wall.

Case E

An eight storey building constructed in the early 1960s with shear walls in the longitudinal direction and frames in the transverse direction.

The principal potential defects includes lack of adequate shear reinforcement in the columns and beam/columns joints of the transverse frames. There is also the potential for a soft storey column hinging collapse mechanism to develop.

Shear walls in the longitudinal direction are tapered down near their bases and are expected to fail in shear.

Case F

A nine storey building built in the mid 1960s with shear walls in the transverse direction and frames in the longitudinal direction. There is also a core with walls effective in each direction.

Generally the shear capacity of beam, column and wall elements does not match their flexural capacity and beam/column joints are also expected to fail before the adjacent members.

Case G

A 12 storey building constructed in mid 1960s with a regular form of shear walls and a perimeter frame with close centred columns. Building is well detailed for its time but column shear failures are still the likely failure mode.

Case H

A 14 storey building constructed in the late 1960s. Seismic resistance provided by shear walls in the longitudinal direction and shear walls and frames in the transverse direction. Non ductile detailing used throughout with only nominal steel in elements such as the coupling beams between walls.

Case I

A five storey building constructed in late 1950s. Perimeter frame is perforated wall/frame type. This acts in conjunction with lift and stair shafts and other frames on the interior of the building.

The lift shaft is expected to rock on its foundation in the transverse direction but fail in shear at ground floor level in the longitudinal direction. As a consequence the ductility demand is expected to be concentrated in the ground floor frame elements in the longitudinal direction but distributed up the building in the transverse direction.

Frame elements have non ductile detailing and shear and/or bond failures due to the use of plain bars is to be expected. Brick infill panels represent a local hazard.

Case J

A 15 storey building constructed in late 1960s with a large shear core and podium wall structural system resisting seismic loads.

Shear failure or, possibly, rocking of podium walls is expected. The core walls will fail in shear rather than flexure even when higher mode amplification of the wall shear forces is ignored.

Case K

A 14 storey shear wall building constructed during late 1950's early 1960s.

The walls are expected to fail in shear and in one direction have only one third the shear strength that would be required by current design codes.

Case L

A 10 storey building built early 1970s designed to ACI 318-71 design code.

This is a shear wall building with architectural fins supporting part of the floor slabs. In the longitudinal direction the walls will rock on their foundations. Resulting deformations imposed on the fin/columns may cause them to separate from the floor slabs and result in at least partial collapse. In the transverse direction failure of the shorter shear walls in shear at ground floor level is expected rather than cracking. The resulting shear deformations may cause a shear failure in the fin/columns.

Case M

A 12 storey building built late 1960s early 1970s with lateral resistance in the longitudinal direction provided by perimeter frames and by shear walls in the transverse direction. Detailing generally conforms to current requirements for limited ductility except for dimensional limitations. Transverse shear walls expected to fail in shear above the podium level or through the piles supporting the walls at foundation level.

Case N

A 10 storey frame structure with brick and reinforced concrete infill panels, constructed in the early 1950s.

Detailing exceptional for its day with closely spaced ties in beam and columns (including joint zones!). Columns generally stronger than beams except for exterior frames/walls. Infill walls can be expected to concentrate ductility demand.

Case O

A 10 storey building constructed in the early 1960s. Corner building with interior boundary walls on adjacent sides. High torsions mean that ductility demand will be concentrated on street frontages in the columns of deep spandrel frames. The columns are not detailed for ductility.

Case P

A 14 storey two way frame building constructed in early 1970s. Design used working stress approach and detailing is non ductile.

Case Q

A six storey frame and shear wall building constructed in the late 1950s early 1960s.

There are no beam/columns joint ties, ties are widely spaced in the columns and the deep spandrels beams will force failure into the columns of the external frames.

Case R

A six storey building constructed in late 1950s. A corner building with adjacent interior boundary walls and a stair tower on the street corner. Flat slab floors are supported on interior columns. Detailing is non ductile.

Case S

A four storey building constructed in early 1960s. Highly torsional response due to boundary wall on adjacent interior faces of the building will concentrate ductility demand in frames located on the street frontages.

Failure will be in the columns and/or beam column joints which have insufficient confining steel to be ductile.

Case T

Four 16 storey buildings constructed in late 1960s and early 1970s. All have substantial shear cores and peripheral gravity frames which make varying though small contribution to the seismic resistance.

Case X

A nine storey building constructed in mid 1960s with shear walls resisting lateral loads in both directions. Some shear walls are offset at 1st floor and, above 1st floor, external walls are perforated with deep spandrels. External perforated wall/frames and internal gravity frames have non ductile detailing (e.g. no beam/column joint ties). 1st floor diaphragm is not designed or detailed for its required function as a transfer diaphragm for the offset shear walls.