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April 1997 Analysis Report
P.G.C. Building
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CHRISTCHURCH DRAINAGE BOARD
OFFICE BUILDING IN CAMBRIDGE TCE

SEISMIC EVALUATION OF EXISTING BUILDING

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EXECUTIVE SUMMARY

The Christchurch Drainage Board office building in Cambridge Terrace has been evaluated for earthquake effects based on the requirements of NZS4203. The evaluations has shown that:

1. Column plastic rotations in the gravity columns exceed their capacity for earthquakes with a return period of from 35 to 70 years (one-third to one-half NZS4203 loading). The consequences of this are severe as the columns would lose gravity support capabilities leading to extensive collapse.

2. Wall shear cracking also initiates at relatively low loads. Cracking is generally limited to coupling beams and around openings. This cracking would lead to permanent damage but the consequences are not as severe as column damage as the wall portions support only small tributary areas of gravity load.

An alternative gravity load support should be provided as a matter of some urgency given the small return period for severe damage and the consequences of this damage.

Strengthening the shear walls be adding concrete to the wall face will reduce damage to the walls but not eliminate it unless all walls are strengthened. Wall strengthening will not significantly reduce the danger of column collapse as foundation rocking and wall flexural yielding imposes rotations on columns regardless of wall shear strength.

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1. INTRODUCTION

The Christchurch Drainage Board office building is a reinforced concrete structure 5 storeys high.

The owner of the building is assessing risk to its assets and as part of this assessment the earthquake safety of the building is being evaluated. This report examines the earthquake performance of the building.

The procedure for this evaluation was as follows:

1. Develop a finite element model of the building, in as-is condition, using drawings as supplied. The model included potential flexural yielding in frame elements, shear degradation in walls and uplift of foundation elements.

2. Prepare earthquakes suitable for the level of seismic risk in Christchurch. A single pair of NZS4203 compatible records was used for this evaluation.

3. Evaluate the damage to the building for increasing levels of seismic load up to the design level earthquake.

4. Report damage patterns versus earthquake level and areas of particular vulnerability.

1.1 Methodology

The New Zealand National Society for Earthquake Engineering (NZNSEE) has published (1996) a draft document entitled "*The Assessment and Improvement of the Structural Performance of Earthquake Risk buildings*". [Reference 1]. This provides procedures for the detailed assessment of structures.

The document provides detailed procedures for assessing the capacity of reinforced concrete elements. However, it does not provide detailed mthods for calculating the demand on buildings such as this.

To calculate demand a detailed non-linear finite element of the building was developed and time history analyses were performed to obtain maximum forces and deformations on all the elements of the building. The requirements of the Loadings Code, NZS4203 [2], were followed in performing the time history analysis. The results from these analyses were then used to evaluate the adequacy of the structural components using the NZNSEE guidelines.

2. STRUCTURAL MODEL

The building was modelled using the computer program ANSR-II [3], a general purpose nonlinear analysis program developed at the University of California, Berkeley. The model was an assemblage of nonlinear elements representing the foundations, shear walls, columns and beams. The general form of the model was similar to that used for unreinforced masonry buildings [4] with appropriate changes to the material properties.

2.1 Model Concept

The basis for non-linear analysis is to identify potential sources of nonlinearity in the building and develop the computer model to capture as accurately as possible the force-deflection reponse of these elements. For this building, potential sources of non-linearity are:

- *Pad foundations and spread footings.* Resistance to overturning is provided only by the weight of the structure itself and so uplift is possible. To model potential uplift, each node at basement level is connected to the ground with a gap element. Gap elements are linear elastic in compression but have no tension capacity. When uplift forces exceed the gravity compression force the elements resist zero load.

- *Shear Wall Flexural Yielding.* The shear wall reinforcing is generally constant with height and so any wall plastic hinges are likely at the point of maximum moment, the base of the wall.

- *Shear Wall Shear Degradation.* Depending on the level of shear stress, the stiffness and/or the strength of the wall in shear may reduce.

- *Column Plastic Hinging.* The columns may yield depending on the level of concurrent axial load and bending moment about each axis.

- *Beam Plastic Hinging.* The beams may yield depending on the level of bending moment.

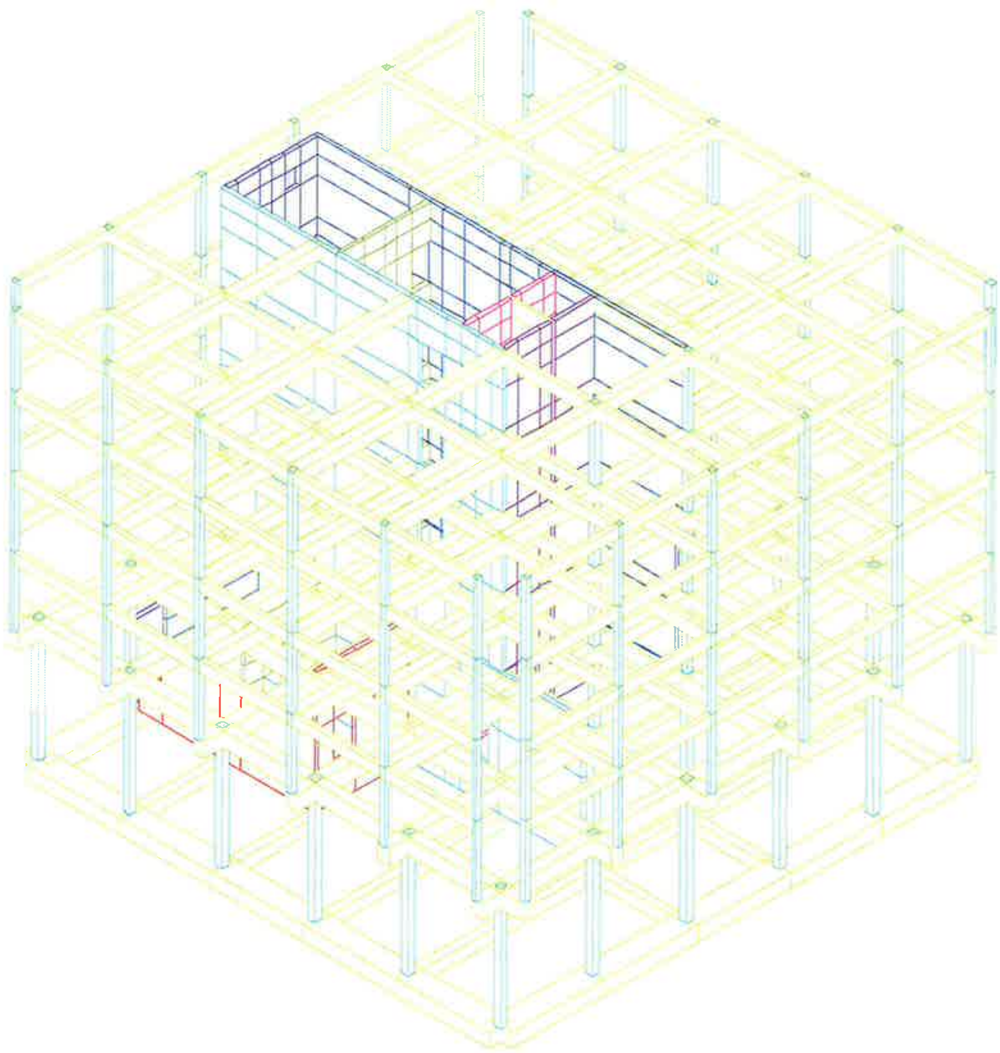
The ANSR-II model was developed to incorporate these sources of non-linearity.

2.2 Geometry

The drawings supplied were well dimensioned and the geometry was based on the centreline dimensions from the drawings. The plan shape was defined by 113 nodes, describing the locations of columns, walls and major openings in the walls. In the vertical dimension nodes were located at each floor level and at 2 or 3 intermediate locations within each storey to enable openings to be defined.

Figure 2.1 shows the model developed in AutoCAD.

FIGURE 2.1 : ANALYSIS MODEL DISPLAYED IN AUTOCAD



2.3 Walls

The walls were modelled as plane stress elements with thicknesses corresponding to the values specified on the drawings. The base of each wall was modelled to incorporate flexural yielding. The shear stiffness of the elements included degradation in strength and stiffness depending on the level of shear stress.

2.3.1 Flexural Yielding

To model flexural yielding each wall node at Ground Floor Level was connected to the wall node below by two elements in parallel, one to model the concrete compression block and the second to model the reinforcing steel.

The concrete compression block was modelled as a gap element, with an area equal to the total tributary area of concrete to that node. The reinforcing steel was modelled as a truss element with an area equal to the total tributary area of vertical reinforcing steel to that node. The truss element was described as bi-linear in compression and tension with a yield strength of 250 MPa.

To define the stiffness of the joint an element length is required. This was set as 50 mm for all walls. The stiffness of each element is then defined as EA/L where A is the area of concrete or steel, E is the appropriate Elastic Modulus and L is the defined 50 mm length.

2.3.2 Shear Strength

The plane stress element elastic stiffness was based on a shear modulus of 10,000 MPa (based on 25 MPa concrete) times 0.80 to provide an effective shear area, as recommended by NZS3101 [Reference 6].

The shear strength of the concrete and horizontal reinforcing was calculated as summarised in Table 2.1. The calculated values of v_c and v_s were then used to define a strength envelope as shown in Figure 2.2.

A strength envelope was developed from published test results for walls without ductile detailing as given in the book by Paulay & Priestley, [Reference 7, Figure 8.4] which is reproduced as Figure 2.3. The calculated values of v_c and v_s define the stress levels on the strength envelope as shown in Figure 2.2.

The walls are linear elastic until the calculated shear strength of the concrete is reached. At this stage the stiffness reduces such that a strength level of $v_c + v_s$ is reached at a strain of 0.0045. At that stage strain hardening occurs so that the steel strength increases by 25% at a strain of 0.0030. Once this peak is reached the strength reduces with increasing strain.

The strength degradation beyond strains of 0.0030 produces a negative stiffness. The wall is in unstable equilibrium and wall collapse will occur if the seismic deformation continues in the same direction and the wall supports vertical load.

The form of the hysteresis envelope is a conservative representation of the envelope shown in Figure 2.3. Figure 2.4 compares the nonlinear model used with the experimental envelope.

TABLE 2.1 : WALL SHEAR STRENGTHS

	6" Walls (152 mm)	8" Walls (203 mm)
Typical Reinforcing		
Horizontal	1/2" @ 12" C	5/8" @ 15" C
Vertical	1/2" @ 12" C	5/8" @ 15" C
ρ_w	0.0027	0.0025
$v_c = (0.07 + 10\,\rho_w)\,\sqrt{f'_c}$	0.48MPa	0.48 MPa
$v_s = \rho_w\,F_{yt}$	0.68 MPa	0.64 MPa

FIGURE 2.2 : HYSTERESIS OF RESPONSE OF SQUAT WALL (from Reference 7, Figure 8.4)

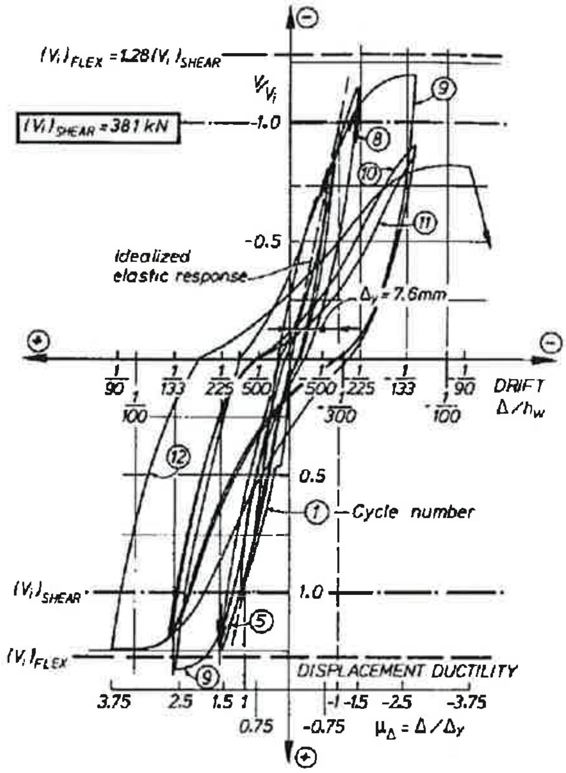


FIGURE 2.3 : WALL SHEAR STRENGTH ENVELOPE

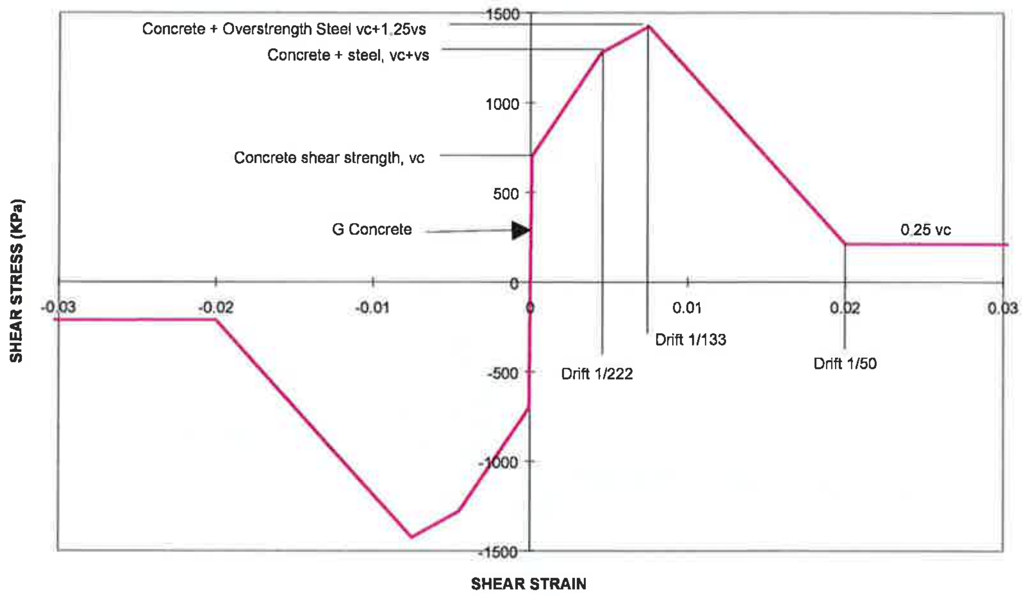


FIGURE 2.4 : COMPARISON OF MODEL WITH EXPERIMENTAL

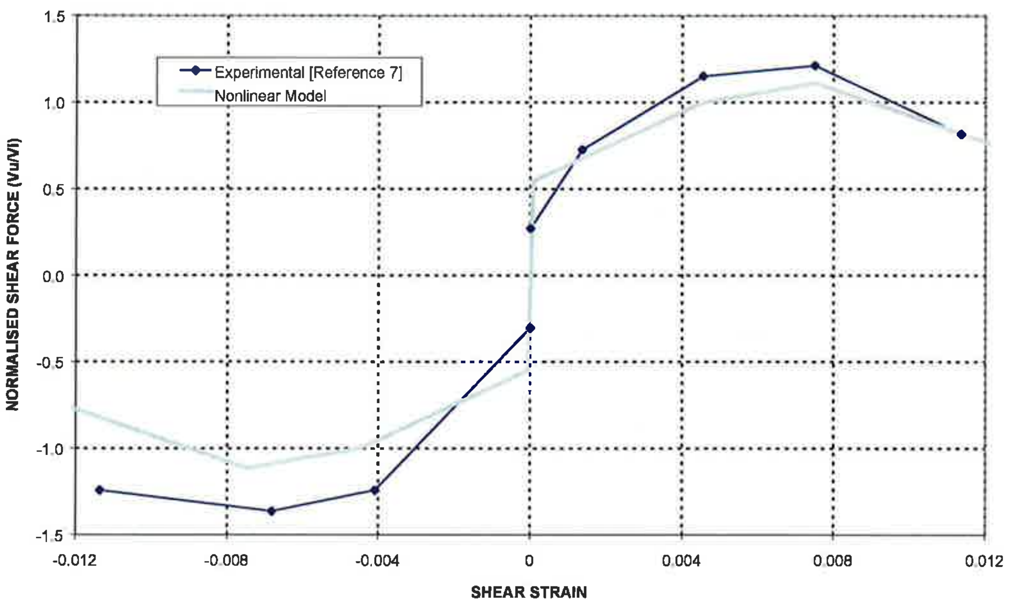
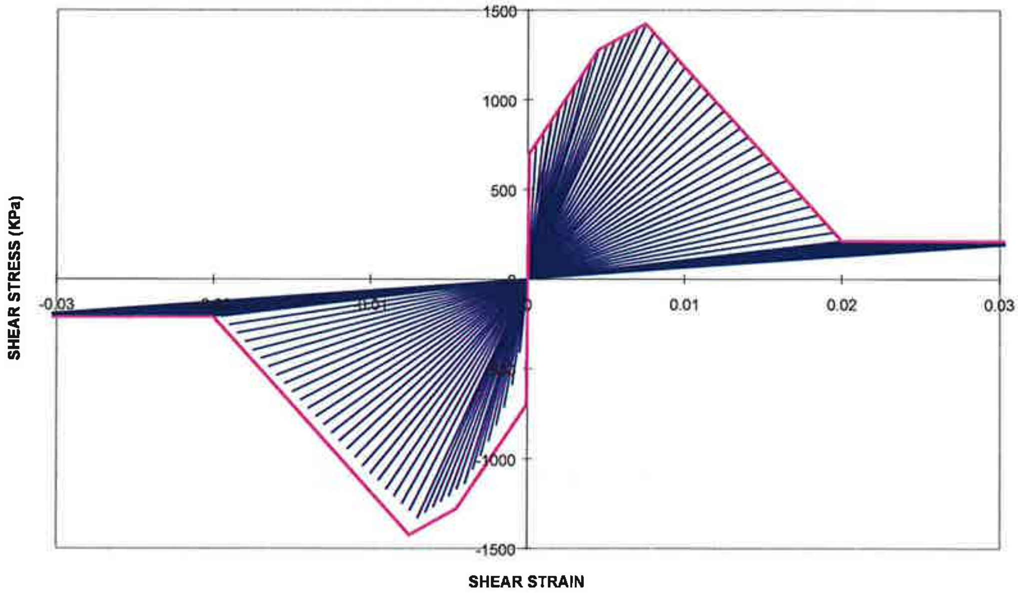


Figure 2.5 shows the hysteresis behaviour of the model under reversing loads as occur during earthquakes [5]. When the shear stress reaches the strength envelope the stiffness reduces to the secant stiffness for unloading and this reduced stiffness is maintained for reversed loading. The stiffness degradation is non-recoverable.

FIGURE 2.5 : WALL HYSTERESIS



2.4 Frame Columns and Beams

The frames columns and beams were modelled as flexural line elements. These elements include shear deformations and rigid joint sizes. The element elastic stiffness was based on 0.6 I_g for the columns and 0.4 I_g for beams, as recommended by NZS3101.

The elements hysteresis is bi-linear. For beams, the yield moment is a function of applied bending moment. Fixed end moments due to gravity loads were applied to ensure the correct total end moments when seismic loads were applied. For columns, yield is based on an interaction surface and is a function of applied axial load and moments about each axis.

For all elements the hysteresis curve was stable (non-degrading) with a strain hardening stiffness of 1% of the elastic stiffness. Flexural elements with hinge properties which degrade under increasing cyclic rotation are available but are generally only used when high degrees of plasticity occur.

2.5 Floors

Rigid diaphragms were assumed at Ground, 1st, 2nd, 3rd, 4th and Roof levels. At these levels all nodes were slaved to the centre of mass node.

2.6 Foundations

Gap elements, linear in compression but with zero tensile strength, were placed at each node at Basement level. These elements were assigned a high elastic stiffness (K = 5,000,000 KPa/m).

2.7 Mass and Weights

The floor masses were calculated from material dimensions with an allowance of 0.5 KPa for superimposed dead load and 0.7 KPa seismic live load. Typical floor weight was 5400 KN and the total seismic weight of the building was 32,800 KN.

2.8 Damping

For the analyses damping of 5% of critical was specified at periods of 0.10 seconds and 1.5 seconds. This spans the range of periods of the walls in both elastic and yielding conditions. A value of 5% is used for most structural analysis for earthquake loadings and design codes are based on this.

3. EARTHQUAKE INPUT

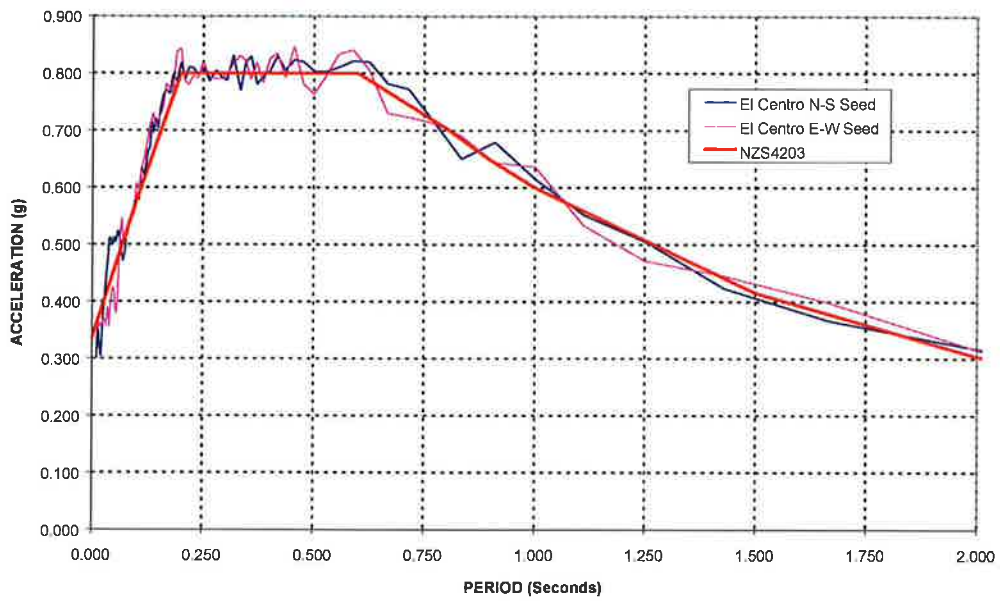
Time history input was prepared following the requirements of NZS4203, "*at least three different earthquake records of acceleration versus time*" and "*scaling shall be such that over the period range of interest for the structure being analysed, the 5% damped spectrum of the earthquake record does not differ significantly from the design spectrum for the limit state being considered*". The period range of interest for the yielding structure is 0.5 to 2.0 seconds

The design spectrum was based on the elastic spectrum for Soft soil sites with Z = 0.8, Sp = 1.0, R = 1.0 and Lu = 1.0

Frequency scaling is used to provides a time history with a close match to a target spectrum. Figure 3.1 shows the response spectra of the El Centro 1940 components after frequency scaling to match NZS4203. This gives a good match for a full range of periods.

To reproduce the effect of a dominant earthquake component, the full component was applied in one direction and a time history scaled to 80% of NZS4203 applied simultaneously in the other direction.

FIGURE 3.1 : FREQUENCY SCALED TIME HISTORIES
5% DAMPED RESPONSE SPECTRA



4. TIME HISTORY ANALYSES

4.1 Sequence of Analyses

The initial analyses considered the as-built building. The earthquake was applied with the major component parallel to the X and the Y axes respectively. Each analysis was performed for 0% and 10% eccectricity. This provided 4 base analyses. The X direction analysis with zero eccentricity was then repeated with scaling factors of 1/3, 1/2, 2/3 and 5/6 applied to the earthquake. The return periods for these motions are as listed in Table 4.1.

TABLE 4.1 : EARTHQUAKE MAGNITUDE

Risk Factor R	Return Period (Years)	Probability of Exceedence at least once in Design Life of 50 Years
0.33	35	76%
0.50	67	53%
0.67	125	33%
0.83	232	19%
1.00	450	10%

After ther initial analyses, the building was strengthened by applying 150 mm of concrete to the face of the end walls of the shear core, full height. This strengthening was assumed reinforced with YD16 @ 200 crs vertical and YD16 @ 150 crs horizontally. The analyses for full NZS4203 loading were then repeated.

Each analysis was for a 20 second earthquake duration at a time step of 0.01 seconds.

TABLE 4.1 : ANALYSES FOR ALL EARTHQUAKES

Analysis	Description
AS-IS BUILDING	
1	X Earthquake, 0% Eccentricity, Full NZS4203 Loading
1B	Y Earthquake, 0% Eccentricity, Full NZS4203 Loading
2	X Earthquake, 10% Eccentricity, Full NZS4203 Loading
2B	Y Earthquake, 10% Eccentricity, Full NZS4203 Loading
1I	X Earthquake, 0% Eccentricity, 1/3 NZS4203 Loading
1J	X Earthquake, 0% Eccentricity, 1/2 NZS4203 Loading
1K	X Earthquake, 0% Eccentricity, 2/3 NZS4203 Loading
1L	X Earthquake, 0% Eccentricity, 5/6l NZS4203 Loading
STRENGTHENED BUILDING	
3	X Earthquake, 0% Eccentricity, Full NZS4203 Loading
3B	Y Earthquake, 0% Eccentricity, Full NZS4203 Loading
4	X Earthquake, 10% Eccentricity, Full NZS4203 Loading
4B	Y Earthquake, 10% Eccentricity, Full NZS4203 Loading

5. RESULTS OF ANALYSES

Table 5.1 summarizes maximum results from each analysis.

TABLE 5.1 : RESULTS FROM EACH EARTHQUAKE

	AS-IS CONDITION				STRENGTHENED			
EARTHQUAKE DIRECTION:	X	Z	X	Z	X	Z	X	Z
ECCENTRICITY:	0	0	10%	10%	0	0	10%	10%
Maximum X Displacement	238	250	229	277	221	228	222	281
Maximum Z Displacement	103	140	95	106	99	129	103	81
Maximum X Base Shear	0.242	0.222	0.231	0.212	0.36	0.291	0.351	0.313
Maximum Z Base Shear	0.258	0.306	0.26	0.284	0.29	0.314	0.281	0.282
WALL RESULTS								
Degraded Panels	42	36	34	27	12	31	17	12
Shear Strain	0.031	0.026	0.031	0.018	0.021	0.022	0.027	0.019
Steel Strain	0.35	0.18	0.24	0.14	0.33	0.18	0.21	0.16
Uplift	19	25	18	18	23	36	20	17
Column Plastic Rotation	0.019	0.017	0.018	0.021	0.015	0.012	0.014	0.016

Figures 5.1 to 5.5 plot the displacements, base shears, maximum shear strain, maximum column plastic rotations and number of damaged wall panels versus earthquake level as a fraction of full NZSW4203 loading.

Figure 5.6 to 5.9 are plots of the extent of wall damage under full NZS4203 loading for the "as-is" structure and for the structure with strengthening applied to the end walls of the shear core.

FIGURE 5.1 : ROOF DISPLACEMENTS

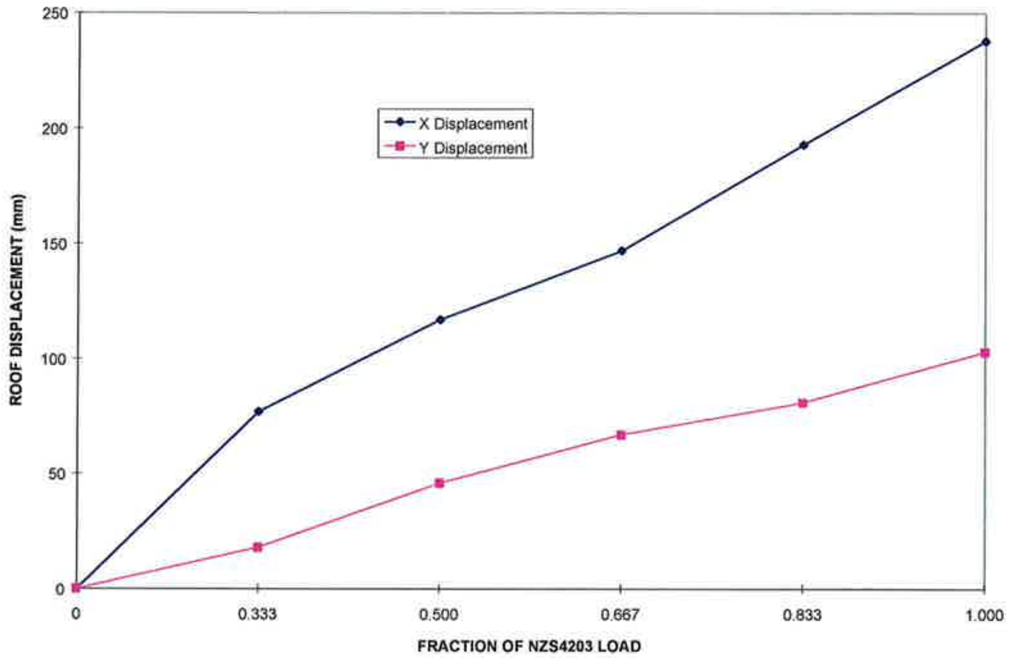


FIGURE 5.2 : BASE SHEARS

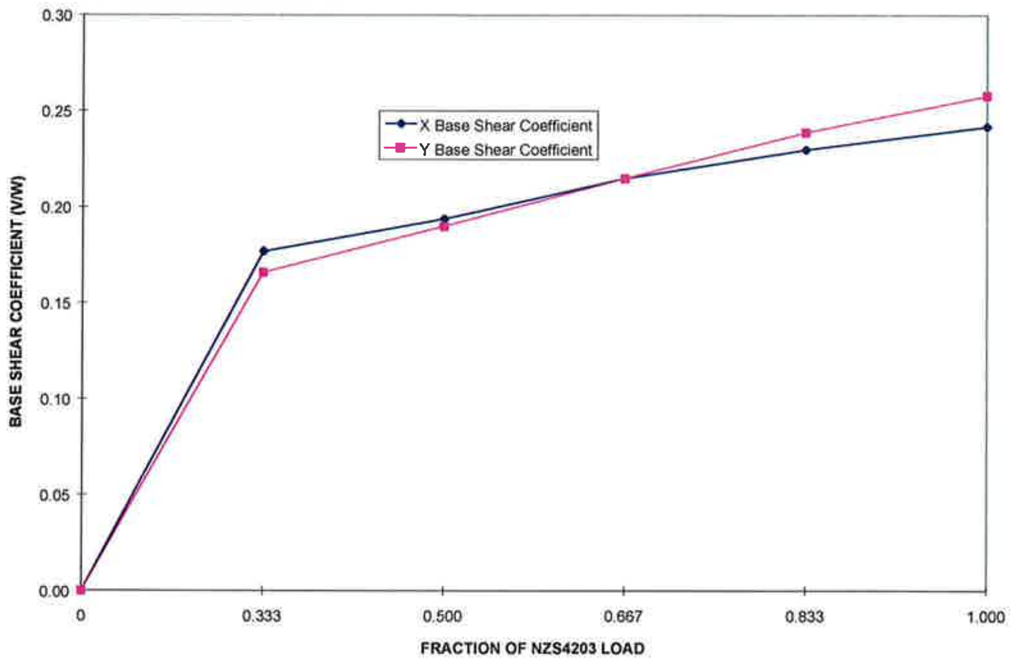


FIGURE 5.3 : MAXIMUM WALL SHEAR STRAIN

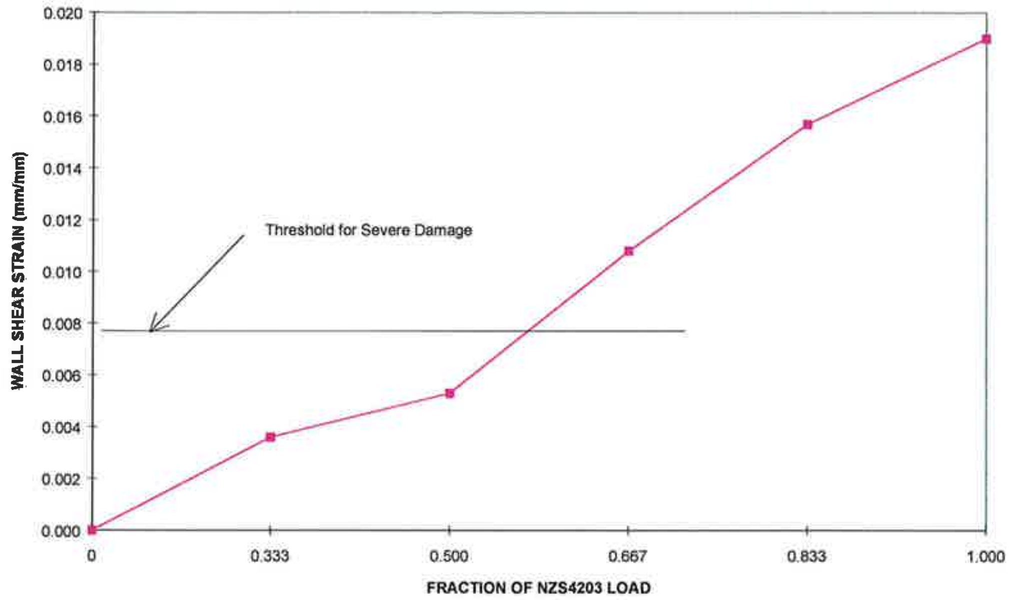


FIGURE 5.4 : MAXIMUM COLUMN ROTATION

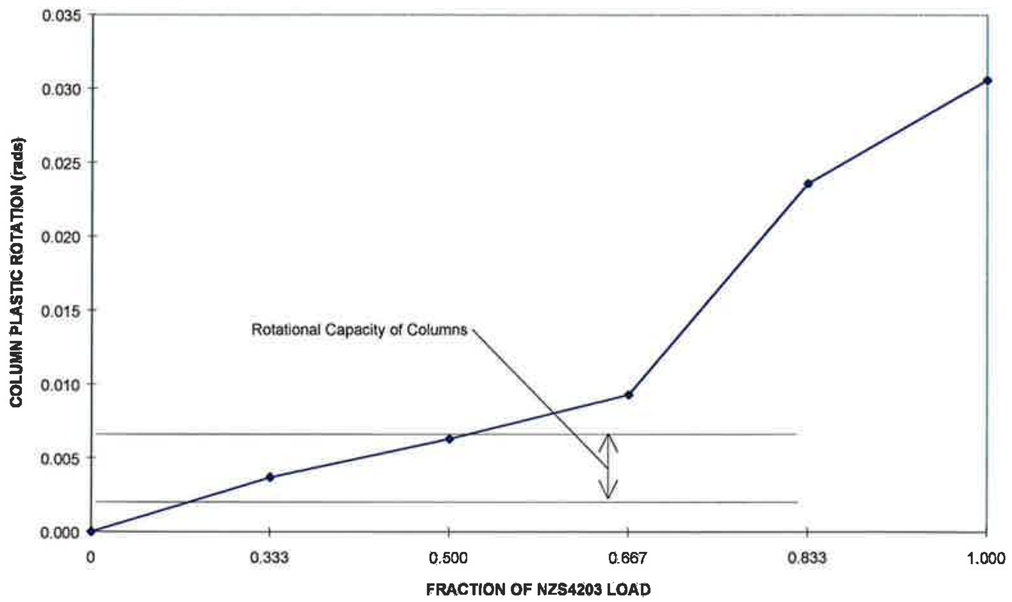


FIGURE 5.5 : NUMBER OF SEVERELY DAMAGED WALL PANELS

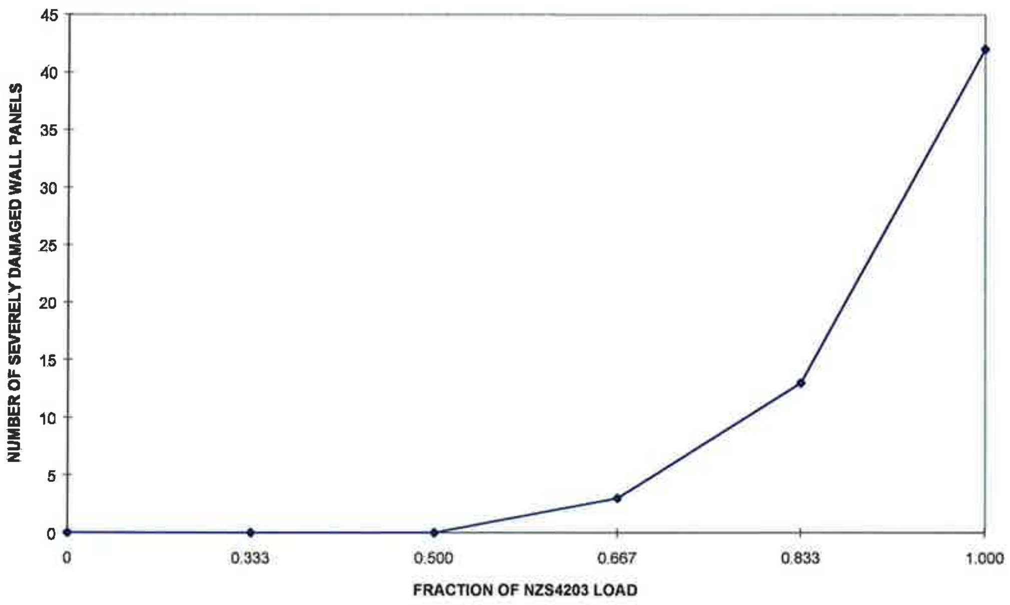


FIGURE 5.6 : AS-IS X LOADING FULL NZS4203

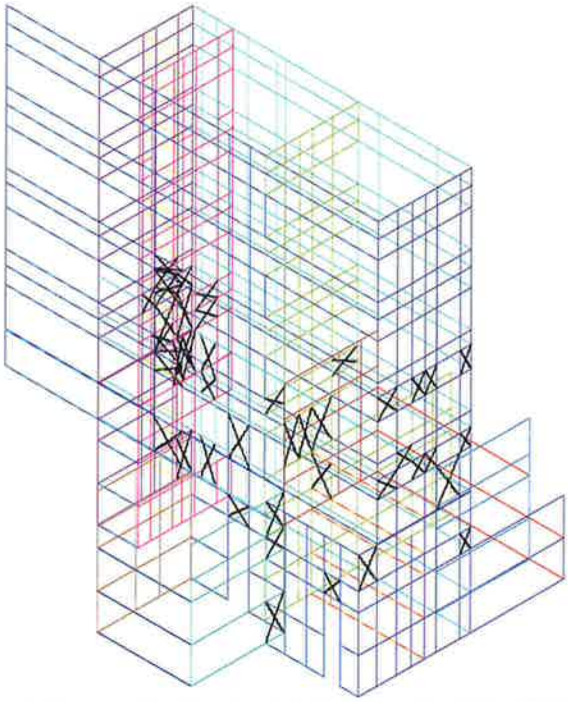


FIGURE 5.7 : STRENGTHENED X LOADING FULL NZS4203

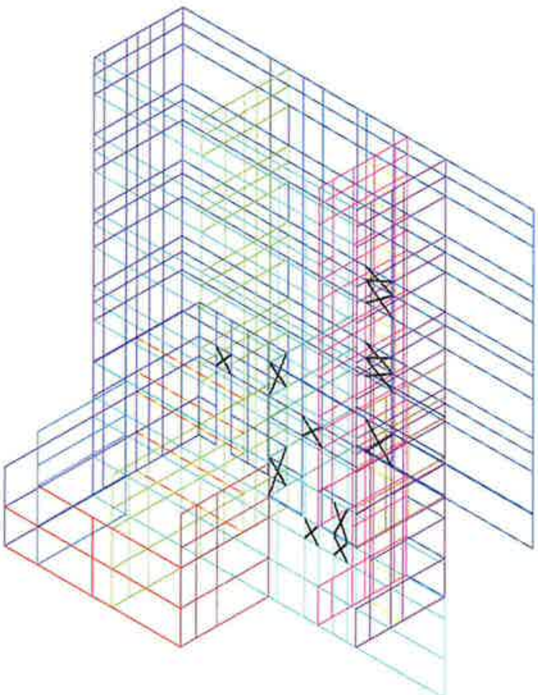


FIGURE 5.6 : AS-IS Z LOADING FULL NZS4203

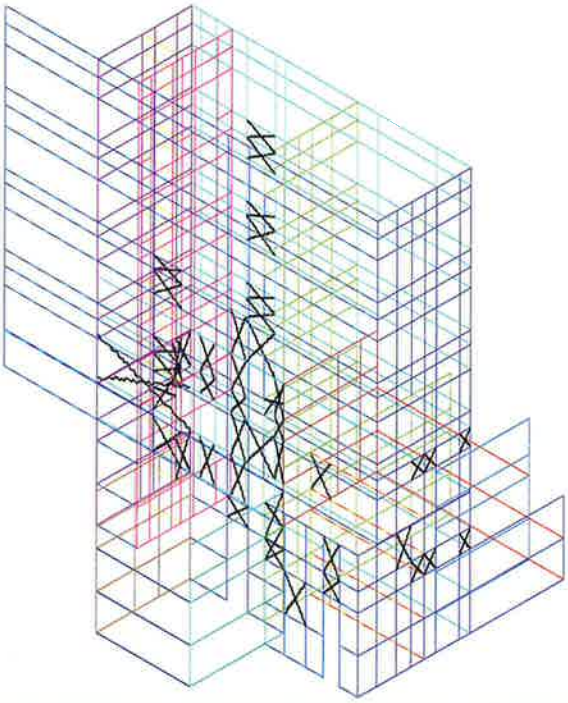
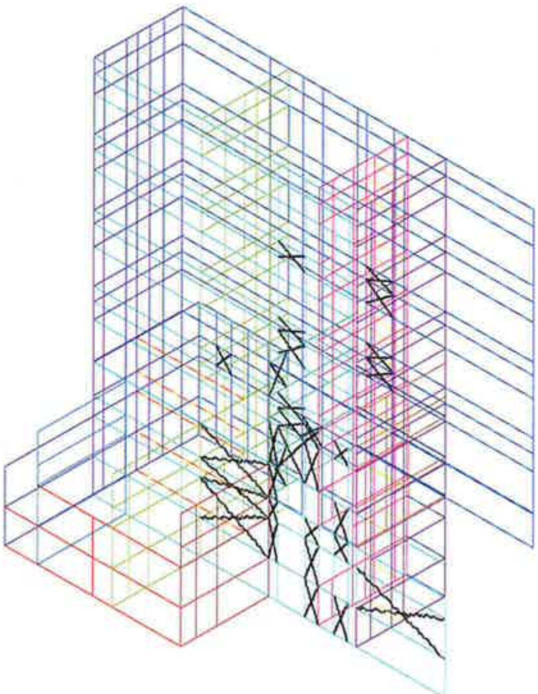


FIGURE 5.7 : STRENGTHENED Z LOADING FULL NZS4203



6. CONCLUSIONS

Under relatively low loads the building exhibits some non-linearity in the form of:

1. Column plastic rotation
2. Wall shear cracking
3. Walll base flexural cracking
4. Foundation uplift

The last two mechanisms, wall base yielding and foundation uplift, are ductile and the only effect on the building is for serviceability due to cracking. Maximum crack widths are up to 3 mm at loads equivalent to 1/3 NZS4203 and 6 mm at 1/2 NZS4203. At full NZS4203 loadings cracks up to 15 mm wide will cause severe cosmetic damage due to concrete spalling.

The columns are small (dimensions as little as 10" x 10", or 254 mm square) and have very light confining reinforcing. The maximum rotational capacity would be in the range of 0.002 to 0.007 radians. At 1/3 NZS4203 maximum applied rotations are 0.004 and at 2/3 NZS4203 are 0.011. Therefore, it is clear that the capacity will be exceeded at relatively low loads. The consequences of exceeding the rotational capacity are severe as the columns could lose vertical load supporting capabilities leading to collapse of extensive portions of the building.

The wall shear strains are also relatively high although cracking is confined to specific portions of the walls, mainly the coupling beams and around openings. The main effect of this cracking is the to reduce the stiffness of the wall but as the cracks do not close there are also potential serviceability problems. For walls which have been strained to the degrading portion of the curve there will be some loss of gravity load carrying capability. However, the consequences of this are not likely to be as severe as for columns as each wall portion supports a relatively small tributary area of floor.

7. REFERENCES

[1] *The Assessment and Improvement of the Structural Performance of Earthquake Risk buildings*, (The Green Book), Draft for General Release, New Zealand National Society for Earthquake Engineering, June, 1996.

[2] *Code of Practice for General Structural Design and Design Loadings for Buildings*, NZS 4203:1992, Standards New Zealand, 1992.

[3] Mondkar, D.P. and Powell, G.H., 1979, *ANSR II Analysis of Nonlinear Structural Response User's Manual*, EERC 79/17, University of California, Berkeley, July.

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[5] E. Kelly, M. R. Button and R. L. Mayes *Seismic Evaluation of Reinforced Masonry Walls*, Structural Engineering in Nuclear Facilities, Volume 1, ASCE, September, 1984.

[6] *Concrete Structures Standard : Part 1 - The Design of Concrete Structures*, NZS 3101:Part 1:1995, Standards New Zealand, 1992.

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