New Zealand Society for Earthquake Engineering

Assessment and Improvement of the Structural Performance of Buildings in Earthquakes

Prioritisation
Initial Evaluation
Detailed Assessment
Improvement Measures

Recommendations fo a NZSEE Study Group on Earthquake Risk Buildings
June 2006
Including Corrigenda Nºs 1 & 2

Assessment and Improvement
of the
Structural Performance
of
Buildings in Earthquakes

Prioritisation
Initial Evaluation
Detailed Assessment
Improvement Measures

Recommendations of a NZSEE Study Group
on Earthquake Risk Buildings

June 2006
Including Corrigenda Nºs 1 & 2

Department of Building and Housing
Te Tari Kaupapa Whare

Foreword

The Building Act 2004 extends the previous definition of the range of buildings that could be earthquake-prone. All but small residential buildings are now covered by the new definition.

Territorial Authorities have been required by the Act to adopt policies on earthquake-prone buildings. Most of these require evaluations of the likely structural performance of buildings that could be earthquake-prone.

Assessment of the structural performance of existing buildings is a challenging task. Each building has unique characteristics and it is often difficult to determine with confidence the extent and quality of structural components and materials.

These NZSEE Recommendations provide authoritative and timely information to assist TAs, owners and their engineers to make assessments of the structural performance of existing buildings, and to determine whether or not they are earthquake-prone.

The document gives information on the background concerns that resulted in the legislation, provides guidance on how a TA might approach the situation, presents a useful Initial Evaluation Procedure, and includes processes for more detailed analysis and evaluation. The inclusion of comprehensive information about measures available to improve the structural performance should help owners and their engineers to find a suitable means to do this.

Use of the Recommendations will promote consistency in assessing the structural performance of existing buildings in earthquakes and contribute to the reduction of earthquake risk in New Zealand.

The Department commends the NZSEE on its achievement and trusts that these Recommendations will prove useful to those responsible for assessing the earthquake-proneness of buildings in relation to s122 of the Building Act.

John Kay

Manager Building Controls

July 2006

Amendments

Description	Date
Corrigendum Nº 1	04/08/2006
Corrigendum Nº 2	15/06/2012
Amendments are marked in the margin.	

Contents

Section 1 - Introduction 1-11

Definitions, Notation and Abbreviations 1-17

Section 2 - Legislative and Regulatory Issues 2-1

2.1 Building Act Requirements 2-1

2.2 Earthquake Prone Buildings 2-1

2.3 Risk Reduction Programmes 2-3

2.4 Advantages of a Formal Policy 2-4

2.5 Adoption/Development of a Formal Policy 2-4

2.6 Policy Content and Options 2-5
- 2.6.1 Policy Content 2-5
- 2.6.2 Implementation Options 2-6

2.7 Implementation Issues for Territorial Authorities 2-6
- 2.7.1 Initial Evaluation Process 2-6
- 2.7.2 Detailed Assessment of Earthquake Performance 2-8
- 2.7.3 Application of Section 112 Requirements (Alterations) 2-8
- 2.7.4 Change of Use Applications 2-8
- 2.7.5 Assessment of the Consequence of Failure 2-9
- 2.7.6 Prioritising Actions 2-9
- 2.7.7 Required Level of Structural Improvement 2-9
- 2.7.8 Timetables for Evaluation and Improvement 2-10
- 2.7.9 Serving Notice 2-10
- 2.7.10 Review Requirements with Owner 2-11
- 2.7.11 Economic Considerations 2-11
- 2.7.12 NZSEE Grading Scheme 2-11
- 2.7.13 Heritage Buildings 2-11
- 2.7.14 Limited Life Buildings 2-11
- 2.7.15 PIM and LIM Notification 2-12
- 2.7.16 Information Systems 2-12
- 2.7.17 Technical Requirements 2-12

2.8 NZSEE Grading Scheme 2-13

2.9 Implementation Options and Steps 2-15
- 2.9.1 Outline Process 2-15
- 2.9.2 Active Programme 2-15
- 2.9.3 Passive Programme 2-17

Section 3 - Initial Evaluation Procedure 3-1

3.1 Background 3-1

3.2 Outline of the Process 3-1

3.3 Summary of Step-by-Step Procedures 3-3

3.4 Background Guidelines and Commentary 3-16
- 3.4.1 Step 1- Collection of information (Table IEP-1) 3-16
- 3.4.2 Step 2- Procedure for assessment of $(\%\ NBS)_b$ (Table IEP-2) 3-16
- 3.4.3 Step 3 - Assessment of performance achievement ratio (PAR) 3-22

3.4.4 Step 4 - Determination of percentage of new building standard *(%NBS)* 3-23
3.4.5 Step 5 - Building Earthquake Prone? 3-26
3.4.6 Step 6 - Building an earthquake risk? 3-26
3.4.7 Step 7 - Seismic grading 3-26

Section 4 - Detailed Assessment – General Issues 4-1

4.1 Introduction 4-1
4.1.1 Context and Background 4-1
4.1.2 Objectives for Assessing Existing Buildings 4-1

4.2 Performance Objectives 4-2
4.2.1 Hierarchy of Performance Measures 4-2
4.2.2 Application 4-5
4.2.3 ULS as Measure of Acceptable Performance 4-5

4.3 Approaches for Performance Assessment 4-6
4.3.1 General 4-6
4.3.2 Global Analysis Considerations 4-8
4.3.3 Approach to Capacity and Demand 4-12

4.4 Building Inspection and Investigation 4-13
4.4.1 Introduction 4-13
4.4.2 General Requirements 4-14
4.4.3 Particular Check Items 4-16

4.5 Relationship with Current Loadings Standards 4-18

4.6 Overall Structural Response Considerations 4-19

4.7 Member Capacity Considerations 4-20

Section 5 - Detailed Assessment - Modelling the Earthquake 5-1

5.1 General 5-1

5.2 Acceleration Response Spectra 5-1

5.3 Displacement Response Spectra 5-2

5.4 Acceleration-Displacement Response Spectra 5-4

5.5 Acceleration Ground Motion Records and Time History Analyses 5-6

5.6 Incorporation of the Structural Performance Factor, S_p 5-6

5.7 Lateral Force/Displacement Requirements 5-6

5.8 $(\%NBS)_t$ factor 5-6

Section 6 - Detailed Assessment - Procedures 6-1

6.1 General 6-1

6.2 Force-Based Methods 6-1

6.3 Displacement-Based Methods 6-4

6.4 Consolidated Force and Displacement Based Procedure 6-6

6.5 Non-Linear Pushover Procedure 6-9

Section 7 - Detailed Assessment of Reinforced Concrete Structures 7-1

7.1 Material Properties and Member Strengths 7-1
7.1.1 Material Strengths 7-1

7.2 Moment Resisting Frame Structures 7-3
7.2.1 Introduction 7-3
7.2.2 Force-Based Procedure for Frame Structures 7-5
7.2.3 Displacement-Based Procedure for Frame Structures 7-19
7.2.4 Determination of Available Ductility Capacity 7-21

7.3 Moment Resisting Frame Elements with Masonry Infill Panels 7-28

7.4 Structural Wall Buildings 7-28
7.4.1 Introduction 7-28
7.4.2 Force-Based Procedure for Wall Buildings 7-29
7.4.3 Displacement-Based Procedure for Wall Buildings 7-35
7.4.4 Deformation Capacities of Wall Elements and the Building System 7-36
7.4.5 Estimation of Equivalent Viscous Damping 7-37

7.5 Dual Frame-Wall Buildings 7-38
7.5.1 Features of Dual Systems 7-38
7.5.2 Assessment Procedure for Dual Frame-Wall Structures 7-39

Section 8 - Detailed Assessment of Steel Structures 8-1

8.1 Introduction and Scope 8-1
8.1.1 Scope 8-1
8.1.2 Useful Publications 8-1

8.2 Material Properties and Member Strengths 8-2

8.3 Evaluation Philosophy and Assumptions 8-2
8.3.1 Approach to be Used for the Evaluation of Existing Steel Seismic Resisting Systems 8-2
8.3.2 Assumptions for the Evaluation 8-3

8.4 Assessing Member and Connection Strength and Rotation Capacity 8-3
8.4.1 General 8-3
8.4.2 Force Transfer through Connections 8-4
8.4.3 General Assessment of the Capacity of Connection Elements and Connections 8-5
8.4.4 Bolted and Riveted Connections 8-6
8.4.5 Welded Beam Flange to Column Connections 8-7
8.4.6 Member Strength and Rotation Capacity 8-8

8.5 Evaluation Procedure for Moment-Resisting Steel Framed Systems 8-10
8.5.1 General 8-10
8.5.2 Determine Strength Hierachy of System 8-11
8.5.3 Allowance for Foundation Strength and Stiffness 8-11
8.5.4 Determine the Structural Ductility Factor necessary to meet the Design Seismic Actions Generated by the Required Strength Assessment Limit 8-12
8.5.5 Determine Inelastic Deflection Limit for System 8-13
8.5.6 Making Allowance for P – Δ Actions. 8-13
8.5.7 Determine Inelastic Behaviour of System from a Pushover Analysis 8-14
8.5.8 Check Stiffness, Strength and Ductility of the System in the Inelastic Range 8-14

8.6 Reporting on Results for Moment-Resisting Steel Frame System Evaluation 8-15

8.7 Evaluation of Moment-Resisting Steel Framed Systems with Infill Panels 8-15

8.8 Evaluation of Braced Buildings 8-16
8.8.1 Lessons Learned from Observed Building Behaviour in Severe Earthquakes 8-16
8.8.2 Evaluation Procedure for Concentrically Braced Framed Systems 8-17

Section 9 - Detailed Assessment of Moment Resisting Frame Elements with Masonry Infill Panels 9-1

9.1 Introduction 9-1

9.2 Solid Infilled Panel Components 9-2
9.2.1 Stiffness 9-2
9.2.2 Strength 9-3
9.2.3 Deformation Capacities 9-5

9.3 Infilled Panel Components with Openings 9-6

9.4 Out-of-Plane Behaviour of Infilled Panel Components 9-6

9.5 The Influence of Infilled Components on Frame Members 9-7
9.5.1 Shear Demands on the Frame Members 9-7
9.5.2 Modified Shear Capacity of Reinforced Concrete Frame Members 9-9

Section 10 - Detailed Assessment of Unreinforced Masonry Buildings 10-1

10.1 General 10-1

10.2 Procedure for the Assessment of Walls Responding In-Plane 10-2
10.2.1 Notation 10-2
10.2.2 Limitations of this Section 10-3
10.2.3 Basis of this Section 10-3
10.2.4 Objective 10-4
10.2.5 General Considerations 10-4
10.2.6 Analysis 10-6
10.2.7 Constitutive Relations and Material Failure Criteria 10-7
10.2.8 Stress and Strain Limits 10-8
10.2.9 Plane Frame Analysis – Strength Limits 10-9
10.2.10 Plane Frame Analysis – Strain Limits 10-10
10.2.11 Common Stress and Strain Parameters 10-11

10.3 Procedure for the Assessment of Walls Responding Out-of-Plane 10-12
10.3.1 Notation 10-12
10.3.2 Basis of this Section 10-13
10.3.3 General 10-14
10.3.4 Procedure for Walls Spanning Vertically between Floors and/or the Roof 10-14
10.3.5 Procedures for Vertical Cantilevers 10-19
10.3.6 Procedures for Gables 10-20
10.3.7 Procedures for Walls Spanning Horizontally or Horizontally and Vertically 10-20

Section 11 - Detailed Assessment of Timber Structures 11-1

11.1 Introduction and Scope 11-1

11.2 Material Properties and Member Strengths 11-1
11.2.1 Material Strengths 11-1
11.2.2 Modification Factors 11-1
11.2.3 Element Properties 11-1
11.2.4 Connections 11-2

11.3 Timber Diaphragms 11-2
11.3.1 Existing Timber Diaphragms 11-3
11.3.2 Strength and Stiffness 11-4

11.4 Timber Shear Walls 11-5
11.4.1 Types of Timber Shear Walls 11-6
11.4.2 Strength and Stiffness 11-6

11.5 Connections 11-7

11.6 Other Timber Elements 11-7

Section 12 - Detailed Assessment - Conclusions 12-1

12.1 General 12-1

12.2 Building Elements 12-1

12.3 Overall Structure 12-1

12.4 Conclusions 12-1

Section 13 - Improvement of Structural Performance 13-1

13.1 General 13-1

13.2 Performance Objectives and Criteria 13-1

13.3 Strategies for Improving Structural Performance 13-2
- 13.3.1 Local Modification of Components 13-2
- 13.3.2 Removal or Lessening of Irregularities and Discontinuities 13-2
- 13.3.3 Global Structural Strengthening and Stiffening 13-3
- 13.3.4 Seismic Isolation 13-3
- 13.3.5 Supplementary Energy Dissipation 13-4
- 13.3.6 Removal of Unnecessary Seismic Mass 13-4
- 13.3.7 Widening Seismic Joints 13-4
- 13.3.8 Linking Buildings Together across Seismic Joints 13-5
- 13.3.9 Seismic Emergency Gravity Supports 13-5
- 13.3.10 Strength and Stiffness Criteria 13-5

13.4 Global Strengthening 13-6

13.5 Strengthening Building Elements 13-8

13.6 Strengthening Unreinforced Masonry or Unreinforced Concrete Buildings 13-14

References 1

Appendices 1

Appendix 2A: Priority Factors 1

Appendix 2B: Factors to be considered when evaluating "as near as is reasonably practicable to that of a new building" 3

Appendix 3A: Typical $(\%NBS)_b$ values for Wellington, Auckland and Christchurch 5

Appendix 3B: Assessment of Attribute Score for URM Buildings 15

Appendix 4A: Typical Pre-1976 Steel Building Systems Used in New Zealand 21

Appendix 4B: Relationships Between Structural Characteristics and Steel Building Performance in Severe Earthquakes 27

Appendix 4C: Assessing the Mechanical Properties of Steel Members and Components 31

Appendix 4D: Potential for Pounding 35

Appendix 4E: Analysis Procedures 39
- 4. Determine the effective height as: 54

Appendix 8A: Bolted and Riveted Joint Moment-Rotation Determination 59

Appendix 8B: Simplified Pushover Analysis for Use in the Evaluation 65

Appendix 10A: Derivation of Instability Deflection and Fundamental Period for Masonry Buildings 67

Appendix 10B: Tests for Assessing the Strength of Masonry and Connectors 79

Appendix 11A: Timber Diaphragm Stiffness 87

Appendix 11B: Timber Diaphragm Strength 89
Appendix 11C: Timber Shear Wall Stiffness 91
Appendix 11D: Timber Shear Wall Strength 93

List of Figures

Section 1 - Introduction 1-11

Figure 1.1: The relationship between the procedures for the design of new buildings and the evaluation of existing buildings 1-13

Definitions, Notation and Abbreviations 1-17

Section 2 - Legislative and Regulatory Issues 2-1

Figure 2.1: Outline of evaluation process 2-7
Figure 2.2: Implementation options and processes 2-15
Figure 2.3: Outline of Steps in Active Programme 2-16
Figure 2.4: Outline of Steps in Passive Programme 2-18

Section 3 - Initial Evaluation Procedure 3-1

Figure 3.1: Diagrammatic representation of Initial Evaluation Procedure 3-2
Figure 3.2: Initial Evaluation Procedure 3-5
Figure 3.3: $(\%NBS)_{nom}$ for Different Building Design Vintages 3-11
Figure 3.4: Structural performance factor, S_p 3-12
Figure 3.5: Extracts from previous Standards showing seismic zoning schemes 3-13
Figure 3.6: Concepts behind scaling factors 3-21
Figure 3.7: Examples of critical structural weaknesses 3-25

Section 4 - Detailed Assessment – General Issues 4-1

Figure 4.1: Strength versus risk and ULS as reference point 4-3
Figure 4.2: Real and modelled responses of buildings to earthquake 4-7

Section 5 - Detailed Assessment - Modelling the Earthquake 5-1

Figure 5.1: Displacement spectra at 5% damping for R = 1 5-3
Figure 5.2: Displacement spectra for different damping levels 5-4
Figure 5.3: Acceleration-Displacement Spectra for different damping levels 5-5

Section 6 - Detailed Assessment - Procedures 6-1

Figure 6.1: Summary of force-based assessment procedure 6-3
Figure 6.2: Derivation of effective stiffness 6-5
Figure 6.3: Summary of displacement-based assessment procedure 6-7
Figure 6.4 Explanation of terms in Eqn 6(3) 6-8
Figure 6.5 Consolidated force / displacement based assessment procedure 6-10
Figure 6.6 Assessment procedure using non-linear pushover analysis 6-11

Section 7 - Detailed Assessment of Reinforced Concrete Structures 7-1

Figure 7.1: Simplified force versus displacement relationships and mechanism outcomes for reinforced concrete frames 7-5
Figure 7.2: Interior beam – column joint subjected to seismic loading 7-7
Figure 7.3: Possible mechanisms of post-elastic deformation of moment resisting frames 7-11
Figure 7.4: Mixed sidesway mechanism of gravity load dominated frames 7-12
Figure 7.5: Typical lateral force-displacement relation of a moment resisting frame 7-12
Figure 7.6: Shear strength capacity as affected by flexure and shear interaction 7-16
Figure 7.7: Degradation of nominal shear stress resisted by the concrete with imposed cyclic curvature ductility factor 7-17
Figure 7.8: Degradation of nominal shear stress resisted by the concrete of beam-column joints with imposed cyclic curvature ductility factor 7-18
Figure 7.9: Mixed sidesway mechanism for a storey 7-19
Figure 7.11: Considerations for beam plastic hinges 7-23
Figure 7.12: Moment-curvature relationships for beam example 7-24
Figure 7.13: Moment curvature response of unconfined columns 7-25
Figure 7.14: Inelastic displacement profile for frames 7-26
Figure 7.15: Summary of force-based assessment procedure for walls 7-30
Figure 7.16: Torsional effects in walled buildings 7-32
Figure 7.17: Required curvature ductility capacity of cantilever wall sections as a function of displacement ductility demand and aspect ratio 7-33
Figure 7.18: Bilinear idealisation of ductile element and system response for a wall building shown in Figure 7.16 7-38
Figure 7.19: The stepwise estimation of the contribution of a frame and a wall element to probable lateral strength and correspondence displacements of a dual system 7-40
Figure 7.20: The bilinear simulation of the force-displacement relation of a dual system and its two elements 7-43

Section 8 - Detailed Assessment of Steel Structures 8-1

Figure 8.1: Typical rivet shank and head diameters 8-6
Figure 8.2: Slender concentrically braced framed building with failure of brace to frame connections, Kobe earthquake 8-17

Section 9 - Detailed Assessment of Moment Resisting Frame Elements with Masonry Infill Panels 9-1

Figure 9.1: Modelling the infill panel of an infilled frame system as an equivalent strut 9-2
Figure 9.2: Modelling the adverse effect of an infill panel on the performance of the perimeter frame showing (a) the placement of the strut, and (b) the moment pattern on the columns 9-8
Figure 9.3: The effect of partial infills on frame performance 9-8

Section 10 - Detailed Assessment of Unreinforced Masonry Buildings 10-1

Figure 10.1: Gable Configurations discussed in this section 10-21

Section 11 - Detailed Assessment of Timber Structures 11-1

Figure 11.1: Distribution of loading for horizontal diaphragm and shear wall system 11-3

Section 12 - Detailed Assessment - Conclusions 12-1

Figure 12.1: Summary of Building Features 12-3

Section 13 - Improvement of Structural Performance 13-1

References 1

Appendices 1

Figure 2A.1: Occupancy Classifications (non-essential buildings) 1
Figure 3B.1: Diaphragm parametersTable 3B.3: Assessment of *%NBS* from Attribute Score 19
Table 3B.3: Assessment of *%NBS* from Attribute Score 20
Figure 4A.1: Riveted steel fabrication details, Government Life Insurance Building, 1937 22
Figure 4A.2: Riveted steel fabrication details, Government Life Insurance Building, 1937 23
Figure 4A.3: Failed beam to column weak axis welded connection from the 1995 Kobe earthquake 24
Figure 4A.4: Braced frame with light tension bracing showing damage but no collapse from the 1995 Kobe earthquake 24
Figure 4A.5: V-braced CBF showing damage but no collapse from the 1995 Kobe earthquake 25
Figure 4B.1: Example of soft storey generated by change from braced to moment frame at bottom storey, 1995 Kobe earthquake 28
Figure 4B.1: Local column crippling failure due to lack of stiffener adjacent to incoming beam flange in a welded, moment-resisting beam to column connection, 1995 Kobe earthquake 29
Figure 4D.1: Example of differing floor elevations in adjacent buildings 37
Figure 4E.1: Plausible force distribution in a flexible diaphragm 41
Figure 4E.2 Diaphragm and wall displacement terminology 44
Figure 4E.3 Beam shears 48
Figure 4E.4 Beam hinges 48
Figure 4E.5 Sway potential 49
Figure 4E.6 Mechanisms 51
Figure 4E.7 Overturning capacity 52
Figure 4E.8 Frame ultimate displacement capacity 53
Figure 4E.9 Strength eccentricity 53
Figure 8A.1: Joint detail 60
Figure 8A.2: Moment–rotation curve 61
Figure 8A.3: Comparison of bare steel and encased moment–rotation behaviour 63
Figure 8A.4: Comparison of monotonic and cyclic moment–rotation behaviour 63
Figure 8B.1: Moment–rotation curve for riveted clip-angle/T – stub connection 65
Figure 10A.1: Configuration at incipient rocking 69
Figure 10A.2: Configuration when rotations have become significant 70
Figure 10A.3: Single cantilever 76
Figure 10 B.1: Bonding requirements for unreinforced masonry walls 79
Figure 10B.2: In-place mortar shear tests 80
Figure10B.2: Bed joint shear test arrangement 82
Figure 10B.3: Schematic of an arrangement for testing doublets 83

List of Tables

Section 1 - Introduction 1-11

Definitions, Notation and Abbreviations 1-17

Section 2 - Legislative and Regulatory Issues 2-1

Table 2.1: Grading system for earthquake risk 2-13
Table 2.2 NZSEE Risk Classifications and Improvement Recommendations 2-14

Section 3 - Initial Evaluation Procedure 3-1

Table IEP-1: Initial Evaluation Procedure – Step 1 3-6
Table IEP-2: Initial Evaluation Procedure – Step 2 3-7
Table IEP-2: Initial Evaluation Procedure – Step 2 continued 3-8
Table 3.1: Return period scaling factor 3-12
Table 3.2: Ductility factors to be used for existing buildings 3-12
Table 3.3: Ductility scaling factor 3-12
Table IEP-3: Initial evaluation procedure – Step 3 3-14
Table IEP-4: Initial evaluation procedure – Steps 4, 5 and 6 3-15
Table 3.4: Guide to severity of critical structural weaknesses 3-24

Section 4 - Detailed Assessment – General Issues 4-1

Table 4.1: Hierarchy of performance measures 4-4
Table 4.2: Summary of recommended analysis procedures and applicability guidelines 4-9

Section 5 - Detailed Assessment - Modelling the Earthquake 5-1

Section 6 - Detailed Assessment - Procedures 6-1

Table 6.1 Typical values of ξ_{eff} for various structural types and materials 6-8

Section 7 - Detailed Assessment of Reinforced Concrete Structures 7-1

Section 8 - Detailed Assessment of Steel Structures 8-1

Section 9 - Detailed Assessment of Moment Resisting Frame Elements with Masonry Infill Panels 9-1

Table 9.1: Out-of-plane infill strength parameters 9-7

Section 10 - Detailed Assessment of Unreinforced Masonry Buildings 10-1

Table 10.1: Notation 10-2
Table 10.2: Strength parameters for preliminary assessments 10-7
Table 10.3: Notation 10-12
Table 10.4: Static instability deflection for uniform walls – various boundary conditions 10-19

Section 11 - Detailed Assessment of Timber Structures 11-1

Table 11.1: Strength values for existing materials 11-4
Table 11.2: Characteristic stresses for visually graded timber [NZS 3603:1993] 11-8

Section 12 - Detailed Assessment - Conclusions 12-1

Section 13 - Improvement of Structural Performance 13-1

Table 13.1: Global strengthening approaches 13-6
Table 13.2 Techniques for strengthening building elements 13-8
Table 13.3: Techniques for strengthening unreinforced masonry or unreinforced concrete buildings 13-14

References 1

Appendices 1

Table 2A.1: Modification factors *K*1 and *K*2 2
Table 3A.1: $(\%NBS)_b$ Wellington, μ = 1.25, Importance Levels 2, 3 and 4 5
Table 3A.2: $(\%NBS)_b$ Wellington, μ = 2, Importance Levels 2, 3 and 4 6
Table 3A.3: $(\%NBS)_b$ Wellington, μ = 3, Importance Levels 2, 3 and 4 7
Table 3A.4: $(\%NBS)_b$ Auckland, μ = 1.25, Importance Levels 2, 3 and 4 8
Table 3A.5: $(\%NBS)_b$ Auckland, μ = 2, Importance Levels 2, 3 and 4 9
Table 3A.6: $(\%NBS)_b$ Auckland, μ = 3, Importance Levels 2, 3 and 4 10
Table 3A.7: $(\%NBS)_b$ Christchurch, μ = 1.25, Importance Levels 2, 3 and 4 11
Table 3A.8: $(\%NBS)_b$ Christchurch, μ = 2, Importance Levels 2, 3 and 4 12
Table 3A.9: $(\%NBS)_b$ Christchurch, μ = 3, Importance Levels 2, 3 and 4 13
Table 3B.1: Assessment of Attribute Score 16
Table 3B.2: Definition of attributes and scores 16
Table 4A.1: Main periods of the structural use of cast iron, wrought iron and steel 21
Table 4C.1: Minimum material properties for steels and rivets manufactured in the USA 32
Table 4C.2: Typical properties of structural steels from the UK for the period 1906–68 32
Table 10A.1: Static instability defection for uniform walls, various boundary conditions 75
Table 10B.1: Determination of design values from in-place mortar shear tests and bed joint shear tests 82
Table 10B.2: Default connector strengths 84

Section 1 - Introduction

Basic Aims of this Document

The underlying aim of the New Zealand Building Act 2004 is to reduce the risk of death or injury that may result from the effects of a significant earthquake on buildings *that represent a higher than normal risk in earthquake*.

The Building Act legislation will greatly increase the awareness of earthquake risk amongst building owners. It will also result in the need to assess the earthquake performance of many more existing buildings than has previously been the case.

This document (the Guidelines) describes approaches, steps and procedures to assist in assessing the earthquake performance of existing buildings of various material types and configurations, notably reinforced concrete, steel, timber and unreinforced masonry. Guidance for improving the performance of such buildings is also given.

The basic aim of this document is to provide a set of guidelines that are helpful to Territorial Authorities, consultants and building owners, and that can be applied consistently to assess the earthquake performance of a building.

Background

Reconnaissance visits mounted by the New Zealand Society for Earthquake Engineering to the scene of major earthquakes over the past two decades have returned with a consistent message regarding the vulnerability of structures designed to early codes. Acknowledging these concerns, and foreshadowing a revision of the Building Act 1991, the Building Industry Authority commissioned the New Zealand National Society for Earthquake Engineering to produce a document setting down the requirements for structural engineers to follow when evaluating and strengthening pre-1976 buildings.

Prior to the enacting of the The Building Act 2004, the term *earthquake risk buildings* related only to unreinforced masonry buildings. The risk posed by such buildings, along with the early concrete and steel structures designed prior to the first New Zealand seismic design code, NZSS 95 published in 1935, is readily apparent. The prime characteristic of these buildings is that wind loading was the only (if any) lateral force considered in their design.

While most buildings designed before the publication of NZS 4203:1976 (SNZ 1976) and associated materials codes have often been designed to similar levels of *strength* as modern structures, they typically do not have either the level of *ductility* or appropriate hierarchy of failure required by current design standards.

Buildings constructed in the decades between 1935 and the early 1970s feature different structural characteristics. Reinforced concrete buildings from the 1940s and the 1950s are typically low-rise with regular and substantial wall elements. Many of these structures would be capable of close to an elastic level of response, with local detailing exceptions. Reinforced concrete buildings from the 1960s and early 1970s are, however, generally taller, less generously proportioned, with less redundancy and greater irregularity often in evidence in frame structures. Steel-framed buildings tended to be riveted up until the early 1940s, with the likely seismic response of these buildings being very dependent on the joint detailing employed.

The level of risk posed by buildings constructed as recently as the early 1970s is now more widely appreciated. The Northridge and Kobe earthquakes have highlighted the vulnerability of this category of structure.

As a consequence of the awareness of this vulnerability, the Building Act was revised to encompass any building which is considered to not be capable of an adequate seismic performance.

Accordingly, the expression *earthquake risk building* is now regarded as applying to any building that is not capable of meeting the performance objectives and requirements outlined in this document.

The Building Act focuses particularly on buildings of high risk. These buildings are referred in the legislation as *Earthquake Prone Buildings* and form a subset (the worst) of earthquake risk buildings

Key Features

The key features outlined in this document include:

- A summary section on legislative and regulatory issues to assist Territorial Authorities in implementation
- Full details of the Initial Evaluation Procedures, previously published separately.
- Assessment procedures for reinforced concrete, steel, timber and unreinforced masonry buildings
- Introduction of approaches and procedures that view structural performance in relation to displacements generated. (Displacement-based approach)
- A section on approaches and techniques commonly used for improving structural performance. (Strengthening and retrofitting).

The approaches and procedures presented have been developed especially with the process of assessment in mind. Such processes differ distinctly from the design processes for a new building. Most of these procedures have yet to be evaluated fully in actual situations, and some refinement can be anticipated as a result of feedback received from Territorial Authorities, consultants and owners.

Several procedures are based on evaluating the performance of individual earthquake-resisting elements. Considerable engineering judgement is required when assessing the implications for individual elements of overall building response, particularly given the configurational shortcomings of earlier structures. Judgement is also required to ensure that elements and components selected for detailed analysis provide a realistic yet conservative assessment of the overall building.

At this stage, the displacement-based approaches and procedures have only been described in detail for structures of reinforced concrete since it is these structures that have received most attention from researchers. However, the displacement-based approaches outlined could quite easily be applied to other materials also. Procedures reflecting a displacement based approach are expected to become more common in future.

Notwithstanding that the aim of this document is to access existing buildings against requirements for new buildings some of the assumptions suggested for existing buildings are less stringent or different from those required for new buildings. This reflects the difference between the objective for an existing building of *predicting the level at which a particular limit state is likely to occur* and

the design objective for a new building of *precluding a particular limit state from occurring.* Less stringent assumptions than used in design also reflect that the building exists and therefore actual material strengths, for example, can be checked.

Guidance given to improve structural performance is general in nature only, due to the wide range of possible options available and of building characteristics. The material presented is intended to assist structural engineers to determine suitable, effective and economical solution. It should be noted that new approaches and techniques are constantly developing.

Purpose and Objective

The purpose of this document is to assist designers and Territorial Authorities in implementing the requirements of the Building Act.

This document:

a) provides a means of assessing the earthquake structural performance of an existing building, and in particular its capability to reach a minimum required level of performance

b) provides approaches to and guidance on techniques for improving seismic performance. .

Subsequent sections define the respective seismic performance criteria, and express them in terms of current design standards.

The relationship between the design of a new building and the assessment and strengthening of an existing building is established in the following sections. This relationship is represented diagrammatically in Figure 1.1. The key principle is that the current loadings standard in conjunction with current materials standards is common to both procedures. Both the philosophies and detailed steps to enable existing buildings to be evaluated in this way are presented in the following sections, including modifications to factors and/or materials values contained in current codes where considered appropriate.

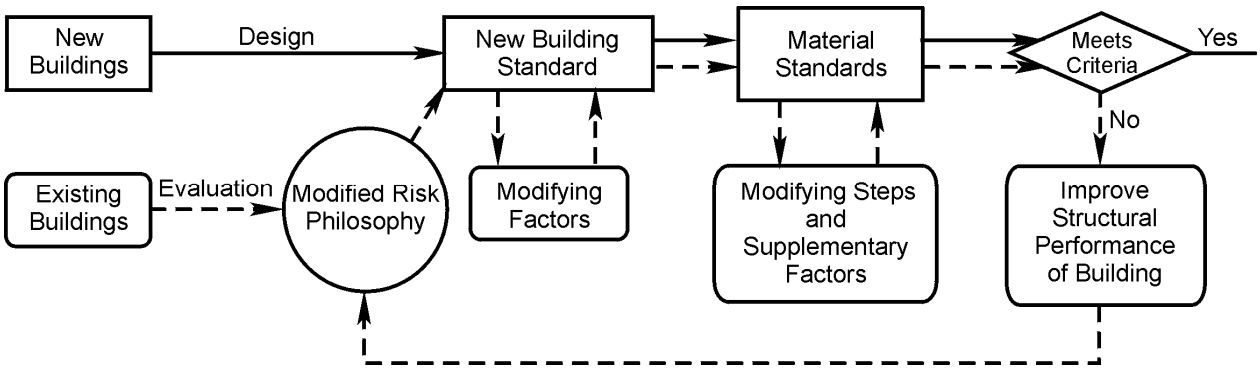


Figure 1.1: The relationship between the procedures for the design of new buildings and the evaluation of existing buildings

Scope

Emphasis in this document is placed on the most common structural configurations that are considered to pose the greatest risk. The seismic resisting elements for which direct guidance is offered in this document are as follows:

- reinforced concrete moment resisting frames
- reinforced concrete structural walls
- reinforced concrete dual wall/frame systems

- structural steel moment resisting frames
- unreinforced masonry buildings
- frame structures (concrete or steel) with masonry infill.
- Timber diaphragms and shear walls

Bridge structures have not been specifically addressed in this version of these Guidelines, although most of the issues and approaches outlined for reinforced concrete frame structures are applicable.

The term *improving the structural performance of* is used in the title of this document rather than *strengthening* in acknowledgment of the wide range of options for structures that are found to be *earthquake risk buildings*. Some of these options involve only the removal or separation of components, and others affect a relatively small number of members. For brevity in this document, however, *strengthening* and *retrofitting* are used most commonly, and should be taken as having the same meaning as "*improving the structural performance of*".

These Guidelines draw together current New Zealand and international knowledge in this field, and will be subject to ongoing refinement and development as further understanding is gained.

This document concentrates on matters relating to life safety; that is to say, performance at the ultimate limit state. Emphasis is therefore placed on the identification and elimination of possible undesirable collapse modes that could affect either part of a building or the entire structure. As well as considering the relative strengths of structural members, there is a need to evaluate the consequences of critical structural weaknesses that could lead to collapse, such as excess torsional responses, or soft or weak stories created by vertical irregularities or adjacent buildings. Lack of seismic separation between structural and non-structural items can also be a life safety issue.

Damage to the building itself is a secondary consideration, and this point along with the associated implications must be made clear to the owners of buildings by users of this document. Insurance considerations are not specifically addressed in the Guidelines due to the many commercial factors involved on a case-by-case basis. Buildings that are either assessed as being acceptable in terms of this document or are actually strengthened could be damaged beyond repair by a significant earthquake.

Although serviceability limit state issues are not specifically addressed, serviceability limit state loads from current design standards can be used to represent the likely onset of damage.

NZSEE Grading Scheme

The NZSEE is promoting the use of a grading scheme for classifying buildings according to earthquake performance. While grades are to be based on assessment scores from procedures in this document, it should be recognised that the determination of a grade is not a requirement of the proposed legislation. Refer Section 2.8. The Grading Scheme proposed in Section 2.8 is likely to influence insurers in their assessment of their risk exposure.

Document Status and Outline

Ultimately it is intended that this document will be nominated in the New Zealand Building Code Handbook as a Guideline Document to assist with compliance with the Building Act. Although definitive procedures are presented, much of the material is commentary and background. This has been judged to be necessary and helpful in bringing together approaches in dealing with the wide variety of existing buildings.

Material that presents assessment procedures has been highlighted with a yellow background (grey when printed on black and white). This device is intended to aid interpretation and application of procedures once a user is familiar with the background.

In Section 2, legislative and regulatory issues are summarised. This section describes the overall approach to evaluating ERBs from a regulatory and compliance perspective. Guidance is given especially to TAs to help them develop formal and consistent policies to deal with the technical, legislative, regulatory, economic and social factors involved.

The section specifically indicates the role of the initial evaluation procedure and detailed evaluation procedure as well as giving guidance on the setting of requirements for improving structural performance.

Section 3 incorporates the Initial Evaluation Procedure. This has been published previously (NZSEE 2000) but has now been revised with NZS 1170.5 forming the "current standard".

Section 4, Detailed Assessment – General Issues, sets out general issues relating to the detailed assessment of buildings.

Section 5 defines the earthquake shaking parameters that should be adopted for a detailed assessment.

In Section 6 the recommended assessment procedeures for a detailed assessment are described.

Sections 7 to 11 contain the bulk of the document. This material is largely new but builds on material previously in the 1996 Document (NZSEE 1996 Green Book). For completeness, unreinforced masonry buildings and timber structures have been included and the steel sections have been extended.

Section 13, Improvement of Structural Performance, provides guidance on performance objectives, approaches and techniques for improving the structural performance of existing buildings.

It is hoped that this document will not only provide guidance to professionals with responsibilities for implementation of the new provision of the Act but also raise awareness amongst owners and the general public of the need to bring many existing buildings closer to the standard required for new buildings.

NZS 4203:1992 and AS/NZ1170:2002

In drafting these Guidelines, the Group has been aware of the need to be up to date. Although NZS 1170.5:2004 has yet to be cited by the New Zealand Building Code as a compliance document at the time of writing, it is expected that this standard will be cited. Therefore these guidelines have been written around NZS 1170.5:2004 as a reference point for new buildings. Until NZS 1170.5 has been cited it may be necessary to also check against NZS 4203:1992 unless the Territorial Authority has agreed to accept NZS 1170.5 as an alternative solution. Particular care is needed in applying the values in these Guidelines so as to ensure that the legally required standards are met.

Acknowledgements

The study group which compiled this document consisted of:

- David Brunsdon (Spencer Holmes)
- Charles Clifton (HERA)
- Carl Devereux (Ian Smith and Partners)
- David Hopkins (David Hopkins Consulting)
- Rob Jury (Beca)
- Professor John Mander (University of Canterbury)
- Professor Nigel Priestley (University of California, San Diego)
- Lou Robinson (Hadley and Robinson)
- Bruce Shephard (Seismic Consultants Ltd)
- Kevin Spring (Ormond Stock and Associates)
- Sulo Shanmuganathan (Opus International Consulting)
- Craig Stevenson (Connell Mott McDonald)
- John Taylor (Christchurch City Council)
- Grant Wilkinson (Holmes Consulting Group)
- Adam Thornton (Dunning Thornton Consultants Limited)
- Peter Moss (Emeritus Professor, University of Canterbury).
- The late Professor Robert Park (University of Canterbury)

The financial assistance provided by the Department of Building and Housing is gratefully acknowledged.

Important Note
It is recommended that those carrying out evaluations and reviews using these guidelines recognise the responsibilities involved and the liabilities to which they may be exposed.

Neither the NZSEE or any member of the Study Group accepts any liability for the application of these Guidelines in any specific instance.

It is recommended that engineers providing advice based on the application of these Guidelines take appropriate steps to define the limits of their responsibilties and liabilities.

Definitions, Notation and Abbreviations

Definitions

For ease of reference, definitions are given in the relevant Section or Chapter

Notation

For ease of reference, notation is given in the relevant Section or Chapter

Abbreviations

Anairp	As near as is reasonably practicable.
CBF	Concentrically braced frame.
CQC	Complete Quadratic Combinations
CSW	Critical structural weakness.
DCB	Design and Construction Bulletin (HERA publication).
(D)MRSF	(Ductile) moment resisting structural frame.
EBF	Eccentrically braced frame.
EMA	Elastic modal analysis – same as MRSA.
EPB	Earthquake prone building – refers to definition in the Building Act 2004 i.e. $\leq$ 33*%NBS*.
ERB	Earthquake risk building – a building assessed as having greater than moderate risk i.e. < 67*%NBS*.
ES(M)	Equivalent static (method).
GSAP	Global structural analysis procedure.
GSM	Global structural model.
HERA	Heavy Engineering Research Association.
HRB	High risk building – a building that does not meet the criteria in the Building Act Section 122.
IEP	Initial Evaluation Procedure.
ITHA	Inelastic time history analysis.
LIM	Land Information Memorandum – refer Local Government Official Information and Meetings Act Section 44a.
LPA	Lateral push-over analysis.
MRSA	Modal response spectrum analysis – same as EMA.
NBS	New Building Standard – i.e. the standard that would apply to a new building at the site. This includes loading to the full requiremnets of the Standard.
NZS	New Zealand Standard.
NZSEE	New Zealand Society for Earthquake Engineering.

PIM	Project Information Memorandum – refer Building Act Section 31.
RSJ	Rolled steel joist.
Section	Section (of an Act of Parliament)
SLaMA	Simple lateral mechanism analysis.
SLS	Serviceability limit state as defined in NZS 1170.5:2004 (or NZS 4203:1992), being the point at which functionality of the structure and its contents become unacceptable.
SANZ	Standards Association of New Zealand
SNZ	Standards New Zealand, formerly SANZ.
SPS	Structural performance score.
SRSS	Square Root of Sum of Squares
T(L)A	Territorial (Local) Authority.
ULS	Ultimate Limit State. This is generally as defined in NZS 1170.5:2004 and AS/NZS1170.0. Refer Section 4.2.3
URM	Unreinforced masonry.
%NBS	Percentage of new building standard
$(\%NBS)_t$	Target percentage of new building standard.

Section 2 - Legislative and Regulatory Issues

2.1 Building Act Requirements

The sections of the Building Act 2004 that have implications for the seismic resistance of existing buildings are;

- Sections 112 and 113 cover buildings undergoing alteration including the situation where the intended remaining life is less than 50 years.
- Section 115 provides requirements for buildings where a change of use is proposed.
- Section 122 and its associated Regulations define an Earthquake-Prone building (EPB).
- Sections 124 to 130 provide power for territorial authorities (TAs) to act on earthquake-prone buildings and set out how this action is to be taken.
- Sections 131 and 132 require the territorial authorities to establish earthquake-prone building policies and specify how the policies are to be established, what they are to include and when they are to be reviewed.

The implications of these requirements on the seismic resistance of existing buildings is discussed in the following sections.

2.2 Earthquake Prone Buildings

The definition of an earthquake-prone building (EPB) is set out in Section 122 of the Act and in its associated Regulations.

Quoting from the Act;

122 Meaning of earthquake prone building

(1) A building is earthquake prone for the purposes of this Act if, having regard to its condition and to the ground on which it is built, and because of its construction, the building –

(a) will have its ultimate capacity exceeded in a moderate earthquake (as defined in the regulations): and

(b) would be likely to collapse causing –

(i) injury or death to persons in the building or to persons on any other property; or

(ii) damage to any other property

(2) Subsection (1) does not apply to a building that is used wholly or mainly for residential purposes unless the building-

(a) comprises 2 or more storeys: and

(b) contains 3 or more household units.

And from the Regulations;

7. Earthquake-prone buildings: moderate earthquake defined-

For the purpose of section 122 (meaning of earthquake-prone building) of the Act, moderate earthquake means, in relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site.

This definition of an EPB is significantly more extensive and more stringent than that provided by the 1991 Building Act. It encompasses all buildings, not simply those constructed of unreinforced masonry or unreinforced concrete, though it exempts small residential buildings. The definition is also linked to the current standard which is significantly more demanding than that provided under the previous Act.

In developing these Guidelines NZSEE has taken the following definitions to apply with respect to the wording of section 122 and its associated regulations:

(a) "ultimate capacity" means ultimate limit state capacity as defined in current design standards.

(b) "likely to collapse causing injury or death to persons in the building" means that collapse and therefore loss of life *could well occur* as a result of the effects of earthquake shaking on the building.

(c) "earthquake that would generate shaking at the site of the building one-third as strong as the earthquake-shaking that would be used to design a new building at that site" means that the inputs of load, displacement, velocity and/or acceleration used for a new building are scaled by one-third, but the duration would be unchanged. Note that this last point becomes very significant if a designer chooses to use time-history analysis to demonstrate acceptable performance.

NZSEE holds the view that the collapse criterion given in subclause 122 (1) (b) of the Act does not relate back to expected performance in a moderate earthquake but rather to an overall expectation. Thus it does not in itself affect the recommendations made in these guidelines. NZSEE recognises however that this is an interpretation of a clause that may be considered to have some ambiguity. NZSEE would like to see this subclause deleted as it is almost impossible to predict collapse and the reference to collapse only has the potential to confuse rather than assist application of the earthquake prone building requirements.

The level of "one-third as strong" (corresponding to a Percentage of New Building Standard (*%NBS*) of 33, (Refer Section 3.2) is considered a reasonable balance (for the present time) between imposing a requirement to upgrade all non-complying buildings (< 100*%NBS*) and the previous position where only URM buildings were addressed. 33*%NBS* corresponds to approximately 20 times the risk of the building reaching a similar condition to that which a new building would reach in a full design earthquake.

It is possible that the threshold of 33*%NBS* could be lifted over time, but if the proposed NZSEE Grading Scheme works as intended, this lower level may suffice as a legislative backstop. Nevertheless, it is recommended that buildings with < *67%NBS* be seriously considered for improvement of structural performance, at least when major alterations or refurbishments are contemplated.

It is arguable whether or not loss of life would occur in a new building under a design earthquake. Structural collapse is generally regarded as making loss of life a certainty (assuming the building is occupied). However, the view is taken that loss of life in or near a properly designed new building could also occur due to factors other than structural collapse. Thus, meeting of earthquake code requirements for a new building may be taken as representing an acceptable probability (or likelihood) that loss of life will occur.

In the same way a building with 33*%NBS*, when subject to earthquake input factored by one-third, could be regarded as having the same acceptable probability of loss of life. However, the generally lower ductility exhibited by older buildings implies more brittle behaviour and more sudden loss of structural integrity. An existing building with 33*%NBS*, subject to a one-third earthquake would generally represent a higher probability of loss of life than a new building subject to a design earthquake.

The wording of the Regulations refers to the response of the site rather than just the building seismic coefficient. This allows engineers the widest possible scope in demonstrating satisfactory performance of the building. For example, they may wish to do a site-specific study of seismic shaking or to carry out time history analyses of the existing building.

2.3 Risk Reduction Programmes

Sections 131 and 132 of the Building Act require TAs to establish a risk reduction policy for EPBs.

The main purpose of the legislation is to reduce earthquake risk in the community. The new requirements recognise the total impracticality of bringing all existing buildings up to the standard of new buildings. The threshold of one-third of the earthquake shaking represents about 20 times the risk of a new building. Buildings below this threshold are categorised by NZSEE as high risk in terms of the hierarchy of performance criteria given in Section 2.1 of these Guidelines.

The Building Act is silent on the level to which an EPB should be strengthened unless a change of use is also involved. It is the view of NZSEE that EPBs should be brought to a standard that is "as near as is reasonably practicable to that of a new building".

Two issues arise:

a) What to do with buildings that pass the one-third criterion, but which still represent a significant risk. Legally, no action is required, but the NZSEE view is that any building below 67*%NBS* should be regarded as a questionable earthquake risk and therefore still an Earthquake Risk Building (ERB). Its structural performance should be improved to protect the interest of both the owner and the community generally.

b) What level of performance improvement represents "as nearly as is reasonably practicable to that of a new building"? This level will vary from case to case and, subject to sound reasoning on the practicability of improving the performance, any level above 33*%NBS* will be legally acceptable. Again, the NZSEE strongly recommends that every effort be made to achieve improvement to at least 67*%NBS*. This reduces the relative risk from around 20 times to around 3 times that of a new building.

Thus, the new legislation targets only the worst buildings – the sort of buildings we see collapsed in other cities following major earthquakes. There will be many buildings that represent a considerably greater earthquake risk than buildings designed and built correctly to current standards.

In order to increase awareness of this fact, the NZSEE is proposing a Grading Scheme for categorising buildings according to their assessed performance in a major earthquake. Refer

Section 2.8 below. This, in conjunction with legislation covering the worst risks, is seen to be an effective way of dealing with the worst buildings in a reasonable timeframe and of achieving ongoing earthquake risk mitigation for the remainder.

These Guidelines are intended to assist building owners, their advisors and TAs to deal with the requirements of the proposed legislation. In particular, this section is intended to encourage and assist TAs to develop a comprehensive risk reduction programme by establishing a formal policy on ERBs and EPBs, through consideration of:

- advantages of a formal policy
- adoption/development of a formal policy
- policy content and options
- implementation options and approaches
- technical requirements and procedures.

In addition, the Guidelines are intended to be of assistance to TAs in the exercise of their discretion in the implementation of the structural requirements of Section 115 of the Building Act covering change of use.

It is emphasised that the recommendations and guidance notes given in this Section are those of NZSEE, and are not intended to imply any additional legal obligation under the Act.

2.4 Advantages of a Formal Policy

The higher than normal risk of many existing buildings is a fact. It is important that TAs determine a clear and comprehensive policy, consistent with the new legislation and the perception of earthquake risk in their communities

It is strongly recommended that all TAs adopt a formal policy, consistent with their particular circumstances. The extent of relevant building stock and the technical and financial resources of the territorial authority, and the community it represents, are clearly considerations.

Regardless of the approach chosen, the decision taken and the reasons for it should be formally made and recorded. In the event of a major earthquake, the decision taken and the reasons for it will need to be capable of being seen as reasonable and defensible, especially in hindsight.

Advantages of a formal policy are:

- a demonstrable recognition by the TA of the risk, and a commitment to a risk reduction programme
- a defensible and logical basis for such a programme
- a means to make building owners and the public aware of the issues involved
- definitive procedures and predictable outcomes for owners and their advisors
- clearly defined requirements based on authoritative Guidelines, such as these NZSEE Guidelines.
- the security and convenience of a consistent nationally accepted standard.

2.5 Adoption/Development of a Formal Policy

The following recommended procedure should assist each TA to reach a decision on which broad policy option is best suited to their particular circumstances. The basic steps are shown in Section 2.9. In summary they are:

a) *Decide to address earthquake risk buildings*. It is recommended that every TA address the potential risk of non-complying existing buildings in its community, even if the decision is made to take no action. This will allow the TA to be fully informed on the nature and extent of the risk in making decisions on what to do about it.

b) *Identify potential earthquake risk buildings*. It is envisaged that a TA would examine its building stock as a "desk-top" study, assessing the numbers of buildings in each age bracket, the total floor area involved, and other data on the physical characteristics. Buildings which are likely to be Earthquake Prone (i.e. high risk) would be identified. It is recommended that all or a representative sample be subjected to the initial evaluation process assessment (refer Section 3 of these Guidelines).

c) *Develop and adopt a formal policy*. Based on the above assessment, a decision should be made on the extent to which the TA will implement a risk reduction strategy. This decision and the reasons for it should be formally recorded, even if the decision is to take no action. On the basis that the decision is to implement the provisions, the TA should:

d) *Decide on a realistic total timeframe* for completion of the risk reduction programme, taking account the impact on the community, and balancing the need to reduce earthquake risk with economic and social constraints.

e) *Select an appropriate implementation option*, taking account of the required timeframe and the particular circumstances of the local community and TA.

The adoption and recording of a formal policy should reflect a genuine commitment to a progressive upgrading of the region's building stock for earthquake resistance. The proposed policy will need to be sustainable with reference to its economic impact on the region's commercial sector, while recognising and responding to the growing knowledge and awareness of earthquake risk, and the variation with time of public perceptions of it.

2.6 Policy Content and Options

This Section sets out suggestions for the content and implementation options of the policy. TAs are also directed to the Policy Guidance for Territorial Authorities document prepared by the Department of Building and Housing and available from their web site. This document draws from the suggestions made in this section of the Guidelines.

2.6.1 Policy Content

The formal policy should include consideration of and clearly defined approaches to the following:

- initial evaluation process
- detailed assessment of earthquake performance
- technical requirements and criteria
- implementation options
- prioritising actions
- application of Section 112 requirements
- approach to Section 115 considerations
- change of use requirements
- assessment of the consequence of structural failure
- required levels of structural performance improvement
- timetables for evaluation and improvement
- serving notice
- review requirements with owner
- economic considerations
- NZSEE grading scheme

- heritage buildings
- limited life buildings
- information systems.

To ensure a consistent and even-handed application of the policy, the TA should adopt formal guidelines based on Building Act requirements and relative safety. The guidelines should include some form of initial evaluation to allow appropriate prioritising of building improvements.

The guidelines should be transparent and defensible, and should define the scope of the envisaged upgrading. This document is intended to assist TAs in this regard.

An outline of the overall evaluation process envisaged is given in Figure 2.1.

2.6.2 Implementation Options

Territorial authorities are obliged by the new legislation to establish a policy on Earthquake Prone buildings. The policy must indicate the approach that is to be followed, the priorities that will be set and how the policy will apply to heritage buildings.

The Territorial Authority has two principal options:

a) an active risk reduction programme

b) a passive risk reduction programme.

In an *active programme*, the TA, using the IEP, would actively identify high risk buildings, set priorities and timeframes for action, and set guidelines for performance levels for upgrading. The TA would then serve notice on the owners requiring them, at their cost, to carry out detailed assessment and/or performance improvement as appropriate. This process will provide a TA with the best possible risk reduction programme as it is able to set and control the timing of mitigation work. There are significant costs to the TA to set up and administer an active programme.

In a *passive programme* assessment and improvement of structural performance would be activated by an application under the Building Act for an alteration (if the TA had reason to believe that the building was Earthquake Prone) or change of use. Assessment of the structural performance of the building would be at the owner's cost. The passive programme therefore has the significant disadvantage of relying on a somewhat haphazard order based on owners intentions for the buildings. This could leave some significant high risk buildings untouched for a long period. The TA may find it difficult to defend a passive programme when viewed with the hindsight of a major event. However the TA's costs to administer the programme will be significantly less than those for the active programme.

Section 2.7 discusses issues which a TA will need to address in developing its formal policy.

2 Section 2.9 outlines the steps required in the development and implementation of the formal policy.

2.7 Implementation Issues for Territorial Authorities

2.7.1 Initial Evaluation Process

Section 3 of these Guidelines details an initial evaluation procedure (IEP) to be applied to buildings. The procedure is intended to be a coarse screening involving as few resources as reasonably possible to identify potentially high risk (or Earthquake Prone) buildings.

The results obtained in the IEP may be used to:

- identify buildings that warrant a detailed assessment of their structural performance
- provide a preliminary score for a comparative risk grading of buildings

- provide a means of determining priorities for improvement of structural performance.

The objective of the IEP is to identify, with an acceptable level of confidence, all high risk buildings. At the same time the process must not catch an unacceptable number of buildings that would, on detailed evaluation be outside the high risk category.

It is expected that those carrying out the IEP would be New Zealand Chartered Professional Engineers with a background of experience in design of buildings for earthquake or having received some specific training.

The initiating circumstances and the responsibility for carrying out the initial evaluation process will vary with the implementation option selected, and is described more fully in Section 2.8.

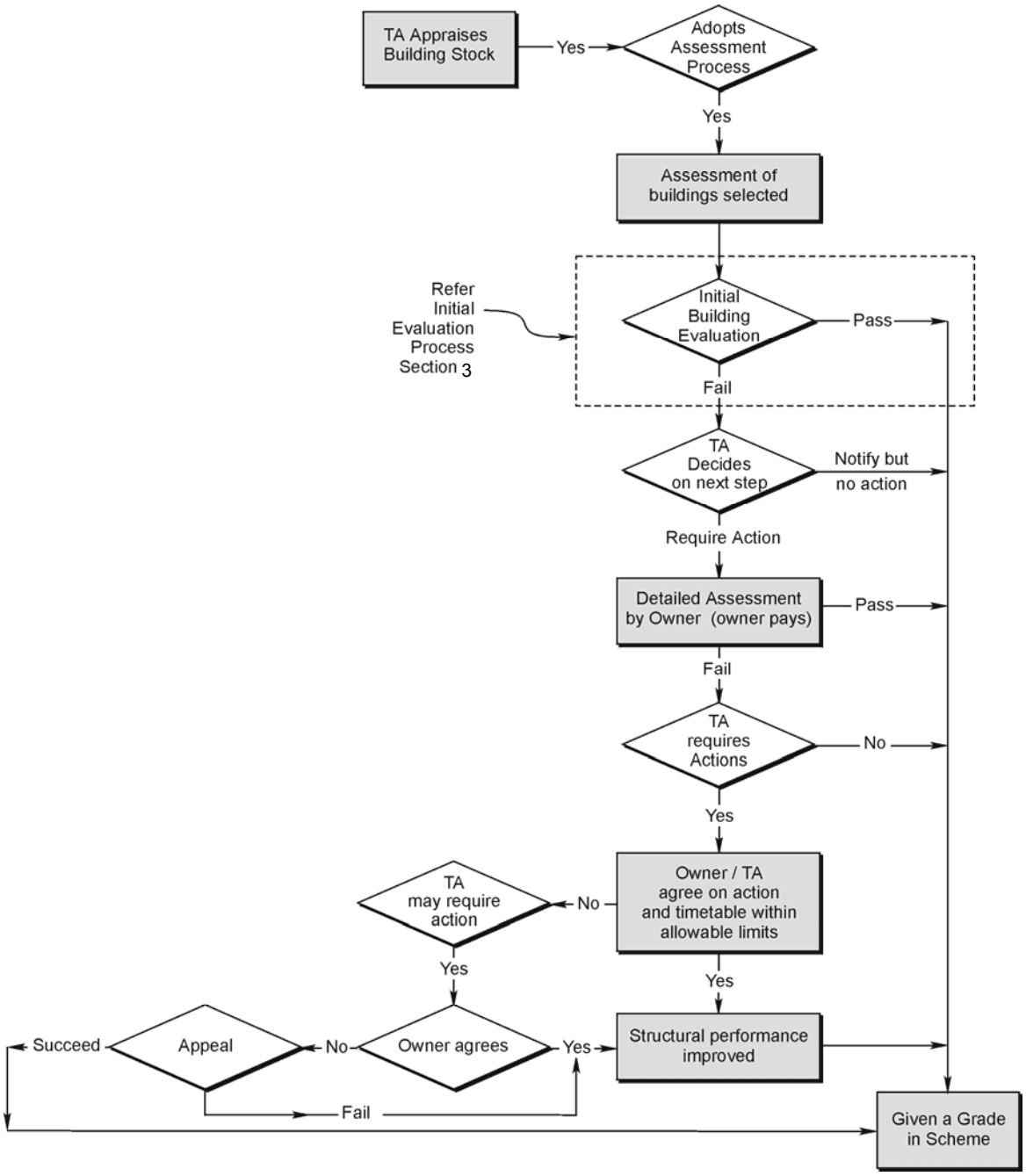


Figure 2.1: Outline of evaluation process

2.7.2 Detailed Assessment of Earthquake Performance

Where an initial evaluation indicates that the building is likely to be high risk (Earthquake Prone), it is desirable that a detailed assessment is carried out as set out in Section 4 of these Guidelines. This will provide a more specific and convincing evaluation on which a final decision can be made on whether or not the building is to be classified as high risk.

The building owner will generally be responsible for submitting the detailed assessment, at the request of the TA. The assessment must be carried out by an engineering consultant suitably experienced in earthquake design.

The initiating circumstances, time required for submission, and follow up requirements will depend on the implementation option selected, and is detailed in Section 2.8.

2.7.3 Application of Section 112 Requirements (Alterations)

On receipt of a consent application for alterations, the TA would be at liberty to require an IEP assessment to be submitted with the application for building consent, if this requirement was part of its formal policy and/or it had reason to believe the building could be Earthquake Prone.

If the TA adopts a passive programme, all applications for a building consent for a building alteration should be assessed under the policy. However as the cost is to be met by the applicant, it may be reasonable to specify a minimum level of work (cost, extent and/or nature) below which, at the TA's discretion, an evaluation would not be required. Alternatively the TA may request an initial evaluation to identify the status of the building even if it elects not to require performance improvement at that time.

If the consent application includes significant structural work, and the building before the alteration is deemed to be Earthquake Prone, it is recommended that the altered building should follow the guidelines for performance improvement under Section 5.

Regardless of the implementation adopted, any structural work required to improve the performance of a building constitutes an alteration to the building. Section 112 of the Building Act therefore applies. In such a case the TA is required to also consider means of escape from fire and the provision of access and facilities for persons with disabilities to the extent required by the Act.

2.7.4 Change of Use Applications

For the TA to approve a change of use under Section 115 of the Act, it is required to believe that the building will meet the structural performance standards of the building code as nearly as is reasonably practicable as if it were a new building.

An assessment should be requested from the owner for all change of use applications. The extent of this assessment will depend on the nature and implications of the change of use.

Any work required to meet structural performance improvement requirement of Section 115 is to be carried out before a Code Compliance Certificate can be issued. Any previous notices or agreements allowing an extended timetable for improvement of structural performance will no longer apply and, if necessary, revised notices will need to be issued to match the change of circumstances.

2.7.5 Assessment of the Consequence of Failure

B1.3.4 of the Building Code requires the owner to make allowance for the consequences of failure in building design.

These Guidelines thus incorporate provision to consider the number of people at risk in the determination of the time prescribed for the mitigation work to be done. Guidance on these priority factors and their application is given in Appendix 2A.

It should be noted that, although, the proposed IEP is based on a comparison with loadings and material standards, this provides a comparative measure of the probability of loss of life. For a new building, the current standards implicitly define the attainment of ultimate limit state as the boundary between acceptable and unacceptable loss of life. By retaining the comparison with current standards when measuring the performance of existing buildings, the same boundary is again implicit.

2.7.6 Prioritising Actions

It is probably not realistic to expect many territorial authorities to carry out a complete evaluation of their entire building stock in the short term, even where they have a genuine commitment to upgrading their buildings.

For an "active" procedure, it will therefore be desirable to easily identify priority buildings for attention. The IEP described in Section 3 was developed for this purpose, focussing on critical structural weaknesses. A TA may elect to deal with different groups of buildings to different timetables to spread workload, provided consistency is achieved, e.g. to focus first on buildings in the CBD or of a particular vintage or type. This could be done based on a simple visual assessment taking account of basic vulnerability features. Buildings so identified would then be assessed using the IEP in Section 3.

The results of the initial evaluation process will give the TA an approximate quantitative measure of building performance, which will form the basis of prioritising for further action. However, because the IEP does not address wider considerations, it may be appropriate to include some quantitative recognition of building importance, building occupancy (number and intensity), and building location as well as the building under-capacity in determining priorities. Appendix 2B provides priority factors to assist territorial authorities to account for this consideration.

A detailed assessment under Section 4 will give a more rigorous measure of the likelihood of failure of the building under earthquake ground motions. It is recommended that, when setting priorities for action, the output from the analysis (i.e. *%NBS*) be adjusted using Appendix 2A. | 2

Note that the priority factors in Appendix 2A are for establishing relative priorities for action only. They must not be applied in determining whether the building is Earthquake Prone in terms of the Act. | 2

2.7.7 Required Level of Structural Improvement

It will be necessary for the TA to decide on a suitable approach for setting expected performance levels appropriate to various buildings that are confirmed as Earthquake Prone. The aim should be to bring as much consistency and fairness as possible to the decision.

It is the recommendation of NZSEE that the expected performance level should be set at as nearly as is reasonably practicable to New Building Standard. Thus the initial target level for improvement should be 100% NBS. In many cases this will not be practicable and it will be necessary to establish a reasoned reduction to an acceptable level. In any event NZSEE

recommends that 67*%NBS* be regarded as a minimum to be achieved in the structural improvement measures notwithstanding that the legal minimum requirement is possibly only 34*%NBS*.

Guidelines should be developed by each TA to deal with the range of buildings likely to be encountered, and particularly special cases such as heritage buildings.

2.7.8 Timetables for Evaluation and Improvement

There are a number of issues that must be considered in determining the total time to complete a TA's risk reduction programme. These include the time for evaluation of the buildings as well as the time to be allowed for the required improvement work on each building to be completed.

For an active programme the TA will need to assess the time required to complete a quantified initial evaluation of the relevant buildings and to serve notice on the owners requiring a detailed evaluation. It is recommended that the period required for completion of the detailed evaluation be no more than one year. This will require consideration of economic and social impact as well as earthquake risk.

On receipt of the evaluation report, the TA is required to determine a reasonable timeframe for completion of improvement work for each building. To reflect the consequence of failure as implied by the Act, suggested priority factors are provided in Appendix 2A. This will allow the timetables set to give quantitative recognition of such things as building importance, building occupancy (number and intensity), and building location as well as structural performance.

If the TA adopts a passive programme, it is recommended that any building for which the policy indicates a time for improvement of less than two years should have the improvement work included in the original consent for change of use or alteration. The two-year timetable indicates a very high risk building with earthquake performance of approximately 10*%NBS*.

If the TA elects to adopt a passive programme, it may decide to define a maximum period for a detailed evaluation for buildings that are not altered or subject to a change of use over a long period. The period could be specified in relation to a intended life of 50 years or a cut-off date (e.g. 2020). This would enable the TA to ensure that all earthquake risk buildings were addressed within the timeframe selected. It would however require the TA to carry out an IEP on all relevant buildings to identify those that do not meet the performance target of the Act.

2.7.9 Serving Notice

For either the active or passive implementation option, the TA should consider serving notice on all buildings that are confirmed by the detailed assessment procedures, not to be safe in terms of the Act. The legislative procedures for serving notice under the Act are detailed in Section 125. The notice is required to specify a time for the work to be done.

For the active programme, it is recommended that the TA issue an initial notice on all buildings identified by the IEP as potentially Earthquake Prone in terms of the Act. The notice should record the result of the IEP and request either a detailed assessment from the owner or a commitment to performance improvement within specified times. It is strongly recommended that the priority list be completed before notices are served, to enable the simultaneous issue of notices, at least for buildings of similar risk profile.

To ensure that its actions are transparent and defensible, any action by the TA in respect of serving notice should be strictly in accordance with the formal policy previously adopted, and would be expected to reflect the relative risks represented by assessed buildings.

2.7.10 Review Requirements with Owner

Buildings identified as high risk are required to be upgraded within a reasonable time. These Guidelines are intended to assist the TA to arrive at appropriate targets based on the Act.

However it could be considered as unreasonable if the TA was not prepared to accept submissions from owners when making decisions affecting their buildings. Valuable information could come to light which was not previously available to the TA, such as lease arrangements, future intentions for the building, and economic considerations (refer 2.7.11 below).

The exact requirements for improvement of structural performance in any particular case would need to be worked out between owner and territorial authority, using the formal policy as basis.

2.7.11 Economic Considerations

The prime concern of the proposed changes is to improve life safety. However each TA is expected to consider the short term and long term costs of the work in determining their requirements. Limited legal precedent confirms that the cost to eliminate the risk is to be balanced against the degree of risk, and that the weighting of the considerations will vary with the circumstances. However, where considerations of human safety are involved, factors which impinge on those considerations must be given appropriate weight".

Concessions for economic hardship are most easily accommodated in the time allowed for the improvement work.

2.7.12 NZSEE Grading Scheme

The proposed NZSEE Grading Scale (refer Section 2.8 below) is intended to assist in raising awareness of the existence of earthquake risk, and to provide an underlying motivation for owners to improve their buildings. TAs are encouraged to actively promote the NZSEE Grading Scheme, and consider using it on PIMs and LIMs.

The initial grading of buildings would usually be based on an IEP. However the grading should be reviewed and amended if necessary if a detailed analysis becomes available. Note that the grading is a measure of likely structural performance.

2.7.13 Heritage Buildings

A TA is required to make particular provision for heritage buildings in its risk reduction policy.

Due to their age, layout, construction type and aesthetic sensitivity, improvement of the structural performance of heritage buildings may be unusually expensive. However in deciding on a suitable standard of performance improvement, the TA will need to consider that, in addition to life safety, protection of the building fabric will be more important than would otherwise be the case.

The TA could consider offering incentives for building owners to achieve an appropriate result.

2.7.14 Limited Life Buildings

Limited life buildings should be assessed in the same way as typical (50 year) buildings, with the seismic design actions for assessment and design of improvements based on the appropriate design return period from the loadings standard.

2.7.15 PIM and LIM Notification

The Building Act 2004 and the Local Government Official Information and Meetings Amendment (No 2) 1991 require the TA to disclose any information known to the authority which is likely to be relevant to the alteration of any building on the site.

The information given should indicate the earthquake risk classification of the building and detail the implications of the classification in terms of the territorial authority's formal policy. If the building has been subject to an IEP or detailed assessment, the result should be identified, by the NZSEE grade or *%NBS* score.

Any requests for detailed assessment or formal notices for rectification in accordance with the Act should be detailed.

Buildings that have been evaluated should be identified accordingly and any likely future requirements or agreed actions should be flagged. For buildings that have not been evaluated, the PIM and LIM should include a statement to that effect.

The notification would be amended or removed if justified by subsequent detailed assessment or performance improvement.

2.7.16 Information Systems

It is recommended that all TAs establish a database of buildings, which should include details of any assessments, grading scores, notices or agreements related to the Act. If appropriately indexed, the data could be of great assistance in a number of ways, including:

- providing a measure of the number of buildings in each risk category or NZSEE Grade at any time
- allowing the territorial authority to review the progress of its overall risk management strategy, including specific risk categories
- providing a convenient index for reminder notices to owners for approaching timetable deadlines for improvement work, or limited life restrictions
- providing information for PIMs and LIMs
- helping demonstrate that a responsible and defensible approach to earthquake risk is being taken.

2.7.17 Technical Requirements

The TA's formal policy should contain clear statements and cross references to loading and material codes, as well as to the technical sections of these Guidelines. It will not be possible to prescribe detailed technical criteria for each situation, but there should be a soundly based framework which territorial authority officials can use to determine detailed requirements for a particular case.

The basis for technical requirements should be these Guidelines, specifically relating to:

- initial evaluation process for identifying ERBs and EPBs (IEP) and detailed evaluation procedures
- approaches for setting design levels for improving structural performance
- timetables for improving structural performance
- the structural performance requirements of stability, strength and displacement
- use of the principles of current standards (loading and material) where possible and applicable

- use of specially developed approaches for assessing performance and for improving structural performance
- use of basic precepts of earthquake resistant design, especially the use of displacement criteria and means of enhancing performance so that displacement demands can be met.

2.8 NZSEE Grading Scheme

In addition to the legislative requirements, the NZSEE is keen to introduce into the property market a system for grading buildings according to their assessed structural performance. The aim is to raise awareness in the industry and allow market forces to work in reducing earthquake risk. In time, owners of lowest grades of buildings would find themselves under pressure to improve them or face loss of revenue.

Table 2.1 indicates the grading scheme proposed. This is linked to the *%NBS* value. Determining the earthquake risk grade of a building would be a simple matter of determining into which grade band the calculated *%NBS* of the building falls.

Note that the grade is not required by the Act, but is seen by NZSEE as a highly desirable mechanism to bring about improvement of structural performance.

Table 2.1 includes an indication of the relative risk for buildings designed at different times. The relative risk represented by the progressively decreasing *%NBS* shows the importance of dealing with those buildings with less than or equal to 33*%NBS* – they have 20 or more times the risk of their strength being exceeded due to earthquake actions.

Table 2.1: Grading system for earthquake risk

Percentage of New Building Standard (%NBS)	Letter grade	Relative risk (approx)	NZS 4203: 1976 or better	1965–76 No CSWs	1935–65 No CSWs	2/3 Chapter 8	Buildings with CSWs
>100	A+	< 1 time					
80–100	A	1–2 times					
67–80	B	2–5 times					
33–67	C	5–10 times					
20–33	D	10–25 times					
<20	E	> 25 times					

Note changes to the relative risk values have been made to line up with the values in Table C4.4.

Notes:

1) *%NBS* is the percentage new building standard score for a particular building
2) Values shown for *%NBS* for building groups are indicative only and will vary with location, assessed ductility, features. Many buildings may have been designed for more than the minimum requriements of the Standards of the day.
3) Letter grade is an indicator of likely performance in earthquake.
4) Relative risk (RR) is the ratio of probabilities that the ultimate strength will be exceeded in any given period of time, i.e. RR = (probability for existing building with *%NBS* value shown) ÷ (probability for building with 100*%NBS*).
5) CSW stands for critical structural weaknesses.

The NZSEE Study Group sought to summarise its views on how buildings of various risk levels should be regarded. The result is shown in Table 2.2. Buildings that do not comply with the minimum requirements of the proposed changes to the Act (i.e. ≤33*%NBS*) are regarded as High Risk Buildings. Those with > 67*%NBS* are regarded as being Low Risk. This leaves a group in

between that meet the requirements of the Act but cannot be regarded as Low Risk. These have been termed Moderate Risk.

These definitions differ from the requirements of the Act. The Act requires that buildings be improved to at least 34*%NBS*. Table 2.2 indicates the difference.

Table 2.2 NZSEE Risk Classifications and Improvement Recommendations

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	Unacceptable	Unacceptable

There are many buildings in New Zealand constructed prior to 1976. The cost to the community of requiring full compliance with current standards would be considerable, and arguably disproportionate to the risk reduction achieved.

The NZSEE considers that the community would accept a higher level of risk in an existing building than for a new building, if only for the reason that it will, in general, be economically more feasible to provide higher levels of dependable strength and reliable ductility in a new building than in an existing one. As a result, existing buildings which can be shown to be able to resist demand corresponding to two-thirds of the design event may be categorised as Low Risk.

The acceptance of a factor of 67% as a minimum for existing buildings to be considered as Low Risk is based on this corresponding to an increase in risk for an existing building of approximately two times that of an equivalent new building. This is judged reasonable and compares well to equivalent levels set for the evaluation of existing buildings in the United States. For example, the approach taken in ASCE 31 leads to approximately 75% of the new building standard.

Whilst this increase in risk could appear high on a building-by-building basis, it appears a reasonable minimum target overall.

The NZSEE recommends upgrading to as nearly as is reasonably practicable to that of a new building. However NZSEE considers it is more important and realistic to identify the high risk buildings, and reduce the risk they pose to a more acceptable level, than to attempt to ensure that all existing buildings comply with the latest standards. The elimination of non-ductile failure mechanisms and critical structural weaknesses is in itself of greater importance than the actual assessment and strengthening level. Building failures during earthquakes rarely occur solely because the design forces have been underestimated. More often than not, poor performance results from some obvious configurational or detailing deficiency.

2.9 Implementation Options and Steps

2.9.1 Outline Process

This section provides the recommended steps in the implementation of the active or passive risk reduction programmes. A diagrammatic representation of the implementation options and processes is given in Figure 2.2.

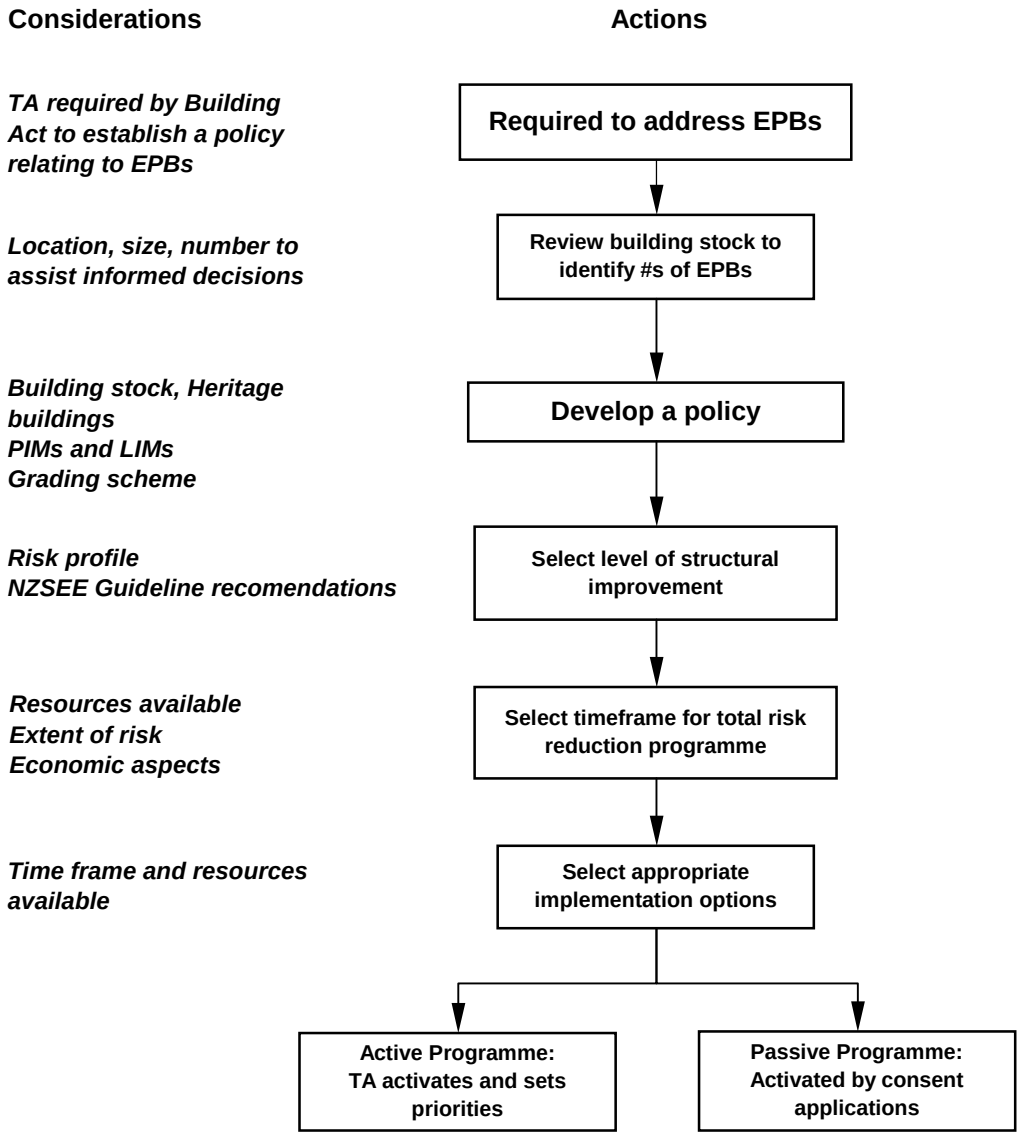
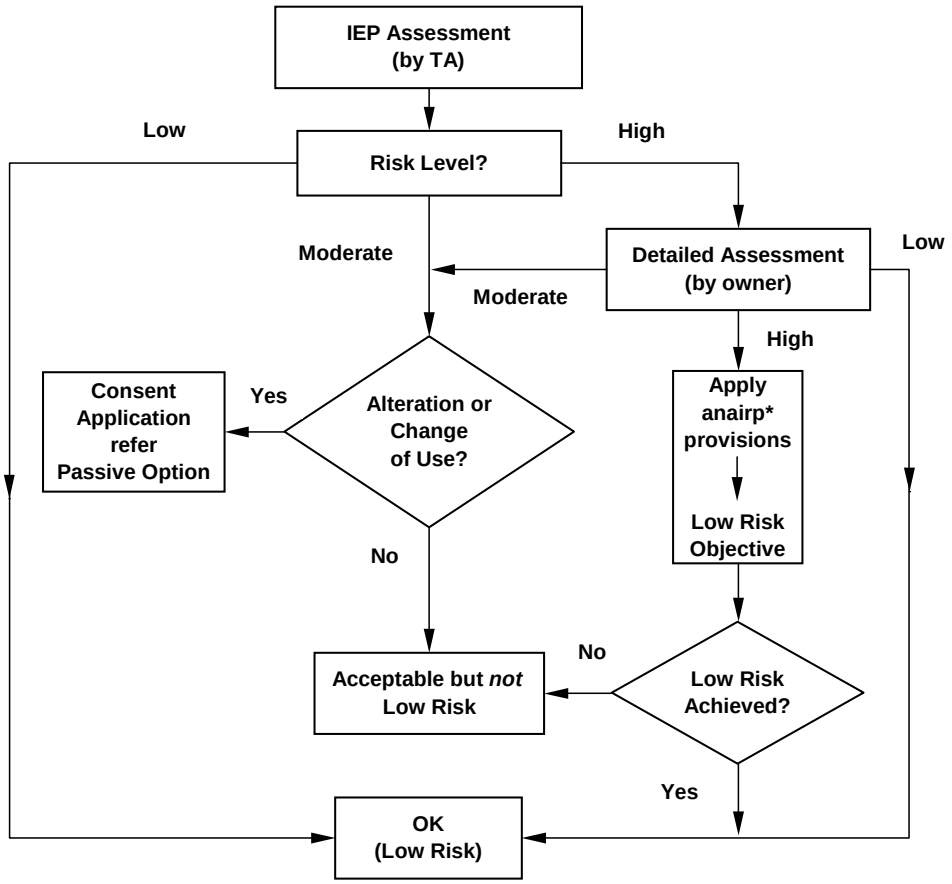


Figure 2.2: Implementation options and processes

2.9.2 Active Programme

The active programme provides for a proactive approach by the TA to identify and address buildings deemed not to be safe in earthquake. The basic steps in implementing an active programme are clearly set out in Figure 2.3. (Note that the NZSEE distinction between high, moderate, and low risk buildings is included in this Figure)

Low Risk is 67*%NBS* or greater. Moderate Risk is between 34 and 66*%NBS*. High Risk is 33*%NBS* or less.



* anairp = as nearly as is reasonably practicable to 100*%NBS*.

Figure 2.3: Outline of Steps in Active Programme

Note 1: A 67*%NBS* is regarded as an acceptable level by NZSEE if improvement beyond this level is difficult to achieve. However, NZSEE recommends that the standard is raised to as nearly as is reasonably practicable to that of a new building. The initial target for improvement should thus always be 100*%NBS*.

In addition to the general comments in Section 2.7, the following should be noted.

a) Initial evaluation process

It is recommended that the territorial authority (or its consultants) would carry out the assessment at the TA's cost. In the interest of transparency and fairness, it would be unreasonable for a TA to initiate an active programme until all potentially high risk buildings have been identified.

b) Decide appropriate performance level

Refer to Section 2.7.7 above.

c) Determine a priority list

Refer to Section 2.7.6 above.

d) Determine timetables

Refer to Section 2.7.8 above.

e) Issue notices to owners

Issue initial notices to owners in accordance with Section 2.7.9.

f) Detailed evaluation by owners

Within the time specified by the TA, owners would be required to have a detailed evaluation carried out and submitted to the TA. Alternatively the owner may elect to accept that the building is high risk, and proceed to (h) below.

g) Building passes criteria – building not a high risk (Earthquake Prone) building

If this shows that the building passes the criteria set down in the Act, the TA should issue a formal notice to that effect, i.e. withdrawing the original notice.

h) Building fails criteria – review by territorial authority/owner

Refer to Section 2.7.10 above.

i) Issue of amended notice

The territorial authority issues an amended formal notice, if appropriate, to record any agreed changes to the previous notice.

j) Action by owner

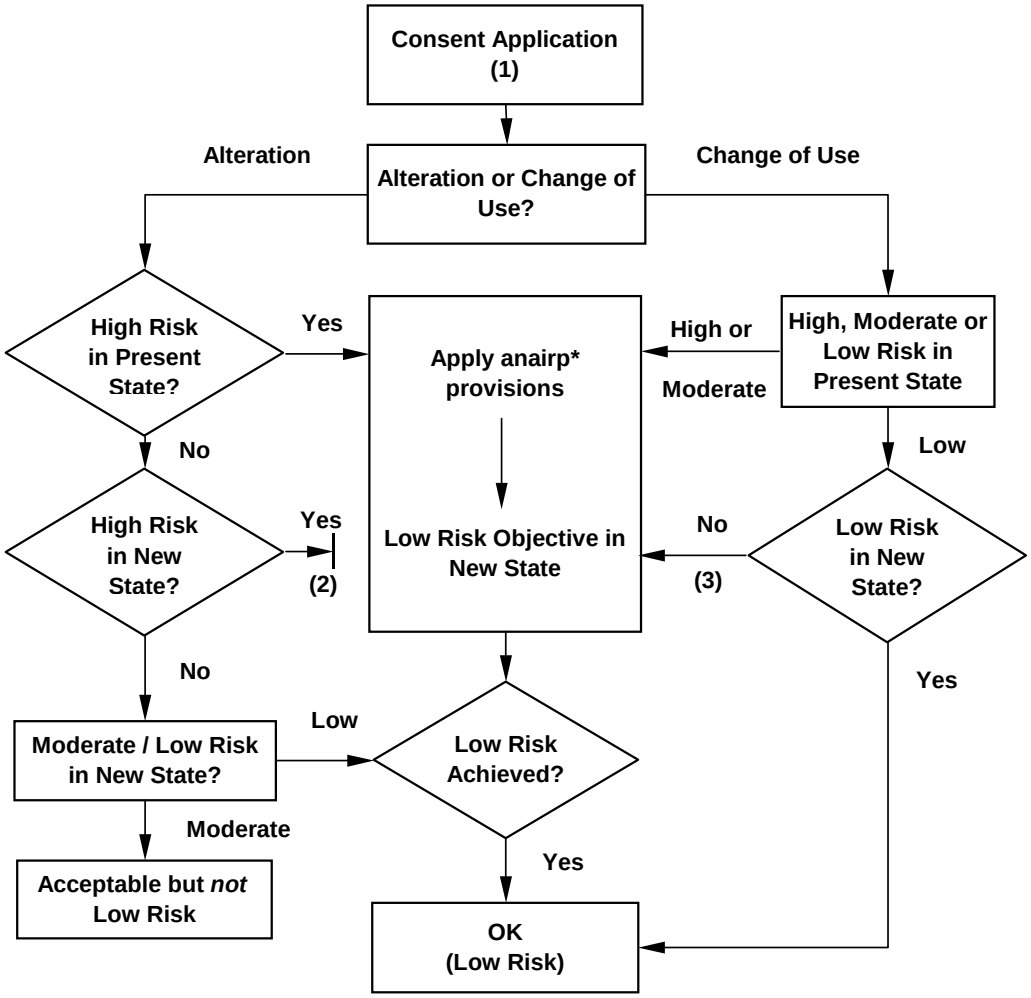
The owner is required to take action to mitigate according the terms of the notice.

k) Monitoring by territorial authority

The TA should use its information systems to give notice in advance of notified dates within which the work is to be completed. The TA may elect to issue a reminder notice to the owner at an appropriate time.

2.9.3 Passive Programme

In the passive programme, the identification and rectification of high risk buildings is initiated by an application by the owner under the Building Act. The basic steps of this process are shown in Figure 2.4.



1 Note that receipt of consent application triggers TA to apply Sections 112 and 115.
2 Cannot be made worse, therefore not possible.
3 Possible especially if new use requires higher performance standard.
* anairp = as nearly as is reasonably practicable to 100%*NBS*.

Figure 2.4: Outline of Steps in Passive Programme

(Refer Note 1 on Figure 2.3 for difference between NZSEE classification and requirements of Act.)

The following comments apply:

a) Adopt policy guidelines for performance improvement

Refer to Section 2.7.7 above.

b) Adopt policy guidelines for timetables for evaluation and improvement

Refer to Section 2.7.8. above

c) Evaluation with consent

An appropriate evaluation, as required by the policy, should be provided by the owner as part of the consent application. Refer to Sections 2.7.3 and 2.7.4 above.

d) Result of evaluation – initial notice

If the IEP indicates that the building is a high risk building, an initial notice should be issued as Section 2.7.9. The territorial authority may consider whether or not to require mitigation work within a specified time. This would help in situations where the owner decides not to proceed with the alteration or change of use.

e) Detailed evaluation by owners

Within the time specified by the territorial authority, owners would be required to have a detailed evaluation carried out and submitted to the territorial authority. Alternatively the owner may elect to accept that the building is high risk, and proceed to (g) below.

f) Building passes criteria – building not a high risk (Earthquake Prone) building

If this shows that the building is not high risk, the TA should issue a formal notice to that effect, i.e. withdrawing the original notice. However the building would still be subject to the requirements of Section 112 or 115 as appropriate.

g) Building fails criteria – review by Territorial Authority/owner

Refer to Section 2.7.10 above.

h) Issue of amended notice

The territorial authority issues an amended formal notice, if appropriate, to record any agreed changes to the previous notice.

i) Action by owner

The owner is required to take action to mitigate according the terms of the notice.

j) Detailed evaluation by owner

This would follow the IEP, either to demonstrate that the building was not high risk or to demonstrate that it would be upgraded to low risk if proposed alteration work/change of use were to be effected.

If the building in its existing condition was shown to be not high risk, the TA should issue a formal notice to that effect. The building would still be subject to the requirements of Section 112 or 115 as applicable. This could include upgrading to low risk for a change of use application.

If the building in its existing condition is shown to be high risk, and will not be upgraded to low risk as a result of the proposed work, the TA will either confirm the previous notice or issue a formal notice to that effect. In all cases this should be a formality only.

k) Review by territorial authority/owner

This step highlights the need for dialogue between the TA and owner. They should each be encouraged to focus on the overall objective which is to reduce earthquake risk.

l) Issue of amended notice

This step is simply to set down the requirements following review by territorial authority and owner. The form of the notice would need to be compatible with the nature of the development.

m) Action by owner

The owner is required to take action to mitigate according to the terms of the notice.

n) Monitoring by Territorial Authority

The TA should use its information systems to give notice in advance of notified dates within which the work is to be completed. The TA may elect to issue a reminder notice to the owner at an appropriate time.

Section 3 - Initial Evaluation Procedure

3.1 Background

The NZSEE recommends a two-stage evaluation process. The initial evaluation procedure (IEP) is intended to be a coarse screening involving as few resources as reasonably possible. It is expected that the IEP will be followed by a more detailed assessment for those buildings identified in the evaluation as likely to be Earthquake Prone (EPB) in terms of the provisions of the NZ Building Act 2004.

Key elements of the procedures are:

a) an initial evaluation (refer to this Section 3)

b) a detailed assessment for buildings not passing the initial evaluation (refer to Section 4)

c) a requirement to improve the structural performance of buildings failing the detailed evaluation (refer to Section 5)

d) provision for an optional earthquake risk grading for all buildings (refer to Section 3.3 below).

This Section 3 of the NZSEE Guidelines describes the Initial Evaluation Procedure (IEP). Procedures for the detailed evaluation and guidelines for the improvement of structural performance are given in Sections 4 and 5 respectively.

Note that the objective of the initial evaluation is to identify, with an acceptable confidence level, all those buildings which will be potentially Earthquake Prone. At the same time the initial evaluation process must not catch an unacceptable number of buildings which on detailed evaluation, pass the test.

It is envisaged that the IEP would be applied by experienced earthquake engineers, with specific training in its application, on behalf of:

- a territorial local authority – to review its building stock as part of its seismic policy, preparatory to issuing notices to building owners
- building owners and managers – as part of overall risk management, and in response to the new legislation.

It is expected that those carrying out initial evaluations would be New Zealand Chartered Professional Engineers, or equivalent, who have:

- sufficient relevant experience in the design and evaluation of buildings for earthquake effects to exercise the degree of judgement required and
- had specific training in the objectives of and processes involved in the initial evaluation procedure.

3.2 Outline of the Process

An outline of the overall evaluation process envisaged is given in Figure 2.1.

An outline of the Initial Evaluation Procedure (IEP) is shown in Figure 3.1.

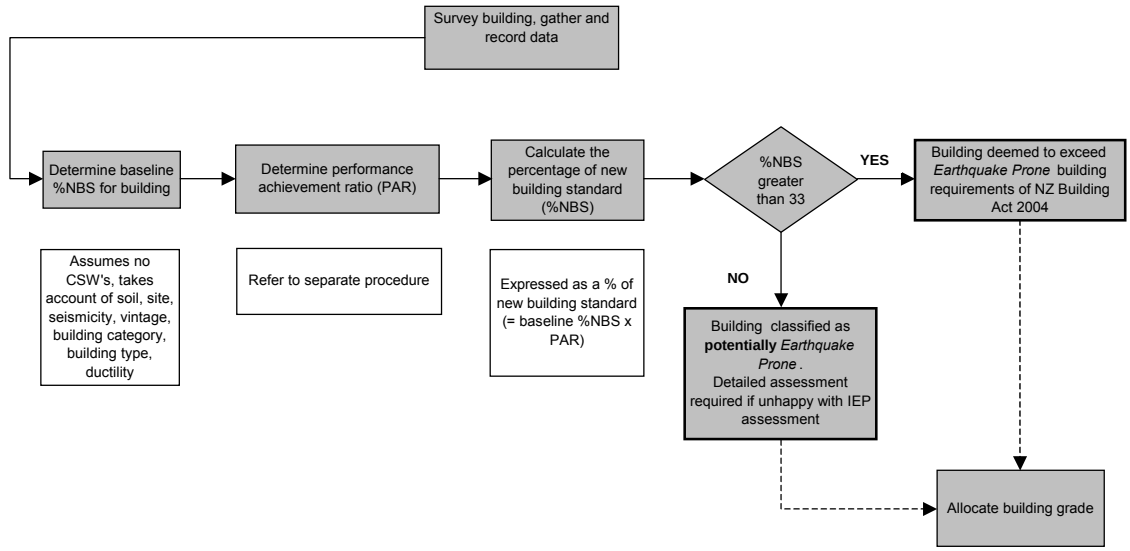


Figure 3.1: Diagrammatic representation of Initial Evaluation Procedure

The process involves making an initial assessment of the performance of existing buildings against the standard required for a new building (i.e. "percentage new building standard" (*% NBS*)).

The IEP outlined below is based on the current standard for earthquake loadings for new buildings in New Zealand, NZS 1170.5:2004. It is assumed that the person carrying out the IEP has a good knowledge of the requirements of this Standard.

The first step is to survey the subject building to gather relevant data on its characteristics, sufficient for use in the IEP.

The next step is to apply the IEP to the candidate building. For each building, the percentage of new building standard (*%NBS*) is determined. *%NBS* is essentially the assessed structural performance of the building (taking into consideration all reasonably available information) compared with requirements for a new building expressed as a percentage. There are several steps involved in determining *%NBS*, as outlined in the following sections.

A *%NBS* of 33 or less means that the building is assessed as potentially Earthquake Prone in terms of the Building Act and a more detailed evaluation of it will typically be required.

A *%NBS* of greater than 33 means that the building is regarded as outside the requirements of the Act. No further action on it will be required by law, however it may still be considered as representing an unacceptable risk and further work on it may be recommended.

A *%NBS* of 67 or greater means that the building is not considered to be a significant earthquake risk.

The IEP is designed as a largely qualitative process involving considerable knowledge of earthquake behaviour of buildings and judgement as to key attributes and their effect on performance.

Due to the qualitative nature of the assessment it should not come as a surprise that in some circumstances assessments of the same building by two or more experienced engineers will differ. This is to be expected, as the evaluation of seismic performance is not an exact science. However, it is also expected that experienced engineers will be able to identify the critical issues that are

likely to effect seismic performance and that, through discussion, a consensus position will be able to be agreed. For the same reason, an IEP assessment that has been independently reviewed is likely to be more robust than one based solely on the judgement of one engineer.

A *%NBS* of 33 or less should only be taken as an indication that the building is potentially Earthquake Prone and a detailed assessment may well show that a higher level of performance is achievable.

The slight skewing of the IEP towards conservatism should give confidence that a building assessed as having a *%NBS* greater than 33 by the IEP is unlikely to be shown, by later detailed assessment, to be earthquake prone. There will be exceptions, particularly when CSWs (other than those used to calculate the PAR) are present that can not be recognised from what is largely a visual assessment from the exterior of the building, or when the original design was deficient (compared with the code of the day).

For a typical multi-storey building, the process is envisaged as requiring limited effort and cost. It would be largely a visual assessment, but supplemented by information from previous assessments, readily available documentation and general knowledge of the building.

The IEP should be repeated if more information comes to hand.

The IEP as presented can be used for unreinforced masonry buildings, however may be difficult to apply in some circumstances. An attribute scoring process (refer Appendix 3B) is suggested as an alternative to Steps 2 and 3 described below but will generally require a greater knowledge of the building than required for the IEP.

3.3 Summary of Step-by-Step Procedures

The IEP is shown in Figure 3.2 and described in the following Steps:

Steps 2 and 3 may not be appropriate for unreinforced masonry buildings and assessors are referred to an alternative approach outlined in Appendix 3B which uses attribute scoring to assess *%NBS* directly.

Step 1: General Information

Use Table IEP-1.

1.1 Add photos of exterior of building for all visible exterior faces, showing features.

1.2 Draw a rough sketch of the building plan that can be ascertained from the exterior of the building, noting relevant features.

1.2 List any particular features that would be relevant to the seismic performance of the building.

1.3 Note any information sources used to complete the assessment.

Step 2: Determine baseline percent new building standard $(\%NBS)_b$

Use Table IEP-2. ***Use a separate form for each orthogonal direction*** unless it is clear from the start which governs:

1.1 Refer to Figure 3.3 for $(\%NBS)_{nom}$.

1.2 Refer to NZS 1170.5:2004 for Near Fault factor.

1.3 Refer to NZS 1170.5:2004 for Hazard factor.

1.4 a) Assess Building Importance Level. Refer to NZS 1170.0:2004.

b) Refer to Table 3.1 for Return Period Scaling Factor.

1.5 a) Assess Ductility of existing structure. Refer to Table 3.2 for maximum.

b) Refer to Table 3.3 for Ductility scaling factor.

1.6 Assess structural performance factor. Refer to Figure 3.4

1.7 $(\%NBS)_b = (\%NBS)_{nom}$ x A x B x C x D x E as shown.

Step 3: Determine performance achievement ratio (PAR)

Use Table IEP-3. ***Use a separate form for each orthogonal direction*** unless it is clear from the start which governs:

- Assess effect on structure of each potential Critical Structural Weakness (CSW). *(Choose from the factors given – do not interpolate.)*
- Refer Section 3.5.2 for guidance.
- PAR = A x B x C x D x E x F as shown.

Step 4: Determine the percentage of new building standard, %NBS

Use Table IEP-4. Compare product of $(\% NBS)_b$ x PAR for each direction (if applicable):

- $\%NBS$ = PAR x $(\%NBS)_b$.

Step 5: Earthquake Prone?

Use Table IEP-4. Assess on basis of *%NBS* in Step 4:

- If $\%NBS > 33$ then not Earthquake Prone.
- If $\%NBS \leq 33$ then a more detailed evaluation is needed.

Step 6: Earthquake Risk?

Use Table IEP-4. Assess on basis of *%NBS* in Step 3:

- If $\%NBS \geq 67$ then not a significant earthquake risk.
- If $\%NBS < 67$ then a more detailed evaluation may be recommended.

Step 7: Provisional Grading based on IEP

Use Table IEP-4. Assess on basis of *%NBS* in Step 4:

- Grade building according to *%NBS* using table provided. (See also Table 2.1, Section 2.8.) Use the worse result as basis if both directions have been assessed.

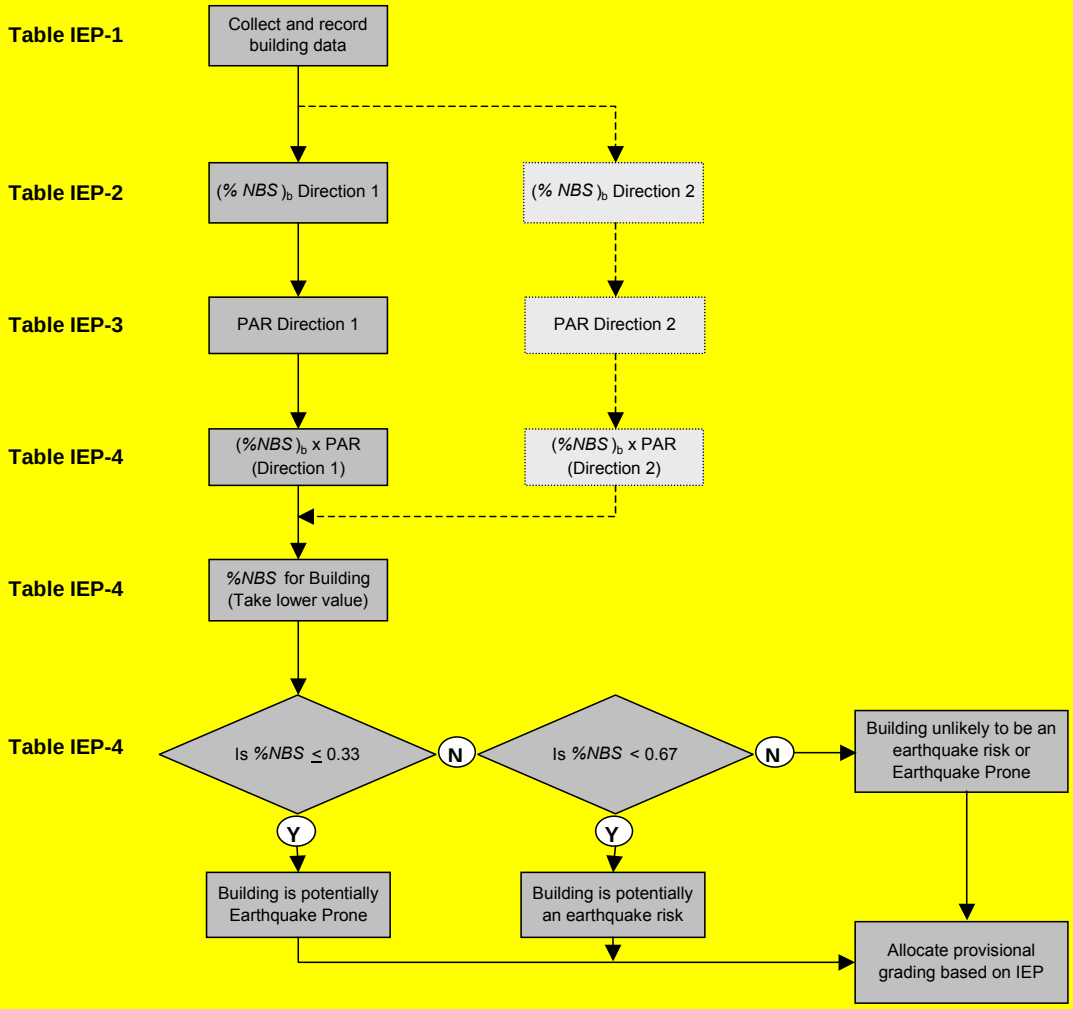


Figure 3.2: Initial Evaluation Procedure

Table IEP-1: Initial Evaluation Procedure – Step 1

Table IEP-1 **Initial Evaluation Procedure Step 1** **Page 1....**

(Refer Table IEP - 2 for Step 2; Table IEP - 3 for Step 3, Table IEP - 4 for Steps 4, 5 and 6)

Building Name	**Ref.**
Location	**By**
	Date

Step 1 - General Information

1.1 Photos (attach sufficient to describe building)

1.2 Sketch of building plan

1.3 List relevant features

1.4 Note information sources

	tick as appropriate
Visual Inspection of Exterior	
Visual Inspection of Interior	
Drawings (note type)	
Specifications	
Geotechical Reports	
Other (list)	

Table IEP-2: Initial Evaluation Procedure – Step 2

Table IEP-2 **Initial Evaluation Procedure Step 2** **Page 2….**

(Refer Table IEP - 1 for Step 1; Table IEP - 3 for Step 3; Table IEP - 4 for Steps 4, 5 and 6)

Building Name	**Ref.**
Location	**By**
Direction Considered: a) Longitudinal b) Transverse	
(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)	**Date**

Step 2 - Determination of $(\%NBS)_b$

2.1 Determine nominal (%NBS) = $(\%NBS)_{nom}$

a) Date of Design and Seismic Zone

tick as appropriate

Pre 1935		☐	See also notes 1, 3
1935-1965		☐	
1965-1976	**Seismic Zone; A**	☐	
	B	☐	
	C	☐	
1976-1992	**Seismic Zone; A**	☐	See also note 2
	B	☐	
	C	☐	
1992-2004		☐	

b) Soil Type

From NZS1170.5:2004, Cl 3.1.3	**A or B Rock**	☐
	C Shallow Soil	☐
	D Soft Soil	☐
	E Very Soft Soil	☐
From NZS4203:1992, Cl 4.6.2.2	**a) Rigid**	☐
(for 1992 to 2004 only and only if known)	**b) Intermediate**	☐

c) Estimate Period, *T* ☐ Seconds

Can use following:

$T = 0.09h_n^{0.75}$	for moment-resisting concrete frames
$T = 0.14h_n^{0.75}$	for moment-resisting steel frames
$T = 0.08h_n^{0.75}$	for eccentrically braced steel frames
$T = 0.06h_n^{0.75}$	for all other frame structures
$T = 0.09h_n^{0.75}/A_c^{0.5}$	for concrete shear walls
$T \leq 0.4$sec	for masonry shear walls

Where h_n = height in m from the base of the structure to the uppermost seismic weight or mass.

$A_c = \Sigma A_i(0.2 + L_{wi}/h_n)^2$

A_i = cross-sectional shear area of shear wall i in the first storey of the building, in m^2

l_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces, in m

with the restriction that l_{wi}/h_n shall not exceed 0.9

d) $(\%NBS)_{nom}$ determined from Figure 3.3 ☐ $(\%NBS)_{nom}$

Note 1: For buildings designed prior to 1965 and known to be designed as public buildings in accordance with the code of the time, multipy $(\%NBS)_{nom}$ by 1.25. ☐

For buildings designed 1965 - 1976 and known to be designed as public buildings in accordance with the code of the time, multiply $(\%NBS)_{nom}$ by 1.33 - Zone A
1.2 - Zone B

Note 2: For reinforced concrete buildings designed between 1976-84 multiply $(\%NBS)_{nom}$ by 1.2 ☐

Note 3: For buildings designed prior to 1935 multiply $(\%NBS)_{nom}$ by 0.8 except for Wellington where the factor may be taken as 1. ☐ ☐ $(\%NBS)_{nom}$

Continued over page

1

Table IEP-2: Initial Evaluation Procedure – Step 2 continued

Table IEP-2 Initial Evaluation Procedure Step 2 continued **Page 3….**

2.2 Near Fault Scaling Factor, Factor A

If $T \leq 1.5$sec, Factor A = 1

a) Near Fault Factor, *N(T,D)*
(from NZS1170.5:2004, Cl 3.1.6)

b) Near Fault Scaling Factor = 1/N(T,D) **Factor A**

2.3 Hazard Scaling Factor, Factor B

a) Hazard Factor, *Z, for site*
(from NZS1170.5:2004, Table 3.3)

b) Hazard Scaling Factor

For pre 1992 = $1/Z$
For 1992 onwards = Z_{1992}/Z

(Where Z_{1992} is the NZS4203:1992 Zone Factor from accompanying Figure 3.5(b)) **Factor B**

2.4 Return Period Scaling Factor, Factor C

a) Building Importance Level
(from NZS1170.0:2004, Table 3.1 and 3.2)

b) Return Period Scaling Factor from accompanying Table 3.1 **Factor C**

2.5 Ductility Scaling Factor, D

a) Assessed Ductility of Existing Structure, μ
(shall be less than maximum given in accompanying Table 3.2)

b) Ductility Scaling Factor

For pre 1976 = k_μ
For 1976 onwards = 1

(where k_μ is NZS1170.5:2004 Ductility Factor, from accompanying Table 3.3) **Factor D**

2.6 Structural Performance Scaling Factor, Factor E

a) Structural Performance Factor, S_p from accompanying Figure 3.4

b) Structural Performance Scaling Factor = $1/S_p$ **Factor E**

2.7 Baseline %NBS for Building, $(\%NBS)_b$

(equals $(\%NSB)_{nom}$ x A x B x C x D x E)

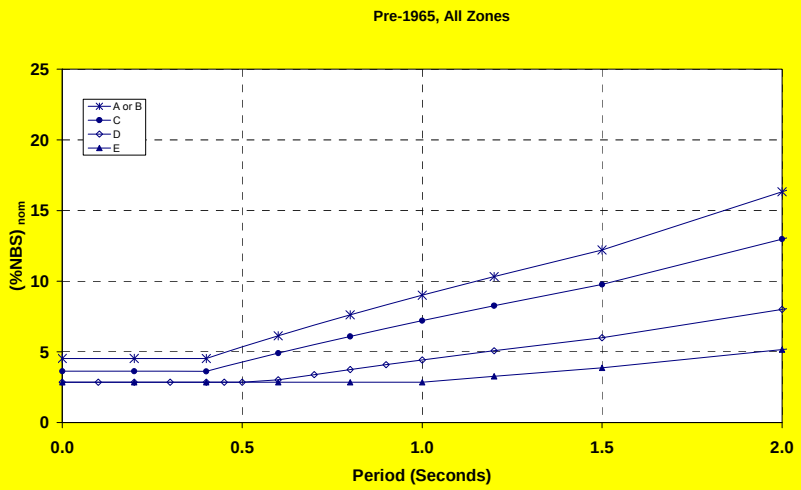


Figure 3.3(a): $(\%NBS)_{nom}$ Pre- 1965, All Zones

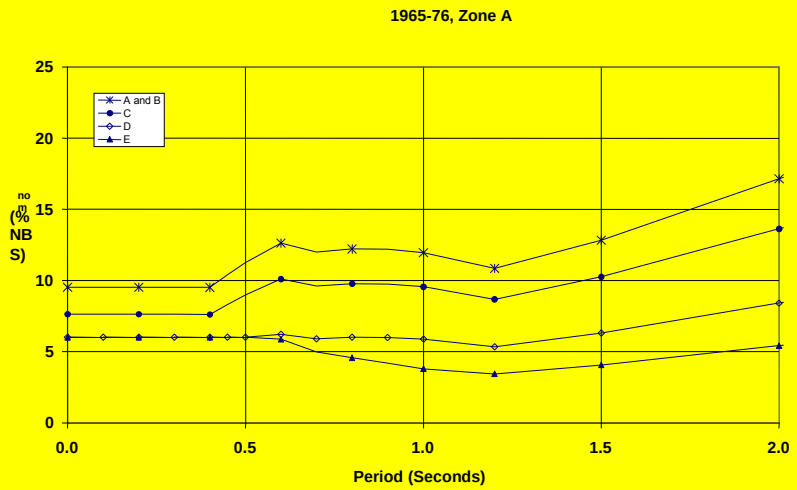


Figure 3.3(b): $(\%NBS)_{nom}$ 1965-76, Zone A

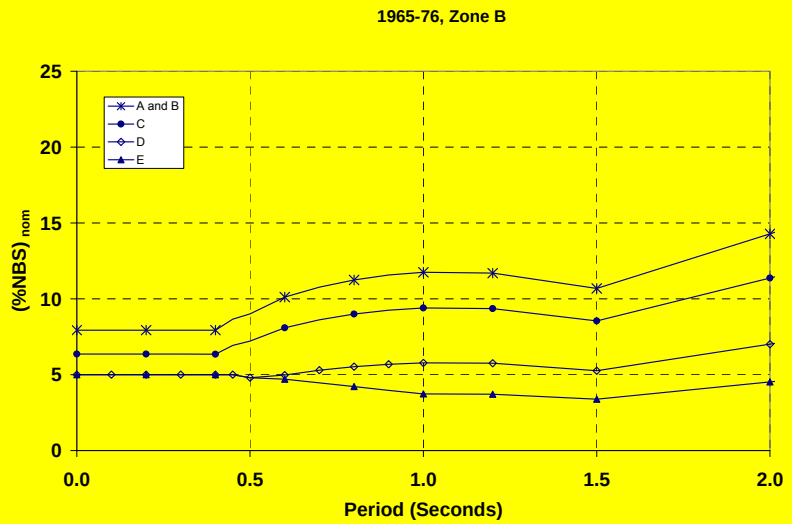


Figure 3.3(c): $(\%NBS)_{nom}$ 1965-76, Zone B

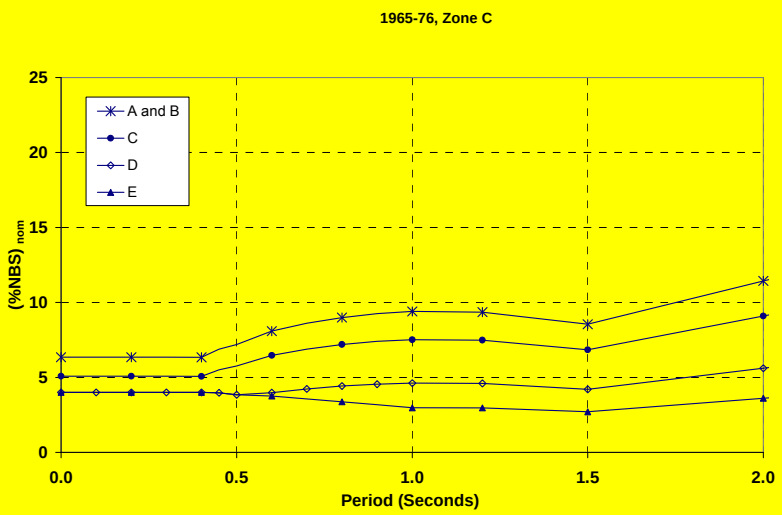


Figure 3.3(d): $(\%NBS)_{nom}$ 1965-76, Zone C

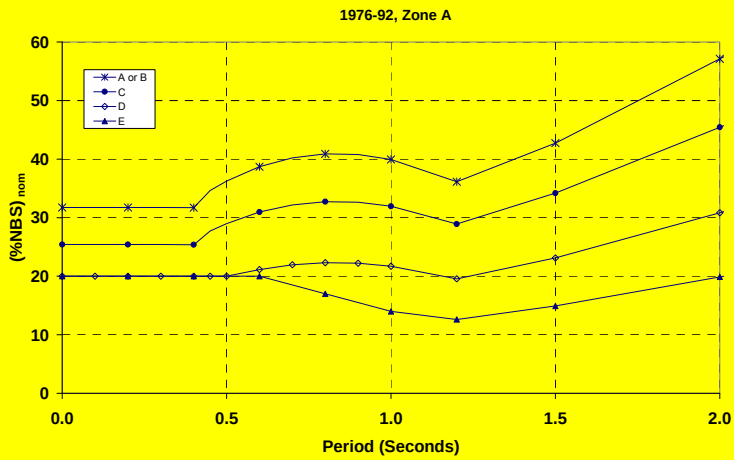


Figure 3.3(e): $(\%NBS)_{nom}$ 1976-92, Zone A

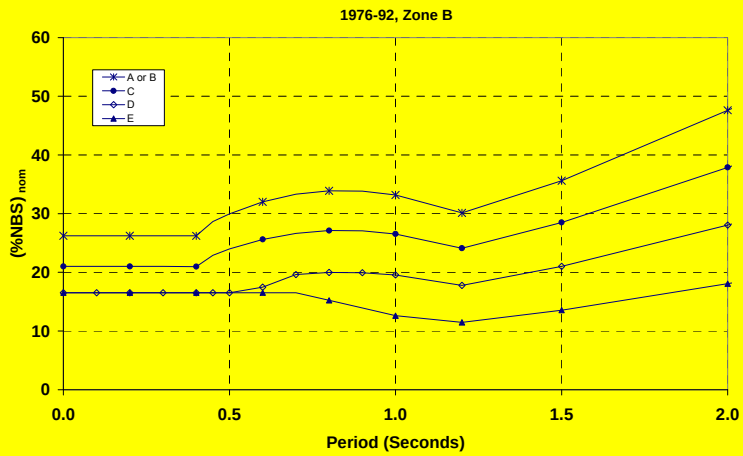


Figure 3.3(f): $(\%NBS)_{nom}$ 1976-92, Zone B

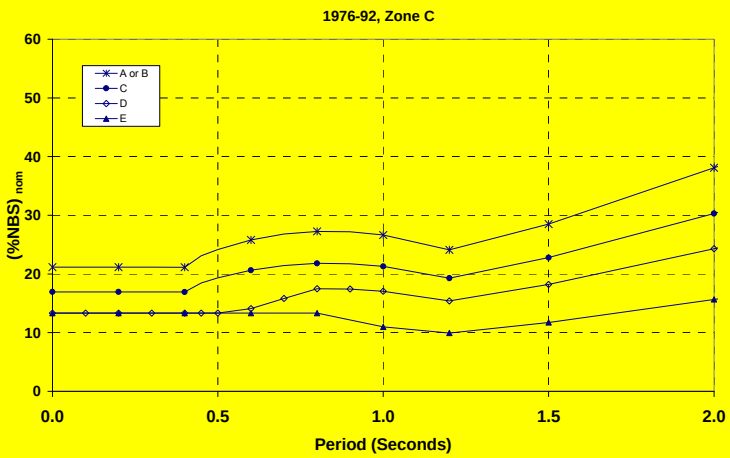


Figure 3.3(g): $(\%NBS)_{nom}$ 1976-92, Zone C

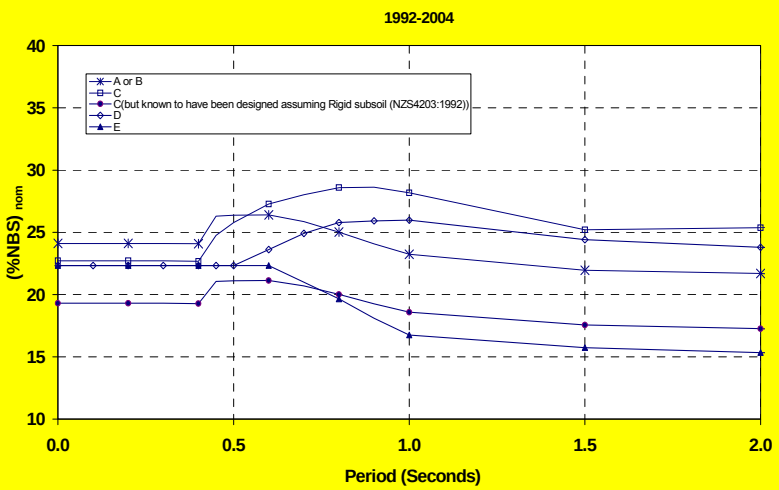


Figure 3.3(h): $(\%NBS)_{nom}$ 1992-2004

Figure 3.3: $(\%NBS)_{nom}$ for Different Building Design Vintages

Table 3.1: Return period scaling factor

NZS1170.5:2004 Return Period Factor, *R*				Return Period Scaling Factor, *C*			
Importance level	Comment	Annual Probability of Exceedance	Return Period Factor, *R*	Pre 1965	1965-76	1976-92	1992-04
1	Minor structures (failure no t likely to endanger human life)	1/100	0.5	2	2	2	1.2
2	Normal structures and structures not falling into other levels	1/500	1	1	1	1	1
3	Major structures (affecting crowds)	1/1000	1.3	0.8	0.8	1	0.9
4	Post-disaster structures (post-disaster functions or dangerous activities)	1/2500	1.8	0.6	0.6	0.9	0.7
5	Exceptional structures are outside the scope of the IEP, special study required.						

1,2

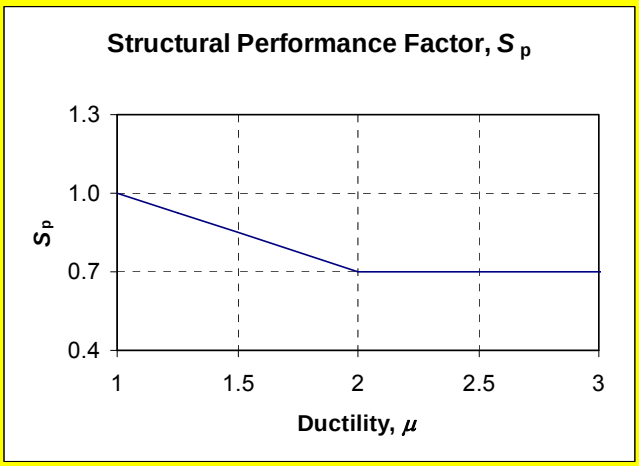
Where R is the return period factor appropriate to the current use of the building, as shown in Table 3.5 of NZS 1170.0:2002

Table 3.2: Ductility factors to be used for existing buildings

Structure Type	Maximum allowable ductility factor for IEP			
	Pre-1935	1935-65	1965-76	1976-2004
All buildings	2	2	2	6

Table 3.3: Ductility scaling factor

	Structural Ductility Scaling Factor, k_μ							
	1.0 or less		1.25		1.50		2	
Soil Type	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E
Period, *T*								
≤ 0.40s	1	1	1.14	1.25	1.29	1.50	1.57	1.70
0.50s	1	1	1.18	1.25	1.36	1.50	1.71	1.75
0.60s	1	1	1.21	1.25	1.43	1.50	1.86	1.80
0.70s	1	1	1.25	1.25	1.50	1.50	2.00	1.85
0.80s	1	1	1.25	1.25	1.50	1.50	2.00	1.90
≥1.00s	1	1	1.25	1.25	1.50	1.50	2.00	2.00



Where S_p is the Structural Performance Factor from NZS1170.5:2004, Cl 4.4.2.

Figure 3.4: Structural performance factor, S_p

Figure 3.5(a) NZS 4203:1984, NZS 4203: 1976 and NZS 1900:Chapter 8 sim.

Figure 3.5(b) NZS 4203:1992

Figure 3.5: Extracts from previous Standards showing seismic zoning schemes

Table IEP-3: Initial evaluation procedure – Step 3

Table IEP-3 Initial Evaluation Procedure Step 3 **Page ……**

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2; Table IEP - 4 for Steps 4, 5 and 6)

Building Name	**Ref.**
Location	**By**
Direction Considered: a) Longitudinal b) Transverse	
(Choose worse case if clear at start. Complete IEP-2 and IEP-3 for each if in doubt)	**Date**

Step 3 - Assessment of Performance Achievement Ratio (PAR)

2 *(Refer 3.4.3)*

Critical Structural Weakness	Building Score	Effect on Structural Performance (Choose a value - Do not interpolate)		
3.1 Plan Irregularity				
Effect on Structural Performance		Severe	Significant	Insignificant
	Factor A	0.4 max	0.7	1
Comment				
3.2 Vertical Irregularity				
Effect on Structural Performance		Severe	Significant	Insignificant
	Factor B	0.4 max	0.7	1
Comment				
3.3 Short Columns				
Effect on Structural Performance		Severe	Significant	Insignificant
	Factor C	0.4 max	0.7	1
Comment				

3.4 Pounding Potential

(Estimate D1 and D2 and set D = the lower of the two, or =1.0 if no potential for pounding)

a) Factor D1: - Pounding Effect

Select appropriate value from Table

Note:
Values given assume the building has a frame structure. For stiff buildings (eg with shear walls), the effect of pounding may be reduced by taking the co-efficient to the right of the value applicable to frame buildings.

Factor D1

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	0.7	0.8	1
Alignment of Floors not within 20% of Storey Height	0.4	0.7	0.8

b) Factor D2: - Height Difference Effect

Select appropriate value from Table

Factor D2

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	0.4	0.7	1
Height Difference 2 to 4 Storeys	0.7	0.9	1
Height Difference < 2 Storeys	1	1	1

Factor D ***(Set D = lesser of D1 and D2 or.. set D = 1.0 if no prospect of pounding)***

3.5 Site Characteristics - *(Stability, landslide threat, liquefaction etc)*

Effect on Structural Performance	Severe	Significant	Insignificant
Factor E	0.5 max	0.7	1

3.6 Other Factors

Factor F **For ≤ 3 storeys - Maximum value 2.5, otherwise - Maximum value 1.5. No minimum.**

Record rationale for choice of Factor F:

……

3.7 Performance Achievement Ratio (PAR)
(equals **A x B x C x D x E x F**)

Table IEP-4: Initial evaluation procedure – Steps 4, 5 and 6

Table IEP- 4 Initial Evaluation Procedure Steps 4, 5 and 6 **Page ...**

(Refer Table IEP - 1 for Step 1; Table IEP - 2 for Step 2; Table IEP - 3 for Step 3)

Building Name	**Ref.**
Location	**By**
	Date

Step 4 - Percentage of New Building Standard (*%NBS*)

	Longitudinal	Transverse	
4.1 Assessed Baseline (*%NBS*)$_b$ (from Table IEP - 2)			2
4.2 Performance Achievement Ratio (PAR) (from Table IEP - 3)			2
4.3 PAR x Baseline (*%NBS*)b			
4.4 Percentage New Building Standard (*%NBS*) (Use lower of two values from Step 4.3)			2

Step 5 - Potentially Earthquake Prone? (Mark as appropriate)	*%NBS* > 33	NO
	%NBS ≤ 33	YES
Step 6 - Potentially Earthquake Risk? (Mark as appropriate)	*%NBS* ≥ 67	NO
	%NBS < 67	YES

Step 7 - Provisional Grading for Seismic Risk based on IEP

Seismic Grade

Evaluation Confirmed by... ..... **Signature**

..... **Name**

..... **CPEng. No**

Relationship between Seismic Grade and *%NBS*:

Grade:	A+	A	B	C	D	E
%NBS:	> 100	100 to 80	80 to 67	67 to 33	33 to 20	< 20

3.4 Background Guidelines and Commentary

3.4.1 Step 1- Collection of information (Table IEP-1)

The first step in the IEP should be to collect relevant information necessary to carry out the assessment and to record this as the basis of the assessment. It is a fundamental premise of the IEP that limited definitive information is likely to be available and the assessment will necessarily be made on the basis of a visual inspection of only the exterior of the building.

Photographs of the building are likely to be taken as part of the IEP and should form part of the permanent record. Likewise a record of the features observed and the extent of information that was available at the time of the assessment will be important considerations if the assessment is questioned in the future. Table IEP-1 provides a means of recording this information.

3.4.2 Step 2- Procedure for assessment of $(\%\ NBS)_b$ (Table IEP-2)

Introduction

One of the first questions asked regarding existing buildings is about their overall expected performance in relation to a building designed to the standard required for new buildings, NZS 1170.5:2004. The comparison available through the IEP provides a simple and convenient measure of relative performance in earthquake.

It must be emphasised that the percentage figure $(\%NBS)_b$ derived is but a first step in any evaluation. It gives only an indication of the likely situation. It does not take full account of the particular characteristics of a building. These may be beneficial (as in the case when extra walls are added for architectural reasons but are nevertheless significant structural elements). Nor does it take into account the effect of critical structural weaknesses (CSWs) that can greatly reduce the overall figures given in the following charts and tables.

Approach

There are a number of variables that feed into the calculation of a *baseline* percent current code ratio. These include:

- the natural period of vibration of the building
- location in relation to seismic risk
- the sub soil characteristics
- the vintage or code to which it was designed
- the equivalent ductility of the building.

Different codes have had different requirements for design over the years. Essentially this boils down to:

- pre-1935: no seismic design (except for buildings in Wellington)
- pre-1965: design for 0.08 g lateral force applied uniformly over the height or 0.06 g applied as an inverted triangle
- 1965-76: design to NZS 1900:1965, Chapter 8.
- 1976-92: design to NZS 4203:1976, some changes for reinforced concrete structures in 1984
- 1992-2004: design to NZS 4203:1992

Definitions

1 $(\%NBS)_{nom}$ ⇒ The assessed performance (strength) compared to NZS 1170.5:2004 assuming ductility of 1.0, Hazard factor of 1.0, Near Fault factor of 1.0, Return Period factor of 1.0, and Structural Performance factor of 1.0 Refer Table IEP-1.

$(\%.NBS)_{nom} / S_p$ is a general measure of the performance (with respect to requirements for a new building) of a particular building in a given location, assuming it is well designed, of regular form, with no critical structural weaknesses and complying with the relevant code provisions at the time it was built.

2 $(\%NBS)_b$ ⇒ Modifies $(\%NBS)_{nom}$ to account for assessed ductility, location (hazard factor and near fault factor NZS1170.5) and occupancy category (i.e. return period factor) but assuming a good structure complying with code of the time it was built.

The resulting value of $(\%NBS)_b$ (baseline *% NBS*) may be regarded as a measure of the performance of a well designed and constructed regular building of its type and vintage on the site in question. It is a "yardstick" against which to measure the effect of critical structural weaknesses which may exist in a particular building of the same type.

Note that an assessment of the likely ductility is required.

3 PAR ⇒ The performance achievement ratio (PAR) may be regarded as the ratio of the performance of the particular building, as inspected, in relation to "a well designed and constructed regular building of its type and vintage on the site in question". Thus "a well designed etc building ..." would have a PAR of 1.0. Refer Table IEP-3.

4 *%NBS*⇒ Percentage of new building standard (*%NBS*). This adjusts $(\%NBS)_b$ to account for particular characteristics of the building especially critical structural weakness. Refer Table IEP-4.

Note: *%NBS* = $(\%NBS)_b$ x Performance Achievement Ratio (PAR)

= a measure, in percentage terms, of the earthquake performance of the building under consideration with respect to NZS 1170.5:2004, taking into account critical structural weaknesses and other relevant features.

Step 2.1: Determine nominal percent of new building standard $(\%NBS)_{nom}$

Note: Consider each orthogonal direction separately unless it is clear from the start which governs.

a) Determine code used in design of building:

- Pre-1965 (0.08 g uniform load)
- 1965-1976 (NZS 1900 Ch. 8):
  - Zone A
  - Zone B
  - Zone C

- 1976-1992 (NZS 4203:1976)
  - Zone A
  - Zone B
  - Zone C
- 1992-2004 (NZS 4203:1992)
- Pre-1935:
  Treat as 80% NZS 1900 Ch. 8 for all buildings, except in Wellington where a seismic code was in place prior to 1935 and 100% NZS 1900 Ch 8 is appropriate. The allowance made for pre 1935 buildings is nominal only. It is expected that major deficiencies, if any, will be picked up in the assessment of the PAR.

b) Determine soil type at the site:

- Use NZS 1170.5:2005 classifications:
  - Class A – Strong rock.
  - Class B – Rock.
  - Class C – Shallow soil sites.
  - Class D – Deep or soft soil sites.
  - Class E – Very soft soil sites.

c) Assess period of building:

- Use any recognised method.
- Note that accurate analysis is not warranted in many cases since results are not highly sensitive to changes in period.
- Simplified period calculations given in Table IEP-1 come from the commentary of NZS 1170.5:2004

d) Use appropriate part of Figure 3.3 to determine $(\%NBS)_{nom}$.

e) Adjust $(\%NBS)_{nom}$ for appropriate Notes 1, 2 and/or 3.

Note 1: Prior to 1976 additional design loads were specified for public buildings. When it is known that this was allowed for at the time of design this should be included in the assessment of *%NBS*.

Note 2: Concrete buildings designed to NZS 4203 up to 1984 were required to be designed using a structural material factor, M = 1.0. This was amended in NZS 4203:1984 to M = 0.8. Hence the adjustment.

Note 3: Prior to 1935, no earthquake provisions were in place in New Zealand except for Wellington. While it would be possible to discount completely the seismic performance of buildings built prior to 1935 this is clearly too severe. The approach taken in the IEP is to assume that buildings built in Wellington prior to 1935 will perform at least as well as those designed to NZSS 95 as they are likely to have been subjected to some design for earthquake. Elsewhere a 20% penalty has been included to reflect that these buildings would not have been required to be designed for earthquake.

Step 2.2: Determine Near Fault scaling factor (Factor A)

a) Use NZS 1170.5:2004 to determine the *N(T,D)* value applicable for a new building at the site of the existing one under consideration.

Step 2.3: Determine Hazard scaling factor (Factor B)

a) Use NZS 1170.5:2004 to determine the Hazard factor, Z, for the site.
b) For 1992 onwards also determine the Zone factor, Z, for the site from NZS 4203:1992.

Step 2.4: Determine Return Period scaling factor (Factor C)

a) Use NZS 1170.0:2004 to determine the Building Importance Level
b) Read Return Period Scaling factor from Table 3.1

Step 2.5: Determine Ductility scaling factor (Factor D)

a) Assess overall ductility of the building in question
   - Refer Table 3.2 for guidance.

b) Read ductility scaling factor from Table 3.3.

For 1976 onwards the ductility is included in the appropriate part of Figure 3.3.

Prior to 1976 it is necessary to calculate the reduction factor to allow for ductility. The ratio varies with period and soil type.

Step 2.6: Determine Structural Performance scaling factor (Factor E)

Use NZS 1170.5:2004 to determine the Structural Performance factor. Refer Figure 3.4.

Step 2.7: Determine baseline percentage of new building standard for building $(\%NBS)_b$

a) Use values from Steps 1.1 to 1.6 to calculate $(\%NBS)_b$ using the following equation:

$$(\%NBS)_b = (\%NBS)_{nom} * A * B * C * D * E$$

$$(\%NBS)_b = (\%NBS)_{nom} * \frac{1}{N(T,D)} * \left(\frac{1}{Z} \text{ or } \frac{Z_{1992}}{Z}\right) * \frac{R_0}{R} * \left(k_\mu \text{ or } 1\right) * \frac{1}{S_p}$$

Where: $(\%NBS)_b$ is the baseline percentage capacity of the building assuming regular, complying construction.

$(\%NBS)_{nom}$	is the nominal value of *(%NBS)* which assumes *N(T,D)* = 1.0, Z = 1.0, R = 1.0, μ = 1.0, and S_p = 1.0
$N(T,D)$	is the near fault factor from NZS 1170.5:2004.
Z	is the hazard factor from NZS 1170.5:2004.
Z_{1992}	is the zone factor from NZS 4203:1992 (for 1992-2004 buildings only).
R	is the return period factor from the accompanying Table 3.1.
R_0	is the equivalent risk factor for the design vintage.

k_μ is the structural ductility scaling factor from accompanying Table 3.3. Note that m can not be greater than the values given in Table 3.2.

S_p is the structural performance factor applicable to the type of building under consideration. Figure 3.4.

Typical values for $(\%NBS)_b$ for Wellington, Auckland and Christchurch can be found in Appendix 3A

Scaling Factors

The above procedure allows calculation of $(\%NBS)_b$ for a particular type of building provided that its location and original design code are known, and an assessment of the equivalent ductility is made.

The values shown in Figures 3.3(a) to 3.3(h) are based on:

- near fault factor of 1.0
- hazard factor of 1.0
- return period factor of 1.0.
- ductility of 1.0
- structural performance factor of 1.0

The values shown are the ratios of the NZS 1170.5:2004 coefficient on the above basis and the coefficient that comes from the Standard used in design (which depends on date of design).

Refer accompanying Figure 3.6.

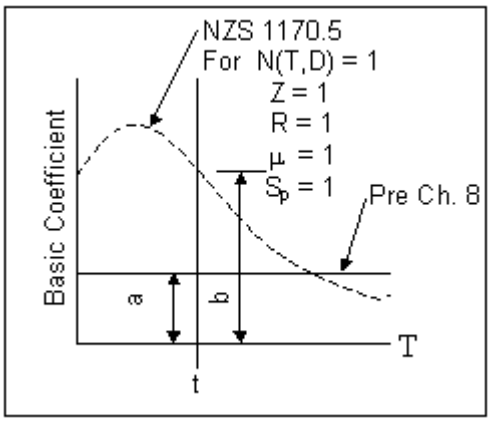


Figure 3.6(a) Pre-NZS 1900: 1965

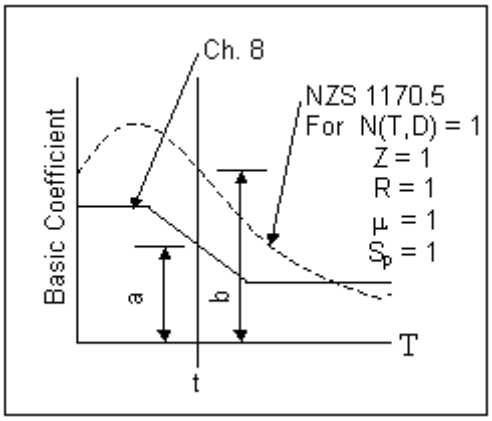


Figure 3.6(b) NZS 1900:Chapter 8: 1965

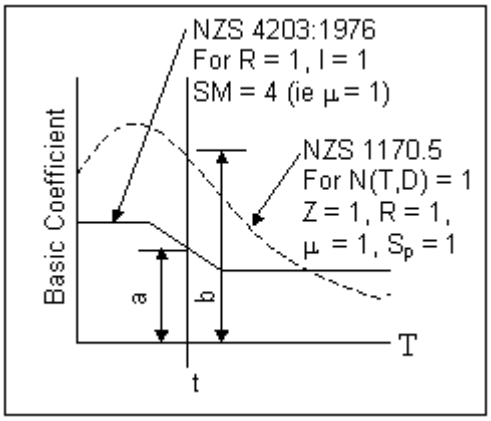


Figure 3.6(c) NZS 4203:1976

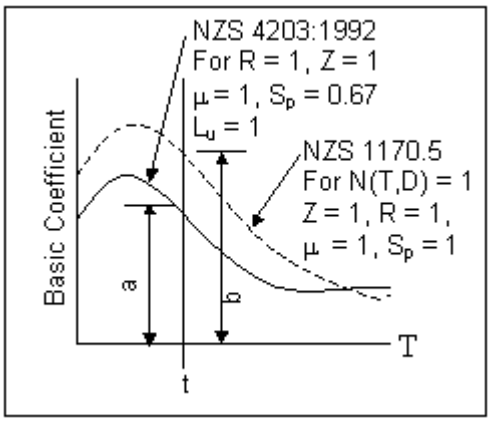


Figure 3.6(d) NZS 4203:1992

Figure 3.6: Concepts behind scaling factors

For a particular T, $(\%NBS)_{nom}$ = a/b

a) to adjust for near fault factor multiply by $A = \dfrac{1}{N(T,D)}$

b) to adjust for hazard factor multiply by $B = \dfrac{1}{Z}$ for pre 1992, or

$B = \dfrac{Z_{1992}}{Z}$ for 1992 onwards

c) to adjust for return period factor multiply by $C = \dfrac{R_0}{R}$

- For pre-Chapter 8 buildings and normal buildings designed to Chapter 8, the adjustment is simply C = 1/R and for public and other buildings designed to Chapter 8, C = 1.25/R.
- For public buildings designed to Chapter 8 1965, the ratio of coefficients used to those for ordinary buildings varied with zone, the ratios (public building/normal building) being 1.33 for Zone A, 1.20 for Zone B and 1.0 for Zone C. Hence the different factors used to determine C

d) to adjust for ductility multiply by $D = k_\mu$ for pre 1976, or

$D = 1$ for 1976 onwards

e) to adjust for structural performance multiply by $E = \dfrac{1}{S_p}$

Assumptions inherent in the method

There are a number of assumptions inherent in the method. These include that;

a) Buildings have been designed and built in accordance with the building standard current at the time, and good practice.

b) The building has been designed for the correct subsoil category. (Make pro-rata adjustments according to NZS 1170.5 spectra, if this is not the case). Note that the rigid subsoil category in NZS 4203:1992 has been split into two categories in NZS 1170.5. The IEP assumes that buildings on subsoil type C (NZS 1170.5) designed to NZS 4203:1992 would have been designed assuming intermediate subsoil. The procedure allows an adjustment if it is known that rigid subsoil was originally assumed.

c) Buildings designed prior to 1965 have had their assessed capacity increased by a factor of 1.5 to convert from allowable stress to ultimate limit state design and divided by 1.4 to convert from a rectangular shear distribution over the height of the building to a triangular distribution with 10% of the base shear applied at the roof. (The basis for this is the ratio of overturning moments derived by the two methods.)

d) Buildings designed to the 1965 code have had their assessed capacity increased by a factor of 1.5 to convert from allowable stress to ultimate strength design methods.

e) Buildings designed to the 1965 code have had their period shifted by a factor of 1.25 to take account of greater assumed flexibility resulting from the allowance for cracking now assumed in modern building design.

f) Buildings designed to the 1976 code are assumed to use the same elastic spectral values as given in the 1984 code. Therefore for a μ of 1 the 1976 values are increased by a factor of 4 (i.e. $SM = 4$).

g) Buildings designed to the 1992 code are assumed to have been designed for an S_p of 0.67. If this is not the case adjust accordingly.

3.4.3 Step 3 - Assessment of performance achievement ratio (PAR)

Assessment of effects of critical structural weaknesses (Steps 3.1 to 3.5)

Note: Consider each orthogonal direction separately unless it is clear from the start which governs.

A critical structural weakness (CSW) shall be deemed to exist if any of the features shown in Table IEP-3 exist. The effect on the structural performance is assessed on the basis of the severity of the CSW in each case.

Definitions of insignificant, significant and severe

Insignificant — The critical structural weakness is not evident or is of such an extent or nature as to have no significant effect on the integrity of the structure or any other element of the building so as to be life threatening *when subject to a design earthquake*.

Significant — The critical structural weakness is evident in part or all of the building and markedly reduces the integrity of the structure or any other element of the building to an extent that partial structural collapse and possible threat to life would result *in a design earthquake*.

Severe — The critical structural weakness is evident in part or all of the building and clearly reduces the integrity of the structure or any other element of the building

to an extent that severe structural collapse and probable threat to life would result *in a design earthquake*.

Compensating provisions (Step 3.6)

It may be that apparent critical structural weaknesses have been compensated for in design. This can be established by viewing building design/construction documentation as part of a simple detailed assessment. Note that even where compensating design has been carried out, a building with discontinuities, such as those nominated as critical structural weaknesses, will still suffer more damage than would a regular geometric/structure building.

Reasons for adopting a compensating factor include:

- more than minimum shear walls
- design for significantly higher gravity loading than current use requires.
- need to compensate for otherwise severe effect of combinations of CSW that are not mutually exclusive.
- any other known factor.

There may be negative factors that are known but have not been included in the IEP assessment e.g. presence of hazardous non-structural items such as URM partitions and fenestration. In such cases it is up to the judgement of the assessor to evaluate the potential life safety risk and adjust the *%NBS* down accordingly. If a reasonable hazard due non-structural items exists it would not be unreasonable to set *%NBS* < 33 with a note that the earthquake prone classification is due to these items.

The maximum value of Factor F has been set at 1.5 (no minimum) unless the building has no more than three storeys in which case a maximum value of 2.5 has been set (also no minimum). The reason for the distinction based on height is that it is felt that there is more scope for judgement for low rise structures where the compensating factors are likely to have a more dramatic effect on earthquake performance.

Factor F is entirely based on the judgement of the assessor and therefore it is a requirement of the IEP that the factors that have led to the decision for Factor F be recorded.

Calculation of performance assessment ratio (PAR) (Step 3.7)

This is simply the product of the factors identified and shown on Table IEP-3. The focus of the review is on the capacity to resist lateral load.

3.4.4 Step 4 - Determination of percentage of new building standard *(%NBS)*

Refer to Table IEP-3. This is a simple calculation:

$$\%NBS = (\%NBS)_b \times PAR$$

What a good building of its type would be

The overall effect of CSWs

Table 3.4: Guide to severity of critical structural weaknesses

	Effect on structural performance		
Critical structural weakness	**Severe**	**Significant**	**Insignificant**
Plan irregularity			
L-shape, T-shape, E-shape	Two or more wings length/ width > 3.0, or one wing length/width >4	One wing length/width > 3.0	All wings length/width ≤ 3.0
Long narrow building where spacing of lateral load resisting elements is …	> 4 times bldg. Width	> 2 times bldg. Width	≤ 4.0 times bldg width
Torsion (Corner Building)	Mass/centre of rigidity offset > 0.5 width	Mass/centre of rigidity offset > 0.3 width	Mass/centre of rigidity offset < 0.2 width or effective torsional resistance available from elements orientated perpendicularly.
``` ```Ramps, stairs, walls, stiff partitions	Clearly grouped, clearly an influence	Apparent collective influence	No or slight influence
Vertical irregularity			
Soft storey	Lateral stiffness varies > 150%	Lateral stiffness varies 100–150%	Lateral stiffness varies < 100%
Mass variation (geometrical)	Mass varies > 150% between adjacent floors	Mass varies 100–150% between adjacent floors	Mass varies < 100% between adjacent floors
Vertical discontinuity	Any element contributing > 0.5 stiffness of the lateral force resisting system discontinues vertically	Any element contributing > 0.3 stiffness of the lateral force resisting system discontinues vertically	Elements contributing to the lateral force resisting systems are continuous vertically
Short columns			
Columns < 70% storey height between floors clear of confining infill, beams or spandrels	Either > 80% short columns in any one side Or > 80% short columns in any storey	> 60% short columns in adjacent sides > 60% columns in a storey are short	No, or only isolated, short columns
Pounding effect			
Floor aligns ≤ 20% storey height	0 < separation < 0.005 *H*	0.005 *H* < separation < 0.01 *H*	Separation > 0.01 *H*
Floor aligns > 20% storey height	0 < separation < 0.005 *H*	0.005 *H* < separation < 0.01 *H*	Separation > 0.01 *H*
	where *H* = height to the level of the floor being considered		
Height difference effect			
No adjacent building, or height difference < 2 storeys	0 < separation < 0.005 *H*	0.005 *H* < separation < 0.01 *H*	Separation > 0.01 *H*
Height difference 2–4 storeys	0 < separation < 0.005 *H*	0.005 *H* < separation < 0.01 *H*	Separation > 0.01 *H*
Height difference > 4 storeys	0 < separation < 0.005 *H*	0.005 *H* < separation < 0.01 *H*	Separation > 0.01 *H*
	where *H* = height of the lower building and separation is measured at *H*		
Site characteristics	Unstable site Extensive landslide from above Probable liquefaction	Potential for site instability Landslide from above Liquefaction potential	Not a significant threat

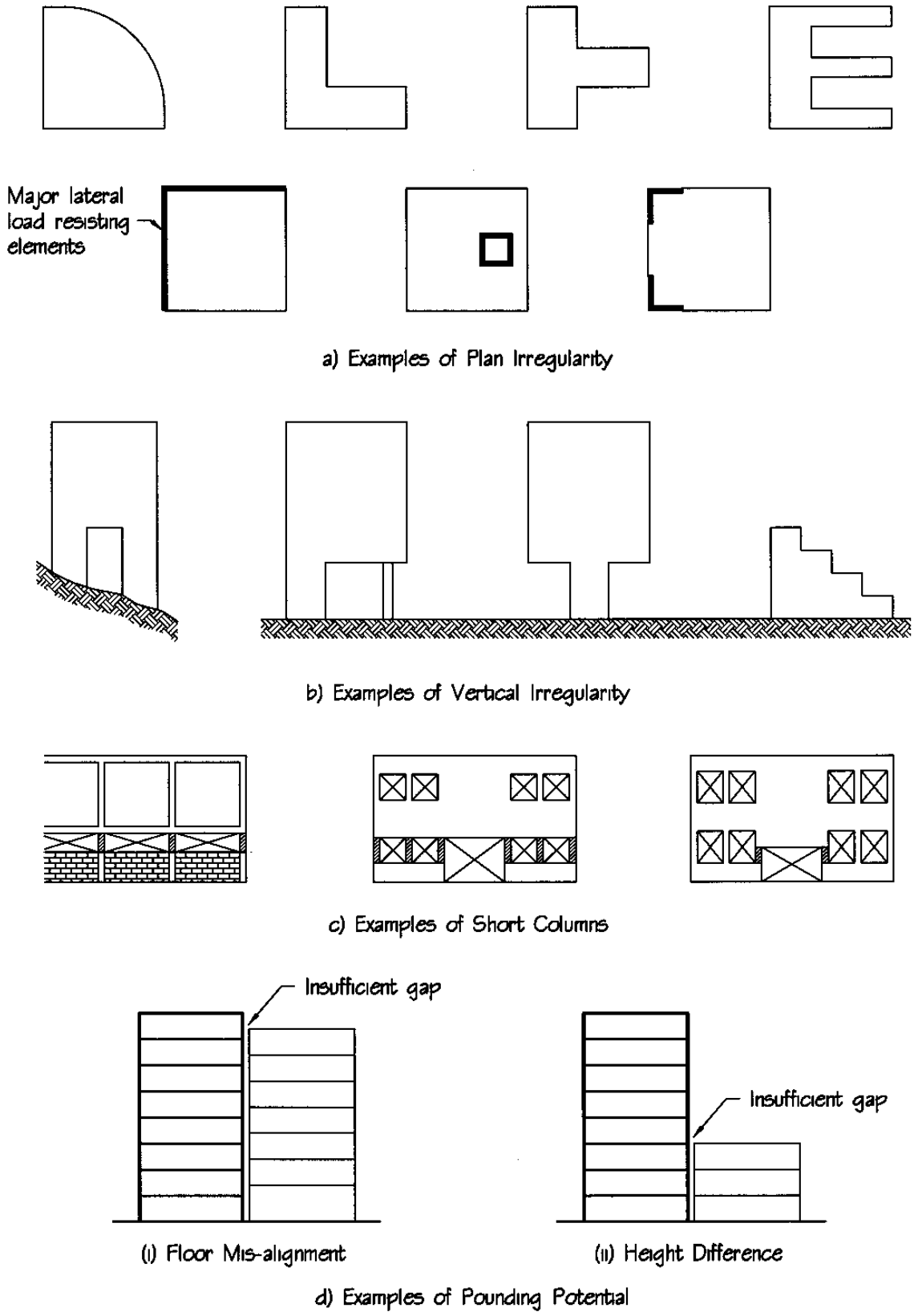


Figure 3.7: Examples of critical structural weaknesses

3.4.5 Step 5 - Building Earthquake Prone?

%NBS greater than 33 → NO – Building does not require further action in terms of the Building Act.

%NBS less than or equal to 33 → YES – Building is potentially earthquake prone in terms of the Building Act. Further action required, e.g. detailed assessment.

3.4.6 Step 6 - Building an earthquake risk?

%NBS greater than or equal to 67 → NO – Building is unlikely to be an earthquake risk.

%NBS less than or equal to 33 → YES – Building is potentially an earthquake risk. Further action recommended, e.g. detailed assessment.

3.4.7 Step 7 - Seismic grading

The grading scheme shown in Table 2.1 (Section 2.8) is being promoted by the New Zealand Society for Earthquake Engineering to improve public awareness of earthquake risk and the relative risk between buildings.

It is not a requirement of the Building Act to provide a seismic grade but it is strongly recommended that this be recorded so as to promote the concept of seismic grading

Seismic grading determined from the results of the IEP should be considered provisional and subject to confirmation by detailed assessment.

Section 4 - Detailed Assessment – General Issues

4.1 Introduction

4.1.1 Context and Background

The initial evaluation procedures described in Section 3 provide an approximate assessment of the likely performance of a building in earthquake. Whether these are applied by a TA or the owner of the building, the approximate nature of the assessment will undoubtedly give rise to concerns regarding the credibility of the result. The detailed procedures for the assessment of structural performance given in this Section 4 are intended to provide a means of more accurate assessment of performance. They allow the engineer to look in more detail at the characteristics of the building, its response to earthquake shaking, the demands it places on structural elements, and the capacity of such elements to meet those demands by maintaining structural integrity under imposed actions and displacements. 2

The focus is the determination of demand on structural elements, resulting from the response of the building, and assessment of the capacity of such elements to meet the demand without causing loss of structural integrity.

The assessment of structural performance is the heart of the process to determine the level of risk represented by a building, and in particular to determine whether or not it meets the requirement of the Act for it to perform satisfactorily in a moderate earthquake (defined as being one third the level of ground shaking at the site as would be required for the design of a new building – i.e. 33% of new building standard (33*%NBS*).

The detailed procedures in this section are intended for implementation following application of the initial evaluation process (IEP) described in Section 3. However, it is not a pre-requisite that the IEP be used. These detailed procedures are compiled to be independent and self-sufficient.

Dealing with existing buildings involves a wide range of structural types, materials and details. No attempt has been made to cover every possible situation. The procedures focus on the most common situations and elements. This should allow an experienced earthquake engineer to adapt and extend the procedures to best match any particular situation.

Situations will vary from small simple buildings to large complex ones. The approach to determine demand and capacity will be up to the engineer responsible for the assessment. This is intended to help the engineer to adopt the simplest available approach consistent with the circumstances.

4.1.2 Objectives for Assessing Existing Buildings

The objectives of these detailed procedures are:

- to provide a means to assess the level at which ULS is reached for existing buildings when subject to earthquake shaking.
- to determine whether or not a building will reach or exceed its ultimate limit state when subjected to earthquake shaking one-third as strong as that required for the design of a new building at the site.
- to provide information and guidance to assist in the assessment of strength and ductility of structural, components, elements and systems.

4.2 Performance Objectives

4.2.1 Hierarchy of Performance Measures

The following performance objectives, performance requirements, performance criteria and verification methods define the hierarchy of performance measures for various building groups. New buildings have been included to provide a frame of reference.

Note that the distinction between high, moderate and low risk buildings is the initiative of NZSEE and is not part of the Building Act. The opportunity has been taken to match the NZSEE definitions of High Risk Buildings with the earthquake prone requirements of the Act.

a) Performance objectives

i) New buildings

People in or adjacent to the building are not exposed to unreasonable risk by the building during a reasonably foreseeable event. (A reasonably foreseeable event is the design earthquake for a similar new building.)

ii) Existing buildings – low risk

People in or adjacent to the building are not exposed to unreasonable risk by the building during a reasonably foreseeable event. (A reasonably foreseeable event is the design earthquake for a similar new building. For existing buildings a higher probability of danger than for a new building is accepted as reasonable.)

b) Performance criteria

i) New buildings: The building shall be shown to attain its ultimate limit state (ULS) when subject to no less than 100% of the design earthquake shaking at the site.

ii) Existing buildings – low risk: The building shall be shown to attain ULS when subject to no less than 67% of the design earthquake shaking at the site.

iii) Existing buildings – moderate risk: The building shall be shown to attain ULS when subject to no less than 33% of design earthquake shaking at the site.

iv) Existing buildings – high risk: A high risk existing building is one which attains ULS when subject to less than 33% of the design earthquake shaking at the site.

Alternatively, a building is not a high risk building if it can be shown by rational and accepted means that it can attain ULS at no less than 33% of the design earthquake shaking at the site.

The lower part of Figure 4.1 gives an indication of how risk increases with reduction in structural performance, expressed as a percentage of the standard required for new buildings. This highlights the fact that the proposed legislation targets only those of high risk. It also supports the NZSEE recommended minimum level for strengthening (improving structural performance) of 67% that for a new building.

c) Verification methods

For all buildings verification methods are

- the requirements of NZS 1170.5:2004 and the relevant and compatible materials standards using full, two-thirds or one-third of the design earthquake loading as applicable.

- other methods, the fulfilment of which can be shown, by rational and accepted means to provide a performance equal to or better than that required to meet the performance objectives and criteria.

For existing buildings:

- adjustment to material properties normally used for new buildings may be permitted to recognise known or measured values, or a greater or lesser confidence than normal in the attainment of strength and similar parameters.

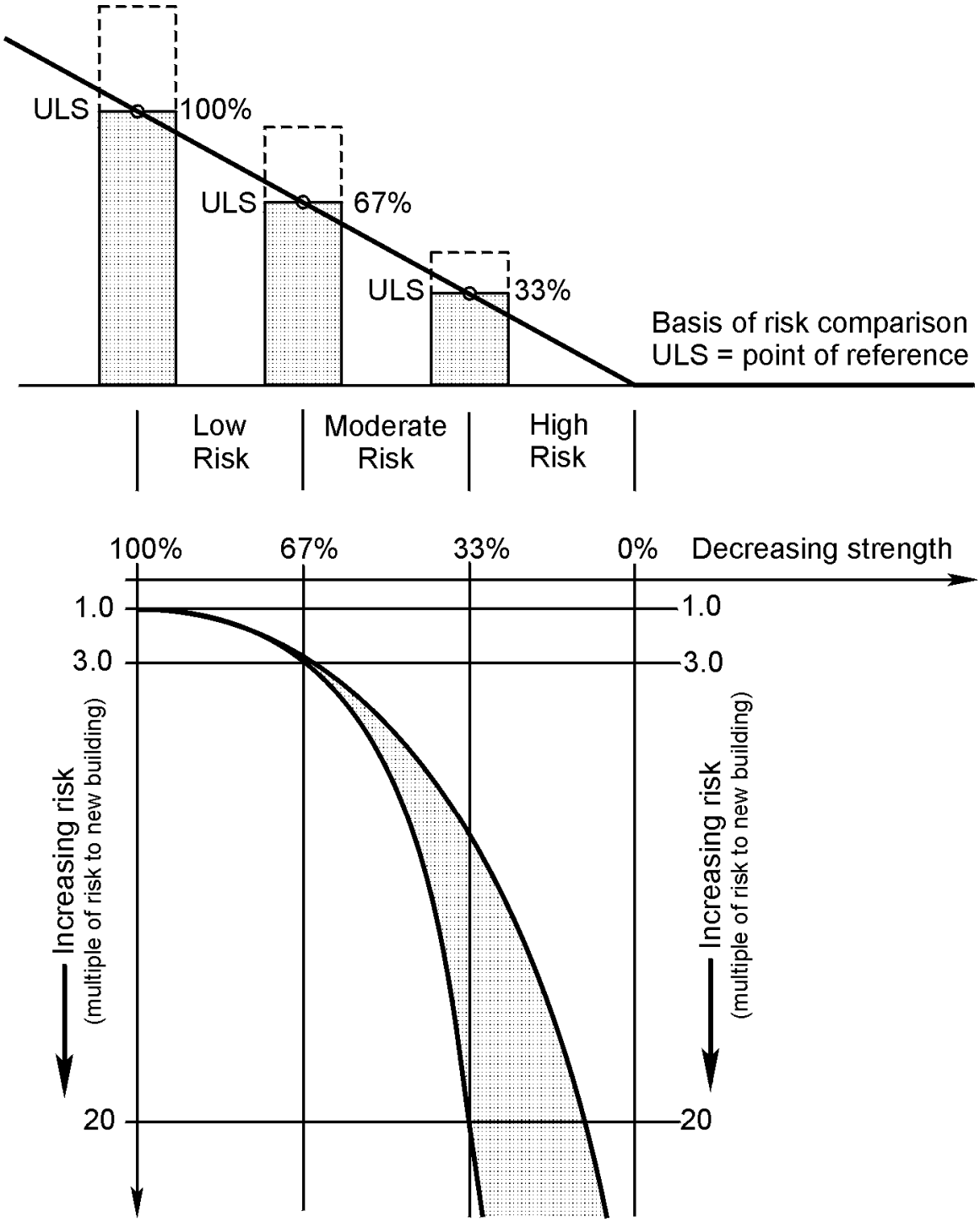


Figure 4.1: Strength versus risk and ULS as reference point

Note that the definitions implicitly define another group of buildings which could be termed existing buildings – moderate risk. This group of buildings would meet performance measures at between 33% and 67% of design earthquake shaking. They would not be classified as EPBs in terms of the Act, but would fall short of the NZSEE recommended minimum performance levels for existing buildings.

These definitions provide the framework for the development of detailed performance requirements. Table 4.1 summarises the hierarchy of performance measures.

Table 4.1: Hierarchy of performance measures

Building group	Performance objectives	Performance requirements	Performance criteria	Verification methods
New buildings	People not unreasonably endangered.	Would lose integrity at 100% or more of new building standard.	ULS attained at not less than 100*%NBS*.	1 NZS 1170.5 (or NZS 4203) and related material codes. 2 Other acceptable.
Existing buildings – low risk	People not unreasonably endangered – higher risk accepted versus new buildings.	Would lose integrity at 67% or more of new building standard.	ULS attained at not less than 67*%NBS*	1 NZS 1170.5 (or NZS 4203) and related material codes. 2 Other acceptable.
Existing buildings – moderate risk	No performance objective. Not HRBs, but less than NZSEE minimum recommended performance.	Would lose integrity at between 33% and 67% of new building standard.	ULS attained at between 33% and 67*%NBS*	1 NZS 1170.5 (or NZS 4203) and related material codes. 2 Other acceptable.
Existing buildings – high risk	These are HRBs by definition. Objective is to make these low risk.	Would lose integrity at 33% or less of new building standard.	ULS attained OR injury likely at ≤33*%NBS*	1 NZS 1170.5 (or NZS 4203) and related material codes. 2 Other acceptable.

Definition (a), Performance Objectives, states the overall objective of seismic design. It is what we would like to achieve in overall performance for all buildings in terms of effect on people. A concession is made for existing buildings in the form of higher acceptable probability of danger. Note that existing buildings – high risk are not covered. The performance objective for these is to improve their performance to make them low risk, i.e. not just medium risk unless attainment of this is clearly impractical.

Definition (b), Performance Criteria, recognises that attainment of ULS is a measure of the acceptability of structural performance. It is recognised that collapse may occur at or above the level at which ULS is attained. However, these definitions of performance criteria in terms of ULS allow use of parameters commonly used in the design of new buildings.

These relationships are illustrated in the upper part of Figure 4.1.

Note that the ultimate limit state (ULS) reference line provides a consistent basis for determining when the threat to life is/is not acceptable, the reason being that ULS is implicitly the reference point for new buildings (refer4.2.3 for further detail).

For existing buildings – high risk, definition (b) converts the general intention of the words in the Act to definitive criteria that are recognised as achieving the same objective as the words in the Act.

Definition (c), Verification Methods, states that compliance with NZS 1170.5:2004 and its companion materials codes (where applicable) under reduced loads will be accepted as a means of complying with definition (b). Definition (c) also provides a reminder that other methods are

admissible as long as they can be demonstrated to achieve equivalent or better performance standards.

4.2.2 Application

Application of NZS 1170.5:2004 and its companion material standards is common for new buildings. The situation is not as clear-cut for existing buildings. Details and materials used may no longer be covered by current codes. Thus, special attention in these procedures is given to providing information to assist in determining both the demand on and capacity of existing building elements.

The general intention is to retain the approaches which apply to new buildings as far as possible, but to provide detailed procedures and additional information to cover those situations for which new building requirements do not provide adequate guidance.

In particular, this is required for forms of construction that are no longer used in new buildings. Instances of this are unreinforced masonry structures and beam-column joints with sub-standard detailing.

4.2.3 ULS as Measure of Acceptable Performance

In the above, the ultimate limit state (ULS) has been deliberately used to define the boundary between what is acceptable performance and unacceptable performance. There may be more sophisticated criteria to test for acceptable performance in terms of risk to life, but adoption of ULS has the advantage of familiarity and simplicity.

Background

AS/NZS 1170.0:2002 and NZS 1170.5 provide a clear basis of a definition of ultimate limit state for new buildings. These indicate quite clearly that the definition is made up of:

a) a limit on strains in elements

b) a requirement for stability (under factored loads)

c) limits on displacement

This is in line with the Building Act/Regulations/Code which require that structures "have a low probability of rupture, becoming unstable, losing equilibrium or collapsing", but provides more detail. The Loadings Standard AS/NZS 1170, defines ULS in terms of limiting displacement or strains to avoid instability and to maintain structural integrity.

The focus on strains, stability and displacement in the definition of ULS leads to greater emphasis on the displacement of a structure during earthquake and the effects these displacements may have. This is seen as particularly helpful when assessing existing buildings and unfamiliar materials.

Application to existing buildings

Applying the test of attainment or not of ULS in an existing building should follow as far as possible the requirements for new buildings. In this regard, the current loadings standard AS/NZS 1170 provides the following:

a) ***Stability*** –Load factors are defined

For force-based analysis, these provide a direct means of checking stability. In displacement based analysis, the determination of stabilizing influences will be less direct, and will usually require the evaluation of the effects of the computed displacements.

b) ***Displacement*** – Limits are prescribed.
These limits, or others imposed for existing buildings, apply to both displacement-based and force-based approaches. In both cases, displacements of the overall structure may also be limited by strains imposed on its elements, or by stability requirements.

c) ***Strain*** – Material codes define or imply limits for common types of members and elements.
For new buildings acceptable performance results from applying design requirements. For existing buildings the challenge is to define practical limits, especially for materials and/or detailing that are no longer used. These Guidelines are intended to provide some assistance in this respect.

4.3 Approaches for Performance Assessment

4.3.1 General

The objective of the structural performance assessment is either:

- to determine the level of performance in relation to current code, or
- to establish whether or not the building meets the one-third threshold.

The approaches chosen to perform the assessment will vary considerably according to circumstances. Many buildings will not require or justify the use of lengthy and detailed analysis. The underlying objective of the legislation is to ensure that physical action is taken to improve the earthquake performance of existing buildings.

Figure 4.2 provides a simple summary of the main approaches available. It also serves as a reminder that whatever analysis and assessment techniques are used, all involve assumptions about the earthquake shaking, the building characteristics, analysis methods, and the likely performance of structural components. The fact that the underlying objective is to enable effective mitigation to be physically done should be borne in mind when choosing an approach to match the circumstances of any particular building.

The "cartoons" in Figure 4.2 indicate the sequence of steps in the assessment procedure.

This graphical summary serves to illustrate that the force-based and displacement-based approaches are different ways of looking at the same issue. In the force-based approach, the performance of components is analysed by examining the forces in critical elements and using rules to assess the limits of integrity of the structural members. In the displacement-based approach, the response of the building structure is considered from the outset on the basis of the displacements by the ground shaking. These are then used to examine the effect of critical structural elements, again using rules to measure the limits of integrity and performance.

The following comments should be noted in relation to Figure 4.2:

a) *Modelling of the earthquake shaking*. This will vary according to the analysis techniques used.

b) *Modelling of the structure*. Numerous assumptions are necessary as to member properties and boundary conditions, and the way these change as a result of earthquake response.

c) *Choice of analysis process or programme to be used*. This will determine the nature and details of response derived, i.e. displacement, shear, moment, axial force..

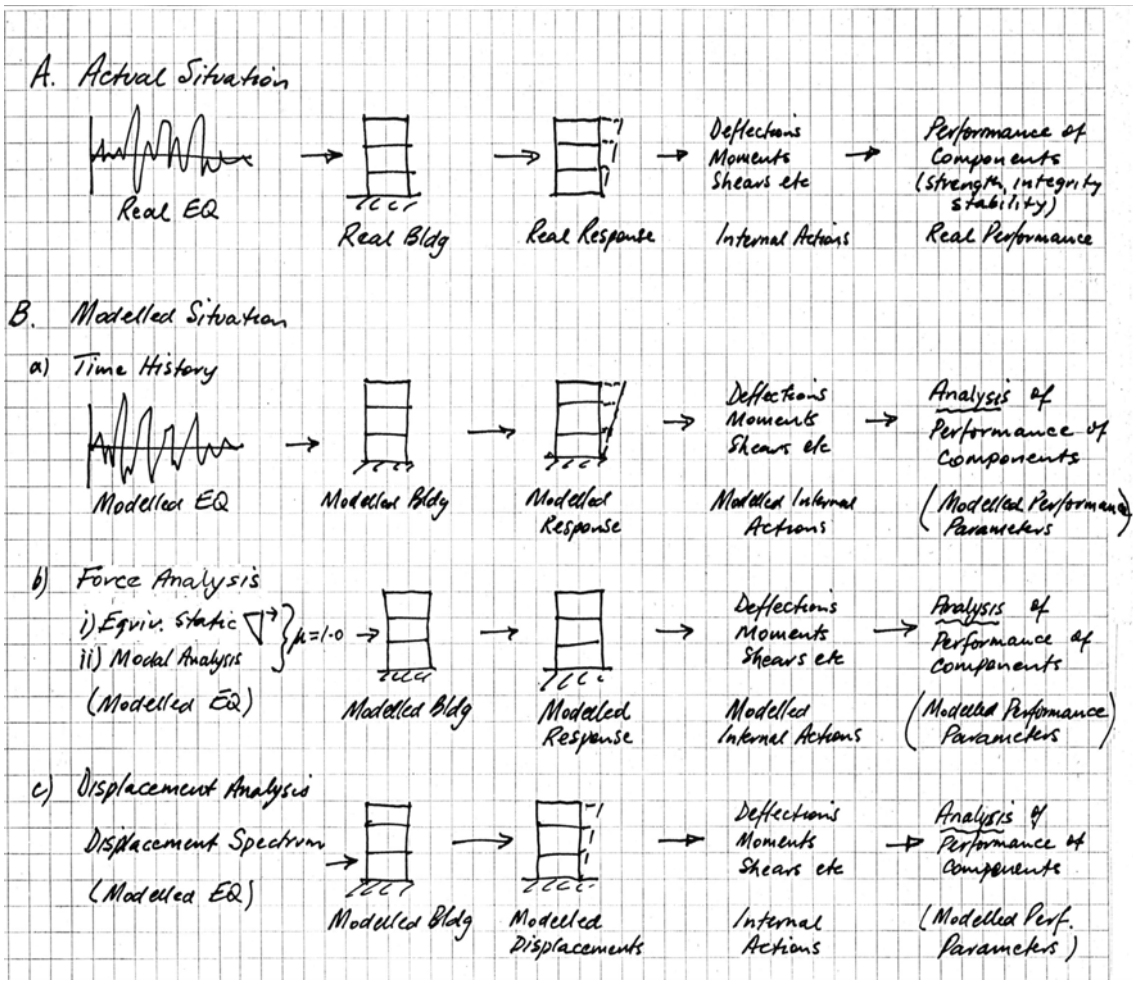


Figure 4.2: Real and modelled responses of buildings to earthquake

d) *Modelling of the capacity of structural elements*. This process is significantly different from that used in the design of new buildings. For new buildings there are prescribed details (e.g. stirrup spacings) which will achieve the ductility assumed. For existing buildings, the ability of elements to deform plastically will depend on the detailing of them. A first-principles approach to assess member ductility is likely to be required.

e) *Comparison of demand and capacity*. On this will rest the result of the assessment.

The way in which the earthquake is modelled will depend on the analysis method. For example, the NZS 1170.5 spectrum may be used for equivalent static or modal analysis, but suitable base earthquake records would be needed for time history analysis. The same applies to the modelling of the building structure and deriving the response.

Hence the presentation of material for Steps a), b) and c) has been split according to the analysis method in order to assist in keeping each consistent with the other. Similarly, modelling of capacity, Step d), is split according to material and structural element type.

In all the approaches the assessment of the building response must be obtained. Regardless of analysis method, this focuses on determining the displacement of the structure. Internal actions generated such as shear, moment and axial load should be considered as consequences of this displacement, not the cause of it. This is the essence of the "displacement based" approach.

Of the approaches available, an appropriate one must be chosen. The basis for this will be, as always, to achieve a credible and practical result.

The extent to which the structure is modelled and the lengths to which other analysis needs to be done requires careful thought. An intuitive overview of the structure will help to identify critical structural weaknesses and/or particularly vulnerable elements.

4.3.2 Global Analysis Considerations

a) Critical structural weaknesses

The main reason for introducing legislation to cover all buildings was the knowledge that those with critical structural weaknesses (CSWs) have a tragic record in major earthquakes. On the other hand, reasonably regular buildings without CSWs have performed satisfactorily even though member detailing may not have been good by current standards.

The likely effects of any CSWs in the building must be fully assessed. These include:

- horizontal irregularity
- vertical irregularity
- short columns
- diaphragm discontinuities
- lack of separation between structural and non-structural elements
- pounding (potential for building-to-building impact).

2 For illustrations of these characteristics refer to Figure 3.7 in Section 3 of these Guidelines. Further information on pounding is given in Appendix 4D.

Irrespective of whether a simple or complex analysis of a structure is about to be undertaken, it is vital to consider how the building as a whole will respond. Very few pre-1970s buildings are either pure wall structures or pure frame structures. Even for those buildings without concrete structural walls, the extensive use of unseparated masonry infill panels means that they cannot be regarded as just a frame structure. Careful consideration of the initial response of such structures, where high initial stiffness may give rise to displacements, strains and internal forces that are appreciably greater than for a structure modelled as a frame. In such situations, upper and lower bound approaches using different combinations of element stiffness should be used to determine the likely worst possible effect on the structure.

Before categorising elements as either secondary or non-structural, careful consideration must be given to their ability to deform with the primary structural elements, and their possible participation in generating unexpected forces/restraints within the structural elements. The displacement capability of designated non or non primary structural elements may well limit the performance of the building as a whole.

Existing load paths must be identified, considering the effects of any past modifications, additions or alterations. Potential discontinuities and weak links should be identified at both the global structural level (e.g. diaphragms) and individual element level (e.g. inadequate anchorage). Components that are essential to the vertical load carrying integrity of the building must also be identified.

Appropriate consideration should be given to foundation elements and the effect of ground conditions. While details of foundation systems will not always be available, the key question to be considered is the extent to which the foundations are capable of developing the strength of the superstructure.

It may be clear from a simple analysis that the building will not sustain the minimum requirement of one-third current code. Further analysis by way of assessment of structural performance would be futile. The effort would be better spent on devising measures to remove or significantly reduce the effects of the CSWs.

b) Global structural analysis procedures (GSAPs)

Different levels of GSAPs may be used for structural assessment, depending on the importance of the structure and the analysis methods available and familiar to the structural engineer. It should be noted that more sophisticated GSAPs do not of themselves ensure improved accuracy of assessment. For example, it may be more difficult to model degrading strength characteristics in an inelastic pushover analysis or a time-history analysis than in a simple displacement-based ultimate lateral mechanism analysis. Brief notes on the strengths and weaknesses of different methods follow. A summary of recommendations on the application and limitation of each method is given in Table 4.2 below. Analysis procedures are discussed further in Appendix 4E.

Elastic methods

Linear elastic methods including modal analysis procedures may be used for the analysis of existing buildings to determine the distribution of member actions due to lateral seismic forces and gravity loading. Standard procedures include both the equivalent static and modal response analysis methods. The advantage of these procedures is that designers are familiar and comfortable with them, and the simplified analysis methods may often allow for design input to be minimised. The disadvantage is that there are difficulties in applying standard provisions applicable to new construction to the complexities of retrofitting buildings.

The results of linear procedures can be very inaccurate when applied to buildings with highly irregular structural systems unless the structure is capable of responding to the design level seismic forces elastically or with a low level of ductility demand.

Table 4.2: Summary of recommended analysis procedures and applicability guidelines

Analysis method	Applicability notes
1 Elastic methods	
1.1 Equivalent static analysis (ESA)	Building height not exceeding 30 m. and No significant vertical stiffness or mass irregularly present, and No significant torsional stiffness irregularity present, and Orthogonal lateral force resisting systems present. *Either*: Elastic responding under 'design level' earthquake, *or* Low ductility demand/capacity (μ < 2.0) under design level earthquake where: no in plan or out of plane discontinuities present in primary lateral force resisting system no significant weak storey irregularity present no significant torsional strength irregularity present in any storey.
1.2 Modal response spectrum analysis (EMA)	Either: Elastic responding under design level earthquake, *Or*

Analysis method		Applicability notes
		Expected low ductility demand/capacity (μ < 2.0) under design level earthquake where: no in plan or out of plan discontinuities present in primary lateral force resisting system no significant weak storey irregularity present no significant torsional strength irregularity present in any storey.
2	**Inelastic methods**	
2.1	Simple lateral mechanism analysis (SlaMA)	No significant torsional stiffness irregularity. Higher mode effects not critical.
2.2	Lateral pushover analysis (LPA)	Higher mode effects not critical.
2.3	Inelastic time history analysis (ITHA)	May be used for any structure, but may not be appropriate for some structures, eg wooden framed structures.

Note: Higher mode effects can be assumed critical in following structures:

- Building fundamental period exceeds approximately one second.
- Shear in any one storey, calculated from a modal analysis considering sufficient modes to achieve at least 90% mass participation, exceeds 130% of the corresponding storey shear resulting from a second analysis considering only the first mode participation.
- Higher modes are not important if 75% or more of mass participates in the first mode in a particular direction.

i) Equivalent static method

Under the equivalent static method, design seismic forces, their distribution over the building height and the corresponding internal forces and building displacements are determined using a linear elastic static analysis.

In the analysis the equivalent static forces are increased from zero until the first plastic hinge forms. The lateral seismic forces corresponding to the development of the first plastic hinge will give a lower bound for the probable lateral force capacity of the structure. In reality however, some inelastic action can be tolerated by components and elements, permitting higher lateral seismic forces to be resisted while further plastic hinges form until a mechanism develops or local failure point is reached. Some limited account may be taken of the post-elastic deformation capacity of the structure, to allow use of a system structural (displacement) ductility factor of $\mu > 1.0$. Assessment of this factor should be based on the ductility capability of the weaker link components in the structure, but should not be taken greater than $\mu = 2$ using an elastic analysis approach.

ii) Modal response spectrum analysis

Elastic modal analysis (EMA) may be used to enable consideration of higher mode effects through superposition rules such as SRSS and CQC methods. This procedure is appropriate for use with structures that are expected to respond elastically to seismic action. EMAs are carried out using linearly elastic response spectra with the resulting forces generally scaled to match the lateral force used in the equivalent static procedure and the components evaluated in the elastic range of strength and serviceability. The post elastic deformation capacity of the structure is addressed in the same way as for the equivalent static method.

The use of EMA as a non-linear procedure to account for anticipated non-linear response is generally inappropriate for assessment of existing structures for the following reasons:

a) There is no simple way of assessing the expected inelastic deformations from an EMA. Common methods, which tend to assume that structure and member ductility levels are identical, are incorrect.

b) EMA's underestimate the force levels associated with higher mode response when member force levels are scaled back to inelastic mechanism strength. Conversely, EMAs overestimate torsional response levels for most buildings that respond inelastically.

c) EMA's cannot consider the influence of seismic axial force variations in columns on the stiffness of columns. This can result in inaccurate estimates of when inelastic action develops in reinforced concrete frame members. Influence of seismic force on member stiffness can directly be included in both SLaMAs and LPAs.

Thus, apart from structural steel and timber structures, and concrete structures that are expected to respond elastically to seismic action, EMA's should not be used to assess existing structure, unless special modifications are made to allow consideration of the above issues.

Inelastic methods

Inelastic methods of analysis include:

i) Simple lateral mechanism analysis (SLaMA)

Either force-based or displacement-based approaches may be used. In both cases a hand analysis is carried out to determine the probable collapse mechanism and its lateral strength and displacement capacity, with simplified consideration of capacity issues (higher mode effects, relative strengths of flexure and shear, etc). Behaviour is considered to be that of an equivalent single-degree-of freedom system. Simplified approaches are used to determine initial stiffness of the structure, and the effective yield displacement. Inelastic mechanisms are developed considering the possibilities of mixed hinging modes including both flexural and shear hinges, where appropriate.

Force-based methods determine expected displacement demand using the elastic stiffness, and assessed structure displacement ductility capacity, and an inelastic acceleration spectra set.

Displacement-based methods use the same methods as for force-based assessment to determine the force-displacement response of the structure. However, expected displacement demand is based on the structure characteristics (effective stiffness and equivalent viscous damping) at maximum displacement capacity rather than initial elastic characteristics. A displacement spectra-set for different levels of elastic damping is used rather than the acceleration spectra set of force-based design. The displacement-based approach enables degrading strength response, and the influence of poor hysteretic response characteristics to be simply incorporated in the analysis.

The main weakness of SLaMAs is that the sequence of development of inelastic action between different members of the structure will not be identified. For structures with low member ductility capacity, there will be a tendency to overestimate structural displacement capacity.

Key elements of SLaMA are presented in Appendix 4E.10.

ii) Lateral pushover analyses (LPAs)

This category of GSAPs is essentially a refinement of the SLaMA approach. An incremental inelastic lateral analysis of the structure is carried out under a lateral vector of floor forces the magnitude of which is gradually increased. The onset of inelastic action of each member can thus be identified, and the inelastic deformation of critical members can be directly tracked, thus identifying the structure "ultimate" capacity more accurately. As with the SLaMAs, LPAs result in a simplified force-displacement response, which can be used with a force-based method to determine displacement demand.

Most LPA programmes cannot deal with negative structural stiffness (falling branch behaviour) since the pushover is carried out with incremental force vectors in each analysis step, so analysis

"blows up" just before lateral strength is achieved, and it can thus be difficult to determine structural displacement capacity.

The choice of the shape of the lateral force vector will affect the results, including, possibly, the location and type of inelastic action. Most engineers are familiar with the inverted triangle distribution of floor forces, but a structure developing a soft-storey sway mechanism should have a force vector essentially uniform with height.

It is difficult to incorporate higher mode effect into LPAs, so in most cases it is still essentially a single mode approach, and collapse mechanisms associated with higher modes may be missed. Current research is being done to improve this. (References needed)

Capacity and relative strength issues may require multiple analyses.

For more information refer to FEMA Document 356.

iii) Inelastic time-history analysis (ITHA)

ITHAs in principle offer the most realistic GSAP representation. They offer the ability to track onset of inelastic response of the LPA methods, while at the same time including higher mode effects in a realistic way. As structural engineers become increasingly familiar with ITHA, and software becomes more readily available, it is expected to become the GSAP of choice for structural assessment, particularly for more important structures. Even so, it requires considerable judgement.

Special care and skill is required to select appropriate modelling approximations. For example, the definition of elastic damping needs careful consideration, as inappropriate definition commonly results in an overestimate of response.

Typically the interactions between flexure, shear and axial load are typically not modelled in ITHA programs, making it impossible to model the onset of shear failure. Similarly, few ITHAs include the influence of axial force in columns on their stiffness. This can influence predictions of onset of inelastic response, and can be critical for structures with brittle failure modes.

Some ITHA programmes cannot model degrading strength characteristics, and few have special elements representing the strength and degradation characteristics of beam-column joints in concrete or steel structures.

The refinements of an ITHA may be inappropriate when the uncertainty associated with the seismic intensity is considered. When ITHA are carried out, it is usuallly necessary to run several analyses with different records representing the design intensity to ensure that variations between different records do not cause a change in the inelastic mechanisms developed. When it is required to determine the actual level of intensity corresponding to a given limit state, rather than assessing a pass/fail result for a reference intensity, it will be necessary to perform multiple analyses, scaling the intensity of the records until the limit state is reached.,

As a consequence of these considerations, ITHA should not be the sole GSAP for a structural assessment, but should be supported by the results of a simplified approach.

4.3.3 Approach to Capacity and Demand

It is most important to recognise that the determination of member capacity, overall structural capacity and demand are not entirely separable. There may be considerable interaction. An obvious example is the need to know the strength of beam and column cross-sections before carrying out an inelastic time-history analysis or push over analysis. Another example is the need

to correctly assess stiffness of members and the structure when doing modal analysis. Initial assumptions of member properties/capacities will have a bearing on the calculation of structural displacement. This in turn will affect the calculated demand on structural elements.

In the face of this, engineers making assessments will need to carefully assess the implications before choosing the most appropriate method of analysis. Considerable judgement will be needed to achieve a credible assessment to match the circumstances and available budget. For example, it may be possible to quickly identify which members/frames/walls are critical and restrict the analysis to those elements.

Without such judgement in the initial stages, there is a danger that the assessment could become unwieldy, uneconomic and ineffective. Those carrying out the assessments need to constantly remind themselves that the objective of the legislation is to reduce seismic risk. It may be better that a fairly crude but effective strengthening measure be carried out than for strengthening work to be postponed while the owner saves up to pay for an unnecessarily expensive analysis.

Assessment of structural performance can be broken in to three stages or aspects, assuming that the reference level of earthquake shaking has been determined:

1) **Member capacity:** Determination of member strength, stiffness and deformation capacity, e.g. flexural and shear strength, elastic and post-"yield" stiffness (this may be negative stiffness for inelastic shear behaviour), plastic rotation or shear deformation capacity. Note that member capacity must be determined before global analysis can be carried out.

2) **Structure capacity:** Incorporation of member stiffness and strength in a global structural model (GSAP). The GSAP is used to determine
   - expected displacements and plastic deformations
   - relative member actions
   - expected inelastic mechanisms.

   The GSAP enables the global capacity to be determined from the member capacity.

3) **Demand versus capacity:** Determination of the earthquake shaking level corresponding to the capacity of the structure. This determination may be made directly within the global analysis, or by direct comparison of global demand with the reference level of earthquake shaking.

4.4 Building Inspection and Investigation

4.4.1 Introduction

Detailed building inspections should be made as part of the assessment of existing performance and before the preparation of strengthening proposals.

Conventional structural analyses are based on the assumption that the roof and floor diaphragms are relatively rigid and that the weight of tributary areas on each level, including the diaphragm, can be lumped to act at points on relatively flexible shear walls. That is, the diaphragms are assumed to distribute the loads to walls parallel to the direction of lateral loading without significant out-of-plane loading of the walls perpendicular to the direction of loading. However, many unreinforced masonry buildings have flexible diaphragms (often constructed of timber) and very rigid shear walls thus invalidating the conventional assumptions.

It is important that the diaphragm flexibility and the out-of plane loading of the walls be correctly included in the analysis model. It is therefore necessary in a detailed inspection to identify the strength and stiffness properties of the diaphragms as well as the main lateral load resisting

elements. It is also important to identify the properties that will influence the out-of-plane strength of the walls as well as their in-plane performance.

It is likely that a visual inspection will have identified the main structural deficiencies and particular attention should be focused on these items during the detailed inspection. The main items to be inspected and the information to be recorded during the detailed inspection are summarised below. In compiling the list of information required it has been assumed that both securement and strengthening may be necessary.

4.4.2 General Requirements

a) Structural configuration

Most of the details of the structural configuration required for an analysis should be available on design or construction drawings. Where detailed plans are unavailable, field measurements will be necessary. As-built checks should also be made. It is recommended that a preliminary inspection be carried out to prepare sketch plans followed by a more detailed inspection to record the detailed dimensions on a set of drawings based on the preliminary work.

The structural configuration information gathered should include the following:

- Plans, elevations and dimensions of frames and walls on each level.
- Location and size of openings in walls and floors.
- Identification of load bearing/non-load bearing walls.
- Identification of any discontinuities in the structural system.
- Arrangement of roof and floor trusses, beams and lintels.
- Identification and location of reinforcing bands, columns and bracing.
- Dimensions of non-structural components to allow storey masses to be reliably assessed.
- Lift and stairwell construction and dimensions.
- Foundation dimensions, type and identification of connections between foundations and between superstructure and foundation.
- Clearances to adjacent buildings.

Identifying the structural configuration will enable both the intended load-resisting elements and the effective load-resisting elements to be identified. Effective load-resisting elements may include both structural and non-structural elements that participate in resisting lateral loads, whether or not they were intended to do so by the original designers. Potential discrepancies in intended and effective load-resisting elements may include discontinuities in the load path, weak links, irregular layouts, and inadequate strength and deformation capacities.

b) Element Properties

The following component properties should be determined:

- Cross-sectional shape and physical dimensions of the primary components and overall configuration of the structure.
- Configuration of connections, size and thickness of connected materials, and continuity of load path.
- Modifications to individual components or overall configuration of the structure.
- Location and dimensions of braced frames and shear walls.
- Current physical condition of components and extent of any deterioration present.

- Reinforcing details in reinforced concrete structures

Structural elements of the lateral-force-resisting system comprise primary and secondary components, which collectively define element strength and resistance to deformation. Behaviour of the components—including shear walls, beams, diaphragms, columns, and braces— is dictated by physical properties such as area; material; thickness, depth, and slenderness ratios; lateral torsional buckling resistance; and connection details. The actual physical dimensions should be measured; e.g., 50 x 100 mm timber dimensions are generally slightly less due to choice of cutting dimensions and later shrinkage. Modifications to members need to be noted. The presence of corrosion, decay or deformation should be noted.

These primary component properties are needed to properly characterize building performance in the seismic analysis. The starting point for establishing component properties should be the available construction documents. Preliminary review of these documents shall be performed to identify primary vertical- (gravity-) and lateral-load-carrying elements and systems, and their critical components and connections. Site inspections should be conducted to verify conditions and to assure that remodeling has not changed the original design concept. In the absence of a complete set of building drawings, the design professional must thoroughly inspect the building to identify these elements, systems, and components. Where reliable record drawings do not exist, an as-built set of plans for the building must be created.

The method of connecting the various elements of the structural system is critical to its performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions. The connection between a diaphragm and the supporting structure is of prime importance in determining whether or not the separate parts of the structure can act together.

If drawings of reinforced concrete buildings are not available it will be necessary to carry out on site investigations to obtain details of sizes and spacing of reinforcing bars. Investigations may include the removal of concrete cover in chosen locations to expose the reinforcing. Radar technology is now available in New Zealand using portable equipment which provides a practical non-destructive investigation alternative for some circumstances.

In the case of steel structures, some useful information is contained in the following three appendices:

- An overview of typical pre-1976 steel building systems used in New Zealand (Appendix 4A)
- Relationships between structural characteristics and steel building performance in severe earthquakes (Appendix 4B)
- Assessing the mechanical properties of members and components used in pre-1975 steel buildings (Appendix 4C)

Other useful background material is contained in the two references below, both of which are available on loan from HERA.

(1) **Appraisal of existing iron and steel structures** (Bussell 1997)

This very comprehensive publication addresses all appraisal topics, except seismic appraisal, and so forms a necessary general companion to a seismic assessment. It covers iron and steel structures built from 1780 to 2000. Although written for the UK, much of the guidance is generic and will be of relevant application in New Zealand. Topics covered are:

- history and manufacture of iron and steel
- iron and steel use in building construction
- appraisal strategy

- properties of structural iron and steel
- defects and deterioration
- identification, examination, measurement and testing
- structural assessment
- assessment of fire performance
- load testing
- repair, strengthening and replacement
- fire and corrosion protection.

(2) Historical record of dimensions and properties for rolled shapes in steel and wrought iron (Ferris).

This publication, from the American Institute of Steel Construction, covers the dimensions and properties of rolled beams and columns from 1873 to 1952. It is the most comprehensive source of early section properties available from HERA.

c) Material properties

In the assessment of an existing structure, realistic values for the material properties, particularly strengths, must be used to obtain the best estimate of the strengths and displacements of members, joints and connections.

Material properties and strengths that were specified in the original design are not appropriate for use in assessment procedures.

The effect of variations in material strength on the hierarchy of failure must be considered.

2 General definitions of material strengths are given in Section 7.1, along with Section 4.7.

4.4.3 Particular Check Items

a) Load bearing masonry wall materials

The type and strength of the masonry in the main load bearing walls is important and should be determined by coring and structural testing.

The thickness of all walls needs to be measured and the masonry lay-up determined. The bonding between the wythes should be identified and the spacing between headers and bonders determined. Coring in typical locations will probably be necessary.

The lay-up at corners should be inspected to identify any lack of bonding.

Scrape testing with a blade screwdriver can be used to investigate the uniformity of the joint mortar. Alternatively, a 3 mm diameter nail punch may be driven with six firm blows of a standard carpenter's hammer into vertical mortar joints. A penetration of 10 mm to 35 mm is typical for lime mortar. In-place mortar shear tests and bed joint shear tests (see Section 10) will need to be carried out to determine the mortar strength if it is likely to be critical.

The continuity of the walls between storey levels and through to the foundation should be checked.

The size and location of cracks in the masonry should be identified and recorded together with notes on any other defects or signs of deterioration in the wall materials.

b) Non-load bearing walls

Non-load bearing walls may stiffen the floor diaphragms and brace the main load bearing walls. The weight of non-load bearing walls may also be a significant component in the total weight.

The materials and construction details of the non-load bearing walls should be recorded.

c) Floor materials

The type of floor construction should be identified and the dimensions of all the supporting structure recorded. The continuity of toppings or timber flooring should be assessed and the location of joints recorded.

The details of any reinforcing at openings should be noted.

Cracking or other signs of distress should be recorded with details of location and severity.

d) Roof materials

The roof materials and supporting structural system should be identified and dimensioned. The effectiveness of any roof bracing system in providing diaphragm action should be assessed.

e) Connections

The size and spacing of connections between floors and walls should be determined. Where the strength of the connections cannot be reliably calculated, site tests should be conducted.
Anchorages in masonry walls and veneer tie locations should be noted and their strength assessed.

An assessment should also be made of the influence of any corrosion or deterioration of the connections on their load capacity.

The type, size and spacing of roof to wall connections should also be identified.

Particular details to determine include:

- Connections between horizontal diaphragms and shear walls and braced frames.
- Size and character of all drag ties and struts, including splice connections in timber diaphragms.
- Connections at splices in chord members of horizontal timber diaphragms.
- Connections of horizontal diaphragms to exterior concrete or masonry walls for both in-plane and out-of-plane walls.
- Connections of cross tie members for concrete or masonry buildings
- Connections of shear walls to foundations for transfer of shear and overturning forces.
- Method of through-floor transfer of wall shear and overturning forces in multi-storey buildings.
- Connections between beams and columns in individual steel or timber frames.

f) Appendages

Any appendages or falling hazards that are likely to affect the safety of the building occupants or passers-by should be assessed. The location, construction, condition and bracing of each item should be recorded.

Specific features that should be considered include:

- chimneys
- veneers
- gables
- parapets, cornices, canopies and ornamentation
- water tanks
- tower-like appendages
- fire escapes
- lift wells
- glass facades
- heavy equipment
- heavy lighting fittings
- Plaster and other heavy renders

g) Condition, maintenance and alterations

The condition of all structural components should be recorded with particular attention given to deterioration such as cracking, spalling, corrosion and decay. Locations and extent of any significant deterioration should be recorded. Any lack of water-tightness in the roof and wall openings should be noted.

The foundation soil type should be determined and a careful inspection made to identify any settlement or indications of foundation distress.

Damage from previous earthquakes or other overloads should be carefully inspected and recorded.

The impact of any building alterations on the performance of the main structural elements should be considered carefully.

4.5 Relationship with Current Loadings Standards

In developing these Guidelines, the provisions of the loadings standard, NZS 1170.5:2004 have been taken to apply in the assessment of existing buildings. Such provisions should be taken as required unless specifically provided for otherwise in this document.

100*%NBS* is taken in these Guidelines as 100% of the standard implied through the application of NZS 1170.5:2004.

Until AS/NZS 1170 and NZS 1170.5 replace NZS 4203 as a Verification Method, it is important that users satisfy themselves that in applying the parameters in this section, they are meeting the relevant requirements of NZS 4203:1992 unless an Alternative Method has been accepted and agreed by the Territorial Authority.

Key provisions that are not modified unless specific reference otherwise is made in the subsequent materials sections include: directionality of earthquake action; horizontal and vertical irregularity and their relationships with methods of analysis; and P-Δ effects under a force-based approach.

The requirements for parts within NZS 1170.5 (Section 8) apply for all secondary and non-structural elements, subject to the application of the appropriate factors from Section 5.

Particular consideration should be given to assessing the displacement capacity of existing parts, given their possibly brittle nature. Interactions with the structure must also be considered.

4.6 Overall Structural Response Considerations

Given the possible existence of critical structural weaknesses (CSWs), the following aspects of overall structural response should receive particular attention.

a) Potential for pounding

For many existing buildings, compliance with the current requirements for building separation have not been met. With insufficient building separation, there is a high risk that building-to-building impact or pounding will greatly impair the performance of both structures. Guidance on how to assess and mitigate the pounding issue is given in Appendix 4D.

Resolving pounding issues will, in many cases, be very difficult due to the different ownership of adjacent buildings. However the issue should not be ignored and the potential detrimental effect on the buildings performance must be included in the assessment.

b) Horizontal irregularity

While modern standards discourage horizontal irregularity and prescribe limitations on eccentricity of load and stiffness, existing buildings may have severe horizontal irregularities. Such irregularities call for special consideration beyond that normally covered in codes for new buildings. The twisting of the building as it responds to earthquake motions can give rise to higher than normal ductility demands on perimeter elements, and requires special consideration.

c) Vertical irregularity

Vertical irregularity is a major threat to structure integrity. It can drive up ductility demands on key structural elements, particularly columns, and in some circumstances, compound horizontal irregularity and other critical structural weaknesses.

It is vital that the structural analysis models the effects of vertical irregularity realistically. Assessment of ductility demand on key elements must take full account of the effects of the irregularities and the displacements generated in the structural members.

d) Short columns

It is vital that both overall analysis and assessment of ductility demand take proper account of the characteristics of short columns. Displacements generated in the structure can have a severe effect on the integrity of these elements by driving up shear forces beyond the capability of the sections.

e) Diaphragms and their interconnection with primary structural elements

For concrete diaphragms, attention is drawn to the provisions of Section 13 of NZS 3101:1995 (SNZ 1995) and clause 6.1.4 NZS 1170.5. The added complication of transfer diaphragms, particularly when not originally designed as such, should be noted.

For existing buildings containing a high level of pre-cast elements, attention is drawn to the interconnection provisions of Section 4.4 of NZS 3101:1995.

4.7 Member Capacity Considerations

a) Material Properties and Member Strengths

In the assessment of an existing structure, realistic values for the material properties, particularly strengths, must be used to obtain the best estimate of the strengths and displacements of members, joints and connections.

Material properties and strengths that were specified in the original design are not appropriate for use in assessment procedures.

The effect of variations in material strength on the hierarchy of failure must be considered.

Definitions of material strengths are given in e) below. Specific guidance is given in the various material sections.

b) Material Strengths

General definitions of material strengths are given below.

The following specific notes relate to reinforced concrete.

Further specific guidance is given in the procedures presented.

c) Use of probable strengths

In determining the strength and deformation capacities of an existing component, calculations shall be based on the probable values of strengths for the materials in the building.

Probable strengths should be used in order to identify the hierarchy of actions, and hence the most likely failure mechanism. The probable or measured nominal strengths are the best estimates of the actual strengths obtained from available information and/or testing.

The actual mean of any tests shall be used as the probable strength in the first instance. However, careful consideration of the effect of variations from the mean (up or down) should be made since this could, for example, affect relative strengths of beams and columns.

The probable strengths are to be based on either actual test results or the default materials strengths given in the subsequent material sections.

Strength reduction factors specific to each material are given in the respective sections.

d) Reliability of information

It may be necessary to account for the uncertainty with regard to the reliability of available information and its effect on the configuration and condition of a component.

It is important to recognise that the strengths of the building materials can vary from member to member, and to take this into account.

Any allowances should be established from the knowledge that the engineer is able to obtain based on access to the original construction documents and/or by surveys and physical testing of representative samples of materials.

It is suggested that, in the absence of other data, variations considered should be one standard deviation or ± 20% of the mean.

ATC 33.03 (ATC 1995) establishes three categories of building information, corresponding to Good, Fair and Poor information classes. Reference to ATC 33.03 may be of assistance in determining what, if any, allowance to make.

e) Definitions of Material Strengths

The following definitions of material strengths are given as an aid to interpretation.

Nominal Strength, S_n

For concrete the nominal strength is the theoretical strength of a member section based on established theory, calculated using the section dimensions as detailed and the lower characteristic reinforcement yield strengths (5 percentile values) and the specified nominal compressive strength of the concrete.

The nominal strength gives a lower bound to the strength of the section and is the value used in design.

For steel the nominal strength is the minimum yield stress and tensile strength based on published data or on test results.

Probable Strength, (Also called "Expected Strength")

The probable strength is the theoretical strength of a member section based on established theory, calculated using the section dimensions as detailed and the mean material strengths. Mean material strengths may be taken as either

- the means of the 5% ile and 95% ile characteristic strengths
- the mean of series of measurements of material strength at the locations in question.

For steel and masonry take the probable strength as equal to the nominal strength.

Overstrength S_o

The overstrength value takes into account factors that may contribute to strength increase such as higher than specified strengths of the steel and concrete, steel strain hardening, confinement of concrete and additional reinforcement placed for construction and otherwise unaccounted for in calculations.

Section 5 - Detailed Assessment - Modelling the Earthquake

5.1 General

Representation of earthquake shaking will vary according to the method of analysis. Regardless of this, the earthquake input to the analysis needs to correspond to the desired level of earthquake shaking applicable to the assessment. In most if not all cases this will involve simple scaling of appropriate parameters, e.g. spectral acceleration, spectral displacement.

The range of earthquake representations available includes:

- acceleration response spectra
- displacement response spectra
- acceleration, velocity or displacement records.

The choice of which representation to use will depend on the analysis method. It is anticipated that for most buildings engineers will adopt the familiar approaches of the latest loadings code (NZS 1170.5:2004 or NZS 4203:1992). Nevertheless, the wide variability of characteristics of existing buildings may require the use of alternative approaches to obtain a realistic assessment of structural performance.

Note
In this section, material presented is in line with the Loadings Standard NZS 1170.5. This choice has been made because this Standard presents the most up-to-date estimates of earthquake hazard in New Zealand and the acceleration hazard spectra are of a stylised form that allow appropriate displacement response spectra, fundamental to displacement based approaches for assessment and design, to be directly derived.

The following sections provide more detail on available representations of earthquakes for analysis purposes and give guidance on selection of appropriate ones for various circumstances.

5.2 Acceleration Response Spectra

Acceleration response spectra are routinely used by designers.

The elastic site hazard spectra spectral accelerations, $C(T)$, used as a basis for these guidelines shall be derived from Section 3 of NZS 1170.5.

Historical buildings of significant cultural significance should be assigned Importance level 3 unless this classification would result in significant disruption to historical fabric. In such cases Importance Level 2 may be assigned but with the expectation of greater damage in a large (low probability) earthquake.

Acceleration spectra for different damping values may be obtained by multiplying $C(T)$ by the factor:

2 $$K_\xi = [7/(2+\xi_e)]^{1/2} \qquad \ldots 5(1)$$

where ξ_e = equivalent viscous damping factor

5.3 Displacement Response Spectra

For displacement-based methods, a displacement response spectrum is required. For the purposes of these guidelines it is considered appropriate to derive the site hazard spectral displacements, $\delta(T)$, from the 5% damped elastic site hazard spectral accelerations, $C(T)$, using the following relationship:

- $\delta(T) = 9800\ C(T)\ T^2/4\pi^2$. …5(2)

Figure 5.1 illustrates the shape of the resulting displacement spectra for Wellington, Christchurch and Auckland for different subsoil conditions.

Displacement spectra for different damping values may be obtained by multiplying the displacement given by eqn 5(2) by the factor, K_ξ, calculated using eqn 5(1):

The effect of the application of K_ξ is illustrated in Figure 5.2.

The periods and damping values used in displacement-based assessments are not identical to those used in force-based assessments. Force-based assessments consider the initial period and damping of the structure. The effect of an increasing period and structural damping corresponding to the ductility demand are included in the scaling down of the elastic forces through the ductility parameter and, to some extent, the application of the Structural Performance Factor, S_p.

On the other hand, displacement-based assessments should be carried out assuming that the structure is responding at the maximum applied displacement. It is appropriate, therefore, to consider the period corresponding to the secant stiffness for the maximum applied displacement and damping as a function of the beyond-yield deformation mechanism. This is described in more details in the relevant sections.

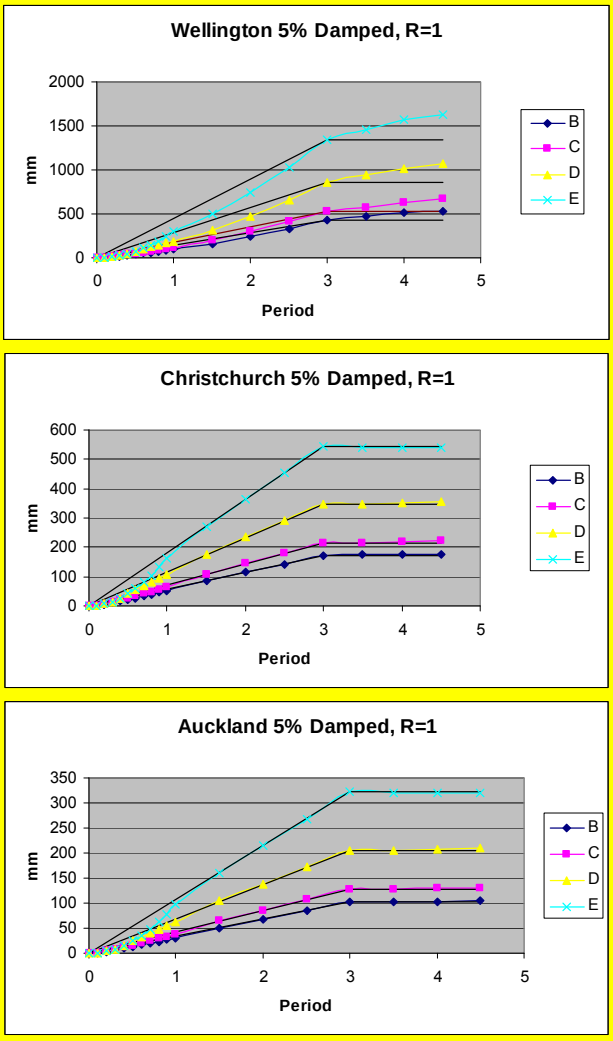


Figure 5.1: Displacement spectra at 5% damping for R = 1

Examination of the displacement spectra in Figures 5.1 and 5.2 reveals a number of interesting points.

First, the significance of soil type is much more apparent when seismicity is expressed in terms of displacement, rather than acceleration spectra.

Second, apart from some non-linearity for low periods, the curves are well represented by straight lines from the origin as drawn on Figure 5.2. For sites where near fault effects are not an issue the displacement spectra are well represented by a bilinear relationship pivoting around the displacement at T=3sec and the leg horizontal beyond 3sec. For a site where near-fault effects are specified the displacement spectra can be approximated by a bilinear relationship between T=0, 3 and 4.5sec. These are approximations, the validity of which will be confirmed during studies expected to commence in 2006. It is expected that the straight-line approximations indicated are sufficiently accurate to be used as the basis for assessments and design of retrofit works but should not preclude a more precise or direct evaluation should circumstances warrant or allow.

Third, the displacement spectra obtained do not represent the tendency of the spectral displacement to converge to the peak ground displacement at long periods but conservatively maintain the spectra at constant peak displacement response values (or increasing in the case of the sites with near-fault effects).

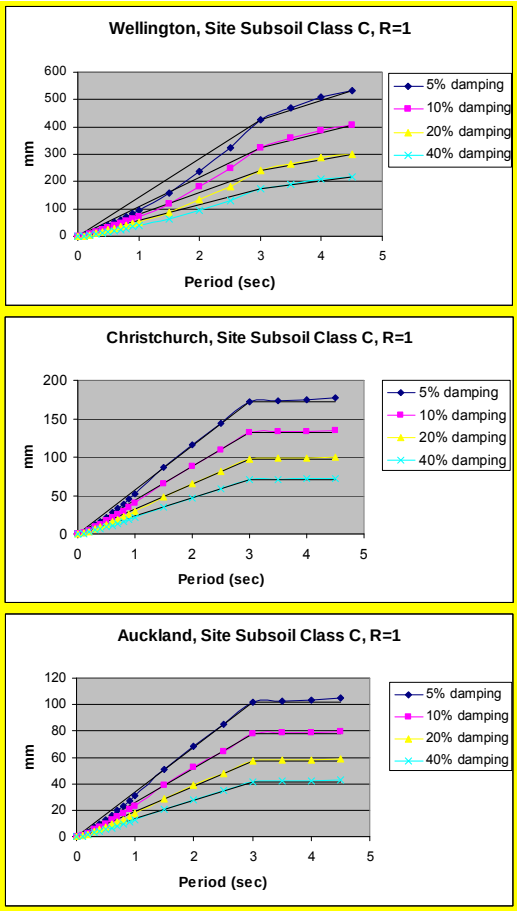


Figure 5.2: Displacement spectra for different damping levels

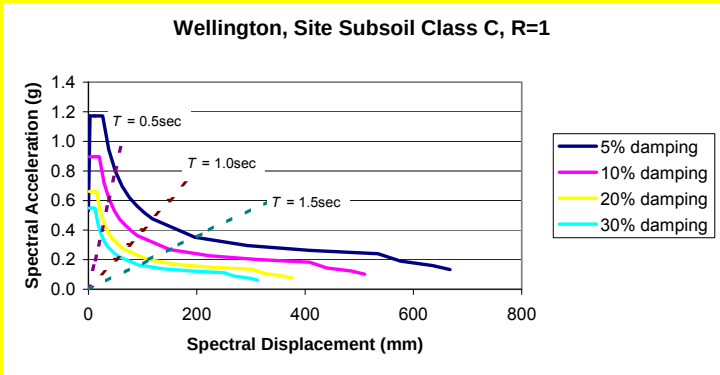
The acceleration spectra defined in NZS 4203 are not in a form that allows direct derivation of realistic displacement spectra, particularly at longer periods. In preference, the NZS 1170.5 acceleration spectra should be used.

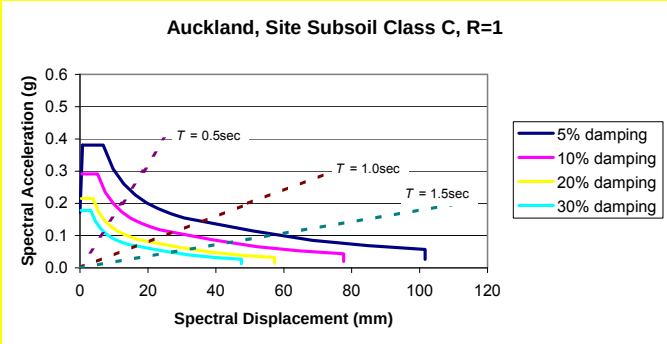
5.4 Acceleration-Displacement Response Spectra

The acceleration and displacement spectra derived above for a particular site and level of damping can be usefully presented in the form of an acceleration-displacement response spectrum (Mahaney et al, 1993). The ordinates of such a spectrum are spectral acceleration and spectral displacement. An example of such a representation is shown in Figure 5.3 for Wellington, 500 year return period (R = 1) and site subsoil class (C).

When constructing an acceleration-displacement spectrum for a particular level of damping both $C(T)$ and $\delta(T)$ must be multiplied by K_ξ.

Acceleration-displacement spectra are particularly useful when assessing the performance of a building from the results of a non-linear pushover analysis. The acceleration and displacement results from a pushover analysis need to be converted to spectral acceleration and spectral displacement before comparisons are possible with the acceleration-displacement spectra described above.





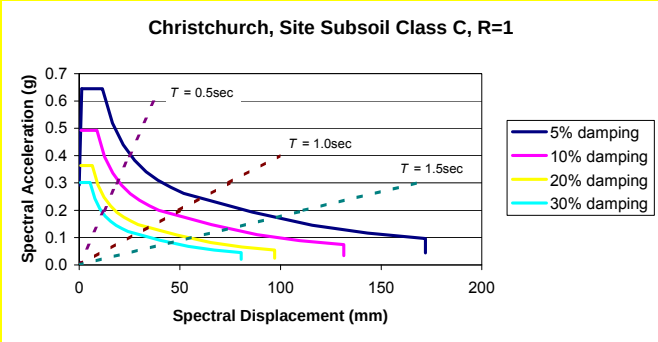


Figure 5.3: Acceleration-Displacement Spectra for different damping levels

The conversion can be carried out as follows (after ATC 40), assuming that elastic response is a good predictor of inelastic response (this will not always be the case);

$$S_a = V/W/\alpha_1 \qquad ...5(3)$$

$$S_d = U_{roof}/PF_{R1} \qquad ...5(4)$$

where V = base shear consistent with U_{roof}

U_{roof} = roof top displacement

α_1 = modal mass coefficient for the first mode (typically taken equal to 1.0 when used with code spectra)

PF_{R1} = modal participation factor for the first mode at roof level

Note that the period, T, can be derived from the relationship;

$$T = 2\pi\sqrt{(S_d/S_a)} \qquad ...5(5)$$

where S_a, S_d are as defined above.

Thus the stiffness of the building (*T*) can be represented by radiating lines from the origin of the acceleration-displacement spectrum. These lines for example periods of 0.5, 1.0 and 1.5sec are shown in Figure 5.3.

ATC 40 (1996) presents an excellent discussion on the way in which the acceleration-displacement spectrum can be derived and used to assess the performance of buildings.

5.5 Acceleration Ground Motion Records and Time History Analyses

The choice and scaling of acceleration ground motion records for use in time history analyses shall meet the requirements of NZS 1170.5 clause 5.5.

The records shall be consistent with the magnitude, fault distance, source mechanisms and ground conditions of the earthquakes dominating the design ground motion. In particular, the records should include adequate representation of near-fault effects for sites where those could significantly contribute to the seismic risk.

Adequate artificial time histories can also be used if suitable historical records are not available. In any case, the input earthquake records shall contain at least 15 seconds of strong motion shaking, or have a strong shaking duration of at least 5 times the fundamental period of the structure, whichever is greater.

Time history analyses require a significant amount of judgement. They shall be conducted in accordance with sound analytical practice, and all modelling of the structure shall be cautiously appraised. Unless otherwise justified, material and structural properties shall be determined from the appropriate material standards modified as required by these guidelines. These include, but are not limited to, adequate consideration of the effects of strain hardening and possible degradation where appropriate. Damping values shall also be realistic.

The calculated structural periods of interest shall take account of potentially significant lengthening of the small displacement periods as the structure undergoes a ductile or large displacement response. P-δ effects shall be included directly in the analysis. Vertical acceleration, excitation in two horizontal directions as well as torsion shall be considered in the analysis.

5.6 Incorporation of the Structural Performance Factor, S_p

The Structural Performance Factor, S_p from NZS 1170.5 may be used either to reduce the demand spectral values calculated above (this is the approach adopted in NZS 1170.5) or used to enhance the capacity as assessed later in these guidelines.

5.7 Lateral Force/Displacement Requirements

The lateral force/ displacement requirements are found by multiplying the demand values calculated as outlined above, by $(\%NBS)_t$, where $(\%NBS)_t$ is the target *%NBS* for the analysis in question.

5.8 $(\%NBS)_t$ factor

This factor has been introduced to emphasise that the earthquake spectra need to be scaled to match the target (*%NBS*) to be used. For example, to check compliance with Building Act triggers would require a target of 33%. An initial upgrading target would be 100%, moderated to a lower value if this was as nearly as is reasonably practicable to 100% that could be achieved. To determine the capacity of a structure may require several iterations using different levels of $(\%NBS)_t$.

Section 6 - Detailed Assessment - Procedures

6.1 General

Historically, the seismic assessment of existing buildings has been undertaken by attempting to invert the design process. This has been neither straightforward nor successful, as the current capacity design procedures for the design of new structures are deterministic in nature.

A first-principles approach was followed by Priestley and Calvi in 1991, resulting in a force-based method for reinforced concrete frames (Priestley and Calvi 1991). This work was developed further by Priestley in 1995, but in the form of a displacement-based approach. (Priestley 1995). The force-based method has been organised into a step-by-step design office procedure by Park in 1996 (Park 1996), using New Zealand seismic hazard acceleration spectra and New Zealand and United States test data for the strength and ductility of reinforced concrete members and their connections.

This section presents the general outline of both force-based and displacement-based procedures, which consolidate this earlier work into comparable step-by-step processes.

While it is generally considered that displacement-based methods produce more rational and less conservative assessment outcomes, it is acknowledged that most designers are currently more familiar with force-based approaches.

Also presented in this Section is an assessment approach that consolidates both the force-based and displacement based procedures outlined below into one assessment procedure and provides the assessor the option of adopting either a force-based or displacement based method of determination of the capacity of the building. The primary objective of this approach is the determination, through a detailed assessment, of the *%NBS* score for the building. A procedure using a non-linear push-over analysis is also presented.

The following outlined procedures are considered to be applicable to all lateral force resisting elements and materials. However they may require some modification in some circumstances. The necessary adaptation for particular materials and structural forms is indicated in subsequent sections.

Section 4.7 contains a more detailed outline of what is involved in the steps of the force-based and displacement-based methods of assessment for a reinforced concrete framed structure.

6.2 Force-Based Methods

The assessment procedure is based on determining the probable strength and ductility of the critical mechanism of post-elastic deformation of the lateral force-resisting elements.

Once the available lateral load strength and displacement ductility of the structure has been established, reference to the 100*%NBS* response spectra for earthquake forces for various levels of structural ductility factor then enables the designer to assess the likely seismic performance of the structure in relation to that of a new building. Such comparisons will need to take account of any modifications to *NBS* requirements necessary to address existing buildings (as given in these Guidelines).

The key steps of a force-based seismic assessment procedure following the recommendation of Park (Park 1996) can be summarised as follows:

Step F1: Determine the probable flexural and shear strengths of the critical sections of the members and joints assuming that no degradation of strength occurs due to cyclic lateral loading in the post-elastic range.

Step F2: Determine the post-elastic mechanism of deformation of the structure that is likely to occur during seismic loading and the probable horizontal seismic base shear *capacity* of the structure, V_{prob}. The post-elastic mechanisms can be investigated using the SLaMA, the Inelastic Time History Method presented in NZS 1170.5 and/or a progressive non-linear pushover analysis in accordance with Appendix 4E.

Step F3: Estimate the fundamental period of vibration, T_1, and calculate the total seismic weight, W_t, of the structure and the structural performance factor, S_p, appropriate for the detailing used in the structure.

Step F4: Determine the implied inelastic spectrum scaling factor, k_μ, corresponding to the probable lateral force capacity of the structure, V_{prob}, found in Step F2 from:

$$k_\mu = \frac{C(T_1)S_p W_t (\%NBS)_t}{V_{prob}} \qquad ...6(1)$$

where:

$C(T_1)$	=	ordinate of the elastic site hazard spectrum for T_1 and for the site, calculated in-accordance with Section 3, NZS 1170.5.
W_t	=	total seismic weight of the structure
S_p	=	structural performance factor
$(\%NBS)_t$	=	target percentage of new building standard (refer Section 5).

Step F5: Determine the *required* structural ductility factor μ corresponding to k_μ using the equations given in Section 5 NZS 1170.5.

Step F6: Evaluate whether the identified plastic hinge regions have the available ductility to match the *required* overall structural ductility factor μ. The element will require retrofitting if the rotation capacity of the plastic hinges is inadequate and/or $(\%NBS)_t$ will need to be reduced.

Step F7: Estimate the degradation in the shear and bond strength of members and joints during cyclic deformations at the imposed curvature ductility factor in the plastic hinge regions. Check whether any degradation in shear and bond strength will cause failure of the members or joints. If it does not, then the assessment apart from Step F8 is complete. If it does, the structure will require strengthening and/or $(\%NBS)_t$ will need to be reduced.

Step F8: Estimate the interstorey drift and decide whether it is acceptable in terms of the requirements of NZS 1170.5.

The sequencing of and interaction between these steps is shown in flowchart form in Figure 6.1.

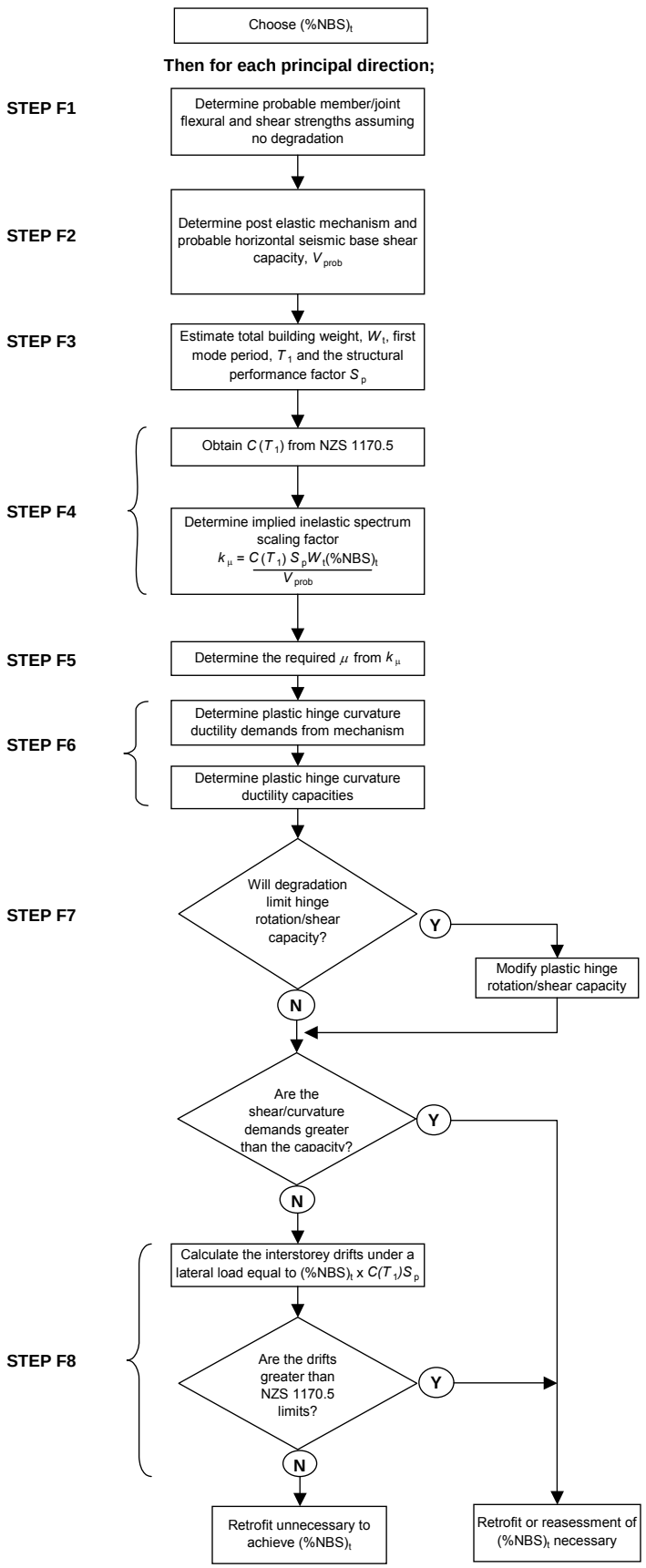


Figure 6.1: Summary of force-based assessment procedure

The procedure for carrying out these steps is discussed in more detail in the following sections for the respective materials. The actual sequence and number of steps varies depending on both the material and configuration, and is modified as appropriate.

6.3 Displacement-Based Methods

Displacement-based methods place a direct emphasis on establishing the ultimate displacement capacity of lateral force resisting elements. Displacement-based assessment utilises displacement spectra which can more readily represent the characteristics of real earthquakes.

The development of procedures encompassing this approach represents a relatively recent development. In 1995 Priestley developed an outline of the key steps for such a procedure for reinforced concrete buildings. He has taken this work further, with appropriate simplifications, to produce the following general procedure which is considered more suitable for use in a design office context. The procedure is elaborated upon further for reinforced concrete frame and wall elements in the subsequent sections.

The modified loading factors from Section 5 are to be applied at appropriate stages during this process. The key steps of a displacement-based seismic assessment procedure can be summarised as follows:

Step D1: Determine the probable flexural and shear strengths of the critical sections of the members and joints assuming that no degradation of strength occurs due to cyclic lateral loading in the post elastic range.

Step D2: Determine the post-elastic deformation mechanism of the structure that is likely to occur during seismic loading, and hence the probable horizontal seismic base shear capacity, V_{prob}, of the structure. The post elastic mechanisms can be investigated using the SLaMA, the Inelastic Time History Method presented in NZS 1170.5 and/or a progressive non-linear push-over analysis in accordance with Appendix 4.7A.

Step D3: Calculate member plastic rotation capacities using moment curvature analyses.

Step D4: Determine whether shear failure will occur before the limits to flexural plastic rotation capacity are reached. The available plastic rotation capacity is reduced if necessary to the value pertaining at shear failure. The storey inelastic drift capacity is estimated from the plastic rotation capacities. Check that the drifts are less than the limits prescribed in NZS 1170.5.

Step D5: The overall structure displacement capacity, U_{sc}, and ductility capacity, μ, are found from the mechanism determined in Step D2 and the critical storey drift. U_{sc} is the sum of the elastic and inelastic displacements (U_{el} +U_{inel}) and μ is the ratio of U_{sc}/U_{el}. U_{sc}, U_{el}, and U_{inel} are measured at the effective height, h_{eff}, of the substitute structure.

Step D6: Calculate the effective stiffness at maximum displacement, and the corresponding effective period of vibration.

Response can be considered directly in terms of displacement, using the substitute-structure approach of Shibata and Sozen (1976). In this, the structural period T is not related to the initial elastic stiffness k_e, but to the effective stiffness k_{eff} at maximum displacement, as shown in Figure 6.2. Thus:

$$T_{eff} = 2\pi\sqrt{M/k_{eff}} = 2\pi\sqrt{W_t/gk_{eff}} \qquad ...6(2)$$

where M is the effective mass of the substitute structure and W_t is the total weight of the structure.

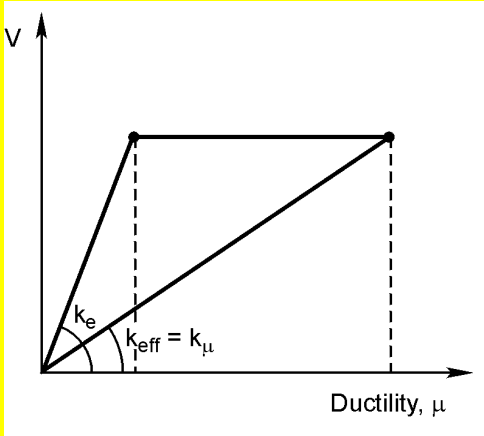


Figure 6.2: Derivation of effective stiffness

Thus seismic response is characterised by an equivalent elastic stiffness and damping corresponding to maximum response, rather than initial values, based on k_e and 5% damping, as typically used in force-based design or assessment.

Alternatively, T_{eff} can be estimated directly at any stage of the analysis using the Rayleigh-Ritz equation;

$$T_{eff} = 2\pi\sqrt{\sum\left(W_i u_i^2\right) / g\sum\left(F_i u_i\right)} \qquad 6(3)$$

where W_i, u_i and F_i are respectively the weight, lateral displacement and the implied inertial force at level i, at any stage of the analysis.

The use of eqn 6(3) avoids the need to estimate h_{eff} (or the displacement at h_{eff}) which as discussed below can be problematical.

Determine the equivalent viscous damping of the structure.

Calculate the structural performance factor, S_p, appropriate for the detailing used in the structure.

Step D7: Determine the structure spectral displacement demand at height h_{eff}, $U_{sd} = S_p(\%NBS)_t\delta(T_{eff})K_\xi$ using $\delta(T_{eff})$ and K_ξ from Section 5.

Step D8: Compare the displacement capacity, U_{sc}, against the demand, U_{sd}, and establish compliance or otherwise.

Displacement spectra to be used are given in Section 5.3.

The sequencing of and interaction between these steps is shown in flowchart form in Figure 6.3.

To enable comparison with the spectral displacement demand, the overall structure displacement capacity, U_{sc}, the elastic displacement, U_{el}, the effective stiffness and ductility, μ, are determined for the equivalent single degree of freedom model of the structure (substitute structure). U_{sc} and U_{el} can be approximated as the lateral deflection at an effective height, h_{eff} of the structure. The determination of h_{eff} is reliant on a good knowledge/understanding of the elastic and inelastic behaviour of the structure and is not readily amenable to simple calculation once the structure is no longer elastic. Some guidance as to appropriate values of h_{eff} are given in the following sections and in Appendix 4E. For the elastic case U_{el} is the top storey displacement divided by the modal participation factor for the roof level. If little more is known about the particular characteristics of the structure under consideration, and there are no column mechanisms, it is considered reasonable to use the same factor to approximate inelastic behaviour.

2 The assessment of the equivalent viscous damping, ξ_e, also requires judgement and care, as the results are quite sensitive to the choice that is made.

2 Various references are available which give guidance on the calculation of ξ_e (Pekcan 1999)(ATC 40)(FEMA 440). However, it is recommended that ξ_e be determined using the method suggested by Pekcan et al (Peckan 1999) as follows;

2

$$\xi_e = \xi_0 + \xi_{hy} + \xi_d = \xi_0 + \frac{2}{\pi}\eta\frac{(1-\alpha_s)(1-1/\mu)}{(1-\alpha_s+\mu\alpha_s)} + \xi_d \qquad ...6(4)$$

where

ξ_0	=	the inherent damping (typically taken as 5%)
ξ_{hy}	=	the hysteretic damping
ξ_d	=	added damping due to supplemental viscous dampers. Taken as zero if there are no dampers present.
μ	=	displacement ductility
α_s	=	post yield to initial stiffness ratio
η	=	efficiency factor, defined as the ratio of the actual area enclosed by the hysteresis loop to that of the assumed perfect bilinear hysteresis.

2 Typical values for ξ_e (expressed as a fraction of 1.0) are shown in Table 6.1.

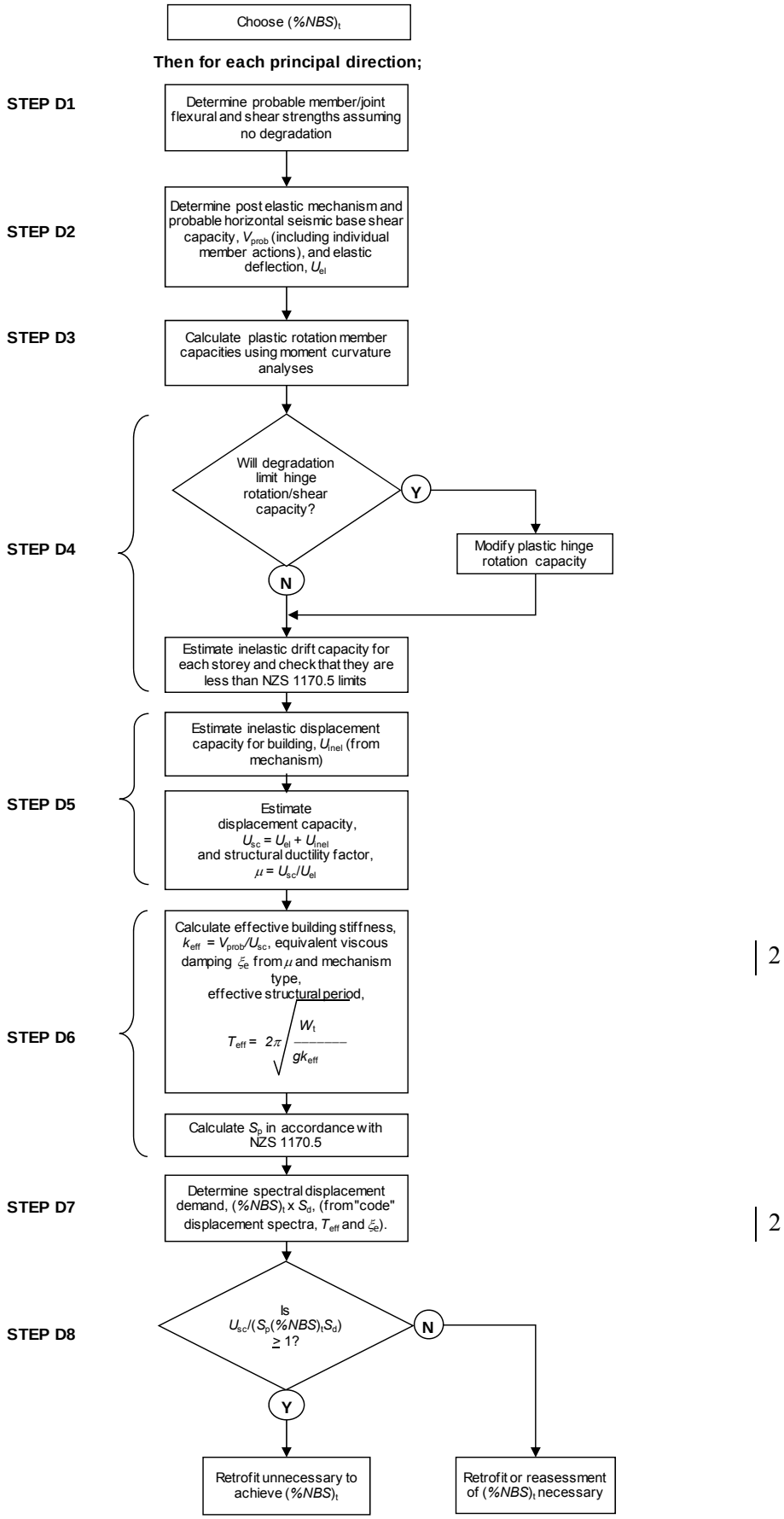
The various parameters given in Eqn 6(3) are shown diagrammatically in Figure 6.4.

2 For unreinforced masonry walls use ξ_e = 0.15 (15%) for walls loaded inplane for the reasons outlined in section 10.2.6 (b). ξ_e = 0.05 (5%) should be used when assessing face loading on masonry walls.

6.4 Consolidated Force and Displacement Based Procedure

It will be apparent that there are similarities in some of the steps for the force and displacement based procedures outlined above. If the same steps from each procedure are put together then a consolidated procedure can be formulated. Such a procedure is shown in Figure 6.5. This general procedure is recommended by these guidelines.

This procedure assesses the *%NBS* that is available from the un-retrofitted or retrofitted building. The basic steps required are described above.



2

2

Figure 6.3: Summary of displacement-based assessment procedure

2 **Table 6.1 Typical values of ξ_e for various structural types and materials**

2

Material	Structural Type	μ	η	ξ_e						
				α_s						
				-0.05	-0.03	0	0.02	0.05	0.1	0.2
Concrete	Ductile	6	0.35	0.31	0.28	0.24	0.22	0.19	0.16	0.12
	Limited Ductile	3	0.3	0.19	0.2	0.18	0.17	0.16	0.15	0.12
	Limited Ductile	2	0.25	0.14	0.13	0.13	0.13	0.12	0.12	0.10
	Nominally Ductile	1.25	0.2	0.08	0.08	0.08	0.07	.07	0.07	0.07
Timber	Limited Ductile	3	0.15	0.12	0.12	0.11	0.11	0.10	0.10	0.09
Steel	Ductile	6	0.65	0.47	0.53	0.39	0.36	0.31	0.26	0.19
	Limited Ductile	3	0.4	0.25	0.24	0.22	0.21	0.2	0.18	0.15
All	Rocking Walls	3	0.15	0.12	0.12	0.11	0.11	0.10	0.10	0.09

Notes After Pekcan et al (Pekcan 1999)

ξ_0 = 5%

Typical values shown shaded.

For unreinforced masonry walls use ξ_0 =0.15 (15%) for walls loaded in-plane and ξ_0 = 0.05 (5%) for walls loaded perpendicular to the face.

The value of μ in the table relates to the displacement ductility experienced at the level of loading considered. Thus, even though a structure may be detailed to achieve μ = 6, the value of ξ_{eff} should be chosen assuming μ = 3, if the structure is only loaded to say half capacity. Generally, assessors will be interested in performance at $(\%NBS)_t$ and so only one value of ξ_{eff} will need to be assessed.

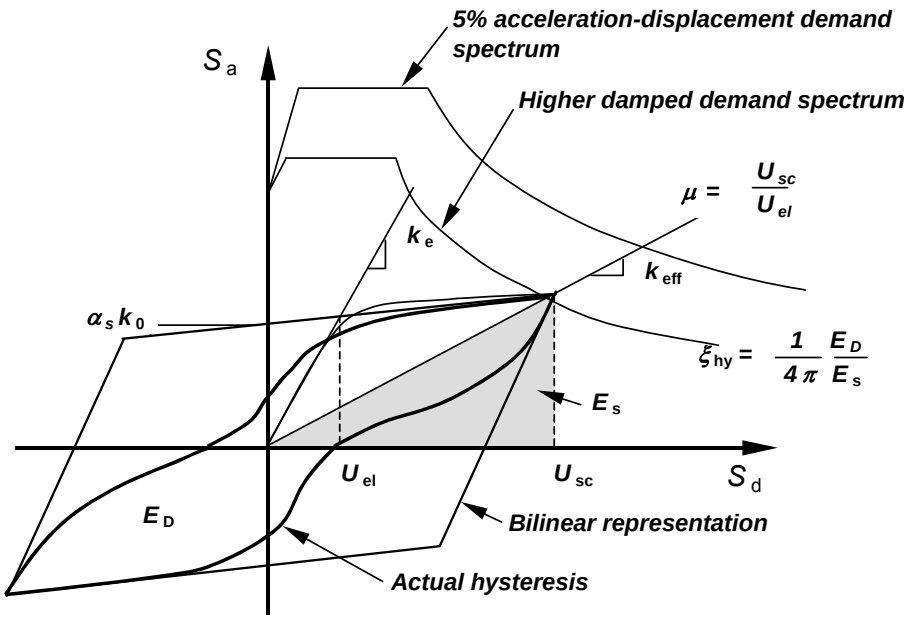


Figure 6.4 Explanation of terms in Eqn 6(3)

6.5 Non-Linear Pushover Procedure

A possible assessment procedure utilising a non-linear push-over analysis is shown in Figure 6.6. As for the consolidated procedure outlined above, the pushover analysis procedure proposed determines the *%NBS* capacity of the building. 2

If the results of the pushover analysis are plotted over the acceleration-displacement demand spectra for the appropriate level of equivalent viscous damping an excellent summary of the performance of the building can be obtained.

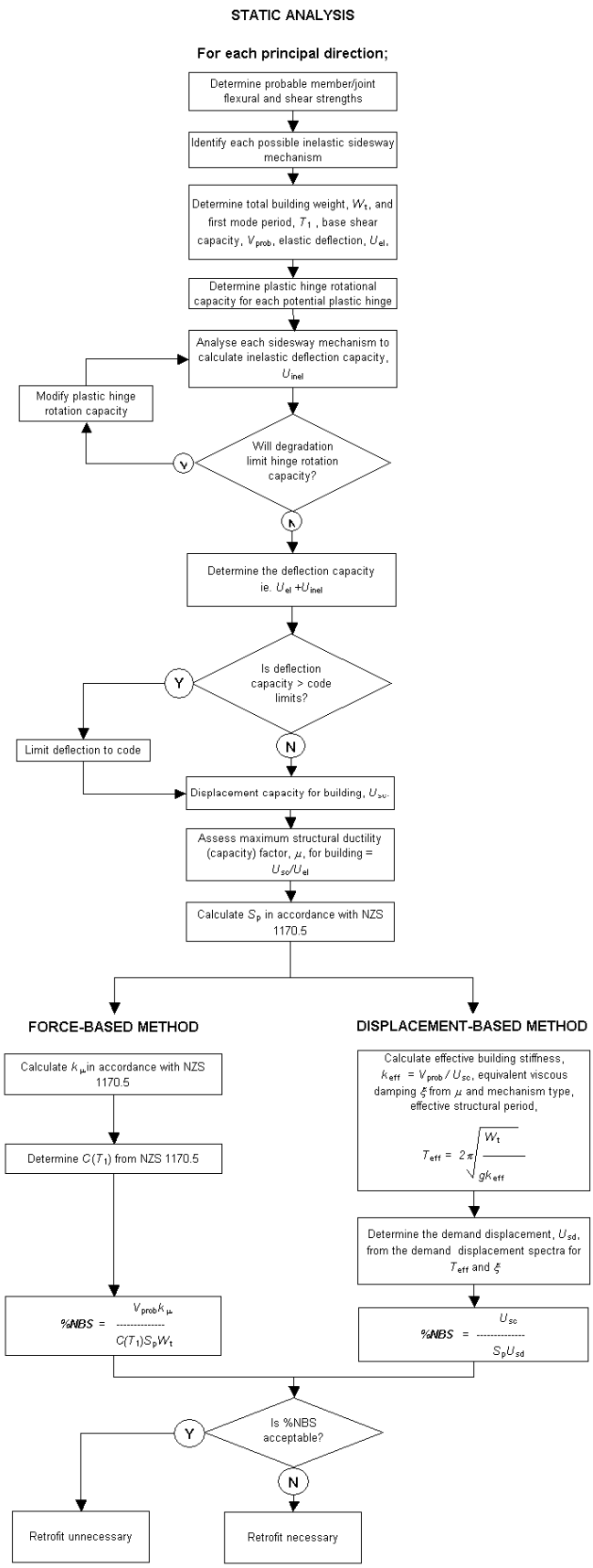


Figure 6.5 Consolidated force / displacement based assessment procedure

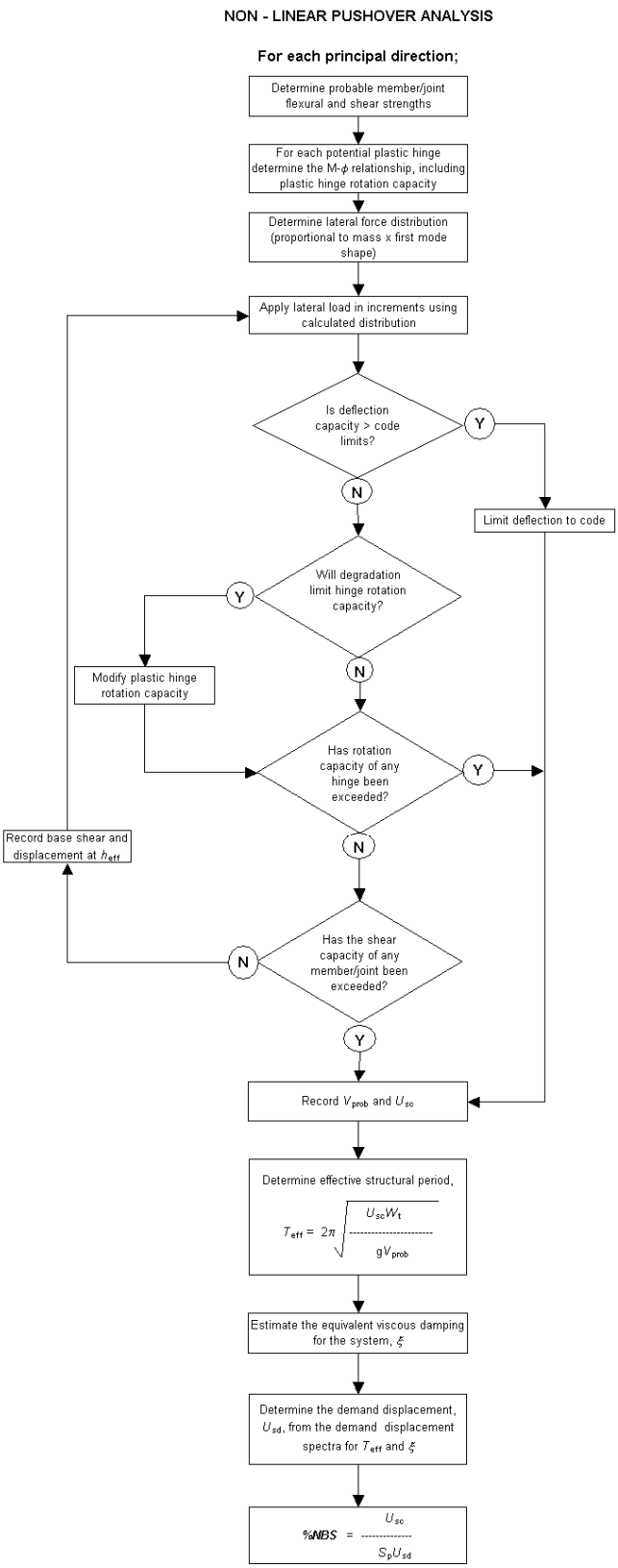


Figure 6.6 Assessment procedure using non-linear pushover analysis

Section 7 - Detailed Assessment of Reinforced Concrete Structures

7.1 Material Properties and Member Strengths

In the assessment of an existing structure, realistic values for the material properties, particularly strengths, must be used to obtain the best estimate of the strengths and displacements of members, joints and connections.

Material properties and strengths that were specified in the original design are not appropriate for use in assessment procedures.

The effect of variations in material strength on the hierarchy of failure must be considered.

The material strengths used are to be as defined below.

7.1.1 Material Strengths

General definitions of material strengths are given in Section 4.7.

The following specific notes relate to reinforced concrete.

Further specific guidance is given in the procedures presented.

a) Probable Strength, S_p

For the steel reinforcement the mean yield strength may be taken as the mean of the upper characteristic (95 percentile value) and the lower characteristic (5 percentile value). The ratio between the upper characteristic to the lower characteristic yield strength will typically be in the range 1.17 to 1.3 depending on source and age. Hence the expected mean yield strength of the reinforcing steel used currently is about 1.08 times the lower characteristic yield strength if the lower end of this range is taken.

Therefore for beams the ratio of probable flexural strength to nominal flexural strength, M_p/M_n can be taken as 1.08.

For concrete, a value of 1.5 times the nominal compressive strength should be used in the absence of more reliable information.

b) Overstrength S_o

For beams the overstrength in flexure is mainly due to the steel properties. For current New Zealand manufactured reinforcing steel, an upper bound for the actual yield strength can be taken as the upper characteristic (95 percentile value).

A further 8% increase in steel stress due to strain hardening can be assumed, (eg see paper by Andriono and Park, Bulletin NZNSEE, Vol. 19, No. 3, 1986, pp 213-246). Hence the ratio of overstrength in flexure to nominal flexural strength, M_o/M_n, can be taken as 1.25 (as is currently assumed in New Zealand for both Grade 300 and Grade 430 steel) and the ratio of overstrength in flexure to probable flexural strength, M_o/M_p, can be taken as 1.16.

2

For columns, confinement can cause a significant increase in the concrete compressive strength and hence in the flexural strength, particularly when the axial compressive load is significant, eg, NZS 3101:1995 gives the following equations for a column confined by the currently specified amount of transverse reinforcement in potential plastic hinge regions:

$$M_o = \left\{1.25 + 2\left(\frac{N^*}{f_c' A_g} - 0.1\right)^2\right\} M_n \geq 1.25 M_n \qquad ...7(1)$$

where M_o = overstrength in flexure of the column
N^* = axial compressive load on column
f'_c = concrete compressive cylinder strength
A_g = gross area of column
M_n = nominal flexural strength of column

This equation is based on University of Canterbury research (eg, see paper by Priestley and Park, ACI Structural Journal, Vol. 84, No. 1, 1987, pp 61-76).

For columns with less confining reinforcement than currently specified in potential plastic hinge regions the enhancement in flexural strength will not be so significant. However, the expected overstrength material strengths should be used to calculate the flexural overstrength.

c) Strength Reduction Factors

In the above considerations, in assessment, the strength reduction factor ø for flexure should be taken as 1.0. A strength reduction factor ø for shear of 0.85 should be built into the shear strength equations.

d) Bounds of Flexural Strength

The bounds of flexural strength are important when assessing moment resisting frames to determine the mechanism of post-elastic deformation.

For beams and columns the lower bound of flexural strength can be taken as the nominal strength, and upper bound as the overstrength.

e) Reinforcing Steel

In the absence of other information, a probable yield strength of about 300 MPa can be used in the assessment of structures reinforced by structural grade reinforcement of the 1930–70 period, this being approximately the expected mean value. Whenever practicable, samples of steel from the structure should be tested to obtain a better estimation of the expected mean yield strength of the reinforcement.

Many existing reinforced concrete structures in New Zealand were constructed using structural grade reinforcing steel with a minimum yield strength of about 227 MPa (33,000 psi); for example as specified in SANZ (1962). Subsequently the minimum yield strength was increased to 275 MPa in the amendment of NZS 1693 and in SANZ (1973). A high yield steel with minimum yield strength of 414 MPa was also available in 1964 SANZ (1964) and subsequent years. Chapman (1991) reports that it has been found by site sampling and testing that in structures built in New Zealand during the 1930–70 period the structural grade reinforcement is likely to possess a lower characteristic yield strength (5 percentile value) which is 15–20% greater than the specified value. Reinforcing steel from the pile caps of the Thorndon overbridge in Wellington constructed in the

1960s has a measured mean yield strength of 318 MPa with a standard deviation of 19MPa (Presland 1999).
Plain round bars were used in New Zealand for longitudinal reinforcement until about the mid-1960s. The development length for plain round bars is at least twice that for deformed bars SANZ (1995). Also, during cyclic loading the bond degradation for plain round bars is more significant than for deformed bars. Hence old structures reinforced by plain round longitudinal bars will show a greater reduction in stiffness during cyclic loading. In recent seismic load tests of a beam-column joint subassembly reinforced by plain round longitudinal bars at the University of Canterbury, the measured lateral displacements were approximately twice those of similar assemblies reinforced by deformed longitudinal bars at similar stages of loading (Liu and Park 1998 and 2001).

f) Concrete

In the absence of specific information a value of 1.5 times the nominal compressive strength can be used to conservatively estimate the expected compressive strength of concrete in assessment. Wherever practicable, cores should be taken from the structure to more accurately assess typical strengths. The quality of the concrete should also be inspected since if compaction was poor, a lower concrete compressive strength may need to be assumed to establish a lower bound of column strength.

The actual compressive strength of old concrete is also likely to considerably exceed the specified value as a result of conservative mix design, age and the less finely ground cement particles. Recent tests on the concrete of 30-year-old bridges in California consistently showed compressive strengths approximately twice the specified strength (Priestley 1995). Concrete from the columns of the Thorndon overbridge in Wellington has a measured compressive strength about 30 years after construction of about 2.3 times the specified value of 27.5 MPa (Park 1996). Similarly, concrete from collapsed columns of the elevated Hanshin Expressway in Kobe, Japan after the January 1995 earthquake has a measured compressive strength almost 30 years after construction of about 1.8 times the specified value of 27.5 MPa (Park 1996), Presland (1999).

In calculating member strength capacities, a strength reduction factor ø of 1.0 should be applied for flexural capacities. A strength reduction factor of 0.85 has already been built into the shear strength Equations 7(5) to 7(11). 2

7.2 Moment Resisting Frame Structures

7.2.1 Introduction

The general steps for the force and displacement-based methods outlined in Section 6, are elaborated on herein for reinforced concrete frame structures. As methods for the determination of available ductility are largely common to both procedures, they are presented separately in Section 7.2.4.

Analyses of existing moment resisting frames typical of early reinforced concrete building structures, and observations of damage caused in recent earthquakes have indicated that the major problem areas are:

a) Inadequate ductility and shear strength of potential plastic hinge regions of beams and columns due to insufficient transverse reinforcement. 2

b) Inadequate anchorage of transverse reinforcement due to poor anchorage details.

c) Inadequate shear strength of beam-column joints due to insufficient transverse reinforcement.

d) Inadequate anchorage of longitudinal reinforcement due to poor anchorage details.

e) Inadequate strength of footings and/or piles and their connections.

2 *f) Uncertain behaviour of the structure as a result of the presence of nonstructural elements, typically infill walls, which can significantly alter the structural behaviour of the frame.*

Priestley (1995), Park (1996), Rodriguez and Park (1991), Hakuto et al (1995), Park et al (1995).

Both the force-based and displacement-based procedures described in this section consider these problem areas in varying levels of detail.

The range of typical failure modes for reinforced concrete frame elements is summarised in Figure 7.1, along with a qualitative representation of possible force vs displacement outcomes.

These include column shear failure (2), column sidesway mechanisms (3) and beam sidesway mechanisms (4), which are considered in more detail in the following subsections.

It must however be emphasised that in reality mixed modes of response represent the most likely outcome. Procedures for evaluating the capacity of such mechanisms are not developed in detail due to practical limitations associated with modelling and analysis. The issue is discussed briefly
2 *subsequently (refer Step FF2 and Figures 7.3 and.7.4), as is the need to model upper and lower bound mechanism scenarios.*

Reference is also made in Figure 7.1 to column bar buckling leading to premature failure (mode (1)). When the buckling of longitudinal column bars occurs, failure develops almost immediately, and usually at a much lower level of force than that associated with the more conventionally assumed mechanisms. Consideration should be given as to the likelihood of premature failure of this type occurring in columns with a high axial load and inadequate transverse reinforcement
2 *(refer Step FF6, Method 1) before undertaking detailed numerical analysis on the more conventional possible failure modes.*

Figure 7.1 also illustrates the fundamental similarities between the force-based and displacement-based approaches, and the relationship between demand and capacity in each case.

This figure shows that a potential column sidesway mechanism with low available ductility capacity (mode (3a)) may well be considered acceptable in a situation of low force or displacement demand.

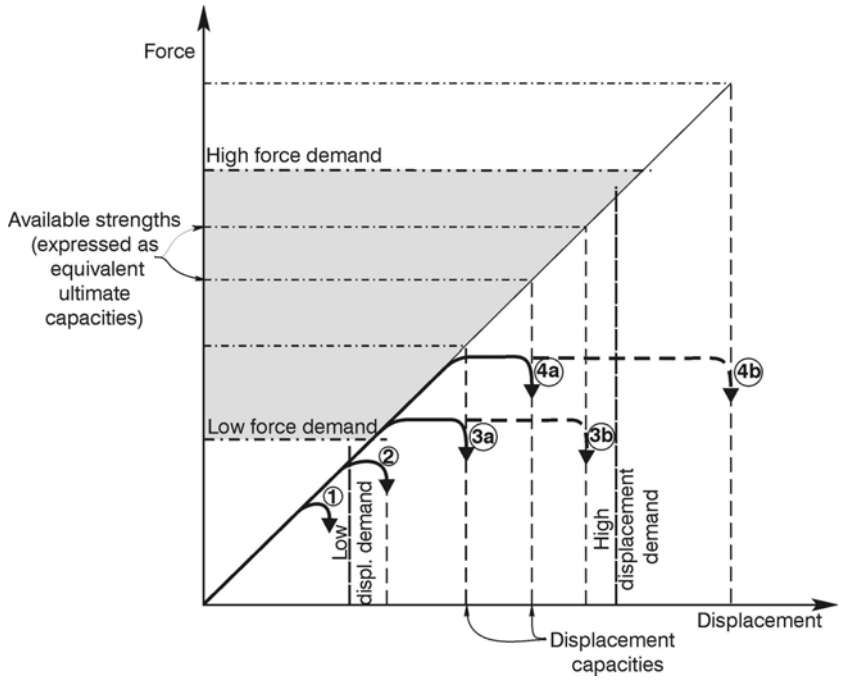


Figure 7.1: Simplified force versus displacement relationships and mechanism outcomes for reinforced concrete frames

Details of the range of possible outcomes in Figure 7.1 are:

1) *Column bar buckling (leading to premature column failure).*
2) *Column shear failure (prior to mechanism forming).*
3) *Column sidesway mechanism:*
   - *(a) Low available section ductility (due to early shear failure)*
   - *(b) High available section ductility*
4) *Beam sidesway mechanism:*
   - *(a) Low available section ductility (due to early shear failure).*
   - *(b) High available section ductility.*

Note: In between outcomes 3 and 4 are mixed mode responses.

7.2.2 Force-Based Procedure for Frame Structures

This procedure follows the steps described in Section 6.2 and shown in the flowchart of Figure 6.1.

Step FF1: Probable flexural and shear strengths

The probable flexural strength of members should be calculated using the expected material strengths and standard theory for flexural strength [Park and Paulay (1975)]. A strength reduction factor $\phi = 1.0$ may be assumed for this flexural strength calculation since the expected properties of the members as built are used.

When calculating the flexural capacity of beams in negative moment regions some of the reinforcement in cast-in-place floor slabs which are integrally built with the beams should be included with the tension top steel of the beams since that slab steel will participate in resisting negative bending moments. It is important to realistically assess the contribution of that slab reinforcement so as to properly determine the flexural strength of the beam.

A detailed assessment of the contributions of slab reinforcement to the negative moment flexural strength of a beam should take into account the width of slab within which the reinforcement can be subjected to significant tension, which is dependent on the boundary conditions of the slab and the level of imposed structure ductility. Also important is consideration of whether the bars in that slab width are adequately anchored to develop their tensile strength, which is dependent on the location of the anchorage of each bar within the slab.

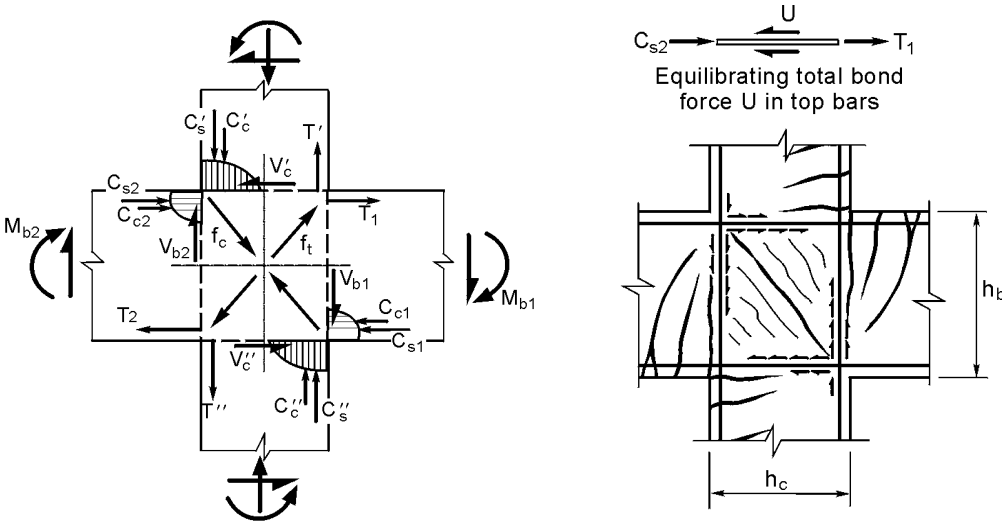
SNZ (1995) gives the following recommendations for width of slabs based on test results [Cheung et al (1991)].

> *In T- and L- beams built integrally with slabs, the slab width within which effectively anchored longitudinal slab reinforcement shall be considered to contribute to the negative moment flexural strength of the beam, in addition to those longitudinal bars placed within the web width of the beam, shall be defined as the lesser of the following criteria:*
>
> (a) One quarter of the span of the beam, extending each side as appropriate from the centre of the beam section;
>
> (b) One half of the span of the slab, transverse to the beam under consideration, extending each side as appropriate from the centre of the beam section;
>
> (c) Where the beam is in the direction at right angles to the edge of the floor and frames into an exterior column, ¼ of the span of the transverse edge beam, extending each side from the centre of the beam section;
>
> (d) Where the beam is in the direction at right angles to the edge of the floor and frames into an exterior column but no transverse edge beam is present, ½ of the column width, extending each side from the centre of the beam section.

The plastic hinges in the beams normally occur at or near the beam ends; hence the longitudinal beam reinforcement is at or near the yield strength at the column faces. This can result in high bond stresses along beam bars which pass through an interior joint core since a beam bar can be close to yield in compression at one column face and at yield in tension at the other column face (see Figure 7.2). During severe cyclic loading caused by earthquake actions, deterioration of bond may occur in the joint. If the bond deterioration is significant, the bar tension will penetrate through the joint core, and the bar tensile force will be anchored in the beam on the far side of the joint. This means that the compression steel will actually be in tension. As a result, the flexural strength and the ultimate curvature of the beam will be reduced.

Hakuto et al (1999) have analysed doubly reinforced beam sections at the face of columns of a typical building frame constructed in New Zealand in the late 1950s. The reinforcement ratios were 1.34% for the top and 0.67% for the bottom. The ratio of column depth to beam bar diameter was 12.5. The effect of stress level in the "compression" reinforcement on the moment capacity of the beam was found to be not so significant. When the bond had deteriorated to the extent that the "compression" reinforcement was at the yield strength in tension the decrease in ultimate moment was up to 10% for positive moment and up to 5% for negative moment compared with those with perfect bond along the beam bars (Hakuto et al 1999). It is evident that the effect of bar slip on flexural strength of beams could be neglected in assessment since it is unlikely that there will be a total loss of bond unless plain round bars are present.

Consideration of the bounds of flexural strength of beams and columns is important when assessing moment resisting frames to determine whether plastic hinging will occur in the beams or columns in the mechanism of post-elastic deformation. The range of expected material strengths should be considered when estimating maximum and minimum likely expected flexural strengths.



(a): Forces from beams and columns acting on the joint

(b): Crack pattern and bond forces after diagonal tension cracking initiates in joint core

Figure 7.2: Interior beam – column joint subjected to seismic loading

Bond slip of lap splice connections

Lap splice connections often occur at the base of columns in frames, particularly in older structures with non-ductile concrete frames. Providing the lap length is sufficient to develop yield ($20d_b$ *for deformed bars*) then the nominal ultimate strength capacity can be attained. However, post-elastic deformations quickly degrade the bond strength capacity and within one inelastic cyclic of loading, the lap splice may be assumed to have failed. This will be evident if longitudinal (tensile) splitting cracks are noticed at the base of the columns.

When the lap splice fails in bond, it does not generally lead to a catastrophic failure as the column is still able to transfer moment due to the presence of the eccentric compression stress block that arises as a result of the axial load in the column.

Thus the moment capacity of a lap splice (M_{lap}) may be determined based on the initial full moment capacity (M_n) and a final moment capacity (M_f) as follows:

$$M_{lap} = M_n - \frac{\theta_p}{0.025}\left(M_n - M_f\right) \qquad \ldots 7(2)$$

where $M_f \leq M_{lap} \leq M_n$ and θ_p = plastic rotation demand on the connection; and M_f is taken as the greater of:

$$M_f = \frac{l_{lap}}{l_d} M_n \qquad \ldots 7(3)$$

and $M_f = 0.5N(D - a)$ …7(4)

where l_{lap} = provided lap length; l_d = theoretical development length; D = the overall width of the member; and a = depth of the compression stress block.

In order to calculate the flexural and shear capacities of the columns, assumptions regarding the earthquake induced axial forces are required. In many cases for multi-bay frames, the earthquake induced axial forces are not significant in comparison to gravity actions. In order to avoid running a frame analysis at this early stage (ie before the available displacement ductility is ascertained),

for critical corner columns the beam shear capacities from the end spans can be summed and factored by R_v from Appendix A of SNZ (1995) as an initial approximation.

Shear strength of members and beam - column joints

The expected shear strength of members and interior and exterior beam-column joints should be calculated using the expected material strengths and theory for the shear strength of members and beam-column joints not undergoing cyclic deformations in the post-elastic range. The effect of the degradation of shear strength due to post-elastic cyclic deformations is considered in Step FF7. A strength reduction factor ϕ = 0.85 has already been built into the shear strength equations since although the probable properties of the members and joints as built are used, the theory is less exact.

Shear strength of beams

The probable shear strength of beams without plastic hinging with rectangular stirrups or hoops is given by:

$$V_p = 0.85(v_c b_w d + A_v f_{yt} d/s)$$

$$= 0.85(k\sqrt{f_c'}b_w d + A_v f_{yt} d / s) \qquad \ldots 7(5)$$

where
- v_c = nominal shear stress carried by the concrete mechanisms
- f_c' = expected concrete compressive strength
- b_w = width of beam web
- d = effective depth of beam
- A_v = area of transverse shear reinforcement at spacing s
- f_{yt} = expected yield strength of the shear reinforcement
- k = 0.2.

This equation assumes that the critical diagonal tension crack is inclined at 45° to the longitudinal axis of the beam.

In the non-seismic provisions of SNZ (1995) k for beams is given as (0.07 + 10 p_w), where $p_w = A_s/b_w d$ and A_s = area of tension reinforcement. SNZ (1995) requires that k so determined be not more than 0.2, nor need it be less than 0.08. The SNZ (1995) equation is a conservative estimate. On the basis of test results, both Hakuto et al (1995) and Priestley (1995) suggest that k = 0.2 could be assumed for beams without plastic hinging. Note that k = 0.2 is conservative for high longitudinal steel contents.

Shear strength of columns

The probable shear strength of columns without plastic hinging can be taken as:

$$V_p = 0.72\,(V_c + V_s + V_n) \qquad \ldots 7(6)$$

where V_c is the shear resisted by the concrete mechanisms and given by:

$$V_c = v_c\, 0.8\, A_g$$

$$= k\sqrt{f_c'}\,0.8 A_g \qquad \ldots 7(7)$$

where $k = 0.29\alpha\ \beta$, v_c = nominal shear stress carried by the concrete mechanisms, A_g = gross area of the column, and $1 \le \alpha = 3\text{- M/VD} \le 1.5$; and $\beta = 0.5 + 20\rho_l \le 1.0$ where M/V is the ratio of moment to shear at the section, D is the total section depth or the column diameter as appropriate and ρ_l is the area of longitudinal column reinforcement divided by the column cross-sectional area. 2

In eqn 7(6), V_s is the shear resisted by the shear reinforcement assuming that the critical diagonal tension crack is inclined at 30° to the longitudinal axis of the column. For rectangular hoops:

$$V_s = \frac{A_v f_{yt} d''}{s} \cot 30^\circ \qquad \ldots 7(8)$$

and for spirals or circular hoops:

$$V_s = \frac{\pi}{2} \frac{A_{sp} f_{yt} d''}{s} \cot 30^\circ \qquad \ldots 7(9)$$

where A_v = total effective area of hoops and cross ties in the direction of the shearforce at spacing s

A_{sp} = area of spiral or circular hoop bar

f_{yt} = expected yield strength of the transverse reinforcement

d'' = depth of the concrete core of the column measured in the direction of the shear force for rectangular hoops and the diameter of the concrete core for spirals or circular hoops.

In eqn 7(6), V_n is the shear resisted as a result of the axial compressive load N^* on the column and is given by:

$$V_n = N^* \tan \propto \qquad \ldots 7(10)$$

where for a cantilever column $\propto$ is the angle between the longitudinal axis of the column and the straight line between the centroid of the column section at the top and the centroid of the concrete compression force of the column section at the base, and for a column in double curvature $\propto$ is the angle between the longitudinal axis of the column and the straight line between the centroids of the concrete compressive forces of the column section at the top and bottom of the column.

The shear strength of columns given by SNZ (1995) is also a conservative estimate. Priestley (1995) and Priestley et al (1994) suggest the above approach as a result of extensive testing. In Equation 7(6), Priestley's suggested values for $V_c + V_s + V_n$ have been multiplied by 0.85 to obtain a closer estimate of the lower bound of his test data and a strength reduction factor of $\phi = 0.85$ for shear has also been applied in addition to this factor.

Shear strength of beam-column joints

For interior and exterior beam-column joints without shear reinforcement, the probable horizontal joint shear force that can be resisted is:

$$\begin{aligned} V_{pjh} &= 0.85 v_{ch} b_j h \\ &= 0.85 k \sqrt{f'} \sqrt{1 + \frac{N^*}{A_g k \sqrt{f'_c}}}\, b_j h \ \le\ 1.92 \sqrt{f'_c}\, b_j h \end{aligned} \qquad \ldots 7(11)$$ 2

where v_{ch} = nominal horizontal joint shear stress carried by a diagonal compressive strut mechanism crossing the joint

b_j	=	effective width of the joint (being normally the column width SNZ (1995)
h	=	depth of column.

It is proposed that the following values for k be used:

- for interior joints, $k = 1.0$
- for exterior joints with beam longitudinal bars anchored by bending the hooks into the joint core, $k = 0.4$
- for exterior joints with beam longitudinal bars anchored by bending the hooks away from the joint core(into the columns above and below), $k = 0.25$.

For beam-column joints without any, or insignificant, shear reinforcement in the joint core and relatively low joint shear stress, SNZ (1995) is quite conservative, particularly if there are no plastic hinges undergoing cyclic deformations in the post-elastic range adjacent to the joint core. Analysis of the test results of Hakuto et al (1995) and those of other researchers suggest the above relationship.

The above recommended values for k are based on the estimated maximum nominal horizontal joint core shear stress, calculated the conventional way, resisted by beam-column joints in tests without joint shear reinforcement and without axial load. The term indicating the influence of axial load, $\sqrt{1+N^*/A_g k\sqrt{f_c'}}$ *was obtained by assuming that the diagonal tensile strength of the concrete was* $k\sqrt{f_c'}$ *and calculating using Mohr's circle for stress the horizontal shear stress required to induce this diagonal (principal) tensile stress when the vertical compressive stress is* N^*/A_g. *[Hakuto et al (2000)]. The above recommended values for k are based on very limited experimental evidence. Further tests are badly needed to improve the accuracy of the assessment of beam-column joints without, or with little, shear reinforcement. A strength reduction factor of 0.85 has been included in eqn 7(11).*

2

Step FF2: The post-elastic mechanisms of the frame and the probable lateral seismic force capacity

Having determined the probable flexural and shear strengths of the members and joints of the frame, the next step in the assessment procedure is to identify the probable location of post-elastic deformations due to severe earthquake forces and hence to determine the critical mechanism of post-elastic deformation.

This will involve determining whether flexural plastic hinges occur in the beams or the columns at each beam-column joint and/or whether shear failure occurs in the members or joints. The imposed shear forces on members should be those associated with the plastic hinge (flexural) mechanism. The imposed horizontal shear forces on beam-column joint cores should be those associated with the adjacent plastic hinges. The horizontal joint shear force is given conventionally by the sum of the tensile forces in the top and bottom longitudinal beam reinforcement minus the column shear force. Comparisons of these calculated imposed shear forces and the expected shear strengths found in Step FF1 will determine whether shear failures occur before the flexural strengths are reached or not.

2

The lateral seismic force capacity associated with the critical mechanism of post-elastic deformation can then be calculated.

Often for a building frame the critical mechanism is not simply a beam sidesway mechanism or a column sidesway mechanism (see Figures 7.3(a) and (b)), but is a mixed mechanism involving flexural plastic hinges at some locations combined with shear failures of members and/or joints at other locations (for example, see Figure 7.3(c)).

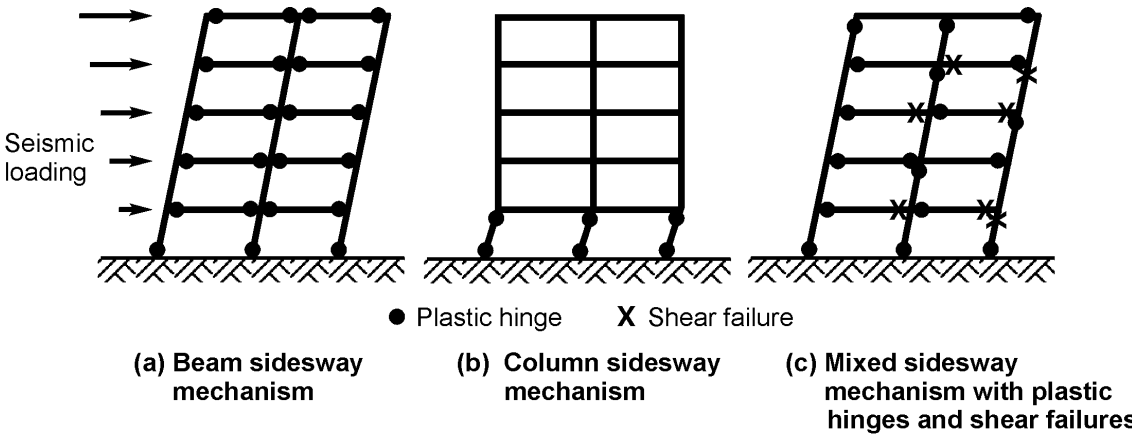


Figure 7.3: Possible mechanisms of post-elastic deformation of moment resisting frames

To investigate whether plastic hinges occur in beams or columns, a sway potential index S_i can be defined for the beam-column joints at a horizontal level by comparing the sum of the expected flexural strengths of the beams and the columns at the joint centroids:

$$S_i = \frac{\Sigma(M_{bl} + M_{br})}{\Sigma(M_{ca} + M_{cb})} \qquad ...7(12)$$

where M_{bl}, M_{br} = beam expected maximum flexural strengths at the left and right of the joint, respectively, at the joint centroid, and M_{ca} and M_{cb} = minimum expected column flexural strengths above and below the joint, respectively, at the centroid of the joint. These are summed for all the joints at that horizontal level.

Lap splice connections often occur at the base of columns in moment resisting frames, particularly in older structures with non-ductile frames.

Equations 7(2) to 7(4) can be used to determine the moment capacity of a column with lap splices. | 2

When $S_i > 1$, column plastic hinges may be expected to form. However, to include the effects of higher modes of vibration, and a possible overestimation of column flexural strength it is suggested [Priestley (1995)] that it be assumed that column plastic hinges form if $S_i > 0.85$. Accordingly, the dynamic magnification factor need not be applied in this procedure.

The use of the dynamic magnification factor, ω_v, in the capacity design of new columns is intended to completely avoid the possibility of column hinge formation. Less conservative measures are appropriate if individual column hinging can be accepted, provided that a full storey column sidesway mechanism does not develop.

A common case for older frames may be the mechanism of post-elastic deformation shown in Figure 7.4. This mechanism has plastic hinges in beams forming only at the faces of exterior columns and plastic hinges forming at the top and bottom of the interior columns and at the column bases. This typically arises as a result of the design gravity loading requiring beams with relatively high flexural strengths and the flexural strengths of the interior columns being relatively weak. This mechanism is common for gravity load dominated frames.

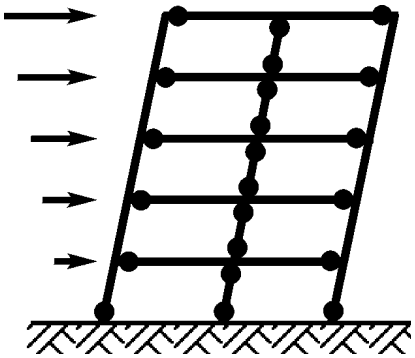


Figure 7.4: Mixed sidesway mechanism of gravity load dominated frames

The typical lateral force-displacement relationship for a moment-resisting frame is represented in Figure 7.5 (Park 1996). This relationship assumes that the probable lateral seismic force capacity of the frame is dependent on the probable flexural strength of members.

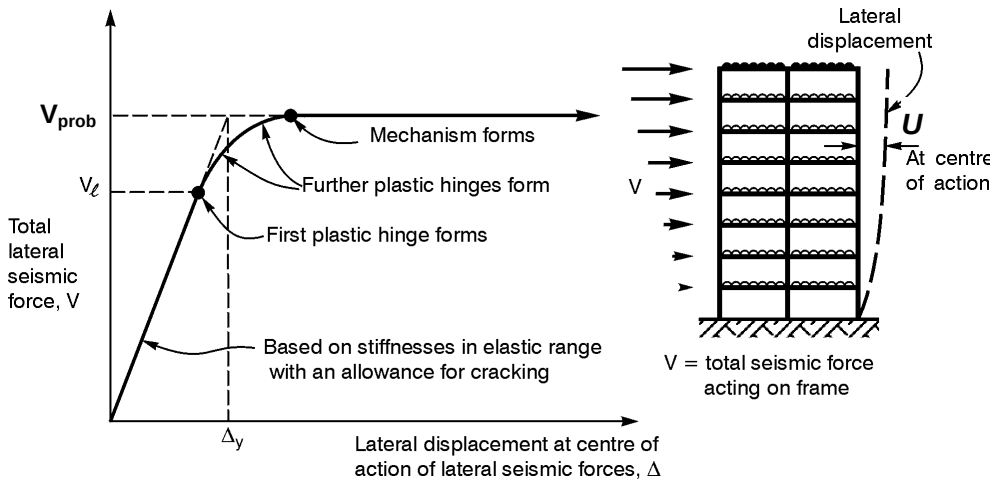


Figure 7.5: Typical lateral force-displacement relation of a moment resisting frame

The lateral capacity of the frame can be found by any one of the following three methods (Park 1996).

Method 1

Linear elastic structural analysis may be used to determine the distribution of bending moments due to lateral seismic forces and gravity loading. In this analysis, to make allowance for concrete cracking, the effective second moments of area of beams and columns can be assumed to be as in Table C3.1 of the commentary on SNZ (1995). In the analysis the equivalent static earthquake forces are increased from zero until the first plastic hinge forms. The lateral seismic force corresponding to the development of the first plastic hinge gives a lower bound to the probable lateral force capacity of the frame (i.e. V_l in Figure 7.5). This lower bound estimate based on the bending moment diagram will always be equal to or less than the actual lateral force capacity. In reality, moment redistribution will permit higher lateral seismic forces to be resisted while further plastic hinges form until a mechanism develops or local failure point is reached.

Method 2

If the mechanism of post-elastic deformation is obvious from the onset, the lateral seismic force corresponding to the mechanism condition can be calculated directly (SLaMA). For example, a

column sidesway mechanism will occur in the bottom storey of the frame if S_i given by Equation 7(12) is greater than 0.85 at the beam-column joints of that storey. In that case the probable lateral force capacity of the frame is given by the sum of the shear forces in the columns of that storey, found from the sum of the probable flexural strengths of the plastic hinges at the top and bottom of the columns of that storey divided by the storey height. This estimate gives an upper bound to the probable lateral force capacity of the frame and will be always equal to or greater than the actual lateral force capacity (ie V_{prob} in Figure 7.5).

The danger of calculating the expected lateral force capacity by the upper bound approach is that the correct mechanism may be missed and the lateral force capacity overestimated as a result. The mechanism giving the least lateral force capacity is the correct one and must be sought.

Method 3

The non-linear lateral pushover structural analysis (LPA) is arguably the most useful method of analysis as mechanisms will be identified. Using LPA the lateral seismic forces acting on the frame are gradually increased until a mechanism forms. The behaviour of the frame is in the elastic range until the first plastic hinge forms and then the post-elastic deformations at the plastic hinges need to be taken into account. The number of plastic hinges forming increases with increase in lateral force until a mechanism develops, giving the actual probable lateral force capacity (ie V_{prob} in Figure 7.5).

A difficulty with the pushover analysis is that a static representation of the distribution of the seismic forces acting on the frame is required. Conventionally an inverted triangular distribution of lateral seismic forces up to the height of the frame could be assumed, but this distribution takes no account of higher mode effects. A sensitivity analysis may need to be conducted assessing the differences in lateral force capacity V of the frame arising from different distributions of seismic load; for example, uniform up the height. This will be of particular interest for taller structures when higher modes will become important. The lateral load distribution obtained from a modal analysis can provide some allowance for higher modes but will only be completely valid while the structure remains predominantly in the elastic range.

A computer program available in New Zealand which is capable of non-linear pushover analysis is RUAUMOKO [Carr (2005)].

Step FF3: Determination of the period of vibration of the structure, total seismic weight and structural performance factor

The fundamental period of vibration of the structure should be calculated including the effect of cracking on the section properties. Table C3.1 of the commentary on SANZ (1995) gives estimates of the effective second moments of area of beams and columns which include the effect of cracking. It is to be noted that the estimates in Table C3.1 are generally on the high side. Also, frames with poorly detailed beam-column joints may undergo a significant reduction in stiffness due to diagonal tension cracking of joints and bond slip of longitudinal bars passing through the joints.

For example, Hakuto et al (1995) tested a poorly detailed beam-column joint which modelled an actual 1950s design but used deformed bar reinforcement and was without axial load on the column. It was found that, after two or three lateral load cycles to about 70% of the yield displacement, to obtain agreement with the measured frame displacements, the displacements needed to be calculated using effective second moments of area of about 0.3 of the gross second moment of area, by using clear spans of members and by multiplying the member contributions due to flexure and shear by 1.2 to account for the additional shear deformation of, and bond slip in, the beam-column joint.

The approximate period calculation given in the commentary to NZS 1170.5 and used in the IEP should not be relied on for a detailed assessment.

The choice of the structural performance factor, S_p, should be appropriate for the detailing used in the structure.

Step FF4: Determination of the implied inelastic spectrum scaling factor

The implied inelastic spectrum scaling factor is found from eqn 6(1) after first deciding the value of the targeted percentage new building standard, $(\%NBS)_t$.

Step FF5: Determination of the required structure ductility factor

Having estimated the implied inelastic spectrum scaling factor, $k\mu$, the required structure ductility factor μ can then be estimated using the equations given in Section 5 NZS 1170.5. Note that use of $\mu > 6$ is not permitted by SNZ (1995). Note that the useable value of μ *may be limited by the*
2 | *permitted interstorey drift (see Step FF8)*

Step FF6: Assessment of whether the plastic hinges have sufficient available ductility to match the required structure ductility

This step involves estimating the likely plastic hinge rotations and/or section ductilities associated with the required structure (displacement) ductility factor μ and checking whether the plastic hinges have sufficient ductility to match that demand. If sufficient rotation capacity at the plastic
2 | hinges is available, then, subject to a satisfactory shear and bond check in Step FF7, the frame does not need to be retrofitted. If sufficient rotation capacity at the plastic hinges is not available, the frame will need to be retrofitted or the target *%NBS* reduced.

The required structure displacement ductility factor μ is given by U_{sd}/U_{el}, where U_{sc} is the maximum required lateral displacement and U_{el} is the yield displacement which can be defined as shown in Figure 7.5.

Any one of three static methods may be used to check the rotation capacity of the plastic hinges, increasing in sophistication from Method 1 to Method 3. In Methods 1, 2 and 3 the available μ is estimated based on the rotational capacity of the plastic hinges and then compared against the
2 | required μ determined in Step FF5.

Method 1

For potential plastic hinge regions in beams of frames where a beam sidesway mechanism is shown to be likely [Priestley 1995]:

- where the stirrups are effectively anchored and the stirrup spacing satisfies $s \leq d/2$ and $s \leq 6d_b$, an available structural ductility factor of $\mu = 6$ may be assumed for the frame, where d = effective depth of beam and d_b = diameter of longitudinal bars
- where the stirrups are not effectively anchored and/or $s > d/2$ or $s > 16d_b$, then an available $\mu = 2$ only may be assumed
- intermediate values of μ may be estimated according to the existing detailing of the members based on the above.

For potential plastic hinge regions at the base of columns where a beam sidesway mechanism is shown to be likely, or for frames of one or two storeys in height, where a column sidesway mechanism is likely [Park 1992]:

- where the hoops are effectively anchored and hoop or spiral spacing satisfies $s \leq d/4$ and $s \leq 6d_b$, and where the ratio of volume of transverse reinforcement/volume of concrete core $\geq 0.01\ \{1 + (2N^*/0.7\ f_c' A_g)\}$ and where the confined length of column at the column base $\geq h\ \{1 + (2N^*/0.7\ f_c' A_g)\}$, then an available structure ductility factor of $\mu = 6$ may be assumed for the frame, where N^* = axial compressive load on the column, f_c' = expected compressive cylinder strength of the concrete, and A_g = gross area of the column
- where either the hoops are not effectively anchored or $s > d/2$ or $s > 16d_b$, then an available $\mu = 2$ only may be assumed.
- where the bottom longitudinal beam bars are lapped in the potential plastic hinge regions as was common in older frames, then an available structure ductility factor of μ *= 2 may be assumed if the bars are deformed or* μ *= 1.25 if the bars are plain round [Wallace 1996].*

For potential plastic hinge regions in columns of frames of more than two storeys in height where a column sidesway mechanism is likely, very high plastic hinge rotations can be required of the critical column regions. An available μ of 1.5 should be assumed unless a more detailed analysis (see Methods 2 and 3) is conducted.

Method 2

A more accurate approach would be to determine the available structure (displacement) ductility factor from the mechanism. In this approach the first step is to determine the available curvature ductility factor ϕ_u/ϕ_y or plastic rotation capacity $(\phi_u - \phi_y)L_p$ at the plastic hinges taking into account the amount of confining reinforcement present, where ϕ_u = available ultimate curvature, ϕ_y = curvature at first yield and L_p = equivalent plastic hinge length. Methods for deriving ϕ_u, ϕ_y and L_p are outlined in Section 7.2.4. Then the critical mechanism is determined and the available structure (displacement) ductility factor is found by pushing the mechanism laterally until the ductility at the critical plastic hinge is exhausted.

It should also be noted that for columns the commentary of SNZ (1995) gives Equations C8.4 and C8.5 for the available ϕ_u/ϕ_y of heavily loaded columns in terms of the content of confining reinforcement and the other column variables. Those two equations could also be used to check the available ϕ_u/ϕ_y of columns.

Determining the available μ from the mechanism by pushing the mechanism laterally until the critical available ultimate curvature ϕ_u is reached is a simplification since, firstly, not all plastic hinges in the mechanism form simultaneously (see Figure 7.5), and secondly, the vertical profile of horizontal displacement of the frame needs to account for the effects of the higher modes of vibration and the type of mechanism that develops. That is, the drift (lateral displacement of a storey divided by the storey height) and the type of mechanism that develops will not be the same for each storey. However, a good approximation for the available μ may be found from the mechanism (see Section 7.2.4).

Method 3

The most complete static approach for determining the available structure (displacement) ductility factor μ is to use a nonlinear lateral pushover structural analysis (LPA) in which the lateral seismic forces on the frame are gradually increased. As the frame is pushed beyond the elastic range the number of plastic hinges forming increases with increase in lateral force until a mechanism develops. The frame is then pushed further, deforming as a mechanism, until the available ultimate curvature is reached at the critical plastic hinge. The available structural (displacement) ductility factor is then determined from that ultimate displacement (see Section 7.2.4).

Step FF7: Effect of ductility demand on the shear strength of beams, columns and their joints and bond strength

The shear strength of beams and columns in plastic hinge regions, and of beam-column joints when plastic hinging occurs adjacent to the joint, depend on the level of the imposed ductility. Hence a mechanism which initiates with flexural plastic hinges may degenerate into plastic hinges with shear failure as the ductility demand increases. Column shear failure is very serious since it could lead to total catastrophic collapse of the structure. Joint shear failure is less likely to cause catastrophic collapse but will result in extreme softening of the frame.

The required structure (displacement) ductility factor μ_{sd} found in Step FF5 was calculated using the flexural and shear strengths determined in Step FF1, which assumes that no degradation of strength occurs due to cyclic lateral loading in the post-elastic range. Degradation of shear strength may reduce the lateral force capacity of the frame and its effect should be checked. Having determined the available curvature ductility factors ϕ_u/ϕ_y in Step FF6, the next step is to determine the resulting shear strength at that ϕ_u/ϕ_y value.

Hopefully the reduced shear strengths will not reduce the lateral force capacity of the frame. However, if the reduced shear strengths are found to be less than the shear forces and the flexural strengths at the plastic hinges and/or beam-column joints for a base shear of V_{prob}, the frame will need to be retrofitted (see Figure 7.6) or the target *%NBS* reduced.

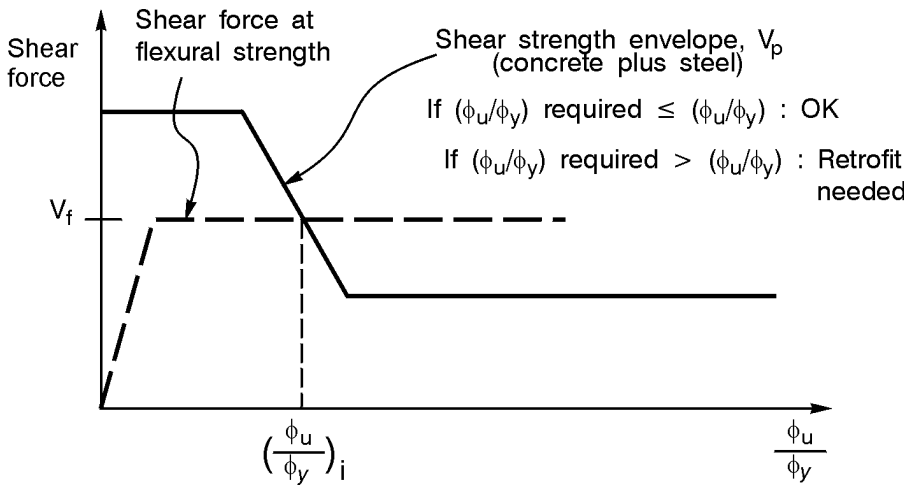


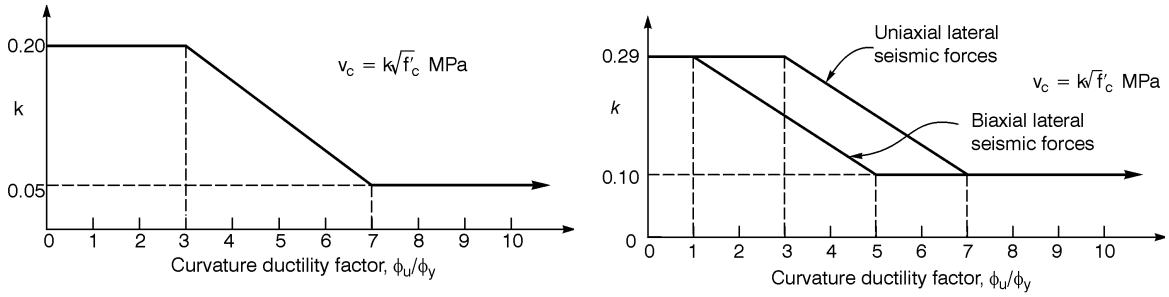
Figure 7.6: Shear strength capacity as affected by flexure and shear interaction

Also, the strength of lap splices in longitudinal reinforcement in plastic hinge regions, and the bond strength of poorly anchored bars passing through beam-column joints, will tend to degrade during imposed cyclic loading in the post-elastic range. An available structural ductility factor of greater than 2 cannot be assumed if lap splices in deformed longitudinal reinforcement exist in plastic hinge regions, unless they are heavily confined. If plain round longitudinal bars are lapped the available structure ductility factor should be taken as 1.0 [Wallace 1996].

Degradation of shear strength of beams and columns

The degradation of the shear strength in plastic hinge regions is due to the reduction of the nominal shear stress v_c resisted by the concrete mechanisms. The nominal shear stress which can be resisted

reduces with increase in ductility imposed by cyclic loading. Figure 7.7 shows proposals for the degradation of the nominal shear stress carried by the concrete, $k\sqrt{f_c'}$ MPa, of beams and columns, as proposed for beams by Priestley (1995) and by Priestley et al (1994), and by consideration of the tests by Hakuto et al (1995), and as proposed for columns by Priestley et al (1994), expressed in terms of the imposed ductility factor ϕ_u/ϕ_y. The probable shear strength is given by Equations 7(5) to 7(7) with the appropriate values of k substituted. The value of v_c is as given in Step FF1 when the imposed curvature ductility factor is zero, reducing linearly during the range of curvature ductility factors shown in the Figure 7.7, and then finally maintaining a residual value. The value of v_c given in Figure 7.7(b) is reduced by multiplying by 0.85 in Equation 7(6). The difference between the magnitudes of the shear resisted by the concrete mechanisms for beams and columns is attributed to the distributed longitudinal reinforcement of columns. Further test evidence is needed, particularly for beams. | 2



(a): Beams **(b): Columns**

Figure 7.7: Degradation of nominal shear stress resisted by the concrete with imposed cyclic curvature ductility factor

Degradation of shear strength of beam-column joints

The nominal horizontal joint shear resisted by the concrete diagonal compression strut crossing the joint core has been found experimentally to reduce with increase in ductility adjacent in plastic hinge regions imposed by cyclic loading [Hakuto et al (1995) and Priestley (1995)]. Figure 7.8 shows the degradation in k proposed. The probable horizontal shear force that can be resisted is given by Equation 7(11) with the appropriate value of k substituted. The value of k is as given by Step FF1 when the curvature ductility factor is zero, reducing linearly during the range of curvature ductility factors shown in the Figure 7.8, and then finally maintaining a residual value. It is to be noted that interior joints are not as vulnerable as exterior joints. Exterior joints with the 90° hooks at the end of the longitudinal beam bars bent away from the joint core (that is, the ends of the top bars are bent up and the ends of the bottom bars are bent down) do not perform well because the beam bar hooks do not properly engage the corner to corner diagonal compression strut [Hakuto et al (1995)]. | 2

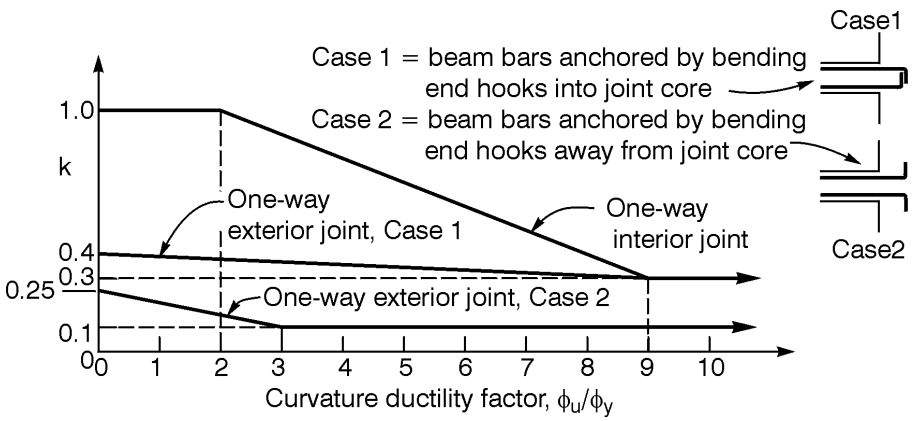


Figure 7.8: Degradation of nominal shear stress resisted by the concrete of beam-column joints with imposed cyclic curvature ductility factor

The values of k in Figure 7.8 are for one-way frames with deformed longitudinal bars and are expected to be conservative for two-way frames. The test evidence on which Figure 7.8 is based is limited. The k = 1.0 for interior joints at low ductility was suggested by Hakuto et al (1995) on the basis of the results of five beam-column joints without joint shear reinforcement tested by five separate investigators in New Zealand, USA and Japan. The k = 0.4 for exterior joints, with beam bar hooks turned into the joint core, at low ductility was suggested by Priestley (1995). A test conducted by Hakuto et al (2000) on this type of joint without shear reinforcement reached k = 0.31 and maintained it during beam plastic hinging up to large ductility factors. A higher k was not reached in this test since the maximum joint shear was governed by the amount of beam reinforcement. The k = 0.25 for exterior joints, with beam bar hooks turned away from the joint core, at low ductility was based on the maximum value for k reached in a test conducted by Hakuto et al (1995). In this test the value of k was found to degrade rapidly down to about one-half of the initial value with imposed ductility.

With regard to interior joints, with deformed longitudinal bars, when the column depth h to beam bar diameter d_b is less than the value specified in SNZ (1995) poor bond performance would be expected. The resulting bar slip would reduce the stiffness of the frame but possibly aid the shear transfer in the beam-column joint [(Hakuto et al 1995; Priestley 1995]. However, two interior beam-column joints without joint shear reinforcement have been tested by Hakuto et al (1995), one with h/d_b = 25 and f_c' = 53 MPa satisfying the requirement of SNZ (1995), and the other with h/d_b = 19 and f_c' = 33 MPa not satisfying the requirement of SNZ (1995). Although slip commenced at a lower ductility factor for the unit with the lower h/d_b ratio, the lateral load versus lateral displacement hysteresis loops for the two units were almost identical for cyclic displacements up to a displacement ductility factor of 6. The maximum horizontal joint shear stresses were $0.47\sqrt{f_c'}$ and $0.6\sqrt{f_c'}$ MPa for the two units. Again, further test evidence is required to improve the accuracy of Figure 7.8.

Mixed sidesway mechanisms

Combinations of beam and column plastic hinges and shear failures make up a variety of possible mixed sidesway mechanisms. As an example, Figure.7.9 shows a line of beam-column joints when beam plastic hinges, with available μ of 6, form except for one beam end where a flexure/ shear failure is predicted with an available μ of 3.

A conservative approach would be to assume the lower bound of μ = 3 for the whole mechanism. However, if it can be assessed that gravity loads can be carried at higher ductilities, it would be reasonable to ignore span 3-4 entirely and to assess the strength on the basis of spans 1-2 and 5-6

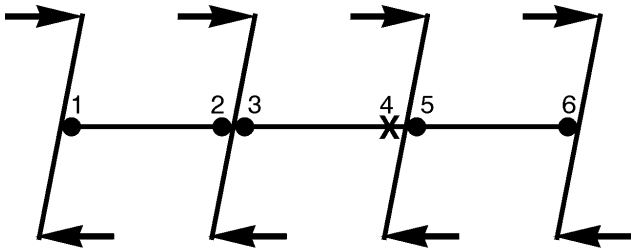
alone. Assuming that the equivalent elastic response strength $S_a(e)$ is proportional to the available μ multiplied by the sum of the flexural strengths, then if $\mu = 3$ for all six plastic hinges:

$S_a(e)$ is proportional to $3 \times 6 \times M_f = 18\ M_f$

and if $\mu = 6$ for only four plastic hinges:

$S_a(e)$ is proportional to $6 \times 4 \times M_f = 24\ M_f$

where M_f is the flexural strength of each beam plastic hinge Priestley (1995). That is, this assumption which is equivalent to removing beam 3-4 from the mechanism, results in a 33% increase in calculated mechanism capacity.



Source: Priestley 1995.

Figure 7.9: Mixed sidesway mechanism for a storey

Step FF8 Check interstorey drift

The interstorey drift of the storeys involving the critical structural element(s) should be checked to ensure that it is not so large as to introduce significant P–Δ effects or to damage non-structural elements. The structure should be stiff enough to satisfy the drift limitations of NZS 1170.5. For the estimation of storey drifts associated with nominal yield rotations of beams and columns, reference may be made to Priestley (1988).

7.2.3 Displacement-Based Procedure for Frame Structures

This procedure follows the steps described in Section 6.3 and shown in the flowchart of Figure 6.3.

Step FD1: Probable flexural and shear strengths

The probable flexural strength of members should be calculated using the probable material strengths and standard theory for flexural strength unless noted otherwise. A strength reduction factor $\phi = 1.0$ may be assumed for the flexural strength calculation since either the probable properties of the members as built or established default values are used.

Comments made regarding beam flexural strengths and earthquake-induced axial forces to use for determining column strengths under Step FF1 in Section 7.2.2 are also applicable for the displacement-based approach.

Calculate the probable shear strength of the beams using Equation 7(6), and the probable column shear strengths using Equations 7(6) to 7(10) (from Step FF1 above).

Step FD2: Post-elastic mechanisms of the frame and probable lateral force capacity

Calculate the sway potential index at each level (refer to Equation 7(12) and Figure 7.3), and establish the likely post-elastic mechanism of the frame.

Step FD3: Member plastic hinge rotation capacity

Calculate the available plastic hinge rotation capacities (see Section 7.2.4).

Step FD4: Shear strength and storey drift checks

Shear strengths of members and joints are then checked to determine whether shear failure will occur before the limits to flexural plastic rotation are reached. The available plastic rotation capacity is reduced, if necessary, to the value pertaining at shear failure.

Establish the available curvature ductility of the member at which shear failure can occur by applying the degradation models of Figures 7.6 and 7.7 (Step FF6). Compare these against the
2 curvature ductilities of the member indicated from the moment-curvature analysis of Step FD3.

Similarly, determine the beam-column joint shear strengths using Equation 7(11) and the degrading capacity model from Figure 7.8.

Using the limiting member curvature ductilities, evaluate the plastic rotation capacities of members in each storey and hence estimate the plastic storey drift capacity (see Section 7.2.4).

Step FD5: Structure displacement and ductility capacity

The overall structure displacement capacity, U_{sc}, and ductility capacity, μ_{sc}, are found from the mechanism of plastic deformation established in Step FD2, and the critical storey drift from Step FD4. For structures that are significantly unsymmetrical in plan, the effect of torsion on the displacement of a frame should be taken into account. U_{sc} is evaluated at h_{eff} (refer section 6.3)

Step FD6: Substitute structure characteristics

Response can be considered directly in terms of displacement, using the substitute-structure approach outlined in Step D6, section 6.3.

Estimates of the equivalent viscous damping available are given in section 6.3. The level of damping assumed depends on the structural ductility demand, μ_{sd}, the expected shape of the hysteresis loops and the predominant form of plastic hinging developed. The energy dissipated in beam plastic hinges is typically larger than in column plastic hinges, but this is not recognised in the estimation of equivalent viscous damping in eqn 6(4).

The choice of the structural performance factor, S_{p}, should be appropriate for the detailing used in the structure.

Step FD7: Structure displacement demand

The maximum displacement demand, U_{sd}, at height, h_{eff}, is found from the displacement response spectra defined in section 5.3, for the appropriate level of equivalent viscous damping and appropriate value of S_{p} (Step FD6) multiplied by $(\%NBS)_{\text{t}}$.

Step FD8: Compare structure displacement capacity against demand

Acceptable performance is indicated by the ratio U_{sc}/U_{sd} being greater than one. If this ratio is less than one, retrofitting is required.

7.2.4 Determination of Available Ductility Capacity

a) Available curvature ductility factor and rotation capacity of plastic hinge regions

The available curvature ductility factor at a plastic hinge is given by ϕ_u/ϕ_y where ϕ_u is the available ultimate curvature and ϕ_y is the curvature at first yield.

The available rotation capacity of a plastic hinge is given by:

$$\theta_p = (\phi_u - \phi_y)L_p \qquad \ldots 7(14)$$

where L_p = equivalent plastic hinge depth.

For a beam the first yield curvature is given by:

$$\phi_y = \frac{\varepsilon_y}{d - kd} \qquad \ldots 7(15)$$

where ε_y = strain at first yield of the longitudinal tension reinforcement and d = effective depth of longitudinal tension reinforcement and kd = neutral axis depth when tension steel reaches the strain at first yield, ε_y. For a column, ϕ_y is generally defined using a bilinear approximation (see Figure 7.13) since the moment-curvature relation for a column does not show a well defined yield curvature.

Priestley and Kowalsky (2000) have shown that the first yield curvature is given with very good accuracy as follows:

For beams $\phi_y = \frac{1.7\varepsilon_y}{h}$ where h = beam depth …7(16)

For circular columns $\phi_y = \frac{2.35\varepsilon_y}{D}$ where D = column diameter …7(17)

For rectangular columns $\phi_y = \frac{2.12\varepsilon_y}{h}$ where h = column depth …7(18)

The available ultimate curvature for a beam or a column is given by:

$$\phi_u = \frac{\varepsilon_{cu}}{c} \qquad \ldots 7(19)$$

where c = neutral axis depth at the ultimate curvature and ε_{cu} the ultimate extreme fibre concrete compressive strain, depends on the extent of confinement of the concrete. For unconfined concrete ε_{cu} = 0.004 can be assumed (Priestley and Park 1987). For confined concrete, a higher value may be used. For confined concrete a conservative value is given by Scott et al (1982) as:

$$\varepsilon_{cu} = 0.004\,(1 + 1.1\,p_s f_{yt}) \qquad \ldots 7(20)$$

where p_s = ratio of volume of transverse reinforcement to volume of concrete core and f_{yt} = probable yield strength of the transverse reinforcement.

Alternatively, and less conservatively, the ultimate concrete strain for confined concrete may be assumed to be as given by Mander et al (1988).

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}} \qquad \ldots 7(21)$$

where the volumetric ratio of transverse reinforcement p_s, may be approximated as:

$$\rho_s = 1.5A_v/b_c s \qquad \ldots 7(22)$$

where A_v = total area of transverse reinforcement in a layer, s = *spacing of layers of transverse reinforcement,*, and b_c = width of column core, measured from centre to centre of the peripheral transverse reinforcement in the web. In eqn 7(15) f_{yh} is the yield strength of the transverse reinforcement, ε_{su} is the steel strainat maximum stress, and f_{cc} is the compression strength of the confined concrete. For older designs, it is recommended that ε_{su} = 0.15 and 0.10 for f_y = 275 and 430MPa transverse reinforcements respectively. In lieu of a more accurate analysis (Scott et al 1982; Priestley et al 1996) $f_{cc} = 1.5f'_c$ may be assumed.

The equivalent plastic hinge length L_p may be approximated (Park 1992; Priestley and Park 1987) as:

$$L_p = 0.5h \qquad \ldots 7(23)$$

where h = section depth, or taken more accurately and less conservatively as:

$$L_p = 0.08L + 0.022f_y d_b \qquad \ldots 7(24)$$

where L = distance of the critical plastic hinge section from the estimated point of contraflexure, f_y = probable yield strength of longitudinal reinforcement, and d_b = diameter of longitudinal reinforcement. The first term on the right hand side of eqn 7(24) represents the spread of plasticity due to tension shift effects and the second term represents strain penetration into the supporting member. (For example, a beam-column joint).

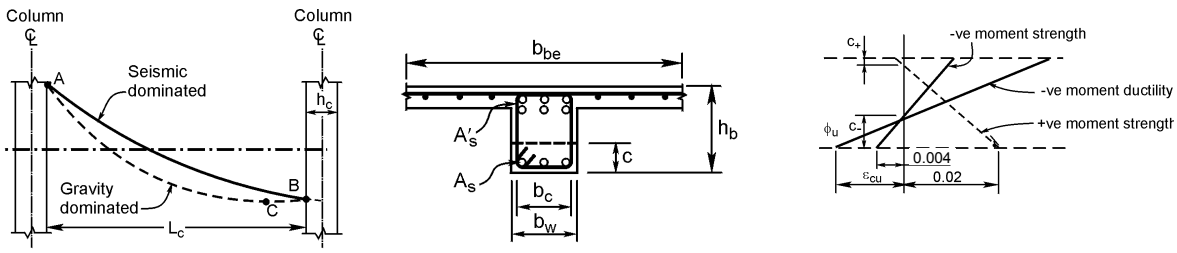
b) Beam plastic rotation capacity

The plastic rotation capacity of the beam plastic hinges defines the plastic story drift in a beam sidesway mechanism. This will depend primarily on the detailing of the transverse reinforcement in the potential plastic hinge regions at the beam ends.

Figure 7.11 shows a beam and the adjacent columns of a seismic resisting frame, and presents information relevant to predicting the available plastic rotation capacity θ_p for beams of typical frames.

As shown in Figure 7.11(a), the distance between the critical section and the point of contraflexure will depend on the relative flexural strength of positive moment and negative moment plastic hinges, and the relative importance of seismic and gravity moments. However, it is suggested that for negative moment plastic hinges, which will generally form against the column face (point A in Figure 7.11(a), a length $L = 0.5L_c$, where L_c = beam clear span, be assumed. This is a reasonable reflection of the fact that (i) negative moment capacity will exceed positive moment capacity, and (ii) high shear stress levels in the plastic hinge region will tend to extend the effective plastic hinge length due to tension shift effects.

The positive moment plastic hinge could form at either the column face (point B in Figure 7.11(a)) or within the span (point C), depending on the influence of gravity loads on the beam. However, the location, which is always hard to define due to uncertainty in the magnitude of gravity loads, and the plastic rotation capacity of the positive moment hinge are of little interest in assessment of plastic rotation because this will generally greatly exceed the rotational capacity of the negative-moment hinge. This is a consequence of (i) the top reinforcement area (including slab contribution) exceeding the bottom reinforcement area and the effective compression zone width b_{be} *for positive moments exceeding the web width* b_w*, appropriate for negative moments (see Figures7.11(b) and 7.11(c)). This results in a greatly reduced compression zone depth* c_+ *for positive moments compared to that for negative moments, as illustrated in Figure 7.11(c). Since compatibility of the storey deformed shape requires that the plastic rotations of all plastic hinges along a beam are essentially equal at any given stage of response, and since plastic hinge lengths for positive moment can be expected to exceed those for negative moment, it follows that the critical condition, corresponding to attaining the ultimate compression strain* ε_{cu} *in a plastic hinge, will always be in a negative moment plastic hinge. It can readily be shown that the theoretically feasible condition of attaining ultimate tensile strain in the positive moment hinge is unrealistic at curvatures corresponding to the ultimate negative moment curvature.*



(a): Span elevation **(b): Beam section** **(c): Strain profiles**

Figure 7.11: Considerations for beam plastic hinges

Figure 7.11(c) shows strain conditions to be used for estimating the flexural strength of the positive and negative moment hinges.
For *'unconfined'* conditions, corresponding to:

- only corner bars restrained against buckling by a bend of transverse reinforcement
- hoop stirrup ends not bent back into the core i.e. 90° hooks.
- spacings of hoop or stirrup sets in the potential plastic hinge such that:
  $s \geq d/2$
  or $s \geq 16d_b$

the ultimate concrete strain ε_{cu} should be assumed to be 0.004, thus corresponding to conditions at determination of flexural strength, where d = effective depth of beam section and d_b = diameter of longitudinal reinforcement.

For *'fully confined'* conditions, corresponding to details satisfying current codes:

- all beam bars in the lower layer (i.e. if more than one) of bottom reinforcement restrained against buckling by transverse reinforcement of diameter greater than $d_b/4$
- all transverse reinforcement anchored by hooks bent back into the core by standard 135° hooks or equivalent anchorages
- spacing of hoop or stirrup sets not less than $s = d/4$ nor $s = 6d_b$.

the ultimate concrete strain ε_{cu} should be calculated as discussed in 7.2.4(a) above.

An example of this approach is given in Figure 7.12, where moment-curvature curves for positive and negative moment bending of a typical beam section are shown. A bay length of 6 m is assumed, which, with a column size of 450 mm square gives an effective clear span of 5.55 m. Top steel area including the contribution of slab reinforcement over a 3000 mm effective width is more than double the bottom steel area. Despite this high steel ratio, the strength of the section in positive and negative bending are not greatly different at high curvatures, due to cover spalling and a deep compression zone depth for negative moments, and strain hardening for positive moments.

At the ultimate curvature for negative bending (ε_{cu} = 0.005) for unconfined concrete the positive moment plastic hinge has a maximum extreme fibre strain of less than 0.0015, even assuming a reduced effective compression zone width of 1000 mm. If the longitudinal reinforcement is properly restrained against buckling by sets of three D10 bars at 100 mm centres, the ultimate negative moment curvature increases from 0.046 radians/m to 0.12 radians/m. At this curvature, spalling of cover concrete for the positive moment hinge is still not expected.

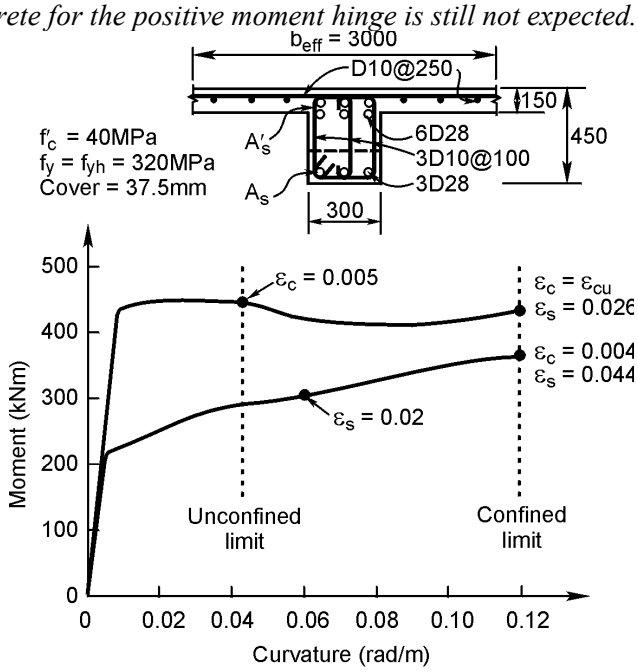


Figure 7.12: Moment-curvature relationships for beam example

The analysis for positive moment bending is simplistic, since under cyclic loading, the bottom reinforcement will be unable to yield the top reinforcement in compression, and thus a steel couple will develop, with slightly reduced moment capacity. Nevertheless, the conclusion that positive moment bending is not critical remains.

For the example of Figure 7.12, an effective plastic hinge length of L_p = 0.08 x 2550 + .022 x 320 x 28 = 401 mm is predicted from eqn 7(24). The more conservative Equation 7(23) gives L_p = 225 mm. With a yield curvature of ϕ_y = 0.009 radians/m (from moment-curvature analysis, or hand analyses), the plastic rotation capacity of the plastic hinge is found to be, for the unconfined case, θ_p=(0.046 – .008) x 0.401 = 0.015 radians.

c) Column plastic rotation capacity

The procedure outlined above also applies, with minor changes, to plastic hinges forming at column bases, or in column sidesway mechanisms. However, the approximation for the volumetric ratio of transverse reinforcement in eqn 7(26) should be replaced by a first principles approach. In fact, it will

often be found that columns in older reinforced concrete frames have only nominal transverse reinforcement, and thus must be considered to be unconfined. Together with reduced plastic hinge length as a consequence of reduced member height compared with beam length, and reduced ultimate curvature as a consequence of axial compression, column plastic rotation capacity will generally be less than values estimated for beams, and values less than $\theta_p = 0.01$ radians will be common.

Since axial load critically affects the ultimate curvature, it is essential that seismic axial forces be included when estimating column plastic rotation. The critical column will be the one with highest axial compression. Moment-curvature analyses will show that, while yield curvature is not greatly affected by axial load level, particularly when yield curvature is expressed in terms of equivalent elasto-plastic response, ultimate curvature, and hence plastic rotation capacity is strongly dependent on axial load.

This is illustrated in Figure 7.13, where an unconfined end column of a frame, with nominal axial load of $P = 0.2f'_{ca}A_g$ is subjected to seismic axial force variations of $P_E = \pm 0.2f'_{ca}A_g$. The yield curvatures differ by less than 10% from the mean, while the ultimate curvatures at P=0 and $P=0.4f'_{ca}A_g$ are 61% and 263% of the value at $P = 0.2f'_{ca}A_g$.

d) Lateral plastic displacement capacity of frames

In the force-based procedure and the displacement-based procedure for assessing moment resisting frames, the available displacement ductility factor μ_{sc} or ultimate horizontal displacement U_{sc} need to be related to the available curvative ductility factors or plastic rotations at the plastic hinge regions. Although the precision with which the plastic drift capacity of existing structures can be predicted is not high, some guidance is given in the following for the cases of a beam sideway mechanism and a column sideway mechanism shown in Figure 7.3.

The lateral displacement at the centre of action of the seismic force at first yield U_{el} may be found by linear elastic pushover analysis. The first yield displacement U_{el} may be defined as in Figure 7.5.

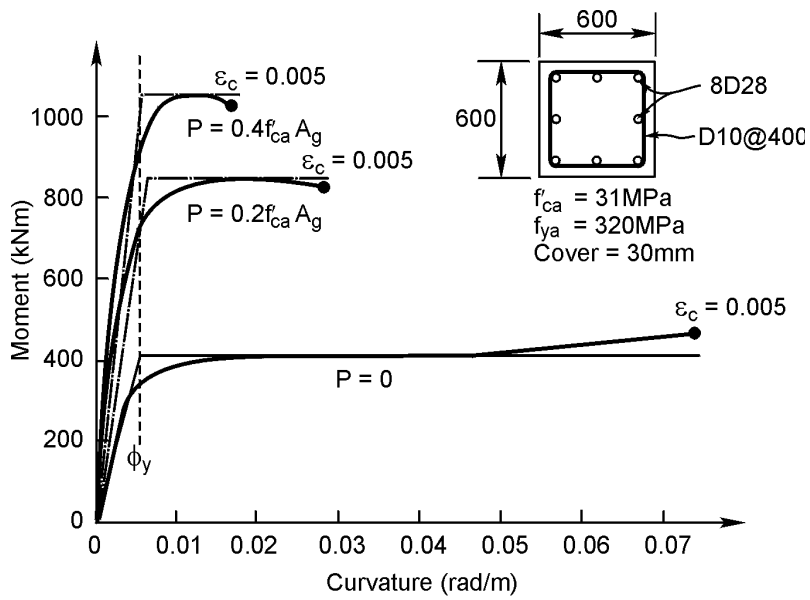


Figure 7.13: Moment curvature response of unconfined columns

The lateral displacement in the post-elastic range at the centre of action of the seismic force will depend on the shape of the plastic deformation profile of the frame. The range of conventional inelastic displacement profiles for frames are shown in Figure 7.14 (Priestley 1995).

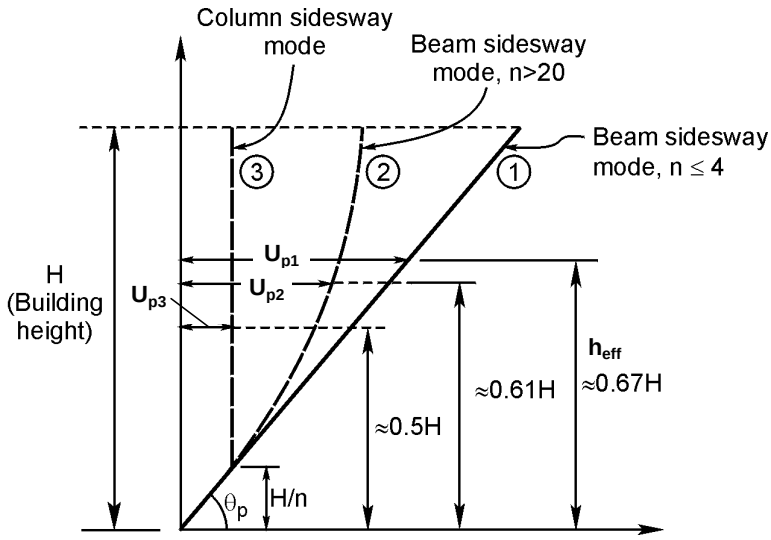


Figure 7.14: Inelastic displacement profile for frames

Ideally, the inelastic displacement profiles for frames should be found from an inelastic frame lateral response analysis, incorporating all potential member nonlinearities. This can be achieved using special purpose 'push over analysis' programmes, or by use of dynamic inelastic time history analyses (e.g. Carr (1994) where the lateral force vector is gradually increased in magnitude sufficiently slowly to ensure that dynamic modes of the structure are not excited. However, this assumes a knowledge of the shape of the lateral force vector, which will typically be assumed to be an inverted triangle, and which may be a reasonable approximation of the elastic displacement profile. If an inelastic deformation mode develops with a displaced shape markedly different from the assumed inverted triangular shape, as would be the case for a column sidesway mode, the vertical distribution of forces in the lateral force vector would gradually deviate increasingly from the inverted triangle shape. To warrant the sophistication of an inelastic push over analysis, it would seem that it would be necessary to be able to modify the shape of the lateral force vector, as plastic displacements increase.

The considerations discussed above are, however, relatively straightforward to implement in a hand analysis, though the degree of precision must be recognised to be rather coarse. Since our ability to determine realistic characteristics for design (or assessment of seismicity is of considerably greater coarseness, this should not be seen to invalidate this simple process.

Consider the inelastic displacement profiles of Figure 7.14. Three cases are considered, all with the same maximum plastic rotation θ_p, assumed to develop in the lowest storey. The linear profile 1 corresponds to a beam sidesway mechanism in a low rise frame (say $n \leq 4$). For much taller frames (say $n \geq 20$), dynamic inelastic analyses indicate that at peak response, the plastic displacement profile is nonlinear, with larger plastic drifts occurring in the lower storeys.

Paulay and Priestley (1992) recommend a peak inelastic drift equal to about twice the average over the building height, though there is some evidence that this may be excessive when hysteretic characteristics are used that are more representative of reinforced concrete behaviour than the elasto-plastic analyses used as a basis for those recommendations.

Profile 2 shows the expected shape, for n > 20 for a beam sidesway mechanism assumed to be parabolic. If a column sidesway mechanism develops in the lowest storey, the inelastic displacement shape is represented by profile 3. Based on these shapes, the inelastic displacement of the centre of seismic force can be estimated. First, however, it must be recognised that the centre of seismic force itself depends on the displaced shape. If an inverted triangle shape is a reasonable approximation of the elastic displacement response, then, initially the effective height of the single degree of freedom representative of the structure is approximately:

$$h_{eff} = 0.67H \qquad \ldots 7(25)$$

where H = height of building.

This is also the effective height for the inelastic displacement profile 1 of the short frame, but profiles 2 and 3 have shapes with lower centroids ($h_{eff} = 0.61H$, $h_{eff} = 0.5\ H$) at very large values of μ_s.

For the beam sidesway mechanisms, the effect is not particularly significant, and it is proposed that, for regular elements, both elastic and inelastic displacements be determined at an effective height of $0.64H$. It is also suggested that the displaced plastic shape be considered to vary linearly from profile 1 to profile 2 as n increases from 4 to 20. The plastic displacement at $0.64H$ can thus be shown to be:

for $n \leq 4$: $U_p = 0.64\ \theta_p H$

$n \geq 20$: $U_p = 0.44\ \theta_p H$

$$4 < n < 20: U_p = (0.64 - 0.0125\ (n-4))\,\theta_p H \qquad \ldots 7(26)$$

For the column sidesway mechanism (profile 3), h_{eff} should reflect the ductility level. Thus, approximately:

$$h_{eff} = [0.64 - 0.14\ (\mu_s - 1)/\mu_s]\ H \qquad \ldots 7(27)$$

where μ_s the displacement (structure) ductility factor.

The plastic displacement U_{inel} is given, for a structure of n equal storey heights h_s, as:

$$U_{inel} = \theta_p h_s$$

$$\therefore U_{inel} = \theta_p H/n \qquad \ldots 7(28)$$

For both the beam sideway and column sideway mechanism, calculating the structural yield displacement U_{el} at the effective height h_{eff}, the ultimate displacement capacity is given by:

$$U_{sc} = U_{el} + U_{inel}$$

and the displacement ductility factor by:

$$\mu_{sc} = U_{sc} / U_{el} = 1 + \frac{U_{inel}}{U_{el}}$$

In eqn 7(28), θ_p is the plastic rotation occurring at the top and bottom of the bottom storey column and at the negative moment plastic hinge at the beam ends. | 2

Hence, in summary, the equations relating the displacement ductility factor μ_s and the ultimate curvatures at the plastic hinges are:

a) For a beam sideway mechanism:

If $n \leq 4$

$$\mu_{sc} = 1 + \frac{0.64(\phi_u - \phi_y)L_p H}{U_{el}}$$

If $n \geq 20$

$$\mu_{sc} = 1 + \frac{0.44(\phi_u - \phi_y)L_p H}{U_{el}}$$

If $4 < n < 20$

$$\mu_{sc} = 1 + \frac{\{0.64 - 0.0125(n-4)\}(\phi_u - \phi_y)L_p H}{U_{el}} \qquad \ldots 7(29)$$

b) or a column sideway mechanism:

$$\mu_{sc} = 1 + \frac{(\phi_u - \phi_y)L_p H}{nU_{el}} \qquad \ldots 7(30)$$

7.3 Moment Resisting Frame Elements with Masonry Infill Panels

The assessment of an infilled frame is obviously dependent on the material constituting the infills and the geometry of both frame and infill. The information provided in Section 9 is intended to provide a reference or starting point for entry into the main frame assessment procedures described in sections 7.2.2.and 7.2.3. While Section 9 focuses on the response of reinforced concrete frame elements with infill panels, the summary of actions and much of the analysis can also relate to structural steel frames.

7.4 Structural Wall Buildings

7.4.1 Introduction

The assessment of structural systems in which seismic resistance has been assigned to reinforced concrete structural walls, is likely to be less elaborate than that of frame systems. In the presence of robust walls, the contribution to seismic resistance of other elements, with a primary role of supporting gravity loads, may often be neglected. The detailing of such frame components need only to satisfy greatly reduced ductility requirements. When the contribution of such frame elements to seismic performance is judged to be more significant, or when the system needs to rely on their seismic contribution to satisfy seismic performance criteria, the system should be treated as a dual frame-wall building, considered in Section 7.5.

The displacement ductility capacity of each wall of the building, and particularly those having the greatest lengths, with a possible maximum value of 5 should be checked. The associated limit displacement of such walls will determine the displacement capacity of the system.

The relationship between ductilities developed in walls with different dimensions and that of the wall system as a whole can be seen in Figure 7.18.

7.4.2 Force-Based Procedure for Wall Buildings

The steps set out below are summarised in the flowchart of Figure 7.15.

Choose $(\%NBS)_t$ and then for each principal direction carry out the following steps;

Step WF1

Evaluate all gravity load related quantities, such as equivalent floor masses, the centre of mass for the building, and the appropriately factored dead and live loads on each of the structural walls.

Compute the corresponding average compression stress over the gross concrete area of each wall, and hence evaluate effective stiffness (Table C3.1 SANZ 1995). | 2

Based on the effective vertical reinforcement at the base, and the gravity loads, determine the probable flexural strength, M_{wp}, of each wall. The neutral axis depth to wall length ratio, c/l_w, a by-product of this calculation, is used subsequently when checking the curvature ductility capacity of each critical wall section.

The probable shear capacity of the plastic region at the base of each wall, $V_{wall,p}$ can be assessed by assuming that the probable contribution of concrete mechanisms to shear strength is quantified, in terms of nominal shear stress, by:

$$v_{cp} = 0.6\sqrt{[(f'_c/25)(N^*/A_g)]} \qquad \ldots 7(31)$$

If the displacement ductility demand is found to be moderate (i.e. less than 3, refer Step WF8), then a higher nominal shear stress may be taken as follows;

$$v_{cp} = (5 - \mu_{sd})(\sqrt{f'_c} + N^*/A_g)/16 \qquad \ldots 7(32)$$

Eqn 7(31) represents a slight adjustment of Equation 9.46 in SANZ (1995), which, for the sake of simplicity, was based on a concrete compression strength of 25 MPa, in order to allow some benefit to be derived when the assessed concrete in the existing structure is stronger. Equation 7(32) is a simplified form of Equation 17–9 in SANZ (1995), applicable to elements of limited ductility. For design purposes the two equations are identical. In certain cases eqn 7(32) would allow more liberal values of concrete shear stress to be used when the estimated ductility demand is between 3 and 4.

The contribution of the existing horizontal shear reinforcement to the total probable shear resistance of each wall may then be determined.

Step WF2

Using the appropriate wall stiffnesses determined in Step WF1, carry out a routine analysis of the elastic structural system. The main purpose of this analysis is to estimate the contribution of each wall to the resistance of the total lateral design forces

Step WF3

From the summation of probable flexural strengths at the wall base sections, estimate the total potential probable lateral force carrying capacity of the structure:

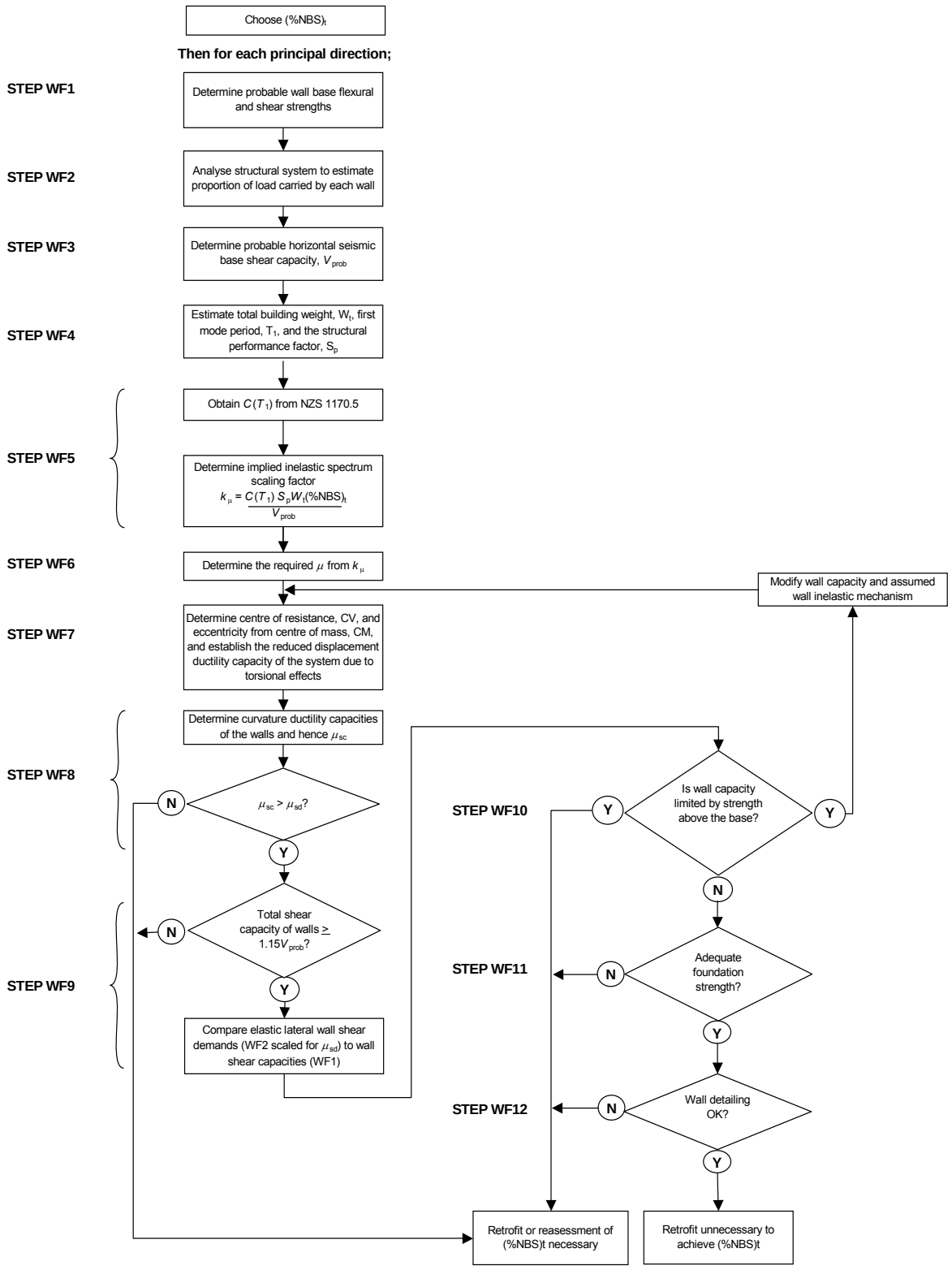


Figure 7.15: Summary of force-based assessment procedure for walls

$$V_{prob} = 1.5\sum M_{wp} / h_w \qquad \ldots 7(33)$$

where h_w = height of the walls, which is assumed here to be the same as the height of the building.

As a result of subsequent investigation of the shear strength and the appropriateness of the detailing of the plastic hinge region at the base of the walls, the probable flexural resistance, M_{wp}, of some or all walls may need to be revised.

Step WF4

Estimate the fundamental period of vibration of the system, T_1, the total weight of the structure, W_t, and the structural performance factor, S_p, appropriate to the level of detailing present in the structure.

Step WF5

Obtain the ordinate of the elastic site hazard spectrum for T_1 for the site from Section 3 NZS 1170.5.

Determine the implied inelastic scaling factor, k_μ, corresponding to the probable lateral force capacity of the structure, V_{prob}, found in Step WF3.

Step WF6

Determine the displacement ductility *demand* on the system (μ_{sd}) from k_μ using the appropriate equations given in Section 5 NZS 1170.5.

The walls are subsequently checked to ascertain whether their ductility *capacity* is adequate to accommodate this demand (Step WF8).

Step WF7

Using the probable strength, M_{wp} (Step WF1), or the corresponding base shear force, determine the centre of resistance, CV, of the system and hence strength eccentricities e_{vy} and e_{vx}, with respect to the centre of mass, CM, of the building.

Although somewhat idealised, Figure 7.16 illustrates relationships with the definition of symbols.

If the strength eccentricity exceeds 2.5% of the relevant lateral dimension of the plan, revise the probable strength of the system derived with eqn 7(36). In such cases reduce the probable strengths of those elements which are responsible for the strength eccentricity obtained, so that with this step the strength eccentricity is eliminated. Using this reduced hypothetical strength of the system, revise the estimation of the displacement ductility *demand* made in Step WF6. | 2

The procedure is based on the assumption that in the absence of strength eccentricity the response of the system may be considered to be governed primary by translatory displacements. In terms of ductile response, effects of stiffness - eccentricity may be ignored.

For example it is found that the relative probable translatory strengths of elements (1), (2) and (3), shown in Figure 7.16, are 46%, 18% and 36% respectively. These result in a negative strength eccentricity of $e_{vx} \approx 0.10A > 0.025A$. *A reliance on only 30% and 13% strength contribution of elements (1) and (2), respectively, to the left of the centre of mass, would result in a total probable strength of only 79%, but no probable strength of only strength eccentricity. The expected displacement ductility demand on the system may then be based on this reduced system strength.*

Under these circumstances displacement demands on element (3) due to system translations and rotations, while developing 100% of the probable system strength, will not be critical.

In traditional design procedures, based on elastic structural behaviour, strengths to elements are assigned in proportion of their assumed stiffness. Subsequently strength redistribution (SNZ 1995) within a 30% limit was permitted to be used, provided that the total seismic strength of the building is not reduced. This restriction on the allocation of seismic strength to elements is now considered to be unnecessary. Hence reliance on the probable strengths of elements, as constructed, may be made, without recourse to analysis of the elastic structure, in evaluating with eqn 7(33) the total strength of the system.

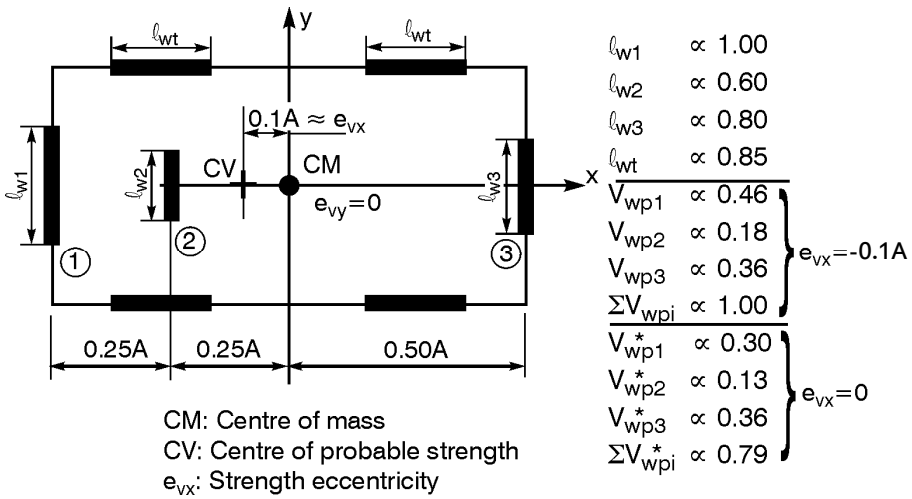


Figure 7.16: Torsional effects in walled buildings

Step WF8

The curvature ductility capacity of walls needs to be checked. However, only in exceptional circumstances, for example when T or angle-shaped sections are used, or when exceptionally large gravity loads are to be carried, might curvature criteria become critical. A simple measure of the curvature ductility capacity of a wall section, based on a maximum concrete compression strain of ε_{cu} = 0.004, is the neutral axis depth to wall length ratio, c/l_w, calculated in Step WF1. When the conservative approximation:

$$c/l_w \leq 0.3 - \mu_{sd} / 27 \qquad \ldots 7(34)$$

is satisfied, it may be assumed that the curvature ductility demand on the wall section can be met and that no confinement of the concrete in the compressed boundary region of the wall is necessary. μ_{sd} is the displacement ductility demand on the wall system assessed in Step WF6.

Eqn 7(34) is based on the assumption that the ductility demand, μ_{sd}, imposed on a wall with an unconfined boundary region, and with an aspect ratio of $A_r = h_w/l_w = 4$, will not exceed the maximum (i.e. 5) currently defined by SNZ (1995). Eqn 7(34) may be unconservative for more slender walls. If refinement is required, Figure 7.17 may be used. If the requirements of eqn 7(34) are not satisfied, the following two avenues may be followed in order to estimate the displacement ductility capacity of the system, μ_{sc}:

a) With the value of the c/l_w ratio obtained from a routine section analysis, the limitation of the displacement ductility capacity of the walls may be obtained from Figure 7.17. With good approximation the curvature ductility ***capacity*** *of a wall section as detailed is:*

$$\mu_\phi = \frac{1.25}{c / l_w} \qquad \text{...7(35)}$$

for the known effective aspect ratio, A_r, of a wall, this value of μ_ϕ may be used to obtain the value of μ_{sc} from Figure 7.17.

b) It may be that a limited amount of effective confining reinforcement in the boundary region is present. This would allow larger concrete compression strains and hence ultimate curvatures to be developed. Provided that the limitations on the maximum spacing of the transverse reinforcement of SNZ (1995) are met, the displacement ductility capacity of the walls may be obtained from the inversion of the equation governing the necessary amount of transverse reinforcement, thus:

$$\mu_{sc} = 40 \frac{A_{sh}}{S_h h''} \Big/ \left[\frac{A^*_g}{A^*_c} \frac{f_c}{f_{yh}} \left(\frac{c}{l_w} - 0.07 \right) - 0.1 \right] \qquad \text{...7(36)}$$

The definition of the effective height, h_{eff}, is provided in Figure 7.17, which also shows usable limits of ductility capacities at which, when different steel grades are utilised, the drift at the vicinity of the effective height of a type of wall will attain 2.5%. | 2

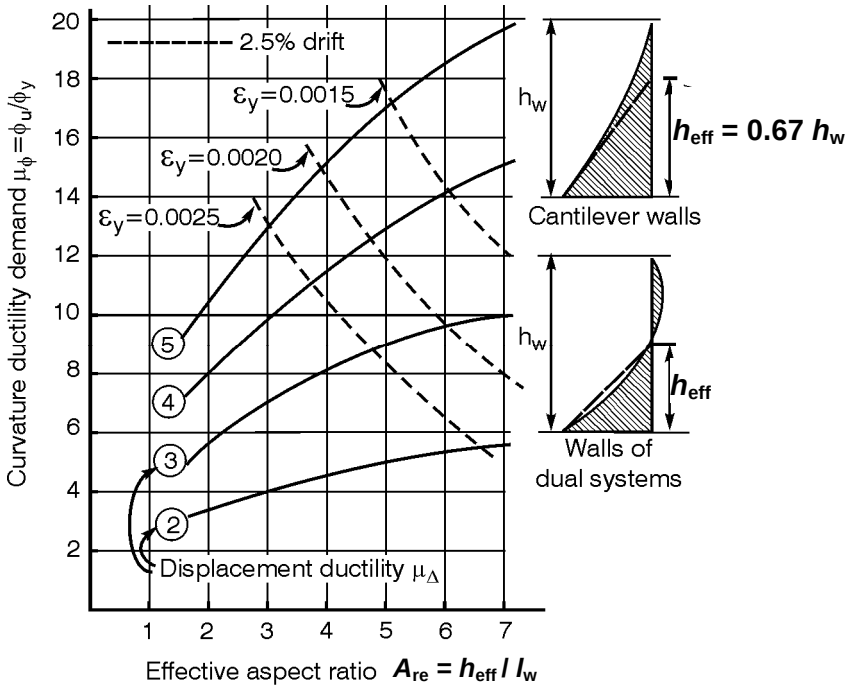


Figure 7.17: Required curvature ductility capacity of cantilever wall sections as a function of displacement ductility demand and aspect ratio

The storey drift in ductile walls is sensitive with respect to the effective aspect ratio, A_{re}, and the yield strain of the steel, ε_y. When different grades of reinforcing steel are used, maximum usable displacement and curvature ductilities are significantly affected. The dashed line curves in Figure 7.17 show ductility limits associated with 2.5% storey drift.

When the spacing limitations of transverse ties are violated, engineering judgement as to their efficiency should be used. For example:

$$A_{sh,\ effective} = \alpha\, A_{sh,\ provided} \qquad \text{...7(37)}$$

where $\alpha < 1.0$ and the value of α must be estimated.

Step WF9

2 With a factor of 1.15 being applied to reflect capacity design principles, the sum of the probable wall shear strengths should be such that:

2 $$\sum V_{\text{wall,p}} \geq 1.15\ V_{\text{prob}} \qquad \ldots 7(38)$$

where V_{prob} is given by eqn 7(33), and noting that the base shear strength corresponding with the elastic response of the building (μ = 1.25) represents an upper bound requirement for shear strength.

The dynamic magnification factor, ω_v, from SANZ (1995) need not be applied in this calculation. This reflects the comparatively short duration of dynamically amplified shear actions, and the aim of *identifying the level at which failure is likely to occur* as opposed to the design objective of *precluding failure*.

If eqn 7(38) is not satisfied (recognising the elastic response upper bound), retrofitting measures need to be undertaken. Even if the equation is satisfied, it is possible that some individual walls have insufficient shear strength to develop their flexural overstrength. Judgement must be exercised in determining the significance of one or more walls being in this situation; factors to be considered include the proportion of walls non-complying in this respect, the position of the non-complying wall(s) with regard to maintaining overall stability and the degree by which an individual wall is unable to develop its flexural capacity.

Step WF10

In terms of the linearised design moment envelopes, recommended in the commentary to SANZ (1995) check the extent of possible deficiency of flexural and shear resistances of the walls at levels above the anticipated plastic region at the base of the building. Flexural deficiencies may result from excessive curtailment with height of the vertical wall reinforcement. In particular examine whether a plastic hinge could develop at any level other than at the base.

Step WF11

Check whether the existing foundation structure is capable of resisting the moment input associated with 1.15 times the probable strength of each wall. If it is found that a particular member of the foundation structure does not possess adequate strength, extend the investigation to include the following features:

a) Evaluate the probable strength of the affected component of the foundation structure, taking into account both the associated shear demand on that member and the quality of the detailing of the existing reinforcement.

b) Examine the possibility of a brittle failure of that component of the foundation structure.

c) If a ductile response of the affected component, corresponding to the overall ductility demand on the building, determined in Step WF6, appears to be assured, reduce accordingly the contribution of the affected wall to the total lateral force resistance of the building at the ultimate limit state.

d) When a brittle failure of the component of the foundation structure is anticipated, disregard the contribution to lateral force resistance of the relevant wall.

e) If the reduction or absence of the wall's contribution to the total lateral force resistance is significant, re-examine the capacity of the entire structural system in terms of the parameters considered in Steps WF2 - WF6.

Step WF12 | 2

Finally, check the adequacy of the walls in terms of dimensional limitations of cross-sections and the quality of the detailing of the reinforcement, particularly in the region of potential yielding. Conclusions drawn from the failure in existing walls to meet relevant current code requirements should be tempered with rational engineering judgement. The criteria to be considered should include:

a) Dimensional limitations relevant to potential for out-of-plane buckling of relatively thin walls should be based on the ductility limits evaluated in Step WF6.

b) The adequacy, particularly in terms of spacing, of the transverse reinforcement in the boundary regions of wall sections within the potential plastic region, to provide lateral restraint against buckling of vertical reinforcing bars.

c) The adequacy of such transverse reinforcement in providing some confinement to the compressed concrete in the boundary regions, when this appears to be necessary following the assessment in Step WF8. | 2

d) The anchorage within the foundation structure of the vertical wall reinforcement, which controls the flexural strength of the walls.

7.4.3 Displacement-Based Procedure for Wall Buildings

With minor modifications the steps shown in the flow charts of Figure 6.3 may be applied to buildings in which seismic performance relies a set of reinforced concrete walls. Changes in terminology and specific requirements for wall buildings are indicated in the steps that follow.

Step WD1 : Probable strength of the building

Based on the probable flexural strength at the base of the constituent walls, M_{wp}, the base shear capacity of the system, V_{prob}, is derived from Equation (33).

Step WD2 : Post-elastic mechanism

Performance evaluation is based on the formation of a plastic hinge at the base of each cantilever wall. As outlined in Step WF10, it is necessary to ascertain that this desirable mechanism, without premature shear failures, can be sustained.

Steps WD3, WD4 and WD5 : Deformation capacities of wall elements

Details of the evaluation of the relevant deformations of the system and its constituent elements, such as nominal yield curvatures, nominal yield displacements, storey drifts and element and system deformation capacities are outlined in Section 7.4.4.

The effects of degradation on the inelastic rotation capacities of walls can be ignored.

Step WD6 : Effective stiffness, effective period of vibration, S_p and equivalent viscous damping

These steps follow the descriptions presented in section 6.3.

Step WD7 : Compare the displacement capacity of the building with expected displacement demand

The strength-independent displacement capacity of a wall system, is evaluated in Step WD3, using eqn (47). The estimation of displacement demand follows the procedure described in Step D8, section 6.3 using displacement spectra presented in section 5.3.

7.4.4 Deformation Capacities of Wall Elements and the Building System

Irrespective of the arrangement and ratio of vertical reinforcement at the base of a wall with length, τ_w, its nominal yield curvature may be estimated by

$$\phi_{wy} = 1.8\ \varepsilon_y / l_w \qquad \ldots 7(39)$$

The corresponding nominal yield displacement of a cantilever wall in terms of its effective height, h_{eff}, shown in Figure 7.17, is in the order of:

$$U_{wy} \approx N_{wy}\ h_{eff}^2/3 \approx 0.6\ \varepsilon_y\ A_{re} h_{eff} \qquad \ldots 7(40)$$

where $A_{re} = h_{eff} / l_w$ is the effective aspect ratio of the wall, and h_w is the full height of the walls.

The associated storey drift at levels above the effective height, h_e, is in the order of:

$$\delta_{wy} \approx N_{wy} h_{eff} / 2 = 0.9\ \varepsilon_y\ A_{re} \qquad \ldots 7(41)$$

Depending on the identified quality of the detailing of a wall, its displacement ductility capacity is to be limited to either μ_{wc} = 5 or 3 (SNZ (1995)) or to a value that does not lead to excessive perceived drift, such as 2.5%.

The displacement ductility capacity of a wall, μ_{wc} may be estimated with the use of eqn 7(44) and Figure 7.18.

When the effective aspect ratio of a wall, A_{re}, approaches 4, drift criteria may well limit the acceptable displacement capacity of the wall. The maximum drift in the vicinity of the effective height of the wall is in the order of:

$$\delta_{w,\,max} = \delta_{wy} + \delta_{wp} \qquad \ldots 7(42)$$

where the drift associated with post-yield displacement, $U_{wall,\,inel}$, of the wall is

$$\delta_{wp} = U_{wp} /(h_{eff} - 0.5L_p) \qquad \ldots 7(43)$$

Noting that $U_{wp} = (\mu_{wc} - 1)\ U_{wy}$ and that, as stated previously, assuming the plastic hinge length of a wall to be $L_p \approx 0.5\ l_w$, it is found that the ductility capacity of a wall, satisfying the 2.5% drift criterion is limited to:

$$\mu_{wc} = 0.025\ (A_{re} - 0.25)\ (l_w / U_{wy}) + 1 \approx 0.04\ (A_{re} - 0.25)/(\varepsilon_y\ A_{re}^2) + 1 \qquad \ldots 7(44)$$

Because displacement capacity of the building is controlled by that of a wall element with the smallest effective aspect ratio, A_{re}, eqns (39), (40) and (44) need to be considered only for the wall of the system with the greatest length.

Equation (39) allows the stiffness of wall elements, while responding essentially in the elastic domain of behaviour, to be defined as:

$$k_w = \frac{M_{wp}}{h_{eff} U_{wy}} \qquad \ldots 7(45)$$

Figure 7.18 illustrates the bilinear modelling of force-displacement relationships for wall elements and the system shown in Figure 7.16. Torsional displacements have not been considered in this illustration. It is seen that each element can be expected to enter the inelastic domain at a different lateral displacement and that the stiffness of an element, implied in the modelling, is proportional to its probable strength. The superposition of element responses leads to the non-linear total response of the system. Although a system does not have a distinct nominal yield displacement, for the purposes of this procedure a reference system nominal yield displacement of :

$$U_{sp} = \Sigma M_{wp} / (h_{eff} \Sigma k_w) \qquad \ldots 7(46)$$

may be used. This then enables the system displacement ductility to be estimated as $\mu_s = U_u / U_{sy}$.

The translatory displacement capacity of a wall building is in general limited by that of its critical element, $U_{wc.min.}$. Therefore, the displacement ductility capacity of the system is:

$$\mu_s = U_{wc,min} / U_{sy} \qquad \ldots 7(47)$$

Its interpretation may be seen in Fig. 7.18. It also defines the effective stiffness of the ductile wall system:

$$k_{eff} = \Sigma M_{wp} / (h_o \mu_s U_{sy}) \qquad \ldots 7(48)$$

shown by the diagonal dotted line, in accord with the displacement-based assessment approach.

7.4.5 Estimation of Equivalent Viscous Damping

Because walls in general are subjected to small axial compression loads, their displacement ductility-dependent equivalent viscous damping is similar to that of adequately detailed beams. However, with reduced aspect ratios, A_{re}, shear deformations in walls are to be expected to be more significant. This could result in some loss in hysteretic damping. Therefore, reduction of the effective damping may be warranted for walls with $A_{re} < 3$.

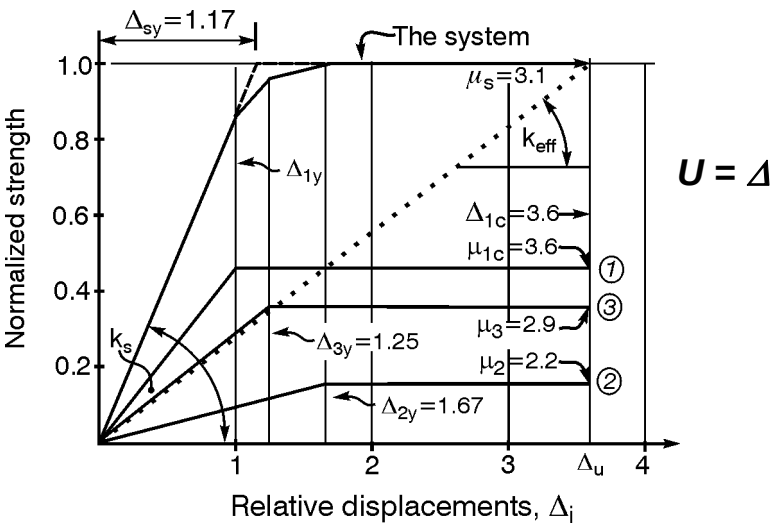


Figure 7.18: Bilinear idealisation of ductile element and system response for a wall building shown in Figure 7.16

7.5 Dual Frame-Wall Buildings

7.5.1 Features of Dual Systems

In dual systems, elements resisting lateral forces in a given direction of the building may have significantly different behaviour characteristics. Mechanisms associated with their ductile response may also be very different. Typical examples are buildings where lateral forces in different parallel vertical planes are resisted by either ductile frames or ductile walls. Walls forming a service core over the full height of the building are common. They may be assigned to resist a major part of the lateral forces, while primarily gravity load carrying frames may also be required to provide a significant fraction of the required seismic strength. Irrespective whether elastic or post-yield behaviour is considered, displacement compatibility requirements (Paulay and Priestley, 1992) over the full height of the building need be considered. The presence of a rigid diaphragm, with an ability to transfer significant in-plane dynamically induced floor forces to the different vertical elements, is a prerequisite. Therefore, the examination of diaphragm-wall connections is particularly important.

During the ductile dynamic response of such systems, very different displacement ductility demands may arise for each of the two types of elements. One purpose of the assessment procedure is to identify the element with the smallest displacement capacity. Wall elements, often representing significant fractions of the probable lateral strength of the system, are typical examples. They control the displacement capacity of the system.

Major advantages of such systems are that displacement ductilities imposed on frames are generally very moderate, and that dynamic displacement demands are not sensitive to modal effects, as in the case of frame systems. Moreover, in comparison with frame or wall systems, dual systems provide superior drift control. Provided that potential plastic hinges are detailed for moderate curvature ductility demands, column sway mechanisms in any storey of the frames are acceptable.

The assessment procedure outlined is applicable to any combination of walls and frames, provided that no gross vertical irregularities, such as discontinuities in walls, exist. It is based on recently introduced displacement focused treatment of ductile reinforced concrete systems (Paulay and Restrepo 1998, Paulay 2000, 2001b and 2002) and on a redefinition of strength-dependent component stiffness (Paulay, 2001a). This enables the same assessment procedure to be carried

out for strength-and displacement-based performance criteria. The displacement ductility capacity of a dual system needs be made dependent on the displacement capacity of its critical element.

7.5.2 Assessment Procedure for Dual Frame-Wall Structures

The following steps are suggested for the assessment of dual frame-wall structures.

Step DD1 : Probable flexural strength of beams and columns

Evaluate the probable seismic strength of beams and columns, following the procedures covered in Step D1, section 7.2.3.

Step DD2 : Post-elastic mechanism of frames and their contribution to lateral force resistance

The hierarchy of column/beam strength may be established following the procedure covered in Step D2, section 7.2.3. Based on the values of the probable strengths of potential plastic hinges in either beams or columns in the vicinity of beam-column joints, the probable mechanism to be developed in each frame, is established.

When all or the majority of frames are similar, this evaluation is relatively simple. When frames, resisting lateral forces in one of the principal directions considered are different, the lateral force resistance of each frame at each level needs to be derived, as in Section 7.2.

Once the sway mechanism of each storey, comprising plastic hinges in beams or columns or the combination thereof, is established, the total probable flexural resistance of a bay in terms of those of the potential plastic hinges at each level, ΓM_{pi}, is determined. This enables the associated storey shear forces, developed in frames, to be estimated. The probable storey shear force, developed in one frame, is

$$V_{pi} = \Sigma M_{pi} / h_s \qquad \ldots 7(49)$$ | 2

where h_s is the height of the relevant storey.

Figure 7.19 illustrates the interpretation of eqn 7(49). It shows a kinematically admissible sway mechanism. Plastic hinges introduce a total moment of ΣM_{pi} to the 4 columns at the level of the beams. This is proportional to the storey shear force, V_{pi}. Overturning moments transmitted from storeys above, by means of axial forces in the columns, are not shown in this figure.

Once the storey shear forces for each frame, $V_{pi,}$ associated with probable flexural strength developed at the level considered, is found, the total storey shear force sustained by all the frames at that level is:

$$V_{si} = \Sigma V_{pi} \qquad \ldots 7(50)$$

This then enables the probable lateral forces sustained at each level by all the frames to be estimated. Hence the contribution of all the frames to sustaining maximum overturning moments, such as M_{fo} at the base, can also be readily evaluated.

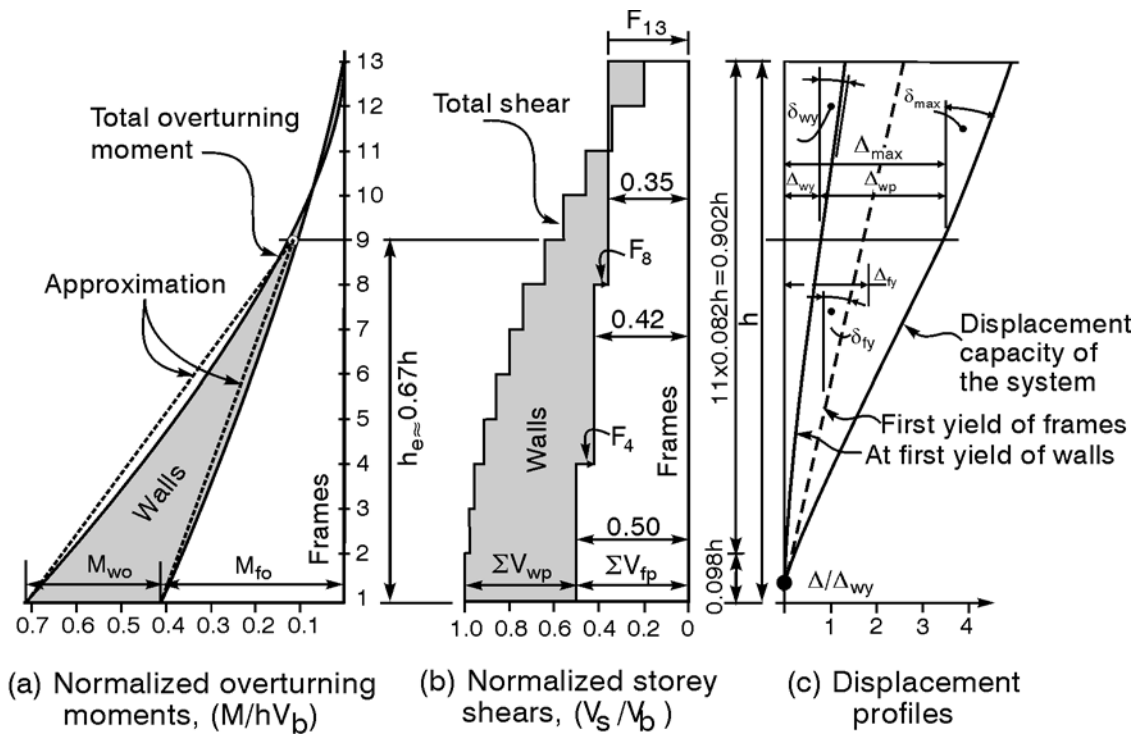


Figure 7.19: The stepwise estimation of the contribution of a frame and a wall element to probable lateral strength and correspondence displacements of a dual system

Figure 7.19 illustrates the stepwise estimation of the contribution to total probable overturning moment capacity and storey shear force of both the frames and the walls.

Step DD2 addresses the contribution to lateral force resistance of frames based on the probable strength of the identified frame mechanisms only. For the 12 storey example structure, it was found that identical detailing of groups of beams, likely to be encountered in existing buildings, resulted in identical storey shear capacities in the first three, the next four and the top five storeys. The plotting of these storey shear forces demonstrates that the combined probable lateral strength of all frames is equivalent to three lateral forces, shown in Figure 7.19(b), as F_{13}, F_8 and F_4. These forces uniquely define the variation of the maximum overturning moments which could be sustained by all the frames (Figure 7.19(a)), when relevant probable strengths developed at all levels.

Step DD3 : The post-elastic mechanism of walls and their contribution to lateral force resistance

The mechanism of the walls of a dual system is expected to comprise plastic hinges at the base of each wall. A detailed study of the wall reinforcement, as in Section 7.4, is required to verify this. Based on the probable strength of the examined base sections of all walls of the system will quantify the total overturning moment that can be sustained by these walls, M_{wo}, subsequently referred to as the wall element. The total probable overturning moment capacity of the dual system at the base is thus

$$M_o = M_{wo} + M_{fo} \qquad \ldots 7(51)$$

as shown in Figure 7.19(a). The expected demands for overturning moments over the height of the building, corresponding with the customary distribution of lateral static design forces (SNZ 1994), can also be readily evaluated (Figure 7.19(a)).

In the chosen example presented, it was shown how the probable storey shear capacities of the frames were evaluated. With this evaluation of the overturning moment capacity of the wall element, M_{wo}, shown in Figure 7.19(a), its probable base shear strength can be estimated from:

$$\Sigma V_{wp} = M_{wo}/h_{eff} \qquad \ldots 7(52)$$

The total probable base shear strength of the system is;

$$V_{prob} = \Sigma V_{wp} + \Sigma V_{fp} \qquad \ldots 7(54)$$

as seen in Figure 7.19(b), where lateral forces and corresponding total storey shear forces are expressed for convenience in terms of the total system base shear, taken a unity. The effective height of the wall element, h_{eff} is given by the approximate position of its point of contraflexure. When a more slender wall element is used, its probable base strength will be smaller. Hence the point of zero wall moment will be at a lower level, resulting in $h_{eff} < 0.67h_w$.

Whereas the storey shear strength provided by the frames can be evaluated with a relatively high degree of precision, the likely shear demand on the walls is less certain, because walls are significantly more sensitive to differences between estimated and real seismic demands. Therefore, comparisons of probable wall storey shear strength, being largely dependent of the horizontal shear reinforcement which has been provided, should be conducted with caution. The shear magnification, 1.15, specified in Step WF9 in section 7.4.2 should be employed. 2

Once the pattern of the total storey shear demands, on both elements, as seen in Figure 7.19(b) is established, that of the corresponding total overturning moment can also be readily determined.

Due to modal effects during the post-elastic dynamic response of the system, moment demands of the wall element may not reduce with height at the same rate as Figure 7.19(a) suggests (SNZ 1995). However, the moment pattern derived may be used to establish the displacement capacity of the building system.

Step DD4 : Estimate the displacement capacity of the dual system

Because, during ductile system response, walls are expected to remain essentially elastic above the plastic region at the base, their deformations will control that of the system. Moreover, the displacement capacity of the walls, rather than that of the frames, should be expected to control the performance limit state. Hence wall benchmark displacements should be estimated and compared with the corresponding displacement ductility demands generated in the frames.

To illustrate the simple procedure leading to displacement capacity estimates, it is reviewed here using again the example structure, data for which are presented in Figure 7.19;

- *Displacement estimates for the walls may be based on a linear variation of bending moments over the effective height, h_e, shown by the dashed line in Figure 7.19(a). Unless more refined values are desired (Paulay, 2001a), the nominal yield displacement of the wall element at the effective height, ie, in the immediate vicinity of the location of zero wall moment, can be estimated with eqns 7(39) and 7(40).*
- *The critical quantity to be considered in gauging displacement ductility demands on the frames, is the maximum drift, δ_{ymax}, in the critical storey. Necessarily this will correspond with the rotation of the walls, expected in the vicinity of the zero wall moments, ie, at height*

h_{eff}. As eqn 7(41) emphasises, critical quantities are the aspect ratio of the walls, A_{re} and the yield strain of the reinforcing steel used, ε_y . The maximum nominal storey yield drift of the walls, δ_{wy}, will immediately indicate whether at this stage any components in that storey of the frame will approach the limit of elastic response. As a general rule, it is found that the critical storey of the frame will enter the inelastic domain only after the walls have been subjected to significant displacement ductility demands.

- *It has been shown (Priestley, 1998) that the nominal yield drift of storeys of reinforced concrete frames, the deformations of which are dominated by those in potential plastic beam hinges, can be estimated as*

$$\delta_{fy} \approx 0.5\ \varepsilon_y\ A_{rb} \qquad \ldots 7(58)$$

where A_{rb} is the mean aspect (span/depth) ratio of the beams.

- *The examination described above addresses only the wall base and the storey subjected to the largest nominal yield drift. At this stage the behaviour of any other part of the structure is not critical and hence of no interest. Figure 7.19(c) shows the typical deflected shape of the critical wall, ie, the one with smallest aspect ration, A_{re}, such as in the example illustrated in Figure 7.19.*

Displacement estimates, illustrated in Figure 7.19(c), furnish the following information:

- *U_{wy}, the nominal yield displacement of the wall element, obtained with eqn 7(39). It enables the displacement ductility capacity, corresponding with the acceptable maximum displacement of the system, U_{max}, to be quantified.*
- *The nominal yield displacement of the frame element, corresponding to near identical nominal storey drifts, δ_{fy}, obtained with eqn 7(54), is associated with a deflection profile shown by the dashed lines in Figure 7.19(c). It has been assumed that at and above level 8, beams shallower than those at lower levels have been used, ie, the aspect ratio of those beams, used in eqn 7(54), is larger.*
- *It is evident that in this example system, the onset of yielding of frame elements can be expected only after the displacement ductility demand on the wall element approaches 2.*
- *The acceptable displacement ductility capacity of the wall $\mu_{wc} = U_{max}/U_{wy}$ must be based on judgement, derived from the study of the details, shear and curvature ductility capacities of the walls. The latter, given in Figure 7.18, is not likely to be critical, unless the effective aspect ratio, A_{re}, of the wall with the greatest length, l_w, of is excessive. If necessary, the curvature ductility demand at the base of the wall due to the post-yield wall displacement, U_{wp}, may be estimated (Paulay and Priestley 1992, Paulay 2001b). Alternatively established recommendations (SNZ 1995), based on eqn 7(35), may be used. The associated storey drift in the vicinity of the effective height, h_{eff}, may be estimated from:*

$$\delta_{max} \approx \delta_{wy} + U_{wp} / h_{eff} \qquad \ldots 7(59)$$

and compared with benchmark values, such as 2.5%. The sensitivity of storey drift with respect to the grade of steel used is emphasised in Figure 7.17.

- *A comparison of displacements at the level of the effective height, shown in Figure 7.19(c), clearly demonstrates that displacement ductility demands imposed on the frames, will be moderate, unless the walls fail.*

Step DD5 : The stiffness and displacement capacity of dual systems

From the contributions of the different lateral force resisting elements to the probable base shear strength, V_{prob}, and the nominal yield displacement at the effective height of the system, U_y, determine its stiffness. When using a strength-based assessment approach, estimate the period T, of the building based on its stiffness

$$k_s = V_{prob} / U_y \qquad \ldots 7(60)$$

and evaluate the expected ductility demand μ_{sd}.

When using a displacement-based assessment approach, the effective stiffness of the dual system is estimated by:

$$k_{eff} = V_{prob} / \mu U_y = k_s / \mu_{sc} \qquad \ldots 7(61)$$

This enables the effective period and the structure displacement demand to be evaluated as shown in section 6.3.

Details of this step, based on bilinear modelling of force-displacement relationships, similar to that shown in Figure 7.5, are summarised with the aid of Figure 7.20. It represents the expected behaviour of the example structure considered in Figure 7.19. To illustrate the simple details of calculations, certain specific assumptions needed to be made.

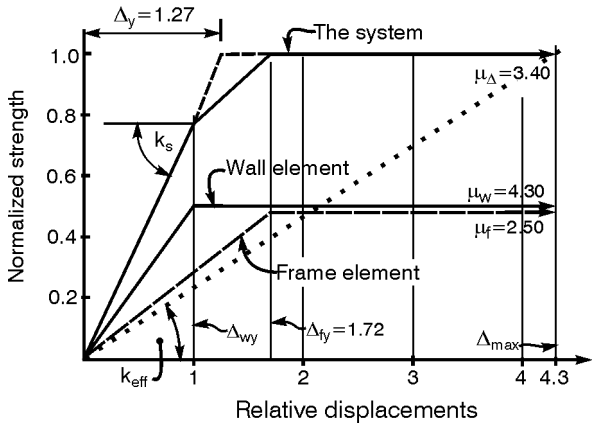


Figure 7.20: The bilinear simulation of the force-displacement relation of a dual system and its two elements

As figure 7.19(b) shows, approximately 50% of the probable base shear strength of the system, V_{prob}, was found to be provided by each the wall and the frame element. The relative nominal yield displacements at level h_e, were found to be for the wall element: U_{wy} = 1.00 and the frame element U_{fy} = 1.72 displacement units. Therefore, the normalised stiffness of these elements are from eqn 7(45): $k_w = V_{wp}/U_{wy}$ = 0.5/1.0 = 0.5 and k_f = 0.5/1.72 = 0.29, respectively. Hence from eqn 7(46) the relative nominal yield displacement of the dual system is U_y = 1.00/(0.5 + 0.29) = 1.27 displacement units. The bilinear idealisation of element and system behaviour, shown in Figure 7.20, records these quantities.

It is assumed that the examination of the details of the wall element resulted in the estimation of its displacement ductility capacity being $\mu_{wc} \approx 4.30$. This then determines its displacement capacity, and hence that of the system, $U_{max} = \mu_{wc} U_{wy}$ = 4.3 units. Because the nominal yield displacement of the dual system is U_y = 1.27 units, the system displacement ductility capacity, an important parameter of a strength-based assessment procedure, is reduced to μ_{sc} = 4.3/1.27 = 3.4.

A displacement-based assessment of the dual system would be based on its displacement capacity, U_{max}. *The equivalent period of vibration of the ductile dual system will then correspond with its effective stiffness*

$$k_{eff} = V_{prob} / U_{max} = V_{prob} / \mu_{sc} U_y = k_s / \mu_s \qquad \ldots 7(62)$$

This condition is simulated in Figure 7.20 *by the diagonal dotted line.*

Step DD6 : Compare structure displacement capacity against demand

In both the strength-and the displacement-based assessment procedure, the displacement and displacement ductility capacities of the dual system were established in the same manner. In a strength-based procedure displacement ductility demands and capacities need to be compared, to establish whether retrofit is required. In a displacement-based procedure the displacement capacity of the system is compared with the expected displacement demand, as outlined in section 6.3.

If strength eccentricity arises, a reduction of the base shear capacity of the system, as defined in Step WF7, section 7.4.2, may be necessary to safeguard critical wall elements against excessive displacement demands.

Virtually identical approaches to displacement capacity and demand comparisons in dual systems were made possible by the recognition that the stiffness of components of ductile reinforced concrete systems are proportional to their probable strength, as defined by eqn 7(45). Stiffness in displacement focused procedures should not be made dependant on strength-independent fractions of traditionally defined flexural rigidities, E_cI, *of components.*

Torsional phenomena in dual systems, due to eccentricity of the total nominal strength of the system, V_{prob}, *may affect wall elements situated at the boundaries of the plan. Wall cores, situated close to the centre of mass, shown as CM in Figure 7.16, are not likely to be significantly affected by system rotations. Additional displacement demands on frames, particularly when situated close to the boundaries of the floor plan, are not likely to jeopardise their displacement capacity because of the moderate translatory displacement ductility demands imposed on frames of dual systems.*

Section 8 - Detailed Assessment of Steel Structures

8.1 Introduction and Scope

8.1.1 Scope

This Section provides detailed guidance on the evaluation of moment-resisting steel framed systems without infill panels and conceptual guidance for evaluation of MRSFs with infill panels and for braced steel systems.

The following are covered:

- Material properties and member strengths (Section 8.2)
- Philosophy and assumptions for the evaluation of existing steel seismic-resisting systems (Section 8.3)
- Assessing member and connection strength and rotation capacity (Section 8.4)
- Evaluation procedure for moment-resisting steel framed systems (Section 8.5)
- Reporting of results from the moment-resisting steel framed system evaluation (Section 8.6)
- Evaluation of moment-resisting steel framed systems with infill panels (Section 8.7)
- Evaluation of braced framed buildings (Section 8.8)

These are also two appendices, containing the following information:

- Determining the moment-rotation characteristics of bolted or riveted joints (Appendix 8A)
- Simplified pushover analysis for use in evaluation (Appendix 8B)

All sections must be applied using sound engineering judgement, as a considerable degree of expert assessment is required in making the evaluation of existing systems.

These Guidelines build upon the first material published on this topic in February 1996, with details from it that are still relevant incorporated into this material (Clifton 1996a).

8.1.2 Useful Publications

The following publications will be of particular assistance to designers making a seismic assessment, which will of necessity form part of a more general assessment of the condition of the building. The first is available on loan or to purchase from HERA.

(1) Seismic design of steel structures, HERA report R4–76 (Feeney and Clifton 1995), in conjunction with Clifton (2000).

(2) NZS 3404 (SNZ 1997) and NZS 1170.5:2004 (or NZS 4203:1992).

8.2 Material Properties and Member Strengths

In the assessment of an existing structure, realistic values for the material properties, particularly strengths, must be used to obtain the best estimate of the strengths and displacements of members, joints and connections.

Material properties and strengths that were specified in the original design are not appropriate for use in assessment procedures.

The effect of variations in material strength on the hierarchy of failure must be considered.

The material strengths used are to be as defined below.

Definitions of Material Strengths

General definitions of material strengths are given in Section 4.7.

Specific guidance is given in the procedures presented below.

8.3 Evaluation Philosophy and Assumptions

8.3.1 Approach to be Used for the Evaluation of Existing Steel Seismic Resisting Systems

The evaluation approach is as follows:

(1) assess the probable strength (flexural and/or axial as appropriate) and rotation capacity available from the individual members and connections of the seismic-resisting system

(2) assemble the probable strengths of the components and members to obtain the strength hierarchy of the system, making allowance for foundation strength and stiffness limitation on the strength hierarchy

(3) determine the actual ductility demand on the system that is required to match the seismic actions generated by the required strength assessment limit (from Section 5 for moderate/high risk determination) with the first yield strength available from the system

(4) determine the inelastic deflection limit for the system, if required

(5) check the strength and ductility of the system in the inelastic range, if required

The evaluation of the actual ductility demand on the seismic-resisting system is made in accordance with the force-based design procedure of Section 6, with the method of analysis used depending on some of the features exhibited by the system.

If this evaluation shows that $\mu_{act} > 1.0 - 1.5$ is required to reduce the seismic actions generated by the required strength assessment limit to the first yield strength available from the system, then the performance under the inelastic regime of behaviour needs to be evaluated. This means that the evaluation procedure covers the system's performance in the inelastic regime, when required. The ductility factor trigger for this check is contained in the appropriate section, eg. in section 8.5.8 for moment-resisting framed systems.

8.3.2 Assumptions for the Evaluation

These are as follows.

a) The form of the connections is such that their flexural strength at first yield and their elastic and post-elastic stiffness can be determined by rational assessment.

b) The steel members consist of either solid I-sections or sections built up by plates, and connected by rivets, bolts or welds, where the strength of the connectors can be determined by rational assessment.

c) The member sizes and connection details can be ascertained with sufficient accuracy to undertake (a) and (b) above. This will typically require the engineering drawings to be available, giving critical details, or else the non-structural and concrete encasement surrounding these joints will need to be removed to allow the assessment to be made.

d) Concrete encasing to the steel frame is designed to fulfil a fire protection role only and is not sufficiently reinforced to contribute significantly to the MRSF strength or stiffness. If the concrete encasing is well-reinforced and likely to contribute to the strength and stiffness of the steel frame, then some of the details presented herein will need to be modified.

e) The designer has access to HERA Report R4–76 (Feeney and Clifton 1995), in addition to NZS 1170.5 and NZS 3404:1997. Until R4–76 is updated to formally be used with the 1997 edition of NZS 3404, information presented in Clifton (2000) will also be of assistance on adapting R4-76 for use with NZS 3404:1997. Reference to the various HERA *Steel Design and Construction Bulletins* e.g. Clifton GC (ed) will provide further useful information.

f) In assessing the strength of elements, the strength reduction factor (ϕ) is set to 1.0 and the minimum material strengths are used. This means that each member / component will typically be slightly stronger than is determined by these guidelines. Appendix 4A gives nominal mechanical properties to use for the steel members and components.

8.4 Assessing Member and Connection Strength and Rotation Capacity

8.4.1 General

The assessment of member and connection strength and rotation capacity is applied to components of moment-resisting and braced systems. This is applied in accordance with the recommendations of HERA Report R4–76 (Feeney and Clifton 1995) for preliminary design of seismic-resisting systems. The results are carried forward into the evaluation of each type of system from the appropriate Sections 8.5 to 8.8.

The assessment, at least for preliminary evaluation, should be undertaken at every fourth floor for a building over 12 storeys (including the roof) in height, or at every third floor for a building over 4 and up to 12 storeys (including the roof) in height, or at every second floor for a building up to 4 storeys (including the roof) in height. In each case the assessment should start at the first level above seismic ground level.

Seismic ground level or “base” level is the level at which the seismic load is first considered to be transmitted directly sidesways (wholly or in part) into the surrounding ground.

The uppermost principal seismic mass level should be included in this assessment.

When the strength hierachy at each level under assessment is determined (see, for example, section 8.5.2, if this shows the hierachy to be the same at all levels then the above will be sufficient. If, however, it changes over the levels, then further levels will need to be included in the assessment until the designer is satisfied that the strength heirachy throughout the seismic-resisting systems is known.

8.4.2 Force Transfer through Connections

The force transfer through the connections must be carefully assessed, weak links determined and their strength and ductility evaluated.

The first stage of this involves determining the load path through the connection. This requires engineering judgement. The general principles given in Sections 10.4 and 10.5 of HERA Report R4–80 (Clifton 1994) will be of assistance in this evaluation.

The following advice will also be of assistance in determining the load path and the weakest link in that path.

1 Determine the internal forces generated in the attached members by the earthquake.

a) An I-section beam not responding inelastically under moment will deliver axial forces through the flanges (tension and compression) and vertical shear through the web.

b) An I-section beam responding inelastically under moment will deliver axial yield forces through the flanges and axial yield forces plus vertical shear through the web.

c) A brace will deliver axial forces (tension is critical) through all its elements.

2 Trace the transfer of forces from elements of the supported member into elements of the supporting member that lie parallel to the incoming force. For example, the incoming axial forces from an I-section beam flange connected to an I-section column must be transferred through the column flange into the column web.

3 Calculate the nominal capacity of all elements along this load path, in accordance with the general assessment provisions of Section 8.4.3. When doing this, note the following.

a) If there are no tension and compression stiffeners in columns adjacent to incoming beam flanges in a moment-resisting beam to column connection, then tensile distortion of the column flange or compression buckling of the column web are likely to occur before the beam can develop its section moment capacity. The former can be assessed using Section 10.9.2 of Clifton (1994), the latter using Clause 5.13 of NZS 3404:1997.

b) The load path may be quite complex. For example, with regard to the load path for the tension force from the beam flange shown in Figure 4A.2, Appendix 4A, into the column web; this involves the following:

- transfer in shear from beam flange to rivets between flange and RSJ connector web
- tension in the RSJ connector web at the minimum cross-section (Line B on Figure 4A.2)
- shear in the of web at the web/flange junction of the RSJ connector
- shear in the rivets between the flanges of the RSJ connector and the gusset plate A which forms the side wall of the column
- local tension in the gusset plate A.

4 The capacity of the load path is determined by the capacity of the weakest component in the load path.

5 The ductility of the load path is determined by the ductility of the weakest component in the load path.

6 If various load paths exist then the stiffest of these will attract the most force.

7 Be particularly aware of situations where the connectors (rivets, bolts or welds) may be the weakest component, as their ductility capacity will be limited. One sided fillet welds in tension or bending are particularly vulnerable in this regard, showing no ductility.

8 Be aware of component forces introduced when an applied force must change direction along the load path.

If one is dealing with a connection of the type shown in Figure 4A.2, then the guidance given in Section 4.4 of Design and Construction Bulletin (*DCB)* No. 18 (Clifton 1996a) will be of assistance in determining the connection flexural strength and ductility.

General provisions for the most common generic form of bolted or riveted beam to column connection are given in Appendix 8A.

The paper by Blodgett (1987) will be of assistance in explaining the concept of load path and illustrating it with various examples.

8.4.3 General Assessment of the Capacity of Connection Elements and Connections

This assessment should be made in accordance with the following:

1 Shear capacity of rivets can be determined from Barker (2000). The key equation is derived from the bolt shear capacity provisions of NZS 3404 and is:

$$V_f = 0.75 f_{uf} k_f n_x A_o \qquad \ldots 8(1)$$

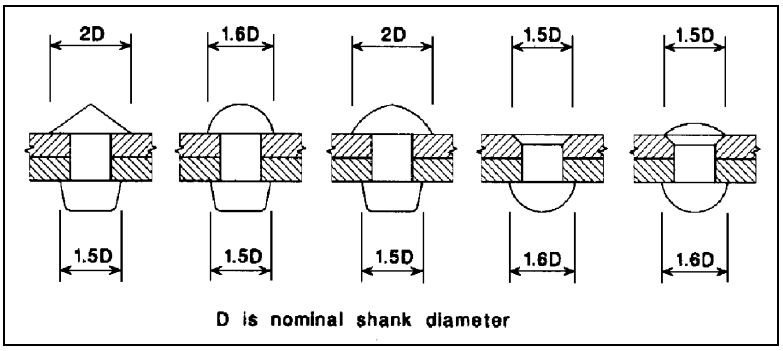
where V_f = nominal shear capacity of rivet. All other notation is from NZS 3404.

2 Tension capacity of rivets is determined using Clause 9.3.2.2 of NZS 3404:1997, with the value of f_{uf} determined from Appendix 4C herein.

3 Assess the diameter of the rivet shank from the diameter of the rivet head in accordance with Figure 8.1.

4 Be aware that some less scrupulous erectors made up some dummy rivets from moulded putty covered in paint on larger groups of rivets. Hitting each rivet with a hammer will soon identify any dummy ones!

5 Assume that concrete encasement, if present and with any amount of confining reinforcement, will prevent local buckling of the steel members. This assumption may not hold for members in regions subject to significant inelastic demand and will need to be assessed more closely for such regions. (See Figure 4B.2, Appendix 4B for such an example, caused by the lack of a tension / compression stiffener in the column adjacent to an incoming rigid welded beam flange.)



Source: Bussell (1997).

Figure 8.1: Typical rivet shank and head diameters

6 For connections of the type shown in Figure 4A.2, Appendix 4A, involving two or more steel plates across the joint subject to major axis shear and bending and confined within a reinforced concrete surround, assume that the joint panel zone will remain elastic or nominally elastic under out of balance shear force induced by the out of balance moments generated by the connection. For the sub-assemblage shown in Figure 4A.2, this has been confirmed by inelastic cyclic testing (Wood 1987).

For connections involving a panel zone web more typical of modern details, determine the nominal panel zone shear capacity from NZS 3404 Clause 12.9.5.3.2. Designer judgement may be required for this.

7 In calculating the connection capacity, assume that:

- the connections to the beam flanges develop and transfer the moment-induced axial force from the beam to the column
- the connections from the beam web to the column transfer gravity and seismic-induced vertical force and also will transfer horizontal actions if a suitably stiff and strong horizontal load path from the beam web into column is available
- if the connection has a direct connection between beam web and column via welded or bolted plates or cleats, with this connection separate to the beam flange to column connection, then for seismic assessment the vertical shear capacity can be assumed to be adequate.

8.4.4 Bolted and Riveted Connections

For connections of the form shown in Figures 4A.1 and 4A.2, use the procedure for moment rotation determination given in Appendix 8A. This is largely based on the experimental work of Roeder et al (1994). This gives connection moment-rotation capacity in the elastic and inelastic regimes, along with a commentary on the derivation of the curve. For the particular connection shown in Figure 4A.2, use Appendix 8A.2 in conjunction with (Roeder et al, 1996 and Clifton, 1996a).

For vertical load carrying capacity, use the provisions of R4-100 (Hyland 1999) to determine the capacity of the beam web to column connection, ignoring the effect of moment on the connection in reducing the shear capacity when making this check.

For other bolted and riveted connections, determine the strength and rotation capacity from first principles using the guidance from Appendix 8A and (Roeder et al, 1996).

8.4.5 Welded Beam Flange to Column Connections

Check if the welded connection can transfer the moment-induced beam actions into the column, as detailed in Sections 3.1 and 3.2 of *DCB* (Issue No. 50, pp. 12–14). If so, then the connection can develop the moment capacity of the incoming beam. If not, as would be the case for an unstiffened column, then assume that the weld at the beam flange will fail early in a severe event and determine the moment capacity as for a semi-rigid connection, to section 8.4.2 and Appendix 8A, based on the moment capacity of the connection to the beam web.

Using the provisions of (Roeder et al, 1996) in conjunction with Appendix 8A, the moment-rotation curve can be constructed as follows:

The general shape of the curve takes the form of Figure 8A.2 with

- θ_y = 3 milliradians
- θ_{p1} = as given by eqn 8A(10)
- θ_{p2} = (θ_{p1} + 5) milliradians …8(1)
- $M_{y,bare}$ = $N_{tfw}d_b$ …8(2)

where:
d_b = depth of beam (m)
N_{tfw} = nominal tension capacity of the weld between the beam flange and the unstiffened column flange, as given by Section 10.9.2 of Clifton (1994).

- $M_{y,encased}$ = $1.3M_{y,bare}$ …8(3)
- For rotations greater than θ_{p2}, $M = M_{y,web}$. …8(4)
- $M_{y,web}$ = moment capacity of the beam web to column alone. This is governed by the moment capacity of the beam web to column connection and needs to be determined from the particular connection detail used. The ranges in web moment capacity are from:

  (i) For webs connected with clip angles, eg. as shown in Figure 4.9.8, use the capacity of that connection, from equation (5) or (6) of Appendix 4.9A.
  (ii) For webs connected with balanced, double sided fillet welds or butt welds of sufficient strength to yield the web in tension, the moment, $M_{y,web}$ = plastic moment capacity of the beam web.

- $M_{y,web}$ is taken as constant from θ_{p2} to θ_u = 40 milliradians.

If the connection is suspected of being welded but is not visible, due to e.g. concrete encasement and with no design or shop drawings being available, then the encasement material must be removed from a representative joint to allow a reasonable assessment to be made. The difference in connection moment-rotation capacity between a joint that can transfer the flange axial forces induced by inelastic beam action dependably into the column and one that cannot is so great that this must be assessed and not guessed.

Similarly the existing state of the weld needs to be assessed using visual inspection techniques; engineers doing this should be familiar with the visual inspection techniques (see Hayward and McClintock 1999) for details.

8.4.6 Member Strength and Rotation Capacity

a) Beams

Bare steel beams, solid sections

i) Use NZS 3404 (1997) Section 5.2 to determine the section status (compact, non-compact, slender) and hence the section moment capacity, M_s or member moment capacity, M_b. Use the former for a beam supporting a concrete slab, and the latter for beams supporting a timber floor. In the timber floor case, the restraint offered can be determined using R4–76 (Feeney and Clifton 1995) and R4–92 (Clifton 1997). In 1(d) below, M_s is used to denote either M_s or M_b as appropriate.

ii) Use NZS 3404 Clause 12.4 and 12.5 to determine the highest possible member category, then:

iii) Use NZS 3404 Table 4.7 (2) to obtain θ_p.

iv) Construct the moment-rotation curve (θ_i, M_i) from the following points:

$(0,0)$; (θ_y, M_s); $(\theta_y + \theta_p, M_s)$

$(\theta_y + 1.25\ \theta_p, 0.5\ M_s)$; $(\theta_y + 1.5\ \theta_p, 0)$

where $\theta_y = 3 \times 10^{-3}$ radians.

3 milliradians is taken as a reasonable first yield rotation for a steel member.

Concrete encased steel beams, solid sections

i) Assume that the concrete encasement suppresses local buckling and provides slight strength enhancement and hence that $M_s = 1.1\ S\ f_y$, with S determined in accordance with NZS 3404 Clause 5.2.3 (see Section 5.2.5.2 of Clifton (1994) for guidance on calculating S).

As stated in section 8.3.2 above, the concrete encasement is assumed not to contribute significantly to the member flexural strength. For typical levels of longitudinal and transverse reinforcement in concrete-encased steel frames and concrete strength/quality, this is realistic (NZS 3404:1997).

ii) Use NZS 3404 Table 4.7(2) to obtain θ_p for member category 2.

iii) Construct the moment-rotation curve from the same points as given in 1(d) above.

b) Columns

Bare steel columns, solid sections

i) Use NZS 3404 (1997) Sections 5.2 and 8.2 to determine the section status (compact, non-compact, slender) and hence the section moment capacity reduced by axial force, M_r.

ii) Use NZS 3404 Clauses 12.4 and 12.5 to determine the highest possible member category, then:

iii) Use NZS 3404 Tables 4.7(2) to 4.7(4) to obtain θ_p.

iv) Construct the moment-rotation curve (θ_i, M_i) from the following points.

- If the member has full lateral restraint
  $(0,0)$; (θ_y, M_r); $(\theta_y + \theta_p, M_r)$; $(\theta_y + 1.25\ \theta_p, 0.5\ M_r)$; $(\theta_y + 1.5\ \theta_p, 0)$

- If the member does not have full lateral restraint
  (0,0); (θ_y, M_o); (1.5 θ_y, 0.5 M_o); (2θ_y, 0)

where:
θ_y = 3 x 10^{-3} radians
M_o = member moment capacity reduced by axial force, N_g^*, to NZS 3404 Clause 8.4.4
M_r = section moment capacity reduced by axial force, N_g^* .

The axial force used in calculating M_r and M_o shall be that from the gravity load associated with earthquake action, i.e. N^*_{G+Qu} . The seismic contribution shall be ignored.

The principal reason for this is because experimental tests (MacRae 1990; Brownlee 1994) have shown that the inelastic behaviour and rotation capacity of a steel beam-column subject to compression and major axis bending is dependant on the magnitude of constant compression force, i.e. that from N^*_{G+Qu} , rather than on the total compression force, which includes the seismic component.

This simplifies the determination of (θ_i, M_i) with little loss of accuracy for columns that are resisting relatively low levels of vertical force, especially the non-seismic component of vertical force. This is typically the case for columns in pre-1976 steel MRSFs.

Concrete encased steel columns, solid sections, little change in cross-section area or moment of inertia of the encased steelwork within a storey height

i) If the concrete encasement complies with the requirements of NZS 3404/NZS 3101:1995 for composite column action (see NZS 3404:1997 Clause 13.8.2), then make the assessment accordingly. In pre-1976 buildings, this is very unlikely.

If the longitudinal and transverse reinforcement levels are much lower than required by (NZS 3404:1997) and (NZS 3101:1995) and the longitudinal reinforcement is plain bar, which is likely in older buildings, then:

- base the nominal moment capacity on that for the steel elements only
- assume that the concrete encasement suppresses local buckling of the encased steel elements and lateral buckling for moment. However member buckling in compression needs to be considered in accordance with Clause 6.3 of Wood (1987), with the effective length factor, k_e ,=1 in accordance with NZS 3404 Clause 12.8.2.4.

ii) Use NZS 3404 Tables 4.7 (2) to 4.7 (4) to obtain θ_p for category 2.

iii) Construct the moment-rotation curve (θ_i, M_i) from the following points:

(0,0); (θ_y, M_r); (θ_y + θ_p, M_r); (θ_y + 1.25 θ_p, 0.5 M_r); (θ_y + 1.5 θ_p, 0)

where θ_y = 3 x 10^{-3} radians.

Concrete encased steel columns, laced and battened sections or solid sections with significant change in the cross-section area or moment of inertia of the encased steelwork within the storey height

i) Determine the nominal section moment capacity for the steel elements only.

ii) Determine the nominal member compression capacity for the steel elements within the section from NZS 3404 Clauses 6.4 or 6.3.4 as appropriate.

iii) Use NZS 3404 Tables 4.7(2) to 4.7(4) to obtain θ_p for category 3.

iv) Construct the moment-rotation curve (θ_i, M_i) from the following points:

(0,0); (θ_y, M_r); ($\theta_y + \theta_p$, M_r); ($\theta_y + 1.25\ \theta_p$, 0)

where θ_y = 5×10^{-3} radians for encased laced and battened sections which exhibit greater elastic flexibility than encased solid sections.

= 3×10^{-3} radians, for encased solid sections.

8.5 Evaluation Procedure for Moment-Resisting Steel Framed Systems

8.5.1 General

The evaluation procedure of this section is aimed at determining whether a system has sufficient first yield capacity to resist the seismic actions generated by the required strength assessment limit. This limit is given in Section 6 as a specified *%NBS*. The seismic actions for this valuation may be further reduced by the ductility capacity of the existing system. This involves determination of the actual ductility demand, μ_{sd}. For strong column / weak beam or weak joint systems, a hand procedure for rapid determination of μ_{sd} is given in NZS 3404 Commentary Clause C12.3.2.3.2.

However, if this assessment shows that $\mu_{sd} > 1.0 - 1.5$ is required, then the influence of inelastic response must be considered. This influence will be less significant on systems exhibiting the following "good features":

a) The strength hierachy (see section 8.5.2 at all levels (except for the uppermost seismic mass level) is to be beam sidesway (ie. weak beam or weak connection) rather than column sidesway.

b) For weak connections, the evaluation of the connection in accordance with Section 8.4 must show the following:

- For elements on the principal load-carrying paths through the connection (these paths will have been determined in section 8.4.2; refer also to NZS 3404 Commentary Clause C12.9.1.2 for guidance on what constitutes the "principal load-carrying path") the weakest component must not be a connector (weld, rivet, bolt), nor involve net tension failure of a component.
- The connection must be able to retain its integrity, with regard to carrying shear and axial force, when its moment capacity is reduced.

c) For all beam to column connections, the connection must not be of a type that has the potential to introduce local buckling or tearing failure in the column (eg. through having no column stiffeners adjacent to an incoming beam flange in a welded beam to column connection – see Figure 4B.2 for an example of this).

d) The assessed inelastic response of the system (this assessment is qualitative rather than quantitative) must be essentially symmetrical in nature and must not contain features that will inevitably lead to a progressive displacement of the building in one direction.

Systems where these features are present gain advantages in terms of first yield assessment and inelastic response evaluation, as given in section 8.5.4.

8.5.2 Determine Strength Hierachy of System

a) Assemble the nominal flexural strengths of the individual connections, beam and column members for each level that has been considered.

Use the guidance in section 8.4.1 to select the number of levels to consider when doing a preliminary evaluation, noting that the number of levels being evaluated may need to be increased in accordance with (f) below.

b) The flexural strength at that level is governed by the weakest of the individual elements. In many instances, this will be the connections.

c) The location of these weakest elements will be the yielding regions. They are the primary elements at that level.

d) Determine if the individual beams of the MRSF at each level under consideration can support the moments from long-term gravity loading ($G + Q_u$) on them in a simply supported condition. If they can't, then halve the plastic rotation capacity from Section 8.4.6 (a), for the beams and that from Section 8.4.6.(b) for the connections. This reflects the monotonic, cumulative nature of inelastic demand on the yielding regions of such members.

e) On the basis of the relative strengths of the frame elements and the location of the weakest elements, determine if the MRSF response will be a beam sidesway or column sidesway mechanism. Note that semi-rigid connections, where these connections are flexurally weaker than the beams or columns, generate a beam sidesway mechanism. Designer judgement is required here. (If the designer has any doubt about this assessment, see Sections 3.5 and 4.1–4.3 of HERA Report R4–76 (Feeney and Clifton 1995) for further guidance on this.)

f) The strength hierachy may change from column sidesway to beam sidesway at different levels with the same system. If this occurs for the levels being checked from (a) above, then the strength hierachy for all levels within the system needs to be determined and the worst case used. This will almost always be the column sidesway mechanism.

g) If the difference in strength between a column sidesway mechanism and a beam sidesway mechanism is less than 15%, then the effect of both needs to be determined.

8.5.3 Allowance for Foundation Strength and Stiffness

The foundation system must be able to transfer the seismic forces between superstructure and ground and to resist any anticipated ductility demands.

A strength assessment of this should be made, involving:

- selecting a dependable load path for transmitting the earthquake plus associated gravity design actions between superstructure and ground
- checking the adequacy of all components along this load path to resist these actions and to sustain any anticipated ductility demands.

The stiffness of the foundation should be modelled as an elastic spring at the column base. Its stiffness can be determined on the basis of its pinned or fixed status, from the strength determination, using NZS 3404 Clause 4.8.3.4.1(a) or (b) as appropriate.

The shear resistance of the foundation needs to be greater than that of the column member; design and detail, if necessary, to achieve this.

8.5.4 Determine the Structural Ductility Factor necessary to meet the Design Seismic Actions Generated by the Required Strength Assessment Limit

a) For systems that exhibit the four "good features" from section 8.5.1, the equivalent static method of analysis from NZS 1170.5:2004 may be used for this determination.

b) For systems that do not exhibit these four "good features" from Section 8.5.1, modal analysis (or numerical integration time history analysis) should be used for this determination.

c) Where appropriate, adjust the seismic design actions in systems with riveted or bolted joints, to account for the magnitude of initial viscous damping. This adjustment may be made for $\mu_{sd} \leq 1.5$ and requires an assessment as to whether that will be met for the initial evaluation. The appropriate damping value is 10%. The adjustment is made in accordance with Clause 12.2.9.2 of NZS 3404.

d) For modal analyses:
- Model the system using the elastic properties of the components from Section 8.4. Use this to obtain the period and the modal participation factors
- Determine the member actions required for the combined modes (SRSS method can be used) and for the first mode response alone.

e) Choose a starting value of μ and compare the member actions from the analysis, for the given value of μ, with the member nominal yield capacities from Section 8.4

i) Increase or decrease the value of μ until the lowest value of μ is obtained for which no components are significantly beyond their nominal yield capacities. Significant means > 10% over nominal capacity for any one component and > 5% for all components in any one storey or at any one level.

ii) The value of μ must be between $1 \leq \mu \leq 6\ C_s$.
where C_s = reduction factor from the 100%*NBS*.

As illustrated in section 4.2.1(b), $C_s = 0.33$ for determining the high risk category, and $C_s = 0.67$ for determining the moderate risk category, or for strengthening of a building which fails the high risk check.

iii) The lowest value of μ for which 8.5.1 is achieved is the actual structural ductility factor, μ_{sd}, for thesystem, in order to meet the required strength assessment limit.

If all the four "good features" of section 8.5.1 are present and $\mu_{sd} \leq 1.5$, then no further steps in this evaluation are required and the system passes the evaluation.

If any of the four "good features" of section 8.5.1 are not present and $\mu_{act} = 1.0$, then no further steps in this evaluation are required and the system passes the evaluation.

If neither of these conditions is met, then proceed with the evaluation of the system's response in the inelastic regime in accordance with sections 8.5.5 to 8.5.8.

8.5.5 Determine Inelastic Deflection Limit for System

a) *Absolute limit*

This represents the extent of inelastic deflection to which the system is allowed to displace. It is given by the most severe (ie. the lowest limit allowed) of the following:

i) The 2.5% interstorey or building height drift limit given by NZS 1170.5 Clause 7.5.1.

ii) The inelastic limit associated with avoidance of instability failure in components tied into the steel system, such as masonry walls.

In the case of (a), the 2.5% drift limit is that specified by NZS 1170.5 for systems analysed by numerical integration time history (NITH) analysis. It is greater than the limits specified by NZS 1170.5) for equivalent static or modal analysis, because the response in the inelastic range is determined much more accurately by NITH analysis, allowing the lateral displacement limit to be relaxed. The authors consider that the system checks made in this procedure, which involve:

- first yield strength adequacy (section 8.5.4)
- extent of inelastic demand expected (section 8.5.4)
- strength loss in the inelastic range (section 8.5.8)
- ductility capacity of the yielding elements of the system (section 8.5.8)

will ascertain the system's response with similar dependability to a NITH analysis, allowing the NITH inelastic drift limit to be used.

In the case of (b), the appropriate limit will depend on the position of any masonry walls relative to the steel frame, the nature of connection between wall and frame and the extent (if any) of seismic isolation. The limit must be determined from a rational limit state method of analysis for the masonry wall. For example:

- for connected masonry walls that are perpendicular to the plane of the frame and hence subject to out of plane displacements due to the in-plane frame deformations, refer to Section 10.
- for connected masonry walls parallel to the plane of the frame (e.g. infill panels), refer to Section 9.

b) P–Δ OK limit

The P–Δ OK limit of NZS 1170.5 Clause 6.5.2 needs to be determined. This involves use of Equation 6.5(1), incorporating the seismic design actions and level of structural ductility required from Step 8.5.4. This check is only required for systems where μ_{sd} from Section 8.5.4 exceeds 1.5.

If the strength hierarchy of the system (refer to section 8.5.2) is beam sidesway over all levels, then the P–Δ OK limit need be applied only over the lower half of the MRSF.

If the strength hierarchy is column sidesway over any level investigated, then apply Equation 6.5(1) over that level as well as over the lower half of the MRSF.

8.5.6 Making Allowance for P – Δ Actions.

This involves determining the inelastic deflection for the system in accordance with NZS 1170.5 Clause 6.5.4, taking into account whether the system is a column sidesway system or a beam sidesway system. The structural ductility factor to use for this is μ_{sd} for the system, determined

from section 8.5.4. Note that, as explained in section 8.5.2, a system may be strong column over some levels and weak column over others, requiring determination of several possible inelastic deflection profiles in order to determine the most critical case (s).

When $\mu_{sd} > 1.5$ and the P – Δ OK limit from section 8.5.5(b) is exceeded, then the effect of P – Δ actions needs to be considered. This can be undertaken (probably conservatively) as follows:

(a) Determine the additional lateral forces to apply to the system from NZS 1170.5 Supp 1 Clause C6.5.4 Commentary.

(b) For seismic actions from the required strength assessment level that have been obtained by equivalent static analyses, combine the two sets of lateral forces and recheck the first yield adequacy of the system from Section 8.5.4(e). This may result in an increase of μ_{sd}.

(c) For seismic actions from the required strength assessment level that have been obtained by modal analysis, combine the actions from the P – Δ induced axial forces with the modal analysis actions arising from the first mode response, so that the member moments from each are additive. Recheck the first yield adequacy of the system from section 8.5.4 (e). This may result in an increase of μ_{sd}.

The system may be subject to P – Δ influence. If so, then this will generate additional actions in the members. These actions are determined by the simple pin-jointed model equilibrium procedure of NZS 1170.5 Supp 1 Figure 4.6.1. This model assumes a first mode type displaced shape (ie. all levels displace in the same direction, with the displacement of level i + 1 exceeding that of level i). Hence the actions generated by the design lateral forces should be combined with those generated by first mode modal forces, when a modal analysis is used, so that the cumulative effect of these forces is additive.

8.5.7 Determine Inelastic Behaviour of System from a Pushover Analysis

Model the system in an elastic-plastic push-over analysis and push it to the calculated inelastic displacement from section 8.5.6. The set of forces used for this are as derived from section 8.5.4 associated with μ_{sd} from that step, plus any P – Δ actions generated from section 8.5.6. This set of forces is multiplied by a scalar value to push the structure to the required displacement limit.

Track the change in the magnitude of the scalar as the system deflects and determine the inelastic rotation demands on the components of the system.

If the engineer does not have inelastic pushover analysis software which can model the second order effects and input a moment rotation curve which has a descending branch, i.e. of the type shown in Figure 8A.2, Appendix 8A, then use the simplified method given in Appendix 8B herein. This method requires the use of an elastic analysis programme with second order capability, so as to allow for P–Δ influences during this analysis.

8.5.8 Check Stiffness, Strength and Ductility of the System in the Inelastic Range

For any seismic-resisting system being evaluated, if $\mu_{sd} = 1.0$, then the inelastic checks below do not need to be undertaken.

If all the four "good features" of section 8.5.1 are present in the seismic-resisting system being evaluated and $\mu_{sd} \leq 1.5$, then the inelastic checks below do not need to be undertaken.

If either of the above do not apply, then checks for stiffness, strength and ductility of the system in the inelastic range must be undertaken, as follows:

a) Check on the inelastic stiffness adequacy of the system

This is met if the inelastic deflections calculated from section 8.5.6 are within the inelastic deflection limit of section 8.5.5(a).

b) Check strength of the system in the inelastic range

Compare the maximum value of the scalar from section 8.5.6 with its value at the attainment of the maximum deflection limit. If the ratio of the latter to the former is greater than 80%, the system has sufficient strength through the inelastic regime of behaviour.

If the ratio is less than 80%, then the loss of strength is too much in the inelastic range and steps will need taking to increase the inelastic strength of the yielding regions for the expected rotation demands.

c) Check ductility capacity of the yielding elements in the system

Compare the inelastic demands from section 8.5.6 with the inelastic rotation capacity from sections 8.4.3 to 8.4.6 for each element of the system. If the capacity is not exceeded for any element, then the system has sufficient ductility capacity.

If it is exceeded, then the ductility demand is too high and either the ductility capacity of the relevant components must be increased or the system should be strengthened.

8.6 Reporting on Results for Moment-Resisting Steel Frame System Evaluation

These should include:

a) The strengths of the components
b) The primary elements and strength heirachy
c) μ_{act} for the system, including P – Δ influence
d) Inelastic deflection limit
e) The critical inelastic deflection profile
f) Results of the checks for stiffness, strength and ductility of the system in the inelastic range.

8.7 Evaluation of Moment-Resisting Steel Framed Systems with Infill Panels

This section gives comments on the evaluation of existing MRSF buildings with infill panels. These are as follows.

a) With bare steel or encased solid section columns, the increased column shear demand from infill panels is not likely to be a problem. This should be confirmed with a check on a critical column.

With an encased laced and battened member, this may not be the case and all such members should be checked using the shear capacity of the steel elements plus that of the concrete.

b) The infill panels should be included as equivalent diagonals or shear spring components, with their elastic yield limit and inelastic strength, stiffness determined in accordance with Section 4.10.

c) Determination of system actual ductility demand and response in the inelastic range then follows Sections 4.9.4 and 4.9.5, but with these infill elements included.

8.8 Evaluation of Braced Buildings

8.8.1 Lessons Learned from Observed Building Behaviour in Severe Earthquakes

In terms of existing braced steel framed seismic-resiting systems, the following general statements are applicable.

a) For the era of buildings covered by this document (pre-1976), braced buildings means concentrically braced frames (CBFs) and, typically, X-braced CBFs. There were no eccentrically braced frames (EBFs) built in this era and very few V-braced CBFs.

b) With CBFs in these buildings, typically the design lateral forces used at the time were low compared with current requirements and either:

- the braces are much weaker than the remainder of the CBF system (i.e. columns and collector beams) or more likely
- the connections between braces, beam and column cannot develop the actions that would be generated in the brace and therefore will fail before the braces yield.

c) The effect of (b) is that, without remedial work, brace failure is expected in a severe earthquake. There were many examples of this reported from Kobe (Clifton 1996b). Where brace failure occurred over most or all storeys of the CBF, this did not result in building collapse and the post-earthquake state of the building (damaged but standing) would have met the performance criteria of these Guidelines, especially where the buildings were relatively squat.

Figures.4A.4 and 4A.5 in Appendix 4A show examples of damaged braced framed buildings subjected to the Kobe earthquake. In these instances, the bracing suffered with some connection failures over all levels, and in the case of the building in Figure 4.4A.5, the building width was greater than its height. Figure 4A.5 shows an example of damage to a slender, tension braced frame building where the semi-rigid frame has limited but quantifiable rotational capacity, once the brace to frame connections have failed.

Figure 8.2: Slender concentrically braced framed building with failure of brace to frame connections, Kobe earthquake

8.8.2 Evaluation Procedure for Concentrically Braced Framed Systems

Designers should be familiar with the general concepts of CBF seismic behaviour prior to commencing this assessment. Read Section 14 of HERA Report R4–76 (Feeney and Clifton 1995).

The evaluation procedure is as follows:

1 Determine the flexural strength and rotation capacity of the beam and column members and the connections between the beam and column, using the provisions in Section 8.4.

2 Determine the axial force transfer capacity of the connections, including brace to beam/ column, beam to column and column splices.

3 Determine the nominal tension capacity of the brace members.

4 Determine the strength hierachy of the system, by means of:

- assemble the nominal strengths of the individual connections, beam, column and brace members for each level that has been considered
- determine the weakest component and expected mode of failure, i.e. brace, brace connection, column
- determine if the collector beam can support the moments from long-term gravity loading ($G + Q_u$) in a simply supported condition.

5. Check if the following good features are present:

- The strength hierachy involves weak brace at all levels except the uppermost seismic level (rather than weak column or weak collector beam)

- The collector beams, columns and the beam to column connections all have sufficient capacity to resist the loads generated by the system at the point of brace failure. This is checked by using the capacity of the braces or brace to frame connections, whichever is the least, in CBF system design provisions of R4-76 (Feeney and Clifton 1995) Section 16-18 as appropriate. Contact HERA for advice on this, if required. For many older braced buildings, this will not be met; the brace to beam/column connections will be the weakest component.

- For all beam to column connections, the connection must not be of a type that has the potential to introduce a local buckling or tearing failure in the column under inelastic rotation (eg. as shown in Fig.4B.2, due to lack of column tension/compression stiffeners).

- The assessed inelastic response of the system (this assessment is qualitative rather than quantitative) must be essentially symmetrical in nature and not contain features that will inevitably lead to a progressive displacement of the building in one direction.

6. Perform an elastic analysis on the system, using the required strength assessment level of seismic loading from Section 5.

   i) If all the good features of step 5 are present, then the equivalent static method of analysis from NZS 1170.5:2004 may be used for this determination.

   ii) If any of the good features of step 5 are not present, modal analysis (or NITH analysis) shall be used for this determination.

7. Where appropriate, adjust the seismic design actions in systems with riveted or bolted joints to account for the magnitude of initial viscous damping. This adjustment may be made for $\mu_{sd} \leq 1.5$ and requires an assessment as to whether that will be met for the initial evaluation. The appropriate damping value is 10%. The adjustment is made in accordance with Clause 12.2.9.2 of NZS 3404.

8. Compare the member actions from the analysis, for the given value of μ, with the member nominal yield capacities from Section 8.4.

   i) Increase or decrease the value of μ until the lowest value of μ is obtained for which no components are significantly beyond yield their nominal yield capacities.

      Significant means > 10% over nominal capacity for any one component and > 5% for all components in any one storey or at any one level.

   ii) The value of μ must be between $1 \leq \mu \leq 6C_s$.

   iii) The lowest value of μ resulting from the application of 8(i) above is the actual structural ductility factor, μ_{sd}, for the system, in order to meet the required strength assessment limit.

9. The evaluation is successfully completed if either of the following is met:

   i) $\mu_{sd} = 1.0$

   ii) $\mu_{sd} \leq 1.5$ and all of the "good features" of step 5 are present.

   If 9 is not met, then proceed with steps 10-13 as follows:

10. Determine inelastic behaviour of the system from a pushover analysis:

   - If the braces or brace connections are the weakest component, assume brace failure occurs early in the inelastic range and model inelastic response for the system as a semi-rigid MRSF.
   - If the braces or brace connections are not the weakest component, then determine the force in the system based on overstrength action in the braces using the appropriate section from Report R4–76 (Feeney and Clifton 1995).
   - Push the system to the inelastic deflection limit using the approach described in section 8.5.7.

11. Check on the inelastic stiffness adequacy of the system
   This will only need checking if the system will be responding in the braces failed condition under the required strength assessment level of seismic actions. In this case it will be responding as a semi-rigid moment-resisting framed system.

   In this instance, check if the inelastic deflections calculated from section 8.5.7 for the system are within the inelastic deflection limit of section 8.5.8(b).

12. Check strength of the system in the inelastic range. This will involve two limits, namely:

   - Loss of strength going from braces intact to braces failed – as an initial limit, base on not more than 50% drop
   - Loss of strength for residual MRSF system (ie. in the braces failed state) over the inelastic displacement – use the 80% limit as for Section 4.9.5.8(b)

13. Check ductility capacity of the yielding elements in the system. This will involve two limits, namely:

   - Rotational capacity of elements under forces prior to brace failure
   - Rotational capacity of elements under forces associated with residual MRSF system (ie. in the braces failed state)

Section 9 - Detailed Assessment of Moment Resisting Frame Elements with Masonry Infill Panels

9.1 Introduction

The assessment of an infilled frame is obviously dependent on the material constituting the infills and the geometry of both frame and infill. The following information is intended to provide a reference or starting point for entry into the main frame assessment procedures. Aspects giving rise to modified frame behaviour in terms of strength and ductility are considered. This section focuses on the response of reinforced concrete and steel. The approach can also be applied to other frame systems, for example concrete or masonry encased structural steel.

Material properties to be used in the following assessments should be those appropriate to the materials involved as detailed in the relevant sections, viz. Sections 7 and 8.

A broad subdivision of the possible effects of infills on frames with particular emphasis on column actions is as follows:

a) The presence of infills does not affect the structural response

This can be the case if the infills are very light and flexible, or completely isolated from the reinforced concrete frame, or so brittle that a total failure is expected even for a moderate ground acceleration. Clearly, in such a case, the possible danger to people in the streets must be carefully considered.

b) The infills are assessed to have a significant contribution on the response, and they are expected to remain in the elastic range

In this case a linear elastic analysis can be performed. The ductility capacity should be set to $\mu_{sc} = 1$, unless inelastic structural wall behaviour can be expected, with columns acting as tension or compression boundary members, and the infill acting as a connecting shear element.

c) The infills are assessed to have a significant contribution to the response, and they are expected to suffer significant damage during the seismic event

In this case the high probability of the formation of a soft storey has to be recognised and taken into account.

In order to decide whether case (1) is applicable to a given situation, the following parameters should be examined:

- details of connections between infill and frame
- ratio of the stiffness of the infilled wall and the stiffness of the bare frame

- ratio of the shear strength of the infilled wall and the bare frame.

When the infill is stiff, but weak, shear failure may occur at a base shear less than that corresponding to the bare frame, and the infill may be ignored, except insofar as it influences the overall response of the structure.

The decision as to whether cases (2) or (3) apply requires consideration of the likely infill failure mechanisms. In some cases it may be necessary to run sensitivity or "what if" scenarios in order to generate lower and upper bound parameters.

9.2 Solid Infilled Panel Components

This section gives means for quantifying the stiffness, strength and deformation capacities for infilled panels.

9.2.1 Stiffness

For Young's modulus and strengths for the infill panel use: (i) for a reinforced concrete infill $E_{ce} = 4700\sqrt{f'_{ce}}$ where f'_{ce} = expected concrete cylinder strength in MPa; and (ii) for clay masonry use $E_{me} = 500\sqrt{f'_{me}}$ where f'_{me} = expected strength of a masonry prism.

Figure 9.1 shows an equivalent diagonal compression strut that can be used in assessing the stiffness of an infill panel. The equivalent strut properties can be calculated using the recommendations based on the early work of Mainstone et al (1970 and 1971).

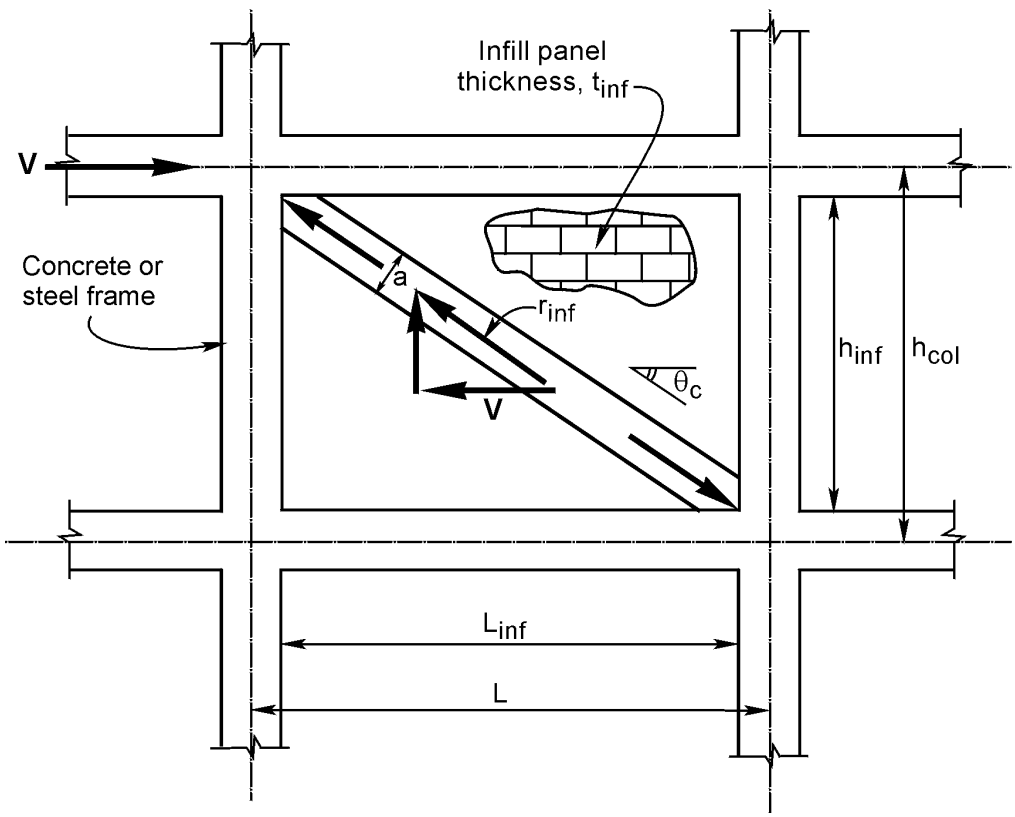


Figure 9.1: Modelling the infill panel of an infilled frame system as an equivalent strut

The equivalent strut is represented by the actual infill thickness that is in contact with the frame *(t_{inf})* and diagonal length *(r_{inf})* and an effective width *a* given by:

$$a = 0.175\ (\lambda_1 h_{col})^{-0.4} r_{inf} \qquad \ldots 9(1)$$

where:

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \sin 2\theta}{4 E_{fe} I_{col} h_{inf}} \right]^{1/4} \qquad \ldots 9(2)$$

in which h_{col} = column height between centrelines of beams (*mm*); h_{inf} = height of infill panel, (*mm*); E_{fe} = expected modulus of elasticity of frame material (*MPa*); E_{me} = expected modulus of elasticity of infill material (*MPa*); I_{col} = moment of inertial of column (mm^4); L_{inf} = length of infill panel (*mm*); r_{inf} = diagonal length of infill panel (*mm*); t_{inf} = thickness of infill panel and equivalent strut (*mm*); θ = angle whose tangent is the infill height-to-length aspect ratio (*radians*) given by the following:

$$\theta = \tan^{-1}\left(\frac{h_{inf}}{L_{inf}} \right) \qquad \ldots 9(3)$$

Unless positive anchorage capable of transmitting in-plane forces from frame members to all masonry wythes is provided on all sides of the walls, only the masonry wythes in full contact with the frame elements need to be considered when computing in-plane stiffness.

9.2.2 Strength

The strength capacity of an infill plane is complex with the potential for several behaviour modes occurring. It is important to analyse several potential failure modes as these will give an indication of potential crack and damage patterns. The following four failure modes are possible.

a) Sliding shear failure

The Mohr-Coulomb failure criteria can be used to assess the initial sliding shear capacity of the infill:

$$V^i_{slide} = (\tau_0 + \sigma_y \tan \phi)\ L_{inf}\, t_{inf} \qquad \ldots 9(4)$$

where τ_0 = cohesive capacity of the mortar beds, where in the absence of this, this may be taken as:

$$\tau_0 = \frac{f'_{m90}}{20} \qquad \ldots 9(5)$$

where ϕ = the angle of sliding friction of the masonry along a bed joint; and $\sigma_y = N/L_{inf}\, t_{inf}$ is the axial stress on the infill panel. Note that $\mu = \tan \phi$ where μ = coefficient of sliding friction along the bed joint which can be found from tests or in the absence of such site specific data assume $\mu = 0.8$.

After the infill's cohesive bond strength is destroyed as a result of cyclic loading, the infill still has some ability to resist sliding through shear friction in the bed joints. As a result, the final Mohr-Coulomb failure criteria reduces to:

$$V^{i}_{slide} = (\sigma_y \tan \phi)\, L_{inf}\, t_{inf} = \mu N \qquad \ldots 9(6)$$

where N = vertical load in the panel.

If deformations are small, then $V_{slide} \approx 0$ because σ_y may only result from the self weight of the panel. However, if these interstorey drifts become large, then the bounding columns impose a vertical load due to shortening of the height of the panel. The vertical shortening strain in the panel is given by:

$$\varepsilon = \frac{\delta}{h} = \theta \frac{\Delta}{h} = \theta^2 \qquad \ldots 9(7)$$

where δ = downward movement of the upper beam as a result of the panel drift angle, θ; h = interstorey height (centre-to-centre of beams); Δ = interstorey drift (a displacement); θ = interstorey drift angle (in radians).

The change in axial load on the infill is:

$$\Delta N = \varepsilon L_{\inf}\, t_{\inf}\, E_{me} \qquad \ldots 9(8)$$

where E_{me} = expected Young's modulus of the masonry which in the absence of tests may be taken as 500 f'_m. Substituting Equations (7) and (8) into (6) gives an expression for the total sliding load capacity:

$$V^{f}_{slide} = \mu(N + L_{\inf}\, t_{\inf}\, E_{me}\, \theta^2) \qquad \ldots 9(9)$$

b) Compression failure

For compression failure of the equivalent diagonal strut, a modified version of the method suggested by Stafford-Smith and Carter (1969) can be adopted. The shear force (horizontal component of the diagonal strut capacity) is taken as:

$$V_c = a\, t_{inf} f_{m90} \cos\theta \qquad \ldots 9(10)$$

where a = effective strut width defined previously; t_{inf} = infill thickness; f'_{m90} = strength of masonry in the *horizontal* direction which may be taken as 50% of the stacked prism strength f'_{me}.

c) Diagonal tension failure of panel

Using the recommendation of Saneinejad and Hobbs (1995), the cracking shear in the infill is given by:

$$V_{cr} = \frac{2\sqrt{2}\, t_{\inf} L_{\inf}\, \sigma_{cr}}{\dfrac{L_{\inf}}{h_{\inf}} + \dfrac{h_{\inf}}{L_{\inf}}} \qquad \ldots 9(11)$$

The cracking capacity of masonry σ_{cr} is somewhat dependent on the orientation of the principal stresses with respect to the bed joints.

In the absence of tests results, the cracking strength may be taken as:

$$\sigma_{cr} = \frac{f'_{m90}}{20} \qquad \ldots 9(12)$$

or

$$\sigma_{cr} \approx v_{me} \qquad \ldots 9(13)$$

where v_{me} = cohesive strength of the masonry bed joint given by:

$$v_{me} = 1.7\sqrt{f'_{me}} \qquad \ldots 9(14)$$

where f'_{me} = expected compressive strength of a stacked masonry prism.

d) General shear failure of panel

Based on the recommendations of Paulay and Priestley (1992), the initial and final contributions of shear carried by the infill panel may be defined as:

$$V_{mi} = A_{vh}\, 0.17\sqrt{f'_{me}} \qquad \ldots 9(15)$$

$$V_{mf} = 0.3 V_{mi} \qquad \ldots 9(16)$$

where V_{mi} = available initial shear capacity that is consumed during the first half-cyclic of (monotonic) loading; V_{mf} = final shear capacity as a result of cyclic loading effects; and A_{vh} = net horizontal shear area of the infill panel. Note for a complete infill with no openings $A_{vh} = L_{inf}\, t_{inf}$ The above values give upper (initial) and lower (final) bounds to the cyclic loading resistance of the infill.

e) The effect of infill panel reinforcement

If either a masonry or concrete infill panel is reinforced, then the reinforcement should improve the shear strength of the panel. The shear carried by the reinforcement is given by the well known code equations that assume a 45 degree truss.

$$V_s = \rho_w f_{ye} A_{vh} \qquad \ldots 9(17)$$

where ρ_w = volumetric ratio of the reinforcement in the infill panel; f_{ye} = expected yield strength of the web reinforcement within the infill panel; and A_{vh} = is defined above.

9.2.3 Deformation Capacities

It is not clear from experimental evidence, nor are there suitable analytical models available, as to what are the deformation capacities for each of the behaviour modes for infilled panel components. Experiments show that diagonal cracking commences with the onset of nonlinear behaviour at interstorey drifts of 0.25% and is essentially complete (from corner-to-corner) in a panel when the panel drift reaches a drift of about 0.5%. Corner crushing commences at this stage, but its extent will depend on the amount of cyclic loading sustained. There is essentially no limit to the ability of

an infill panel to deform in sliding shear—other behaviour modes usually govern. Thus, limits governed by the general shear behaviour mode affect the displacement capability of infill panels:

- Brick masonry 1.5%
- Grouted concrete block masonry 2.0%
- Ungrouted concrete block masonry 2.5%

9.3 Infilled Panel Components with Openings

Although the strength of infills with openings is best assessed using rational strut and tie models with sub-components of materials given in other sections of these recommendations, a simplified approach based on the work of Dawe and Seah (1988) follows. If an infill is pierced with either a door or window opening, then the strength and stiffness may be reduced by the factor $\lambda_{opening}$ given by the equation:

$$\lambda_{opening} = 1 - \frac{1.5 L_{opening}}{L_{\inf}}; \qquad \lambda_{opening} \geq 0 \qquad \ldots 9(18)$$

where $L_{opening}$ the maximum width of opening measured across a horizontal plane. Note the above equation implies that if the opening exceeds two-thirds of the bay width it may be assumed that the infill has no influence on the system performance.

9.4 Out-of-Plane Behaviour of Infilled Panel Components

Angel and Abrams (1994) describe methods for assessing infill capacity. Based on these recommendations the following formulae can be used for assessing the infill strength in which w is the uniform pressure on the wall which will cause out-of-plane failure.

$$w = \frac{2 f'_m}{(h/t)} \lambda R_1 R_2 \qquad \ldots 9(19)$$

in which f'_m = masonry strength; λ = slenderness parameter defined in Table 9.1; R_1 = out-of-plane reduction factors, taken as R_1 = 1 for no damage and for moderate and severe damage see Table 9.1; R_2 = lack of stiffness reduction factor for bending frame members given by:

2 $$R_2 = 0.35 + 2.45(10)^{-5}\, EI \leq 1 \qquad \ldots 9(20)$$

where EI = flexural rigidity of the weakest frame on the non-continuous side of the infill panel
2 (units: kN-m^2).

Table 9.1: Out-of-plane infill strength parameters

Height to thickness ratio $\frac{h}{t}$	Slenderness parameter λ	Strength reduction factors	
		Moderate damage	Severe damage
5	0.130	1.0	1.0
10	0.060	0.9	0.9
15	0.035	0.9	0.8
20	0.020	0.8	0.7
25	0.015	0.8	0.6
30	0.008	0.7	0.5
35	0.005	0.7	0.5
40	0.003	0.7	0.5

9.5 The Influence of Infilled Components on Frame Members

The flexural and shear strength assessment of any structural steel or reinforced concrete frames that surround infill panels should be based on Sections 7 and 8 of this document. However, it should be emphasized that the presence of infills modifies and magnifies the shear demands on the frame members by shortening the distance between in-span plastic hinges. | 2

9.5.1 Shear Demands on the Frame Members

The shear demand will be a maximum when flexural plastic hinges form at each end of this so-called "short column", thus:

$$V_{col} = \frac{2M_p^{col}}{l_{ceff}} \qquad \ldots 9(21)$$

where M_p^{col} = plastic moment capacity of the column based on the expected material strength properties; and l_{ceff} = the effective length of a "short" fixed-fixed column as shown in Figure 4.10.2 which may be found from:

$$l_{ceff} = \frac{a}{\cos\theta_c} \qquad \ldots 9(22)$$

where a = effective width of a longitudinal compression strut defined above; and θ_c = the diagonal strut angle which may be found from solving the following equation:

$$\tan\theta_c = \frac{h_{inf} - \left(\frac{a}{\cos\theta_c}\right)}{L_{inf}} \qquad \ldots 9(23)$$

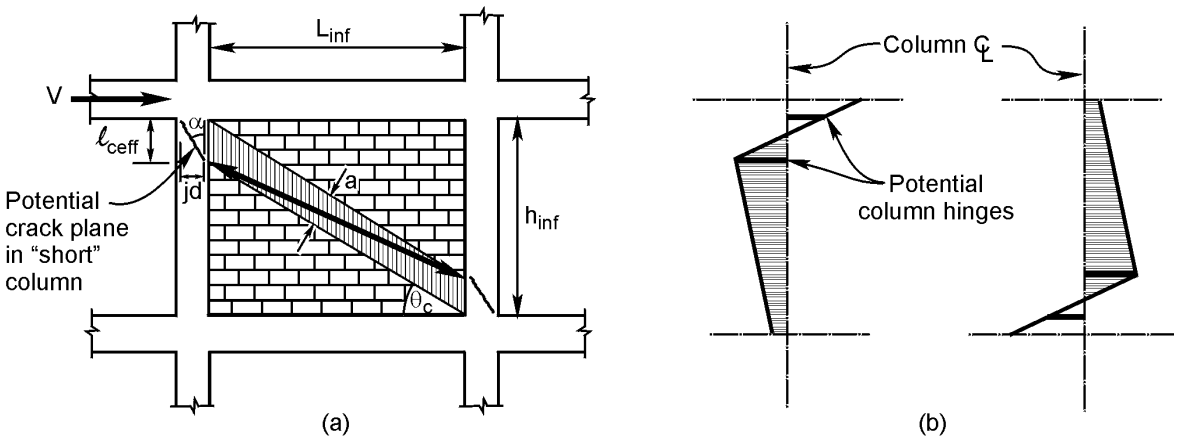


Figure 9.2: Modelling the adverse effect of an infill panel on the performance of the perimeter frame showing (a) the placement of the strut, and (b) the moment pattern on the columns

For infills with a sliding shear failure it may be assumed that the potential column hinges form at mid-height of the infill, thus:

$$l_{ceff} = 0.5h_{\inf} \qquad \ldots 9(24)$$

Figure 9.3 shows the effect of a partial height infill on the surrounding frame. Although the infill does not strengthen the system per se, it has the effect of placing greater shear demands on the columns.

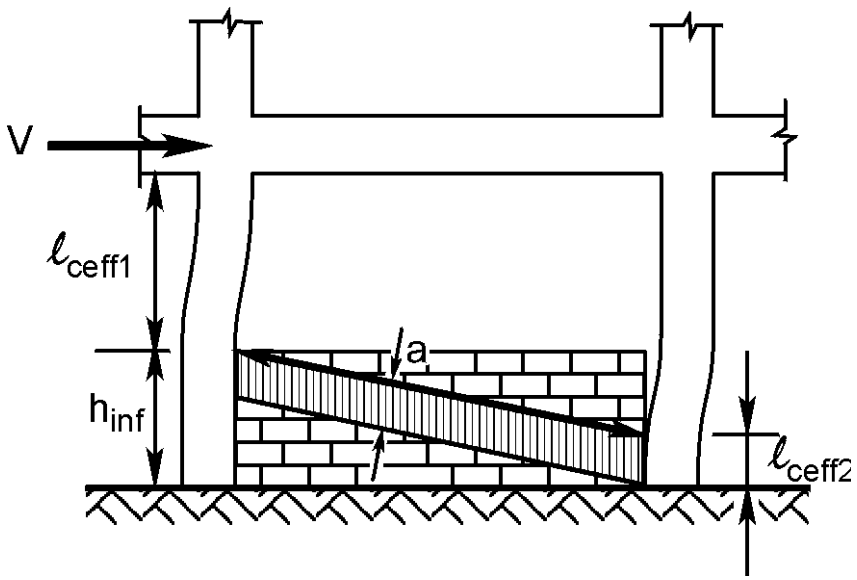


Figure 9.3: The effect of partial infills on frame performance

For the leeward column (the right hand column in Figure 9.3) the above recommendations for ascertaining l_{ceff2} and the associated shear demands should be used. However, for the windward column (the left column in Figure 9.3) the following should be adopted:

$$l_{ceff1} = h_{col} - h_{\inf} \qquad \ldots 9(25)$$

9.5.2 Modified Shear Capacity of Reinforced Concrete Frame Members

a) Steel Frames

When a "short-column" effect is present in the frame of an infilled frame system it is essential to check the shear strength based on the modified geometry. from the shear capacity of steel frames can be calculated in the usual fashion, which is the shear yield capacity of the steel webs. This strength should then be compared with the increased shear demand due to the short column effects induced by the presence of the infills as described above.

b) Concrete Frame

For reinforced concrete columns a corner-to -corner crack angle may form between the column hinges, thus modifying the shear resistance mechanism of the reinforced concrete column, this potential crack angle can be calculated from:

$$\alpha_c = \tan^{-1}\frac{jd}{l_{ceff}};\ 20° < \alpha_c < 45° \qquad \ldots 9(26)$$

where jd = internal lever arm within the column member, which in lieu of a more precise analysis may be taken as 80% of the overall member width.

The approach recommended by Priestley et al (1996) is can be adapted to provide an estimation of shear capacity of reinforced concrete members in the presence of large diagonal cracks. The shear resistance using this approach is given by:

$$V_r = V_s + V_n + V_c \qquad \ldots 9(27)$$

where V_s, V_n and V_c is the shear carried by steel, compressive axial strut force, and concrete mechanism, respectively. These are defined below.

- V_s = shear carried by the transverse reinforcement is given by:

$$V_s = A_{sh} f_{yhe} \frac{jd}{s} \cot \alpha_c \qquad \ldots 9(28)$$

in which A_{sh} = area of steel in one transverse hoop set; f_{yhe} = expected yield strength of the transverse reinforcement; jd = internal lever arm which in lieu of a more precise analysis may be taken as 0.8D; D = member depth; s = centre to centre spacing of the hoop sets; and α_c =corner-to-corner crack angle measured to the axis of the column.

- V_n = the shear carried by axial load (strut action) in a column which is given by:

$$V_n = N \tan \alpha_c \qquad \ldots 9(29)$$

where N = axial load in the frame member; α_c = as defined above.

- V_c = the shear carried by the concrete and is given by:

$$V_c = k\sqrt{f'_{ce}}\, b_w d \qquad \ldots 9(30)$$

in which b_w = web width; d = effective member depth; and k = coefficient depending on the displacement ductility of the member which may be defined as follows:

$$k = 0.33 - 0.06\Lambda^2 \frac{E_s}{f_{ye}} \tan \alpha_c\, \theta_p \qquad \ldots 9(31)$$

and $0.05 \leq k \leq 0.29$

where θ_p = plastic hinge rotation in the column (or beam) hinges, α_c = corner-to-corner crack plane angle such that tan α_c = jd/L_{eff}, E_s = Young's modulus of the longitudinal reinforcement, and $\Lambda =$ fixity condition between plastic hinges in the frame such that $\Lambda = 1$ for fixed-pinned (one hinge only), and $\Lambda = 2$ for the fixed-fixed case of two hinges.
Note that the above equation implies that for a member ductility of $\mu \leq 2$, $k = 0.29$, while for $\mu \geq 8$, $k = 0.05$, and for $2 < \mu < 8$ linear interpolation is used to determine the value for k.

For eqns 9(28) and 9(29) to apply, f'_{ce} is in MPa units.

Section 10 - Detailed Assessment of Unreinforced Masonry Buildings

10.1 General

Unreinforced masonry buildings have been the subject of legislation in relation to their earthquake performance since 1968. Since that time three successive NZSEE publications have provided guidance:

- "Recommendations for the Classification of High Earthquake Risk Buildings" published in 1972 and known as the "Brown Book"
- "Earthquake Risk Buildings – Recommendations and Guidelines for Classifying, Interim Securing and Strengthening" published in 1985 and known as the "Red Book"
- "Draft Guidelines for Assessing and Strengthening Earthquake Risk Buildings" published in 1995 and known as the "1995 Red Book".

Each of these has built on the approach and content of its predecessor, and has taken account of changing circumstances, technical developments, legislation and knowledge of behaviour of URM buildings in earthquake. Each has been written as a stand alone document specifically for URMs, covering requirements for inspection, strength levels, condition assessment, and material properties. In these new Guidelines, URM buildings form only part of the scope of buildings covered. This has meant that much of the m aterial in the 1995 Red Book has been superseded by more general requirements covered in Sections 2,3 and 4 of these current Guidelines. Even so, some material in the 1995 Red Book has been retained:

- Material providing guidance on detailed inspection.
- The attribute score assessment. Although this is superseded by the IEP, it is considered to be a valuable complementary tool for assessing the performance of URM buildings. It is included as an appendix to the Initial Evaluation Section
- The chapter on strength of materials, included as Appendix 10B, in unmodified form.

Sections 10.2 and 10.3 present material for the assessment of in-plane and out-of-plane performance of unreinforced masonry walls.

10.2 Procedure for the Assessment of Walls Responding In-Plane

10.2.1 Notation

Table 10.1: Notation

Symbol	Meaning	Comments
A	Depth of equivalent rectangular stress block	Determined as for reinforced concrete or reinforced masonry.
C	Cohesion	The adhesion of the mortar to the bricks. It is related to moisture absorption in bricks, but less so by the absorption qualities of individual types. It is not greatly influenced by keying of the brick surface (e.g. holes, lattices or patterning).
D	Overall depth of member	
E	Young's Modulus	
f_{bc}	Compressive strength of bricks	Measured on the flat.
f_{bt}	Direct tensile strength of bricks	May be taken as 85% of the stress derived from splitting tests or as 50% of stress derived from bending tests.
f_c	Compressive strength of the masonry	Due to the confining influence of the bricks on relatively thin mortar courses, this may be assumed to be twice the compressive strength of the mortar, f_{mc}, but not greater than the compressive strength of the bricks, f_{bc}.
f_{mc}	Compressive strength of mortar	
N	Normal force on a cross-section	
T	Thickness of web	
V	Shear strength	
X	Horizontal direction	
Y	Vertical direction	
Z	Distance from the compression edge of the section to the line of action of N	
γ	Shear strain	
ν	Poisson's ratio	
ε_{xx}	Normal strain in x direction	The stress and strain measures noted in Greek letters are those normally employed in finite element work. They are engineering stress and strain taken at a point (not average values), with tensile strains and stresses taken as positive. The x direction is defined as horizontal and parallel to the bedding planes. The y direction is vertical.
ε_{yy}	Normal strain in y direction	
μ	Coefficient of friction	
σ_{xx}	Normal stress in x direction	
σ_{yy}	Normal stress in y direction	
τ_{xy}	Shear stress	

10.2.2 Limitations of this Section

The material in this section has in mind brick masonry laid in running bond (more particularly common bond). Where the wall has more than one wythe, wythes should be adequately interconnected with header course at least every sixth course.

The principles may also be applied to stone masonry that is well coursed and essentially laid in running bond, but strength should be specifically determined.

Cohesion, for example, will often be small to zero in impervious dense stone.

When stone walls are laid with two wythes, there should be adequate header stones connecting the wythes, even if the cavity is filled.

Properties of the fill should be determined from tests. Voids are common in the fill. Their presence can be detected by ultrasonic or radar testing, or by opening out the stonework. Analytic treatment of the fill in the cavity is subject to the same reservations as noted for rubble masonry.

The material in this section should not be applied to rubble stone masonry.

For these structures failure criteria other than those covered in this section need to be examined. Among these is the possibility of sliding or other shear failure along oblique directions, which involve more generalised application of the Mohr-Coulomb yield criteria (or other criteria) than is applied in this section.

10.2.3 Basis of this Section

Much of this section is based on experimental and analytic studies by Magenes (1992).

Gambarotta and Lagomarsino (1997) have researched damage models for seismic response and have suggested modelling and failure criteria for brick masonry structures based on concepts borrowed from fracture mechanics. Their formulations are more suited to analyses of structures that are modelled using plane stress finite elements.

Magenes and Calvi (1997) and Magenes and Della Fontana (1998) reviewed previous analytic and physical tests and suggested formulations suitable for use with plane frame analysis software, employing suggestions made by Mann and Müller (1982). Subsequent researchers, including Magenes and Calvi, tested correlation of their simpler methods with more sophisticated methods, including those of Gambarotta and Lagomarsino.

Research has included dynamic analyses and has correlated the analytical results of physical tests on prototypes that have been tested statically and on shake tables.

Useful information on materials, inspection and assessments is contained in FEMA (1998)

The material in this section is largely drawn from the Magenes and Calvi publication. Some default stress values have been adapted from experimental results reported by Magenes and others, information in the FEMA documents (1998), and tests at the University of Cardiff (Kitching (1999), the Georgia Institute of Technology (Foss (2001)) and elsewhere (Benedetti and Petrini (1996); Abrams (1994)).

10.2.4 Objective

The objective of analysis for in-plane response of unreinforced masonry is to demonstrate that the structure is not unacceptably distressed when it is deformed by the maximum likely displacements that occur during the prescribed earthquake event.

10.2.5 General Considerations

a) Interconnection

This section assumes that there is sufficient interconnection at each level so that all walls and assemblages in the same vertical plane deflect essentially the same amount at each level when subjected to earthquake.

Such interconnection need not necessarily be with rigid diaphragms.

b) Modes of failure

Unreinforced masonry buildings are commonly characterised by deep members (walls, piers and spandrels). This usually makes these structures more forgiving of distress in individual members than are skeletal structures in modern framed buildings, principally because the spectral displacements are unlikely to be a significant fraction of the member dimensions. Sliding displacements at the base of a wall, for example, can be tolerated because the wall is unlikely to become unstable due to the shear displacements.

Nevertheless, certain failure modes are less acceptable than others are. In general, the preferred failure modes (if any) are rocking or sliding of walls or of individual piers. These modes have the capacity to sustain high levels of resistance during large inelastic straining.

c) Modelling of the structure

Several means of modelling the structure may be employed, including those employed for commonly used plane frame software. However, because the cross-sectional dimensions of elements are large compared with the length of the elements, some modifications to modelling that might be commonly employed are warranted. These include specific modelling for shear distortion and mass distribution, and facilities for coping with inelastic action and the conduct of pushover analyses.

Plane frame analysis will provide reasonably accurate predictive capacity if the following considerations are observed.

- *Because of the importance of shear deformation in these structures, the approach of NZS 3101 in incorporating shear deformation through a reduced flexural stiffness is not recommended - shear and longitudinal strains should each be specifically allowed for.*
- *Regions common to two or more intersecting members (joint panel zones) may require special attention. Parametric studies indicate that the member cross-sections may be assumed to penetrate unmodified to the intersection point or may be assumed to have stiffer (or rigid) end sections. Examination of stresses in the joint regions is seldom necessary.*
- *Much of the seismic mass of most unreinforced masonry buildings (particularly those with timber floors) lies in the walls themselves. This, coupled with the size of the members, suggests greater inertia effects may be involved. Rocking modes in particular, but also other modes, will involve rotational accelerations and vertical accelerations associated with these that are of greater importance than in modern skeletal structures. Modelling of the mass distribution is therefore important.*

- *To be able to predict inelastic strains, some form of pushover analysis is desirable. However, this need not necessarily require sophisticated software – simple hand procedures will often suffice.*

The preferred method is to model the elements as assemblages of plane stress elements, shell elements, or solid three-dimensional elements (so-called "brick" elements). Reasons for this include the relative ease with which the structure can be accurately described and the strains and stresses can be monitored. However, sufficiently sophisticated software is not readily accessible and the computational costs can be high.

Plane stress element modelling might use one of the following three levels, with each level representing progressively greater abstraction.

- *Modelling may treat the mortar as separate elements from the bricks. Given the relative thinness of the mortar bed joints it seems appropriate to model them as line elements, with quadrilateral elements used for the bricks. It is common to ignore the effects of head joints, except in determining failure criteria. Separate failure criteria are used for each of the bricks and the mortar. Gap elements can be introduced to model separation of layers (or laminae) at bed joints. Properties of the materials (or of the earthquake effects at the site) are seldom known sufficiently fully to warrant this level of modelling, which might be used for very important historic buildings or for smaller sections or larger assemblages.*
- *The properties of the mortar joints and of the bricks can be combined using homogenisation techniques. The weighted properties are used then to produce continuum elements. For best results, sampling and/or integration points are aligned with the bed joints, with or without gap elements. It is common to use the initial (elastic) stiffness throughout –with appropriate iteration at each load step.*
- *The most abstract modelling combines the stiffness and strength properties of the bricks and mortar as above, but otherwise treats the masonry as a continuum. It is common to assume that the masonry is isotropic. Gap elements are seldom included. This level is suited to large structures and produces adequate results for assessment purposes.*

For complex structures, particularly those with ineffective or flexible diaphragms, modelling of the structure in three dimensions as an assemblage of walls or frames is often helpful. However, it should then be appreciated that for modal response spectrum techniques a large number of modes may need to be considered to ensure that an adequate proportion of the total mass participates (as required by NZS 1170.5). Note that the total effective mass at 90% of the total mass, as specified in NZS 1170.5, may be difficult to achieve

Plane stress elements can be employed for modelling in three dimensions. In that these elements have only two degrees of freedom at each node, however, some contrivance is necessary to prevent numerical instability. One means, in which spring elements are added to the necessary degrees of freedom, is often sufficient. However, the allocation of mass to the nodes must then be made using other techniques, usually at the expense of accurate predictive capacity. Use of shell elements or three-dimensional "brick" elements, overcome this difficulty but at some computational expense. Suitable approximations can be often employed to allow adequate rigour using two-dimensional analyses. These approximations include:

- *Inclusion of return walls that might be lifted when the wall under study is deflected. The width of return wall that is affected can be assessed from the shear stresses at the intersection of the two walls. However, this approach should be pursued only with caution, as there may be discontinuous bonding at corners.*
- *Because masonry structures are commonly very stiff, even after cracking, ordinary timber floors and roof systems do not possess sufficient stiffness in comparison with the masonry elements to qualify the floors and roofs for the normal assumption of rigidity. Collections of*

smaller assemblages of a few walls or frames can then be chosen, with the immediately tributary areas used for accumulation of masses and gravity loads to these assemblages. Since deflections are often small, even should the separate assemblages act out-of-phase, relative deflections between them will seldom pose any serious problems. Care needs to be taken to ensure that the building as a whole is torsionally stable.

Whatever the modelling employed, it is usually not necessary to consider interaction between the in-plane and out-of-plane behaviour. Throughout this section, in-plane behaviour is discussed as if out-of-plane behaviour is not involved at all.

Three-dimensional analysis that allows for the interaction of in-plane and out-of-plane effects can be conducted, but the necessary software is not yet readily available and its use requires good modelling and interpretative skills.

It should be appreciated that computer programs are an aid to rapid design and evaluation. They are not essential to the assessments procedures described in this section. However, computer programs are almost indispensable for rigorous elastic analyses and for modal response spectrum analyses.

10.2.6 Analysis

a) Ductility factor

The concept of a ductility factor is rather meaningless for many structures, and this is the case for most unreinforced masonry structures. It remains important to assess the displacement demands, but rather more directly. In any event it is very difficult to determine the deflections of an unreinforced masonry structure at the end of its elastic phase (when shear cracking commences, or when bed joints begin to open, for example). The determination of a ductility factor (deflection at ultimate divided by the elastic deflection) is correspondingly fraught with difficulty.

Analysis should assume that response is elastic (ductility factor of unity). An initial assessment of period is not essential. Any period will provide an estimation of the pattern of deflection in the structure. The final deflections at ultimate conditions can then be used to deduce an effective period, from which the displacement demand can be assessed.

It is considered appropriate to take $S_p = 1$ for unreinforced masonry

Commonly, the effective period will still be short, so even this final step is not essential to the assessment of the structural capacity at ultimate when a force-based approach is adopted. However, it is recommended that a displacement-based analysis be conducted, for which an assessment of the displacement demand is required. Inertia forces need not be calculated, but may be calculated if the designer prefers to use a force-based approach as a means of controlling other aspects of the analysis.

b) Damping

Part of the role of a ductility factor is to recognise damping that occurs due to plastic straining. While unreinforced masonry does not respond in a classical elastic-plastic manner, exhibiting pinched hysteretic loops, often close to non-linear elastic response, there is nevertheless significant damping. Damping includes Coulomb (or “dry”) damping associated with sliding shear, radiation damping, pinched hysteretic damping associated with flexural/diagonal shear softening, and equivalent damping due to impact on rocking.

An overall equivalent viscous damping ratio of 15% of critical is recommended for both the force based and displacement based methods of analysis.

The adoption of damping at 15% of critical introduces about 35% reduction in response compared to response associated with 5% damping used in NZS 1170.5. This reduction applies in addition to relief from period shift (if any). An alternative approach would be to use a ductility factor of 1.5 to assess required elastic deflections (and inertia forces, if preferred). However, it should not then be assumed that the final deflection is required to be 1.5 times the elastic deflections so derived. It is therefore considered preferable to use the 35% reduction but to preserve the concept of elastic response (or, more accurately, a non-linear elastic response) by using a ductility factor of 1.0.

c) Modal response spectrum analysis

Much of the mass of walled buildings lies in the walls themselves, particularly where floors are constructed of timber. The equivalent static method commonly concentrates masses at nodes and includes accelerations in the lateral direction only. This leads to rather poor modelling for the class of structure considered here. Vertical and rotational inertia have important effects in these structures. It is recommended that modal response spectrum analyses be undertaken, including modelling of mass degrees of freedom in at least the two orthogonal directions (horizontal and vertical). Inclusion of rotational inertia is important in plane frame analyses.

10.2.7 Constitutive Relations and Material Failure Criteria

Masonry, by its nature, is not an isotropic material. Bricks and the mortar that binds them have different properties, and the elastic properties are different in the horizontal and vertical directions. Shear stiffness is correspondingly difficult to assess. However, sufficiently accurate results can be obtained if the composite masonry is assumed isotropic. Typical values that are suitable for preliminary assessments are suggested in Table 10.2 for both the mortar and the bricks, from which the values for the composite masonry can be assessed.

Much of the research that has been undertaken has concluded that the Mohr-Coulomb yield criterion is reasonably applicable, but known problems associated with excess dilatancy when an associated flow rule is assumed have been avoided by assuming a non-associated flow rule. Some interesting variations based on fracture theory have also emerged. Within the methods suggested in this section, simpler criteria have been adopted. Values of strength parameters that are suitable for preliminary assessments are suggested in Table 10.2.

Table 10.2: Strength parameters for preliminary assessments

	Visual characteristics and hand tests [1]	Stresses (MPa) and friction					Stiffness (GPa)	
		c	μ	f_{mc}	f_{bc}	f_{bt}	E	ν
Mortar								
Stiff	High Portland cement content (typical cement: lime: sand =1: 0.25: 3). Punch test < 10 mm.	0.4	0.8	8.0			12	0.11
Firm	Lime based (lime/sand =1/3), but in interior locations (not weathered). Punch test < 20 mm .	0.2	0.6	4.0			9	0.07
Soft	Lime binding possibly mildly leached, can be raked out of joint, but stays bound. Punch test < 30 mm.	0.1	0.4	1.0			7	0.05
Non-cohesive	Lime-based mortar that is heavily leached and weathered. Sand-like, easily raked out by hand, aggregate is unbound. Not suitable for earthquake resistance.	0.0	0.0	0.0				
Bricks								
Hard	Dense (heavy heft), hard surface, well fired, dark reddish brown.				20.0–30.0	2.0–3.0	18	0.2
Stiff [2]	Common brick, can be scored with a knife, red. Lower figures if split.				10.0–20.0	1.0–2.0	13	0.2
Soft	Weathered, pitted, distinct colour variation with depth (bright orange), probably under-fired.				1.0–5.0	0.1–0.5	4	0.35

1 Punch test uses a standard carpenter's nail punch (3 mm diameter at the tip), which is firmly driven with a standard carpenter's hammer for 6 blows. The total penetration is recorded.

2 Should only be considered if the mode of failure is likely to be in sliding shear (at the ULS) but the behaviour at the required deflection is essentially elastic.

10.2.8 Stress and Strain Limits

a) Plane stress analysis

Analysis may use plane stress elements for the assessment of structural elements. Where this method is used, the following limits on stresses and strains should be observed, unless a more detailed analysis, including the effects of strain softening and strength degradation, is undertaken.

i) Where there are no pre-existing cracks along bedding planes, such as those associated with settlement or damp-proof courses, tensile resistance to σ_{yy} may be assumed until c/3 is reached; thence no tensile resistance to σ_{yy} should be assumed.

It is consistent to assume tensile strength when cohesion is assumed for shear resistance. The theoretical value for this tension is c/μ according to the Mohr-Coulomb criterion, but tests indicate that the tension will be limited more by the direct tensile strength of the mortar. The suggested value is reasonably typical, but there is considerable variation. Once the bond is lost it is irrecoverable, and tensile strength is reduced to zero. In critical circumstances, zero tension might be assumed throughout. However, allowing a small tension will allow some predictions to be made of crack propagation.

ii) σ_{yy} in compression may be assumed linear with strain to $0.7f_c$ and rising to $0.85f_c$ for compressive strains not exceeding 0.005.

Stresses above $0.7\ f_c$ are seldom encountered. Greater compressive strengths add little to the flexural strength in deep members in any event. Once a certain upper limit of strain is reached, there is rapid strength degradation. More detailed stress-strain relationships are provided by Magenes (1992), and elsewhere. These show strong non-linearity and rapid loss of strength at high strains.

iii) σ_{xx} should be limited in tension to $0.5f_{bt}$, and when this limit has been reached, σ_{xx} should then be taken as zero.

The factor of 0.5 is to account for the effects of common bond, which has on average only half the bricks effective at any given vertical section. Ignoring the weakening effects of header courses is somewhat compensated by ignoring also any cohesion in the head joints and possible stagger between header head joints and regular course head joints. Note that this criterion applies to horizontal stresses, not to principal stresses as in the plane frame formulation.

iv) In circumstances where σ_{yy} may be assumed to be tensile, as provided in 1, shear strength may be assumed to be $|c - \mu\sigma_{yy}|$. Once the tensile resistance to σ_{yy} is exhausted, shear stress $|\tau|$ should thence not exceed $|\mu\sigma_{yy}|$. Shear strains $|\gamma|$ should not exceed 0.005.

The manner in which cohesion varies as a function of longitudinal and shear strains is not straightforward. Gambarotta and Lagomarsino use a damage parameter to trace the reduction in cohesion and other strength parameters, but the relationships involved require knowledge of many variables that would be difficult in the ordinary course of events to acquire with confidence. The relationships suggested here are offered pending further research and refinement. Where outcomes are important an alternative formulation taking the cohesion as zero whenever longitudinal strains are tensile might be employed (as is implied in the frame analysis procedures following). However, this may lead to a conclusion that sliding shear, a favourable mode, is involved, whereas less favourable modes may in fact occur. The reversing nature of earthquake effects also suggests that the influence of cohesion should be ignored in compressed regions that have previously failed in tension. It

should be appreciated that spandrels will fail under gravity load if no cohesion and associated tension are present and there are no separate lintels supporting the spandrel masonry.

10.2.9 Plane Frame Analysis – Strength Limits

Where the analysis is based on modelling the structure as a plane frame, suggested strength limits are given in the following. The strengths are based on the stresses provided for plane stress analysis above, with additional simplifications employed to derive equivalent limits and conversion to forces from stresses to reflect the normal outputs from plane frame analyses. The rationale behind each is provided in comments.

The maximum **nominal shear strength** is given by:

$$V_n = MIN(V_s, V_j, V_b) \qquad \ldots 10(1)$$

The shear strengths have the following values.

$$V_s = \frac{3czt + \mu N}{1 + \frac{3\alpha cdt}{N}} \qquad \ldots 10(2)$$

This is intended to control sliding shear at the end of the member.

This limit is derived from equilibrium of forces acting over the compression zone of the cross-section. The neutral axis depth is calculated from an assumed linear variation of stress with depth. The neutral axis depth is then $3(z–M/N) = 3d(z/d–M/Nd) = 3d(z/d–\alpha V/N) = \beta d$. *The shear strength, assuming that cohesion is only effective over this depth, is therefore* $V_n = \beta dct + \mu N$. *This is equal to the applied shear, V, so* $V = 3d(z/d–\alpha V/N)ct + \mu N$, *or* $V(1+ 3\alpha ct/N) = 3zct + \mu N$. *The relationship then follows. However, the previous comments (in the discussion on plane stress) about prior tensile failure in areas of compression should be kept in mind here.*

$$V_j = \frac{cdt + \mu N}{1 + \alpha} \qquad \ldots 10(3)$$

This is intended to reflect damage to mortar in joints near points of contraflexure.

The numerator in this expression is the usual Mohr-Coulomb failure criterion applied over the full depth of the section. Experiments show that it is a good approximation where the shear ratio, M/Vd, is small. However, shear strength has been shown to decrease hyperbolically as this ratio increases (i.e. the inverse of the shear strength increases linearly as the shear ratio increases). This is taken into account by the denominator in the above expression

$$V_b = \frac{\sqrt{f_{bt} dt (f_{bt} dt + N)}}{2.3(1 + \alpha)} \qquad \text{where } (1 + \alpha) \le 2.5 \qquad \ldots 10(4)$$

This is intended to limit shear associated with diagonal tension failure involving cracking through the bricks near points of contraflexure.

The term in the numerator is the value of the average shear that would be required to produce a principal tension equal to f_{bt} *in the presence of the axial force N. Both the shear and the axial*

forces are assumed to produce uniform stress. The factor 2.3 is to account for nonuniformity of stress and other factors. The formulation for shear stress to this stage (without the (1+α) term) is due to Mann and Müller. The additional term (1+α) was introduced by Magenes and Calvi to account for the influence of aspect ratio, in the same manner as for the expression for V_j.

In the above expressions:

- c is the bond (cohesion)
- μ is the coefficient of friction
- $\alpha = M/Vd$ is the effective aspect ratio (calculated at the point of maximum moment)
- N is the normal (axial) force on the cross-section
- f_{bt} is the direct tensile strength of the bricks.

The direct tensile strength can be approximated as 85% of the splitting strength from splitting tests or as 50% of the brick modulus of rupture from bending strength tests.

When the shear reaches V_n plastic flow at constant shear is assumed, up to the limit of permitted deflection of the element.

The maximum nominal flexural resistance is given by:

$$M_n = N(z - a/2) = N\left(z - \frac{1}{2}\frac{N}{0.85 f_c t}\right) \qquad \ldots 10(5)$$

Here, f_c is the compressive strength of the masonry, and a is the depth of the equivalent rectangular stress block.

This equation is similar to those used for reinforced concrete or reinforced masonry, and relates the flexural strength to the mid-depth of the section. The crushing strength of the masonry may be assumed to be twice the compression strength of the mortar, but not greater than the compression strength of the bricks.

When the applied moment reaches M_n plastic flow at constant moment is assumed. There is no effective limit on element deflections, but 1% is suggested as a usable maximum.

10.2.10 Plane Frame Analysis – Strain Limits

The maximum deflection in any element should not exceed the following, depending on the mode of failure of the element. The effects of flexural and shear deformation (including elastic and plastic components of both) should be included in calculating deflections. The maximum deflection so calculated is to be measured transverse to the longitudinal axis of members.

a) For walls and piers

For elements failing in moment (a so-called rocking mode), or for elements failing in sliding shear, the maximum deflection should not exceed 1% of the clear span of the element.

Care should be taken in considering rocking modes. Control of strength by flexure does not imply that rocking occurs in the classical sense of unconstrained overturning of an effectively rigid body. Strains along the compression diagonal, which involve corner-to-corner shortening, make this

unlikely in most cases. The limit of 1% is rather artificial, but greater deflections are unlikely to be useful.

For all other modes of failure, the maximum deflection should not exceed 0.5% of the clear span of the element.

The limit on shear strain is imposed to prevent excessive strength degradation.

b) For spandrels

The maximum deflections should not exceed 0.5% of the clear span of the spandrel.

This is likely to be the limiting criterion for many structures. Where it is, consideration might be given to also investigating the response of a reduced structure without spandrels. Where the remainder of the structure is capable of providing the necessary resistance, the spandrels can be ignored if they would not suffer so much damage that they would disintegrate and fall from the building. Disintegration is unlikely where a spandrel is supported by an independent lintel (usually constructed from concrete, steel or timber).

10.2.11 Common Stress and Strain Parameters

Common values of the various stress parameters are shown in Table 10.2. These are not to substitute for values selected from tests that are conducted in accordance with Appendix 10B, but may be used for routine preliminary assessments. The stresses in the table may be used for analysis using plane stress elements or for plane frame analysis.

The tabulated values of cohesion, c, and friction, μ, are to be factored by κ, where:

$$\kappa = \frac{1}{1 + 2\mu \dfrac{h_b}{l_b}} \approx \frac{1}{1 + 0.7\mu} \qquad \ldots 10(6)$$

Here, h_b and l_b are the height and length of the individual bricks.

Relations for strength have been adjusted to match results from detailed finite element analyses. Those finite element analyses have not directly modelled head joints. The expression is intended to take some account of the influence of head joints, and is due to Mann and Müller. It is noted that the finite element analyses have been correlated with only a few actual physical tests.

10.3 Procedure for the Assessment of Walls Responding Out-of-Plane

10.3.1 Notation

Table 10.3: Notation

Symbol	Meaning	Comments
A	Angular deflection (rotation) of the top and bottom parts of a wall panel relative to a line through the top and bottom restraints.	The angle is in radians. It is measured as if there were no interstorey deflection.
a	Parameter given by equation.	
b	Parameter given by equation.	
δb	Change in parameter b due to interstorey slope, Ψ.	
C_{fi}	Floor coefficient at level i.	Refer to NZS 1170.5.
C_{fn}	Floor coefficient at level n.	Level n is the top of the structure.
C_{fo}	Floor coefficient at level o.	Level o is the base of the structure.
C_m	Value of the seismic coefficient that would cause a mechanism to just form.	Uniform acceleration to the entire panel is assumed in finding C_m.
C_{ph}	Seismic coefficient for a part at the level of the wall panel using a ductility factor of unity for the panel and the period T_p. The coefficient is for an earthquake intensity that would apply to a new building.	Refer to NZS 1170.5.
C_{pho}	Seismic Coefficient for a part at the level of the wall panel using a short period.	"Short period" is 0.45 seconds.
D_{ph}	Displacement response (demand) for a wall panel subject to an earthquake of the intensity specified in NZS 1170.5 for a new building.	
e_b	Eccentricity of the pivot at the bottom of the panel measured from the centroid of W_b.	
e_o	Eccentricity of the mid-height pivot measured from the centroid of W_b.	
e_p	Eccentricity of P measured from the centroid of W_t.	
e_t	Eccentricity of the mid-height pivot measured from the centroid of W_t.	
G	Acceleration of gravity, 9.81 m/s^2.	
H	Clear height of the storey.	The clear height can be taken at the centre-to-centre height between lines of horizontal restraint. In the case of concrete floors, the clear distance between floors will apply.
h_l	Height from the base of the building to the mid-height of the wall panel	
h_n	Height from the base of the building to the top of the building	Usually the roof for URM buildings, but possibly the uppermost principal floor otherwise.

Symbol	Meaning	Comments
I_D	Proportion of the NZS 1170.5 intensity earthquake that the wall panel is able to sustain without collapse through excessive deflection.	The "NZS 1170.5 intensity" is taken as the value used for the design of new buildings in this section.
J	Rotational inertia of the wall panel and attached masses.	
J_{anc}	Rotational inertia of ancillary masses.	
J_{bo}	Rotational inertia of the bottom part of the panel about its centroid.	
J_{to}	Rotational inertia of the top part of the panel about its centroid.	
P	Load applied to the top of the panel.	P is assumed to act through the pivot at the top of the wall.
R	Risk factor as defined in NZS 1170.5	
S_p	Structural performance factor	See text (S_p = 1.0)
T	Effective thickness.	Varies with position.
t_{nom}	Nominal thickness.	Varies with position.
T_1	Fundamental period of the building	
T_p	Effective period of a wall panel.	
W_b	Weight of the bottom part of the panel.	
W_t	Weight of the top part of the panel.	
y_b	Height of the centroid of W_b from the pivot at the bottom of the panel.	
y_t	Height from the centroid of W_t to the pivot at the top of the panel.	
Z	Zone factor as defined in NZS 1170.5	
γ	Participation factor	This factor relates the deflection at the mid-height hinge to that obtained from the spectrum for a simple oscillator of the same effective period and damping.
Δ	Horizontal deflection at the mid-height of a wall panel relative to the mean of the deflections at the top and bottom restraints; or the horizontal deflection at the top of a parapet relative to its base.	Δ is thus measured as if there were no interstorey deflection.
Δ_i	Deflection that would cause instability.	W_b, W_t and P are the only forces applying for this calculation.
Δ_m	An assumed maximum useful deflection = $0.6\ \Delta_i$.	Used for calculating period and deflection response capacity.
ψ	Interstorey slope.	Interstorey deflection divided by the storey height.

2

10.3.2 Basis of this Section

Procedures for the assessment of face-loaded walls spanning vertically in one direction are based on displacement response that includes strongly non-linear effects. These procedures have been verified by research (Blaikie 2001, 2002) using numerical integration time history analyses and by

laboratory testing that included testing on shake tables. This research extended the preliminary conclusions reached in Blaikie and Spurr (1993). Other research has been conducted elsewhere, some of which is listed in various other research studies (Yokel 1971; Fattal 1976; Hendry 1973, 1981; Haseltine 1977; West 1977; Sinha 1978; ABK Consultants 1981; Kariotis 1986; Drysdale 1988; Lam 1995; Mendola 1995).

Procedures provided for in earlier drafts of this document (1995 Red Book), which are based on the concept of equating total energy (strain energy of deformation plus potential energy due to shifts of weights) of the rocking wall to that for an elastic oscillator have been shown deficient. They give inconsistent results and are unsafe particularly where walls are physically hinged at floor levels (as when they seat onto a torsionally flexible beam with no wall under it) or when walls are made of stiff masonry (high Young's modulus).

Procedures for the assessment of face-loaded walls that span one-way horizontally or two-way horizontally and vertically are based on response that includes only weak non-linear effects (i.e. elastic or nominally elastic response). Only general guidelines are provided for these procedures. They are based on less rigorous research and are therefore not as well developed as the procedures for walls spanning vertically. Caution therefore needs to be exercised in the use of these procedures.

10.3.3 General

Walls are to be assessed in every storey, and for both directions of response (inwards and outwards). The rating of the wall is to be set at the least value so found, as failure in any one storey for either direction of loading will lead to progressive failure of the whole wall.

Walls are commonly analysed as spanning vertically in one direction between a floor and another floor or the roof or as vertically cantilevered (as in partitions and parapets). Lateral restraint of the floors and the roof assumed for all such walls is to be assured. If the restraint cannot be assured then the methods presented here for one-way vertically spanning walls cannot be used. However, it might still be possible to assess such walls by analysing them as spanning horizontally between other walls, columns or other elements or as two-way assemblages.

For walls of several wythes, the designer should check that the walls are capable of acting as integral units, as is assumed for the procedures given in this section.

2 | *For example, and with reference to Figures 10A.1 and 10A.2 (of Appendix 10A), there is a vertical shear acting on the centreline of the lower wall that is equal to* $P + W_t + 0.5W_b$. *This shear needs to be resisted by header bricks crossing the centreline. For this purpose, each header brick may be assumed to contribute a shear resistance of* $2f_rbt^2/l$, *where b, t and l are the breadth, depth and length of the header, and* f_r *is its modulus of rupture in bending.*

10.3.4 Procedure for Walls Spanning Vertically between Floors and/or the Roof

a) General

The following steps are those required to assess the displacement response capability and the displacement demand, from which the adequacy of the walls can be determined. Some guidance on methods for determination of key parameters is provided in Appendix 10A. Refer to Figure 10.1 for the notation employed.

The wall panel is assumed to form hinge lines at the points where effective horizontal restraint is assumed to be applied. The centre of compression on each of these hinge lines is assumed to form

a pivot point. The height between these pivot points is the effective panel height h. At mid-height between these pivots, height $h/2$ from either, a third pivot point is assumed to form.

1 Divide the wall panel into two parts, a top part bounded by the upper pivot and the mid-height between the top and bottom pivots, and a bottom part bounded by the mid-height pivot and the bottom pivot.

The division into the two parts is based on the assumption that a significant crack will form at the mid-height of the wall, where an effective hinge will form. The two parts are then assumed to remain effectively rigid. These assumptions are not always correct. For example, in the upper part of top-storey walls significant deformation occurs, and, particularly where the tensile strength of the mortar is small, the third hinge will not necessarily form at the mid-height. However, errors introduced by the approximations are not significant.

2 Calculate the weight of the wall parts, W_b of the bottom part and W_t of the top part, and the weight acting at the top of the storey, P.

The weight of the wall should include any render and linings, but these should not be included in t_{nom} or t unless the renderings are integral with the wall. The weight acting on the top of the wall should include all roofs, floors (including partitions and ceilings and the seismic live load) and other features that are tributary to the wall.

3 From the nominal thickness of the wall, t_{nom}, calculate the effective thickness, t.

The effective thickness is the actual thickness less the depth of the equivalent rectangular stress block. The reduction is intended to reflect that the walls will not rock about their edge but about the centre of the compressive stress block. Calculation of the depth of the equivalent rectangular stress block should use caution, as the depth determined for static loads may increase under earthquake excitation. Appendix 10A suggests a reasonable value based on experiments, which is $t = t_{nom}(0.975 - 0.025\ P/W)$. The thickness calculated by this formula may be assumed to apply whatever the mortar, provided it is cohesive. For weaker (and softer) mortars, greater damping will compensate for any error in the calculated t.

4 Assess the maximum distance, e_p, from the centroid of the top part of the wall to the line of action of P, and similarly e_b, e_t and e_o. Usually, the eccentricities e_b and e_p will each vary between 0 and $t/2$ (where t is the effective thickness of the wall). Exceptionally they may be negative.

Figure 10.1 shows the positive directions for the eccentricities for the assumed direction of rotation (angle A at the bottom of the wall is positive for anti-clockwise rotation).

The walls do not need to be rigidly attached or continuous with a very stiff section of wall beyond to qualify for an assumption of full flexural restraint.

Care should be taken not to assign the full value of eccentricity at the bottom of the wall if the foundations are indifferent and may themselves rock at moments less than when the wall rocks. In this case the wall might be considered to extend down to the supporting soil where a cautious appraisal should then establish the eccentricity. The eccentricity is then related to the centroid of the lower block in the usual way.

5 Calculate the mid-height deflection, Δ_i, that would cause instability under static conditions. The following formula may be used to calculate this deflection.

$$\Delta_i = \frac{bh}{2a} \qquad \ldots 10(7)$$

where:

$$b = W_b e_b + W_t(e_o + e_b + e_t) + P(e_o + e_b + e_t + e_p) - \Psi(W_b y_b + W_t y_t) \qquad \ldots 10(8)$$

and

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph \qquad \ldots 10(9)$$

The deflection that would cause instability in the walls is most directly determined from virtual work expressions, as noted in Appendix 10A.

6 Assign the maximum usable deflection, Δ_m, as 0.6 Δ_i.

The lower value of the deflection for calculation of instability limits reflects that response predictions become difficult as the theoretical limit is approached. In particular the response becomes overly dependent on the characteristics of the earthquake, and minor perturbances lead quickly to collapse. Some compensation is subsequently made for this conservatism. The reduced deflection limit is also used for calculation of period.

7 Calculate the period of the wall, T_p, as four times the duration for the wall to return from a displaced position measured by Δ_m to the vertical. The period may be calculated from the following equation.

$$T_p = 6.27\sqrt{\frac{J}{a}} \qquad \ldots 10(10)$$

Where J is the rotational inertia of the masses associated with W_b, W_t and P and any ancillary masses, and is given by the following equation.

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b\left[e_b^2 + y_b^2\right] + W_t\left[(e_o + e_b + e_t)^2 + y_t^2\right] + P\left[(e_o + e_b + e_t + e_p)^2\right]\right\} + J_{anc} \qquad \ldots 10(11)$$

The relations are derived in Appendix 10A. The method used there may be employed to assess less common configurations as necessary.

8 Calculate the seismic coefficient ($C_p(T_p)$) for an elastically responding part ($\mu_p = 1$) with this period (T_p), that applies at this elevation in the building.

Note that only 5% damping should be applied, not the greater damping suggested for in-plane response. Experiments show that expected levels of damping from impact are not realised: the mating surfaces at hinge lines tend to simply fold onto each other rather than impact.

9 Calculate γ the participation factor for the rocking system. This factor may be taken as

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg} \qquad \ldots 10(12)$$

It relates the response deflection at the mid-height of the wall to the response deflection for a simple oscillator of the same period and damping. Its value varies from a maximum of 1.5 to significantly less when the aspect ratio h/t_{nom} of the wall is small and the ratio P/W is large.

10 From $C_p(T_p)$, T_p, R_p *and* γ calculate the displacement response, D_{ph} from;

$$D_{ph} = \gamma(T_p/2\pi)^2.C_p(T_p).R_p.g \qquad ...10(13)$$

The given relationship implies that the amplification of demand due to elevation in the building is the same for displacement as it is for acceleration. The factor γ is the participation factor for the rocking system found in step 9.

Note that while the primary response is calculated on the assumption of 5% damping for the panels, 15% damping may be assumed for the overall structure if it is constructed of unreinforced masonry.

11 Calculate $\%NBS = 100[(1.2)(0.6)\Delta_i]/[D_{ph})] = 72(\Delta_i/D_{ph})$ …10(14) | 2

The two multipliers are introduced for these reasons:

(i) The 0.6 factor applied to Δ_i reflects that response becomes very dependent on the characteristics of the earthquake for deflections larger than $0.6\Delta_i$. This reduced deflection is also used for the calculation of the period of the wall, which would be infinitely long for $\Delta = \Delta_i$.

(ii) The 1.2 factor compensates to some degree for the conservatism introduced by the factor of 0.6 used for period and other calculations.

If > 33%NBS but < 67%NBS, then the wall may be classed as of moderate hazard. If ≥ 67%NBS, then the wall may be classed as of low hazard. %NBS ≤ 33 is not acceptable. | 2

12 Calculate the horizontal accelerations that would just force a mechanism to form. The acceleration may be assumed to be constant over the height of the panel, reflecting that it is associated more with acceleration imposed by the supports than with accelerations associated with the wall deflecting away from the line of the supports. Express the acceleration as a coefficient, C_m, by dividing by g.

Again, virtual work should prove the most direct means for calculating the acceleration. Appendix 10A shows how, and derives the following expression for C_m in which the ancillary masses are assumed part of W_b and W_t.

$$C_m = \frac{b}{(W_b y_b + W_t y_t)} \qquad ...10(15)$$

To account for enhancement of strength due to tensile strength of mortar and possible rendering it is recommended that a possibly greater value be used for the design of connections to the roof and floors. In addition, the value of C_m may be too large to use for the design of connections. Accordingly, it is recommended that C_{pho} should be used for the design of connections.

13 Calculate the ratio $C_p(0.4)$, which is the value of $C_p(T_p)$ for a part with a short period.

$C_p(0.4)$ *is required to assess an upper bound on reactions for the design of supports, which would otherwise be dictated by values of C_m that are either too high or too low. High values are likely to be more applicable to squat walls (small height/thickness ratios) with large values of the ratio P/W.*

14 Calculate the support reactions to ensure that the strengths of connections to floors are not less that the demand from the required reactions.

This is best assessed from the value of $C_p(0.4)$ calculated above.

Note that it is recommended that the reactions be stronger than the wall, so that if strengthening is envisaged (as will usually be the case), the FULL reaction potential, limited only be what would be required of a new building, should be allowed for.

b) Simplifications for regular walls

Where walls panels are uniform within a storey (approximately rectangular in vertical and horizontal section and without openings), and the interstorey deflection does not exceed 1% of the storey height, the following approximations may be employed. Otherwise the general procedure should be used. The steps that follow parallel those of the general procedure above, and results are summarised in Table 10.4 following.

1 Divide the wall as before.

2 Calculate the weight of the wall, W, and the weight applied at the top of the storey, P.

3 Calculate the effective thickness as before, noting that it will be constant.

4 Calculate the eccentricities, e_b, e_t and e_p, which may usually each be taken as either $t/2$ or 0.

5 Calculate the instability deflection, Δ_i from the formulae in the table for the particular case.

6 Assign the maximum usable deflection as 60% of the instability deflection.

7 Calculate the period, which may be taken as $6.27\sqrt{(J/a)}$, where J and a are given in Table 10.4. Alternatively, where the wall is fairly thin (h/t is large), the period may be approximated at:

$$T_p \approx \sqrt{\frac{0.67h}{(1+2P/W)}} \qquad \ldots 10(16)$$

in which h is expressed in metres.

8 Calculate $C_p(T_p)$.

9 Calculate the participation factor as for the general method, with the numerator of the expression expanded to give $\gamma = Wh^2/8J$. This may be taken at the maximum value of 1.5 or may be assessed by using the simplified expression for J that is shown in Table 10.4.

10 Calculate D_{ph} from $C_p(T_p)$, T_p *and* γ in the same manner as for the more general method.

11 Calculate *%NBS* in the same manner as for the more general method.

12 Calculate C_m.

13 Calculate $C_p(0.4)$.

14 Calculate the reactions from the weight, W, and from C_m and $C_p(0.4)$.

Table 10.4: Static instability deflection for uniform walls – various boundary conditions

Boundary Condition Number	0	1	2	3
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	t/2
b	$(W/2+P)$t	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)$h	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)t}{(2W+4P)}$	t
J	$\{(W/12)[h^2+7t^2]$ $+Pt^2\}/g$	$\{(W/12)[h^2+16t^2]$ $+9Pt^2/4\}/g$	$\{(W/12)[h^2+7t^2]$ $+9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2]$ $+4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

2

10.3.5 Procedures for Vertical Cantilevers

Parameters for the assessment of vertical cantilevers, such as partitions and parapets are derived in Appendix 10A, which should be consulted for general cases. For parapets of uniform rectangular cross-section, the following approximations may be employed. The item numbers parallel the steps for the general procedure for walls spanning between a floor and an upper floor or roof.

1 The parapet need not be divided. Only one pivot is assumed to form—at the base.

2 The weight of the parapet is W. P is zero.

3 The effective thickness is $t = 0.98t_{nom}$.

4 Only e_b is relevant. It is equal to $t/2$.

5 The instability deflection *measured at the top of the parapet* $\Delta_i = t$.

6 The maximum usable deflection *measured at the top of the parapet* $\Delta_m = 0.6\ t$.

7 The period may be calculated from:

$$T_p = \sqrt{2.67h\left[1+\left(\frac{t}{h}\right)^2\right]} \qquad \ldots 10(17)$$

in which h, the height of the parapet above the base pivot, is expressed in metres.

8 Calculate $C_p(T_p)$

9 Calculate $\gamma = 1.5/[1+(t/h)^2]$ …10(18)

10 Calculate D_{ph} from $C_p(T_p)$, T_p *and* γ and as before.

11 Calculate *%NBS* as for the general procedure for walls spanning between a floor and an upper floor or roof, from;

$$\%NBS = 72\Delta_i/D_{ph} = 72t/D_{ph}. \qquad \ldots 10(19)$$

2

12 Calculate $C_m = t/h$. …10(20)

13 Calculate $C_p(0.4)$.

14 Calculate the base shear from W, C_m and $C_p(0.4)$. This base shear adds to the reaction at the roof level restraint.

10.3.6 Procedures for Gables

Figures 10.1 (a) and (b) illustrate two gables. Figure 10.1 (a) shows a gable that: is free along the vertical edge; is simply supported along the top edge (at roof level); and is continuous at the bottom edge (ceiling or attic floor level). This somewhat unusual case is useful in establishing parameters for more complex cases. For the gable in Figure 10.1 (a), the following parameters can be derived.

$$a = \frac{h}{6}\left(2W + 3P\right) \qquad \ldots 10(21)$$

$$J = \frac{W}{24g}\left(32t^2 + h^2\right) + \frac{9Pt^2}{4g} \qquad \ldots 10(22)$$

NOTE: In the above equations, W and P are total weights, not weights per unit length. It should also be noted that the participation factor now has a maximum value of 2.0 (t << h, P = 0)

For the gable in Figure 10.1 (b), the above results can be used to provide a cautious appraisal of performance. This effectively ignores the enhanced performance for this case.

Several factors that enhance performance occur in gables of the kind illustrated in Figure 10.1(b), all of which relate to the occurrence of significant membrane actions. Guidance on this aspect will be provided in future versions of this document when the necessary research (including testing) has been undertaken. (See also the following section on walls spanning horizontally and vertically).

10.3.7 Procedures for Walls Spanning Horizontally or Horizontally and Vertically

General considerations

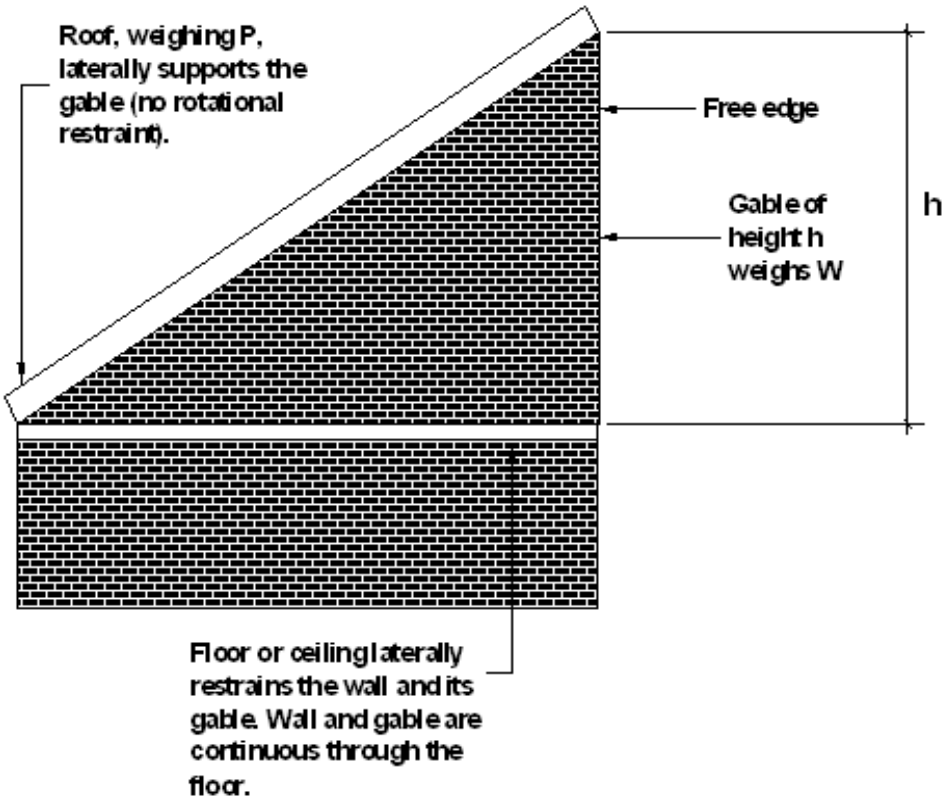
The wall panel should be modelled as a slab, shell, or assemblage of block elements, or as one or more slabs interconnected with headers or cavity ties, and continuous with adjoining panels beyond the supports.

Interaction of header courses in providing flexural, torsional and shear coupling of wythes (through the header courses acting as “beams” or “stiffeners”) may be considered with caution.

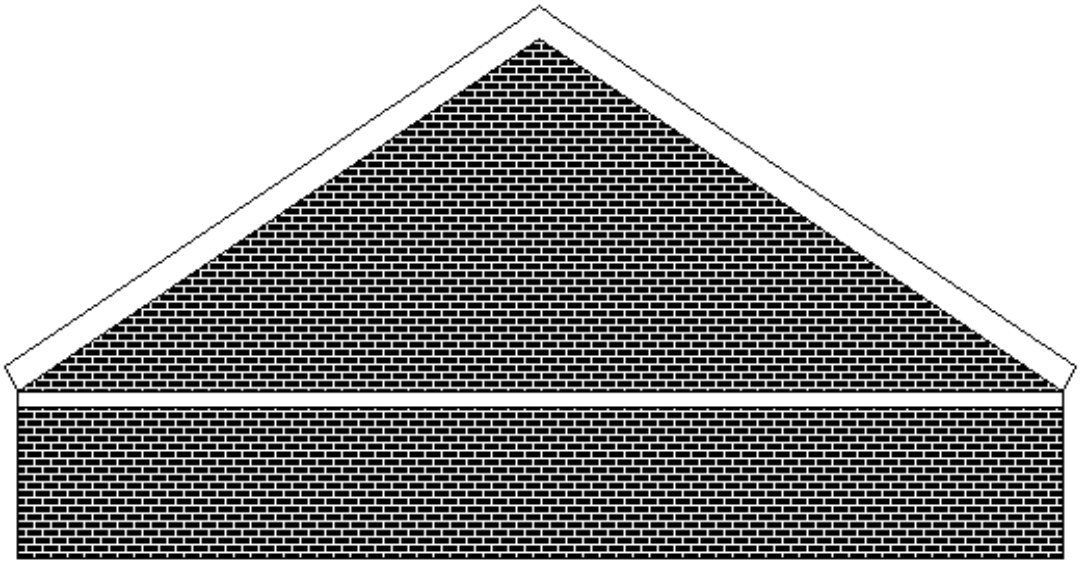
An elastic analysis should then be conducted.

Redistribution of moments in the vertical direction may be undertaken in all cases.

For panels spanning horizontally, Redistribution of moments in the horizontal direction should only be considered if failure is unlikely to occur through the bricks. If failure may occur through the bricks, a brittle failure mode is expected and a local failure could lead to widespread failure. The flexural strength may be taken as that moment that leads to a stress equal to the modulus of rupture in bending.



(a) Basic gable used for defing parameters.



(b) Typical gable for which results from (a) can be applied.

Figure 10.1: Gable Configurations discussed in this section

For panels spanning horizontally, in which failure is likely to occur in the mortar along a path around the bricks, then redistribution may be considered. In this event, the moment capacity for one-way slabs may be approximated by the expression:

$$\frac{M}{M_m} = \left(\frac{\phi}{\phi_m}\right)^{-0.8} \quad \text{where } \phi \geq \phi_{\mathrm{m}} \qquad \ldots 10(23)$$

where ϕ is the curvature, M is the moment, and subscript m refers to the condition when the maximum moment is reached. The value of the peak moment, M_m, should use a cautious appraisal of the shear strength of the mortar.

Further information may be obtained from Hansen (1999) and Kitching (1999)

For two-way action, interaction between moment, twist and shear should be taken into account.

For wall panels that are thick relative to their spans compressive membrane action may be taken into account.

Section 11 - Detailed Assessment of Timber Structures

11.1 Introduction and Scope

This section provides guidance on the assessment of strength of timber structures in buildings. In particular it is intended to assist in providing information on common forms of construction and their strength parameters.

Timber has been used extensively in unreinforced masonry buildings for floor joists, roof framing, floors and sarking under roofs. For this reason, this chapter may need to be read in conjunction with the preceeding section detailing the assessment of unreinforced masonry buildings.

It is particularly important to determine the state of the connection between the floors and the supporting walls as this will have a direct bearing on whether or not the floors (and also the roof if sarking has been used) can act as a diaphragm in distributing the seismic floor loads to the walls and tying the walls together. Thus the state of the wall/diaphragm connection will determine the possible load paths for transferring seismic actions down to the foundations.

11.2 Material Properties and Member Strengths

In the assessment of an existing structure, realistic values for the material properties, particularly strengths, must be used to obtain the best estimate of the strengths and displacements of members, joints and connections.

Material properties and strengths that were specified in the original design are not appropriate for use in assessment procedures.

The effect of variations in material strength on the hierarchy of failure must be considered.

11.2.1 Material Strengths

General definitions of material strengths are given in Section 5.3.

Strength assessments of existing materials may be made from the results of tests. If no test results are available, either tests should be conducted or conservative values of strength assessed by comparison with the properties of similar timbers as given in NZS 3603 (SNZ 1993), the NZ Timber Industry Federation Manual (NZTIF 2001), or other recognised source.

11.2.2 Modification Factors

The modification factors given in NZS 3603 should be used where appropriate.

11.2.3 Element Properties

The following component properties will need to be determined as set out in Section 4.4

Structural elements of the lateral-force-resisting system comprise primary and secondary components, which collectively define element strength and resistance to deformation. Behavior of the components—including shear walls, beams, diaphragms, columns, and braces— is dictated by physical properties such as area; material grade; thickness, depth, and slenderness ratios; lateral torsional buckling resistance; and connection details. The actual physical dimensions should be measured; e.g., 50 x 100 mm stud dimensions are generally slightly less due to choice of cutting dimensions and later shrinkage. Connected members include plywood, bracing, stiffeners, chords,

sills, struts, and hold-down posts. Modifications to members include notching, holes, splits, and cracks. The presence of decay or deformation should be noted.

These primary component properties are needed to properly characterize building performance in the seismic analysis. The starting point for establishing component properties should be the available construction documents. Preliminary review of these documents shall be performed to identify primary vertical- (gravity-) and lateral-load-carrying elements and systems, and their critical components and connections. Site inspections should be conducted to verify conditions and to assure that remodeling has not changed the original design concept. In the absence of a complete set of building drawings, the design professional must thoroughly inspect the building to identify these elements, systems, and components as indicated in Section 4.4. Where reliable record drawings do not exist, an as-built set of plans for the building must be created.

11.2.4 Connections

Details of all the connections need to be determined as outlined in Section 4.4.

The method of connecting the various elements of the structural system is critical to its performance. The type and character of the connections must be determined by a review of the plans and a field verification of the conditions. The connection between a timber diaphragm and the supporting structure is of prime importance in determining whether or not the two parts of the structure can act together.

11.3 Timber Diaphragms

Conventional structural analyses are based on the assumption that the roof and floor diaphragms are relatively rigid and that the weight of tributary areas on each level, including the diaphragm, can be lumped to act at points on relatively flexible shear walls. That is, the diaphragms are assumed to distribute the loads to walls parallel to the direction of lateral loading without significant out-of-plane loading of the walls perpendicular to the direction of loading. However, many unreinforced masonry buildings have flexible diaphragms (often constructed of timber) and very rigid shear walls thus invalidating the conventional assumptions.

It is important that the diaphragm flexibility and the out-of plane loading of the walls be correctly included in the analysis model. It is therefore necessary in a detailed inspection to identify the strength and stiffness properties of the diaphragms as well as the main lateral load resisting elements. It is also important to identify the properties that will influence the out-of-plane strength of the walls as well as their in-plane performance.

The expected strength of wood diaphragms should be taken as the yield capacity of the diaphragm assembly. The effects of openings in wood diaphragms also need to be considered. The presence, or lack, of chords and collectors will affect the load carrying capacity of the diaphragm. Connections between diaphragms and other components, including shear walls, drag struts, collectors, cross ties, and out-of-plane anchors, must also be considered.

The behavior of horizontal wood diaphragms is influenced by the type of sheathing, size and amount of fasteners, existence of perimeter chord or flange members, and the ratio of span length to width of the diaphragm. The presence of any but small openings in wood diaphragms will cause a reduction in the stiffness and yield capacity of the diaphragm due to a reduced length of diaphragm available to resist lateral forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings will reduce the loss in stiffness of the diaphragm and limit damage in the area of the openings. The presence of chords at the perimeter of a diaphragm will significantly reduce the diaphragm deflection due to bending, and increase the stiffness of the diaphragm over that of an unchorded diaphragm. However, the increase in stiffness due to chords in a single straight sheathed diaphragm is minimal due to the flexible nature of these diaphragms.

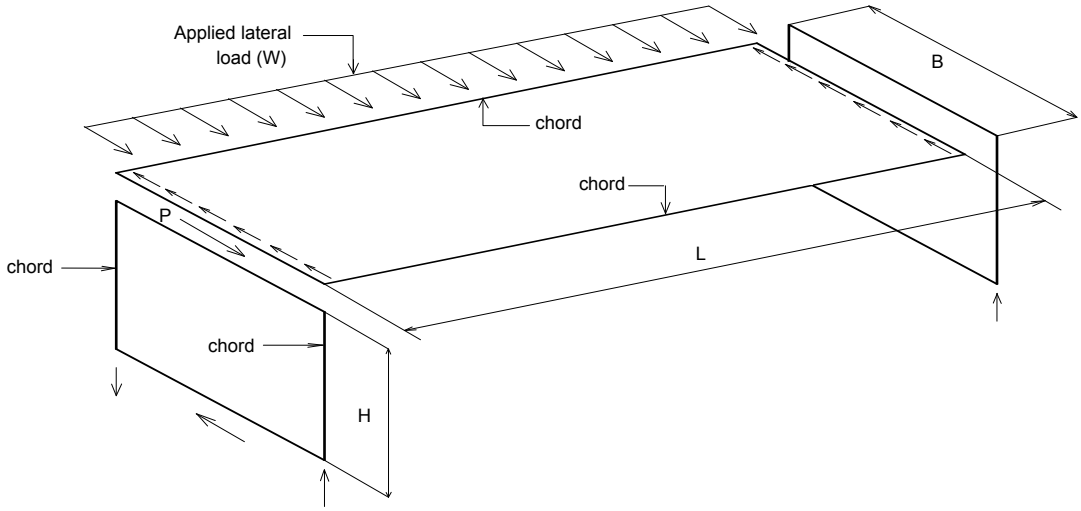


Figure 11.1: Distribution of loading for horizontal diaphragm and shear wall system

11.3.1 Existing Timber Diaphragms

These may have been constructed in one of several different forms including:

a) Square sheathing:

This consists of 25 or 50 mm thick boards, usually 100-200 mm wide, nailed in a single layer at right angles to the cross members such as joists in a floor or rafters in a roof. In a floor, the boards were usually tongue and groove in order to improve the interconnection between the boards and thus improve the load sharing ability of the system.

The sheathing serves the dual purpose of supporting gravity loads and resisting shear forces in the diaphragm. Most often, the sheathing was nailed with 8d or 10d nails, with two or more nails per sheathing board at each support. Shear forces perpendicular to the direction of the sheathing are resisted by the nail couple. Shear forces parallel to the direction of the sheathing are transferred through the nails in the supporting joists or framing members below the sheathing joints.

b) Single diagonal sheathing:

This consists of sheathing boards of 25 or 50 mm thickness and 100-200 mm wide, nailed in a single layer at a 45° angle to the cross members.

The sheathing supports gravity loads and resists shear forces in the diaphragm. Commonly, the sheathing wais nailed with 8d nails, with two or more nails per board at each support. The shear capacity of the diaphragm is dependent on the size and quantity of the nails at each sheathing board. This type of diaphragm has greater strength and stiffness than straight sheathing.

c) Double diagonal sheathing:

This consists of two layers of diagonal sheathing, one on top of the other, with the boards in one layer at a 90° to the boards in the other.

This type of diaphragm is considerably stiffer than either straight or single diagonal sheathing. One layer of sheathing is in axial tension and is counteracted by the other layer which is in compression.

d) Panel sheathing:

This consists of wood structural panels, such as plywood or oriented strand board, placed on framing members and nailed in place. Different grades and thicknesses of wood structural panels are commonly used, depending on requirements for gravity load support and shear capacity. Edges at the ends of the wood structural panels are usually supported by the framing members. Edges at the sides of the panels can be blocked or unblocked.

Nailing patterns and nail size can vary greatly. Nail spacing is commonly in the range of 75 to 150 mm on centre at the supported and blocked edges of the panels, and 250 to 300 mms on centre at the panel interior. Staples are sometimes used to attach the wood structural panels.

11.3.2 Strength and Stiffness

The strength should be based on an assessment of the materials making up the particular diaphragm and their individual strengths. Depending on the type of diaphragm, the formulae given in
2 Appendix 11B can be used to determine the diaphragm strength. In the absence of tests results, the
2 maximum values contained in Table 11.1 may be used in lieu of more detailed calculations.

2 Formulae for calculating diaphragm stiffness are given in Appendix 11A. For many diaphragms, the major component affecting the stiffness is the nail slip. In the case of initial assessment, it is sufficiently adequate to base the stiffness on the nail slip component of deformation.

Table 11.1: Strength values for existing materials

Item	Materials	Strength values	ϕ
1	**Horizontal diaphragms**		
	a Roofs with straight sheathing (sarking) and roofing applied directly to the sheathing.	6 kN/m	0.7
	b Roofs with diagonal sheathing and roofing applied directly to the sheathing.	15 kN/m	
	c Floors with straight tongue and groove sheathing.	6 kN/m	
	d Floors and roofs with sheathing and existing plaster renailed to the joists or rafters.	Add 2 kN/m to the values for Items 2(a) and 2(c)	
2	**Timber framed walls**		
	a Timber framed stud walls with wood or metal lath and plaster.	4 kN/m each side	0.7
	b Gypsum wall board, unblocked edges.	3 kN/m each side	
3	**Timber**		
	Use values from NZS 3603:1993 Table 2.2[1] as follows:		
	a Radiata Pine, Douglas Fir, Larch	Characteristic stresses as' No. 1 framing' grade	Refer to NZS 3603
	b Other timbers	Characteristic stresses for 'building' grade	

NOTE : 1 - See Table 11.2

11.4 Timber Shear Walls

Wood and light frame shear walls can be categorized as primary or secondary elements. Walls that are considered part of the lateral-force-resisting system should be considered primary elements. Walls that are not considered part of the lateral-force-resisting system, but must remain stable to support the gravity loads during seismic excitation, can be considered secondary elements.

Dissimilar wall sheathing materials on opposite sides of a wall should not be combined when calculating the capacity of the wall. Different walls sheathed with dissimilar materials along the same line of lateral-force resistance should be analyzed based on only the wall sheathing with the greatest capacity. The walls shall be analyzed based on the relative rigidity and capacity of the materials to determine if the performance of the secondary elements is acceptable.

For overturning calculations on shear wall elements, stability should be evaluated in accordance with AS/NZS 1170.0. Net tension due to overturning shall be resisted by uplift connections.

The effects of openings in wood shear walls must be considered. Where required, reinforcement consisting of chords and collectors should be added to provide sufficient load capacity around openings to meet the requirements for shear walls.

The expected strength of wood and light frame shear walls should be taken as the yield capacity of the shear wall assembly.

The behavior of wood and light frame shear walls is complex and influenced by many factors, the primary factor being the wall sheathing. Wall sheathings can be divided into many categories (e.g., brittle, elastic, strong, weak, good at dissipating energy, poor at dissipating energy). In many existing buildings, the walls were not expected to act as shear walls (e.g., a wall sheathed with wood lath and plaster). Most shear walls are designed based on values from monotonic load tests and historically accepted values. The allowable shear per unit length used for design was assumed to be the same for long walls, narrow walls, walls with stiff tie-downs, and walls with flexible tie-downs. Only recently have shear wall assemblies— framing, covering, anchorage—been tested using cyclic loading.

Another major factor influencing the behavior of shear walls is the aspect ratio of the wall. The NEHRP Recommended Provisions (BSSC, 2000) limit the aspect ratio (height-to-width) for structural panel shear walls to 2:1 for full design shear capacity and permit reduced design shear capacities for walls with aspect ratios up to 3.5:1. The interaction of the floor and roof with the wall, the end conditions of the wall, and the redundancy or number of walls along any wall line would affect the wall behavior for walls with the same aspect ratio. In addition, the rigidity of the tie-downs at the wall ends has an important effect in the behavior of narrow walls.

The presence of any but small openings in wood shear walls will cause a reduction in the stiffness and yield capacity due to a reduced length of wall available to resist lateral forces. Special analysis techniques and detailing are required at the openings. The presence or addition of chord members around the openings will reduce the loss in overall stiffness and limit damage in the area of openings

For wood and light frame shear walls, the important limit states are sheathing failure, connection failure, tie-down failure, and excessive deflection. Limit states define the point of life safety and, often, of structural stability. To reduce damage or retain usability immediately after an earthquake, deflection must be limited. The ultimate capacity is the maximum capacity of the assembly, regardless of the deflection.

11.4.1 Types of Timber Shear Walls

a) Transverse sheathing:

This consists of 25 or 50 mm thick boards, usually 100-200 mm wide, nailed in a single layer at right angles to the studs.

The sheathing serves the dual purpose of resisting the in-plane shear force caused by lateral loading. The perimeter members carry axial loading from the gravity loads and the lateral loading whereas the intermediate studs are not loaded axially by the lateral loading.

Nail slip is the dominant cause of lateral deflection in shear walls of common dimensions. Flexural strains in the chord members, and shear distortion in the sheathing itself also contribute to the total deflection.

b) Single diagonal sheathing:

The shear force applied to the shear wall is carried by tension or compression in the 45° diagonal sheathing and is transferred to the perimeter members by the nails.

c) Double diagonal sheathing:

Two layers of sheathing on the same side of the framing significantly improve the shear characteristics of a shear wall. When double diagonal sheathing is used, one layer acts in tension and the other in compression and the shear is assumed to be shared; thus, the two layers act as a shear membrane.

d) Panel sheathing:

This consists of wood structural panels, such as plywood or oriented strand board, placed on framing members and nailed in place. Different grades and thicknesses of wood structural panels, or gypsum board, may have been used on each side of the wall, depending on requirements for gravity load support, shear capacity, and fire protection. Edges at the ends of the structural panels are usually supported by the framing members. Edges at the sides of the panels can be blocked or unblocked.

Nailing patterns and nail size can vary greatly. Nail spacing is commonly in the range of 75 to 150 mm on centre at the supported and blocked edges of the panels, and 250 to 300 mms on centre at the panel interior.

11.4.2 Strength and Stiffness

The strength should be based on an assessment of the materials making up the particular shear wall and their individual strengths. Depending on the type of shear wall, the formulae given in
2 Appendix 11D can be used to determine the diaphragm strength. In the absence of tests results, the
2 maximum values contained in Table 11.1 may be used in lieu of more detailed calculations.

2 Formulae for calculating shear wall stiffness are given in Appendix 11C. For many shear walls, the major component affecting the stiffness is the nail slip. In the case of initial assessment, it is sufficiently adequate to base the stiffness on the nail slip component of deformation.

11.5 Connections

Frequently connections to masonry are nominal and cannot be relied upon for engineering purposes. Some information about the likely performance of timber diaphragm to masonry wall connections is given in section 10B.4.1 of Appendix 10B, and in Beattie (1999). However, the performance of such connections depends greatly on the level of deterioration that may have taken place in both the masonry and the timber members, and any corrosion of the bolts themselves. 2

11.6 Other Timber Elements

Whilst timber is frequently used in large residential and commercial buildings, it rarely constitutes the primary structural supporting system. Exceptions are some notable churches and the incorporation of glue-laminated timber in many industrial and some commercial buildings, largely post-1976.

Approaches taken in assessing the structural performance of timber buildings should follow the same principles, when appropriate, as those for steel and concrete buildings. Certainly, similar general performance criteria apply covering displacement, integrity and strength.

The strength values in Tables 11.1 and 11.2 may be used in assessing the strength of these elements – unless specific tests are carried out.

Table 11.2: Characteristic stresses for visually graded timber [NZS 3603:1993]

1. Moisture condition – Dry (m/c = 16% or less)							
Species	**Grade**	**Bending**	**Compression parallel**	**Tension parallel**	**Shear in beams**	**Compression perpendicular**	**Modulus of elasticity (GPa)**
Radiata pine	No.1 Framing	17.7	20.9	10.6	3.8	8.9	8.0
Douglas fir	No.1 Framing	17.7	22.1	10.6	3.0	8.9	8.0
Larch	No.1 Framing	22.7	27.1	13.6	3.5	8.9	9.6
Rimu	Building	19.8	20.1	11.8	3.8	10.9	9.5
Kahikatea	Building	14.5	19.5	8.6	3.0	5.9	6.8
Silver beech	Building	23.6	24.8	14.2	3.5	7.1	9.3
Red beech	Building	28.0	30.4	16.8	5.3	12.4	13.4
Hard beech	Building	29.5	26.6	17.7	5.0	14.2	13.6
2. Moisture condition – Green (m/c = 25% and greater)							
Radiata pine	No.1 Framing	14.8	12.7	8.9	2.4	5.3	6.5
Douglas fir	No.1 Framing	14.8	14.5	8.9	2.4	4.7	6.5
Larch	No.1 Framing	15.0	17.4	8.9	2.7	5.6	7.7
Rimu	Building	15.0	14.5	8.9	2.7	6.8	8.3
Kahikatea	Building	13.9	14.2	8.3	2.4	4.4	6.0
Silver beech	Building	20.7	19.2	12.4	2.7	3.8	7.5
Red beech	Building	25.1	18.3	15.0	3.8	7.7	11.3
Hard beech	Building	28.3	24.2	17.1	4.4	10.6	12.1

NOTE –

1. Modulus of rigidity may be estimated from G = E/15.
2. Modulus of elasticity in compression perpendicular to the grain may be estimated from E_p = E/30.
3. Grades are as specified in NZS 3631:1988.
4. For standard names of commercial timbers in New Zealand, refer to NZS 3621.

Section 12 - Detailed Assessment - Conclusions

12.1 General

The preceding Sections 4, 5 and 6 provide guidance on performance requirements, performance assessment, analysis procedures and approaches, and modelling earthquake effects on structures. Sections 7 to 11 provide detailed procedures and criteria for reviewing the demand on and capacity of building structures of concrete, steel, unreinforced masonry and timber.

To complete the performance assessment of the structure, the results of the various analyses need to be brought together and reviewed in the context of the overall performance of the structure. In particular, it will be important to identify those characteristics which impact most on structural performance. The following provide brief comments on the elements in this process.

12.2 Building Elements

Steps should include:

- review of the results of the various analyses of demand versus capacity
- identification of the critical elements in terms of overall structural performance.

12.3 Overall Structure

Steps should include:

- review of compatibility of deformations of the component elements
- review of:
  - displacements and their implications on structural performance
  - stability of the building and its components, including P–Δ effects
  - overall structural integrity.

It will be necessary to determine which of the effects governs the overall performance of the structure and to record the reasons and the results in terms of percentage of New Building Standards.

12.4 Conclusions

During the course of the detailed assessment of an existing building there will be a wide variety of issues to be addressed. Each will require engineering judgement and assumptions as to material quality, detailing and even structural configuration.

It is vital that the overall result be determined in the context of the whole building and the particular combination of elements it has, structural and "non-structural". The initial evaluation process and associated forms provide a reasonable check list of issues to be considered in this regard.

In order to provide a focus for the assessment, written conclusions on the following should be recorded:

- Overall, considering all the aspects reviewed, what is the percentage of new building standard, *%NBS*?
- Given the *%NBS*, what is the allocated grading of the building on the NZSEE Scale?

- What are the key issues to be addressed to improve the structural performance of the building to an acceptable level?
- In particular, what are the critical structural weaknesses?

A succinct and carefully reasoned summary of the engineer's assessment will provide the best possible basis for determining the actions necessary to safeguard the interests of the owner, the relevant territorial authority, and the community in general.

Figure 12.1 has been prepared to assist those making assessments to record key deails about the building.

Assessment of Structural Performance of Buildings in Earthquake
Summary of Building Features

Building Name:		**Ref:**
Location:		**Date:**
		By:
Principal Use:	**Year built:**	**Design Code:**

General Description:

Structure Description

Element	*Material*	*Type*		*Material Properties*		*Comment*
Roof						
Walls						
Frames						
Floors						
Ground Floor						
Basement						
Foundations		Strip	Pad	Raft	Piles	Other

Earthquake System/Parameters

	System (eg shear walls)	Period (sec)	Ductility (μ)	Coeff (C_h)	RP Factor (R_u)	SP Factor (S_p)	Sep,n (mm)
Direction 1:							
Direction 2:							

Comment: e.g. CSW's:

Zone Factor:		**Site Subsoil Class:**				**Return Period:**
Assessment Basis:	**ESA**	**EMA**	**SLaMA**	**LPA**	**ITHA**	**Other**

Site Subsoil Characteristics

Description:

Strength Parameters:	**Cohesive**		**Cohesionless**		**Comment:**
	udss (kPa)	**depth** (m)	**SPT** (N)	**depth** (m)	

Gravity Loads

	Roof	**Floor**	**Floor**	**Gr. Floor**	**Basem't**	**Comment:**
DL (kPa)						
LL (kPa)						

Sketch / Other Information:

Figure 12.1: Summary of Building Features

Section 13 - Improvement of Structural Performance

13.1 General

This section provides guidance on ways to improve the structural performance of buildings in earthquake. It is essentially an expanded check list of possible solutions, both global and detailed. In expanding the check list, descriptions of the techniques are given, together with a commentary on design considerations for each.

While the range of approaches and solutions is reasonably comprehensive the lists do not claim to cover every possible approach or technique.

13.2 Performance Objectives and Criteria

The basic aim of improving structural performance in the context of these Guidelines and the proposed legislation is to reduce the earthquake risk from existing buildings. The approaches and methods given in this section apply equally to buildings deemed to be not safe in earthquake according to the legislation, and to other buildings of lesser risk. The aim of structural performance improvement should be to achieve as near as reasonably practicable to 100*%NBS*. Considerable judgement is required to determine a level of improvement appropriate to any particular case. The NZSEE strongly recommends that the attainment of not less than 67% of New Building Standard (60*%NBS*)). Even this level represents a significantly higher risk than for a building of 100% NBS.

The hierarchy of Performance Measures and relationship of performance to attainment of Ultimate Limit State (ULS) is given in Section 4: Performance Objectives. Particular attention should be paid to the way relative risk increases as the performance measure (percentage of new building standard - *%NBS*) goes down. Figure 4.1.

The standard required for improving structural performance for any particular building should be a matter of discussion between the owner, structural designer and the territorial authority. The starting point for such discussions should be the achievement of 100*%NBS*. Although the legal minimum for a building of ≤ 33*%NBS* is , by default (as it is not mentioned in the Act) 34*%NBS*, it is the NZSEE's strong recommendation to bring the building to "as near as is reasonably practicable" to that of a new building. There should be a resolve by all parties to achieve 100*%NBS* if that is practicable. In any case improvement should be at least to 67*%NBS* unless special circumstances exist that can be used to justify the additional risk involved to occupants.

Even if a building passes the 33% threshold, serious consideration should be given to improving its performance, particularly if it is below 67*%NBS*. This may allow the improvement to be planned to coincide with a general refurbishment or change of use.

Nevertheless, it is recognised that a wide range of buildings and circumstances will be involved. Ideally, any building should be brought up to 100*%NBS*, and this should be done if it can be done economically. However, the underlying aim of the legislation is to cause a reduction in earthquake risk represented by existing buildings. It will be far better to bring a building from say 35 to 60*% NBS* than to do nothing because achievement of 67% involved a quantum jump in expenditure.

The approaches and techniques available to improve structural performance in earthquake give considerable scope to arrive at a solution that is effective, economical and not unnecessarily intrusive to the functions within the building.

13.3 Strategies for Improving Structural Performance

Improving the structural performance of buildings in earthquake may be achieved by adopting one or more of the strategies outlined in this section of these Guidelines.

Designers are required to carefully consider issues of relative stiffness and relative ductilities between the existing structure and new strengthening elements.

Strategies include identification of weak or brittle elements that form part of the seismic resisting structure for strengthening.

Other strategies involve structural improvements to mitigate poor building global behaviour such as soft storey mechanisms or highly torsional responses.

Ideally, unstrengthened and/or strengthened buildings will have an adequate level of redundancy so that localised failure or overload of a few elements will not precipitate overall instability or collapse of the building.

13.3.1 Local Modification of Components

While some existing buildings have substantial strength and stiffness, often some of their structural components are understrength or they have inadequate deformation capacity.

A strategy for this type of building, could involve local improvements to those components that are inadequate while retaining the basic form of buildings' lateral force resisting system.

Local improvements that can be considered include improving component connectivity, component strength, and/or component deformation capacity.

This strategy can be a cost-effective method to improve the seismic performance of a building when only a limited number of components are inadequate.

Local strengthening could include measures such as adding a plywood overlay diaphragm over an existing timber floor or by adding concrete facings to the column elements of heavily perforated wall-type of structures.

Local improvements that improve the deformation capacity or ductility of components can allow them to survive large displacements without necessarily increasing the component strength. For example, placing steel jackets around reinforced concrete columns can allow the columns to deform without loss of strength through spalling, degrading flexural reinforcement splices or shear failure in plastic hinge zones.

13.3.2 Removal or Lessening of Irregularities and Discontinuities

Stiffness, mass and strength irregularities are common causes of inadequate seismic performance of buildings. Checking seismic displacements, and forces often identifies high concentrations of forces within one storey or on one side of a building. Similarly, when checking mode shapes and building deformations, unbalanced displacements will indicate the presence of a discontinuity in the structure. For example, shear wall type buildings with shear walls of differing heights will generally develop very high floor (transfer) diaphragm shear stresses. Removal or separation of the

structural element causing an irregularity or discontinuity may be sufficient to reduce seismic demand on the overstressed elements to acceptable levels.

Sometimes, building performance can be improved by deliberately weakening some elements. For example with a highly torsional shear wall type of building carefully selected walls may be split vertically so as to reduce their flexural capacity and lower the shear demand on those walls. By splitting the walls in that manner the torsional response of the building can be reduced to achieve benefits of reduced displacement ductility demand on many of the structural elements.

13.3.3 Global Structural Strengthening and Stiffening

Some flexible structures will have poor seismic performance because critical components or elements do not have adequate ductility to resist the large seismic deformations usually associated with that type of structure.

For structures with many such elements a cost effective way to improve performance is to stiffen the structure so as to reduce the ductility demand on those critical components. By stiffening the structure the building period will reduce and the elastic strength demands on the lateral force resisting system will typically increase. Stiffening a structure is usually accompanied with an increase in seismic strength.

Construction of new braced frames, moment resulting frames or shear walls within an existing structure are effective methods for adding both additional stiffness and strength.

By providing supplementary strength to the lateral force resisting systems, it is possible to raise the threshold of seismic intensity at which the onset of damage occurs.

Care is needed to ensure that the new strengthening elements are compatible with the stiffness of the existing elements so as to avoid premature or brittle failure of those elements.

13.3.4 Seismic Isolation

An alternative to strengthening a weak building is to substantially isolate it from damaging seismic ground motions.

Base isolation produces a structural system, incorporating superstructure and isolation bearings, with a fundament response that corresponds to nearly rigid body translation of the superstructure above the isolation plane. By base isolating a building the building period is usually extended out to 2 or 3 seconds – substantially reducing the seismic response into the superstrucutre. Also, the isolation bearings are usually designed and built to incorporate a high level of seismic damping that further reduces the seismic response into the superstructure.

The seismic demands on the superstructure, the non-structural components and contents are greatly reduced.

Base isolation is often an appropriate strategy to achieve the enhanced levels of seismic protection required for heritage buildings, buildings housing valuable collections or critical equipment.

Most of the seismic deformation induced in a base isolated system occurs over the height of the bearings. Deformations of up to 300–600 mm are common with base isolated buildings.

Base isolation is most effective for relatively stiff low height buildings with a large seismic mass. This technique is less effective for light, flexible structures and tall buildings. Base isolation of existing buildings is technically complex, and it usually involves very detailed and careful

underpinning and foundation strengthening techniques. It can be a relatively costly technique to implement.

13.3.5 Supplementary Energy Dissipation

More technologies are becoming available that allow the seismic energy imparted to a structure by ground motion to be dissipated in a controlled manner through the action of special devices such as hydraulic cylinders yielding plates, yielding braces or friction joints, resulting in an overall reduction in the displacements of the structure.

The most common devices dissipate energy through friction, hysteretic or visco-elastic processes. The energy dissipated is proportional to the amount of displacement induced in hysteretic devices or the instantaneous velocity for visco-elastic devices. These systems are generally most effective in structures that are relatively flexible and have some inelastic deformation capacity.

Depending on the characteristics of the device; either static or dynamic stiffness is added to the structure as well as energy dissipation capacity (damping).

In some cases, although the structural displacements are reduced, the seismic forces acting on the structure can be increased (as a result of the building period being shortened). Like base isolation, this is a technically complex strategy that requires specialised analysis for design. It tends to be more cost effective though than base isolation.

13.3.6 Removal of Unnecessary Seismic Mass

Many older style existing buildings have heavy non structural components. Removal of some of the heavy elements can assist by reducing the overall seismic mass of the building. Roof mounted concrete water tanks, heavy masonry interior non structural partitions, exterior veneers, brick infills and/or heavy masonry chimneys can be considered for removal.

By removing these elements, it can save the expense of seismically strengthening them. In addition, removal of some elements such as infill panels and heavy non-structural partitions can lead to improvements to the structural performance of the building. Care is needed though, to ensure the removal of the unnecessary non-structural elements does not create a discontinuity or lead to increased eccentricity of seismic mass at any floor level.

13.3.7 Widening Seismic Joints

Many existing buildings, or existing building elements have insufficient seismic separation to move freely during an earthquake. Insufficient seismic separation between buildings will result in seismic pounding between buildings with resulting local damage that can precipitate more serious loss of stability of columns and the like.

Insufficient separation between structural frames and non-structural walls can lead to a mid height shear friction type of failure in the walls that can in turn lead to high column shears if the effective lengths of the columns are reduced.

Generally, only very limited opportunities are available to widen seismic gaps between buildings. Sometimes, cantilever concrete floors on one side of a seismic joint can be cut back but most times pounding between floor slabs (where they are approximately at the same level) is inevitable.

Widening out seismic joints between frames and infill panels is often possible along with establishing sliding connections at the tops of those panels. Generally, new face load supporting

structure will be required to support infill panels separated from adjacent columns and from the underside of the beam at the top of the panel.

13.3.8 Linking Buildings Together across Seismic Joints

Some buildings are comprised of several seismically separate structures often with completely inadequate width seismic joints between them. Often these structures can benefit from the installation of linkage nodes between the separated structures that can transfer axial loads and seismic shears in a controlled manner. With careful design and detailing it is possible to achieve controlled "articulated" movement between buildings and to use the excess seismic strength of one building to assist in supporting its neighbour.

Careful analysis is required particularly where neighbouring buildings have quite different strengths, stiffnesss and building periods.

13.3.9 Seismic Emergency Gravity Supports

Some columns on existing buildings can be vulnerable to severe damage leading to collapse during an earthquake. For example, with a frame building that has columns of varying heights the shortest columns will generally hog the seismic storey shear until they fail in shear. Often columns on some buildings have insufficient displacement ductility capacity to survive a design earthquake.

Rather than strengthen the columns to allow them to survive the design earthquake an alternative strategy is to install supplementary seismic emergency columns immediately adjacent to them.

These emergency columns are usually fabricated from steel, are nominally pin ended and are fitted from slab face to slab face or from top of beam to underside of beam at the level above.

This technique can be especially useful to provide alternative primary supports at exterior wall lines because other column jacketing or fibre wrapping techniques generally require removal of the building façade or exterior claddings.

Clearly, the damaged columns cannot be relied upon to assist the lateral strength of the building but they can allow the full development of the seismic resisting elements of the building while preventing the premature collapse associated understrength non-ductile columns.

13.3.10 Strength and Stiffness Criteria

The assessment of strength and deformation capacities of existing components and elements should be based on the probable or expected values of material properties in the building unless otherwise specified in the material sections of this document .

Where rehabilitation or structural enhancement of existing lateral force resisting components are undertaken, it is appropriate to adopt the stiffness assumptions, strength criteria and acceptable deformations applicable to the existing elements. Unless other procedures are specified in this document, the design of such rehabilitation or structural enhancement shall be in accordance with the procedures specified in current material standards and/or recognised guidelines except the strength reduction factor may be taken as unity. Where however the rehabilitation results in a considerable enhancement in strength, in excess of (say) 50 % of that of the original component strength, the strength reduction factor of the applicable material standard should be adopted unless lower bound material strengths are used in the assessment of the strength of the rehabilitated components.

Default lower bound values shall be taken as expected strengths divided by a factor corresponding to 1.5, 1.3. 1.25 and 1.1 for concrete, masonry, steel reinforcement and structural steel respectively unless otherwise determined by testing (from statistical mean minus one standard deviation).

Where new support elements are incorporated to add strength and stiffness to an existing building, stiffness assumptions, strength criteria and acceptable deformations associated with these shall follow the requirements set forth in current material codes and/or recognised guidelines, adopting the strength reduction factors laid down in these standards**.**

13.4 Global Strengthening

The following table provides descriptions of global strengthening options for dealing with the whole building, or at least stabilising it in one direction.

Table 13.1: Global strengthening approaches

Description	Design comment
1 **Shear walls** New concrete shear walls or concrete overlay shear walls on substantial foundations or strengthened existing foundations can be used to increase the total seismic resistance of a building. An alternative approach is infilling existing frame openings with reinforced concrete to convert existing frames to shear walls.	The strength and behaviour of many buildings can be significantly improved with the addition of new shear walls. The ductility of these walls can be set to match the available ductility of the existing structure. Often the new shear walls will significantly stiffen the building, shorten its period, and thus increase the seismic base shear co-efficient. The walls can be designed to current concrete or steel design standards. Where concrete wall infills are used provision of new wall boundary elements or upgrading (eg.jacketing) of the existing frame columns may be required. Steel or concrete drag ties may be required to engage sufficient length of the floor diaphragms.
2 **Pin based "strong back" walls** Pin based walls are walls that act as "strong backs" up the height of the building. These walls are well tied to the foundations and to the floor diaphragms at each level. No attempt is made to transfer flexural actions from the wall to the foundations nor to the beams at each level. The connections at all levels to the strong back walls are notional pin connections.	A very useful technique to suppress the critical weakness of a soft storey mechanism. Many buildings have irregular floor heights, irregular vertical stiffness or a strong torsional response at one level. With careful design pin based "strong back" walls can prevent soft storey behaviour by redistributing the seismic actions up the height of the building. The forces required to even out the deflected shape of the building are used for the flexural design of strong back walls. Wall shear strength and the "pinned" diaphragm connections should be designed for overstrength actions. Special care is needed to tie the compression edges of these walls to the building structure to ensure wall edge stability.
3 **Moment resisting frames** Moment resisting steel or concrete frames are often added to existing "shop front" type buildings that have very little seismic resistance along the glazed "shop front" of building. The moment resisting frames minimise the physical intrusion into the building. Concrete and steel frames are detailed for the level of ductility required of them.	The strengthening frames usually include a "foundation" beam so that the full flexural strength of the columns can be developed both at their top and bottom. The frames are usually required to be relatively stiff so that they are reasonably compatible with other existing lateral resisting structures. Frames of limited ductility are often required to suit stiffness and strength demands. Floor diaphragm enhancements are usually necessary to transfer the diaphragm design forces to/from the strengthening frames.

Description	Design comment
4 **V-braced frames** Steel eccentric and concentric V braced frames can be added to existing buildings to increase the total seismic resistance of a building. They are usually comprised of two new steel columns that support a steel collector beam at each floor level. Angled steel braces run from the column/beam junction at each floor level to the midspan of the steel beam at the floor above.	Concentric V braced frames should be designed for elastic response design actions in accordance with NZS 3404. Eccentric V braced frames should be designed for an inelastic response with a yielding/ductile shear link situated mid length of the collector beams, between the two brace connections. All columns, braces, foundations and connections are designed to resist the overstrength actions of the yielding/ductile shear link. The design procedure is in full accordance with NZS 3404. Often steel or concrete drag ties are required to engage sufficient length of the floor diaphragms.
5 **Cross braced frames** Cross braced frames installed vertically in the planes of walls or horizontally in ceiling, floor or roof planes have many applications for strengthening existing buildings. The cross braces are generally steel installed between new or existing beams and columns. Steel braces are often notched for predictable tension yielding.	Concentric cross braced frames will generally be relatively stiff and are often well suited to strengthening relatively “brittle” types of existing structure. Care is needed to avoid unnecessary slip in connections of steel braces to existing concrete frame elements. Generally, scabbling concrete surfaces, roughening steel plates and high strength grouting between steel base plates and existing concrete surfaces will be required.
6 **Yielding braced frames** In existing concrete or steel frame buildings of low strength and/or ductility compression/tension yielding steel braces can be installed in a “chevron” pattern between adjacent beam/column joints. The yielding braces usually take the form of a yielding steel flat or angle continously supported within a concrete filled steel tube. The braces have carefully designed overstrength end regions.	The braces have non-linear axial behaviour with excellent hysteretic behaviour. They can deliver high levels of element ductility. Care is needed to correctly model their non-linear behaviour when they are providing supplementary strength to otherwise elastically or nominally ductile buildings. Detailing the braces for axial deformations is very important. Correct stability support for the braces when they yield in compression is essential. Steel or concrete drag ties may be required to engage sufficient length of the floor diaphragms.

13.5 Strengthening Building Elements

Techniques for strengthening building elements are included in Table 13.2, covering the following:

- columns/piers
- beams
- beam/column joints
- footings
- floor roof and ceiling diaphragms
- shear walls.

Table 13.2 Techniques for strengthening building elements

Description	Design comment
1 **Columns/piers** 1.1 ***Concrete columns steel jackets*** Seismic behaviour of columns can be improved using circular or elliptically shaped thin steel jackets to encase existing rectangular or circular columns. The jackets are continuously site welded then grout or concrete filled. The jackets will normally extend from floor level to the undersides of the beams above.	Steel jackets can be designed to increase column confinement, provide restraint against buckling of longitudinal bars, provide additional shear strength and provide additional lap bond strength. Generally the steel jackets don't increase the flexural strength of the column and but they will usually reduce the effective plastic hinge length of the column in column yielding sideways mechanisms.
1.2 ***Concrete columns – composite fibre wrapping*** Seismic behaviour of circular and/or rectangular columns can be improved by wrapping the columns with specialist synthetic fibres bedded into an epoxy or other bonding material. High strength composite fibres such as carbon fibre or glass fibre are generally used. The fibre wraps are predominantly unidirectional to provide good confinement. Rectangular column corners need to be radiused to suit the bend radii of the specified fibres.	The fibre wraps can be designed to increase column confinement, provide restraint against buckling of longitudinal bars and provide additional shear strength and lap bond strength. Generally, the fibre wraps don't increase the flexural strength nor stiffness of the columns and they will usually allow the columns to develop full length plastic hinge zones in column yielding sideways mechanisms. Refer to the manufacturers of the fibres for specific design guidance and construction specification requirements.
1.3 ***Additional Concrete Jackets/Skins to Columns and Piers*** Concrete jacketing can be used to improve the deformation and shear capacity of columns and piers. The concrete jacketing incorporates transverse confinement reinforcement at reasonably close centres, comprising outer hoops around the perimeter of the existing column section and, commonly, cross ties drilled and anchored into the core of the existing column or passed right through the column. Nominal longitudinal reinforcement is provided to support the transverse stirruping but can be used to improve flexural strength where fully anchored into adequate foundations and/or continuous through the floor system at each level.	The concrete jacketing is designed to increase column confinement, provide restraint against buckling of longitudinal bars in the existing column section, provide additional shear strength and lap bond strength. Additional flexural strength can be provided where the longitudinal reinforcement (and jacketing) is continuous through the floor system at each level. If the purpose of the jacketing is to increase the ductility but not the flexural strength of the column, the longitudinal reinforcement in the concrete jackets should be discontinued a short distance from the connection with adjacent components.

Description	Design comment
	Concrete jackets placed to improve ductility may also enhance the flexural strength due to the increase in section size. Where jacketing is not continuos this may shift the ductility demands to adjacent sections. This needs to be checked and appropriate steps such as extending the extent of jacketing considered. Measures need to be implemented to provide shear transfer between new and existing materials where composte action is required, such as for increase in flexural strength, such as scabbling of the surface of the existing column.
2 **Beams** 2.1 ***Concrete beams*** ***Steel hoops for shear/confinement*** Hoops are installed in two pieces and full strength site welded to encapsulate the beam. Holes are drilled or cut at regular centres through the floor slab adjacent to the sides of the beam. The top surface cover of the concrete beam is removed so that the steel flat hoops can be recessed below floor level. The sides of the beam are scabbled at the location of the hoops and the gap between the hoops and the beam are pressure grouted with high strength cementatious grout.	To provide adequate confinement and to develop strut-tie actions to improve the shear strength of concrete beams over plastic hinge zones it is usually necessary to place steel flat hoops right around the beams at regular centres. The spacing of the steel hoops is determined by shear and the antibuckling requirements of the longitudinal reinforcement.
2.2 ***Concrete beams*** ***Composite fibre wrapping for shear/confinement*** Composite fibre hoops are tightly wrapped completely around the concrete beams at regular centres to act as stirrups for shear enhancement and to provide antibuckling restraint and confinement to the longitudinal beam bars. Holes are drilled at regular centres through the floor slab adjacent to the sides of the beam. The top and bottom edges of the beam are carefully radiused to suit the fibre wrapping.	It is usually necessary to fibre wrap right around the concrete beams to develop strut-tie actions to improve shear strength and to provide the antibuckling restraint over the length of the plastic hinge zones. The spacing of the hoops of fibre is determined by shear and antibuckling requirements. Refer to the manufacturers of the fibres for specific design guidance and construction specification requirements.
2.3 ***Concrete beams*** ***External post tension to enhance flexural and shear strength*** Post tensioning may serve to increase the flexural and shear strength of concrete beams. Deficiencies in reinforcement development and splices may also be reduced given tension stress levels are reduced. Post tensioned reinforcement should be unbonded within a distance equal to twice the effective depth from sections where inelastic action is expected.	It is preferred not to bond the post-tensioned reinforcement in regions where inelastic response is expected. Bonded reinforcement is more likely to undergo inelastic strain that may relieve the post tensioning stress. Anchorage zones should also be located away from inelastic regions because of the potential for anchorage damage in these regions. Joint shear stength may also be increased by post-tensioning.

Description	Design comment
3 **Beam/column joints** 3.1 ***T-brackets to develop MRF actions*** The seismic capacity of existing post and beam types of construction can often be increased by adding fabricated steel channels, welded to form large T shaped brackets to the post and beam junctions. Existing timber framed warehouses can often be strengthened by bolting these purpose design and made T brackets to the post and beam junctions to develop moment resisting frame actions.	Often existing timber framed warehouses have utilised hardwoods for posts and beams. Bearing capacities of bolts in these timbers can be impressive. Usually, the steel T bracket will be designed to provide a nominally ductile or limited ductile response and bolts will be design to develop the overstrength actions from the T brackets. Limiting the slip in these connections can often be achieved by using oversize holes and grout spaces between the bracket and the timber beam then filling the oversize holes and grout spaces with epoxy mortar or high strength cementatious grouts.
3.2 ***Steel or concrete jacketing of joint zones*** Where concrete jacketing is used to increase the flexural strength of columns, jacketing needs to be passed through the floor system to ensure the transfer of enhanced strength between columns and beams in addition to improving confinement. The new column reinforcement is passed thorugh the floor system and encased in concrete jacketing. Steel jacketing may be used to increase the flexural strength of columns at each level using longitudinal flat steel plates or angle sections at column corners passed through the floor system and transversly linked by steel straps or rods passed through holes drilled though the adjacent beams.	Where used for enhancement of flexural strength with jacketing, longitudindal reinforcement or steel strap reinforcement is passed and grouted through holes cored or brokenthrough the floor slab adjacent the corners of the existing column section. The transverse reinforcement to jacketing is commonly installed though holes drilled through the concrete beams framing into the column at each floor level.
4 **Footings** 4.1 ***Extra footing area and/or overlay pads*** The seismic load capacity of pad-type footings can be increased by increasing the footing area by casting a concrete surround to the perimeter of the pad. In many cases, the existing pads will require an overlay “slab” well anchored to the existing pad and to the base of the column to increase the strength of existing pads.	When widening existing footings it is often necessary to increase the flexural, shear and punching shear capacity of those footings. Overlay slabs are usually designed to deliver composite action with the existing pad. Careful consideration of interface shear strength demands is required between the new overlay slab and the existing foundation pad and the base of the column.
4.2 ***Rocking foundations*** Many existing buildings have been designed for low seismic overturning actions. Often existing (or new) shear walls will commence rocking well below the design seismic load level for the building. Often, new concrete shear wall facings on existing walled structures, will have insufficient length to preclude rocking. Basic foundation integrity is required with the connection of shear walls to foundations so that predictable and reliable rocking action can occur.	Provided the ground conditions are suitable, inelastic foundation rocking actions at the design seismic level can be a satisfactory means of seismic energy dissipation. A non-linear analysis should be used and good information on the likely soil stiffness is required. High levels of inelastic actions through rocking should be avoided and the deformation consequences of rocking on the other building elements should be checked to ensure that adequate load paths are maintained and their displacement capacity is not exceeded.

Description	Design comment
4.3 *Additional piles for tension and/or compression enhancement* The seismic load capacity of shear wall or pad type footings can be increased by attaching additional piles and pile caps to those foundations. Pile selection will be dependent on the founding soil type and the practicalities of piling in close proximity to existing building elements. Pile caps and/or overlay pads and foundations are usually required to attach the new piles to the existing footings.	Comments noted in 4.1 apply to the pile caps and/or overlay slabs. Some pile types, for example steel screw piles will have a very low modulus of stiffness as they transition from compression to tension (i.e. "sloppy"). If concentric placement of piles is not possible then care is needed to properly design for all eccentric design actions. Generally, additional flexural and shear strength is required as part of the foundation pad strengthening detail.
5 **Diaphragms** 5.1 ***Diaphragm drag ties/collectors*** Drag ties can be constructed as a reinforced concrete tension/compression element or as a steel flat, angle or channel tension only element. Shear transfer to/from the drag tie can be by reinforcing rods epoxy grouted into existing concrete members or by using headed stud connectors to steel members. Steel ties are often positioned just clear (say 20–25 mm) of the concrete elements they connect to so that high strength cementatious grout can be pressure injected into the gap for maximum shear transfer. Concrete surfaces are scabbled and steel surfaces are often deliberately roughened with random runs of overlay weld.	Many existing buildings have inadequate load paths to transfer seismic forces in and out of shear walls or frames. Steel or concrete drag ties can be designed to "gather up" the inertia forces from adjacent floor or other building elements and tie those elements back to shear walls etc. Similarly, drag ties can be used to connect shear walls and/or frames together to achieve a better distribution of lateral forces to these elements. The drag ties are generally aligned along the axis of the shear walls unless stabilising ties are used to balance out the out of plane forces associated with drag ties angled to the main axis of the shear wall.
5.2 ***Concrete Diaphragm struts and ties*** Diaphragms transmit inertial forces in a structure to the lateral load resisting systems. Concrete diaphragms typically comprise slabs, collectors and chords. Diaphragm action may alternatively be considered as a structural truss in the horizontal plane comprising struts, ties and chords. The strength of diaphragms may be enhanced by the provision of additional reinforcement and concrete encasement or of structural steel plate or alternative sections along appropriate strut and tie lines to the slab diaphragm. Shear transfer to/from the existing diaphragm slab can be by reinforcing rods expoxy grouted into the existing concrete slab and beam members or by epoxy fixing and bolting steel plate sections to the slab.	A strut and tie approach to diaphragm design is becoming more commonly adopted in building design, particularly in slabs with major openings, irregular floor plans and irregular spaced lateral load resisting systems. Improvement of the strength of the individual strut and tie components of a diaphragm will often prove more cost effective than alternatives. Diaphragm thickness may be increased as an alternative approach but the added weight will increase the seismic load as well as increase footing loadings.

Description	Design comment
5.3 ***Diaphragm steel cross bracing*** Horizontal steel cross bracing can be used to strengthen or replace a weak existing diaphragm, They are typically used in buildings with structural steel framing systems with the existing steel floor beams used as the truss chord elements. The bracng arrangement may be one of various structural shapes. For lightly loaded conditions diagonal threaded rods are commonly used as tension only members. For more heavily loaded diaphragms steel tubes, rectangular hollow sections or column/beam sections are used acting in compression as well as tension. Truss element connections are usually concentric to maximise stiffness and ensure truss members act under axial loading only.	Concentric diagonal steel diaphragm bracing should be designed for seismic loads derived from the Parts and Portion requirements of NZS1170 Part 5 and in accordance with NZS 3404. Existing chord components may be strengthened by the addition of steel plates or sections to existing steel elements or the provision of additional steel plate/section reinforcement fixed to existing concrete beam or slab elements acing as chords. They should be bonded and bolted to existing concrete chord elements to enhance the composite action.
6 **Shear walls** 6.1 ***Concrete Skin Walls*** Concrete "skin" walls are often used to increase the shear and flexural strength of existing walls or heavily perforated wall type structures. Constructing a "skin" on the inside (or outside) of these walls with carefully designed shear reinforcement can significantly increase the shear capacity and ductility of these walls. Concrete skin walls are also used to enhance the shear strength (and sometimes flexural strength) of plain cantilever walls.	Many existing concrete buildings have heavily perforated walls for regular patterns of windows or doors. The concrete piers between the openings are often relatively thin and prone to diagonal shear cracking as inelastic seismic actions are concentrated in the piers. Often, relatively little flexural reinforcement is required (or desired) to limit the overstrength shear capacity of the piers. The flexural reinforcing in the existing walls is often adequate and the focus of the strengthening regime is to provide overstrength shear reinforcement and detailing for ductility.
6.2 ***Post tensioning concrete Shear walls*** The in-plane flexural and shear capacity of walls can be enhanced using bonded or unbonded post tensioned tendons, either fixed to the exterior of the walls or within cores through the wall interior.	The use of unbonded post tensioned reinforcement is preferred as bonded reinforcement is more likely to undergo inelastic strain in regions where inelastic response is anticipated that may relieve the post tensioning stress. Where bonded reinforcement is used this should be well away from the inelastic regions. The tendons need to be well anchored at foundation level to ensure the levels of prestressed can be attained. Allowance needs to be made for loss of prestress force due to creep and shrinkage. The shear capacity of the wall needs to be checked to ensure the flexural strength of the wall can be developed. Additional shear strength enhancement may be required over that provided by the increase in axial load resulting from the addition of post tensioned tendons.

Description	Design comment
6.3 ***Composite fibre overlays*** The use of high strength composite overlays, such as carbon or glass fibre sheets or bands, epoxied to the surface can be used to enhance the stiffness and strength of existing concrete and masonry walls. They are used as tensioning reinforcing and can therefore increase both the in-plane and out of plane strength of walls.	Carbon or glass fibres, woven into fabric sheets are appled to the surface of the wall using an epoxy resin binder and can be orientated in one or two directions. Several layers and orientations can be used depending on the design requirements. Carbon fibres have a modulus of elasticity and tensile strength greater than that of steel, whilst glass fibres have a lower modulus of elasticity and tensile strength. Both glass and carbon fibres exhibit brittle behaviour in tension. Debonding of the fibres from the wall usually results under out of plane loadings.

13.6 Strengthening Unreinforced Masonry or Unreinforced Concrete Buildings

Techniques for improving the performance of URM buildings are given in Table 13.3, covering the following:

- in-plane strengthening
- face load strengthening
- combined face load/in-plane strengthening
- diaphragm strengthening
- chimneys towers and appendages.

Unreinforced masonry or unreinforced concrete buildings require special consideration as they are often quite brittle. Generally, the walls are poorly connected to adjacent floors and roof structures. Most of this type of buildings have timber suspended floors with limited diaphragm integrity or strength.

The quality of the mortar in masonry building is extremely variable with some lime mortars deteriorated to have almost no reliable shear strength.

Table 13.3: Techniques for strengthening unreinforced masonry or unreinforced concrete buildings

Description	Design comment
1 **In-plane strengthening** 1.1 ***Concrete shear walls and wall facings*** Overlay concrete shear walls to existing masonry walls can be used to improve both the seismic resistance and ductility of unreinforced masonry buildings. Overlay walls can be constructed from insitu concrete or reinforced sprayed or shotcrete concrete. These are usually distributed throughout the building to minimise concentration of strength demands and additionally provide increased face loading strength.	The use of shotcrete or insitu concrete overlay walls provides a strengthening system of comparable stiffness to the original masonry, ensuring the added strength can be mobilised prior to onset of unacceptable or excessive damage to the original building structure. The walls are usually designed to mobilise the weight of the existing structure to resist overturning demands. Commonly the shotcrete or concrete layer is designed to resist all lateral forces, however the masonry wall with the concrete skin can be considered to behave as a composite section. Adequate anchorage needs to be provided at the concrete/masonry interface for shear transfer of both gravity and lateral loads. Walls from 100 mm (minimum for structural purposes) to up to 300 mm thick can be successfully sprayed. Design should be to current concrete standards, for limited ductile or nominally elastic response design actions as the low aspect ratio of the walls to most masonry buildings precludes achievement of a ductile flexural behaviour. Account needs to be taken of the insitu concrete strength of shotcrete concrete, as measured from core testing, often only reaching as low as two thirds of corresponding cylinder strengths.

Description	Design comment
1.2 ***Concrete frames*** Concrete movement resisting frames can be used to provide strengthening to a building without significantly increasing a building's stiffness. Given they are relatively open structures, they can often be installed with minimal impact on a building's architecture or floor space.	Deflection compatibility with the existing masonry structure requires careful consideration. The new concrete frames should be sized to provide similar stiffness to that of the masonry walls. Relatively stiff concrete frames can be useful to provide additional seismic strength to buildings with heavily pierced masonry walls. Many masonry buildings have relatively low deformation capability and hence the usefulness of framing strengthening can be quite limited.
1.3 ***Flexural rods to rocking piers*** The in-plane flexural capacity of unreinforced masonry piers can be enhanced beyond their rocking strength, by the installation of reinforcing steel bars grouted or cemented into drilled holes through their core or alternatively exterior reinforcement fixed to the outer faces of the piers.	Reinforced-cored or exterior reinforced masonry piers can be considered to act as composite reinforced masonry piers as long as sufficient bond between the new reinforcement and masonry is achieved. The vertical reinforcement should be well anchored beyond the base of the piers. New vertical reinforcement can be considered to contribute to the shear (sliding joint shear) capacity of the piers.
1.4 ***Axial post strengthening*** The in-plane flexural and shear capacity of rocking piers can be alternatively enhanced using bonded or unbonded post tensioned tendons, either fixed to the exterior of the piers or within a core through the pier interior.	Post tensioned masonry piers or walls should be considered to behave as unreinforced masonry walls with increased vertical compression load. Allowance needs to be made for loss of prestress force due to creep and shrinkage. Care is required with anchorage zones to spread the anchorage stresses. Low levels of prestress are recommended so as to avoid excessive build up of potential energy that will release when the prestressed element ultimately fails.
1.5 ***Composite fibre overlays*** The use of high strength composite overlays, such as carbon fibre sheets, epoxied to the masonry surface can be used to increase the shear capacity of existing masonry wall panels.	A coated masonry wall can be considered to behave as a composite section, as long as adequate bond is achieved at the coating and masonry wall interface. Load distribution between the masonry and coating system should be determined on the basis of the elastic moduli of each material. Refer to the manufacturers of the fibres for specific design guidance and construction specification guidance.
1.6 ***V-brace frames*** See 4 of Table 13.1	Braced steel frames can be used to enhance the seismic resistance strength of existing masonry buildings. Typically brace frames provide lower levels of stiffness than do shear walls. They should be designed for elastic or limited ductile response design actions to preclude early onset of unacceptable or excessive damage to the original building structure at relatively low levels of seismic loading.

Description	Design comment
1.7 ***Infilling wall openings*** The seismic resistance of existing pierced masonry walls incorporating windows and/or door openings can be enhanced by infilling of such openings to provide single continuous solid wall elements.	Masonry infill can comprise masonry clay brick units, concrete masonry (reinforced or unreinforced) units or cast insitu concrete. Infilled openings can be considered to act compositely with the surrounding masonry structure as long as adequate anchorage or interlocking is provided at the interface of the new infill and existing masonry, to ensure an equivalent shear strength to the existing wall material. Where the infill is of different thickness and/or material to the existing, consideration needs to be given to the different strengths and elastic moduli between new and old in assessment of the lateral loading carried by the composite section.
1.8 ***Plywood faced shear walls*** Where seismic loading demands are relatively low, the seismic lateral resistance of a building can be enhanced by the addition of plywood sheathed shear wall elements. The plywood sheathing is fixed directly to timber studs and connected via top and bottom plates to each level. Connections need to be designed for hold down of chord members and for horizontal shear.	Plywood shear walls provide less strength and are more flexible than equivalent concrete or concrete masonry walls. Appropriate use of plywood shear walls in masonry buildings would be as internal bracing walls in the upper storeys. Where used they should be distributed across a building in a balanced manner to reduce the loading on each wall. Plywood shearwalls should be designed for elastic or limited ductile response design actions in accordance with NZS 3603.
2 **Face load strengthening** 2.1 ***Floor roof and ceiling level ties*** All masonry walls should be firmly anchored at floor and roof levels. Connections between walls and floors can be improved through use of wall ties or anchors. These are commonly fabricated from steel rods and plate, with the rods grouted into or bolted through the brick wall and bolted via the plate to existing floor joists or blocking between joists to develop the required forces.	Out of plane loading is commonly resisted by wall components spanning between floor levels and ceilings or roofs acting as diaphragms. Commonly timber floor joists and roof rafters will be found to be fixed into wall sockets with only a nominal mechanical connection to the wall. Tie connections should be sized for out of plane lateral forces assessed from Section 8 of NZS 1170 Part 5. Note that floor ties may also be used for in plane diaphragm shear transfer and tying requirements for this action need to be separately assessed.
2.2 ***Replacement veneer ties*** Proprietary epoxy resin ties, helical steel ties or expanding metal ties can be used to replace or supplement existing steel ties to cavity brick construction.	Veneers need to be checked to ensure they have ties to the main wythe of the wall, and that they are in sound condition. Existing steel ties can often be found to be corroded within the mortar joints or inadequate to carry the veneer inertia loads. Replacement or supplementary ties should have tensile capacity in excess of the lateral loads developed for the area tributary to the tie.
2.3 ***Rosehead washers*** The tensile capacity of floor/ roof ties can be maximised by the use of rosehead washer bearing plates to the outer wall face. These are appropriate for use in solid masonry walls or when local packing is provided across cavities in cavity wall construction.	Rosehead washer plates are a traditional method of securing masonry walls to support floors or roofs. The design of the plates should be sympathetic to the style of the building wherever possible.

Description	Design comment
2.4 ***Mullion supports and/or girt supports*** Where out of plane failure of a wall is likely under the design level seismic accelerations, vertical mullions or horizontal girt supports can be introduced to reduce the span of masonry wall panels.	Structural steel sections are commonly used as mullion/girt bracing supports. These bracing elements are proportioned to resist a tributary portion of out of plane lateral load. Deflection limits rather than strength will often dictate section sizing. Out of plane deflection of such members should not exceed one tenth of the total wall thickness under the ultimate limit state design loads. Adequate connections between the masonry wall and bracing members need to be provided.
2.5 ***Parapet bracing (or removal of parapets)*** Parapets and exterior wall appendages incapable of sustaining out of plane seismic loading or displacement demands need to be braced back to the building roof structure or alternatively reduced in height or removed altogether.	Parapets are particularly vulnerable to damage, as earthquake accelerations are greatest at the top of a building. Bracing would typically comprise structural steel struts, ties and /or truss elements bolted to the parapet and tied back to the primary roof structure of the building.
2.6 ***Cantilever columns*** Cantilever columns provide additional lateral support to walls, allowing for two way spanning of wall panels in a similar way as buttressing (see 6.2.8). Adequate foundations and connections to the base of the column section need to be provided to ensure cantilever action is obtained.	The flexibility of the column element needs to be considered in assessing the wall spanning action.
2.7 ***Composite fibre flexural strips*** The use of high strength composite overlay strips, such as carbon fibre sheets, epoxied to the masonry surface each side of a wall panel can be used to provide tensile strength and accordingly increase the out of plane flexural capacity of existing masonry wall panels	A coated masonry wall can be considered to behave as a composite section, as long as adequate bond is achieved at the coating and masonry wall interface.. Refer to the manufacturers of the fibres for specific design guidance and construction specification guidance. Check the face load shear capacity of the wall-supports as additional shear connections are often required to match the increased flexural strength of the wall.
2.8 ***Buttressing or propping*** Buttressing or propping of walls provides additional lateral support and allows for two way spanning of wall panels, rather than in one direction, increasing the wall resistance to face loading.	Buttressing can be provided by new crosswalls or infilling or propping to existing crosswalls. Adequate anchorage connections between the wall and the buttressing element need to be provided to transfer wall inertia forces to the buttress or element, particularly when these are directed away from the buttress support line (i.e. in tension).
2.9 ***Helical steel through ties*** Helical steel through-ties can be used to improve bonding and tying between brickwork layers and ensure composite action through the full depth of wall panels.	Refer to the manufacturers of the helical ties for complete design recommendations and criteria.

Description	Design comment
2.10 ***Concrete Overlay Walls*** Insitu or shotcrete overlay walls can be used to enhance the out of plane strength of masonry walls. The flexural capacity and behavour will be assymetrical for loading in opposing directions as the compression zone will alternate between the concrete and masonry.	Masonry walls with concrete overlays can usually be considered as composite provided adequate bond/anchorage is provided at the concrete masonry interface.
3 **Combined face load and in plane strengthening** 3.2 ***Vertical and/or horizontal post tensioning*** The out of plane flexural and shear capacity of wall panels can be enhanced using bonded or unbonded post tensioned tendons, either fixed to the exterior of the piers or within a core through the pier interior.	Post tensioned masonry walls should be considered to behave as unreinforced masonry walls with increased compression load. Allowance needs to be made for loss of prestress force due to creep and shrinkage. Care is required with anchorage zones to spread the anchorage stresses. Low levels of prestress are recommended so as to avoid excessive build up of potential energy that will release when the prestressed element ultimately fails. Anchorages are often eccentricly loaded and cables eccentric to the neutral axis of the wall elements. Care is needed in design and construction to fully allow for the induced moments that result.
3.3 ***Deep drilling and reinforcing of walls*** The out of plane and in plane flexural capacity of unreinforced masonry walls can be enhanced by the installation of reinforcing steel bars grouted or cemented into drilled holes through their core	Reinforced-cored masonry wall panels can be considered to act as composite reinforced masonry walls as long as sufficient bond between the new reinforcement and masonry is achieved. The vertical reinforcement should be well anchored beyond the base of the walls. New vertical reinforcement can be considered to contribute to the shear (sliding joint shear) capacity.
3.4 ***Grouting Rubble Filled Walls*** Many stone masonry walls comprise an outer and inner wythe of stone (or stone on the outside and brick on the inside) with the cavity between the wythes infilled with stone rubble that is very poorly bonded and very porous (no fines). These walls generally have very low in-plane and out-of-plane strength and integrity. The structural/properties of these walls can be significantly improved by carefully infilling the porous rubble with a cement based grout. Relatively low grout strengths are usually all that is needed and grouts that include a lime content and additives to improve their flow are often used. Extreme care is needed to avoid a damaging build up of hydrostatic pressure from the grout. Slow grouting rates are recommended and/or grouting in several low lifts through grout holes drilled through one wythe will limit excessive grout pressures.	

Description	Design comment
3.5 ***Concrete Overlay Walls*** See 1.1 and 2.10 above.	
4 **Diaphragm strengthening** 4.1 ***Plywood Overlay Diaphragms*** Existing timber strip flooring often has limited reliable seismic diaphragm capacity. Plywood sheeting laid over the existing flooring and well nailed to all edges will provide an "engineered" diaphragm. Staggered plywood sheet layouts are usually used and all sheet edges and ends must be trimmed to abut adjacent sheets to allow nailing to a common strip of flooring. Joining sheets by nailing the ply through (say) 400x400x1mm corner plates avoids the need to trim sheets to suit the strip flooring and provides reliable load transfer between sheets.	The ply overlay diaphragms can be designed to NZS 3603 requirements. Generally, the strength of the existing strip floor diaphragm is ignored as its diaphragm action is often destroyed by the installation of services below the floor or is very weak where floor joists abut one another over a steel or timber main support beam (for example) or load bearing wall.
4.2 ***Boundary Connections, Diaphragm Chords, Drag Ties*** Steel strips or steel angles are generally installed around the perimeter of the ply overlay diaphragms to provide a reliable chord element/shear collector element/and a point of attachment of the wall face load ties to the ply overlay. Steel strip chords say 200x1mm can be laid under (or over) the ply adjacent to the masonry walls and well nailed into position. Steel edge angles, drilled for nails to the ply diaphragm and to attach to floor level ties are used for some installations.	Reliable chords, shear collectors and drag ties around openings or to transfer tension/compression loads from diaphragms to adjacent walls are usually required for engineered diaphragms. Often the chord forces are not large, given the properties of the diaphragms, and can be resisted by thin metal strips say 200x1mm. Generally, flat head "product" nails can be used to nail through the ply and steel strip and into the existing flooring to achieve limited ductile connections.
4.3 ***Steel flat overlays*** Additional to their use as diaphragm chords and drag ties (see 4.2), steel straps can be used as tension cross bracing on or under existing timber and concrete floors to enhance shear strength of diaphragms. (see also 5.3 of Table 13.2)	
4.4 ***Concrete topping overlays*** Concrete topping overlays can be cast over existing concrete slab diaphragms to increase their thickness and shear capacity. Tying or bonding of the new overlay to the existing slab is generally required to prevent out of plane buckling and maximise the shear capacity of the composite diaphragm.	Concrete slab diaphragm thickness may be increased using a topping overlay but the added weight will increase the seismic load as well as increase footing loadings. Concrete diaphragms can be designed to NZS 3101 requirements.

Description	Design comment
4.5 ***Roof and ceiling diaphragms*** The shear capacity of existing timber sheathing (sarked) diaphragms to rooves and ceilings can be enhanced by the addition of nails through each sheathing board into their underlying supports. Increased strength can be provided by the addtion of a new plywood or plaster sheet panel diaphragm over an exisiting diaphragm (or replacing the existing diaphragm). Where the new panel diaphragm is placed over an existing sheathed diaphragm, the joints of the new panel diaphragm should be placed so they are near the centre of the sheathing boards or at 45-degrees to the joints between sheathing boards.	Removal of existing roofing or ceiling lining will be required to install additional nailing or to place and nail the new overlay panel diaphragm. New panel overlay diaphragms can be designed to NZS 3603 requirements or in accordance with the requirements of the manufacturer of the proprietary sheet material.
5 **Chimney, towers and appendages** 5.1 ***Attaching Chimneys and Towers to Diaphragms and/or Walls*** Providing adequate fulcrum supports for cantilever chimneys and towers is often very difficult to achieve. Generally, very significant strengthening is required at roof and/or ceiling level to cope with the large seismic reaction loads. Struts, ties and strengthened diaphragms at these levels often are required to extend over some distance from the chimney or tower to dissipate the fulcrum reaction. Raising the level of support above the roof level can lower the seismic reactions significantly but can expose the strut/ties to view. Often a steel girdle is required around the chimney or tower to provide adequate support and anchorage for the fulcrum connection.	Seismic loads derived from Section 8 of NZS 1170 Part 5 on Parts and Components are high to account for the relative height of chimney and towers together with the seismic whip-lash effects on these vertical cantilevers. Introducing some limited ductility into the supporting structure is usually necessary so that the seismic loads being transferred back into the supporting walls, diaphragms and connections become more manageable and practical.
5.2 ***Wire Tying Appendages to Arrest Falling*** Some building components such as pediments, finials, gargoyles, crosses and other roof ornamentation are impractical to strengthen. Instead, a valid strategy is to lasso them to the building with wire ties so that when they are dislodged in an earthquake their fall is arrested.	Arresting the fall of a falling object will generate several g's of deceleration (kinematic equations). The wires must be well secured to the objects and well tied back to the building. The wire ties can include a spring element or a ductile clip to reduce the deacceleration force. Ensure that the portion of the building that the wires are anchoring on to do not become dislodged by the high restraint forces involved.

References

Section 1

AS/NZS 1170.0:2002, Structural Design Actions, Part 0: General requirements – Aust/New Zealand, Standards New Zealand.

NZBC 1992, New Zealand Building Code

NZSEE 1985, "Earthquake Risk Buildings – Recommendations and Guidelines for Classifying, Interim Securing and Strengthening", December

NZSEE 1996, "The Assessment and Improvement of the Structural Performance of Earthquake Risk Buildings". Draft for General Release, June, 122pp. (Prepared for Building Industry Authority)

NZSEE 2000, "An Initial Evaluation Process for Identifying Buildings Not Safe in Earthquake", August, 27pp. (Prepared for Building Industry Authority)

SNZ 1976, "General Structural Design and Design Loadings for Buildings", New Zealand Standard NZS 4203:1976, Standards New Zealand

NZS 1170.5:2004, Structural Design Actions, Part 5: Earthquake actions – New Zealand, Standards New Zealand.

Section 3

NZS 1170.5:2004, Structural Design Actions, Part 5: Earthquake actions – New Zealand, Standards New Zealand.

SNZ 2004,

Section 4

FEMA 356 (2000). Prestandard and Commentary for the Seismic Rehabiliation of Buildings, Federal Emergency Management Agency, Washington, DC, USA

Bussell M. 1997. *Appraisal of Existing Iron and Steel Structures*. The Steel Construction Institute, Ascot, England, SCI Publication 138.

Ferris HW. *Rolled Shapes – Beams and Columns – Period 1873 to 1952*. American Institute of Steel Construction, New York, USA.

Section 5

ATC 40 Seismic evaluation and retrofit of concrete buildings, Vol 1 & 2 Applied Technology Council, Nov 1996.

FEMA 440 Improvement of nonlinear static seismic analysis procedures, Federal Emergency Management Agency, Preprint Edition, 2005

Pekcan G Mander J B Chen SS Fundamental considerations for the design of non-linear viscous dampers Earthquake Engineering Structural Dynamics 28, 1405-1425, 1999

Section 6

ATC. 1995. Guidelines for the Seismic Rehabilitation of Buildings (75% complete draft). Applied Technology Council Project ATC 33.03, California.

Eurocode 8. 1994. Design Provisions for Earthquake Resistance of Structures (EC8).

Park R. 1996. A static force-based procedure for the seismic assessment of existing reinforced concrete moment resisting frames. Proceedings of NZNSEE Annual Conference. New Plymouth.

Priestley MJN, Calvi GM. 1991. Towards a capacity-driven assessment procedure for reinforced concrete frames. Earthquake Spectra 7(3): 413–37.

Priestley MJN. 1995. Displacement-based seismic assessment of existing reinforced concrete buildings. Proceedings of Pacific Conference on Earthquake Engineering 2: 225–44. Melbourne.

SNZ. 1995. Concrete Structures Standard NZS 3101:1995, Volume 1 Code of Practice and Volume 2 Commentary. Standards New Zealand, Wellington.

Section 7

Carr AJ. 2005. *RUAUMOKO Users Guide*. Department of Civil Engineering, University of Canterbury.

Chapman HE. 1991. Seismic retrofitting of highway bridges. *Bulletin of New Zealand National Society for Earthquake Engineering* 24(2): 186–201.

Cheung PC, Paulay T, Park R, 1991. *Mechanisms of slab contributions in beam - column sub assemblages. Design of Beam - Column Joints for Seismic Resistance.* Special Publication SP-123 American Concrete Institute; 259-289.

Hakuto S, Park R, Tanaka H. 1995. *Retrofitting of Reinforced Concrete Moment Resisting Frames*. Research Report 95–4, Department of Civil Engineering, University of Canterbury.

Hakuto S, Park R, Tanaka H. 1999. Effect of deterioration of bond of beam bars passing through interior beam-column joints on flexural strength and ductility. *Structural Journal of American Concrete Institute* 96(5): 858–64, September–October.

Hakuto S, Park R, Tanaka H. 2000. Seismic load tests on interior and exterior beam-column joints with substandard reinforcing details. *Structural Journal of American Concrete Institute* 97(1): 11–25, January–February.

Liu A, Park R. 1998. Seismic load tests on two interior beam-column joints reinforced by plain round bars designed to pre-1970s seismic codes. *Bulletin of the New Zealand Society for Earthquake Engineering* 31(3): 164–76.

Liu A, Park R. 2001. Seismic behaviour and retrofit of pre-1970s as-built exterior beam-column joints reinforced by plain round bars. *Bulletin of the New Zealand Society for Earthquake Engineering* 34(1): 68–81.

Mander JB, Priestley MJN, Park R. 1988. Observed stress-strain behaviour of confined concrete. *Journal of Structural Engineering, American Society of Civil Engineers* 114(8): 1827–49.

Mander JB, Priestley MJN, Park R. 1988. Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering, American Society of Civil Engineers* 114(8): 1804–26.

Park R, Paulay T. 1975. *Reinforced Concrete Structures*. John Wiley and Sons, New York.

Park R, et al. 1995. The Hyogo Ken Nanbu Earthquake (The Great Hanshin Earthquake) of 17 January 1995. Report of the NZNSEE Reconnaissance Team. *Bulletin of the New Zealand National Society for Earthquake Engineering* 28(1): 1–98.

Park R. 1992., *Seismic Assessment and Retrofit of Concrete Structures: United States and New Zealand Developments*. Proceedings of Technical Conference of New Zealand Concrete Society, Wairakei, New Zealand, pp 18–25.

Park R. 1996. A static force-based procedure for the seismic assessment of existing reinforced concrete moment resisting frames. *Bulletin of the New Zealand National Society for Earthquake Engineering 30(3):213-226.*.

Paulay T, Priestley MJN. 1992. *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley and Sons, New York.

Paulay T. 1993. *Simplicity and Confidence in Seismic Design*. John Wiley and Sons, New York.

Paulay, T, Restrepo J I, 1998. Displacement and ductility compatibility in buildings with mixed structural systems. *Journal of the Structural Engineering Society, New Zealand*. 11(1) :7-12.

Paulay, T. 2000. Principles of displacement compatibility. *Journal of the Structural Engineering Society, New Zealand*. 13(2):14-21.

Paulay, T. 2001a. A re-definition of the stiffness of reinforced concrete elements and its implications in seismic design. *Structural Engineering International* 11(1): 36-41.

Paulay, T. 2001b. Seismic response of structural walls: Recent developments. *Canadian Journal of Civil Engineering*. 28: 922-937.

Paulay, T. 2002. An estimation of displacement limits for ductile systems. *Earthquake Engineering and Structural Dynamics* 31:583-599.

Presland RA. 1999. Seismic performance of retrofitted reinforced concrete bridge piers, PhD Thesis, Department of Civil Engineering, University of Canterbury

Priestley MJN, Kowalsky MJ. 2000. Direct displacement-based seismic design of concrete buildings. *Bulletin of the New Zealand Society for Earthquake Engineering* 33(4): 421–443, December.

Priestley MJN, Park R. 1987. Strength and ductility of concrete bridge columns under seismic loading. *Structural Journal of American Concrete Institute* 84(1): 61–76.

Priestley MJN 1988. Brief Comments on the Elastic Flexibility of Reinforced Concrete Frames and Significance in Seismic Design. *Bulletin of the New Zealand Society for Earthquake Engineering. 31 (4):2 246-259*

Priestley MJN, Seible F, Calvi GM. 1996. *Seismic Design and Retrofit of Bridges*. John Wiley and Sons, New York.

Priestley MJN, Verma R, Xiao Y. 1994. Seismic shear strength of reinforced concrete columns. *Journal of Structural Engineering, American Society of Civil Engineers* 120(8): 2310–29.

Priestley MJN. 1995. Displacement-based seismic assessment of existing reinforced concrete buildings. *Proceedings of Pacific Conference on Earthquake Engineering* 2: 225–44, Melbourne.

Priestley, MJN. 1988. Brief comments on elastic flexibility of reinforced concrete frames and significance in seismic design. *Bulletin of the New Zealand Society for Earthquake Engineering*. 31(4): 246-259.

Rodriguez M, Park R. 1991. Repair and strengthening of reinforced concrete buildings for earthquake resistance. *Earthquake Spectra* 7(3): 439–59.

SANZ. 1962. *Deformed Steel Bars of Structural Grade for Reinforced Concrete NZS 1693:1962*. Standards Association of New Zealand, Wellington.

SANZ. 1964. *Hot Rolled Steel Bars of HY60 Grade (60,000 psi) for Reinforced Concrete NZS 1879:1964*. Standards Association of New Zealand, Wellington.

SNZ. 1973. *Hot Rolled Steel Bars for the Reinforcement of Concrete NZS 3402P:1973*. Standards Association of New Zealand, Wellington.

SNZ. 1992. *Code of Practice for General Structural Design and Design Loadings for Buildings NZS 1170.5:2004 (OR NZS 1170.5 (OR NZS 4203):1992), Volume 1 Code of Practice* and *Volume 2 Commentary*. Standards New Zealand, Wellington.

SNZ. 1995. *Concrete Structures Standard NZS 3101:1995, Volume 1 Code of Practice and Volume 2 Commentary*. Standards New Zealand, Wellington.

Scott BD, Park R, Priestley MJN. 1982. Stress-strain behaviour of concrete confined by overlapping hoops at low and high strain rates. *Journal of the American Concrete Institute Proceedings* 79(1): 13–27.

Shibata A, Sozen M. 1976. Substitute structure method for seismic design in reinforced concrete. *Journal of Structural Engineering, American Society of Civil Engineers.* 102(1).

SNZ. 1990. *Code of Practice for the Design of Masonry Structures NZS 4230:1990*. Standards New Zealand, Wellington.

Wallace JL. 1996. *Behaviour of Beam Lap Splices Under Seismic Loading*. Master of Engineering Thesis, Department of Civil Engineering, University of Canterbury.

Section 8

AS 1817:1991. *Metallic Materials – Vickers Hardness Test*. Standards Australia, Sydney, Australia.

ASM International. 1976. *Guide to Materials Engineering Data and Information*. ASM International, Ohio, USA.

Astaneh-Asl A. 1995. *Post-Earthquake Stability of Steel Moment Frames with Damaged Connections*. Third International Workshop on Connections in Steel Structures, Italy.

Barker GF. 2000. Assessment of existing structures: a strength limit state appraisal of hot driven rivets. *Steel Construction* 34(1): 1–15, March.

Blodgett OW. 1987. Weld failures: they could be the result of violating simple design principles. *Australia Welding Journal* 32(2).

Brownlee SA. 1994. *Axial Force and Plate Slenderness Effects on the Inelastic Behaviour of Structural Steel Beam-Columns*. University of Canterbury, Christchurch, New Zealand.

Bussell M. 1997. *Appraisal of Existing Iron and Steel Structures*. The Steel Construction Institute, Ascot, England, SCI Publication 138.

Clifton GC (ed). *HERA Steel Design and Construction Bulletin*. Bimonthly Technical Publication from HERA, Manukau City. Issue numbers as referenced in the text.

Clifton GC. 1994. *Steelwork Limit State Design Guides*. Vol 1. HERA, Manukau City, HERA Report R4–80.

Clifton GC. 1996a. Tentative guidelines for assessing the seismic performance of pre-1975 moment-resisting steel framed buildings. *HERA Structural Design and Construction Bulletin* 18: 1–10.

Clifton GC. 1996b. *Steel Building Performance in Two Recent Major Earthquakes*. IIW Asian Pacific Welding Congress, Auckland, New Zealand.

Clifton GC. 1997. *Restraint Classifications for Beam Member Moment Capacity Determination to NZS 3404:1997*. HERA, Manukau City, HERA Report R4–92.

Clifton GC. 2000. *Tips on Seismic Design of Steel Structures*. HERA, Manukau City.

Feeney MJ, Clifton GC. 1995. *Seismic Design Procedures for Steel Structures*. HERA, Manukau City, HERA Report R4–76.

Ferris HW. *Rolled Shapes – Beams and Columns – Period 1873 to 1952*. American Institute of Steel Construction, New York, USA.

Hayward P, McClintock A. 1999. *What Every Engineer Should know about Welding and Inspection*. CBIP/NZWC, HERA, Manukau City.

Hyland C. 1999. *Structural Steelwork Connections Guide*. HERA, Manukau City, HERA Report R4–100.

MacRae GA. 1990. *The Seismic Response of Steel Frames*. University of Canterbury Research Report 90–6. University of Canterbury, Christchurch, New Zealand.

NZS 1170.5:2004. Structural Design Actions, Part 5 : Earthquake Actions-New Zealand, Standards New Zealand.

NZS 3101:1995 (incorporating Amendments 1 and 2, 1997). *Concrete Structures Standard*. Standards New Zealand.

NZS 3404:1997 plus Amendment No. 1: 2001. *Steel Structures Standard*. Standards New Zealand, Wellington.

NZS 4203: 1992, *Loadings Standard*; Standards New Zealand.

Roeder C, et.al. 1994. *Strength, Stiffness and Ductility of Older Steel Structures under Seismic Loading*. University of Washington Department of Civil Engineering, Washington, USA, Report No. SGEM 94–4.

Roeder C, et.al. 1996. Seismic Behaviour of Older Steel Structures. *Journal of Structural Engineering,* Vol 122, No 4, April 1996, pp 365 - 373

Wood PJ. 1987. *Cyclic Testing of a Concrete Encased Riveted Beam Column Joint*. Ministry of Works and Development, Wellington, New Zealand, Report Number 5–82/2.

Section 9

Angel R, Abrams DP. 1994. Out-of-plane strength evaluation of URM infill panels. *Proceedings of the NCEER Workshop on Seismic Response of Masonry Infills*. DP Abrams (ed), NCEER Technical Report NCEER–94–0004.

Dawe JL, Seah CK. 1988. Lateral load resistance of masonry panels in flexible steel frames. *Proceedings of the Eighth International Brick and Block Masonry Conference*, Trinity College, Dublin, Ireland.

Mainstone RJ, Weeks GA. 1970. The influence of bounding frame on the racking stiffness and strength of brick walls. *2nd International Brick Masonry Conference*.

Mainstone RJ. 1971. On the stiffness and strength of infilled frames. *Proceedings of the Institution of Civil Engineers Sup.*, pp 57–90.

Paulay T, Priestley MJN. 1992. *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley & Sons, NY.

Saneinejad A, Hobbs B. 1995. Inelastic design of infilled frames. *Journal of Structural Engineering*, ASCE, 121(4): 634–50.

Stafford-Smith B, Carter C. 1969. A method of analysis for infilled frames. *Proceedings of the Institution of Civil Engineers* 44: 31–48.

Section 10

Abrams D. 1994 Seismic response of unreinforced masonry buildings: research accomplishments. *Multidisciplinary Center for Earthquake Engineering Research*. National Centre for Earthquake Engineering Research. Available at http://mceer.buffalo.edu/publications/.

Benedetti D, Petrini V. 1996. *Shaking Table Tests on Masonry Buildings. Results and Comments*. ISMES, Bergamo.

EL Blaikie,EL & Spurr,DD. 1993. *Earthquake Vulnerability of Existing Unreinforced Masonry Buildings*. EQC funded research by Works Consultancy Services, under Project 91/87, February.

EL Blaikie,EL. 2001. *Methodology for the Assessment of Face-Loaded Unreinforced Masonry Walls under Seismic Load*. EQC funded research by Opus International Consultants, under Project 99/422.

EL Blaikie,EL. 2002. *Methodology for Assessing the Seismic Performance of Unreinforced Masonry Single Storey Walls, Parapets and Free Standing Walls.* Report prepared for the EQC Research Foundation.

Federal Emergency Management Agency (FEMA), 1998. Publications (prepared by Applied Technology Council (ATC) for the Partnership for Response and Recovery). FEMA 306, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings – Basic Procedures Manual* (ATC-43 Project). FEMA 307, *Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings – Technical Resources*; FEMA 308, *Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*.

Foss M, 2001. *Diagonal Tension in Unreinforced Masonry Assemblages*. MAEC ST-11: Large Scale Test of Low Rise Building System. Georgia Institute of Technology.

Gamborrotta L, Lagomarsino S. 1997. Damage models for the seismic response of brick masonry shear walls. Part I: The mortar joint model and its applications; Part II: The continuum model and its applications. *Earthquake Engineering and Structural Dynamics* 26: 423–62.

Hansen KF, 1999. *Bending and Shear Tests with Masonry.* Danish Building Research Institute, SBI Bulletin 123.

Kitching N, 1999. *The Small Scale Modelling of Masonry*. Masonry Research, Civil Engineering Division, Cardiff School of Engineering.

Magenes G, Calvi GM. 1997. In-plane seismic response of brick masonry shear walls. *Earthquake Engineering and Structural Dynamics* 26: 1091–112.

Magenes G, dell a Fontana A. 1998. Simplified non-linear seismic analysis of masonry buildings. *Proceedings of the Fifth International Masonry Conference*. British Masonry Society, London.

Magenes G. 1992. *Comportamento Sismico di Murature di Mattoni: Resistenza e Mecahnismi di Rottura di Maschi Murari*. Departimento di Meccanica Strutturale, Universita di Pavia, Februaio.

Mann W, Müller H. 1982. Failures of shear-stressed masonry: an enlarged theory, tests and application to shear walls. *Proceedings of the British Ceramic Society* No. 30, September.

Various other research studies include those of Yokel (1971), Fattal (1976), Hendry (1973, 1981), Haseltine (1977), West (1977), Sinha (1978), ABK Consultants (1981), Kariotis (1986), Drysdale (1988), Lam (1995), Mendola (1995). A summary of these papers is presented in a preliminary ABAQUS study of historic masonry structures in Italy, available at http://urban.arch.virginia.edu/struct/pompeii/masonry/masonry-2.html.

Section 11

SNZ 1993. *Timber Design Standard NZS 3603*. Standards New Zealand, Wellington.

Beattie, G.J. 1999. Earthquake load sharing between timber framed and masonry walls. Pp 368-375 *in* Walford G.B. and Gaunt D.J.(Ed) Proceedings of the Pacific Timber Engineering Conference, Vol. 3, March 1999, Rotorua, NZ. *New Zealand Forst Research Institute, Forest Research Bulletin No. 212.*

Appendix 4D

Anagnostopoulas SA, Spiliopoulos KV. 1992. An investigation of earthquake induced pounding between adjacent buildings. *Earthq Eng Struct Dyn* 21: 289–302.

Carr AJ, Moss PJ. 1994. Impact between buildings during earthquakes. *Bulletin of the NZNSEE* 27(2) June.

Conoscente JP, Hamburger RO. 1992. *Dynamic Analysis of Impacting Structural Systems*. Proceedings of the 10th World Conference on Earthquake Engineering.

Kasai K, Jeng V, Patel PC, et al. 1992. *Seismic Pounding Effects – Survey and Analysis*. Proceedings of the 10th World Conference on Earthquake Engineering.

Kasai K, Maison BF, Patel DJ. 1990. *An Earthquake Analysis for Buildings Subjected to a Type of Pounding*. Proceedings of the Fourth US National Conference on Earthquake Engineering, EERI, Oakland, CA.

FEMA 273, 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, 1997, Washington, D.C. (FEMA Publication No. 273).

Appendix 4E

Bracci, J. M., Kunnath, S. K., and Reinhorn, A. M., 1995, "Simplified Seismic Performance and Retrofit Evaluation," submitted to the *Journal of the Structural Division*, American Society of Civil Engineers, New York, New York.

BSSC, 2000, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, 2000 Edition, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, D.C.

Eberhard, M. O., and Sozen, M. A., 1993, "Behavior-Based Method to Determine Design Shear in Earthquake-Resistant Walls," *Journal of the Structural Division*, American Society of Civil Engineers, New York, New York, Vol. 119, No. 2, pp. 619–640.

Fajfar, P., and Fischinger, M., 1988, "N2—A Method for Non-Linear Seismic Analysis of Regular Structures," *Proceedings of the Ninth World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan.

NZS 1170.5:2004, Structural Design Actions, Part 5: Earthquake actions – New Zealand, Standards New Zealand.

FEMA 273, 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, 1997, Washington, D.C. (FEMA Publication No. 273).

FEMA 274, 1997, *NEHRP Commentary on the Guidelines for Seismic Rehabilitation of Buildings,* prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, D.C. (FEMA Publication No. 274).

FEMA 310, 1998, *Handbook for the Seismic Evaluation of Buildings—A Prestandard,* prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C. (FEMA Publication No. 310).

FEMA 356, 2000, Prestandard and Commentary for the Seismic Rehabiliation of Buildings, Federal Emergency Management Agency, Washington, DC, USA

Appendices

Appendix 2A: Priority Factors

2A.1 Occupancy Classification

The occupancy classification (OC) should be determined by considering both the occupant load (O_L) and the intensity of occupation (O_I).

O_L = The maximum number of people exposed to risk during the normal functioning of the building.

$$O_I = \frac{\text{Occupant Load}}{\text{Gross Floor Area (100s of m}^2\text{)}} \times \frac{\text{Weekly hours of normal occupancy}}{40}$$

The occupancy classification is determined as follows:

- For essential buildings: $OC = 1$
- For all other buildings: OC is determined from Figure 2A.1.

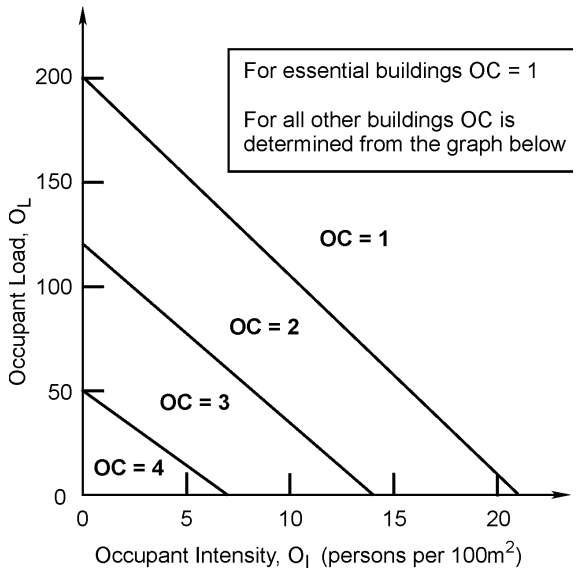


Figure 2A.1: Occupancy Classifications (non-essential buildings)

2A. 2 Risk to People Outside the Building

The risk to people outside the building is a function of building location, accessibility and use. The intention of this factor is to recognise that larger numbers of people, other than the occupants, may be at risk in the event that parts of a building may collapse during an earthquake. Examples are:

- high risk: inner city retail shopping areas adjacent to busy footpath, exitways, malls and public places
- medium risk: inner or outer city commercial business areas with street frontage
- low risk: outer city/suburb industrial warehouse areas not frequented by pedestrians.

2A.3 Prioritising for Detailed Evaluation

- The following relationship may be used to assist with prioritising buildings that have undergone the IEP procedure.
- The procedure should not be used for comparison of buildings in different earthquake zones, and is intended for use with buildings identified as potentially not safe in an earthquake.

$$PS = \frac{\%NBS}{(K1 \times K2)}$$

where:

PS = Prioritised Structural Performance Score
$\%NBS$ = Percentage of New Building Standard from the IEP analysis
$K1$, $K2$ = Factors from Table 2A.1

Table 2A.1: Modification factors *K*1 and *K*2

Description	Classification	Factor
Occupancy Classification (refer Figure 2.5)	1	*K*1 = 1.2
	2	1.0
	3	0.9
	4	0.8
Risk to people outside (refer commentary below)	High	*K*2 = 1.1
	Medium	1.0
	Low	0.9

2A.4 Timetable for Improvement

Time to complete performance improvement (T_c) to be:

$$T_c = \frac{\%NBS}{5 \times K1 \times K2}$$

where:

$1.0 < T_c < 20$ (years)
$\%NBS$ = Percentage of New Building Standard
$K1$, $K2$ = As above

Note:

- The *%NBS* is the earthquake performance of the building compared with requirements for a new building, expressed as a percentage. If a detailed evaluation of the building is available, this should be used to determine the *%NBS*. Otherwise, at the territorial authority's discretion, the IEP score may be used.
- For a change of use application, the work is to proceed immediately as part of the consent.

Appendix 2B: Factors to be considered when evaluating “as near as is reasonably practicable to that of a new building”

The following factors should be considered by TAs and designers/assessors when evaluating *“as near as is reasonably practicable to that of a new building”*;

a) The size of the building;
b) The complexity of the building;
c) The location of the building in relation to other building, public spaces, and natural hazards;
d) The intended life of the building;
e) How often people visit the building;
f) How many people spend time in or in the vicinity of the building;
g) The intended use of the buildings, including any special traditional and cultural aspects of the intended use;
h) The expected useful life of the building and any prolongation of that life;
i) The reasonable practicality of any work concerned;
j) In the case of an existing building, any special historical or cultural value of that building;
k) Any other matter that the territorial authority considers to be relevant.”

Appendix 3A: Typical (%*NBS*)$_b$ values for Wellington, Auckland and Christchurch

Table 3A.1: (%*NBS*)$_b$ Wellington, μ = 1.25, Importance Levels 2, 3 and 4

WELLINGTON

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
WGTN 1931-1935	A or B Rock	17	38	62
	C Shallow Soil	14	30	49
	D Soft Soil	11	19	30
	E Very Soft Soil	12	12	19
1935-1965	A or B Rock	14	30	49
	C Shallow Soil	11	24	39
	D Soft Soil	9	15	24
	E Very Soft Soil	10	10	16
1965-1976	A or B Rock	29	40	52
	C Shallow Soil	23	32	41
	D Soft Soil	18	20	25
	E Very Soft Soil	20	13	16
1976-1992	A or B Rock	86	108	138
	C Shallow Soil	69	86	110
	D Soft Soil	54	59	74
	E Very Soft Soil	54	38	48
1976-1984 Reinforced Concrete	A or B Rock	103	129	165
	C Shallow Soil	82	104	132
	D Soft Soil	65	70	89
	E Very Soft Soil	65	45	58
1992-2004	A or B Rock	78	75	63
	C Shallow Soil	73	91	73
	D Soft Soil	72	84	69
	E Very Soft Soil	72	54	44

WELLINGTON

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
WGTN 1931-1935	A or B Rock	14	30	49
	C Shallow Soil	11	24	39
	D Soft Soil	9	15	24
	E Very Soft Soil	10	10	16
1935-1965	A or B Rock	14	30	49
	C Shallow Soil	11	24	39
	D Soft Soil	9	15	24
	E Very Soft Soil	10	10	16
1965-1976	A or B Rock	30	41	53
	C Shallow Soil	24	33	42
	D Soft Soil	19	20	26
	E Very Soft Soil	21	13	17
1976-1992	A or B Rock	94	119	152
	C Shallow Soil	75	95	121
	D Soft Soil	59	64	82
	E Very Soft Soil	59	42	53
1976-1984 Reinforced Concrete	A or B Rock	113	142	182
	C Shallow Soil	90	114	145
	D Soft Soil	71	77	98
	E Very Soft Soil	71	50	63
1992-2004	A or B Rock	70	68	56
	C Shallow Soil	66	82	66
	D Soft Soil	65	76	62
	E Very Soft Soil	65	49	40

WELLINGTON

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
WGTN 1931-1935	A or B Rock	10	21	34
	C Shallow Soil	8	17	27
	D Soft Soil	6	10	17
	E Very Soft Soil	7	7	11
1935-1965	A or B Rock	10	21	34
	C Shallow Soil	8	17	27
	D Soft Soil	6	10	17
	E Very Soft Soil	7	7	11
1965-1976	A or B Rock	22	30	38
	C Shallow Soil	17	24	30
	D Soft Soil	14	15	19
	E Very Soft Soil	15	9	12
1976-1992	A or B Rock	86	108	138
	C Shallow Soil	69	86	110
	D Soft Soil	54	59	74
	E Very Soft Soil	54	38	48
1976-1984 Reinforced Concrete	A or B Rock	103	129	165
	C Shallow Soil	82	104	132
	D Soft Soil	65	70	89
	E Very Soft Soil	65	45	58
1992-2004	A or B Rock	55	53	44
	C Shallow Soil	51	64	51
	D Soft Soil	51	59	48
	E Very Soft Soil	51	38	31

Table 3A.2: $(\%NBS)_b$ Wellington, μ = 2, Importance Levels 2, 3 and 4

WELLINGTON

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	64	104
	C Shallow Soil	20	51	83
	D Soft Soil	16	32	51
	E Very Soft Soil	17	20	33
1965-1976	A or B Rock	53	85	55
	C Shallow Soil	43	68	44
	D Soft Soil	34	42	27
	E Very Soft Soil	36	27	17
1976-1992	A or B Rock	113	143	182
	C Shallow Soil	91	114	145
	D Soft Soil	71	78	98
	E Very Soft Soil	71	50	63
1976-1984 Reinforced Concrete	A or B Rock	136	171	219
	C Shallow Soil	109	137	174
	D Soft Soil	86	93	118
	E Very Soft Soil	86	60	76
1992-2004	A or B Rock	103	100	83
	C Shallow Soil	97	121	97
	D Soft Soil	96	111	91
	E Very Soft Soil	96	72	59

WELLINGTON

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	64	104
	C Shallow Soil	20	51	83
	D Soft Soil	16	32	51
	E Very Soft Soil	17	20	33
1965-1976	A or B Rock	54	87	56
	C Shallow Soil	44	70	44
	D Soft Soil	34	43	27
	E Very Soft Soil	37	28	18
1976-1992	A or B Rock	125	157	201
	C Shallow Soil	100	126	160
	D Soft Soil	79	85	108
	E Very Soft Soil	79	55	70
1976-1984 Reinforced Concrete	A or B Rock	150	188	241
	C Shallow Soil	120	151	191
	D Soft Soil	94	102	130
	E Very Soft Soil	94	66	84
1992-2004	A or B Rock	93	90	75
	C Shallow Soil	87	109	87
	D Soft Soil	86	100	82
	E Very Soft Soil	86	65	53

WELLINGTON

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	18	45	73
	C Shallow Soil	14	36	58
	D Soft Soil	11	22	36
	E Very Soft Soil	12	14	23
1965-1976	A or B Rock	39	63	40
	C Shallow Soil	32	51	32
	D Soft Soil	25	31	20
	E Very Soft Soil	27	20	13
1976-1992	A or B Rock	113	143	182
	C Shallow Soil	91	114	145
	D Soft Soil	71	78	98
	E Very Soft Soil	71	50	63
1976-1984 Reinforced Concrete	A or B Rock	136	171	219
	C Shallow Soil	109	137	174
	D Soft Soil	86	93	118
	E Very Soft Soil	86	60	76
1992-2004	A or B Rock	72	70	58
	C Shallow Soil	68	85	68
	D Soft Soil	67	78	64
	E Very Soft Soil	67	50	41

Table 3A.3: $(\%NBS)_b$ Wellington, μ = 3, Importance Levels 2, 3 and 4

WELLINGTON

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	113	171	219
	C Shallow Soil	91	137	174
	D Soft Soil	71	93	118
	E Very Soft Soil	71	60	76
1976-1984 Reinforced Concrete	A or B Rock	136	205	263
	C Shallow Soil	109	164	209
	D Soft Soil	86	112	142
	E Very Soft Soil	86	72	91
1992-2004	A or B Rock	103	100	83
	C Shallow Soil	97	121	97
	D Soft Soil	96	111	91
	E Very Soft Soil	96	72	59

WELLINGTON

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	125	188	241
	C Shallow Soil	100	151	191
	D Soft Soil	79	102	130
	E Very Soft Soil	79	66	84
1976-1984 Reinforced Concrete	A or B Rock	150	226	289
	C Shallow Soil	120	181	230
	D Soft Soil	94	123	156
	E Very Soft Soil	94	79	100
1992-2004	A or B Rock	93	90	75
	C Shallow Soil	87	109	87
	D Soft Soil	86	100	82
	E Very Soft Soil	86	65	53

WELLINGTON

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	113	171	219
	C Shallow Soil	91	137	174
	D Soft Soil	71	93	118
	E Very Soft Soil	71	60	76
1976-1984 Reinforced Concrete	A or B Rock	136	205	263
	C Shallow Soil	109	164	209
	D Soft Soil	86	112	142
	E Very Soft Soil	86	72	91
1992-2004	A or B Rock	72	70	58
	C Shallow Soil	68	85	68
	D Soft Soil	67	78	64
	E Very Soft Soil	67	50	41

Table 3A.4: (*%NBS*)$_b$ Auckland, μ = 1.25, Importance Levels 2, 3 and 4

AUCKLAND

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	43	94	151
	C Shallow Soil	34	75	120
	D Soft Soil	27	46	74
	E Very Soft Soil	30	30	48
1965-1976	A or B Rock	60	83	106
	C Shallow Soil	48	66	84
	D Soft Soil	38	41	52
	E Very Soft Soil	42	26	33
1976-1992	A or B Rock	133	167	213
	C Shallow Soil	106	134	170
	D Soft Soil	84	91	115
	E Very Soft Soil	84	59	74
1976-1984 Reinforced Concrete	A or B Rock	159	200	256
	C Shallow Soil	127	160	204
	D Soft Soil	100	109	138
	E Very Soft Soil	100	70	89
1992-2004	A or B Rock	120	116	97
	C Shallow Soil	113	141	113
	D Soft Soil	111	130	106
	E Very Soft Soil	111	84	68

AUCKLAND

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	43	94	151
	C Shallow Soil	34	75	120
	D Soft Soil	27	46	74
	E Very Soft Soil	30	30	48
1965-1976	A or B Rock	61	84	108
	C Shallow Soil	49	67	86
	D Soft Soil	39	41	53
	E Very Soft Soil	42	27	34
1976-1992	A or B Rock	146	184	235
	C Shallow Soil	117	147	187
	D Soft Soil	92	100	127
	E Very Soft Soil	92	64	82
1976-1984 Reinforced Concrete	A or B Rock	175	220	282
	C Shallow Soil	140	176	224
	D Soft Soil	110	120	152
	E Very Soft Soil	110	77	98
1992-2004	A or B Rock	108	104	87
	C Shallow Soil	102	127	102
	D Soft Soil	100	117	95
	E Very Soft Soil	100	75	62

AUCKLAND

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	30	65	106
	C Shallow Soil	24	52	84
	D Soft Soil	19	32	52
	E Very Soft Soil	21	21	34
1965-1976	A or B Rock	44	61	78
	C Shallow Soil	36	49	62
	D Soft Soil	28	30	38
	E Very Soft Soil	31	19	25
1976-1992	A or B Rock	133	167	213
	C Shallow Soil	106	134	170
	D Soft Soil	84	91	115
	E Very Soft Soil	84	59	74
1976-1984 Reinforced Concrete	A or B Rock	159	200	256
	C Shallow Soil	127	160	204
	D Soft Soil	100	109	138
	E Very Soft Soil	100	70	89
1992-2004	A or B Rock	84	81	68
	C Shallow Soil	79	98	79
	D Soft Soil	78	91	74
	E Very Soft Soil	78	59	48

Table 3A.5: (*%NBS*)$_b$ Auckland, μ = 2, Importance Levels 2, 3 and 4

AUCKLAND

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	78	198	320
	C Shallow Soil	63	158	255
	D Soft Soil	49	97	157
	E Very Soft Soil	53	63	101
1965-1976	A or B Rock	109	175	112
	C Shallow Soil	88	140	89
	D Soft Soil	69	86	55
	E Very Soft Soil	75	56	35
1976-1992	A or B Rock	175	221	282
	C Shallow Soil	140	177	224
	D Soft Soil	111	120	152
	E Very Soft Soil	111	77	98
1976-1984 Reinforced Concrete	A or B Rock	211	265	339
	C Shallow Soil	168	212	269
	D Soft Soil	133	144	183
	E Very Soft Soil	133	93	118
1992-2004	A or B Rock	159	153	128
	C Shallow Soil	150	186	150
	D Soft Soil	147	172	140
	E Very Soft Soil	147	111	90

AUCKLAND

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	78	198	320
	C Shallow Soil	63	158	255
	D Soft Soil	49	97	157
	E Very Soft Soil	53	63	101
1965-1976	A or B Rock	112	179	114
	C Shallow Soil	89	143	91
	D Soft Soil	70	88	56
	E Very Soft Soil	76	57	36
1976-1992	A or B Rock	193	243	310
	C Shallow Soil	154	194	247
	D Soft Soil	122	132	168
	E Very Soft Soil	122	85	108
1976-1984 Reinforced Concrete	A or B Rock	232	292	373
	C Shallow Soil	185	233	296
	D Soft Soil	146	159	201
	E Very Soft Soil	146	102	130
1992-2004	A or B Rock	143	138	115
	C Shallow Soil	135	167	135
	D Soft Soil	133	154	126
	E Very Soft Soil	133	100	81

AUCKLAND

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	55	139	224
	C Shallow Soil	44	111	178
	D Soft Soil	35	68	110
	E Very Soft Soil	37	44	71
1965-1976	A or B Rock	81	130	83
	C Shallow Soil	65	104	66
	D Soft Soil	51	64	41
	E Very Soft Soil	55	41	26
1976-1992	A or B Rock	175	221	282
	C Shallow Soil	140	177	224
	D Soft Soil	111	120	152
	E Very Soft Soil	111	77	98
1976-1984 Reinforced Concrete	A or B Rock	211	265	339
	C Shallow Soil	168	212	269
	D Soft Soil	133	144	183
	E Very Soft Soil	133	93	118
1992-2004	A or B Rock	111	107	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	77	63

Table 3A.6: $(\%NBS)_b$ Auckland, μ = 3, Importance Levels 2, 3 and 4

AUCKLAND

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	263	397	508
	C Shallow Soil	210	318	404
	D Soft Soil	166	216	274
	E Very Soft Soil	166	139	177
1976-1984 Reinforced Concrete	A or B Rock	315	477	609
	C Shallow Soil	252	381	484
	D Soft Soil	199	259	329
	E Very Soft Soil	199	167	212
1992-2004	A or B Rock	159	153	128
	C Shallow Soil	150	186	150
	D Soft Soil	147	172	140
	E Very Soft Soil	147	111	90

AUCKLAND

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	289	437	558
	C Shallow Soil	231	350	444
	D Soft Soil	182	238	301
	E Very Soft Soil	182	153	194
1976-1984 Reinforced Concrete	A or B Rock	347	524	670
	C Shallow Soil	278	420	533
	D Soft Soil	219	285	362
	E Very Soft Soil	219	184	233
1992-2004	A or B Rock	143	138	115
	C Shallow Soil	135	167	135
	D Soft Soil	133	154	126
	E Very Soft Soil	133	100	81

AUCKLAND

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	263	397	508
	C Shallow Soil	210	318	404
	D Soft Soil	166	216	274
	E Very Soft Soil	166	139	177
1976-1984 Reinforced Concrete	A or B Rock	315	477	609
	C Shallow Soil	252	381	484
	D Soft Soil	199	259	329
	E Very Soft Soil	199	167	212
1992-2004	A or B Rock	111	107	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	77	63

Table 3A.7: (*%NBS*)$_b$ Christchurch, μ = 1.25, Importance Levels 2, 3 and 4

CHRISTCHURCH

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	55	90
	C Shallow Soil	20	44	71
	D Soft Soil	16	27	44
	E Very Soft Soil	18	18	28
1965-1976	A or B Rock	43	59	75
	C Shallow Soil	34	47	60
	D Soft Soil	27	29	37
	E Very Soft Soil	29	19	24
1976-1992	A or B Rock	122	153	196
	C Shallow Soil	98	123	156
	D Soft Soil	77	83	106
	E Very Soft Soil	77	54	68
1976-1984 Reinforced Concrete	A or B Rock	146	184	235
	C Shallow Soil	117	147	187
	D Soft Soil	92	100	127
	E Very Soft Soil	92	65	82
1992-2004	A or B Rock	94	90	75
	C Shallow Soil	88	110	88
	D Soft Soil	87	101	83
	E Very Soft Soil	87	65	53

CHRISTCHURCH

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	55	90
	C Shallow Soil	20	44	71
	D Soft Soil	16	27	44
	E Very Soft Soil	18	18	28
1965-1976	A or B Rock	44	60	77
	C Shallow Soil	35	48	61
	D Soft Soil	27	29	38
	E Very Soft Soil	30	19	24
1976-1992	A or B Rock	134	169	216
	C Shallow Soil	107	135	172
	D Soft Soil	85	92	116
	E Very Soft Soil	85	59	75
1976-1984 Reinforced Concrete	A or B Rock	161	203	259
	C Shallow Soil	129	162	206
	D Soft Soil	101	110	140
	E Very Soft Soil	101	71	90
1992-2004	A or B Rock	84	81	68
	C Shallow Soil	79	99	79
	D Soft Soil	78	91	74
	E Very Soft Soil	78	59	48

CHRISTCHURCH

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	18	39	63
	C Shallow Soil	14	31	50
	D Soft Soil	11	19	31
	E Very Soft Soil	12	12	20
1965-1976	A or B Rock	32	43	56
	C Shallow Soil	25	35	44
	D Soft Soil	20	21	27
	E Very Soft Soil	22	14	18
1976-1992	A or B Rock	122	153	196
	C Shallow Soil	98	123	156
	D Soft Soil	77	83	106
	E Very Soft Soil	77	54	68
1976-1984 Reinforced Concrete	A or B Rock	146	184	235
	C Shallow Soil	117	147	187
	D Soft Soil	92	100	127
	E Very Soft Soil	92	65	82
1992-2004	A or B Rock	66	63	53
	C Shallow Soil	62	77	62
	D Soft Soil	61	71	58
	E Very Soft Soil	61	46	37

Table 3A.8: (*%NBS*)$_b$ Christchurch, μ = 2, Importance Levels 2, 3 and 4

CHRISTCHURCH

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	46	117	190
	C Shallow Soil	37	94	151
	D Soft Soil	29	58	93
	E Very Soft Soil	32	37	60
1965-1976	A or B Rock	78	124	80
	C Shallow Soil	62	99	63
	D Soft Soil	49	61	39
	E Very Soft Soil	53	39	25
1976-1992	A or B Rock	161	203	259
	C Shallow Soil	129	162	206
	D Soft Soil	102	110	140
	E Very Soft Soil	102	71	90
1976-1984 Reinforced Concrete	A or B Rock	194	244	311
	C Shallow Soil	155	195	248
	D Soft Soil	122	132	168
	E Very Soft Soil	122	85	108
1992-2004	A or B Rock	124	120	100
	C Shallow Soil	117	145	117
	D Soft Soil	115	134	109
	E Very Soft Soil	115	86	70

CHRISTCHURCH

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	46	117	190
	C Shallow Soil	37	94	151
	D Soft Soil	29	58	93
	E Very Soft Soil	32	37	60
1965-1976	A or B Rock	79	127	81
	C Shallow Soil	63	101	65
	D Soft Soil	50	62	40
	E Very Soft Soil	54	40	26
1976-1992	A or B Rock	177	223	285
	C Shallow Soil	142	179	227
	D Soft Soil	112	121	154
	E Very Soft Soil	112	78	99
1976-1984 Reinforced Concrete	A or B Rock	213	268	342
	C Shallow Soil	170	214	272
	D Soft Soil	134	146	185
	E Very Soft Soil	134	94	119
1992-2004	A or B Rock	112	108	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	78	63

CHRISTCHURCH

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	32	82	133
	C Shallow Soil	26	66	106
	D Soft Soil	20	40	65
	E Very Soft Soil	22	26	42
1965-1976	A or B Rock	58	92	59
	C Shallow Soil	46	74	47
	D Soft Soil	36	45	29
	E Very Soft Soil	39	29	19
1976-1992	A or B Rock	161	203	259
	C Shallow Soil	129	162	206
	D Soft Soil	102	110	140
	E Very Soft Soil	102	71	90
1976-1984 Reinforced Concrete	A or B Rock	194	244	311
	C Shallow Soil	155	195	248
	D Soft Soil	122	132	168
	E Very Soft Soil	122	85	108
1992-2004	A or B Rock	87	84	70
	C Shallow Soil	82	101	82
	D Soft Soil	80	94	77
	E Very Soft Soil	80	60	49

Table 3A.9: $(\%NBS)_b$ Christchurch, μ = 3, Importance Levels 2, 3 and 4

CHRISTCHURCH

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	161	244	311
	C Shallow Soil	129	195	248
	D Soft Soil	102	132	168
	E Very Soft Soil	102	85	108
1976-1984 Reinforced Concrete	A or B Rock	194	292	374
	C Shallow Soil	155	234	297
	D Soft Soil	122	159	202
	E Very Soft Soil	122	102	130
1992-2004	A or B Rock	124	120	100
	C Shallow Soil	117	145	117
	D Soft Soil	115	134	109
	E Very Soft Soil	115	86	70

CHRISTCHURCH

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	177	268	342
	C Shallow Soil	142	214	272
	D Soft Soil	112	146	185
	E Very Soft Soil	112	94	119
1976-1984 Reinforced Concrete	A or B Rock	213	322	411
	C Shallow Soil	170	257	327
	D Soft Soil	134	175	222
	E Very Soft Soil	134	113	143
1992-2004	A or B Rock	112	108	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	78	63

CHRISTCHURCH

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	161	244	311
	C Shallow Soil	129	195	248
	D Soft Soil	102	132	168
	E Very Soft Soil	102	85	108
1976-1984 Reinforced Concrete	A or B Rock	194	292	374
	C Shallow Soil	155	234	297
	D Soft Soil	122	159	202
	E Very Soft Soil	122	102	130
1992-2004	A or B Rock	87	84	70
	C Shallow Soil	82	101	82
	D Soft Soil	80	94	77
	E Very Soft Soil	80	60	49

Appendix 3B: Assessment of Attribute Score for URM Buildings

For URM buildings built prior to 1935, the IEP can be carried out using the attribute scoring method outlined in this Appendix. The *%NBS* is then determined directly from the Total Attribute Score as described below.

The recommended procedure is;

1. Complete the attribute scoring Table 3B.1 using the guidance provided in Table 3B.2.
2. From the Total Attribute Score determine the *%NBS* from Table 3B.3

Interpolation may be used for intermediate attribute scores. While attributes may differ for each principal direction, it is the intention that the attribute score apply to the building as a whole. Given that local collapse is viewed as having the same implications as total collapse, attributes should correspond to the weakest section of a building where relevant.

The derivation of *%NBS* using the attribute scoring method outlined, assumes that all appendages likely to present a hazard have been adequately secured or measures taken to remove the risk to life, e.g. provision of appropriately designed canopies or designated “no go” zones adjacent to the building.

Table 3B.1: Assessment of Attribute Score

Item		Attribute ranking				Assessed score	
		0	1	2	3	Long	Trans
1	**Structure continuity**	Excellent	Good	Fair	Poor or none		
2	**Configuration**						
2a	Horizontal regularity	Excellent	Good	Fair	Poor		
2b	Vertical regularity	Excellent	Good	Fair	Poor		
2c	Plan regularity	Excellent	Good	Fair	Poor		
3	**Condition of structure**						
3a	Materials	Sound	Good	Fair	Poor		
3b	Cracking or movement	Not evident	Minor	Moderate	Severe		
4	**Wall (URM) proportions**						
4a	Out of plane	Good			Poor		
4b	In-plane	Excellent	Good	Fair	Poor		
5	**Diaphragms**						
5a	Coverage	Excellent	Good	Fair	Poor		
5b	Shape	Excellent	Good	Fair	Poor		
5c	Openings	None		Significant			
6	**Engineered connections between floor/roof diaphragms and walls, and walls and diaphragms** capable of spanning between	Yes		No			
7	**Foundations**	Excellent	Good	Fair	Poor		
8	**Separation from neighbouring buildings**	Adequate		Inadequate			
		Total Attribute Score;:		***for each direction***			
				for building as a whole:			

Notes:

For definition of grading under each attribute refer Table 3B.2

Table 3B.2: Definition of attributes and scores

	Atribute score[1]
Attribute Item (1): Structure continuity	
Totally un-reinforced masonry	3
Some continuity, e.g. un-reinforced masonry with a concrete band at roof *or* floor level	2
Good continuity, e.g. un-reinforced masonry with reinforced bands at *both* roof and floor levels	1
Full continuity (i.e. vertical stability not reliant on URM), e.g. reinforced concrete or steel columns and beams with un-reinforced masonry walls/infill or separate means of vertical support provided to floors and roof	0

	Atribute score[1]
Attribute Item (2): Configuration	
(a) Horizontal regularity	
Severe eccentricity, i.e. distance between storey centre of rigidity and the centre of mass for all levels above that storey, $e_d \geq 0.3\ b$ (b = longest plan dimension of building perpendicular to direction of loading)	3
$e_d \leq 0.3\ b$	2
$e_d \leq 0.2\ b$	1
Building symmetrical in both directions	0
(b) Vertical regularity	
Vertical stiffness discontinuities or discontinuities in load paths present	3
All walls continuous to foundations	2
and no soft storeys *and* minimal vertical stiffness changes	1
and no weak storeys *and* no significant mass irregularities	0
where:	
soft storey is a storey where the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average stiffness of the three storeys above	
weak storey is a storey where the storey strength is less than 80% of the strength of the storey above	
a *mass irregularity* exists if the mass varies by more than 50% from one level to another (excluding light roofs which should be considered as a *part* of the building).	
(c) Plan regularity	
Sharp re-entrant corners present where the projection of the *wing* beyond the corner > 0.15 b	3
Regular in plan	0
Attribute Item (3): Condition of structure	
(a) Materials	
Poor, i.e. considerable deterioration, fretting or spalling, etc., or lime or other non-competent mortar or rubble wall construction	3
Fair, i.e. deterioration leading to reduced strength	2
Good, i.e. minor evidence of deterioration of materials	1
Sound	0
(b) Cracking or movement	
Severe, i.e. a considerable number of cracks or substantial movement leading to reduced strength *or* isolated large cracks	3
Moderate	2
Minor	1
Non-evident	0

	Atribute score[1]
Attribute Item (4): Wall (URM) proportions	
(a) Out of plane performance	
Poor,	3
for one storey buildings $h_w/t \geq 14$ *and* $l_w/t \geq 7$	
for multistorey buildings:	
top storey $h_w/t \geq 9$ *and* $l_w/t \geq 5$	
other storeys $h_w/t \geq 20$ *and* $l_w/t \geq 10$	
Good (not poor)	0
Where h_w = height of wall between lines of positive lateral restraint and l_w = length of wall between lines of positive lateral restraint	

(b) In plane performance[2]

	A_p/A_w			
	One storey building	2 and 3 storey buildings		
		Top storey	Other stories	
Poor	≥25	≥20	≥17	3
Fair	>20	>15	>12	2
Good	>15	>10	>7	1
Excellent	≤15	≤10	≤7	0

Where A_w = cross sectional area of all URM walls/wall sections extending over full height of storey

A_p = plan area of building above storey of interest.

For buildings of greater than 3 stories take attribute score = 3

Attribute Item (5): Diaphragms (Refer Figure 3B.1)

(a) Coverage

No diaphragm	3
Full diaphragm	0

To achieve an attribute ranking of 0 requires a diaphragm to be present at each level, including roof level, covering at least 90% of the building plan area at each level. Interpolation for attribute rankings of 1 and 2 may be made using judgement on the extent of coverage. Note that unless the diaphragm is continuous between walls, its effectiveness may be minimal.

(b) Shape

	Limiting span to depth ratios for diaphragms of different construction material				
	Concrete	***Sheet materials***	***T&G timber***	***Steel roof bracing***	
Poor	> 4	> 4	> 3	> 5	3
Fair	< 4	< 4	< 3	< 4	2
Good	≤ 3	≤ 3	≤ 2	≤ 3.5	1
Excellent	As for good, but in addition the projection of "wings" beyond sharp re-entrant corners < 0.5b.				0

	Atribute score[1]
(c) Openings	
Significant openings	3
No significant openings	0
Interpolation for attribute rankings of 1 and 2 may be made using judgement. Significant openings are those which exceed the limiting values given below. Refer Figure 3B.1 for definition of terms	

Diaphragm construction material	Limiting values of	
	X/b	Y/D
Concrete	0.6	0.5
Sheet material	0.5	0.4
T&G timber	0.4	0.3

Attribute Item (7): Foundations	
Separate foundations with no interconnection *or* un-reinforced piles (unless ramification of pile failure is assessed to be minor).	3
Pads, strips or piles with some interconnection. Concrete piles to be reinforced unless ramification of pile failure is assessed to be minor.	2
Pads, strips or piles with good interconnection in both directions.	1
Concrete raft with sound connections to walls	0
Attribute Item (8): Separation	
Inadequate – no separation provided or obviously inadequate provisions for separation	3
Adequate – separation provided	0

Notes

1 Individual attribute scores may be interpolated.

2 This is an index describing the extent of b rick walls within the building. The numb ers given are on ly loosely related to lateral load capacity.

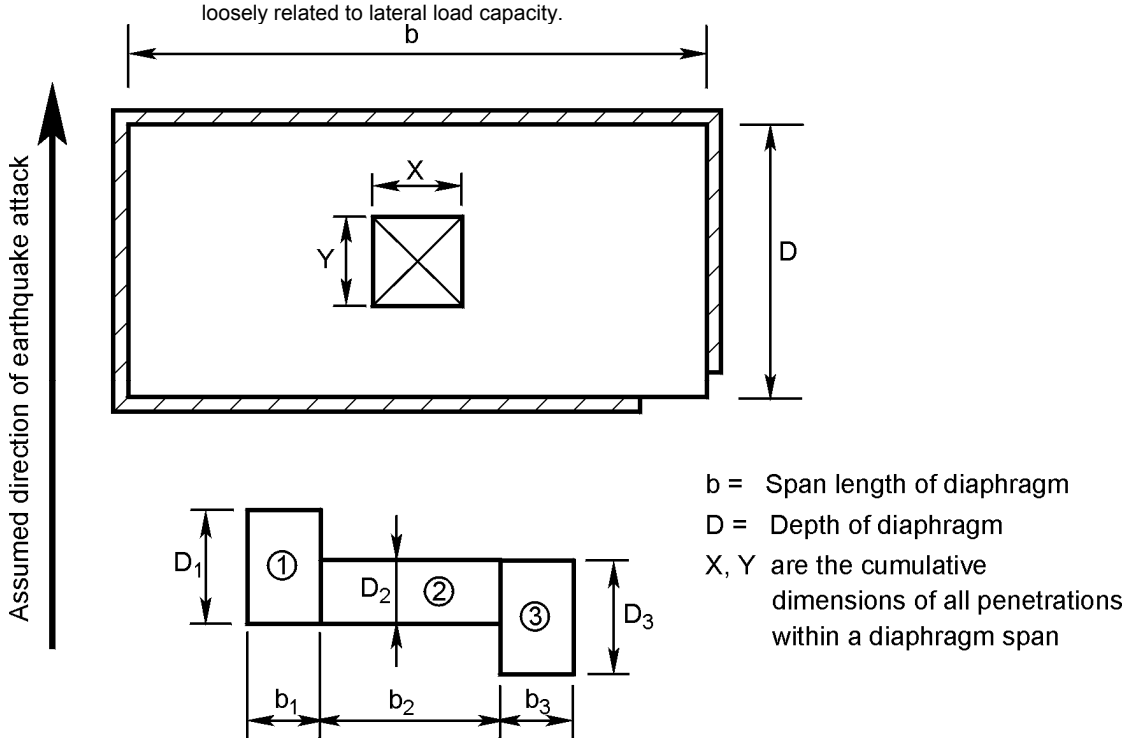


Figure 3B.1: Diaphragm parameters

Table 3B.3: Assessment of *%NBS* from Attribute Score

Item	Attribute Score	*%NBS*
1	A score of 0 for all attribute scoring items	67
2	Less than or equal to 1 for all of attribute scoring items 1 to 6 inclusive, and less than 2 for each of attribute scoring items 7 and 8	35
3	As for 2 but a score of 0 for attribute scoring item 1	40
4	5 < Total Attribute Score ≤ 10	20
5	10 < Total Attribute Score ≤ 15	15
6	15 < Total Attribute Score ≤ 25	10
7	Total Attribute Score > 25	5

Appendix 4A: Typical Pre-1976 Steel Building Systems Used in New Zealand

4A.1 General

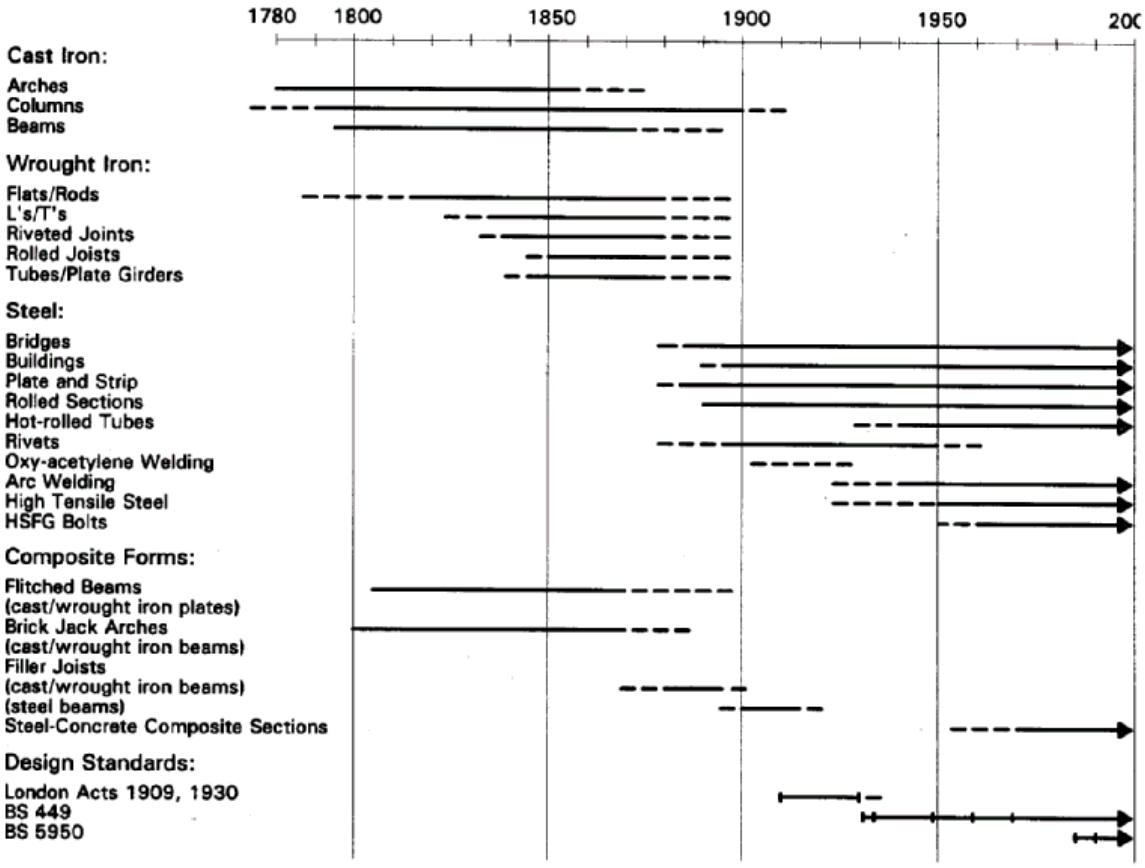
This section gives general guidance on the typical pre-1976 steel building systems used in New Zealand. The material presented is based on published material and details supplied by design engineers. It is intended that this section be extended as more buildings are assessed in the future.

4A.2 Use of iron and steel in existing buildings

Bussell (1997) gives a good summary of the use of iron and steel in structures from 1780 to the present day. In the New Zealand context, the relevant period covers ≈ 1900 to 1976. The main periods of use of the various materials is summarised in Table 4A.1.

Most ferrous material in existing New Zealand buildings will be steel, which was the preferred material for structural members in buildings from 1880 onwards. The exception is columns, especially gravity carrying columns functioning as vertical props for the floor. Cast iron was used for these through to just after 1900 and cast iron columns are found in some of the oldest New Zealand buildings. How to identify such columns is identified in Appendix 4C, section 4C.1.

Table 4A.1: Main periods of the structural use of cast iron, wrought iron and steel



Source: Bussell (1997).

4A.3 Moment-resisting frames

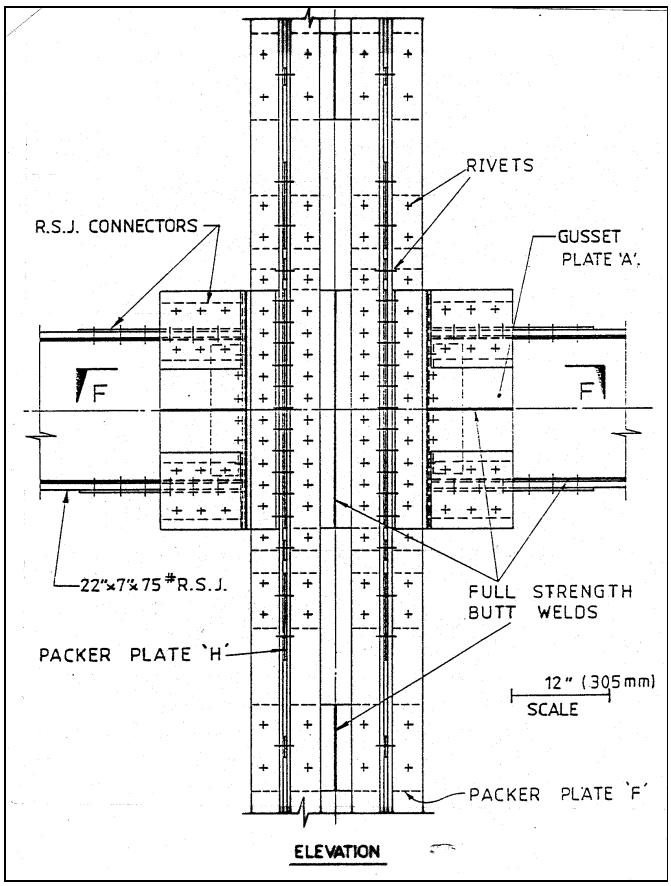
1 Beams: these were typically rolled steel joist (RSJ) sections, which are I-sections where the inside face of the flanges is not parallel to the outside face, being at a slope of around 15%. This makes the flanges thicker at the root radius than at the tips.

The flange slenderness ratios of RSJ sections are always compact when assessed to NZS 3404:1997.

These beams were typically encased in concrete for fire resistance and appearance, with this concrete containing nominal reinforcement made of plain round bars or, sometimes, chicken wire.

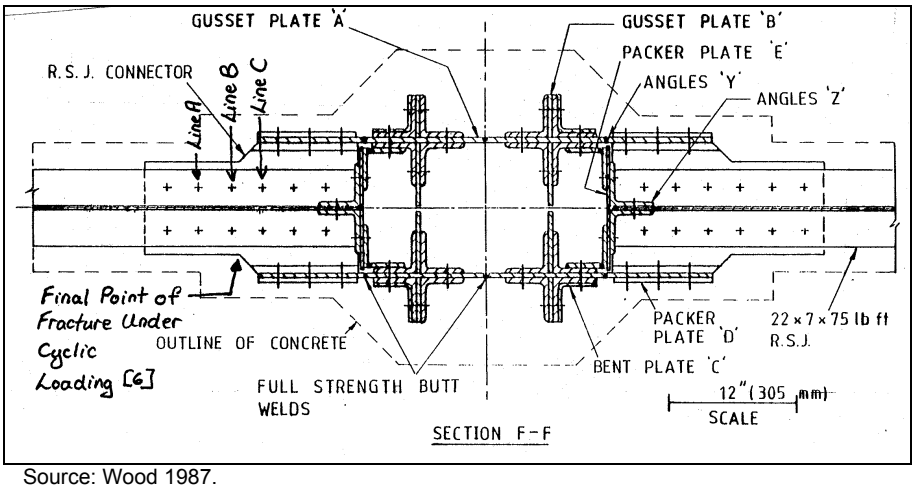
2 Columns formed from hot-rolled sections used either hot-rolled steel columns (RSCs) or box columns formed by connecting two channels, toes out, with a plate to each flange. The columns were encased in lightly reinforced concrete containing nominal reinforcement made of plain round bars.

3 Compound box columns were also formed from plates, joined by riveted or bolted angles into a box section and encased in concrete. Examples of this type of construction are shown in Figures 4A.1 and 4A.2.



Source: Wood 1987.
Note: See also Figure C9.2.

Figure 4A.1: Riveted steel fabrication details, Government Life Insurance Building, 1937



Source: Wood 1987.

Figure 4A.2: Riveted steel fabrication details, Government Life Insurance Building, 1937

4 Beam to column connections in the earlier moment frames typically comprised semi-rigid riveted or bolted connections. The RSJ beam flanges were bolted to Tee-stubs or angles bolted to the column flanges or to lengths of RSJ bolted to side extensions of the column plates. An example of the latter is shown in Figure 4A.2.

The RSJ beam web was connected by a double clip angle connection to the column flanges, again as shown in Figure 4A.2.

A simpler version of a semi-rigid connection used in some pre-1976 buildings is shown in Figure 8A.1 of Appendix 8A.

These joints generally involved the use of rivets up to 1950 and HSFG bolts after 1960, with a changeover from rivets to bolts from 1950 to 1960.

5 Beam to column connections from about 1940 onwards were also arc welded. The strength and ductility available from welded connections requires careful evaluation and attention to load path. This topic is addressed in section 8.4.2 and its importance is illustrated in Figure 4A.3. That figure, taken from a building collapsed by the Kobe earthquake of 1995, shows a failed beam to column minor axis connection, forming part of a moment-resisting frame in that direction. The beam was welded to an endplate which was fillet welded to the column flange tips. Unlike the connection detail shown in Figure 4A.2, there was no way to reliably transfer the concentrated axial force in the beam flanges, that is induced by seismic moment, from the beam into the column, with the weld between endplate and column flange unzipping under the earthquake action.

While this example is from Japan, the detail is also relevant to some early New Zealand buildings and the concept is certainly relevant.

Figure 4A.3: Failed beam to column weak axis welded connection from the 1995 Kobe earthquake

Figure 4A.4: Braced frame with light tension bracing showing damage but no collapse from the 1995 Kobe earthquake

6 Splices in columns. These typically involved riveted (pre-1950) or bolted (post-1950) steel sections, with the rivets or bolts transferring tension across the splice and compression being transferred by direct bearing. Figures 4A.1 and 4A.2 show plated box columns connected by riveted angles, while Figure 4A.3 shows a bolted UC splice detail in the column, this being a fore-runner to the bolted column splice details of HERA Report R4-100 (Hyland 1999). Such bolted splices generally perform well.

4A.4 Braced frames

For the pre-1976 buildings covered by this document, braced frames incorporating steel bracing involve concentrically braced framing (CBF), either x-braced CBFs or V-braced CBFs.

Figure 4A.4 shows an X-braced CBF with relatively light bracing and Figure 4A.5 V-braced CBF. Both are from Kobe, Japan but are similar to details used in early New Zealand buildings.

Figure 4A.5: V-braced CBF showing damage but no collapse from the 1995 Kobe earthquake

Appendix 4B: Relationships Between Structural Characteristics and Steel Building Performance in Severe Earthquakes

A small number of pre-1975 steel framed buildings (older steel-framed buildings) were damaged in the 1994 Northridge earthquake and a significant number in the 1995 Hyogo-ken Nanbu (Kobe) earthquake. From the pattern and extent of damage observed, some general recommendations can be made in order to guide the evaluation of this type of building. A background to these recommendations is now given, followed by details of the recommendations themselves.

The Los Angeles Northridge earthquake, in January 1994, caused considerable damage to modern, ductile moment-resisting steel frames (DMRSFs). This damage took the form of fracture between the beam flange to column flange connection of the rigid beam to column connections. Further details on the nature of the damage and reasons for it are given in Clifton (1996b).

The failures turned the initially rigid connections into semi-rigid connections, with the connection as the weakest flexural link relative to the moment capacity of the beam or the column. The vertical load-carrying capacity remained adequate and the connections retained a reduced moment capacity. Thus the inelastic demand on the frame was concentrated into the connections, which in semi-rigid form retained appreciable ductility.

The hysteretic performance (cyclic moment-rotation curves) representative of the damaged connections is described in Astaneh-Asl (1995). The nature of these curves can be described as being:

a) pinched hysteretic loops with little energy absorption
b) broadly elastoplastic in nature, but not symmetrical, due to the influence of the floor slab
c) susceptible to minor degradation over successive cycles.

While over 100 buildings suffered joint damage in this earthquake, the general response of these buildings was good. Most showed no outward non-structural signs of distress after the earthquake, such as permanent lateral drift, nor were there indications of unexpectedly large interstorey lateral deflections developed during the earthquake. Thus the nature of MRSF response, where the weak link was in the connections, was satisfactory under the high-intensity Northridge Earthquake, which had maximum spectral accelerations in the 0.2–0.8 second period range. (This is reasonably representative of the NZS 1170.5:2004 (or NZS 4203:1992) design spectra for intermediate and stiff soil sites.)

The Hyogo-ken Nanbu (Kobe) earthquake in Japan, in January 1995, caused damage to a range of steel framed buildings, but principally to older, medium – rise commercial and industrial buildings. Large numbers of these older (pre-1981) buildings suffered damage. Their poor performance was due to one or more of the following reasons (Clifton 1996b):

(i) poor distribution of strength/stiffness over successive storeys, leading to soft storey formation

(ii) lack of provision for an adequate load path through the connections, leading to partial or complete connection failure, especially loss of vertical load-carrying capacity

(iii) inadequate strength of the overall seismic-resisting system

(iv) inadequate stiffness of the overall seismic-resisting system

(v) in the case of some older residential buildings, corrosion of the steel frame due to long-term build up of condensation in the external walls envelope.

The pattern of damage from both earthquakes has showed that, for seismic-resisting systems which exhibited inelastic response, three factors are important in order to achieve a good performance of the overall building. These are:

1) the beam to column connections retain their integrity, with regard to carrying shear and axial force, if their moment capacity is reduced
2) inelastic demand is minimised in the columns: both member rotational demand due to general plastic hinging and localised deformation due to local buckling or tearing failure. The former demand can arise from soft-storey formation, as for example is illustrated in Figure 4B.1. In this instance, the soft storey demand has arisen due to the bracing system encompassing all except the bottom storey, resulting in the ductility demand being concentrated into that level. The latter demand is most typically caused by inappropriate detailing for transfer of forces through the connection of incoming beam or brace members into the column. An example is shown in Figure 4B.2 and this concept is covered in detail in section 8.4.2.
3) the inelastic response is essentially symmetrical in nature and does not lead to a progressive displacement of the building in one direction.

These three factors are embodied in the guidelines for evaluation which follow.

Figure 4B.1: Example of soft storey generated by change from braced to moment frame at bottom storey, 1995 Kobe earthquake

Figure 4B.1: Local column crippling failure due to lack of stiffener adjacent to incoming beam flange in a welded, moment-resisting beam to column connection, 1995 Kobe earthquake

Appendix 4C: Assessing the Mechanical Properties of Steel Members and Components

4C.1 Is it cast iron, wrought iron or steel?

The earliest steel framed buildings likely to be requiring a seismic assessment would have been built in the 1880s. As shown in Table 4A.1 of Appendix 4A, the use of cast iron from that time, until its discontinuance around 1910, was confined to columns. These would have typically been used for gravity load carrying columns only. They are typically "chunky" with thick sections, often ornate or complex profile (fluted or plain hollow circular or cruciform columns). Their surface is typically pitted with small blowholes. More detailed visual characteristics are given in Table 7.1 of the SCI Publication 138 (Bussell 1997).

Cast iron is a low-strength, low ductility material not suitable for incorporation into a seismic-resisting system. However, if used as a propped gravity column, with the supports for the beams assessed and reinforced if necessary (e.g. with steel bands) to avoid local fracture under seismic-induced rotation, they can be dependably retained. For more guidance on their assessment for this application (see Bussell 1997).

Wrought iron has good compressive and tensile strength, good ductility and good corrosion resistance. Its performance in this regard is comparable to that of steels from the same era, which largely ended around the 1880s and 1890s. The principal disadvantage of wrought iron was the small quantities made in each production item (bloom), being only 20 to 50 kg. This meant that the use of wrought iron in structural beam and column members required many sections to be joined by rivets. For that reason it was rarely used in building structures in New Zealand. If a building being assessed contains members built up from many small sections of I sections, channels and/or flats and which dates from earlier than 1900, then the use of wrought iron in these members should be further assessed, using the guidance in Sections 3.4 and 7 of Bussell (1997).

All other ferrous components in buildings under assessment can be considered as being made from steel.

If in doubt, the visual assessment criteria in Table 7.1 of Bussell (1997) can be used for more detailed visual consideration.

4C.2 Expected yield and tensile strengths of steels, fasteners and weld metals

The following information is taken from Bussell (1997) and Ferris. The values given are minimum values, being consistent with the requirements from NZS 3404 for the material properties used to be the minimum specified values. This information is given in Table 4C.1 for steels from America and Table 4C.2 for steels from the UK. In the case of the UK, the minimum properties given should be used in the assessment. Properties of UK steels and rivets prior to 1906 can be obtained from Bussell (1997).

Table 4C.1: Minimum material properties for steels and rivets manufactured in the USA

Time period	Application	Minimum yield stress (MPa)	Minimum tensile strength (MPa)
< 1900	Buildings Rivets	240 205	400 340
1900–10	Buildings Rivets	240 205	410 340
1910–25	Buildings Rivets	190 170	380 330
1925–32	Buildings Rivets	210 170	380 314
1932–50	Buildings Rivets	225 195	410 355
1950–76	Buildings (mild steel)	250	410
	Buildings (HT steel)	350	480

Source: Ferris (year?).

Table 4C.2: Typical properties of structural steels from the UK for the period 1906–68

Property (values in N/mm² unless noted)	Typical value (or range of values)	Notes
Ultimate tensile strength:		
BS 15: 1906	432-494	BS 15 covered mild steel
BS 15: 1912-1941	432-509	
BS 15: 1948-1961		
Rivet bar	386-463	
Other	432-509	
BS 548: 1934-1942		BS 548 covered high tensile steel
Rivet bar	463-540	
Other	571-664	
BS 968: 1941	As BS 548: 1934	BS 968 covered weldable high tensile steel
BS 968: 1943	509-633	
BS 968: 1962	494-602	
Yield strength:		
BS 15: 1948-1961	225-235	
BS 15: 1961-1968	230-250	No change in UTS.
BS 548: 1934-1942	293-355	No requirements for rivet bar; values depended on steel thickness, being lower for thicker sections
BS 968: 1941	As BS 548: 1934	
BS 968: 1943	293-324	
BS 968: 1962	340-355	
Elongation at failure (%):		
BS 15: 1906-1941		
Rivet bar	25 (min.)	
Other	20 (min.)	
BS 15: 1948-1961		
Rivet bar	26-30	Cold bend test
Other	16-24	
BS 548: 1934-1942		
Rivet bar	22-27	
Other	14-18	
BS 968: 1941-1943		
Plates	14-18	
Sections and bars	14-22	
BS 968: 1962		
Standard test pieces	15-23	

Source: Bussell (1997).

4C.3 Confirming tensile strength by test

Older steels have an inherently greater variability than modern steels, so it is important to undertake a minimum degree of non-destructive testing to gain sufficient assurance that the materials have the properties used in the assessment.

This testing should also be able to identify material that may exhibit brittle behaviour under seismic condition.

There is an approximate relationship between material hardness and tensile strength. Material hardness is represented in a number of ways, however the best relationship for the range of material strengths of interest (400 to 700 MPa) is given by the *Vickers Hardness*, H_v. Testing for Vickers Hardness is carried out to AS 1817 *Metallic Materials – Vickers Hardness Test* (1991).

That relationship is tabulated in ASM International (1976) and can be expressed in equation form as:

$$f_u = 3.09\ H_v + 21.2 \qquad \text{…4C(1)}$$

where H_v = Vickers Hardness from test.

This expression is valid for $100 \le H_v \le 300$, corresponding to $330 \le f_u \le 950$ MPa.

Vickers Hardness tests are readily undertaken on the insitu steel elements and there are a number of materials testing organisations which can perform this task.

The purpose of the tests is to:

- determine the general material strengths of the critical components
- identify components which have unexpectedly high or low strengths and hence need further investigation
- identify components that might be subject to brittle fracture under seismic conditions.

The steps involved in determining which elements to test and the number of tests to conduct are as follows:

Step 1: Determine the components to be tested, i.e. beams, columns, critical connection components and connectors. Those elements identified as critical from the connection evaluation in section 8.4.2 and the strength heirachy evaluation in section 8.5.2 should be subject to the most detailed testing, plus a lesser frequency of testing for other beam, column and brace members.

Step 2: Determine a frequency of testing. Use the guidance in Section 7.5 of Bussell (1997) and DCB No. 44, pp. 2–3 [Clifton (ed.)], aimed at covering 15% of the total sample of each type of component being tested for critical components; increasing this to 25% if the results show a significant number of suspect samples.

Step 3: Use eqn 4C(1) to obtain the tensile strength.

Step 4: Compare with the expected strengths from Section 4C.2 and make a judgement on the material's suitability. Any materials with $H_v < 100$ or $H_v > 230$ should be investigated more thoroughly by tensile sampling and visual inspection. Any materials with $H_v > 230$ should also be treated as potentially prone to brittle fracture.

4C.4 Suppression of brittle fracture

This becomes a issue for further investigation if the testing from Section 4C.2 shows up a steel with a H_v of over 230 and/or if the thickness of any element of existing steelwork is over 32 mm thick, when that element is in the "principal load-carrying path through the seismic-resisting system" (NZS 3404:1997) and is carrying axial or bending induced tension force. In those cases material from those elements needs to be removed for Charpy Impact Testing, as specified from NZS 3404, to determine the energy absorption. These tests should be conducted at 0^oC for elements of external steelwork and at 20^oC for elements of internal steelwork.

There is not a direct relationship between tensile strength and brittle fracture, however the susceptibility to brittle fracture increases with increasing tensile strength. The elongation also decreases with increasing strength. This guidance is therefore a threshold, requiring more appropriate testing for potential brittle fracture performance if it is not met.

Appendix 4D: Potential for Pounding

4D.1 Evaluation of Potential for Pounding

The effects of pounding need to be considered where both of the following criteria apply.

a) Either of the following conditions exist:

   i) Adjacent buildings are of different heights and the height difference exceeds two storeys or 20% of the height of the taller building, whichever is the greater.

   ii) Floor elevations of adjacent buildings differ by more than 20% of the storey height of either building.

b) Separation between adjacent buildings at any level is less than a distance given by:

   $S = \sqrt{U_1^2 + U_2^2}$

   where U_1 = estimated lateral deflection of Building 1 relative to ground under the loads used for the assessment.

   and U_2 = estimated lateral deflection of Building 2 relative to ground under two-thirds of the loads used in the assessment.

However, the value of 'S' calculated above need not exceed 0.028 times the height of the building at the possible level(s) of impact.

Where adjacent buildings are of similar height and have matching or similar floor levels, no account need be taken of the effects of pounding on either building irrespective of the provided separation clearances.

4D.2 Assessment of Pounding Effects

Where required to account for the effect of pounding in 1 above, the following alternative approaches may be adopted.

4D.2.1 Analytical approach

A proper substantiated analysis shall be undertaken that accounts for the transfer of momentum and energy between the buildings as they impact. Elements and components of the building structures shall be capable of resisting the forces resulting from impact, giving due consideration to their ductility capacity and need to sustain vertical forces under such impact loading.

4D.2.2 Approximate approach

i) For the case of two unequal height buildings where their floor elevations align, the impact-side columns of the taller building should have sufficient strength to resist the following design actions.

   - 175% of the column design actions (shear, flexural and axial) occurring under the application of the seismic lateral loading of NZS 1170.5:2004, assuming the building is free standing, applied above the height of the building corresponding to that of the adjacent shorter building. | 2
   - 125% of the column design actions occurring under the application of the seismic lateral loading of NZS 1170.5:2004, assuming the building is free standing, over the height of the building corresponding to that of the adjacent shorter building.

All other columns remote from the building side suffering impact shall have sufficient strength to resist 115% of the column design actions occurring under the application of the seismic lateral loading of NZS 1170.5:2004, assuming the building is free standing, over the full height of the building.

ii) For the case where the floor elevations of adjacent buildings differ, with the potential for mid-storey hammering of each building, the impact-side columns of the building(s) which may be impacted between storeys should have sufficient strength to resist design actions resulting from imposition of a displacement on the columns, at the point of impact, corresponding to one half of the value of '*S*' derived in 4A.1(b) above.

The imposed displacements need only be applied at any one level. However critical design actions shall be derived considering application of the imposed displacements at any level over the building height where impact could occur.

In addition, where the buildings are of unequal heights, in accordance with 4D.1(a)(i) above, the requirements of 4D.2.2 (i) shall also apply.

4D.3 Alternative Mitigation Approaches

Alternative means to mitigate the effects of pounding may be considered. These include:

- permanent connection of adjacent buildings. This approach may prove practical for a row or block of buildings of similar height and configuration.
- provision of additional structural elements and components away from the points of impact to compensate for components that may be severely damaged due to impact.
- provision of strong collision shear walls to act as buffer elements to protect the rest of the building (Anagnostopoulas and Spiliopoulos 1992). The use of collision shear walls would prevent mid storey impact to columns of adjacent buildings, reducing potential for local damage and partial or total collapse.

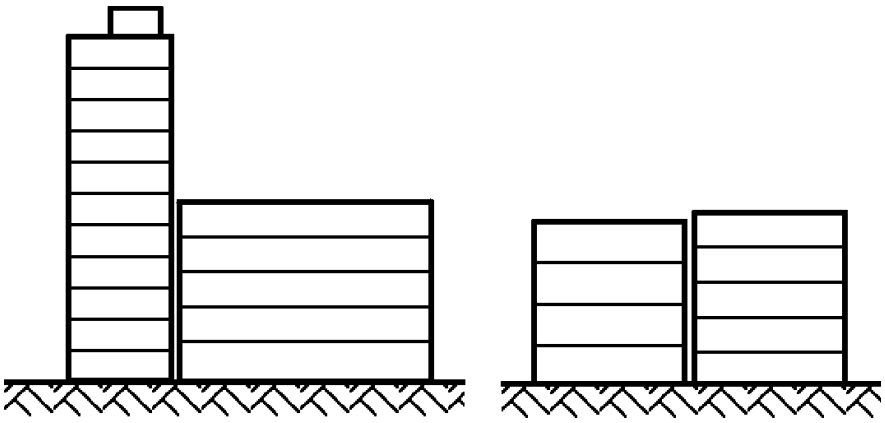
Older buildings have often been built up to property boundary lines, with little or no separation to adjacent buildings. Buildings with inadequate separation may consequently impact each other or pound during an earthquake. Such impacts will transmit short duration, high amplitude forces to the impacting buildings at any level where pounding occurs with the following consequential effects:

- *High "in-building" accelerations in the form of short duration spikes.*
- *Modification to the dynamic response of the buildings, the pattern and magnitude of inertial demands and deformations induced on both structures. Response may be amplified or de-amplified and is dependent on the relative dynamic characteristics of the buildings, including their relative heights, masses and stiffness', as well as ground conditions that may give rise to soil-structure interaction and the magnitude and direction of travel of the earthquake motions.*
- *Local degradation of strength and/or stiffness of impacting members.*

Numerous pounding damage surveys and numerical and analytical pounding studies have been undertaken in the last 10–15 years, especially after the 1985 Mexico City earthquake that caused an unusually large number of building failures. It is clear that pounding is a complex problem with numerous circumstances under which it can be encountered. The results of the studies that have been undertaken are sensitive to the many parameters related to the building structures (and their numerical modelling) in addition to the prevalent soil conditions and the characteristics and

direction of seismic attack. However based on these studies and evidence from past earthquakes, it is possible to draw the following general conclusions.

- *Where buildings are significantly different in height, period and mass, large increases in response from pounding can be expected.*
- *Differences in height in particular between neighbouring buildings can result in significant pounding effects, producing large response increases in the upper part of the taller building (refer Figure 4D.1(a)). The shears in the impact-side columns for the taller building can be up to 50–70% higher than in the no pounding case at the levels immediately above the lower building, and 25–30% at levels higher up, as the shorter building acts as a buttress to the taller building. In soft ground conditions where soil-structure interaction and through-soil coupling occurs, the impact-side shears can be enhanced by a further 25–50%.*
- *For buildings of similar height and having similar mass and stiffness, in most cases the effects of pounding will be limited to some local damage, mostly non-structural and nominal structural, and to higher in-building accelerations in the form of short duration spikes. In such conditions, from a practical viewpoint, the effects of pounding on global responses can be considered insignificant.*
- *Where building floors are at different elevations, the floor slabs of one structure can impact at the mid-storey of the columns of the others, shearing the columns and initiating partial or total collapse (refer Figure 4D.1(b)). Particularly susceptible to such action are buildings overtopping a shorter neighbouring building whose columns may be impacted at mid-storey by the uppermost level of the shorter building.*
- *The local high amplitude, short duration accelerations induced by colliding buildings will increase the anchoring requirements for the contents of the buildings as well as architectural elements.*



(a) Buildings of unequal height **(b) Buildings of equal height**

Figure 4D.1: Example of differing floor elevations in adjacent buildings

The potential or likelihood of pounding needs to be evaluated, using calculated drifts for both buildings. The SRSS combination of structural lateral deflections of both buildings is proposed, as adopted in FEMA 273 (NEHRP Guidelines), to check the adequacy of building separation. This approach has been adopted to account for the low probability of maximum drifts occurring simultaneously in both buildings whilst they respond completely out of phase. It is not intended that detailed analysis or modeling be undertaken to determine building drifts but rather general estimates be used.

Approximate analytical methods have been proposed for assessing the effects of pounding, including time history analyses (Johnson, Conoscente and Hamburger 1992) and elastic response spectrum analyses (Kasai, Maison and Patel 1990). Use of such approaches however may not prove practical for many buildings or within the capability of many design practitioners.

An alternative simplified approach has been proposed, based on simple factoring of earthquake design forces applicable to the building, to ensure some account of pounding effects is made. Both moment/shear capacities and p-delta effects need to be considered. Studies (Kasai, Maison and Patel 1990; Kasai, Jeng, Patel, et al 1992; Carr and Moss 1994) have shown that column and storey shears in the taller building above the pounding level can be increased by anywhere up to or exceeding 100%. The level of increase is dependent on many factors including initial separation distances and relative mass and stiffness of the adjacent buildings. A midrange increase in design shear has been adopted for the simplified approach at this stage. Whilst it is recognised that this approximate approach is relatively crude it has the benefit of ease of application without the need for use and familiarity with sophisticated analyses tools. It is expected that as further research on pounding is undertaken more appropriate and practical means to evaluate and mitigate pounding will become available.

Appendix 4E: Analysis Procedures

NOTE

This Appendix is based on material contained in FEMA 356.

Other background information can be found in FEMA 273 and 274.

This information is presented as commentary material to assist assessors in the application of the analysis procedures outlined in Section 4 and 6.

4E.1 Introduction and Scope

This appendix sets out the requirements for analysis of buildings and describes the general analysis requirements for mathematical modelling including basic assumptions, consideration of torsion, diaphragm flexibility, and P-Δ effects. Five methods that can be used to analyse a building are then described in detail.

Section 4.3.2and Table 4.2, summarise several elastic and inelastic analysis methods that can be used to assess strength and displacement demands that a building might be subjected to during and earthquake. Of the elastic methods, the Equivalent Static Method is a linear elastic procedure, while the Modal Response Spectrum Method is a linear dynamic procedure. In the case of the inelastic methods, the SLaMA and the Pushover Method are nonlinear static procedures whereas the Inelastic Time History Method is a nonlinear dynamic procedure.

Linear procedures are appropriate when the expected level of nonlinearity is low. Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Dynamic procedures are required for tall buildings, buildings with torsional irregularities, or non-orthogonal systems.

The Nonlinear Static Procedure is acceptable for most buildings, but should be used in conjunction with the Linear Dynamic Procedure if mass participation in the first mode is low.

The term "linear" in linear analysis procedures implies "linearly elastic." The analysis procedure, however, may include geometric nonlinearity of gravity loads acting through lateral displacements and implicit material nonlinearity of concrete and masonry components using properties of cracked sections. The term "nonlinear" in nonlinear analysis procedures implies explicit material nonlinearity or inelastic material response, but geometric nonlinearity may also be included.

4E.2 Mathematical Modelling

A building should be modelled, analysed, and evaluated as a three dimensional assembly of elements and components. However, use of a two dimensional model can be justified when:

1. The building has rigid diaphragms and horizontal torsion effects are not large or the horizontal torsion effects have been accounted for, or
2. The building has flexible diaphragms.

If two dimensional models are used, the three-dimensional nature of components and elements should be taken into account when calculating stiffness and strength properties.

If the building contains out-of-plane offsets in vertical lateral force-resisting elements, the model should explicitly account for these offsets when determining the demands on the diaphragms.

For nonlinear procedures, a connection should be modelled explicitly if the connection is weaker, has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.

For two-dimensional models, the three-dimensional nature of components and elements should be recognized in calculating their stiffness and strength properties. For example,

shear walls and other bracing systems may have "L" or "T" or other three dimensional cross-sections where contributions of both the flanges and webs should be accounted for in calculating stiffness and strength properties.
In these recommendations, component stiffness is generally taken as the effective stiffness based on the secant stiffness to yield level forces.
Examples of where connection flexibility may be important to model include the panel zone of steel moment-resisting frames, the "joint" region of perforated masonry or concrete walls, and timber diaphragms.

4E.3 Horizontal Torsion

The effects of horizontal torsion should be considered. Torsion need not be considered in buildings with flexible diaphragms as defined in Section 5(a) herein. The total horizontal torsional moment at a storey is given by the sum of the actual torsional moment and the accidental torsional moment as given in NZS 1170.5:2004, Clause 6.3.5.

Actual torsion is due to the eccentricity between the centres of mass and stiffness. Accidental torsion is intended to cover the effects of the rotational component of the ground motion, differences between computed and actual stiffnesses, and unfavourable distributions of dead and live load masses.

4E.4 Primary and Secondary Elements and Components

Elements and components may be classified as primary or secondary. Elements and components that affect the lateral stiffness or distribution of forces in a structure, or are loaded as a result of the lateral deformation of a structure should be classified as primary or secondary, even if they were not intended to be part of the lateral force resisting system.

Primary elements and components are those that provide the capacity of the structure to resist collapse under the seismic forces induced by the ground motion in any direction. Other elements and components can be classified as secondary. Primary elements and components should be checked for earthquake induced forces and deformations in combination with gravity load effects. Secondary elements and components should be checked for earthquake deformations in combination with gravity load effects.

NOTE
This definition of primary and secondary elements is not the same as used in NZS 3404 for steel structures.

4E.5 Diaphragms

4E.5.1 Classification of Diaphragms

Diaphragms should be classified as flexible when the maximum horizontal deformation of the diaphragm along its length is more than twice the average interstory drift of the vertical lateral-force-resisting elements of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm should be used.

Diaphragms should be classified as rigid when the maximum lateral deformation of the diaphragm is less than half the average interstory drift of the vertical lateral-force-resisting elements of the associated story.

Diaphragms that are neither flexible nor rigid should be classified as stiff.

For the purpose of classifying diaphragms, interstory drift and diaphragm deformations should be calculated using the pseudo lateral load specified in Equation (3-10). The in-plane deflection of the

diaphragm should be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, and all in-plane lateral forces associated with offsets in the vertical seismic framing at that diaphragm level.

4E.5.2 Mathematical Modelling

Mathematical modelling of buildings with rigid diaphragms should account for the effects of horizontal torsion as specified in Section 4E.3 above. Mathematical models of buildings with stiff or flexible diaphragms should account for the effects of diaphragm flexibility by modelling the diaphragm as an element with an in-plane stiffness consistent with the structural characteristics of the diaphragm system. Alternatively, for buildings with flexible diaphragms at each floor level, each lateral force-resisting element in a vertical plane may be permitted to be designed independently, with seismic masses assigned on the basis of tributary area.

Evaluation of diaphragm demands should be based on the likely distribution of horizontal inertia forces. For flexible diaphragms, such a distribution may be given by eqn 4E(1)) and illustrated in Figure 4E.1.

$$f_d = \frac{1.5\,F_d}{L_d}\left[1-\left(\frac{2x}{L_d}\right)^2\right] \qquad \text{...4E(1)}$$

where:
f_d = Inertial load per foot
F_d = Total inertial load on a flexible diaphragm
x = Distance from the centre line of flexible diaphragm
L_d = Distance between lateral support points for diaphragm

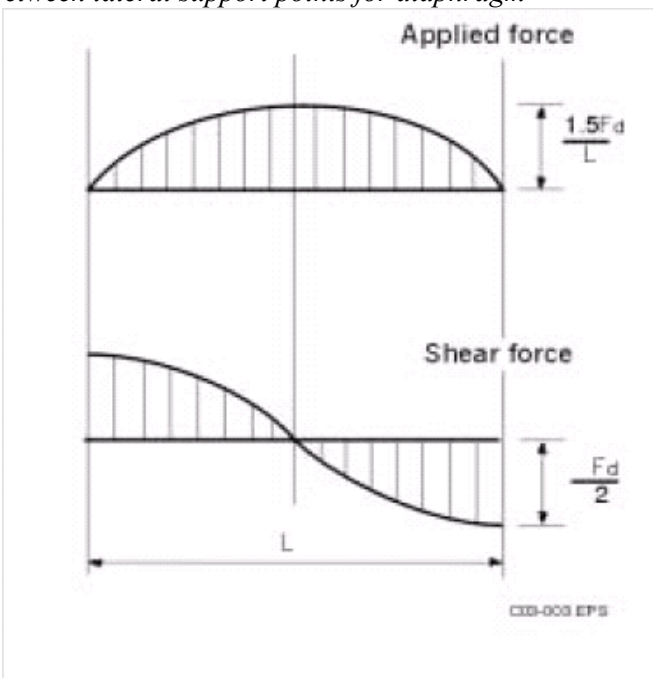


Figure 4E.1: Plausible force distribution in a flexible diaphragm

4E.6 P-Δ Effects

Buildings should be checked for P-Δ effects as set out in Section 6.5 of NZS 1170.5:2004.

P-Δ effects are caused by gravity loads acting through the deformed configuration of a building and result in increased lateral displacements.

A negative post-yield stiffness may significantly increase interstory drift and the target displacement. Dynamic P-Δ effects are introduced to consider this additional drift. The degree by which dynamic P-Δ effects increase displacements depends on the following:

1. The ratio α of the negative post-yield stiffness to the effective elastic stiffness;
2. The fundamental period of the building;
3. The strength ratio, R, (being the ratio of the yield strength to the ultimate strength);
4. The hysteretic load-deformation relations for each story;
5. The frequency characteristics of the ground motion; and
6. The duration of the strong ground motion.

4E.7 Methods of Analysis

Selection of an appropriate analysis method should be based on Table 4.2.

4E.8 Equivalent Static Analysis

4E.8.1 Period Determination

The fundamental period of the building can be calculated for the direction under consideration using one of the following analytical, empirical, or approximate methods.

a) Method 1 – Analytical

Dynamic (eigenvalue) analysis of the mathematical model of the building can be carried out to determine the fundamental period of the building.

For many buildings, including multi-storey buildings with well-defined framing systems, the preferred approach to obtaining the period for design is Method 1. In this method, the building is modelled using the modelling procedures of Section 5 through 8 and 11, and the period is obtained by Eigenvalue analysis. Flexible diaphragms may be modelled as a series of lumped masses and diaphragm finite elements.

b) Method 2 – Empirical

The fundamental period of the building shall be determined in accordance with:

1. $$T_1 = 1.25\, k_t\, h_n^{0.75} \qquad \text{…4E(2)}$$

where ;

k_t = 0.075 for moment resisting concrete frames
0.11 for moment-resisting steel frames
0.06 for eccentrically braced steel frames
0.05 for all other frame structures

h_n = height in m from the base of the structure to the uppermost seismic weight or mass.

2. Alternatively, the value k_t for structures with concrete shear walls may be taken as

$$k_t = 0.075 / \sqrt{A_c} \qquad \text{…4E(3)}$$

where

$$A_c = [A_i \{0.2 + (l_{wi} / h_n)\}^2]$$

and

A_c = total effective area of the shear walls in the first storey in the building, in m^2,

A_i = effective cross-sectional area of shear wall i in the first storey of the building, in m^2,
h_n = as in item 1 above,
l_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/h_n shall not exceed 0.9.

3. The estimation of T_1 may be made using the following expression:

$$T_1 = 2\sqrt{d} \qquad \text{…4E(4)}$$

where

d = the lateral elastic displacement of the top of the building, in m, due to gravity loads applied in the horizontal direction.

Empirical equations for period, such as that used in Method 2, intentionally underestimate the actual period and will generally result in conservative estimates of pseudo lateral load. Studies have shown that depending on actual mass or stiffness distributions in a building, the results of Method 2 may differ significantly from those of Method 1.

c) Method 3 - Approximate

1. For any building, the Rayleigh-Ritz method can be used to approximate the fundamental period.

The largest translational period in the direction under consideration, T_1, may be calculated from eqn 4E(5).

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n}\left(W_i d_i^{\,2}\right)}{g\sum_{i=1}^{n}\left(F_i d_i\right)}} \qquad \text{…4E(5)}$$

where

d_i = *the horizontal displacement in m of the centre of mass at level i, ignoring the effects of torsion*
F_i = *the displacing force in kN at level i*
g = *acceleration due to gravity in m/s^2*
i = *the level under consideration of structure*
n = *number of levels in a structure*
W_i = *the seismic weight in kN at level i.*

2. For one-story buildings with single span flexible diaphragms, eqn 4E(6) may be used to approximate the fundamental period.

$$T = \left(3.94\ U_w + 3.07\ U_d\right)^{0.5} \qquad \text{…4E(6)}$$

where U_w and U_d are in-plane wall and diaphragm displacements in metres, due to a lateral load in the direction under consideration, equal to the weight of the diaphragm.

3. For one-story buildings with multiple-span diaphragms, eqn 4E(6) may be used as follows: a lateral load equal to the weight tributary to the diaphragm span under consideration is applied to calculate a separate period for each diaphragm span. The period that maximizes the pseudo lateral load is used for design of all walls and diaphragm spans in the building.

4. For unreinforced masonry buildings with single span flexible diaphragms, six stories or less in height, eqn 4E(7) may be used to approximate the fundamental period.

$$T = (3.07\,U_d)^{0.5} \qquad \text{...4E(7)}$$

where U_d is the maximum in-plane diaphragm displacement in metres, due to a lateral load in the direction under consideration, equal to the weight tributary to the diaphragm.

Method 3 is appropriate for systems with rigid vertical elements and flexible diaphragms in which the dynamic response of the system is concentrated in the diaphragm. Use of Method 2 on these systems to calculate the period based on the stiffness of the vertical elements will substantially underestimate the period of actual dynamic response and overestimate the pseudo lateral load.. Eqn 4E(7) is a special case developed specifically for URM buildings. In this method, wall deformations are assumed negligible compared to diaphragm deflections. For illustration of wall and diaphragm displacements see Figure 4E.2. When calculating diaphragm displacements for the purpose of estimating period using eqns 4E(6) or 4E(7), the diaphragm should be considered to remain elastic under the prescribed lateral loads.

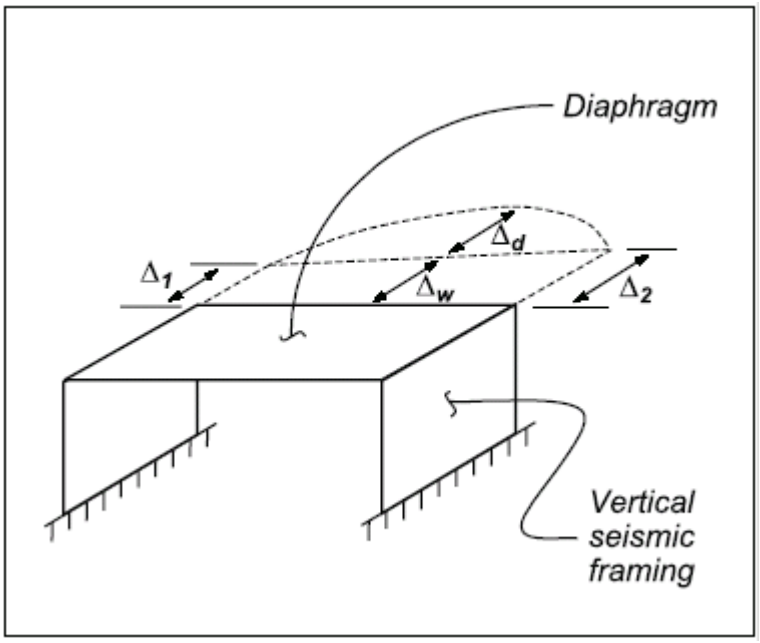


Figure 4E.2 Diaphragm and wall displacement terminology

4E.8.2 Pseudo Lateral Load

The pseudo lateral load in a given horizontal direction can be determined from eqn 4E(8). This load is applied to the vertical elements of the lateral force resisting system.

$$V = C_1\,C_2\,C_3\,C_m\,S_a\,W_t \qquad \text{...4E(8)}$$

where:

V = Pseudo lateral load

C_1 = Modification factor to relate expected maximum inelastic displacements to those calculated for linear elastic response. Values suggested in FEMA 356 are:

$C_1 = 1.5$ for $T < 0.10$ second.
$C_2 = 1.0$ for $T \geq T_s$ second.

Linear interpolation may be used to calculate C_1 for intermediate values of T.

T = The fundamental period of the building in the direction under consideration, calculated as in Section 8.1 herein.

T_s = The characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

C_2 = Modification factor to represent the effects of pinched hysteresis shape, stiffness degradation, and strength deterioration on the maximum displacement response. C_2 should be taken as 1.0 for the case of linear elastic analysis.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects listed in Section 4B.6 herein. For values of the stability index, 2_i , (see Section 6.5 of NZS 1170.5:2004), less than 0.1 in all stories, C_3 shall be taken as 1 + 5(2 - 0.1)/T using 2 equal to the maximum value of 2_i of all stories.

C_m = Effective mass factor to account for higher mode mass participation effects and can be taken as 1.0 for one and two storey structures, or if the fundamental period, T, is greater than 1.0 seconds. In the case of steel or concrete buildings of three or more stories, a value of 0.9 can be used for C_m .

S_a = Response spectrum acceleration at the fundamental period and damping ratio of the building in the direction being considered and taken from Section 3 of NZS 1170.5:2004.

W_t = The effective seismic weight of the building.

Coefficient C_1. *This modification factor is to account for the difference in maximum elastic and inelastic displacement amplitudes in structures with relatively stable and full hysteretic loops. The values of the coefficient are based on analytical and experimental investigations of the earthquake response of yielding structures. See FEMA 356, Section 3.3.3.3 for further discussion.*

Coefficient C_2. *This coefficient adjusts design values based on component hysteresis characteristics, stiffness degradation, and strength deterioration. See FEMA 274 for additional discussion.*

Coefficient C_3. *For framing systems that exhibit negative post-yield stiffness, dynamic P-Δ effects may lead to significant amplification of displacements. Such effects cannot be explicitly addressed with linear procedures. No measure of the degree of negative post-yield stiffness can be explicitly included in a linear procedure.*

4E.8.3 Vertical Distribution of Seismic Forces

The vertical distribution of the pseudo lateral load should be as specified in this section for all buildings except unreinforced masonry buildings for which the pseudo lateral loads should be distributed as setout below. The lateral load Fx applied at any floor level x should be determined in accordance with Eqn 4E(8) and Eqn 4E(10):

$$F_x = C_{vx}V \qquad \text{…4E(9)}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \ldots 4E(10)$$

where:

C_{vx} = Vertical distribution factor
k = 2.0 for T ≥ 2.5 seconds
= 1.0 for T ≤ 0.5 seconds
Linear interpolation shall be used to calculate values of k for intermediate values of T.
V = Pseudo lateral load
w_i = Portion of the total building weight W located on or assigned to floor level i
w_x = Portion of the total building weight W located on or assigned to floor level x
h_i = Height (in m) from the base to floor level i
h_x = Height (in m) from the base to floor level x

For unreinforced masonry buildings with flexible diaphragms for which the fundamental period is calculated using Eqn 4E(10), the pseudo lateral loads can be calculated and distributed as follows:

1. For each span of the building and at each level, calculate period
2. Calculate pseudo lateral load for each span.
3. Apply the lateral loads calculated for all spans and calculate forces in vertical seismic-resisting elements using tributary loads.
4. Diaphragm forces for evaluation of diaphragms are determined from the results of step 3 above and distributed along the diaphragm span considering its deflected shape.
5. Diaphragm deflection should not exceed 300 mm for this method of distribution of pseudo lateral loads to be applicable.

4E.8.4 Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building should be distributed according to the distribution of mass at that floor level.

4E.8.5 Diaphragms

Diaphragms should be designed to resist the combined effects of the inertial force, F_{px}, calculated in accordance with eqn 4E(11), and horizontal forces resulting from offsets in or changes in the stiffness of the vertical seismic framing elements above and below the diaphragm. Forces resulting from offsets in or changes in the stiffness of the vertical seismic framing elements should be taken as the forces due to the pseudo lateral load without reduction, unless smaller forces are justified by a limit-state or other rational analysis, and should be added directly to the diaphragm inertial forces.

$$F_{px} = \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \qquad \ldots 4E(11)$$

where:

F_{px} = Total diaphragm inertial force at level x
F_i = Lateral load applied at floor level i given
w_i = Portion of the effective seismic weight W located on or assigned to floor level i
w_x = Portion of the effective seismic weight W located on or assigned to floor level x

The seismic load on each flexible diaphragm is then distributed along the span of that diaphragm, proportional to its displaced shape.

4E.9 Modal Response Spectrum Analysis

The horizontal ground motion should be either a response spectrum taken from Section 3 of NZS 1170.5:2004, or else a response spectrum determined by a site-specific investigation.

Modal spectral analysis is carried out using linearly elastic response spectra that are not modified to account for anticipated nonlinear response. It is expected that the method will produce displacements that approximate maximum displacements expected during the design earthquake, but will produce internal forces that exceed those that would be obtained in a yielding building. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements

4E.9.1 Response Spectrum Method

Should be carried out in accordance with Clause 6.3 of NZS 1170.5:2004.

4E.10 Simple Lateral Mechanism Analysis (SLaMA)

A hand analysis is carried out to determine the likely collapse mechanism and its lateral strength and displacement capacity. This is then compared to the earthquake demand on the structure determined using either a force- or displacement-based method. The following sets out a possible SLaMA procedure for a framed building.

4E.10.1 Lateral frame capacities

For each lateral frame (with or without walls):

1. Calculate the beam gravity moments, M_{BG}, and the gravity shear forces, V_{BG} (approximately).

2. Calculate the column and wall gravity loads, N_G.

3. Determine the beam moment capacities, MBN. Where the reinforcing comprises smooth bars, assume both top and bottom reinforcement is in tension regardless of the position of the neutral axis.

4. Determine the beam shears at the moment capacities as illustrated in Figure 4E.3.

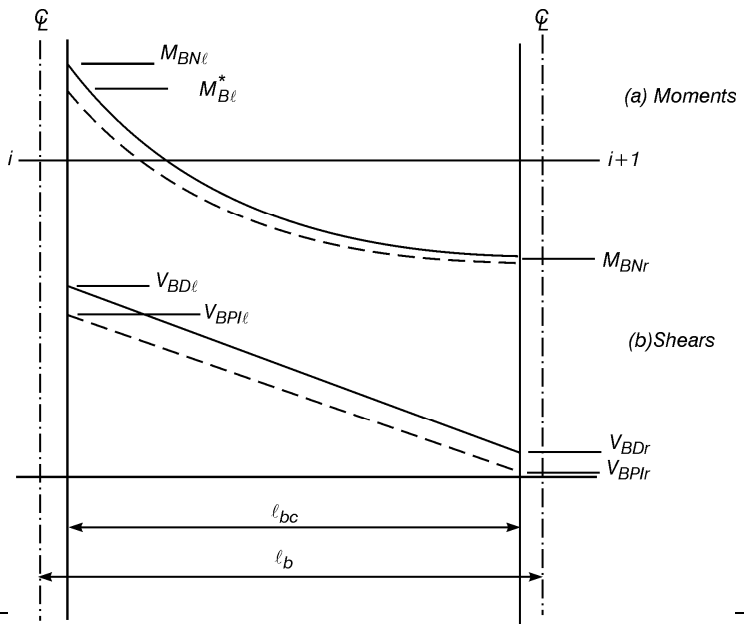


Figure 4E.3 Beam shears

i.e. $V_{BDl} = V_{BGl} + V_{BEl} = V_{BGl} + (M_{BNl} + M_{BNr})/l_{bc}$ 4E(12)

5. Determine the initial probable beam shear capacity, V_{BPI}, using eqn 7(5).

6. Check the initial beam shear strength to determine whether it is greater than the beam shear, V_{BD}, at the beam moment capacity. If $V_{BPI} > V_{BD}$, then reduce the effective beam moment capacity to (se Fig. 4E.3):

$$M^*_{Bl} = (V_{BPIl} - V_{BGl})l_{bc} - M_{BNr} \quad 4E(13)$$

7. Check the beam/column joint capacity demand as follows:

   (a) assume the top beam forms beam 'hinges' based on the moments, M_{BN}, from Step 3 or the reduced moments , M_B, from Step 6, i.e. Equation 4E(13).

   (b) Determine the joint shear strength using Equation 7(11), and the principal tensile stress, $p_t = k\sqrt{f_c}$'

   (c) If the joint capacity demand is too high, the beam moment capacity will need to be reduced.

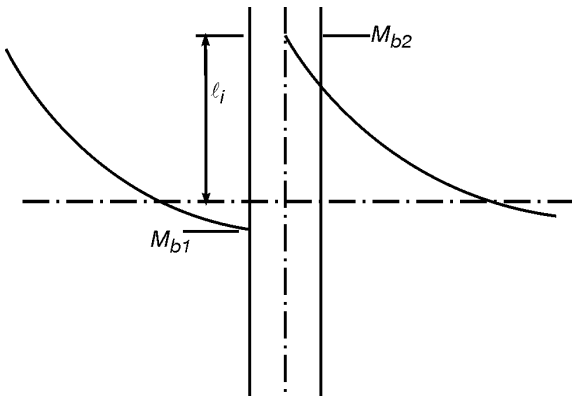


Figure 4E.4 Beam hinges

$$V_{ij} \approx \frac{(M_{b1} + M_{b2})}{0.9\,h_b} - V_{col}$$

$$= \frac{\sum M_b}{0.9\,h_b} - V_{col}$$

where h_b is the beam depth

$$\therefore V_{col} \approx 0.5 \sum M_b \frac{l_b}{l_{bc}} \frac{1}{l_c}$$

$$\approx \frac{1.2 \sum M_b}{l_c}$$

where l_b = beam length
l_{bc} = clear beam length
l_c = column height, between beam centrelines

$$\therefore V_{jh} \approx \frac{1.1\sum M_b}{h_b} - \frac{1.2\sum M_b}{l_c}$$

$$= \sum M_b \left(\frac{1.1 l_c - 1.2 h_b}{h_b l_c} \right) \qquad \text{4E(14)}$$

$$\therefore \sum M_b = (M_{b1} + M_{b2}) = V_{jhc} \left[\frac{h_b l_c}{1.1 l_c - 1.2 h_b} \right]$$

where $V_{jhc} = p_t \sqrt{1 + \frac{N^*}{A_g p_t}}$

(d) Determine the column seismic axial forces, N_E*, below the beam arising from the seismic beam shears using the reduced beam moments (if necessary).

(e) Repeat (a) to (d) for each floor level down to the lowest level.

8. Determine the column shears and check the column shear demand/capacity.

(a) Using values of N* from Step 7 above (seismic plus gravity), calculate the column shear strength, V_{CPI}, using Equation 7(6).

(b) Calculate the column flexural strength under N*.

(c) Check whether the footings will rock or not. If they will, then reduce the column base moment capacity.

(d) Check the joint sway potential.

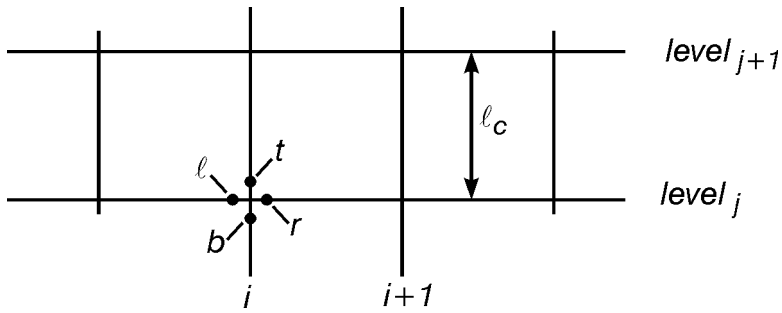


Figure 4E.5 Sway potential

The sway potential at the joint on column i at level j (see Fig. 4E.5) is given by

$$S_{Pij} = \frac{M_{bijl} + M_{bijr}}{M_{cijt} + M_{cijb}}$$

based on the full moment capacity at the joint centroid. If $S_{Pij} > 0.85$, assume that the column hinges at t and/or b.

(e) The column shear demand is given by:

$$V_{CD} = w_V \frac{\left(M_{bijl} + M_{bijr} + M_{bij+1,l} + M_{bij+1,r}\right)}{2kl_c}$$
$$\leq \frac{\left(M_{cijt} + M_{cij+1,b}\right)}{kl_c}$$

At the column base, use M_{CiO} instead of the beam moments.

(f) Check the initial column shear failure, i.e. is $V_{CPI} > V_{CD}$?

If the check is satisfactory, go to the next frame.
If $V_{CPI} < V_{CD}$, then the column is likely to fail in a brittle manner. In this case, μ_s = 1, and the beam moments and N* must be reduced proportionally.

(g) Check the next frame.

4E.10.2 Check the storey sway potential at each level.

1. Determine the storey sway potential for each frame where

$$S^*_{pjk} = \frac{\sum_i \sum_k \left(M_{bijkl} + M_{bijkr}\right)}{\sum_i \sum_k \left(M_{cijkt} + M_{cijkb}\right)}$$

where i = the column number
j = the storey number, and
k = the frame number

The beam and column moments are those extrapolated to the joint centroid.

2. Check whether S_{pjk}* > 0.85 k.
If it does, then sway potential exists.

3. Check possible sway mechanisms as illustrated in Fig. 4E.6.

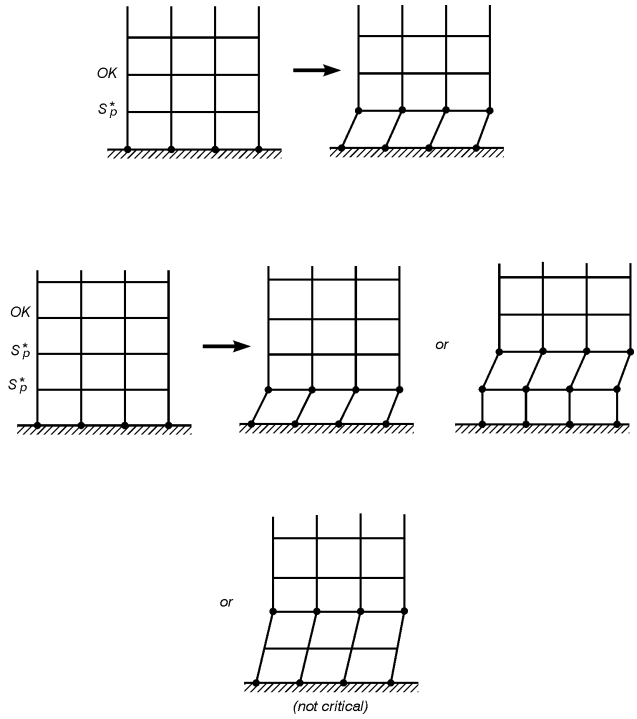


Figure 4E.6 Mechanisms

4E.10.3 Force-based Assessment of Demand

1. Calculate the overturning moment capacity of each frame in the structure (see Fig. 4E.7).

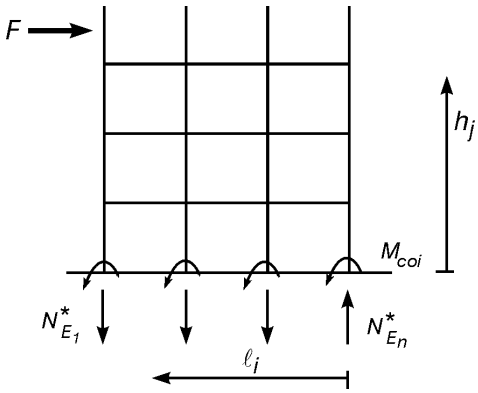


Figure 4E.7 Overturning capacity

Note: determine OTM^1 for unreduced beam moments, or
OTM^2 for beam moments reduced for ultimate joint shear, or
OTM^3 for beam moments reduced for the collapse mechanism.

2. Calculate the overturning moment capacity of the whole structure as:

$$V_{CD} = w_V \frac{\left(M_{bijl} + M_{bijr} + M_{bij+1,l} + M_{bij+1,r}\right)}{2kl_c}$$

$$\leq \frac{\left(M_{cijt} + M_{cij+1,b}\right)}{kl_c}$$

$$\text{Total}\, OTM = \sum_k OTM_k \quad \text{for k frames}$$

3. Determine the height of the lateral force resultant from

$$h_{eff} = \sum m_j h_j^2 \Big/ \sum m_j h_j$$

where m_j = mass at storey j.

$$OTM_n = \sum_i M_{coi} + \sum N_{Ei} l_i$$

4. The base shear capacity can be determined from

$$V_B = OTM / h_{\text{eff}}$$

5. The yield displacement, μ_y, is given by

$$\Delta_y = \left[0.5\varepsilon_y \frac{l_b}{h_b} h_{eff} \right] \frac{OTM_1}{OTM_2}$$

where l_b = full beam length (see Fig. 4E.3) and d_b is the beam depth.

6. Calculate the frame ultimate displacement capacity for the assessed yield mechanism as given in Figure 4E.8.

7. Determine whether the structure is torsionally eccentric.
   (a) If it is, then determine the strength eccentricity. With reference to Figure 4E.9.

$$\bar{y} = \sum V_{Bk}\, y_k \Big/ \sum V_{Bk}$$

$$e = \bar{y} - y_{mass\ centre}$$

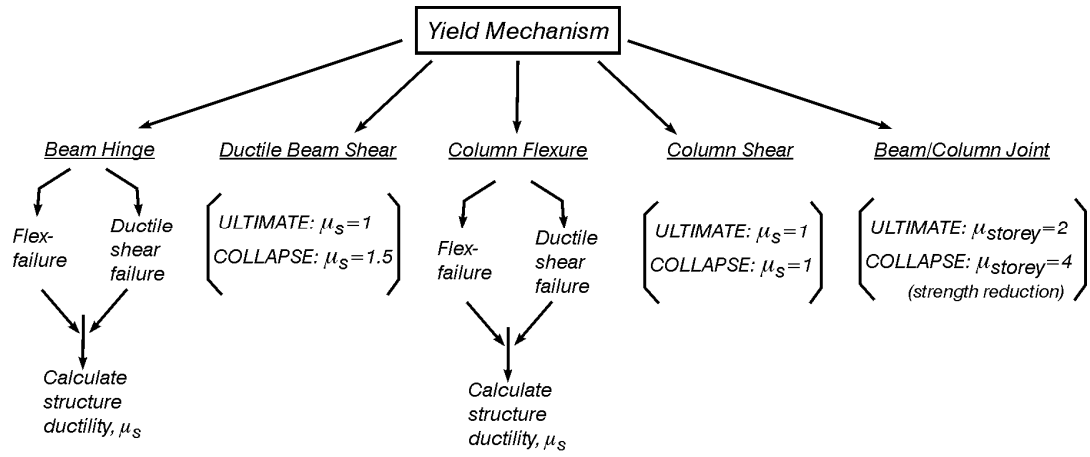


Figure 4E.8 Frame ultimate displacement capacity

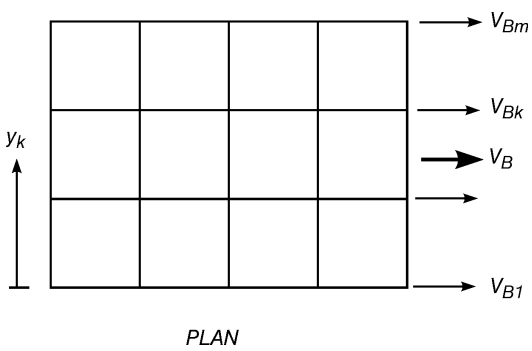


Figure 4E.9 Strength eccentricity

8. Determine which frame is subjected to the critical ultimate displacement.

9. Taking twist into account, determine the ultimate displacement, μ_u, at the centre of mass (or the displacement, μ_c, at collapse).

10. The structure displacement capacity is then given by

$$\mu_{sc} = \Delta_u \big/ \Delta_y$$

11. Determine the elastic stiffness, K_e, where

$$K_e = V_B / \Delta_y$$

12. Determine the effective mass, M_e, from

$$M_e = \frac{\sum m_i h_i}{h_e}$$

Also check the situation where the effective mass in the first mode is less than 100%.

13. Determine the elastic period, T, as

$$T = 2\pi\sqrt{\frac{M_e}{K_e}}$$

14. The ductility demend, μ_{SD}, can be determined from (V_B / M_e)and the spectrum.

15. The (*%NBS*) is given by:

$$(\%NBS) = \frac{\mu_{SC}}{\mu_{SD}}$$

4E.10.4 Displacement-based Assessment of Demand

1. Determine the overturning moment for each frame of the structure as for the force-based assessment (FBA).

2. Determine the overturning moment for the structure as for the FBA

3. Determine the ultimate displacement profile for each frame.

4. Determine the effective height as:

$$h_{eff} = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i}$$

5. Determine the base shear capacity, V_B, as for FBA.

6. Determine the yield displacement, Δ_y, as for FBA.

7. The structure ultimate displacement capacity, Δ_{UC}, can be determined as in steps 7-10 for FBA.

8. The effective mass is determined by

$$M = \frac{\sum m_i \Delta_i}{\Delta_{UC}}$$

Check the situation where the effective mass is less than 100% in the first mode.

9. The effective stiffness is:

$$K_e = V_B / \Delta_{UC}$$

10. The effective damping, ξ_{eff}, needs to be determined for the particular μ_{SC} (= μ_u / μ_y) using Equation 6(3).

11. Calculate the effective period as in step 13 of the FBA.

12. Calculate the displacement demand, μ_{UD}, from the displacement spectrum and the effective damping.

13. Calculate the (*%NBS*) as

$$(\%NBS) = \frac{\mu_{UC}}{\mu_{UD}}$$

4E.11 Lateral Pushover Analysis

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures should comply with the requirements of Section 4E.11.1

The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. A method for determining suitable target displacements is described in Section 3.3.3.3 of FEMA 356 (2000).

4E.11.1 Modelling and Analysis Considerations

The selection of a control node, the selection of lateral load patterns, the determination of the fundamental period, and analysis procedures should comply with the requirements of this section.

The relation between base shear force and lateral displacement of the control node should be established for control node displacements ranging between zero and 150% of the target displacement, Δ_t.

The component gravity loads should be included in the mathematical model for combination with lateral loads as specified in AS/NZS 1170.0. The lateral loads should be applied in both the positive and negative directions, and the maximum seismic effects should be used for design.

The analysis model is discretised to represent the load-deformation response of each component along its length to identify locations of inelastic action. All primary and secondary lateral-force-resisting elements should be included in the model.

The force-displacement behavior of all components can be explicitly included in the model using full backbone curves that include strength degradation and residual strength, if any.

Alternatively, a simplified analysis can be used. In such an analysis, only primary lateral force resisting elements are modeled, the force-displacement characteristics of such elements are bilinear, and the degrading portion of the backbone curve is not explicitly modeled. Elements not meeting the acceptance criteria for primary components are designated as secondary, and removed from the mathematical model.

When using the simplified analysis, care should be taken to make sure that removal of degraded elements from the model does not result changes in the regularity of the structure that would significantly alter the dynamic response. In pushing with a static load pattern, the simplified analysis does not capture changes in the dynamic characteristics of the structure as yielding and degradation take place.

In order to explicitly evaluate deformation demands on secondary elements that are to be excluded from the model, one might consider including them in the model, but with negligible stiffness, to obtain deformations demands without significantly affecting the overall response.

4E.11.2 Control Node Displacement

The control node should be located at the center of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse should be regarded as the level of the control node. The displacement of the control node in the mathematical model should be determined for the specified lateral loads.

4E.11.3 Lateral Load Distribution

Lateral loads are applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load should be applied. One pattern shall be selected from each of the following two groups:

1. A modal pattern selected from one of the following:

   a) A vertical distribution proportional to the values of C_{vx} given in eqn 4E(10). Use of this distribution should be used only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is also used.
   b) A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution should be used only when more than 75% of the total mass participates in this mode.
   c) A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum. This distribution should be used when the period of the fundamental mode exceeds 1.0 second.

2. A second pattern selected from one of the following:

   a) A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.
   b) An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution should be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution will depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure. Use of more than one lateral load pattern is intended to bound the range of design actions that may occur during actual dynamic response.

In lieu of using the uniform distribution to bound the solution, changes in the distribution of lateral inertial forces can be investigated using adaptive load patterns that change as the structure is displaced to larger amplitudes. Procedures for developing adaptive load patterns include the use of story forces proportional to the deflected shape of the structure (Fajfar and Fischinger), the use of load patterns based on mode shapes derived from secant stiffnesses at each load step (Eberhard and Sozen), and the use of load patterns proportional to the story shear resistance at each step (Bracci et al.). Use of an adaptive load pattern will require more analysis effort, but may yield results that are more consistent with the characteristics of the building under consideration.

4E.12 Inelastic Time History Analysis

Where an inelastic time history analysis carried out for the seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building should be subjected to earthquake shaking represented by ground motion time histories in accordance with Clause 6.4 of NZS 1170.5:2004 to obtain forces and displacements.

The calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis should be carried out with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Appendix 8A: Bolted and Riveted Joint Moment-Rotation Determination

8A.1 Clip angle type connections

A comprehensive procedure for evaluating the nominal moment capacity and rotation available from riveted or early bolted steel connections is given in (Roeder et al 1996). This procedure is applicable for beam to column connections formed with either tee-stub or clip angle connections between beam flange and column flange, as shown in Fig. 1 of (Roeder et al. 1996).

The procedure includes a method for calculating the effective yield moment for a riveted connection, along with expressions for the rotational capacity at maximum strength of the connection, (ie. the rotation limit above which the moment capacity falls significantly below that given by calculated nominal yield moment. Both yield moment and degradation threshold are a function of the expected mode of failure of the connection to the beam flanges. Roeder et al (1996). require three modes of failure to be checked for the critical case, ie.:

(1) Tensile failure of the stem or outstanding leg (OSL) of the angle or tee section connection onto the supported beam flange.
(2) Shear yielding/failure of the connectors, and
(3) Flexural yielding of the leg(s) of the angle or tee-stem connecting onto the supporting column flange.

The failure mode giving the least capacity of these three becomes the failure mode for the connection, in terms of this evaluation.

Most older riveted or bolted beam to column joints in New Zealand have used clip angles, as shown in Fig. 8A.1. A simplified procedure for calculating the yield moment and the moment-rotation characteristics is given below.

This procedure is based around the critical failure mode being that associated with flexural yielding of the legs of the angle or tee-section connecting onto the column. The first two failure modes need to also be assessed and only when the third failure mode is shown to govern can the procedure given in this simplified section be used.

If either tensile failure or shear yielding /failure of the connectors governs, then use the procedure in Section 8A.2.

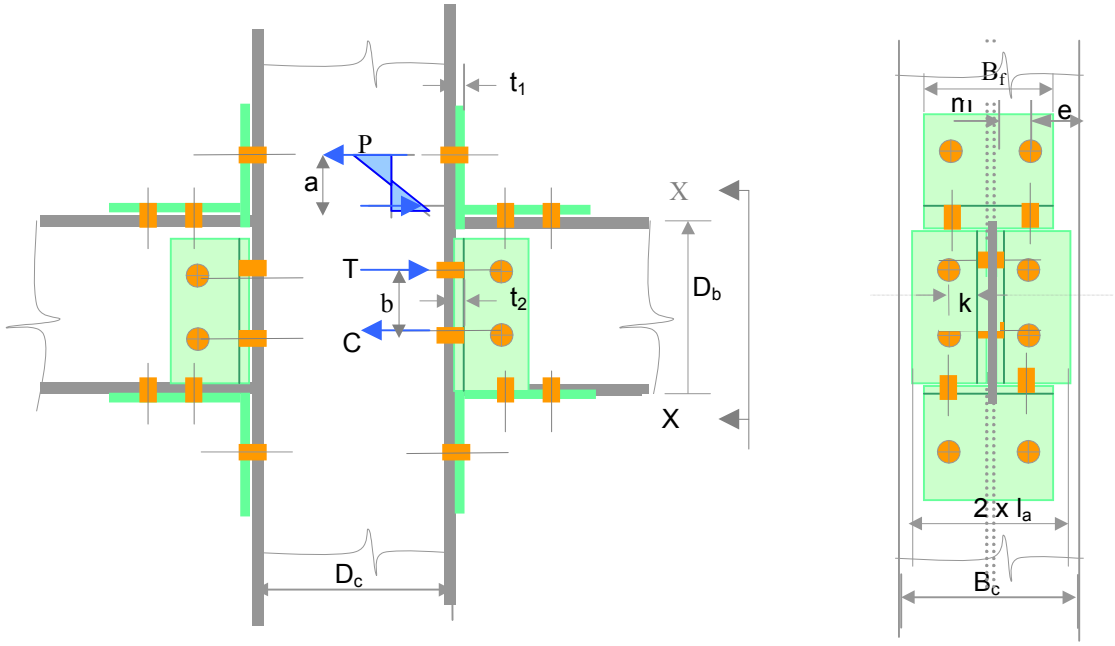


Figure 8A.1: Joint detail

Bending moment capacity of flange cleat angle:

$$M_f = \frac{B_f t_1^2}{4} f_{ya} \qquad \text{…8A(1)}$$

where B_f is minimum of (beam flange width; angle length), t_1 is thickness of flange cleat angle leg, and f_{ya} is design yield strength of the angle section.

From eqn 8A(1), tensile force in the flange cleat bolts/rivets:

$$P = \frac{2M_f}{a} \qquad \text{…8A(2)}$$

where a is the distance between bolt centreline to the flange cleat angle leg.

Bending moment capacity of web cleat angle:

$$M_w = \frac{2 l_a t_2^2}{4} f_{ya} \qquad \text{…8A(3)}$$

where l_a is the length of web cleat angle face and t_2 is thickness of web cleat angle leg.

From eqn 8A(3), tensile force in the web cleat bolts/rivets:

$$T = \frac{2M_w}{k} \qquad \text{…8A(4)}$$

where k is the distance between bolt centreline to the web cleat angle leg.

Tension strength of the column flange:

$$T_c = (4m + 1.25e) t_c f_{yc} \qquad \text{…8A(5)}$$

where m is the distance from centre of bolt hole to radius root at web, e is distance from rivet centre to flange edge, and t_c is thickness of the column flange and f_{yc} is the yield stress of the column flange.

Yield moment capacity of the joint is:

$$M_y = PD_b + Qb \qquad \text{…8A(6)}$$

where Q is either T from eqn 8A(4) or T_c from eqn 8A(5), whichever is less, and b is the distance between the centroid of tension and compression forces in the web cleat.

8A.1.1 Moment – rotation behaviour

Figure 8A.2 shows the proposed moment–rotation behaviour of riveted clip angle/T-stub connection based on Roeder et al experimental studies on seismic resistance on older steel structures at the University of Washington and University of Minnesota (Roeder et al 1994).

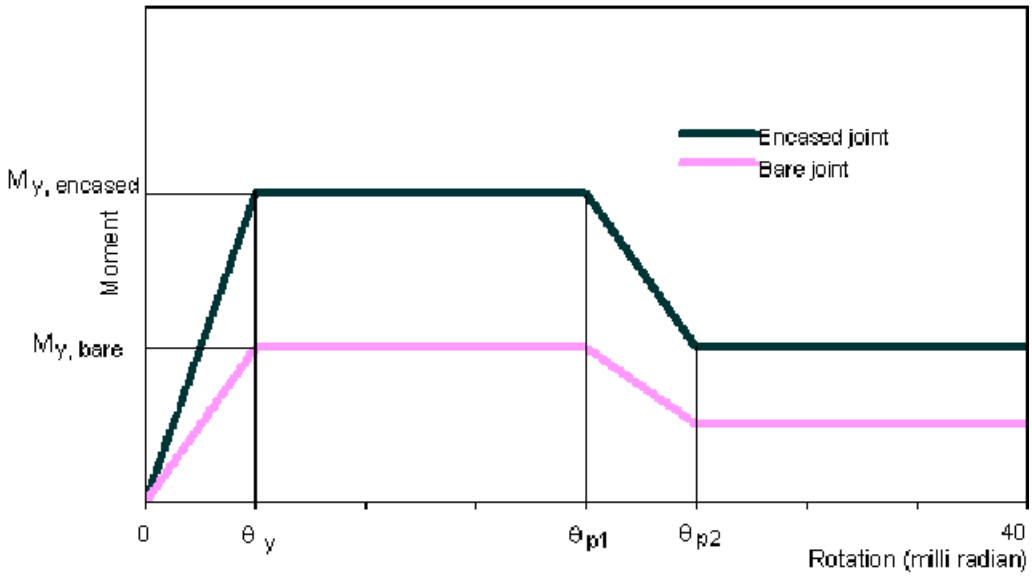


Figure 8A.2: Moment–rotation curve

In Fig. 8A.2

θ_y = 5 milliradians, for a clip angle type connection

$$\theta_{p1} = \frac{12.5}{d_b} \text{ milliradians} \qquad \text{…8A(7)}$$

d_b = depth of beam, in metres

$$\theta_{p2} = (\theta_{p1} + 5) \text{ milliradians} \qquad \text{…8A(8)}$$

$M_{y,bare}$ = as given by eqn 8A(6) for a bare steel connection

$$M_{y,encased} = 2\, M_{y,bare} \text{ for a clip angle type connection} \qquad \text{…8A(9)}$$

When the joint is rotated from θ_{p1} to θ_{p2} , the moment reduces by a factor of 0.5 and then remains constant up to θ = 40 milliradians, after which zero moment capacity is assumed.

In regard to the above:

- θ_y = 5 milliradians is an appropriate rotation at first yield for this pre-1975 building connection
- eqn 8A(7) is from (Roeder et al. 1996), for connections with flexural yield of connecting elements
- the experimental tests undertaken show that the degradation in moment capacity occurs over a rotation of approx. 5 milliradians, hence this is the difference used between θ_{p1} and θ_{p2} .
- the enhancement factor for $M_{y,encased}$ compared with $M_{y,bare}$ is that recommended by (Roeder et al. 1996) for this, the most flexible form of semi-rigid connection.

8A.1.2 Joint deterioration

The joints tested by Roeder, both concrete encased and bare joints generally experienced degradation at rotation 20–25 milliradians. It was also observed that the concrete encased composite joint had a better performance over the bare joints. The concrete encasement prevented any local deformation of the joint until the concrete crushed when the joint capacity deteriorates to that of a bare joint. The enhancement provided by the composite action of concrete encasement and floor slabs to connection capacity was found to be substantial and in the range of 30–100% increase to that of bare joint moment capacity. The higher increase of capacity was noted in the weaker joints such as clip angles.

In bare joints without concrete encasement the joint capacity deteriorated significantly when the clip angle to the beam flange failed but the capacity did not drop to zero because of the resistance provided by the web cleat angle connection.

8A.1.3 Background to Roeder's experiments

Roeder et al (1994, 1996) focused their experimental work on issues that were not addressed previously by researchers in determining the seismic resistance of older steel building. Some of the key objectives of their work were:

- to study the cyclic behaviour of these older steel structures considering the change in stiffness at large inelastic deformation. The past research work were primarily under monotonic loading.
- to study the effect of concrete encasement provided for fire resistance on connection stiffness, strength, and ductility
- to understand the effect of rivets on seismic behaviour of joints
- to develop a model to establish the strength, stiffness, and ductility of these older steel structures based on their experiments.

The research work was a joint effort between the University of Washington, the University of Minnesota, and Preece/Goudie & Associates. As part of the testing programme they tested 23 large-scale specimens including bare steel and encased joints with clip angle, T-stub, and stiffened seat connections.
The main findings of the research were:

1 The hysteretic behaviour of the connections was relatively poor but the connections often were able to sustain large deformations. They behaved as partially restrained connections. Clip angle connections were generally weaker and more flexible than the other connections.

2 Concrete encasement significantly increased the strength and stiffness of weaker and more flexible joints such as clip angle connections and modestly increased for stiffer and stronger connections. See Figure 8A.3 taken from Roeder (1994).

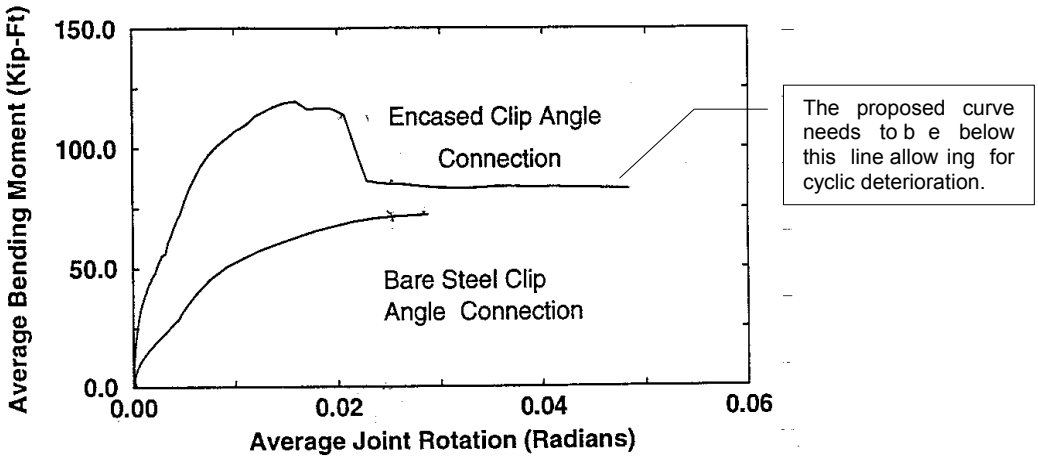


Figure 8A.3: Comparison of bare steel and encased moment–rotation behaviour

3 The tests showed that mode of failure for the cyclic loading was very similar to the monotonic loading. Both monotonic and cyclic load tests deteriorate or fail at very similar deformations as shown in Figure 8A.4. The monotonic tests typically provided an upper bound envelope for the cyclic tests. T-stub and clip angle connections for both bare steel and encased connections displayed this behaviour.

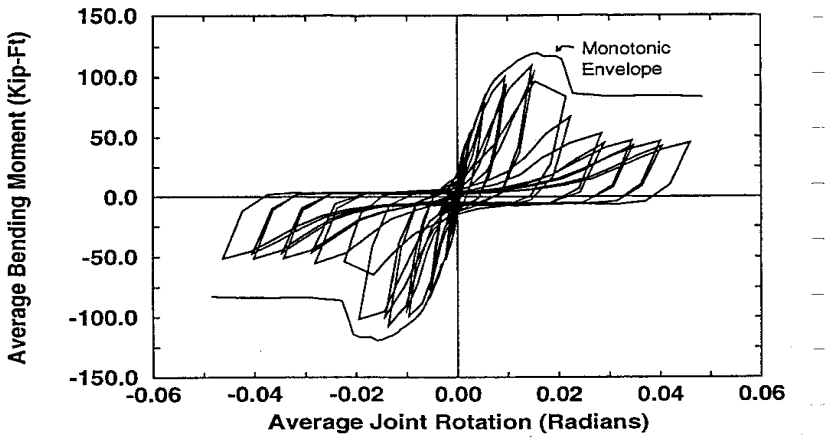


Figure 8A.4: Comparison of monotonic and cyclic moment–rotation behaviour

4 All connectors failed at almost the identical deformation for both bare steel and encased connection. However, the initial failure of these connectors did not result in a complete loss of the resistance of the connection. See the moment rotation behaviour in Figure 8A.2. Considerable resistance was provided by the web angles and composite action provided by the concrete encasement even after the initial failure.

The above experimental studies were on riveted connections. It should be noted that bolted connections would be stiffer and have more rotational capacity than the comparable riveted connections. However, the limits on the overall system inelastic displacement would be such that the bolted connections cannot attain its full capacity. For example, when the connection is the

weakest element, then the connection rotation will be around 30 milliradians maximum for a frame displacement of 2.5% of the interstorey height. Thus the 40 milliradians limit on rotation is a practical upper limit for the system as a whole, even if the individual joint is capable of greater rotations while maintaining a dependable level of moment capacity.

8A.2 Other bolted and riveted connections

For bolted and riveted connections in general - especially other than clip angle connections of the form shown by Fig. 8A.1 – use the procedure from (Roeder et al., 1996) to determine the moment capacity M_y. (This is termed M_u in that paper). This involves using the seven step procedure on pages 370 and 371 of that paper.

The moment-rotation curve is then constructed in a similar manner to Fig. 8A.2, using the following key values for rotation and moment:

θ_y = 5 milliradians for clip angle connections
= 3 milliradians for tee stub connections

θ_{p1} = $\frac{3.75}{d_b}$ milliradians, for failure mode being tensile yielding of the stem of the tee stub or clip angle connected to the beam flange …8A(10)

θ_{p1} = $\frac{7.5}{d_b}$ milliradians, for failure mode being shear yielding of the connectors …8A(11)

θ_{p1} = as given by eqn 8A(7), for failure mode being flexural yielding of the connecting elements

θ_{p2} = $(\theta_{p1} + 5)$ milliradians …8A(12)

$M_{y,\,bare}$ = as given by (Roeder et al., 1996)

$M_{y,\,encased}$ = $C_1 M_{y,\,bare}$ …8A(13)

C_1 = 1.3 for a tee-stub type connection
= 2.0 for a clip angle type connection

Appendix 8B: Simplified Pushover Analysis for Use in the Evaluation

The analysis must have the capability to take into account the P-Δ action by large displacement analysis and the modelling of joint elastic springs in the system.

1 Take the force vector from Section 4.9.7(d) and assign a unit Load Factor (LF) to it.

2 Increase the LF until past the yield moment (M_{yield}) in approximately one-quarter of the joints on any level.

3 Reduce the joint elastic stiffness on that level to the first inelastic value, that is, as shown on Figure 8B.1 and reapply loads using LF_{max} from Step 2.

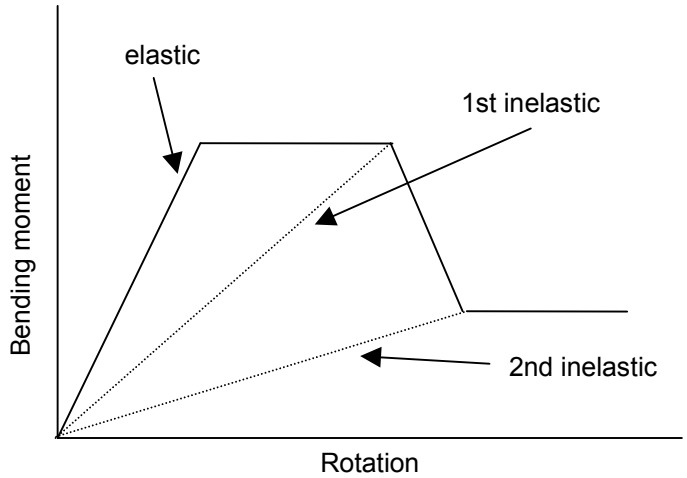


Figure 8B.1: Moment–rotation curve for riveted clip-angle/T – stub connection

4 Check all levels to see if M_{yield} is exceeded in approximately one-quarter of the joints. If so, reduce the joint elastic stiffness in all joints on that level and reanalyse. Keep the top one-third (or three) joints elastic throughout to model the concentration of demand in lower levels.

5 Check the rotation in the joints at the lower levels. If > 20 milliradians, then reduce the joint stiffness to the 2nd inelastic level and reanalyse. Reduce LF if necessary to keep within the deflection limits if these limits are exceeded when the joint stiffness on a given layer is reduced to the second inelastic level.

6 When the deflection limit is attained, check if $LF \geq 0.8\ LF_{max}$.

Appendix 10A: Derivation of Instability Deflection and Fundamental Period for Masonry Buildings

10A.1 General considerations and approximations

It should be appreciated that there are many variations that need to be taken into account in considering a general formulation for unreinforced masonry walls that might fail out-of-plane. Among these considerations are the following.

- Walls will not in general be of constant thickness in a building, or even within a storey.
- Walls will have embellishments, appendages and ornamentation that may lead to eccentricity of masses with respect to supports.
- Walls may have openings for windows or doors.
- Support conditions will vary.
- Existing building may be rather flexible, leading to possibly large inter-storey displacements that may adversely affect the performance of face-loaded walls.

To simplify the analysis while taking into account important factors, the following are the approximations that are employed.

1 Deformations due to distortions (straining) in the wall are ignored. Deflections are assumed to be entirely due to rigid body motion.

This is equivalent to saying that the change in potential energy due to a disturbance of the wall from its initial position is due mostly to the movement of the masses of the elements comprising the wall and the movements of the masses tributary to the wall. Strain energy contributes less to the change in potential energy.

2 It is assumed that potential rocking occurs at the support lines (at roof or floor levels, for example) and, for walls that are supported at the top and bottom of a storey, at the mid-height. The mid-height rocking position divides the wall into two parts of equal height, a bottom part (subscript b) and a top part (subscript t). The masses of each part are not necessarily equal.

It is implicit within this assumption and that in (1) above, that the two parts of the wall remain undistorted when the wall deflects. For walls constructed of softer mortars or for walls where there is little vertical prestress from storeys above, this is not actually what occurs—the wall takes up a curved shape, more particularly in the upper part. Nevertheless, the errors that occur from the use of the stated assumptions have been found to be small and acceptably accurate results are still obtained.

3 The thickness is assumed to be small relative to the height of the wall, and the slope, A, of both halves of the wall is assumed to be small, in the sense that $cos(A) \approx 1$ and $sin(A) \approx A$.

The approximations for slope are likely to be sufficiently accurate for reasonably thin walls. For thick walls where the height to thickness ratio is smaller, the formulations that are developed in this appendix are likely to provide less accurate results. However, for walls of this kind force-based approaches provide an alternative.

4 Inter-storey slopes due to deflection of the building are assumed to be small.

Approximate corrections for this effect are noted in the method.

5 In dynamic analyses, the moment of inertia is assumed constant and equal to that applying when the wall is in its undisturbed position, whatever the axes of rotation.

It should be appreciated that the moment of inertia is dependent on the axes of rotation. During excitation the axes continually change position. The approximation assumes that the inertia is constant. Within the context of other approximations employed, this is reasonable.

6 Damping is assumed at the default value in NZS 1170.5:2004 (or NZS 4203:1992), which is 5% of critical.

For the aspect ratio of walls of interest, additional effective damping due to loss of energy on impact is small. Furthermore it has been found that the surfaces at rocking (or hinge) lines tend to fold onto each other rather than experience the full impact that is theoretically possible, reducing the amount of equivalent damping that might be expected. However, for in-plane analysis of buildings constructed largely of unreinforced masonry, adoption of a damping ratio that is significantly greater than 5% is appropriate.

7 It is assumed that all walls in storeys above and below the wall under study move "in phase" with the subject wall.

This is found to be the case in analytical studies. One reason for this is that the effective stiffness of a wall as it moves close to its limit deflection (as measured by its period, for example) becomes very low, affecting its resistance to further deflection caused by accelerations transmitted to the walls through the supports. This assumption means that upper walls, for example, will tend to restrain the subject wall by exerting restraining moments.

2 | 10A.2 One-way vertically spanning face-loaded walls

10A.2.1 General formulation

Figures 10A.1 and 10A.2 show the configuration of a wall panel within a storey at two stages of deflection. The wall is intended to be quite general. Simplifications to the general solutions for walls that are simpler (e.g. of uniform thickness) are made in a later section.

Figure 10A.1 shows the configuration at incipient rocking. Figure 10A.2 shows the configuration after significant rocking has occurred, with the wall having rotated through an angle A and with mid-height deflection Δ, where $\Delta = Ah/2$.

In Figure 10A.1 the dimensions e_b and e_t relate to the mass centroids of the upper and lower parts of the panel. e_p relates to the position of the line of action of weights from upper storeys (walls, floors and roofs) relative to the centroid of the upper part of the panel. The arrows on the associated dimensioning lines indicate the positive direction of these dimensions for the assumed direction of motion (angle A at the bottom of the wall is positive in the anti-clockwise sense). Under some circumstances the signs of the eccentricities may be negative, for example for e_p when an upper storey wall is much thinner than the upper storey wall represented here, particularly where the thickness steps on one face.

In the figures the instantaneous centres of rotation (marked ICR) are shown. These are useful in deriving virtual work expressions.

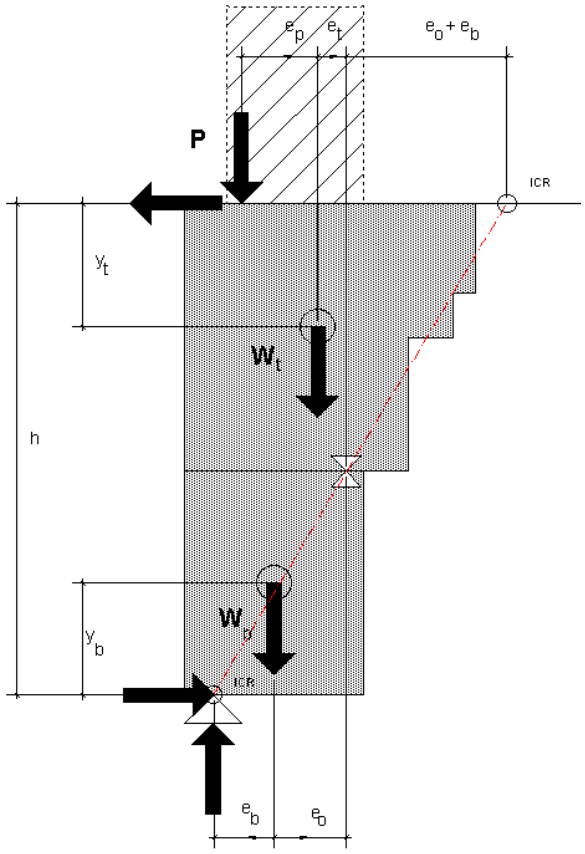


Figure 10A.1: Configuration at incipient rocking

10A.2.2 Limiting deflection for static instability

With reference to Figure 10A.2, and using virtual work, the equation of equilibrium can be directly written. For static conditions this is given by:

$$W_b(e_b - Ay_b) + W_t\left(e_o + e_b + e_t - A\left(\frac{h}{2} + y_t\right)\right) + P(e_o + e_b + e_t + e_p - Ah) = 0 \qquad \ldots 10A(1)$$

Writing:

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph \qquad \ldots 10A(2)$$

and

$$b = W_b e_b + W_t(e_o + e_b + e_t) + P(e_o + e_b + e_t + e_p) \qquad \ldots 10A(3)$$

and collecting terms in *A,* the equation of equilibrium is rewritten as:

$$-aA + b = 0 \qquad \ldots 10A(4)$$

from which:

$$A = \frac{b}{a} \qquad \ldots 10A(5)$$

when the wall becomes unstable.

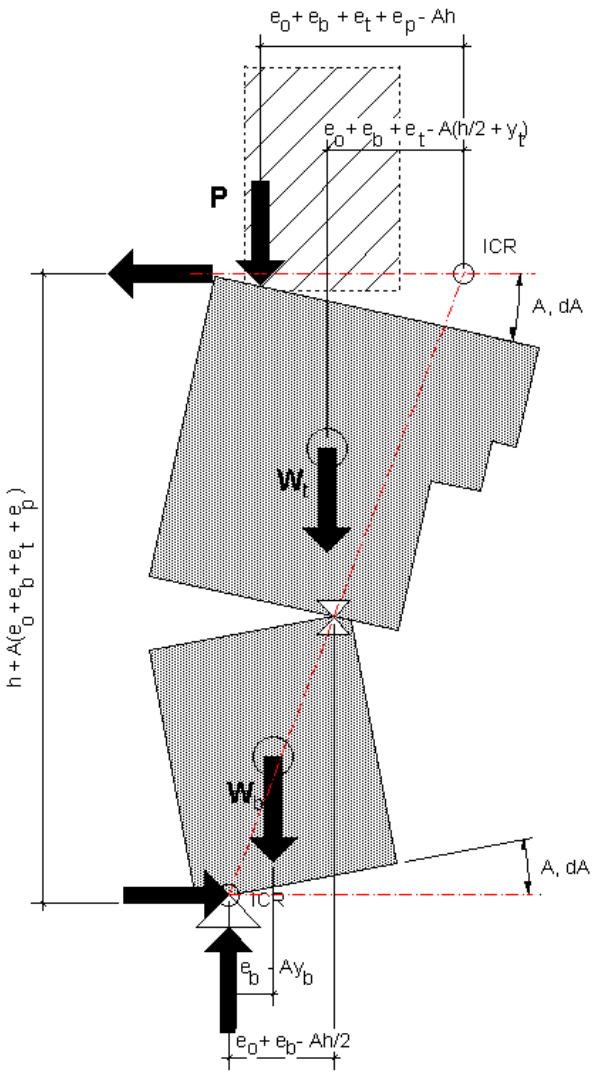


Figure 10A.2: Configuration when rotations have become significant

The critical value of the deflection at mid-height of the panel, at which the panel will be unstable, is therefore:

$$\Delta_i = A\frac{h}{2} = \frac{bh}{2a} \qquad \ldots 10A(6)$$

It is assumed that Δ_m, a fraction of this deflection, is the maximum useful deflection. Experimental and analytic studies indicate that this fraction might be assumed to be about 0.6. At larger displacements than $0.6\Delta_i$, analysis reveals an undue sensitivity to earthquake spectral content and a wide scatter in results. Some compensation is made for taking this fraction as less than unity when the final assessment for the likely performance of the wall is made.

10A.2.3 Equation of motion for free vibration

When conditions are not static the virtual work expression on the left-hand side in the equation above is unchanged, but the zero on the right-hand side of the equation is replaced by the mass times acceleration, in accordance with Newton's law. Thus we have:

$$-aA + b = -J\ddot{A} \qquad \text{…10A(7)}$$

where the usual notation for acceleration using a double dot to denote the second derivative with respect to time is used, in this case indicating angular acceleration, and J is the rotational inertia.

The rotational inertia can be written directly from the figures, noting that the centroids undergo accelerations vertically and horizontally as well as rotationally, and noting that these accelerations relate to the angular acceleration in the same way as the displacements relate to the angular displacement. While the rotational inertia is dependent on the displacements, the effects of this variation are ignored. Accordingly the rotational inertia is taken as that when no displacement has occurred. This then gives the following expression for the rotational inertia.

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b\left[e_b{}^2 + y_b{}^2\right] + W_t\left[(e_o + e_b + e_t)^2 + y_t{}^2\right] + P\left[(e_o + e_b + e_t + e_p)^2\right]\right\} + J_{anc} \qquad \text{…10A(8)}$$

where J_{bo} and J_{to} are respectively the moments of inertia of the bottom and top parts about their centroids, and J_{anc} is the inertia of any ancillary masses, such as veneers, that are not integral with the wall but that contribute to its inertia.

Note that in this equation the expressions in square brackets are the squares of the radii from the instantaneous centres of rotation to the mass centroids, where the locations of the instantaneous centres of rotation are those when there is no displacement. Some CAD programs have functions that will assist in determining the inertia about an arbitrary point (or locus), such as about the ICR shown in Figure 10A.2.

Collecting terms and normalising the equation so that the coefficient of the acceleration term is unity, we have the following differential equation of free vibration.

$$\ddot{A} - \frac{a}{J}A = -\frac{b}{J} \qquad \text{…10A(9)}$$

10A.2.4 Period of free vibration

The solution of the equation for free vibration derived in the previous section is:

$$A = C_1 \sinh\left(\sqrt{\frac{a}{J}}\tau\right) + C_2 \cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \qquad \text{…10A(10)}$$

The time, τ, is taken as zero when the wall has its maximum rotation, $A\ (=\Delta/2h)$. Using this condition and the condition that the rotational velocity is zero when the time $\tau = 0$, the solution becomes:

$$A = \left(\frac{2\Delta}{h} - \frac{b}{a}\right)\cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \qquad \text{…10A(11)}$$

For the period of the "part", T_p, we take it as four times the duration for the wall to move from its position at maximum deflection to the vertical. Then the period is given by:

$$T_p = 4\sqrt{\frac{J}{a}}\cosh^{-1}\left(\frac{\frac{b}{a}}{\frac{b}{a}-\frac{2\Delta}{h}}\right) \qquad \ldots 10A(12)$$

However, this can be further simplified by substituting the term for Δ_i found from the static analysis and putting the maximum value of Δ as Δ_m to give:

$$T_p = 4\sqrt{\frac{J}{a}}\cosh^{-1}\left(\frac{1}{1-\frac{\Delta_m}{\Delta_i}}\right) \qquad \ldots 10A(13)$$

2 | If we accept that the deflection ratio of interest is 0.6 (ie $\frac{\Delta_m}{\Delta_i} = 0.6$), then this becomes:

$$T_p = 6.27\sqrt{\frac{J}{a}} \qquad \ldots 10A(14)$$

10A.2.5 Maximum acceleration

The acceleration required to start rocking of the wall occurs when the wall is in its initial (undisturbed) state. This can be determined from the virtual work equations by assuming that $A=0$. Accordingly:

$$\ddot{A}_{\max} = \frac{b}{J} \qquad \ldots 10A(15)$$

However, a more cautious appraisal assumes that the acceleration is influenced primarily by the instantaneous acceleration of the supports, transmitted to the wall masses, without relief by wall rocking. Accordingly:

$$C_m = \frac{b}{\left(W_b y_b + W_t y_t\right)} \qquad \ldots 10A(16)$$

where C_m is the acceleration *coefficient* to just initiate rocking.

10A.2.6 Adjustments required when inter-storey displacement is large

When inter-storey displacement is large, as measured by the slope ψ (equal to the inter-storey displacement divided by the storey height), the following adjustment can be made.

The parameter b is reduced by δb in the determination of the static displacement, where:

$$\delta b = \left(W_b y_b + W_t y_t\right)\psi \qquad \ldots 10A(17)$$

Otherwise there is no undue complication. A typical limit on ψ is 0.025.

10A.2.7 Participation Factor

The participation factor can be determined in the usual way by normalising the original form of the differential equation for free vibration, modified by adding the ground acceleration term. For the original form of the equation, the ground acceleration term is added to the RHS. Written in terms of a unit rotation, this term is ($W_b y_b + W_t y_t$) times the ground acceleration. The equation is normalised by dividing through by J, and then multiplied by h/2 to convert it to one involving displacement instead of rotation. The participation factor is then the coefficient of the ground acceleration. That is

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg} \qquad \ldots 10A(18)$$

10A.2.8 Simplifications for regular walls

Simplifications can be made where the thickness of a wall within a storey is constant, there are no openings and there are no ancillary masses. Further approximations can then be applied:

- The weight of each part (top and bottom) is half the total weight, W.
- $y_b = y_t = h/4$
- The moment of inertia of the whole wall is further approximated by assuming that all e are very small relative to the height (or, for the same result, ignoring the shift of the ICR from the mid-line of the wall), giving $J = Wh^2/12g$. Alternatively, the simplified expressions for J that are given in Table 10A.1 can be used.

a) Approximate displacements for static instability | 2

The following table gives values for a and b and the resulting mid-height deflection to cause static instability when e_b and/or e_p are either zero or half of the effective thickness of the wall, t. In the table e_o and e_t are both assumed to be equal to half the effective wall thickness. While these values of the eccentricities are reasonably common, they are not the only values that will occur in practice.

The effective thickness may be assumed given by the expression:

$$t = \left(0.975 - 0.025\frac{P}{W}\right)t_{nom} \qquad \ldots 10A(19)$$

where t_{nom} is the nominal thickness of the wall.

Experiments show that this is a reasonable approximation, even for walls with soft mortar. Where there is soft mortar, greater damping occurs that reduces response, which compensates for errors in the expression for the effective thickness.

b) Approximate expression for period of vibration | 2

Noting that:

$$a = \left(\frac{W}{2} + P\right)h \qquad \ldots 10A(20)$$

and using the approximation for J relevant to a wall with large aspect ratio, the expression for the period is given by:

$$T_p = 6.27\sqrt{\frac{2Wh}{12g(W+2P)}} \qquad \ldots 10A(21)$$

where it is to be noted that the period is independent of the restraint conditions at the top and bottom of the wall (i.e. independent of both e_b and e_p).

If the height is expressed in metres, then this expression further simplifies to:

$$T_p = \sqrt{\frac{0.67h}{(1+2P/W)}} \qquad \ldots 10A(22)$$

a value confirmed from experimental results. It should be appreciated that periods may be rather long. For example, if a storey height is 3.6 m and there is no surcharge (i.e. $P=0$), then the period is about 1.55 seconds for an initial displacement that is 60% of the displacement that would cause static instability (typically in the order of the wall thickness – see Table 10A.1).

This approximation errs on the low side, which leads to an under-estimate of displacement demand and therefore to slightly incautious results. The fuller formulation is therefore preferred.

2 | **c) Participation Factor**

Suitable approximations can be made for the participation factor. It could be taken at the maximum value of 1.5. Alternatively, the numerator can be simplified as provided in the following expression, and the simplified value of J shown in Table 10A.1 can be used.

2 | **d) Maximum acceleration**

By making the same simplifications as above, the maximum acceleration is given by:

$$\ddot{A}_{\max} = \frac{b}{J} = \frac{12bg}{Wh^2} \qquad \ldots 10A(23)$$

2 | Or, more cautiously, the *acceleration coefficient*, C_m, is given in Table 10A.1 for the common cases regularly encountered.

2 | 10A.2.9 Adjustments required when inter-storey displacement is large

Using the common limit on ψ of 0.025, and substituting for $W_b = W_t = W/2$ and $y_b = y_t = h/4$, δb is found to be $Wh/160$. Taking h/t = 25, then, in the absence of any surcharge, the percentage reduction in the instability deflection is as follows for each case shown in Table 10A.1: 31% for Cases 0 and 2; and 16% for Cases 1 and 3. These are not insignificant, and these affects should be assessed especially in buildings with flexible principal framing such as steel moment-resisting frames.

Table 10A.1: Static instability defection for uniform walls, various boundary conditions

Boundary Condition Number	0	1	2	3
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	$t/2$
b	$(W/2+P)t$	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)t}{(2W+4P)}$	t
J	$\{(W/12)[h^2+7t^2]+Pt^2\}/g$	$\{(W/12)[h^2+16t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+7t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2]+4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

10A.3 Vertical cantilevers

10A.3.1 General formulation

Figure 10A.2 shows a general arrangement of a cantilever. The wall that is illustrated has an overburden load at the top, but this load will commonly be zero, as in a parapet. Where a load does exist it is important to realise that the mass associated with that load can move horizontally, so that the inertia of the wall is affected by the overburden to a greater extent than for the walls that are supported horizontally at the top. If the top load is supported onto the wall in such a way that its point of application can change, as when it is through a continuous beam or slab that cross the wall, then the formulation for the analysis of the wall will differ from that noted here.

Sometimes several walls will be linked, as when a series of face-loaded walls provide the lateral resistance to a single-storey building. This case can be solved by methods derived from the general formulation, but express formulations for it are not provided here.

For the single wall illustrated, it is assumed that P is applied to the centre of the wall at the top and that point of application remains constant. It is straightforward to obtain the following parameters:

$$a = Ph + Wy_b \qquad \text{…10A(24)}$$

$$b = (P + W)e_b \qquad \text{…10A(25)}$$

$$J = \frac{W}{12g}\left(h^2 + t_{nom}^2\right) + \frac{W}{g}\left[\left(y_b^2 + e_b^2\right)\right] + \frac{P}{g}\left[\left(h^2 + e_b^2\right)\right] \qquad \text{…10A(26)}$$

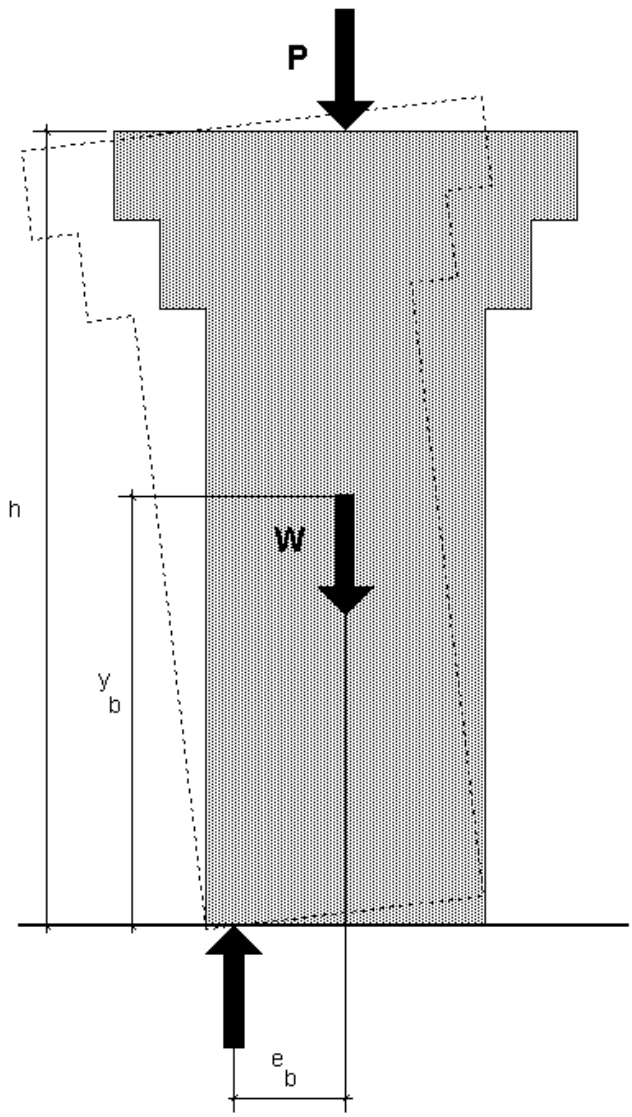


Figure 10A.3: Single cantilever

10A.3.2 Limiting deflection for static instability

When the wall just becomes unstable, the relationship for *A* remains the same as before, but the deflection is *Ah*. Thus, the limiting deflection is given by:

$$\Delta_i = Ah = \frac{bh}{a} = \frac{(P+W)he_b}{Ph+Wy_b} \qquad \ldots 10A(27)$$

For the case where $P=0$ and $y_b=h/2$ this reduces to $\Delta_i = 2e_b = t$.

10A.3.3 Period of vibration

The general expression for period remains valid. Where $P=0$, $e_b=t/2$, $y_b=h/2$, approximating $t=t_{nom}$ and expressing h in metres, the period of vibration is given by:

$$T_p = \sqrt{2.67\left[1+\left(\frac{t}{h}\right)^2\right]} \qquad \text{…10A(28)}$$

10A.3.4 Participation Factor

The expression for the participation factor remains unaffected. That is, $\gamma = Wh^2/2Jg$. This may be simplified for uniform walls with P=0 (no added load at the top) by inserting the specific expression for J. This gives 2

$$\gamma = \frac{3}{2\left(1+\left(\frac{t}{h}\right)^2\right)} \qquad \text{…10A(29)}$$

10A.3.5 Maximum acceleration

Using the same simplifications as above:

$$C = \frac{t}{h} \qquad \text{…10A(30)}$$

Appendix 10B: Tests for Assessing the Strength of Masonry and Connectors

Source: 1995 Red Book.

10B.1 Notation

ϕ Strength reduction factor.
v_a Maximum in-plane shear stress at the ultimate limit state.

10B.2 Existing materials

Strength assessments of existing masonry may be made from the results of tests. If testing is undertaken, the results of *all* tests should be recorded and reported.

For unreinforced masonry walls to be considered as structural members providing vertical support to roofs and floors or for resisting lateral loads the following conditions should be satisfied (see Figure 10B.1:

- The bonding of such walls should be such that each face of the wall surface is comprised of headers comprising not less than 4% of the wall surface and extending not less than 90 mm into each wythe.
- The distance between adjacent full-length headers should not exceed 600 mm either vertically or horizontally.
- In walls in which a single header does not extend through the wall, bonders from opposite sides should be covered with another bonder course overlapping the bonder below by at least 90 mm. If the masonry does not comply it should be removed, strengthened, or treated as a veneer or two separate skins.

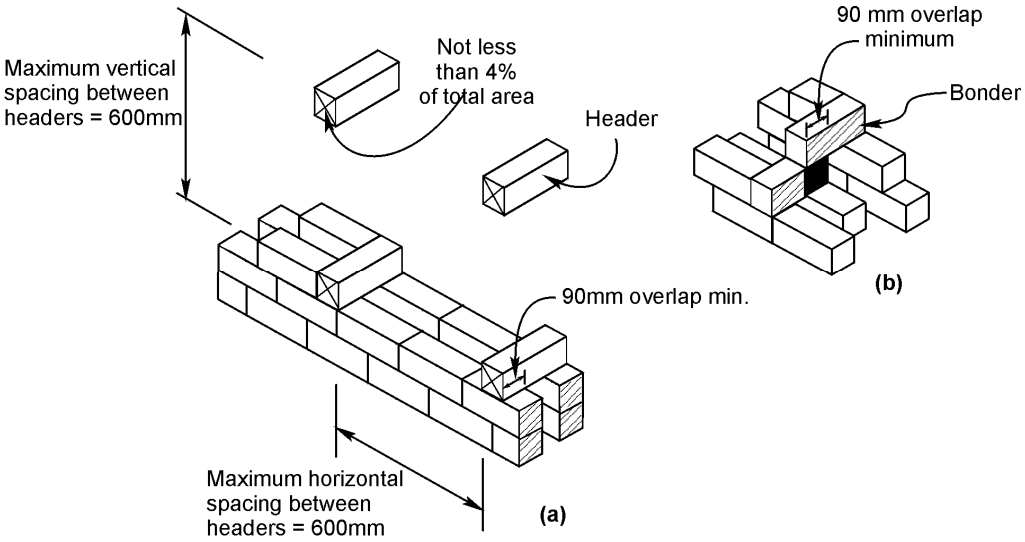


Figure 10 B.1: Bonding requirements for unreinforced masonry walls

10B.3 Tests for Masonry Strengths

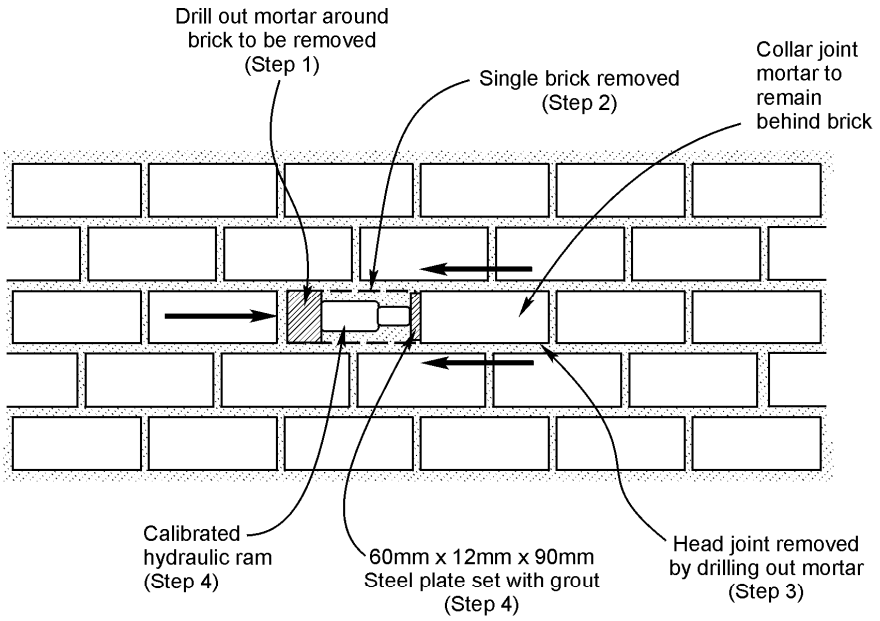
The designer may choose to conduct tests on existing masonry to establish design values. The test procedures described in this section are considered to be acceptable.

10B.3.1 In-place mortar shear test

Note: This test is thought to give unreliable results where the mortar strength has low cohesion. This is because in the process of frictional sliding the expansion of the mortar normal to the sliding plane is prevented, and this gives rise to confining pressures that will not necessarily arise during earthquake response. Core tests or tests on doublets or triplets are therefore generally preferred.

Preparation of sample

The bed joints of the outer wythe of the masonry shall be tested in shear by laterally displacing a single brick relative to the adjacent bricks in the same wythe. The head joint opposite the loaded end of the test brick shall be carefully excavated and cleared. The brick adjacent to the loaded end of the test brick shall be carefully removed by sawing or drilling and excavating to provide space for a hydraulic ram and steel loading blocks (see Figure 10B.2).



Procedure:

1. Existing mortar drilled out with 8mm masonry drill by 100mm long
2. Remove brick
3. Drill out head joint mortar by 100mm deep
4. Install jack and test
5. Test shear strength of mortar (kPa) = $\dfrac{\text{P (Load in Newtons) x 1000}}{\text{2 x Flat area of brick (mm}^2\text{)}}$

Figure 10B.2: In-place mortar shear tests

Application of load and determination of results

Steel blocks, the size of the end of the brick, shall be used on each end of the ram to distribute the load to the brick. The blocks shall not contact the mortar joints. The load shall be applied horizontally, in the plane of the wythe, until either a crack can be seen or a slip occurs. The strength of the mortar shall be calculated by dividing the load at the first cracking or movement of the test brick by the nominal gross area of the sum of the two bed joints.

Test frequency

Test positions shall be distributed such that the conditions are representative of those of the entire structure expected to be utilised for seismic resistance. The minimum number of tests shall be as follows:

a) At each of the first and top storeys, not less than two tests per wall or line of wall elements providing a common line of resistance to lateral forces

b) At all other storeys, not less than one test per wall or line of wall elements providing a common line of resistance to lateral forces.

c) In any case, not less than one test per 500 sq m of wall surface nor less than a total of eight tests.

Determination of design values from tests

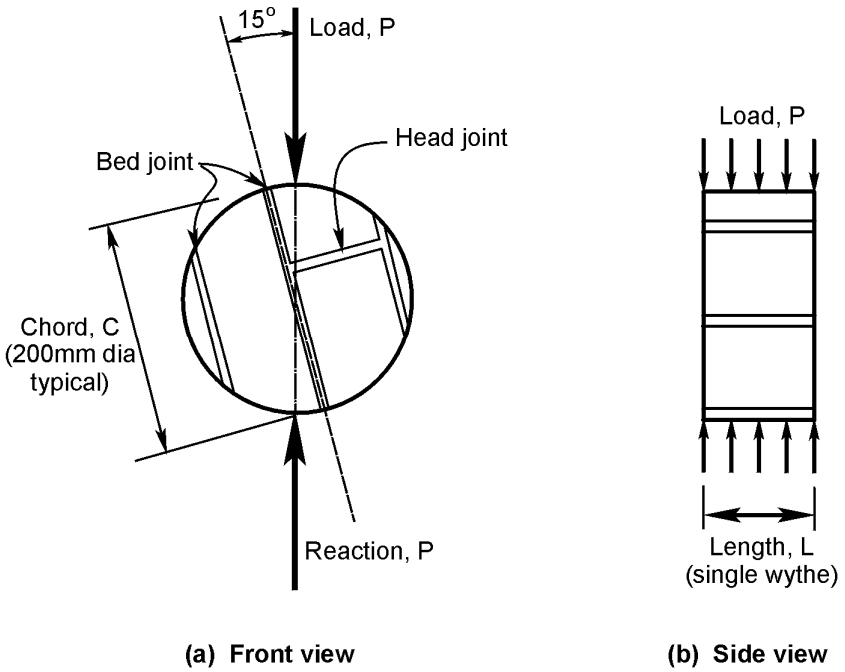
The relationship between the test results and the maximum ultimate limit state design shear stress, v_a, is given in Table 10B.1.

10B.3.2 Bed joint shear test

Note: This test will only provide the total shear strength (cohesion and friction). However, the effects of friction are unlikely to be large where the test is undertaken on reasonably competent mortar, so the shear strength recorded might be assigned entirely to cohesion. Alternatively, a representative value of μ may be assumed to enable evaluation of the true cohesion.

Preparation of sample

A core of typically 200 mm diameter shall be taken through the wall, centred on a horizontal mortar joint (see Figure 10B.3). | 2



2 **Figure10B.3: Bed joint shear test arrangement**

Application of load and determination of results

The core shall be placed between the platens of a compression testing machine with the plane of the horizontal mortar joint aligned at 15° to the vertical. The strength of the mortar shall be calculated by dividing the load at failure by the nominal gross area of the mortar joint.

Test frequencies

Test frequencies shall be as for the in-place mortar shear test.

Determination of design values from tests

The relationship between the test results and the ultimate limit state design shear stress, v_a, is given
2 in Table 10B.1.

Table 10B.1: Determination of design values from in-place mortar shear tests and bed joint shear tests

In-place mortar shear	Bed joint shear	Ultimate limit state in-plane shear stress v_a (kPa)
80% of test results not less than (kPa) χ + axial stress	Average test results of cores (kPa) 0.7 χ	 χ (maximum 1000 kPa) (refer note 2)

Notes:

1 These values may only be used when the wall response is not dominated by flexural action (i.e. significant flexural cracking not expected)

2 Shear stress may be increased by the addition of 30% of the dead weight stress of the wall above.

Example of application of Table 10B.1: if 80% of in-place mortar shear test results were not less than 400 kPa and the axial stress was 100 kPa, then the ultimate limit state in-plane shear stress would be (400–100) + 0.3(100) = 330 kPa. 2

If bed joint shear tests were carried out on samples taken from the same location and the average result was 230 kPa, then the ultimate limit state in-plane shear stress would be (210/0.7) + 0.3(100) = 330 kPa.

10B.3.3 Tests on Doublets and Triplets

Testing of doublets and triplets are possibly the best and most reliable means of determining strength parameters of masonry. An advantage of the methods is that clamping forces can be independently varied, so that separate values of friction and cohesion parameters can be obtained.

Figure 10B.3 shows a schematic of a test set-up for doublets. Further information on the testing procedures and details of suitable test rigs are given in Hansen (1999).

Testing on triplets require less sophistication.

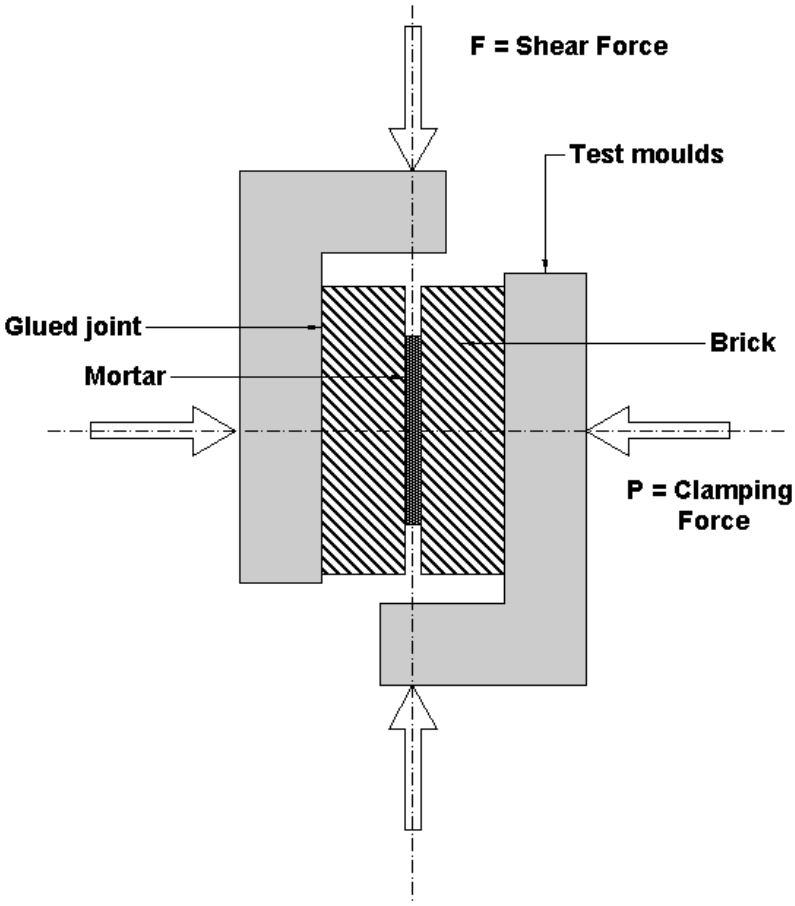


Figure 10B.4: Schematic of an arrangement for testing doublets 2

10B.4 Tests on Connectors

10B.4.1 Default Strength for Bolts

The following Table 10B.2 lists design strengths that may be adopted for bolts connecting components to masonry. Larger values may be adopted if justified by tests conducted in accordance with b).

Table 10B.2: Default connector strengths

Item	Type	Comment	Strength	ϕ
1	**Shear Connectors**	Bolts should be centred in an oversized hole with non-shrink grout or epoxy resin grout around the circumference.		0.7
	Shear bolts and shear dowels embedded at least 200 mm into unreinforced masonry walls.		M12 bolt: 6 kN M16 bolt: 9 kN M20 bolt: 14 kN	
2	Tension Connectors	The designer should also ensure that the connection to other components is adequate. 25% of all new anchors should be tested to the following torques: —M12: 54 Nm —M16: 68 Nm —M20: 100 Nm		0.7
	Tension bolts extending entirely through the masonry, and secured with a bearing plate at least 138 x 138 or 155 diameter.		29 kN (all sizes)	
	Tension bolts and reinforcing bars grouted (cementitious or epoxy resin) 50 mm less than the thickness of the masonry	Bolts grouted with epoxy may lose strength and fail abruptly id wall cracking occurs at the bolt. The designer should ensure that failure cones from adjacent bolts do not overlap.	11 kN (all sizes)	

10B.4.2 Tension strength of anchors

This section outlines procedures for preliminary testing where the designer may wish to conduct tests on new anchors to derive greater design values than suggested in Table 10B.2.

Application of load and determination of results

The masonry wall should support the test apparatus. The distance between the anchor and the test apparatus support should not be less than the wall thickness. The tension test load reported should be the load recorded at 3 mm relative movement of the anchor and the adjacent masonry surface. For the testing of existing anchors, a preload of 1.5 kN shall be applied prior to establishing a datum for recording elongation. Anchors should be installed in the same manner and using the same materials as intended to be used in the actual construction.

Test frequency

A minimum of *five* tests for each bolt size and type should be undertaken.

Determination of design values from tests

The ultimate limit state strength of tested existing wall anchors should be taken as the mean of all results less 0.8 times the standard deviation for each bolt size. A strength reduction factor of 0.7 should be used to determine the design strength.

Appendix 11A: Timber Diaphragm Stiffness

The mid span deflection of a horizontal diaphragm Δ_h can be calculated from

$$\Delta_h = \Delta_1 + \Delta_2 + \Delta_3 \qquad \text{...11A(1)}$$

where

Δ_1 = diaphragm flexural deformation considering chords acting as a moment resisting couple (mm)
Δ_2 = diaphragm shear deformation resulting from beam action of the diaphragm (mm)
Δ_3 = deformation due to nail slip for horizontal diaphragm (mm)

For transverse sheathing:

$$\Delta_1 = 0$$
$$\Delta_2 = 0$$
$$\Delta_3 = \frac{Le_n}{2s} \qquad \text{…11A(2)}$$

For single diagonal sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \qquad \text{...11A(3)}$$
$$\Delta_2 = \frac{WL}{4EBt} \qquad \text{…11A(4)}$$
$$\Delta_3 = \frac{(1+a)me_n}{2} \qquad \text{…11A(5)}$$

For double diagonal sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \qquad \text{...11A(6)}$$
$$\Delta_2 = \frac{WL}{8EBt} \qquad \text{…11A(7)}$$
$$\Delta_3 = \frac{(1+a)me_n}{2} \qquad \text{...11A(8)}$$

For panel sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \qquad \text{...11A(9)}$$
$$\Delta_2 = \frac{WL}{8GBt} \qquad \text{…11A(10)}$$
$$\Delta_3 = \frac{(1+a)me_n}{2} \qquad \text{...11A(11)}$$

where

a = Aspect Ratio of each sheathing panel:
= 0 when relative movement along sheet edges is prevented,
= 1 when square sheathing panels are used,
= 2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)
A = Sectional area of one chord (mm^2)
B = Distance between diaphragm chord members (mm)
e_n = Nail slip resulting from the shear force V (mm)

E = Elastic modulus of the chord members (MPa)
G = Shear modulus of the sheathing (MPa)
L = Span of a horizontal diaphragm (mm)
m = Number of sheathing panels along the length of the edge chord
t = Thickness of the sheathing (mm)
W = Lateral load applied to a horizontal diaphragm (N)

Appendix 11B: Timber Diaphragm Strength

11B.1 Square sheathing:

The strength of transversely sheathed diaphragms, i.e. where the sheathing runs perpendicular to the diaphragm span, depends on the resisting moment furnished by nail couples at each stud crossing. If the nail couple, $M = F_n.s$, then the shear force per metre length, v , that can be resisted is

$$v = \frac{F_n}{l} \cdot \frac{s}{b} \qquad \text{…11B(1)}$$

and the total shear strength is

$$V = \frac{2F_n\, s\, B}{b\, l}. \qquad \text{…11B(2)}$$

If the boards have not shrunk apart, then friction between the board edges could possibly increase the load carrying capacity by the addition of a term, 2Bv', where

v' = 74 N/m for 25 mm sawn boards,
= 148 N/m for 50 mm sawn boards, and
= 222 N/m for tongue and groove boards.

The in-plane stress in the sheathing is given by the expression

$$V = \frac{2F_b\, z\, B}{b\, l} \qquad \text{…11B(3)}$$

where;

F_n = nominal nail strength
s = nail spacing
l = spacing between joints
b = width of sheathing board
B = depth of diaphragm
z = section modulus of the sheathing board = $\frac{b^2\, t}{6}$.

2

11B.2 Single diagonal sheathing:

As above, the strength of the diaphragm depends on the resisting moment produced by the nail couples at each joint crossing. The total load that can be resisted is;

$$V = \frac{F_n\, N\, B}{b} \qquad \text{…11B(4)}$$

2

where;

N is the total number of nails.

The in-plane stress in the sheathing is given by the expression,

$$V = F_c B t. \qquad \text{…11B(5)}$$

2

where;

2 t = thickness of the sheathing board

Other symbols as defined in 11D.1.

The chord members need to be checked for combined bending and axial stresses (refer to NZS3603).

11B.3 Double diagonal sheathing:

The total load that can be resisted by the nail couples at each joist crossing is the same as for the single diagonal sheathing and the load resisted by the in-plane stress in the sheathing is;

$$V = 2F_cBt. \qquad \text{…11B(6)}$$

where;

2 F_c = characteristic stress in the sheathing board in compression parallel to the grain

Other symbols as defined in 11B.1.

11B.4 Panel sheathing:

The strength values in Table 11.1 should be used in assessing the strength of these elements – unless specific tests are carried out.

Appendix 11C: Timber Shear Wall Stiffness

The horizontal inter storey deflection in one storey of a shearwall Δ_w can be calculated from:

$$\Delta_w = \Delta_4 + \Delta_5 + \Delta_6 + \Delta_7 \qquad \text{...11C(1)}$$

where

Δ_4 = deformation due to support connection relaxation
Δ_5 = wall shear deformation
Δ_6 = deformation due to nail slip
Δ_7 = deformation due to flexure as a cantilever (may be ignored for single storey shear walls).

For transverse sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{...11C(2)}$$

$$\Delta_5 = 0$$

$$\Delta_6 = 2\frac{H}{s}e_n \qquad \text{...11C(3)}$$

$$\Delta_7 = H\theta$$

For single diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{…11C(4)}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{...11C(5)}$$

$$\Delta_6 = 2\sqrt{2}\,e_n \quad \text{for the case where H} \le \text{B, OR} \qquad \text{…11C(6)}$$

$$= 2\sqrt{2}\,\frac{H}{B}e_n \quad \text{for the case where H} \ge \text{B}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \qquad \text{...11C(7)}$$

For double diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{…11C(8)}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{...11C(9)}$$

$$\Delta_6 = \sqrt{2}\,e_n \quad \text{for the case where H} \le \text{B, OR} \qquad \text{…11C(10)}$$

$$= \sqrt{2}\,\frac{H}{B}e_n \quad \text{for the case where H} \ge \text{B} \qquad \text{…11C(11)}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \qquad \text{...11C(12)}$$

For panel sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{…11C(13)}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{...11C(14)}$$

$$\Delta_6 = 2(1+a)me_n \qquad \text{...11C(14)}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \qquad ...11C(15)$$

where;

a	=	Aspect Ratio of each sheathing panel:
	=	0 when relative movement along sheet edges is prevented,
	=	1 when square sheathing panels are used,
	=	2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)
A	=	Sectional area of one chord (mm^2)
B	=	Distance between diaphragm or shear wall chord members (mm)
e_n	=	Nail slip resulting from the shear force V (mm)
E	=	Elastic modulus of the chord members (MPa)
G	=	Shear modulus of the sheathing (MPa)
H	=	Height of the storey under consideration (mm)
m	=	Number of sheathing panels along the length of the edge chord
t	=	Thickness of the sheathing (mm)
V	=	Shear force in storey under consideration (N)
θ	=	Flexural rotation at base of storey under consideration (radians)
δ_c	=	Vertical downward movement (mm) at the base of the compression end of the wall (this may be due to compression perpendicular to the grain deformation in the bottom plate)
δ_t	=	Vertical upward movement (mm) at the base of the tension end of the wall (this may be due to deformations in a nailed fastener and the members to which it is anchored).

Appendix 11D: Timber Shear Wall Strength

11D.1 Transverse sheathing:

The strength of transversely sheathed shear walls depends on the resisting moment furnished by nail couples at each stud crossing. If the nail couple, $M = F_n$. s, then the shear force per metre length, v , that can be resisted is;

$$v = \frac{F_n}{l} \cdot \frac{s}{b} \qquad \text{…11D(1)}$$

and the total shear strength is;

$$V = \frac{F_n \, s \, B}{b \, l} \, . \qquad \text{…11D(2)}$$

If the boards have not shrunk apart, then friction between the board edges could possibly increase the load carrying capacity by the addition of a term Bv', where

v' = 74 N/m for 25 mm sawn boards,
= 148 N/m for 50 mm sawn boards, and
= 222 N/m for tongue and groove boards.

The in-plane stress in the sheathing is given by the expression;

$$V = \frac{F_b \, z \, B}{b \, l} \qquad \text{…11D(3)}$$

where;

F_n = nominal nail strength
s = nail spacing
l = spacing between studs
b = width of sheathing board
B = depth of diaphragm
z = section modulus of the sheathing board = $\frac{b^2 \, t}{6}$.

2

11D.2 Single diagonal sheathing:

The horizontal shear, V_i, carried by each board is;

$$V_i = \frac{1}{\sqrt{2}} N \, F_n \qquad \text{…11D(4)}$$

giving a total strength of;

$$V = \frac{F_n \, NB}{2b} \qquad \text{…11D(5)}$$

Since the axial force in the sheathing is the same on both sides of any intermediate stiffener, no load is transferred into the stiffeners from the sheathing. However, the perimeter members are subjected to both axial loads and bending and must be designed for the combined stresses (see NZS3603). The bending in the chord members is caused by a UDL of;

$$w = \frac{N F_n}{b} \quad . \qquad \text{…11D(6)}$$

The in-plane strength of the sheathing is given by;

$$V = \frac{F_c\, bt}{2} \, . \qquad \text{…11D(7)}$$

where;

2 t = thickness of the sheathing board

Other symbols as defined in 11D.1.

11D.3 Double diagonal sheathing:

Based on the strengths of the nail couples, the strength of the shear wall is given by;

$$V = \frac{F_n\, NB}{2b} \qquad \text{…11D(8)}$$

The in-plane stress in the sheathing boards is given by the expression;

$$V = F_c\, Bt \, . \qquad \text{…11D(9)}$$

where;

2 F_c = characteristic stress in the sheathing board in compression parallel to the grain

Other symbols as defined in 11D.1.

The stress in the chords is given by;

$$V = \frac{F_c\, BA}{H} \qquad \text{…11D(10)}$$

while the stress in the plates is given by;

$$V = F_c\, A_p \, . \qquad \text{…11D(11)}$$

11D.4 Panel sheathing:

The strength values in Table 11.1 should be used in assessing the strength of these elements – unless specific tests are carried out.