Section B

ANALYSIS AND DESIGN METHODS

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This paper is the result of deliberations of the Society's Study Group for the Seismic Design of STEEL STRUCTURES.

2. INTRODUCTION

2.1 General Philosophy

This paper introduces the general concepts that should be followed when analysing and designing steel structures. Analysis is dealt with briefly as the techniques are well established and adequately documented elsewhere. Comments are made on aspects that relate specificly to steel structures. For design purposes structures are defined according to their level of seismic resistance. All steel structures can be classified one of three categories.

(i) Fully Ductile Structures

These are capable of sustaining or required to sustain large inelastic deformations without a significant loss of strength or reduction in energy dissipating capacity. Capacity design procedures shall be used.

(ii) Structures of Limited Ductility or Low Ductility Demand

These are capable of sustaining small to moderate inelastic deformations without a significant loss of strength or stiffness. Capacity design procedures may be used with modifications to design and detailing requirements.

(iii) Elastic Structures

These structures are expected to respond elastically to large earthquake motions and do not require any special detailing requirements. Elastic detailing based on the Code requirements will ensure that most steel structures will not fail even with minor excursions into the post elastic range.

2.1 Definitions

Ideal Strength - This is calculated using the specified yield stress and specified member dimensions.

Dependable Strength - For steel structures this is equal to the ideal strength, i.e. $\phi = 1.0$

Overstrength - This is calculated taking into account all of the possible factors that may increase the strength above the ideal value, such as higher yield stresses, strain hardening, increases in section dimensions, etc. Section 3.6 gives recommended overstrength factors.

3. ANALYSIS

3.1 Classification of Structures

Ductility demand for different structural types shall be classified by either the 'Structural Type Factor' 'S'(2) or by μ in 'Recommendations for Seismic Design of Petrochemical Plants' (S.D.P.P.) (3) and shall be as follows:

Ductility Demand	S	μ
Full	$S < 2.0$	$\mu > 2.0$
Limited	$2.0 \leq S < 6.0$	$1.0 > \mu \geq 2.0$
None	$S = 6.0$	$\mu = 1.0$

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Because of the different philosophical approach between NZS 4203 and SDPP a direct correlation between 'S' and 'μ' is not possible. Examples of 'S' and 'μ' factors for various structural types are given in Appendix A.

In some cases structural members may be required to be proportioned on the basis of stiffness rather than strength. If there is a sufficient strength reserve then these structures may become Limited rather than Fully Ductile for design purposes.

Note: If a structure is capable of ductility a maximum S value of 5/1.5 is recommended otherwise the overstrength factor of 1.5 will produce loads equivalent to elastic loads but ductile detailing would still be required.

3.2 Loadings

(i) Gravity, wind and snow loadings shall be as per NZS 4203.

(ii) Seismic loadings shall be determined either by a static or dynamic means as outlined in NZS 4203 or Seismic Design of Petrochemical Plants.
- Static -
A standard triangular distribution (modified if necessary for tall slender structures) shall be used.
- Dynamic -
A spectral modal analysis supplemented by numerical integration response analysis may be carried out to modify the distribution of the applied loads. Often steel structures will be more flexible than similar structures in other materials and the participation of higher modes may be significant.

3.3 Factors Affecting Analysis

One or more of the following factors may need to be considered during an analysis of a structure:

(i) Shear deformations in members with low span/depth ratios

(ii) Effect of rigid end panels.

(iii) Joint panel deformation.

(iv) Axial deformation.

(v) Buckling strength of elements.

(vi) Stiffness degradation of elements.

(vii) Reduced flexural yield capacity due to axial and/or shear and/or torsion stresses.

(viii) P-Delta effects.

(ix) Effects of cracked or uncracked cover concrete on stiffness.

(x) Effects of concrete slabs and their connections on member stiffness.

(xi) Foundation and soil deformations.

(xii) Effect of secondary elements.

(xiii) Torsion on whole structure.

(xiv) Stiffness (fixity of joints) - the analytical model chosen must reflect the actual joint stiffness.

3.4 Damping

For elastically responding structures S values of 5.0 for concrete and 6.0 for steel are related to approximate elastic damping of 3.5% and 2.0% of critical damping respectively. Modification to a steel structure by the use of bolted joints and the addition of such elements as concrete to columns and concrete slabs built integral with beams will significantly increase damping.

For these situations S = 5.0 instead of 6.0 for elastically responding structures is acceptable. Damping factors for all steel frames responding elastically should be in the following range:

Type	% of critical damping
Fully welded structure	1% - 2%
Fully bolted structure	5% - 7%

For structures designed to respond inelastically damping factors are likely to be in the following ranges:

Type	% of critical damping
Fully welded structure	5% 7%
Fully bolted structure	10% - 15%

The Table below lists the recommended values of initial elastic (viscous) damping for use in the seismic design of steel structures.

Type of Structure	Behaviour	Welded Connections	Bolted Connections
Clad	Elastic	2.5	5.0
Clad	Ductile	5.0	10.0
Unclad	Elastic	2.0	5.0
Unclad	Ductile	5.0	7.5

Notes:

A clad steel structure is one with concrete or composite steel flooring systems integrally connected to the supporting steel beams. The structure may be open or fully enclosed around its perimeter. Examples include multistorey car parking buildings and typical commercial office construction.

An unclad steel structure is one with an open floor system and no exterior covering. Steel grid panel flooring systems laid on steel beams do not constitute cladding. A multi-level tank farm support structure is an example of unclad construction.

3.5 Moment Redistribution

Redistribution of design moments by elastic analysis may only be carried out in accordance with the following provisions:

- Equilibrium between internal and external forces must be maintained for each combination of applied loads.
- The reduction in peak bending moments should not exceed the following values unless justified by special study.

Fully Ductile	30%
Limited Ductility	15%
Elastic	0%

(Note: steel sections having large form factors may not provide sufficient rotation capacity if the maximum redistribution allowed is carried out and a high ductility demand is required.)

Redistribution may lead to excessive deformations in very flexible structures.

3.6 Overstrength Factors

The energy dissipating elements should be chosen and suitably designed and detailed while other structural elements (including connections) should be provided with sufficient reserve strength to ensure that the chosen energy mechanisms are maintained during large deformations in the structure.

In locations where yielding is designed to occur stress raisers such as abrupt changes of section or sharp corners or notches should be avoided.

Limited data is available on structural steel sections used in New Zealand and results available indicate a significant variation in material properties. In the absence of more detailed information the following overstrength factors are recommended:

	Fully Ductile Structures	Limited Ductility Structures
Strain Hardening	1.10	1.0
Material Variation	1.35	1.35
Overstrength Factor	1.50	1.35

In the absence of specific design data these factors are to be used for mild steel structural sections assuming a nominal yield strength of 250 MPa. For high yield steel use a strain hardening factor of 1.25 and 1.1 respectively for Fully and Limited Ductile structures.

Note:

Where drift limit considerations dictate the size of structural members they need only be designed or detailed as for the limited ductile case.

3.7 P-Delta Effects

Differences between the actual response of a structure to loading and that predicted by a linear elastic analysis result from a number of causes. A number of non-linear effects combine to produce these differences with the P-delta effect being one of them.

In most design situations the P-delta effects are not significant until the structure undergoes large inelastic deformations. Current codes place a limit on the elastically calculated interstorey drift and if these are adhered to no further calculation is required. It should be noted however that there are very few studies of the inelastic response of steel structures, covering these aspects.

Where P-delta demand is a problem it is preferable to strengthen the structure rather than to attempt to reduce the drift by increasing the stiffness.

4. DESIGN METHODS

4.1 Strength Design Method

This method is recommended for all structures of more than two seismic mass levels whether they are designed for full or limited ductility. (Structures of any number of mass levels may be designed using the Allowable Stress Design Method, (refer section 4.3). Structures up to four mass levels may be designed using plastic design methods (refer section 4.2).

The design principles as set out in NZS 4203 shall be applied. Forces in the members are determined assuming linear elastic behaviour under the design loads. Forces thus obtained may then be redistributed in accordance with section 3.5

Capacity design procedures are then applied. Elements of the primary lateral load resisting system are chosen and suitably designed and detailed for energy dissipation under severe or moderate deformations depending on ductility demand. All other structural elements are then provided with sufficient overstrength so that the chosen hierachy for the collapse mechanism is maintained.

Sizing of members for strength shall be based on the requirements of section 5.

Satisfying the requirements for local buckling and lateral instability shall vary according to the ductility demand required in the structure.

For structural elements of structures limited ductility the requirements of AS1250 (clauses 10.8 - 10.10) are deemed to be sufficient.

For structural elements of fully ductile structures the recommendations of the study group should be used to ensure that full plastic deformations can be obtained.

(Note: The rotation of plastic hinges under severe seismic action is considerably in excess of those envisaged by section 10 of AS1250 and hence more stringent stability requirements may be needed to ensure the plastic capacity of the section is maintained.)

4.2 Plastic Design Methods

This method may be used for all classes of structures having up to four seismic mass levels. Once the collapse mechanism has been postulated all members shall be then proportioned according to the requirements of Chapter 10 of AS1250.

Load combinations shall be as for Strength Design in NZS 4203. Chosen collapse mechanisms shall not lead to deformations that are in excess of those permitted by NZS 4203. In addition mechanisms that have an insufficient number of plastic hinges shall be avoided against. (For example, column storey mechanisms.)

4.3 Allowable Stress Design Method

This method may be used for structures of any height with no or low ductility requirements. If an S factor of 6.0 is used a seismic load of 0.8E shall be used and the allowable stresses increased by 1.33. Elastic section moduli shall be used.

It is recognised that because of the ductile properties of steel most structures will exhibit some post elastic behaviour even with very basic detailing.

4.4 Capacity Design Methods

The principles of capacity design are well documented and the assessment of forces on beams in a weak beam strong column concept is relatively straightforward. The realistic assessment of column loads is more difficult.

The Concrete Code (NZS 3101) proposes a method for determining column actions using:

(i) "$\emptyset_o$' and 'ω' factors to enhance column moments.

(ii) "R_m' factor to reduce the enhanced moment depending upon the magnitude of the axial load and whether it is tension or compression.

(iii) "R_m' factor to modify axial loads dependent upon the number of floors hinging above the level being considered.

(iv) "$\emptyset_o$' and numerical factors to enhance the design shear forces.

The biaxial effects due to concurrent loading and the participation of higher modes are addressed by reducing the problem to an unaxial one and using larger numerical values for the above factors.

The method outlined in the Concrete Code purports to be a method for the evaluation of column actions in ductile multistorey frames and its use is recommended for steel frames until further research data is available. However the following points should be noted:

(i) The presence of an axial load whether it be tensile or compressive will reduce the flexural capacity of the steel section. Unlike concrete sections though, the presence of moderate compressive loads will not adversely affect the ductile capacity.

(ii) Biaxial design of steel sections is relatively straight forward and hence consideration should be given to designing for loading in two directions simultaneously rather than a modified unaxial approach.

(iii) Shear failures in steel are not necessarily as brittle as those in concrete and hence assessment of shear forces may be too great.

4.5 Studies of Steel Structures

Limited studies in the dynamic behaviour of a three storey 'gravity dominated' steel frame and a twelve storey ductile steel frame have been carried out using the 'Drain 2D' programme. The purpose of these studies was to verify whether the method proposed in the concrete code was applicable to steel structures.

The two frames selected were two reinforced concrete ones redesigned in steel. These were taken from a University of Canterbury report, "The Seismic Response of Reinforced Concrete Multistorey Frames", by D.M. Tompkins (13).

The results to date are inconclusive

A. Three Storey Frame

Designed to an 'S' factor of 0.8 and no 'ω' (dynamic magnification) factor. Elastic damping was taken as 2% of critical and strain hardening at 5% of E.
The structure was analysed using the following acceleration records:

(i) 10 seconds N-S El Centro

(ii) 3 seconds 0.46 Horizontal Acceleration

(iii) 10 seconds 0.6 x Pacoma Dam Record

Analysis of the above results indicated that at no stage did a collapse mechanism occur and ductility demand at plastic hinges was not excessive for steel sections.

B. Twelve Storey Frame

This frame was designed using current codes. The choice of frame type (i.e. as defined by Tompkins) meant that stiffness rather than strength governed member sizes. The yield stress in both beams and columns was 250 MPa. An overstrength factor for beams was used and the moment distributed to the columns on the basis of elastic distribution. No dynamic magnification factor (ω) was used.

The dynamic analysis using El Centro N-S as the forcing function gave deflections up to twice the design deflections but relatively few plastic hinges. This would indicate that if member sizes are significantly larger due to stiffness requirements then the magnitude of forces on columns in particular needs to be reduced.

C. Future Research

It is hoped that further work can be done on realistic steel frames in order that sufficient data can be obtained to make recommendations regarding the use of 'ω' factors.

5. RESISTING STRENGTH OF MEMBERS

Primary energy dissipation in structural steel members can be by any of the modes listed below. The values refer to compact sections only. Combinations of loadings may occur that reduce the capacity of the structural member in its prime energy dissipating mode. Refer to other papers for details of these requirements.

5.1 Flexure

$$M_u = F_y.S$$

where M_u = design load moment
F_y = nominal yield stress of material
S = plastic section modulus

(Note: For sections with high shape factors this method of determining design strength may not be satisfactory for high ductility demand structures. Large strains in extreme fibres may lead to local instability or fracture of the section.)

5.2 Axial Tension

$$P_u = A_n.F_y$$

where P_u = design tensile load
A_n = net section area

5.3 Axial Compression

$$P_u = A_s.F_y$$

where P_u = design compression load
F_s = compression stress in member allowing for length and local instability effects
A_s = gross section area

5.4 Shear

$$V_u = A_w.0.55F_y$$

where V_u = design shear
A_w = shear area of section

5.5 Torsion

$$T_u = \psi.0.55F_y$$

where T_u = design torque
ψ = stress factor assuming yield over the whole section

6.0 MATERIAL PROPERTIES

Steel shall conform to clauses 2.1.1 and 2.1.2 of NZS 3404:1977. A material type factor M = 0.8 shall be used for such steel.

In addition to the above requirements for steel in yielding members the following criteria shall apply:

	Full Ductility	Limited Ductility
Max. specified yield stress	360 MPa	450 MPa
Max. ratio of yield stress to ultimate stress	0.70	0.9
Min. length of yield plateau	$10 \times \varepsilon_y$	$3 \times \varepsilon_y$

For steel that does not conform to the above requirements a higher material type factor shall be used. The product of S and M need not be greater than 6.0.

7. REFERENCES

1. NZS 3404 "Code for Design of Steel Structures",Standards Assoc. of N.Z., Wellington, New Zealand.

2. NZS 4203:1976 "Code of Practice for General Structural Design and Design Loadings for Buildings", Standards Assoc. of N.Z., Wellington, New Zealand.

3. "Seismic Design of Petrochemical Plants", prepared by N.Z. Ministry of Works and Development for the Ministry of Energy, Wellington, New Zealand, March 1981.

4. AS1250-1981, "SAA Steel Structures Code", Standards Assoc. of Australia, Sydney, Australia.

5. Erasmus, L.A. "Statistical Analysis of Mechanical Properties Obtained in Structural Steel Sections", Special Report R4-01, N.Z.H.E.R.A. Manakau City, New Zealand.

6. Erasmus, L.A. "The Mechanical Properties of Strucutral Sections and the Relevance of These Properties to the Capacity Design of Structures", Trans. IPENZ, Civil Eng. Sect., Vol. 11, No. 3/CE, Nov. 1984.

7. Evison, R.D. and Mowat, A.F. "The Seismic Design of Industrial Plants", Trans. IPENZ Civil Eng. Div., Vol. 9, December 1982.

8. HERA "SANZ Steel Structures Code", draft prepared by N.Z. Heavy Engineering Research Assoc., Manukau City, New Zealand.

9. Beamish, M.J. "Seismic Behaviour of Steel Portal Frame Knee Joints". Report submitted to N.Z.H.E.R.A., Auckland, 1984.

10. Lay, M.G. "Source Book for the Australian Steel Structures Code AS1250", Third Edition, 1982.

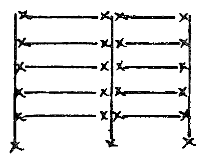
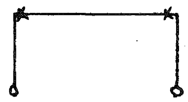
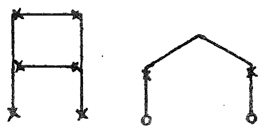
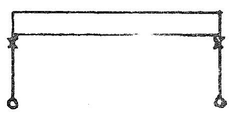
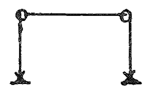
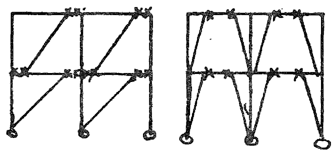
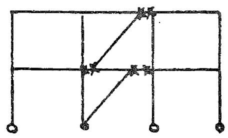
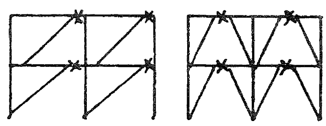
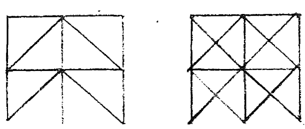
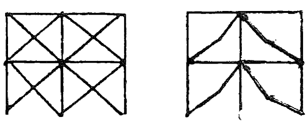
11. "Design of Earthquake Resistant Structures", Edited by E. Rosenbleuth, Pentech Press, 1980.

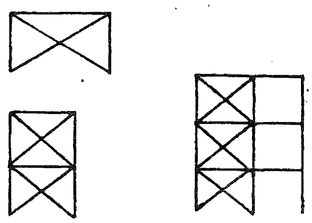
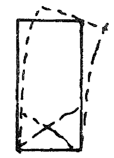
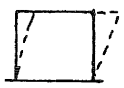
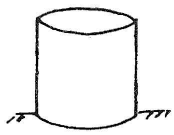
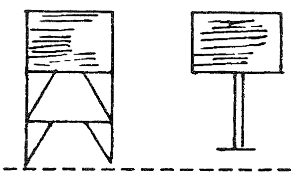
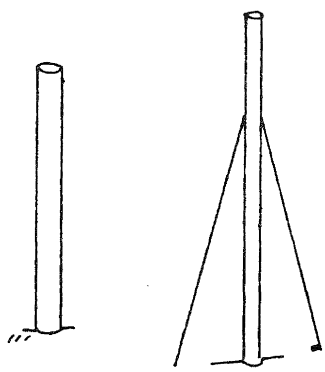
12. A. Aggarwal and R. Coates, "Structural Damping in Bolted Beam-Column Connections". Proceedings of Metal Structures Conference, Melbourne, Australia, 1985.

13. Tompkins, D.M., "The Seismic Response of Reinforced Concrete Multistorey Frames", Research Report No. 80-5, Civil Eng. Dept., University of Canterbury, February 1980.

APPENDIX A

S - FACTORS FOR STEEL STRUCTURES

TYPE OF STRUCTURE	DIAGRAM	S	μ
1 MOMENT RESISTING FRAMES			
1.1 Frame designed by Strength Design Method with adequate number of plastic hinges (refer C3.4.2 NZS 4203)		0.8	4.0
1.2 Frame designed by Strength Design Method with inadequate number of plastic hinges		1.0	3.0
1.3 Frame designed by Plastic Design methods up to four seismic levels		1.2	3.0
1.4 Frames using truss as hinge forcing mechanism Hinge forms in columns		1.0	3.0
1.5 Frame designed by Alternate Design Method		5.0	1.0
1.6 Vertical Cantilever		2.0	2.0
2.0 ECCENTRIC BRACED FRAMES			
2.1 Eccentric Braced frame Strength Design Method Flexural Mode			
i) Adequate no. of hinges		0.8	4.0
ii) Inadequate no. of hinges		1.2	3.0
2.2 Eccentric Braced frames Strength design Method Shear Mode			
i) Adequate no. of hinges		1.0	3.0
ii) Inadequate no. of hinges		1.4	3.0
3.0 CONCENTRIC BRACED FRAMES			
3.1 Concentric braced frame capable of plastic yielding in tension and compression		1.4-1.6	4.0-5.0
3.2 Concentric braced frame Capable of plastic yielding in tension and controlled compres sion buckle yield		1.7-2.0	3.0-3.9

TYPE OF STRUCTURE	DIAGRAM	S	μ
3.3 Concentric braced frame capable of plastic yielding in tension only			
i) single storey		2.0	3.0
ii) two or three storeys		2.5-3.0	1.8-3.0
4.0 SHEAR WALLS			
4.1 Steel plate wall capable of yielding in flexure		1.2	3.0
4.2 steel plate wall capable of yielding in shear		1.6	3.0
5.0 STEEL TANKS			
5.1 Steel tanks on ground			
i) ductile skirt pedistal		1.6	3.0
ii) stepping foundations		2.0	2.0
iii) tensile yielding of holding down bolts		1.6	3.0
iv) fully elastic		5.0	5.0
5.2 Elevated steel tanks Support frame as for relevant structure above tank to remain elastic		--	--
6.0 CHIMNEYS STACKS etc.			
6.1 Ductile skirt pedistal		1.6	3.0
6.2 Stepping foundations		2.0	2.0
6.3 Tensile yielding of holding down bolts		1.6	3.0
6.4 Guyed		5.0	1.0
6.5 Fully elastic cantilever		5.0	1.0