

Practice Note PN19

Seismic Resistance of Pressure Equipment

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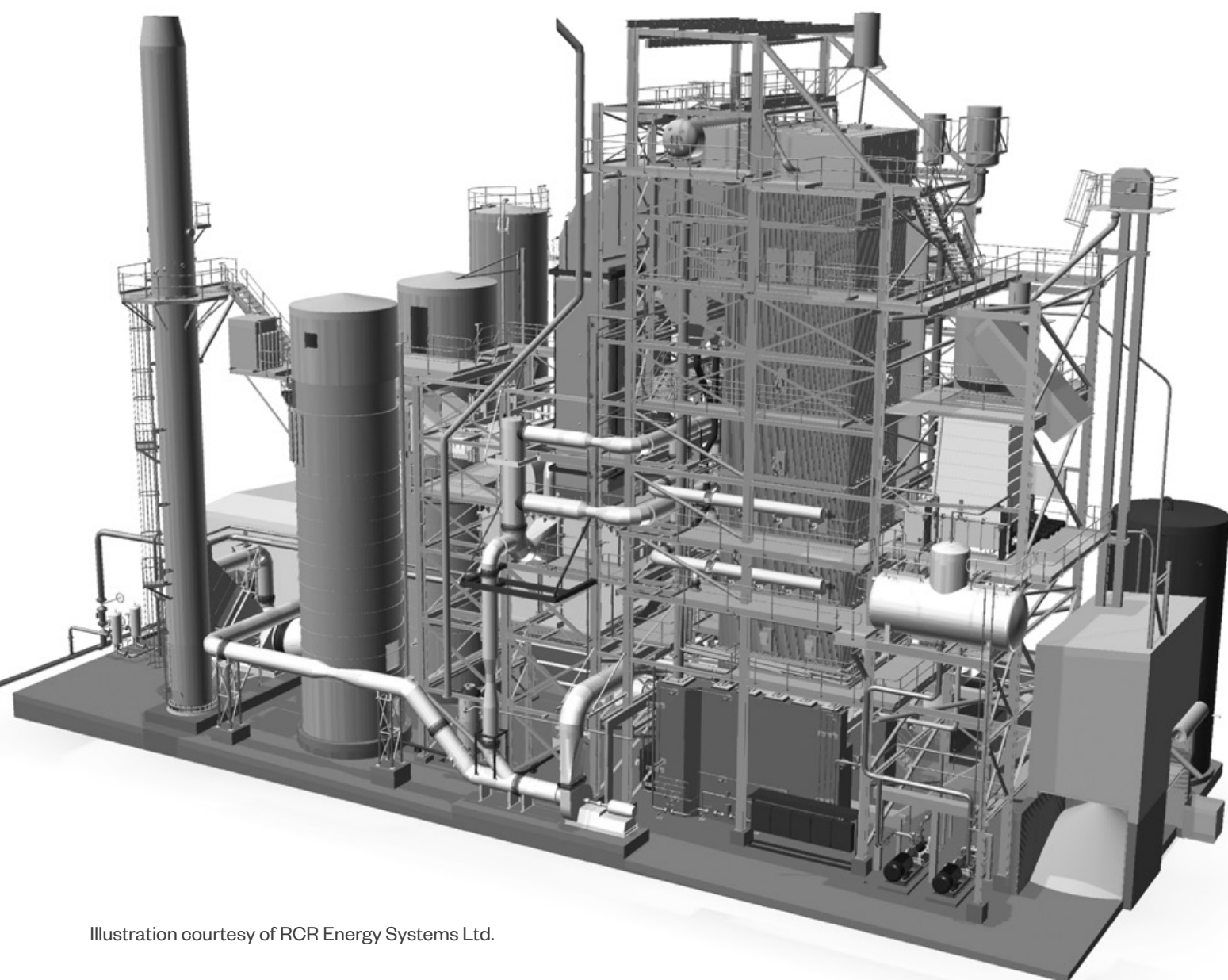


Illustration courtesy of RCR Energy Systems Ltd.

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TERMINOLOGY AND ABBREVIATIONS

ACPB	Approved Code of Practice for the Design, Safe Operation, Maintenance and Servicing of Boilers	MRF	Moment Resisting Frame
ACPPE	Approved Code of Practice for Pressure Equipment (Excluding Boilers)	MRSA	Modal Response Spectrum Analysis
ACENZ	Association of Consulting Engineers New Zealand	NB	Nominal Bore (of a pipe)
ALA	American Lifelines Alliance	NITHA	Numerical Integration Time History Analysis
APE	Annual Probability of Exceedance (AS/NZS 1170)	NZBC	New Zealand Building Code (First Schedule to the Building Regulations)
API	American Petroleum Institute	NZIA	New Zealand Institute of Architects
AS	Australian Standard	NZS	New Zealand Standard
ASCE	American Society of Civil Engineers	NZSEE	New Zealand Society for Earthquake Engineering
ASME	American Society of Mechanical Engineers	PE	Pressure Equipment
BA	Building Act 2004	PECPR Regs	Health and Safety in Employment (Pressure Equipment, Cranes, and Passenger Ropeways) Regulations 1999
CPEng	Chartered Professional Engineer	Pressure equipment in this Practice Note has the meaning given to it in the Pressure Equipment, Cranes and Passenger Ropeways (PECPR) Regulations. It includes boilers, boiler piping, compressors, fired heaters, gas turbines, steam turbines, hot water boilers, piping components, pressure fittings, pressure piping, pressure vessels, pumps, and steam engines.	
DWL	Design Working Life	QA	Quality Assurance
ESA	Equivalent Static Analysis	SAM	Seismic Anchor Motion
FEMA	Federal Emergency Management Agency (United States)	SDPP	Seismic Design of Petrochemical Plant
HD	Hold Down (bolts)	Secs	Seconds
HSE Act	Health and Safety in Employment Act 1992		
HVAC	Heating, Ventilating, Air Conditioning		
IL	Importance Level (AS/NZS 1170)		
LFRS	Lateral Force Resisting System		

SLS Serviceability Limit State

SQUG Seismic Qualification User Group
(useful experience collected by
American Electric Power Research
Institute)

Shall and Should: The word “shall” in this
Practice Note indicates a mandatory
requirement when claiming
compliance with this Practice Note.
The word “should” indicates a
recommendation.

SRSS Square root of the sum of the squares

SSI Soil Structure Interaction

TA Territorial Authority (unit of local
government)

ULS Ultimate Limit State

Worksafe WorkSafe New Zealand, formerly part
of the Ministry of Business, Innovation
and Employment, and before that, the
Department of Labour

WSD Working Stress Design

δ Deflection of a member

Δ Deflection of an entire structure

μ , μ Structural Ductility Factor

ξ Damping (% of critical)

1.0 INTRODUCTION

1.1 Introduction

Predicting earthquake performance for buildings, building-like, and non-building structures is a skill. Applying earthquake design according to any standard requires a method involving many approximations to arrive at the equations in the standard. While the design engineer must select the numerical values carefully, undue levels of accuracy within the calculation method are not warranted.

Design engineers need to consider the assumptions the structural design is based on when applying the standard to nonbuilding industrial structures. As there is little research into the way these structures perform in earthquakes, design engineers must develop a feel for how nonbuilding structures behave.

A conservative design approach for these structures is warranted until their behaviour is more fully documented and researched.

1.2 Brief history of seismic resistant pressure vessel design

This Practice Note is based on a paper Graeme Lindup presented to the New Zealand Society for Earthquake Engineering Technical Conference in 2006 and published in the Society's Bulletin in September 2007.

That paper used the 1997 American Society of Civil Engineers (ASCE) Task Committee on Seismic Evaluation and Design of Petrochemical Facilities "Guidelines for Seismic Evaluation and Design of Petrochemical Plants" and adapted these for the use with NZS 1170.5. Although aimed at petrochemical plants, the ASCE guidelines were able to be used for other industrial plants that have nonbuilding structures not similar to buildings.

The ASCE recommendations have been incorporated into other design codes commonly used for industrial facilities' design in the United States of America. These include ASCE 7-02, Section 9.14 "Nonbuilding Structures" and FEMA 450, Chapter 14 "Nonbuilding Structure Design Requirements". A second edition of the ASCE guidelines was published in 2010 and can be purchased through their online shop.

During 2007 engineers engaged in the geothermal power industry advised Engineering New Zealand that NZS 1170.5 was written from the point of view of habitable buildings which were being designed using "limit state" methods. These methods were notably different from the "working stress" methods mechanical engineers use for designing boilers, pressure vessels and pressure piping.

In the same year a project to write a New Zealand Standard on pressure equipment suitable for international standardisation purposes came to the attention of Engineering New Zealand. The need to include seismic provisions coincided with the matters arising in the geothermal industry.

Attention focused on the Ministry of Business, Innovation and Employment's (MBIE) Approved Code of Practice for Pressure Equipment (Excluding Boilers) 2001 (ACPPE) seismic provisions which mixed limit state and working stress concepts in a way that could be interpreted to give designs that would underperform in a dangerous manner. The Engineering New Zealand Engineering Practice Board drew this problem to MBIE's (now WorkSafe) attention, who heeded the warning and agreed to consider seismic resistance in a revision of their ACPPE (2001). MBIE (now WorkSafe) later agreed to extend consideration of the Engineering New Zealand work to a revision of their Approved Code of Practice for the Design, Safe Operation, Maintenance and Servicing of Boilers.

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Engineering New Zealand took the view that the Lindup paper was the best available guidance on the problem, and promoted its use within the pressure equipment industries.

The need to get overseas structural and mechanical design engineers to be familiar with the 1976 version of NZS 4203 with its preference for strength design for the impending “Think Big” projects led to the commissioning by the Ministry of Energy and subsequent publishing in 1981 of the document “Seismic Design of Petrochemical Plants” (SDPP). NZS 4203, revised in 1984 and in 1992, moved progressively to a limit state approach to the design of new buildings. This has been continued with the more recent series of “Structural Design Actions” standards AS/NZS 1170 Parts 0, 1, 2 and 3 and NZS 1170.5 “Earthquake Actions – New Zealand”.

1.3 Scope

This Practice Note is an attempt to integrate the Structural Design Actions standards setting out the minimum requirements for structural actions (AS/NZS 1170) with guidelines incorporated into overseas standards for the design of the nonbuilding structures not similar to buildings commonly found in industrial and petrochemical plants. It applies to all new pressure equipment constructed or installed in New Zealand.

Pressure equipment may be found, for example, in hotels, hospitals, and educational institutions.

This Practice Note updates the Lindup paper, uses relevant legislative requirements and updates from members of the Practice Note Working Party to develop design methods for the typical components of an industrial or petrochemical plant. Typical seismic coefficients have been determined in Appendix E to illustrate the design methods.

The worked examples in Appendix E feature pressure equipment. The principles that are discussed are also applicable to other types of heavy industrial plant such as refrigeration plant and electrical transformers.

Appendix D is intended to apply to pressure equipment alterations within existing plant.

Designers of some smaller pressurised items which do not require seismic design or design verification under the ACPPE may benefit from the technical guidance in this Practice Note's guidance.

1.4 Purpose

This Practice Note provides guidance to engineering designers and design verifiers to:

- Ensure the continuing containment of pressurised fluids during and after the design earthquake
- Derive seismic loads on mechanical parts for use in working stress design (WSD) after using NZS 1170.5 to determine the limit state loads applicable to structures
- Ensure that structures and foundations supporting pressure equipment are designed in accordance with the needs of the pressure equipment
- Ensure rotating equipment continues to function or safely stop during or after the design earthquake.
- Integrate new equipment into existing plant, and repair or modify existing plant, according to the law.
- Provide design statements templates for the certifying of supports and foundations for pressure equipment.

2.0 REGULATORY ENVIRONMENT

In general, all work on New Zealand soil for industrial plant is subject to the:

- Resource Mangement Act 1991
- Building Code, (a Schedule to the Building Act's Building Regulations)
- Pressure Equipment, Cranes and Passenger Ropeways Regulations 1999 (PEOPR) made under the Health and Safety in Employment (HSE) Act 1992
- Hazardous Substances and New Organisms (HSNO) Act 1996.

The items that come under the PEOPR Regulations and the HSNO Act are also subject to mechanical design considerations, the appropriate approvals and design verification. These requirements are set out in approved codes of practice (ACOP) or compliance guides produced by the WorkSafe for the HSE Act or the Environmental Protection Authority for the HSNO Act.

This Practice Note is expected to be referenced as a means of compliance in the revised ACPPE (2001).

New Zealand loadings standards have been written with an emphasis on the requirements for new building and nonbuilding structures similar to buildings (Clause C1.1 of NZS 1170.5 Supp 1). New Zealand loadings standards have been developed based on building orientated research and on building performance in service and during severe winds and earthquakes.

The use of loadings standards and the extent of the Building Act's compliance requirements can become problematic to designers of industrial plants. This is because many of the structures involved do not resemble a building. Figure 1 shows typical industrial plant items. Further examples are included in Appendix A.



Figure 1. Horizontal pressure vessel on short pedestals

Structures typically within industrial complexes can have:

- Little redundancy in their structural load paths
- Little or no cladding
- No internal partitions
- Few structural connections between members to produce the level of damping assumed to be present for buildings.

Some structures support equipment weighing many times their own weight. The equipment may retain pressure and/or contain hazardous materials. Their design is governed within New Zealand by the PECPR Regulations and/or the Hazardous Substances and New Organisms Act (HSNO).

The standards mechanical engineers use to design these items remain in the domain of working stress methods. The methodology of translating the ultimate earthquake base shear equations developed in the limit state based Loading Standard into an acceleration able to be used by a mechanical engineer for the design of their equipment or pressure piping has become uncertain as there is now no guidance given on this in the Loading Standard or its commentary.

An earlier version of the Building Code, Clause B1 "Structures" Verification Method VM1 referred the designer to NZS 4203:1984 for items that are not designed by the limit state method. However, this reference was removed in the changes to the Compliance Document for the Building Code Clause B1 Structure released at the end of 2008.

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The primary focus of the New Zealand Loading Standards on buildings and their performance under wind, snow and earthquake has led to differences in the way these Standards' seismic aspects are interpreted and applied to nonbuilding structures not similar to buildings, pressure equipment and distributive systems typically found in industrial plant.

3.0 STRUCTURAL SYSTEMS

3.1 General

Structures found in industrial and petrochemical facilities support non-structural process, mechanical and electrical items located above ground. These supporting structures are the primary means ground shaking from an earthquake is introduced to these non-structural items.

The way the structures respond to earthquakes directly influences the level of shaking non-structural items experience during a major earthquake. This affects the structures' classification and the approach for earthquake actions set out in the ASCE guidelines (2010).

3.2 Building Code provisions

The Building Code classifies structures into seven categories:

- Housing
- Communal Residential
- Communal Non-residential
- Commercial
- Industrial
- Outbuildings
- Ancillary

While the first four categories will not typically be present within an industrial facility, the last three categories may be present within a large plant's boundaries. To avoid confusion, in this Practice Note, any use of the words *Industrial*, *Outbuilding* or *Ancillary* with regard to the Building Code classification (items 5-7 in the above list) is italicised.

3.3 Types of structure

Industrial structures can be separated into two structural types:

- Building structures
- Nonbuilding structures.

The "Nonbuilding structures" divide into two sub-categories:

- Nonbuilding structures similar to buildings
- Nonbuilding structures not similar to buildings.

Each structure should have its applicable building code category and structural type determined before detailed structural analysis is undertaken. A diagram showing the process to ascertain the structural type has been included at the end of this section. See Figure 2.

3.4 Building structures

Building structures are those the building code and associated standards can apply to directly without modification. In industrial sites, these are typically administration buildings, buildings providing weather and/or hygienic protection to people who produce, repair or store goods, substations, maintenance buildings, shelters, compressor houses etc and normally are single storey structures.

If these structures support pressure equipment within their framing or on their foundations, their design is covered by both the Building Act and the PECPR Regulations. Thus parts of this Practice Note apply when determining their seismic actions. However, their seismic response is well documented elsewhere and is not covered in this Practice Note.

Their NZBC classification will generally be *Industrial* or *Outbuilding*.

3.5 Nonbuilding structures

All structures within industrial/petrochemical facilities (other than actual buildings) are typically nonbuilding structures. Some have structural systems that resemble those of buildings such as multi-storey modules or pipe racks. These types of structure are classified as nonbuilding structures similar to buildings.

Other structures whose structural systems do not resemble buildings are classified as “nonbuilding structures not similar to buildings” include circular storage tanks and horizontal pressure vessels.

The Building Code classifies most of these structures as “*Ancillary*”. Photographs of some typical industrial nonbuilding structures are included in Appendix A.

3.5.1 Nonbuilding structures similar to buildings

These structures (generally unclad) have Lateral Force Resisting Systems (LFRS) similar to those of buildings; they can be analysed according to the methods described in Section 6. Examples of nonbuilding structures similar to buildings found in typical industrial facilities include:

- Multilevel moment resisting frames or braced frames supporting non-structural items. Items that individually weigh less than 20% of the structure’s total weight, such as stairs, ducts, hoppers and/or equipment such as heat exchangers, horizontal pressure vessels, fall into this category
- Pipe racks with LFRS that are moment resisting frames (usually in the transverse direction) or braced frames (usually in the longitudinal direction)
- Rectangular vertical furnaces or boilers.

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Typically the structures named in the first two bullet points support pressure equipment within their framing or on their foundations, and their design must satisfy both the Building Act and the PECPR Regulations.

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Typical third bullet point structures are pressure equipment which must satisfy only the PECPR Regulations. However, their foundations must satisfy PECPR and the Building Act. Walkways and ladders providing access must satisfy the Building Act, but if they also support pressure equipment such as pressure piping they must also comply with the PECPR Regulations.

3.5.2 Nonbuilding structures not similar to buildings

These structures can be subdivided into four subcategories:

1. Rigid structures with a natural period of vibration $T_1 < 0.06$ seconds (secs). These include skids, pumps or compressors supported by ground slab foundations, and squat horizontal vessels supported on short stiff piers.

A design seismic coefficient can be determined from the equivalent static analysis method described in Section 6.4. This assumes system damping has little effect and a structural ductility factor $\mu = 1.0$ or 1.25 depending on whether any limited ductility can be utilised.

2. Flat-bottomed tanks. These respond to earthquakes in a unique manner. The NZSEE document "Seismic Design of Storage Tanks" (2009) and API 650 Appendix E both provide appropriate seismic analysis methods.
3. Others. Examples include skirt supported vertical vessels, spheres, guyed structures and vertical fired heaters supported on braced legs. These are generally either slender or vertically irregular and frequently require a modal response spectrum analysis (see Section 6.5). Appendix E Example E1 illustrates the proposed equivalent static analysis design method for a vertical skirt supported pressure vessel.
4. Combination structures. These support non-structural items (such as process related equipment or piping) whose weight exceeds 20% of the overall structure's weight. Examples would include large horizontal pressure vessels, heat exchangers or fin-fan coolers supported above grade on braced or moment resisting frames.

The analysis method depends on whether the non-structural element is rigid or flexible. The supporting structure is generally building-like in structural form. Appropriate analysis methods are discussed in Section 7. Appendix E Example E3 illustrates a proposed design method for these structures.

Self-supporting equipment items or rigid body structures at grade generally make up the majority of nonbuilding structures not similar to buildings in industrial plants. Example E4 in Appendix E illustrates the proposed design method for these structures.

Type 1 structures typically are pressure equipment governed by the PECPR Regulations, with foundations governed by both the Building Act and the PECPR Regulations.

Type 2 structures are usually not pressure equipment, and their design is covered by the HSNO Act. Their foundation design must satisfy the HSNO Act and the Building Act.

Type 3 structures may be pressure equipment and if so must comply with the PECPR Regulations. Their foundation design must satisfy the Building Act and the PECPR Regulations.

Type 4 structures that support pressure equipment within their framing or on their foundations must satisfy both the Building Act and the PECPR Regulations. If not, their design must comply only with the Building Act.

Access up and around Type 3 or 4 structures, e.g. walkways, platforms and ladders, must be designed in accordance with the Building Act. If the access ways also support pressure equipment such as pressure piping, they must comply with the Building Act and the PECPR Regulations.

3.6 Parts and components

When a non-structural item's combined weight is less than 20% of the structure's total weight, it shall be considered a part or as a component. The seismic forces can be determined using either NZS 1170.5 Section 8 "Requirements for Parts and Components" or the simplified method given in NZS 4219:2009. Appropriate analysis methods are discussed below in Section 8.

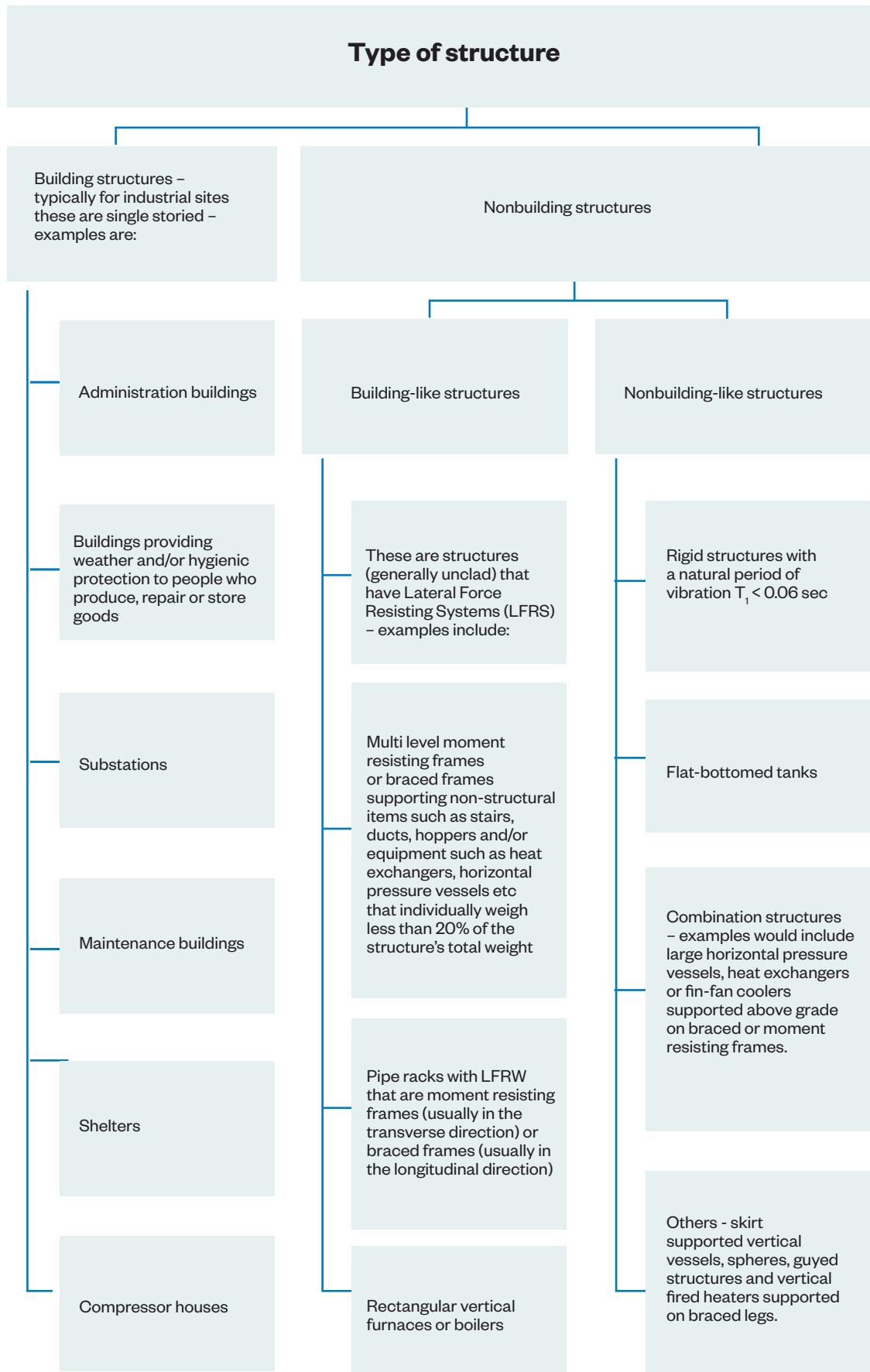
Examples of non-structural items classed as either a part or a component typically found in industrial facilities include:

- Components such as pressure equipment (e.g. horizontal vessels, exchangers, piping, mechanical, electrical and other items like tubing, cable tray/ladders, lights, conduit, instruments, low pressure pumps and ductwork).
- Parts such as access platforms and ladders.

Appendix E, Examples E2, E5, E6 and E7 illustrate the proposed design method based on NZS 1170.5 Section 8 for these items.

The supports for pressure equipment must comply with both the Building Act and the PECPR Regulations. Supports for components such as lights or cable ladders must comply only with the Building Act.

Figure 2: Diagram illustrating the structural type selection process.



4.0 APPLICABLE DESIGN STANDARDS

Industrial structures are generally constructed using the same materials and standards used in the New Zealand construction industry.

Special structures e.g. storage tanks and those structures specifically excluded by AS/NZS 1170.5 have their own legislation and design methodology.

A selected list of publications that may help with interpreting legislation or outline design standards is below.

1. AS/NZS 1170 *Structural Design Actions*
2. NZS 3404 *Steel Structures Standard*
3. NZS 3101 *Concrete Structures Standard*
4. NZS 4230 *Design of Reinforced Concrete Masonry Structures*
5. AS/NZS 4600 *Cold-formed Steel Structures*
6. NZSEE *Seismic Design of Storage Tanks*
7. API 650 *Welded Storage Tanks for Oil Storage*
8. ASME *Boiler and Pressure Vessel Code*
9. NZS PD 5500 *Specification for Unfired Fusion Welded Pressure Vessels*
10. ASME B31.1 *Code for Power Piping*
11. ASME B31.3 *Code for Chemical Plant and Petroleum Refinery Piping*
12. ASME B31E *Standard for the Seismic Design and Retrofit of Above-Ground Piping Systems*
13. ASME B31.5 *Code for Refrigeration Piping and Heat Transfer Components*
14. ASME STS-1 *Steel Stacks*
- V4 April 2016 | 15. *Intentionally left blank*
16. AS 1210 *Pressure Vessels*
17. AS 4343 *Pressure Equipment – Hazard Levels*
- V4 April 2016 | 18. WorkSafe Approved Code of Practice for Pressure Equipment (Excluding Boilers)
19. WorkSafe Approved Code of Practice for the Design, Safe Operation, Maintenance and Servicing of Boilers
- V4 April 2016 | 20. *Intentionally left blank*

Notes:

V4 April 2016 | NZS 4203 has been referenced in this document for historical reasons.

AS 1210 has been revised recently to align with the AS/NZS 1170 set of standards in respect of wind, snow and earthquake loading, and is used as a guide for the design of pressure equipment generally, not just vessels.

5.0 SEISMIC DESIGN PHILOSOPHY

There are a number of issues to be considered before undertaking the detail design of industrial plant. Some of these relate to AS/NZS 1170 standards and others to the observed and recorded behaviour of industrial structures and components during earthquakes.

5.1 Design life, design working life, durability life or intended life?

A project's Basis of Design document is the usual document that specifies the process design life for an industrial facility. That is the most probable time period for which the feedstock, for example natural gas, geothermal energy or forestry, will be available. For most petrochemical facilities this is typically taken as 25 years but it can be less. For other facilities, the plant may be intended to be operated to some unknown long term date in the future.

5.1.1 AS/NZS 1170.0 requirements

The DWL to be determined which is used to select the probability of exceedance of different actions such as wind, snow and earthquake. The AS/NZS 1170.0 definition is:

Duration of the period during which a structure or structural element, when designed, is assumed to perform for its intended purpose with expected maintenance but without major structural repair being necessary.

It is theoretically possible to extrapolate these requirements down to justify very small seismic design loads for very short DWLs for pressure equipment. However, if a large earthquake occurred, the public is unlikely to tolerate the fact that so little consideration was given in design to the possible consequences. The equipment designer should therefore follow the lead given in NZS 1170.0 Table 3.3 which specifies the lower limits on annual probabilities of exceedance. It is thought likely that future revisions of this Standard will emphasise this aspect more.

5.1.2 Building Act requirements

The Building Act requires the owner or their agent to obtain building consents from the appropriate territorial authority for non-exempt structures. The intended life of these is required to be nominated in the application for any building consent. To comply with the Building Act and the Building Code, a structure is deemed to have an "indefinite life". However, if a structure's intended design life is less than 50 years, a specified intended life may be nominated in the application for its building consent.

At the end of a structure's specified intended life, owners must tell the territorial local authority at that time whether they propose to demolish, extend its life or change its use. Upgrading or refurbishment may be required if the structure is not to be demolished. Therefore, specifying a limited life may lead to a condition being attached to the building consent requiring alteration, removal or demolition at the end of the specified life. Consequently some plant owners do not want to have this limiting the use of their plant after the specified life and require the wind and earthquake forces to be determined using a DWL = 50 years. Others consider this to be unwarranted pre-investment and require the minimum values to be used.

The Building Code sets out minimum durability lives for all items making up a structure. The primary systems providing structural stability must have a durability life equal to either the specified intended life or 50 years. This also applies to non-structural components that are difficult to replace or whose failure would go undetected in normal use and maintenance. Most new buildings are designed having an AS/NZS 1170.0 DWL of 50 years.

Non-structural components that are moderately difficult to replace or whose failure would go undetected in normal use but would be detected during normal maintenance must have a minimum durability life of 15 years. Non-structural components that are easily replaced and whose failure would be detected in normal use must have a minimum durability life of five years.

These non-structural components are secondary structural items, and are not normally designed using wind, earthquake or snow actions determined using a DWL equal to their durability lives. Instead, their design reflects the need to protect human life or the ability of people to safely evacuate in the event of failure.

5.1.3 HSE Act – pressure equipment

Designers of new pressure equipment, or those making alterations to existing pressure equipment are required to state the hazard level and the design life in the equipment's design documents. The design life should also be specified in the owner's operating and maintenance instructions provided with the equipment.

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Paragraph removed

For pressure vessels, AS 1210:2010 Appendix J outlines what the DWL, according to AS/NZS 1170.0 should be. It also provides a relationship between the Importance Level (IL) derived from AS/NZS 1170.0 and the Hazard Level from AS 4343.

The Australian Standard, AS 1210, provides guidance on what is an acceptable DWL for pressure equipment. Although AS 1210 excludes piping from its scope, Engineering New Zealand recommends AS 1210's DWL guidance is extended to pressure piping. In addition, AS 1210 Appendix J refers to AS/NZS 1170.0 Table 3.3 for the annual probability of exceedance rather than those in Appendix F which apply to Australia. Thus the design load levels for wind, snow and earthquake would be acceptable for New Zealand. Recommended minimum seismic coefficients are provided below in Appendix H. How these values have been determined is outlined in Appendix H.

5.1.4 HSE Act requirements – boilers

Appendix H recommends minimum seismic coefficients and shows how these have been determined.

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However, water tube package boilers have a minimum coefficient of 0.6 if the NZS 1170.5 Hazard factor Z is less than 0.45.

5.1.5 Summary

Before AS/NZS 1170.5 was issued, minimum seismic coefficients (based on NZS 4203:1992) assumed all items had a 50 year design life. After AS/NZS 1170.5 was issued, however, it became difficult for pressure equipment design engineers to agree on the required DWL. The mechanical design life was equated with the concept of durability life, which was used as the DWL when determining design earthquake actions for mechanical items and pressure equipment. This aimed to keep the seismic design forces to a minimum.

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AS 1210:2010 requires all pressure vessels to be assigned a minimum Design Working Life (DWL) of 25 years, so there is now a clear precedent set for the minimum Design Working Life and this has been carried through this Practice Note.

5.1.6 Recommended design working life

For the industrial plant design engineer, the term “design life” has multiple meanings. The above legislation requires the applicable parts of a plant to have a design life, intended life and a durability life to be set during its design phase.

The DWL along with the chosen IL sets the required return periods for wind, snow and earthquake forces for the design ultimate and serviceability limit states. It has a major effect on the required strength, displacements and foundation sizing requirements.

Table 1 gives a recommended approach based on the proposed length of time the plant will be operating.

Table 1: Recommended Design Working Lives for Earthquake Load Level Determinations

PROCESS DESIGN LIFE OR PERIOD OF FEEDSTOCK AVAILABILITY	MECHANICAL DESIGN LIFE FOR PECPR ITEMS ⁽⁴⁾	DESIGN WORKING LIFE/ INTENDED LIFE FOR BUILDING ACT COMPLIANCE
< 6 months	25 years ⁽³⁾	6 months ⁽¹⁾
≤ 25 years	25 years	25 years ⁽¹⁾
> 25 years	25 years	50+ years ⁽²⁾

Notes:

1. The owner should agree to the specific intended life for the items covered by the Building Act.
2. All items must be designed for the earthquake with a return period determined using a DWL = 50 years.
3. To comply with AS 1210:2010.
4. If the mechanical design standard's design life is greater, this shall apply.

5.2 Pressure equipment hazard levels

The hazard level of pressure equipment is usually determined using AS 4343. The nature of the contained substance, various operating parameters, diameter or volume and other variables combine to set a hazard level that can range from A (high) to E (negligible).

The hazard level given to each pressure equipment item is used by the industrial structural engineer usually in consultation with safety and risk engineers. Between them, they determine the nature of the hazard if there were leaks following a major earthquake event. The number of items with a hazard level of A or B then can be used to determine whether the IL(s) should be set at a value greater than 2, the value for a normal facility.

5.3 Importance Levels

Tables 3.1 and 3.2 of AS/NZS 1170.0:2002 are used to determine what IL applies for each component, structure or the plant as a whole. A decision should be made early in the design process to identify the structures or items in the plant that are to have a separate IL applied or whether a single IL shall apply for the whole site. Large plants may have enough separation to allow blocks of plant to have a common IL.

Table 2 compares SDPP Table 2.2, NZS 4203:1992 Table 2.3.1, the ASCE Guidelines (2010) Table 2.1 and AS/NZS 1170 and shows how each document endeavours to categorise the importance of a range of structures and components. The relationship in Table 2 between the AS 4343 Hazard Level and AS/NZS 1170.0 IL is the same as that set out in AS 1210 Appendix J.

NZS 4203:1992 and AS/NZS 1170 have five classifications. The more specialised SDPP and the ASCE guidelines (2010) have only four classifications.

A mainstream hydrocarbon gas conditioning facility upstream of the main gas distribution network usually has some items, if not the whole plant, classed as having an IL of 4. However, if the plant is a minor network supplier and/or is in a remote area, or a downstream industrial facility it would have an IL of 2 or 3. However, a fire in a Takaka dairy factory in June 2005 that caused the Takaka township's evacuation shows careful consideration of the contained materials within a plant is necessary. IL 4 covers facilities containing hazardous materials capable of causing hazardous conditions that extend beyond the plant. This was the case in the Takaka fire.

The ILs set should be decided in conjunction with the plant's owner and/or operator using a risk based approach. Safety and Risk engineers can be consulted to review the risk approach taken.

The chosen IL is used with AS/NZS 1170.0:2002 Table 3.3 to select the applicable return periods for the Ultimate Limit State and Serviceability Limit States and from Clause 2.1.4, NZS 1170.5:2004 determines how many serviceability states must be considered.

Table 2: Comparison of Structure Classifications

SDPP SEISMIC CLASSIFICATION	NZS 4203:1992 STRUCTURE CLASSIFICATION	ASCE OCCUPANCY CATEGORIES	AS 4343 HAZARD LEVEL	AS/NZS 1170.0 IMPORTANCE LEVELS	NZS 1170.5 PART CLASS	DESCRIPTION OF STRUCTURE OR ITS INTENDED ROLE
-	-	-	A	5		<ul style="list-style-type: none"> Exceptional structures Structures that have special functions Structures whose failure poses catastrophic risk to a large area Structures whose failure poses catastrophic risk to a large number of people Major dams Extreme facilities Supporting pressure equipment with a high hazard level
A	I	Essential	A	4	P.1 or P.4	<ul style="list-style-type: none"> Preservation of life Severe impact on society Components critical to safe plant shutdown Components whose failure represents a severe hazard beyond plant In plants of national importance, components that cannot be readily repaired, replaced or by-passed Post disaster functions or dangerous activities Utilities or emergency supplies or backup facilities for other IL 4 structures Facilities containing hazardous materials capable of causing hazardous conditions that extend beyond plant Supporting pressure equipment with a high or average hazard level

SDPP SEISMIC CLASSIFICATION	NZS 4203:1992 STRUCTURE CLASSIFICATION	ASCE OCCUPANCY CATEGORIES	AS 4343 HAZARD LEVEL	AS/NZS 1170.0 IMPORTANCE LEVELS	NZS 1170.5 PART CLASS	DESCRIPTION OF STRUCTURE OR ITS INTENDED ROLE
B	II	Substantial Hazard	B	3	P.2	<ul style="list-style-type: none"> Contains people in crowds Components' failure represents a severe hazard within plant Major structures affecting crowds Public utilities not designated as post-disaster Facilities not designated as post-disaster containing hazardous materials capable of causing hazardous conditions that do not extend beyond plant Supporting pressure equipment with an average hazard level
	III	Other				<ul style="list-style-type: none"> Public buildings with contents of high value Components that cannot be readily repaired, replaced or by-passed Major structures affecting crowds
C	IV	Low Hazard	C	2	P.3 (See Note 1)	<ul style="list-style-type: none"> Normal structures Critical components that can be readily repaired, replaced or by-passed Supporting pressure equipment with a low hazard level
D	V	-	D and E	1	P.3 (See Note 1)	<ul style="list-style-type: none"> Secondary nature Components other than Class I-IV above whose failure represents a minimal safety hazard Supporting pressure equipment with an extra low or negligible hazard level Minor structures whose failure is not likely to endanger human life

Note:

¹ For classification of parts that have Hazard Levels C, D or E in AS 4343, and if the criteria specified in Table 8.1 of NZS 1170.5 for Classifications P.1, P.2 or P.4 are applicable, then a Part Risk Factor $R_p = 1.0$ shall be used.

Examples E1 and E2 in Appendix E illustrate the effects of setting IL = 4. Examples E3 to E7 illustrate the proposed design method for the more common IL = 2 structures.

5.4 Strength considerations

5.4.1 Building Act

Unless the process demands more stringent requirements, all structures' performance should meet the AS/NZS 1170:2002 requirements. They shall be designed to have adequate strength, stability, ductility and stiffness under seismic effects to satisfy the:

- Ultimate Limit State (ULS), so in an ULS earthquake, the structural system does not collapse
- Serviceability limit state SLS2, so for an SLS2 earthquake, structures or items given an IL = 4 or 5 from Table 3.2 remain operational
- Serviceability limit state SLS1 so for an SLS1 earthquake, the structure/item does not require repair.

The serviceability limit states SLS1 and SLS2 for main structures are deflection based checks, not ones where the strength capacity is normally checked. This is not the case for parts or components: refer to Section 5.5 of this Practice Note.

It should be emphasised to owners that the purpose of design code levels is to prevent loss of life and not to prevent damage to property. If the owners want to prevent earthquake damage to their plant it is likely the design will need to consider a longer return period design earthquake. An estimate of the maximum credible earthquake can be part of the scope of any site specific seismic hazard study. Once this is known, the owners and their design team can decide on the approach they want to take. In lieu of a special study a 2500 year return period earthquake may be used as the maximum credible earthquake¹ – refer NZS 1170.5 supp 1 Section C2.1.

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5.4.2 Items coming under PECPR Regulations

Pressure equipment items are designed according to the working stress standards listed in ACPPE (2001) Schedule C. These standards deal mainly with pressure containment and strength, give little guidance on ductility and collapse, and do not recognise the concepts of ultimate and serviceability limit states.

The New Zealand seismic design requirements for pressure equipment were for many years set out in AS/NZS 1200:2000 Appendix I. Towards the end of 2015 a new edition of AS/NZS 1200 dropped references to a standard that had become obsolete and required seismic design requirements for New Zealand to be determined, as appropriate, from NZS 1170.5 with guidance from this Practice Note.

AS/NZS 1200:2000 required the seismic coefficient to be calculated from a loading standard (NZS 4203) which soon became obsolete, and also to be taken from a table, with the more severe value used in design. A corresponding process is now required, by calculating according to NZS 1170.5, making reference to the tabulated value in PN 19 Appendix H Table H1, and using the more severe value in design.

If a site-specific hazard study, including any proposal to elevate the equipment to a higher level, has been undertaken for the site where the pressure equipment is to be installed, the ACPPE Section 5.4.4 (10) allows the use of seismic coefficients derived from that study. The study must include appropriate geological and structural investigation, NZS 1170.5, and the guidance of this Practice Note.

The intention is that all pressure equipment shall be designed to have adequate strength and stability during earthquakes to satisfy the:

- Pressure containment limit state so that in a design period earthquake leak tightness and position retention are maintained. Complying with recognised pressure equipment standards demonstrates this.
- Functionality limit state so in an SLS2 return period earthquake items or systems given an IL 4 or 5 from AS/NZS 1170.0 remain operational.

1 As the term "maximum credible earthquake" is controversial it is best avoided by most authors.

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5.5 Limit State Objectives

The objectives for an ULS return period earthquake are:

- To avoid the structural system's collapse
- To avoid injury and loss of life
- To minimise damage to equipment
- To avoid collapse or loss of support of parts of structures classified as P.1, P.2, P.3 or P.4 in NZS 1170.5 Table 8.1
- To maintain vital services.

The serviceability limit state SLS2 objectives for the appropriate return period earthquake are:

- For the structural system to maintain operational continuity
- To avoid collapse or loss of support of parts of structures classified P.5 in NZS 1170.5 Table 8.1.

The serviceability limit state objectives SLS1 for the appropriate return period earthquake are:

- For the structural system and non-structural components to not require repair
- To avoid collapse or loss of support of parts of structures classified as either P.6 or P.7 in NZS 1170.5 Table 8.1.

The objectives' of the general serviceability limit state are to:

- Avoid disruption to any service
- Ensure deformations and vibrations are within acceptable limits for occupational comfort, process operations and material fatigue.

It should be noted that for major structures' design, the design serviceability limit state focuses on deformations and vibrations, and not on strength.

However, NZS 1170.5 Section 8 and NZS 4219 suggest that for parts or components with a classification of P.5 to P.7, the "serviceability level" earthquake assessment to meet general serviceability is also used for part or component strength design. It is also used for fixing these parts or components to the main supporting structure.

It is recommended all pressure equipment covered by the PECPR regulations should be designed for earthquakes having a return period corresponding to the Ultimate Limit State in NZS 1170.5. To avoid confusion, Part Categories P.5 to P.7 are not included in Table 2. Parts or components with an AS 4343 Hazard Level A, B, C, D or E shall be selected from Category P.1 to P.4.

5.6 Performance objectives

The structural performance objectives for buildings are set out in AS/NZS 1170.0 Clause 3.2.

In industrial situations these encompass:

- Structural integrity
- Containment
- Functionality.

5.6.1 Structural Integrity

Structures and components should not collapse or otherwise fail under the design loads determined for the site from AS/NZS 1170 or any site specific seismic hazard studies. In general, maintaining structural integrity does not mean maintaining the structure's functionality unless the set IL requires this.

Strength, ductility and deformation limits are the main considerations to ensure structural integrity performance requirements are met.

5.6.2 Containment

Structures and components containing hazardous materials should be designed so they do not permit significant releases into the environment during and after a major earthquake. Ensuring containment means strength, deformation limits and structural and mechanical details of elements with respect to potential leak paths must be considered at the design stage.

5.6.3 Functionality

Industrial structures and components that must operate after a major earthquake should be designed to maintain their designated function during and after such events.

Systems, such as emergency depressurising systems, flare lines, flare stacks, electrical supply systems, communication towers and fire fighting systems, and the like, require attention to strength, ductility, deformation limits, stress levels, structural details, seismic interaction and protection from falling or collapsing objects.

5.7 Ductility considerations

5.7.1 ULS ductility factors

Ductility, even for nominally ductile structures, is a vital requirement for structures to resist seismic loadings. The ductility limits chosen for each structure and its accompanying deformations and limit state, should be taken from the appropriate material design standard. If these are not available, from the appropriate material design standard, they should not exceed the requirements set out below. Ductility limits may also be demonstrated through independent testing.

The measure of a structure's ductility in response to seismic events is indicated by the value of its "Structural Displacement Ductility Factor". This is an assessment of the structure's overall ability to sustain cyclic inelastic displacements. NZS 1170.5 Clauses 2.2.1 to 2.2.4 give the allowable ranges of the ULS Structural Displacement Ductility Factor for four different types of structure responding to earthquakes:

- Brittle or elastic structures, $\mu_{uls} = 1.0$
- Nominally ductile structures, $1 < \mu_{uls} \leq 1.25$
- Structures with limited ductility have a μ_{uls} in the range of $1.25 < \mu_{uls} < 3$
- Ductile structures have a μ_{uls} in the range of $1.25 < \mu_{uls} \leq 6$

Before assigning the ULS Structural Displacement Ductility Factor the structural systems resisting seismic forces should be carefully considered using the appropriate material design standard. This is especially the case when designing nonbuilding structures not similar to buildings. See Section 3.5.2.

The value chosen also has a bearing on whether capacity design is required or is an option (NZS 1170.5 Section 5.6). Capacity design generally requires an additional amount of design effort to verify the structure and has a bearing on what methods are used to design verify it – see NZS 1170.5 Clause 5.3.1.

5.7.2 SLS1 and SLS2 ductility Factors

NZS 1170.5 Clause 4.3.2 sets out the limits for the values for μ for the two possible serviceability limit states:

- SLS1 $1.0 < \mu_{s1} \leq 1.25$
- SLS2 $1.0 < \mu_{s2} \leq 2.0$

If either the equivalent static or the modal response spectrum method of analysis is to be used for an SLS1 or SLS2 analysis, a value for μ needs to be assigned. NZS 1170.5 Equation 5.2(4) that sets out the serviceability limit state horizontal modal response spectrum has incorrectly left out the use of the factor k_{μ} . This error has been corrected in the verification method B1/VM1.

The higher the ULS Structural Displacement Ductility Factor used, the higher the value of a SLS1 or SLS2 Structural Displacement Ductility Factor may be justified within the limits given above.

5.8 Analysis methods

The equivalent static method of analysis (ESA) for seismic effects generally is appropriate. However, a dynamic analysis of irregular or slender structures may be required. The designer should use judgement in deciding whether a dynamic analysis is appropriate. NZS 1170.5 Clause 6.1.3.1 outlines the limitations to the ESA method for buildings and nonbuilding structures similar to buildings. Structures within petrochemical facilities are often inter-linked by platforms, large diameter piping and the like, and a modal response spectrum analysis (MRSA) of the total system may be more appropriate. The Clause 6.1.3.1 limitations on the ESA method's appropriateness can also be used for nonbuilding structures not similar to buildings.

5.9 Acceptance criteria

Limit state methods shall generally be used in establishing acceptance criteria for structures. All limit states shall comply with the method shown in AS/NZS 1170.0:2002 Section 7.

Components designed according to mechanical codes (such as piping, pressure vessels) generally use allowable or working stress methods to establish acceptance criteria. The structural engineer therefore needs to provide the mechanical engineer with seismic accelerations and displacements that can be accommodated within the mechanical acceptance criteria. This means the structural supporting system for the component and how it would behave under severe shaking must be considered.

Table 3 provides guidelines for the allowable deflection limits for serviceability limit states. AS/NZS 1170.0 Appendix C and some material design standards give guidelines for serviceability limit states.

NZS 4219 Clause 2.2 (c) requires components needed for operational continuity (classification P5) in IL = 4 buildings to be restrained so that the system can continue to perform its functions after an SLS2 earthquake. Clause 2.2 (d) requires all components to retain structural and operational integrity after an SLS1 earthquake.

5.10 Component review

Each system and its components should be assessed to determine their susceptibility to strong ground shaking. Many systems and components are seismically rugged, but those that are part of hazardous or essential elements need to be considered closely.

The non-structural component review aims to ensure performance meets the objectives described in Sections 5.5 and 5.6 of this Practice Note. The review may include considering records of experience, vendor data, analysis, testing or any combination of these.

Table 3: Suggested Drift Ratio Limits for the two Serviceability Limit States

State of damage	STRUCTURAL SYSTEM				
	Ductile MRF	Non-ductile MRF	MRF with infills	Ductile walls	Squat walls
SLS1 (no damage)	< 0.2%	< 0.1%	< 0.1%	< 0.2%	< 0.1%
SLS2 – (repairable light damage)	0.4%	0.2%	0.2%	0.4%	0.2%
SLS2 – (repairable moderate damage)	< 1.0%	< 0.5%	< 0.4%	< 0.8%	< 0.4%

5.11 Seismic interaction between components

Process, mechanical and piping design requirements and the desire to minimise plot space encourages placing components close together. This creates the potential for damaging interaction between structures and components during an earthquake especially in skid mounted systems. It is essential structural engineers review proposed layouts and address any concerns they have about interaction between plant items.

Impact and differential displacement generally have adverse effects on the performance of sensitive systems and components. Separating these items from each other is a basic design principle. If this is not practical, seismic interaction should be accounted for in the analysis and design.

Examples of potential structure and component interaction are:

- Tall vertical vessels with platforms or pipe in close proximity to each other
- Vessels with shared working platforms
- Interconnecting pipes and pipe ways
- Distributive systems that connect structures with different lateral force resisting systems (LFRS) or foundations. Piping for instance is often routed vertically from a high elevation on a pipe rack into (for example) the flanges or nozzles of turbines and vessels at grade. Large differential displacements of these pipes' supports during an earthquake may cause leaks at the flanges. This in turn could lead to possible fires and/or explosions.

Maximum horizontal deflections do not occur at the same instant as the earthquake induced peak accelerations. Also peak horizontal displacements for separate structures may not occur at the same instant but as the amount of inelastic behaviour increases, the way adjacent structures deflect with time becomes a complex issue.

Therefore when reviewing the physical interaction between adjacent structures, the NZS 1170.5 Section 7.4.1.2 requirement that no contact is made between adjacent structures with both deflecting at their design horizontal displacements should be complied with.

Good practice to avoid or minimise physical interaction involves:

- Setting out all structures as regular structures whenever possible
- Providing adequate space between components (this also helps for maintenance activities)
- Checking the maximum displacements and ensuring there is enough room between the structures and components they support
- Providing, in conjunction with the pipe stress engineer, support and restraint configurations for pipes to minimise transfer of load across flanges and couplings and to prevent pipes falling off their supports
- Providing, in conjunction with the pipe stress engineer, more flexible piping systems to mitigate the coupling effects of large diameter pipes spanning between structures
- Minimising the ULS inter-storey deflections to levels substantially less than the 2.5% allowed in NZS 1170.5 Section 7.

5.12 Redundancy within structures

Experience of earthquakes indicates the more load paths provided in a structure (i.e. greater redundancy) the better its seismic performance. Industrial and petrochemical facilities tend to consist of structures that lack redundancy. Consideration should be given to providing redundant structural systems to the maximum extent practical. If sufficient redundancy is unable to be provided, the chosen structural performance factor S_p should reflect this.

The designer should therefore be cautious of using material standard designated S_p values as these are typically applicable to building structures. They do not generally apply to the types designated as nonbuilding structures not similar to buildings described in Section 3.5. The use of $S_p = 1.0$ for the design of steel tanks as per NZSEE (2009) is an example of this, while a steel building such as a single storey portal frame structure could have a S_p value of either 0.7 or 0.9 (NZS 3404:1997 Clause 12.2.2.1).

5.13 P-Delta effects

When a structure is displaced laterally, P-Delta actions reduce the structure's resistance to further displacement in the same direction. For ductile structures, this becomes important in the ultimate limit state, as each time the inelastic range is entered, there is a tendency for the displacement to increase and it may be possible for sway mechanisms to form. If a sway mechanism develops during an earthquake, the structure may collapse if the earthquake is of sufficient duration.

P-Delta effects in a structure increase with:

- An increase in ductility demand on it
- The duration of the severe ground motions
- The inverse of the fundamental period of the structure.

The need to allow for P-Delta effects should be assessed as outlined in NZS 1170.5 Clause 6.5.2.

5.14 Horizontal torsion effects

NZS 1170.5 Clauses 5.3.2 and 6.3.5 set out the recommended minimum requirements for torsion load effects in buildings. These requirements are described in terms of a minimum eccentricity required between the applied seismic forces and the centre of mass. The 0.1b eccentricity is intended to allow for variations in structural properties, the distribution of mass, participation of non-structural elements and the effect of ground rotation about the vertical axis. Rigid diaphragms are required to transfer these eccentricities to the LFRS.

However, in industrial and petrochemical facilities, the mass location for most structures can be determined with reasonable accuracy and rigid diaphragms are usually not present. For these structures, the torsion effects are minimal. However, if the mass distribution cannot be determined with accuracy or the structure has a rigid diaphragm, an allowance should be made for accidental torsion.

For nonbuilding structures not similar to buildings, the allowance for accidental torsion, if required, shall be as follows. The centre of mass shall be assumed to have shifted in each horizontal direction from its calculated value by a distance equal to 5% of the structure's dimension perpendicular to the earthquake direction being considered.

5.15 Directions of earthquake forces

For ductile structures, NZS 1170.5 Clause 5.3.1.1 allows that the independent design for each of the principal LFRS will provide adequate resistance for earthquakes acting in any direction. Therefore, for ductile structures the design seismic forces may be assumed to act non-concurrently. The corner columns and foundations that are part of the LFRS in both horizontal directions however, are the exception to this. See NZS 1170.5 Supp 1 Clause C5.3.1.

For nonbuilding structures in industrial facilities, the requirement that earthquake forces are to be considered to come from any direction should be carefully evaluated. The NZS 1170.5 Clause 5.3.1 requirement that orthogonal effects be considered shall be satisfied by designing elements using the applicable method outlined in either Clause 5.3.1.1 for ductile structures or Clause 5.3.1.2 for nominally ductile or brittle structures.

There is no requirement in NZS 1170.5 Section 8 for parts and components to be analysed for F_{ph} acting concurrently in a similar manner to Clause 5.3.1 of NZS 1170.5. Parts sensitive to vertical accelerations, however, must be designed for vertical actions F_{pv} as described in NZS 1170.5 Eqn 8.5(2) concurrent with the horizontal actions from NZS 1170.5 Eqn 8.5(1). FEMA 450 Clause 6.2.6 specifically requires no horizontal concurrency. In this case F_{ph} is applied in two orthogonal directions independently but requires a concurrent vertical seismic action equal to 20% of the peak ESA elastic site hazard spectrum.

It is recommended therefore that F_{ph} be applied in two orthogonal directions independently and parts sensitive to vertical accelerations shall be designed for concurrent vertical actions F_{pv} .

5.16 System damping

The appropriate system damping values must be considered for each structure. The spectra curves given in NZS 1170.5 Table 3.1 have been created using the structural damping value = 5% of critical damping for all modes of vibration. This is based on the assumption that 5% is applicable for most buildings for the level of accuracy claimed by a seismic analysis.

A significant number of an industrial plant's building-like and nonbuilding structures not similar to buildings and the components they support are either welded steel pressure retaining items that must remain elastic or nominally ductile unclad items with few or no bolted joints.

The assumption that 5% damping will exist during a major earthquake therefore needs to be examined carefully for these and other industrial structures. Damping ratios for elastic fully welded structures are typically used for vertical skirt supported vessels and horizontal vessels supported on saddles.

The level of damping applying is strain dependant, and the strain levels experienced under the various SLS and ULS return period earthquakes differ. This effect is shown in the Brookhaven National Laboratory (2006) report for the U.S. Nuclear Regulatory Commission (USNRC), NUREG/CR-6919.

NUREG/CR-6919 (2006) Table 2 sets out the recommended levels of damping for cases when an earthquake causes stresses that are 80% or less than the applicable design standard's stress limits. These are lower than those recommended for cases when the structural response to the design earthquake will be close to the applicable design standard stress limits.

Damping also has less effect on rigid structures. Standards such as FEMA 450 reduce the effect of damping in the short period range so at $T = 0.0$ seconds the scaling factor $C_f(\xi) = 1.0$ for all values of ξ .

Where the particular material standard indicates the applicable damping values are not equal to 5% of critical; appropriate scale factors $C_f(\xi)$ should be applied to the seismic response spectrum to determine the design forces.

For example, for steel structures, NZS 3404:1997 Clause 12.2.9 recommends ULS damping values and a method for determining the damping scaling factor for standard structures. For the unclad steel structures that are normally used for modular industrial construction, $2\% \leq \xi \leq 7\%$ and $C_f(\xi)$ varies between 1.333 and 0.895 respectively. However, the NZS 3404 Clause 12.2.9 values for $C_f(\xi)$ do not decrease for short periods.

NUREG/CR-6919 (2006) has recommended damping values for structures, piping, electrical distribution systems, HVAC items, mechanical and electrical components. For piping subsystems, NUREG/CR-6919 (2006) recommends damping values in the range 3% to 4% for SLS2 and ULS level responses. Some pipe stressing software programs have a default damping value of 3% for their MRSA analyses. However, the American Lifelines Alliance (2002) recommends that for both ESA and MRSA, piping design is undertaken using 5% damped spectra.

As systems respond to seismic loadings in an increasingly inelastic manner ($\mu \geq 2$), the level of damping becomes less significant in determining the ultimate structural response. This largely arises from the global damping induced by the yielding of the highly stressed components within the system.

Hence, different levels of damping should be considered for the ULS, SLS1 and SLS2 earthquakes. For the ULS, if the damping levels are overestimated in the design, during an ULS return period earthquake, the structure will start to yield earlier than expected and the damping effects are likely to increase to a level approximately that assumed in the design. The overall effect is not likely to be very different from the original design assumptions.

Because the damping for each individual system will vary, the level of damping to be used in the ULS, SLS1 and SLS2 designs should be determined as part of the initial process when assessing seismic performance. Lower bound values for damping should be considered for critical items; refer to Table 4 for recommended values. Higher values of ξ may be used if justifiable or if premature yielding can be tolerated.

The NZSEE Study Group on Earthquake Risk Buildings (2006) recommendations give a method for modifying the 5% damped spectra for other values of damping. Their equation has been given below.

It is recommended for structures that need to be analysed using damping values different to 5% that the following damping adjustment factor $C_f(\xi)$ is used:

$$\text{For } T \geq 0.2 \text{ sec:} \quad C_f(\xi) = [7 / (2 + \xi)]^{0.5}$$

$$\text{For } T \leq 0.06 \text{ sec:} \quad C_f(\xi) = 1.0$$

For $0.06 < T < 0.2$ sec, linearly interpolate between the above values.

Table 4: Recommended lower bound structural damping levels for vital items as a % of critical damping

ITEM	VALUES FOR ξ WHEN $T \geq 0.2$ SECONDS			
	SLS1 Earthquake	SLS2 Earthquake	ULS ⁽¹⁾ Earthquake	
			$\mu_{uls} \leq 1.25$	$\mu_{uls} \geq 2.0$
Reinforced Concrete	1.0%	4.0%	7.0%	5.0%
Reinforced Masonry	1.0%	4.0%	7.0%	5.0%
Prestressed Concrete	0.5%	3.0%	5.0%	5.0%
Unclad steel welded or bolted with friction connections	0.5%	1.0%	2.0%	5.0%
Unclad Steel with bearing type bolted connections	1.0%	2.0%	5.0%	5.0%
Pressure vessels, heat exchangers, pumps and valves	0.5%	2.0%	3.0%	5.0%
Electrical cabinets, panels, motor control centres	0.5%	2.0%	3.0%	5.0%

Note:

¹Use ULS values only if the significant stresses due to load combinations that include the ULS earthquake are at least 80% of the applicable design standard's stress limits.

5.17 Soil-structure interaction

Soil-structure interaction (SSI) refers to the dynamic interaction effects between a structure and the underlying soil during an earthquake. SSI effects are pronounced for heavy items founded on soft or medium soil such as large diameter storage tanks. In general SSI effects can be ignored for light surface founded structures or structures founded on stiff or rock material.

Methods for considering SSI effects are available in FEMA 450 and “Seismic Design of Storage Tanks” (NZSEE 2009). Generally, values for the shear wave velocities for the underlying soil are required to ascertain soil stiffness as part of these procedures.

SSI effects can potentially modify the structure's response. This depends on the site-specific conditions, and the structure and soil's combined dynamic characteristics. As a rule when SSI effects are significant, the period of the system increases and there is a reduction in the structural response to an earthquake. The exception is when there is resonance between the structure and the soil. Flexible parts or components may have an increased response, so their detailing needs specific attention. The structure and soil's combined response tends to increase the lateral and rocking displacements. Deflection related actions therefore need to be considered.

5.18 Working Stress Methods

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5.18.1 Converting ULS actions to WSD loads

The hazard curves calculated using NZS 1170.5 Equation 3.1(1) give the predicted earthquake accelerations for various return periods at any location within New Zealand. These can be presumed to be independent of the analysis method used to determine the earthquake resistance of items experiencing the earthquake.

In the Ultimate Limit State (ULS), the “ultimate strength” of members within a structure is equated to the actual gravity plus earthquake loading. In the Working Stress Design method (WSD), a load factor is applied to the seismic loadings to reduce them to a level appropriate for allowable stresses. This includes their overstress allowance. In early versions of NZS 4203, the WSD factor was 0.8. ALA (2002) use 1/1.4 (= 0.71) and both FEMA 450 and ASCE 7-10 use 0.7 as this factor. AS/NZS 1170.2 and AS 1210 use 1/1.5 (= 0.67) to convert ULS forces to those to be used with WSD design codes.

In theory, different scaling factors may be required for different actions, as the factor depends on the ratio of the allowable stress (perhaps increased for load cases that include wind or earthquake depending on the code used) to the ultimate stress. ASCE 7-10, Section C11.1 says the 0.7 figure is a compromise for the various building related materials for which the standard ASCE 7-10 would be used.

Table C11-1 of ASCE 7-10 shows ratios of dead and live loads to earthquake load. In some situations WSD analysis, using the 0.7 scaling factor, gives a section where the capacity is less than that required using limit state design.

Designers using the allowable stress methods need to check the load combinations used to derive the combined actions are also compatible with the allowable stress methods. Sections 2.4.1, 12.4.2 and 12.4.3 of ASCE 7-10 outline these requirements. It is important the strength and stability combinations are both used. Table 5.1 in the ASCE Guidelines (2010) also provides guidance for allowable stress design load combinations for industrial plant. These are used as a basis for the combinations given in Section 9 of this Practice Note.

Of the current New Zealand standards, NZS 4219 and AS/NZS 1170.2 are the only ones providing guidance on how allowable stress design methods can be used with AS/NZS 1170 earthquake and wind actions. NZS 4219 Clause 4.3 provides a conversion factor for use with allowable stress designs. The demand or inertia effects are reduced by a factor of 0.8. To remain consistent with NZS 4219 the Working Stress Design method applied to the types of structures designed to WSD standards shall be based on:

1. Design actions being calculated assuming linear elastic behaviour without any redistribution
2. Permissible stresses within the limits of the approved WSD design standards. These include the ASME B31 codes (including ASME B31Ea-2010) for pressure piping, and NZS PD 5500 or approved equivalents for pressure vessels
3. The nominal earthquake loads derived from AS/NZS 1170:2002 and this Practice Note, modified as follows:
 - Multiplied by 0.80 for ULS earthquake loads
 - Multiplied by 1.0 for earthquake loads derived for the serviceability limit states SLS2 and SLS1
 - Incorporating load factors of 1.0 for other load components with load factors equal to or greater than 1.0
 - Incorporating load factors of 0.90 for other load components with load factors less than 1.0, when considering stability.

Values for seismic shear or seismic actions on a part using μ_p or $\mu_p > 1.25$ indicate substantial yielding is expected to provide ductility and energy absorption. A working stress analysis using a ductility factor greater than 1.0 has to be undertaken with some caution.

FEMA (2009) in the commentary to Chapter 12, C12.1 “Structural Design Basis” outlines the ASCE 7-10 philosophy permitting both Strength or ULS design and Allowable Stress Design to use a response modification factor R greater than 1.0. The factor R in ASCE 7-10 Equation 12.8-2 performs much the same function as S_p/k_{μ} does in NZS 1170.5 Equation 5.2(1): it modifies the elastic site response spectrum to the design response spectrum for the chosen level of ductility.

The way the component response modification factor R_p in ASCE 7-10 Equation 13.3-1 modifies the non-structural components’ seismic response is similar to the Part Response Factor C_p for the chosen part’s ductility level. When used with ASCE 7-05, ASME B31Ea (2010) permits R_p to increase to a maximum of 3.5. NZS 1170.5 Table C8.2 outlines some suggested ductilities for various mechanical items: e.g. for pressure pipe $\mu_p = 2.0$.

NZS 1170.5 Clause 8.6 implies the designer must be confident the accelerations from the design earthquake will result in the part or component yielding before a $\mu_p > 1.0$ can be used. If the earthquake induced stresses in the item to be designed do not pass the yield point for a $\mu_p = 1.0$ analysis, the part’s ductility factor may not be further increased.

The part ductility factors outlined in NZS 1170.5 Supp 1 and NZS 4219 may be used to determine earthquake loads as long as:

- The designer proves the part or component will yield as required by NZS 1170.5 Clause 8.6 when subject to accelerations from the design earthquake.
- Both horizontal and vertical earthquake actions are applied concurrently.

ALA (2002) Chapter 7 presents several advanced analysis techniques for piping systems that investigate the plastic range of the stress-strain relationship. These allow pipe stress engineers to use a more complex limit state analysis approach for the pressure piping expected to yield under ULS design actions.

This could be useful when analysing existing lines that cannot be easily strengthened or seismically restrained during retrofits or where new pressure piping lines are tied into existing, older piping systems.

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5.18.2 Brittle fracture

ASME B31E does not directly address brittle fracture. However, it includes a requirement for the ductility of the piping system to be at least 15% at the temperature reasonably expected during an earthquake. Some cast irons may be excluded particularly if the application is normally at temperatures lower than ambient.

ASME B31E increases the allowable occasional stress for the ductile failure mechanism during an earthquake. Note that ASME B31E does not supersede any other provision of the ASME B31.x Standards. The allowable stress to be used in the ASME B31.x clauses for protection against brittle fracture remains unchanged by ASME B31E.

5.18.3 Australian Standards

The material on seismic resistance in the appendices to AS 1210 and AS 1228 is not applicable to equipment destined for New Zealand. Similarly, AS 1170.4 does not apply in New Zealand.

5.19 Seismic displacements

NZS 1170.5 Section 7 “Earthquake Induced Deflections” sets out the requirements for considering seismically induced displacements for buildings. This includes requirements for total displacements and for design inter-storey displacements.

The ULS inter-storey deflection limit is set in NZS 1170.5 Clause 7.5.1 as being 2.5% of the corresponding storey height. ASOE (2010) indicates that it is not necessary to meet the ULS inter-storey deflection limits for nonbuilding structures not similar to buildings. However, it also recommends that if the drift limit is exceeded, P-Delta effects and the effects on any component containing hazardous or flammable material should be considered.

The ULS horizontal seismic displacements are taken as the larger of the values determined from NZS 1170.5 Clause 7.2.1.1(a) or (b). In the Commentary to NZS 1170.5, Figure C7.1 attempts to show the methodology outlined in Clause 7.2.1.1(b) for a structure where sidesway mechanisms are not suppressed by the application of capacity design. However, the entire diagram was not printed. Figure C7.1 should actually be the same as that shown in the Commentary to NZS 4203:1992, Figure C4.7.1.

The NZS 1170.5 Fig C7.1 and the Commentary Clauses C7.2 and C7.3 are thus guidelines only and not comprehensive. The NZS 1170.5 Commentary Clause C7.2.1.1 does not outline whether the deflection profile (1) showing the elastic deflections was determined from an ESA or MRSA. Nor does it show how the ESA deflection scaling factor k_d obtained from NZS 1170.5 Clause 6.2.3 should be accounted for if an ESA is used. Also missing is how the scaling factor for P-Delta effects using NZS 1170.5 Clause 6.5.4 Method A should be applied when this method is used to account for P-Delta effects.

If required, the design inter-storey deflections between adjacent levels can be determined from NZS 1170.5 Clause 7.3. The ULS inter-storey deflections use the drift modification factor k_{dm} obtained from Table 7.1. Category 1 and 2 steel concentrically braced frames (CBF) designed according to NZS 3404 Clause 12.12, require a further deflection factor. This is to be considered for total and inter-storey deflections set out in NZS 3404 Clause 12.12.5.2(h). Detailed guidance on how ULS deflections can be determined is provided in Section 19.2 of HERA report R4-76 (1995). This can be adapted and used with NZS 1170.5. While the procedure set out in R4-76 (1995) Section 19.2 is for CBF structures it can be adapted and used with any structure in lieu of guidance from the Commentary to NZS 1170.5.

The Royal Commission on the Canterbury Earthquakes recommended that an explanation be added to the commentary to NZS 1170.5 (Recommendation 38) regarding the difference between design and peak inter-storey drifts. Submissions on this issue regarding the poor performance of some inter-storey stairs in multi-storey buildings during the major Canterbury earthquakes lead to a Technical Report to the Royal Commission by Des Bull (2011). It was recommended that the peak inter-storey deflections be determined from the ULS drift multiplied by $1/S_p$. Bull also recommended that for escape paths, the peak displacements

were determined from the maximum credible Earthquake which for most structures is a 2500 year return period earthquake. Examples E1 and E2 in Appendix E includes the derivation of the ULS horizontal displacements plus the peak inter-storey drifts based on the HERA approach, NZS 1170.5 Clauses 7.3 and 8.5.3 and the Technical Report to the Royal Commission.

5.20 Seismic anchor motion for distributive systems

ALA (2002) Section 4.1.4 “Seismic Anchor Motion” recommends pressure equipment items should be designed for both the inertia effects resulting from the supporting structure moving under earthquake stress and the differential motion between support attachment points or “seismic anchor motion” (SAM). This is usually only relevant for distributive systems such as piping and ducting networks, as they are frequently supported by a number of different structures along their route.

NZS 1170.5 indirectly includes this effect in Clause 8.5.3. This includes the requirement to account for inter-storey deflection induced actions calculated according to Clause 7.3, where parts are attached to the primary structure on more than one level. However, guidance is provided in NZS 4219:2009 Clause 4.2.3 on how the inertia and displacement induced effects from earthquake induced actions should be combined.

In the distributive system analysis, SAM is accounted for by inputting the seismic displacements at the support attachments or equipment nozzles as separate analyses or load cases. Various deflection profiles resulting from the possible inter-storey movements need to be used in the analyses where distributive systems are supported from many levels in both individual or different structures. Note: Each support attachment may restrict only some of the six possible degrees of freedom.

The resulting stresses in the system are combined with the stresses due to inertia by the square root of the sum of the squares method.

In NZS 4219 this approach is shown in Clause 4.2.3 as:

$$(M_i^2 + M_d^2)^{0.5} = M^*$$

In the seismic standard for ASME B31 pressure piping, ASME B31Ea-2010, the equation in Section 3.4 can be modified similarly to:

$$PD/(4t) + 0.75i\{M_{\text{sustained}} + (M_i^2 + M_a^2)^{0.5}\}/Z < S_s$$

where:

$M_{\text{sustained}}$ = Moment due to sustained loads

M_i = Moment due to earthquake inertia (derived from $0.8 * E_u$)

$M_a = M_d$ = Moment due to earthquake displacement of supports or SAM (derived from E_u)

M^* = Total seismic action on component

P = System operating pressure

D = Outside pipe diameter

t = Pipe wall thickness deducting corrosion allowance but not the mill tolerance

i = Applicable stress intensification factor from the B31 standards

Z = Pipe elastic section modulus deducting corrosion and erosion allowances but not mill tolerances

$S_s = \text{Min}[2.4S; 1.5S_y; 408 \text{ MPa}]$

S = ASME B31 allowable stress at normal operating temperature

S_y = Specified minimum yield stress at normal operating temperature.

Examples E2, E6 and E7 in Appendix E show the sequence for deriving both the M_i and M_a effects based on the NZS 4219 approach.

5.21 Buckling

The structural design of equipment such as water-tube boilers with large flat panels may need to consider buckling. This can result from compression or compressive bending caused by horizontal or vertical earthquake accelerations. Taking buckling into account can be a complex engineering activity. It may depend on information such as internal pressure or vacuum, metal temperature, and non-linear elastic-plastic material behaviour.

5.22 Input data summary

Important data to be determined before detailed structural design starts for industrial structures includes:

- Building code building category
- Structural type; building or nonbuilding structure, part or component
- Intended life
- Design working life
- Importance level
- Applicable design standards
- Analysis method
- Site subsoil class
- Damping levels for design earthquakes.

6.0 EARTHQUAKE ANALYSIS

6.1 Analysis method selection

Two seismic analysis options are commonly used:

- Equivalent Static Analysis (ESA).
- Modal Response Spectrum Analysis (MRSA).

For the majority of cases, the ESA method is appropriate for industrial structures. Its limitations according to NZS 1170.5 Clause 6.1.3.1 should be investigated, and a MRSA used when appropriate.

Numerical integration time history analyses (NITHA) – either linear or non-linear – according to NZS 1170.5 Section 6.4 are not usually undertaken in design offices. The analysis and design of important (IL = 5) and unusual structures would generally be the only ones warranting the expense of this method.

The lack of readily available earthquake records suitable for each site subsoil class, and for those with and without the forward directivity effects associated with near fault motions, means NITHA will only be associated with projects that can afford this work being subcontracted to specialists.

6.2 Site elastic hazard spectrum $C(T)$

If a site specific seismic hazard study has not been undertaken, the values for the 5% damped elastic site spectrum $C(T)$ shall be determined from NZS 1170.5 Clause 3.1.1 for the earthquake return periods required for each applicable limit state.

$$C(T) = C_h(T) Z R N(T_p, D) \quad \text{Eqn 3.1(1)}$$

Where:

$C_h(T)$ = Spectral shape factor for chosen Site Subsoil Class, see NZS 1170.5 Clause 3.1.2

Z = Hazard Factor for the chosen site, see NZS 1170.5 Clause 3.1.4

R = Return Period Factor for the chosen IL and earthquake return period, see NZS 1170.5 Clause 3.1.5

$N(T_p, D)$ = Near-fault factor for the chosen site, see NZS 1170.5 Clause 3.1.6.

Site specific hazard studies should provide one of the following:

- Curves similar to those developed by Eqn 3.1(1) and shown in NZS 1170.5 Figures 3.1 and 3.2, with return periods that match those in AS/NZS 1170.0 Table 3.3
- A more refined estimate for Z , the hazard factor determined in NZS 1170.5 Clause 3.1.4.

Site specific hazard studies should also consider all local faults, not just those listed in NZS 1170.5 Table 3.8.

NZS 1170.5 provides five different generalised hazard spectra. These are typical for five different site subsoil classes, Subsoil Classes A, B, C, D and E. The level of earthquake shaking experienced by a structure depends on the soil layers beneath the site. An assessment of the site's subsoil class is needed to determine the design earthquake.

6.2.1 Site subsoil class

The criteria for determining whether a site has a Subsoil Class of A, B, C, D or E shall be according to NZS 1170.5 Clause 3.1.3. Any geotechnical investigations for the site's foundations should also consider the underlying soil strata and make a recommendation as to the site subsoil class.

6.2.2 Spectral shape factor $C_h(T)$

Once the site's subsoil class has been determined, either NZS 1170.5 Table 3.1 or Figures 3.1 and 3.2 shall be used for the generalised 5% damped Spectral Shape Factor $C_h(T)$.

6.2.3 Hazard factor Z

NZS 1170.5 Table 3.3 shall be used to determine the appropriate hazard factor unless the location is not listed. Then Z shall be interpreted from NZS 1170.5 Figures 3.3 or 3.4 or the values for adjacent locations listed in NZS 1170.5 Table 3.3.

The hazard factor for Christchurch City, Selwyn District and Waimakariri District shall apply to all structural periods of less than 1.5 seconds. This earthquake region's structures with periods in excess of 1.5 seconds should be subject to specific investigation, pending further research¹.

6.2.4 Return period factor R

The return period factor for SLS2 and SLS1 and for ULS, shall be determined from NZS 1170.5 Table 3.5 for the return periods determined from AS/NZS 1170.0 Table 3.3 for the chosen IL and DWL.

In the Canterbury earthquake region, the return period factor for the serviceability limit state SLS1 shall be taken as $R_s = 0.33$.

6.2.5 Near-fault factor $N(T_i, D)$

Long period structures ($T_i > 1.5$ secs) experience greater accelerations than previously expected when located within 20 kilometres of the fault line from which the earthquake originated. NZS 1170.5 Clause 3.1.6 shall be used to determine the near-fault factor $N(T_i, D)$ that endeavours to cater for this effect.

NZS 1170.5 Table 3.3 lists and shows for each major New Zealand location whether it is within 20 km of the 15 most active fault lines. The major faults to consider regarding near-fault factors are listed in NZS 1170.5 Table 3.6. These are shown graphically in NZS 1170.5 Figure 3.5.

6.3 Design earthquake actions

The general equation for seismic forces adapts the 5% damped elastic hazard curves to allow for the assumed level of ductility, building performance, actual damping and P-Delta effects. This is shown in NZS 1170.5 equations 5.2(1), 5.2(3) and 5.2(4). These can be further modified to suit industrial structures:

$$C_d(T) = C(T) * S_p / k_\mu * C_f(\xi) * K$$

Where:

$C_d(T)$ Design action coefficient for the appropriate limit state

$C(T)$ Site elastic hazard spectrum; Equation 3.1.1 of NZS 1170.5

S_p Structural Performance factor for the appropriate limit state; see Section 6.3.3 of this Practice Note

k_μ Inelastic Spectrum Scaling Factor for the appropriate limit state and soil class; see Table 9 in Section 6.3.4 and Table 10 in Section 6.3.4.2 of this Practice Note

$C_f(\xi)$ Damping Factor = 1.0 unless the structural damping **is not** equal to 5% of critical damping for the appropriate limit state; see Table 5 in Section 6.3.1 of this Practice Note.

K Scaling factor that accounts for a variety of ULS effects that may need to be considered. These include P-Delta effects, steel braced frame seismic load modifiers, modal response spectrum method, and ULS scaling factor k etc. For SLS1 and SLS2, $K = 1.0$.

6.3.1 System damping factor $C_f(\xi)$

Appropriate structural damping values shall be selected for analysing the structural system. This is because the spectra curves given in Table 3.1 of NZS 1170.5 have been created using the structural damping value = 5% of critical damping for all modes of vibration.

The damping factor then is determined as per Section 5.16 above. The factor $C_f(\xi)$ for periods > 0.2 seconds is given in Table 5 for different values of ξ .

Table 5: $C_f(\xi)$ values for different levels of damping for $T_1 \geq 0.2$ seconds

% OF CRITICAL DAMPING (ξ)	$C_f(\xi)$
0.5	1.67
1.0	1.53
2.0	1.32
3.0	1.18
5.0	1.00
10.0	0.76
15.0	0.64

6.3.2 P-Delta scaling factor k_1 or k_3

The scaling factor obtained by using NZS 1170.5 method A of Clause 6.5.4.1 to account for P-Delta effects requires the least amount of analysis. However, for higher structural ductility factors, the factor significantly increases the applied seismic forces. The more computationally heavy method B of Clause 6.5.4.2 may be more economical. Depending on the value of T_1 and the site subsoil class, the factor may be different for an ESA (k_1) or for a MRSA (k_3).

For example, a structure with $T_1 = 0.2$ seconds, 500 year return period ULS earthquake, $Z = 0.30$, Soil Class = C, 5% damping, the P-Delta scaling factors vary according to μ as shown in Table 6.

Table 6: Typical P-Delta scaling factors using method A

μ	k_p	k SCALING FACTOR	
		ESA (k_1)	MRSA (k_3)
1	0.0150	1.021	1.017
1.25	0.0169	1.029	1.024
2	0.0225	1.071	1.057
3	0.0300	1.130	1.104
4	0.0300	1.164	1.132
5	0.0300	1.199	1.160
6	0.0300	1.233	1.188

If P-Delta effects are required to be considered the following is recommended:

6.3.2.1 EQUIVALENT STATIC ANALYSIS

Method A in NZS 1170.5 Clause 6.5.4.1 shall be used to increase the applicable seismic forces and displacements using the P-Delta scaling factor k_1 .

6.3.2.2 MODAL RESPONSE SPECTRUM ANALYSIS

These are usually undertaken by computer software packages. Most modern software programs allow either small or large displacement options to be used in the analysis. Small displacement theory does not directly allow for P-Delta effects and should only be used for serviceability limit state analyses. If the large displacement option accounts directly for P-Delta effects by updating the displacement matrix and member stiffness, the P-Delta scaling factor $k_3 = 1.0$. If not and if P-Delta effects need to be considered, the input spectrum shall be increased by the scaling factor k_3 determined from NZS 1170.5 Clause 6.5.4.1.

6.3.3 Structural performance factor S_p

NZS 1170.5 uses a structural performance factor S_p and a displacement ductility factor, to reduce the elastic seismic loads to obtain the design loads on a structure.

The S_p shows:

- The calculated loads correspond to peak accelerations that happen only for a limited time, so have limited time to inflict significant damage
- The construction material's inherent ductility
- The total structural capacity is greater than predicted through redundancy and non-structural elements
- The structure's energy dissipation is typically higher than assumed through damping from non-structural elements and the foundation
- The limit state being considered
- The designer's ability to provide detailing consistent with ductile behaviour.

S_p values need to be determined for buildings, nonbuilding structures similar to buildings and for the main structural support systems for nonbuilding structures not similar to buildings for each limit state. Rigid non-structural elements of combination structures, parts and components are supported elements and are designed for forces derived from NZS 1170.5 Section 8 and do not need to be given an S_p value.

The value for the S_p factor in NZS 1170.5 varies from 0.7 to 1.0. This depends on the chosen structural ductility factor μ for the limit state under consideration. For industrial structures there is less redundancy than for general buildings. They may also contain fewer non-structural items, which give the structure greater damping and strength during a strong earthquake.

For steel or steel/concrete composite items that contain brittle elements that would not respond in a ductile fashion in an earthquake, the S_p factor should equal 1.0.

Jury (2004) indicates the choice of the S_p factor is a matter of judgement as often the performance of actual structures is better than that predicted by analysis. For those structures that are outside the immediate application of the loadings standard, a conservative approach should be adopted. Until there is data available on the seismic performance of the type of structure in question under strong earthquake motions, the designer of industrial structures needs to consider carefully whether the NZS 1170.5 default S_p values can apply to the structure being designed or whether a more conservative value should apply.

The S_p values in Table 7 are recommended unless the relevant material standard specifies a greater value for the material or form of LFRS.

Table 7: Recommended Structural Performance Factors

S_p	ITEM
0.70	<ul style="list-style-type: none">• Steel, concrete, masonry and timber structures designed according to the relevant NZ limit state standards where $\mu \geq 2.0$.
0.925	<ul style="list-style-type: none">• Steel, concrete, masonry and timber structures designed according to the relevant NZ limit state standards where $\mu = 1.25$.• Pressure vessels and piping where $\mu = 1.25$ and limited ductility can be tolerated.
1.00	<ul style="list-style-type: none">• Steel, concrete, masonry and timber structures designed according to the relevant NZ standards where $\mu = 1.0$.• Tanks, pressure vessels and piping where $\mu = 1.0$.• Items for which NZ material standards are not used.• When considering lateral stability against sliding or toppling of a whole structure.

6.3.4 Inelastic spectrum scaling factor k_{μ}

The inelastic lateral load design actions are obtained by factoring the elastic actions by $1/k_{\mu}$. The factor represents the change in the dynamic response of the structure undertaking plastic deformations. To evaluate requires an estimate of the structure's fundamental period of translation vibration, T_1 for the limit state in question.

Table 8 lists recommended values for μ for typical industrial structures.

Table 8: Recommended Maximum Structural Ductility Factors

DESCRIPTION	STRUCTURAL STEEL	REINFORCED CONCRETE	PRE-STRESSED CONCRETE
1. Concrete or steel tanks on ground	As per NZSEE (2009)		
2. Chimneys or stacks, cantilevered piers or columns ^{(1) (6)}	3 ⁽⁴⁾	3	3
3. Guyed chimneys or stacks ^{(4) (6)}	3	–	–
4. Interconnecting pipe work supported above grade ^{(2) (6)}	To be treated as a subsystem		
5. Interconnecting pipe work supported at grade by rigid supports ^{(2) (6)}	1.25	–	–
6. Low pedestal supports for pipe work	1.25	1.25	–
7. A ductile skirt supporting a vertical vessel ^{(4) (6)}	3	–	–
8. Vessels with hold down bolts designed for yielding under the design earthquake ^{(3) (6)}	3	–	–
9. Horizontally mounted pressure equipment ^{(5) (6)}	To be treated as a subsystem or a combination structure		

Notes:

¹ Slender process columns, chimneys or stacks greater than 15 metres high shall be subject to dynamic analysis.

² For Item 4, in situations where very little damage can be tolerated or inadequate ductile capacity is available, the part structural ductility factor μ_p shall be taken as 1.25 otherwise as per Table C8.2 of NZS 1170.5-Supp 1.

³ For Item 8, a higher value of μ may be used, subject to ratification by rational analysis that shall include a time history analysis of a detailed analytical model of the system.

⁴ μ values for skirt and shell to comply with Table 12.5 of NZS 3404.

⁵ For structures supporting equipment whose weight exceeds 20% of the supporting structures, the required μ depends on whether the equipment item is rigid or not, see Table 11.

⁶ Pressure envelope to remain within pressure design code allowable stresses.

6.3.4.1 SOIL CLASSES A, B, C AND D

To allow for the behaviour of inelastic structures with fundamental periods less than 0.7 secs, this factor is adjusted.

$$k_{\mu} = \mu \quad \text{For } T_1 \geq 0.7 \text{ second}$$

$$k_{\mu} = [(\mu - 1) * T_1 / 0.7] + 1 \quad \text{For } T_1 < 0.7 \text{ second}$$

To calculate k_{μ} , T_1 shall not be taken less than 0.4 secs. See NZS 1170.5 Clause 5.2.1.1.

Table 9: Soil Classes A, B, C and D, values of inelastic spectrum scaling factor k_{μ}

μ	$T_1 = 0.4 \text{ SECS}$	$T_1 = 0.5 \text{ SECS}$	$T_1 = 0.6 \text{ SECS}$	$T_1 \geq 0.7 \text{ SECS}$
1.0	1.00	1.0	1.0	1.0
1.25	1.14	1.18	1.21	1.25
2.0	1.57	1.71	1.86	2.0
3.0	2.14	2.43	2.71	3.0
4.0	2.71	3.14	3.57	4.0
5.0	3.29	3.86	4.43	5.0
6.0	3.86	4.57	5.29	6.0

6.3.4.2 SOIL CLASS E

To allow for the behaviour of inelastic structures with fundamental periods less than 1.0 secs, this factor is adjusted.

$$k_{\mu} = \mu \quad \text{For } T_1 \geq 1 \text{ second or } \mu < 1.5$$

$$k_{\mu} = [(\mu - 1.5) * T_1] + 1.5 \quad \text{For } T_1 < 1 \text{ second and } \mu \geq 1.5$$

To calculate k_{μ} , T_1 shall not be taken less than 0.4 secs. See NZS 1170.5 Clause 5.2.1.1.

Table 10: Soil Class E, values of inelastic spectrum scaling factor k_{μ}

μ	$T_1 = 0.4 \text{ SECS}$	$T_1 = 0.6 \text{ SECS}$	$T_1 = 0.8 \text{ SECS}$	$T_1 \geq 1.0 \text{ SECS}$
1.0	1.00	1.00	1.00	1.0
1.25	1.25	1.25	1.25	1.25
2.0	1.70	1.80	1.90	2.0
3.0	2.10	2.40	2.70	3.0
4.0	2.50	3.00	3.50	4.0
5.0	2.90	3.60	4.30	5.0
6.0	3.30	4.20	5.10	6.0

6.4 Equivalent static analysis method

6.4.1 Ultimate limit state horizontal design actions

The main ULS equation is set out using NZS 1170.5 equations 3.1(1), 5.2(1) and 5.2(2). This is modified for the general situation as follows:

$$\begin{aligned} C_d(T_1) &= C(T_1) * S_p / k\mu * C_f(\xi) * K \\ &= C_h(T_1) R Z N(T_1, D) * S_p / k\mu * C_f(\xi) * K \\ &\geq (Z/20 + 0.02) * R \end{aligned}$$

but not less than 0.03R

$C_h(T_1) R Z N(T_1, D)$ represents the ordinate of the site's hazard spectra. This is outlined above for the fundamental translational period of vibration for the chosen ULS return period earthquake.

S_p ULS Structural Performance Factor.

$k\mu$ ULS Ductility Factor set out in NZS 1170.5 Clause 5.2.1.1. This requires an estimate of the structure's ULS fundamental period of translation vibration, T_1 .

$C_f(\xi)$ ULS Damping Factor = 1.0 unless the structural damping is not equal to 5% of critical damping.

K ULS scaling factor that accounts for a variety of effects that may need to be considered. Generally: $= k_1 * k_2$

k_1 ESA P-Delta scaling factor. If P-Delta effects must be considered, see the approximate method in NZS 1170.5 Clause 6.5.4 Method A.

k_2 Material code earthquake actions modifier factor. Other factors can modify the basic seismic acceleration coefficient. These are in the relevant material codes. In the Steel Structures Standard NZS 3404 for example, there is the additional scaling factor C_s .

C_s Steel Concentrically Braced Frame (CBF) seismic load modifier for CBF's with bracing effective in tension and compression. Refer to NZS 3404 Clause 12.12.3:

$$1.0 \leq C_s \leq 2.1. \text{ As } \mu \text{ increases, so does the value of } C_s.$$

6.4.2 Serviceability limit state horizontal actions

The main SLS equation is taken from NZS 1170.5 Clause 5.2.1.2. This refers to NZS 1170.5 Equations 3.1(1) and 5.2(1). It is modified for the general situation as follows:

$$C_d(T_1) = C(T_1) * S_p / k\mu * C_f(\xi)$$

Where:

$C(T_1)$ = the ordinate of the site's elastic hazard spectra outlined above for the fundamental translational period of vibration T_1 for the chosen SLS1/SLS2 return period earthquake. For SLS1/SLS2 T_1 can be different from that for the ULS. The structure is generally stiffer for lower amplitude effects. This effect is shown in NZS 1170.5 Supp 1 Clause C4.1.2.2.

S_p SLS1/SLS2 Structural Performance factor.

$k\mu$ = SLS1/SLS2 Ductility Factor. This is calculated using the chosen ductility factor for the SLS1/SLS2 limit state. This factor requires an estimate of the structure's fundamental period of translation vibration, T_1 appropriate for the chosen serviceability limit state. See NZS 1170.5 Clause 5.2.1.1.

$C_f(\xi)$ SLS1/SLS2 Damping Factor = 1.0 unless the structural damping is not equal to 5% of critical damping for the particular serviceability limit state.

6.4.3 Ultimate limit state vertical design actions

The main ULS equation is set out in NZS 1170.5 Equations 3.2(1) and 5.4(1):

$$\begin{aligned}C_{vd}(T_v) &= C_v(T_v) * S_p * C_f(\xi) \\&= 0.7 C_h(T_v) R Z N(T_v, D) * S_p * C_f(\xi)\end{aligned}$$

Where:

$C_h(T_v) Z R N(T_v, D)$ represents the ordinate of the site's hazard spectra outlined above for the fundamental vertical period of vibration T_v for the chosen ULS return period earthquake.

T_v The fundamental vertical period of the structure or element under consideration.

S_p ULS Structural Performance Factor.

$C_f(\xi)$ ULS Damping Factor = 1.0 unless the structural damping is not equal to 5% of critical damping.

NZS 1170.5 Clause 5.4.1 sets $T_v = 0.0$ second for buildings. For industrial structures, the mass distribution may be such that significant items are supported on beams and not set over columns; a $T_v > 0.0$ second may be more appropriate.

6.4.4 Serviceability limit state vertical actions

The main ULS equation is set out in NZS 1170.5 Equations 3.2(1) and 5.4(1):

$$\begin{aligned}C_{vd}(T_1) &= C_v(T_v) * S_p * C_f(\xi) \\C_{vd}(T_1) &= 0.7 C_h(T_v) R Z N(T_v, D) * S_p * C_f(\xi)\end{aligned}$$

Where:

$C_h(T_v) Z R N(T_v, D)$ represents the ordinate of the site's elastic hazard spectra outlined above for the fundamental vertical period of vibration T_1 for the chosen SLS return period earthquake.

T_v The fundamental vertical period of the structure or element under consideration.

Other terms are as above.

S_p SLS1 and/or SLS2 Structural Performance factor.

$C_f(\xi)$ SLS1 and/or SLS2 Damping Factor = 1.0 unless the structural damping is not equal to 5% of critical damping.

6.5 Modal Response Spectrum Analysis (MRSA)

6.5.1 Ultimate limit state equation for horizontal design actions

The main ULS equation is set out in NZS 1170.5 Equations 3.1(1) and 5.2(3):

$$C_d(T) = C(T) * S_p / k_\mu * C_f(\xi) * K$$
$$= Ch(T) R Z N(T_p, D) * S_p / k_\mu * C_f(\xi) * K$$

$C_h(T) R Z N(T_p, D)$ represents the site's elastic hazard spectra outlined above for the chosen ULS return period earthquake.

S_p ULS Structural Performance Factor.

k_μ Ductility Factor set out in NZS 1170.5 Clause 5.2.1.2. It requires an estimate of the structure's fundamental period of translation vibration, T_p . See NZS 1170.5 Clause 5.2.1.1.

$C_f(\xi)$ ULS Damping Factor = 1.0 unless the structural damping is not equal to 5% of critical damping.

K ULS scaling factor that accounts for a variety of effects that may need to be considered. Generally:
 $K = k_3 * k_4 * k_5$

k_3 MRSA P-Delta scaling factor. If P-Delta effects must be considered, see the approximate method as set out in NZS 1170.5 Clause 6.5.4.1 Method A.

k_4 MRSA effects scaling factor k . If base shear comparisons with ESA base shear must be considered see NZS 1170.5 Clause 5.2.2.2.

k_5 Material code earthquake actions modifier factor. Other factors can modify the basic seismic acceleration coefficient and are set out in the relevant material codes. In the Steel Structures Standard NZS 3404 for example, there is the additional scaling factor C_s .

C_s Steel Concentrically Braced Frame (CBF) seismic load modifier for CBFs with bracing effective in tension and compression. Refer to NZS 3404 Clause 12.12.3:

$$1.0 \leq C_s \leq 2.1. \text{ As } \mu \text{ increases, so does the value of } C_s.$$

6.5.2 Serviceability limit state equation for horizontal design actions

The main SLS equation is given in the corrected version of NZS 1170.5 Equation 5.2(4). This can be written as:

$$C_d(T) = C(T) * S_p / k_\mu * C_f(\xi)$$

Where:

$C(T)$ = the site's elastic hazard spectra outlined above for the chosen SLS return period earthquake.

S_p SLS1 and/or SLS2 Structural Performance Factor.

k_μ = SLS1/SLS2 Ductility Factor calculated using the chosen ductility factor for the SLS1/SLS2 limit state.

$C_f(\xi)$ SLS1 and/or SLS2 Damping Factor = 1.0 unless the structural damping is not equal to 5% of critical damping.

6.5.3 Limit state equations for vertical design actions

The main ULS and SLS equations are set out in NZS 1170.5 Equations 3.2(1) and 5.4(1). If they must be considered, for both limit states the vertical design spectra shall be taken as 70% of the horizontal spectra set out above, with $k_\mu = 1.0$ for the ULS.

7.0 GUIDELINES FOR EARTHQUAKE ANALYSIS OF COMBINATION STRUCTURES

7.1 General

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Combination structures are structural systems that support nonbuilding items that weigh 20% or more of the weight of the structure i.e. $W_p \geq 0.2 (W_p + W_s)$. See Section 3.5.2 for further detail. Most have difficulty meeting the horizontal and/or vertical regularity requirements of NZS 1170.5 Section 4.5.

Depending on the overall height and the combined system's fundamental period, the equivalent static analysis method may not be applicable. See NZS 1170.5 Clause 6.1.3.1. Figures 7 and 8 in Appendix A show some typical industrial combination structures.

Where the examples in Appendix E relate to the guidelines for the seismic design of combination structures, these have been referred to in Section 7.3.

W_p = Weight of nonbuilding item

T_p = Fundamental horizontal period of nonbuilding item

W_s = Weight of support structure

μ_p = Structural ductility factor for nonbuilding item

μ_s = Structural ductility factor for support structure

S_{pe} = Structural performance factor for nonbuilding item

S_{ps} = Structural performance factor for support structure

$C_f(\xi)_p$ = Damping factor for nonbuilding item

$C_f(\xi)_s$ = Damping factor for support structure

$k\mu$ = Inelastic spectrum scaling factor

R_p = Part risk factor for nonbuilding item

C_{ph} = Part horizontal response factor for nonbuilding item.

7.2 Analysis method

Section 4.4.3 and Appendix 4.B of the ASCE guidelines (2010) outlines the recommended analysis approaches. The analysis method differs depending on whether the nonbuilding item is flexible ($T_p > 0.06$ secs) or not ($T_p \leq 0.06$ secs). The two approaches have been outlined in more detail below.

ASCE recommends the design value for the structural ductility factor for analysing the supporting structure is the minimum of that for the nonbuilding item or the supporting structure: $\mu = [\mu_p, \mu_s]_{\min}$. This is one aspect of ASCE's approach that has not been universally adopted.

FEMA 450 Clause 14.1.5 differs. For rigid nonbuilding items ($T_p \leq 0.06$ secs), the combined system is designed using the structural ductility factor for the supporting structure: $\mu = \mu_s$. For flexible nonbuilding items, FEMA 450 restricts μ to being ≤ 3.0 .

The FEMA 450 approach is less conservative for the design of the supporting structure. This is especially true for structures supporting critical items that are designed by working stress methods and may not be able to exhibit any great levels of ductility.

Table 11 outlines the two design methods (Case 1 and Case 2) and details how the seismic coefficients shall be derived for the ULS. Case 2 can always be used in lieu of undertaking a Case 1 analysis.

In the past structures supporting pressure equipment covered by PECPR, were often governed by the minimum seismic coefficients set out in NZS 1200 Appendix I. This arose from ACPPE (2001) Part 3 Clause 3.4.2 where:

“...associated foundations and support structures shall be designed for seismic, wind and snow loading in accordance with this code...”

For pressure vessels and pressure piping these minimum values could be reduced if a site specific seismic hazard study was undertaken according to ACPPE (2001) Clauses 5.4.4 (10) and 6.4.3 (2) and used to determine the seismic coefficients. This Practice Note recommends that this applies to all pressure equipment. It is also recommended the design value determined from the study be used, even if this is less than the coefficient given in Table H1.

Unless the site has a specific seismic hazard study, one effect of Table H1's minimum values is they may limit the maximum possible value of μ for the supporting structure. This effect is illustrated in Examples E3 and E5 in Appendix E.

Table 11: Analysis Methods for Combination Structures

DESCRIPTION	CASE 1	CASE 2
Period of supported nonbuilding item	$T_p \leq 0.06$ secs	$T_p > 0.06$ secs
Fundamental horizontal period of system	T_1 determined for combined system	T_1 determined for combined system
Structural ductility factor	$\mu = \mu_s$	$\mu = [\mu_p, \mu_s]_{\min}$ and $\mu \leq 3$
Structural performance factor	$S_p = S_{ps}$	$S_p = [S_{pe}, S_{ps}]_{\max}$
System damping	$C_f(\xi) = C_f(\xi)_s$	$C_f(\xi) = [C_f(\xi)_p, C_f(\xi)_s]_{\max}$
Seismic Weight	$W_t = W_p + W_s$	$W_t = W_p + W_s$
Heights to seismic masses	Heights to masses adjusted for mass of nonbuilding item	Floor heights include those to masses of nonbuilding items
ULS Inelastic Spectrum Scaling factor	$k\mu$ determined for T_1 based on the site soil class and fundamental period (with T_1 taken not less than 0.4 secs)	$k\mu$ determined for T_1 based on the site soil class and fundamental period (with T_1 taken not less than 0.4 secs)
Horizontal design action coefficient	$C_d(T_1)$ based on system's period	$C_d(T_1)$ based on period of combined system
Base Shear	$V = C_d(T_1) W_t$ For structures supporting pressure equipment; $C_d(T_1) \geq$ applicable value from Table H1	$V = C_d(T_1) W_t$ For structures including modelled pressure equipment; $C_d(T_1) \geq$ applicable value from Table H1
Support structure's member forces	Determined from model by either equivalent static analysis or modal response spectrum analysis	Determined from combined model by either equivalent static analysis or modal response spectrum analysis
Nonbuilding item's member forces	Forces on nonbuilding item F_{ph} determined from Section 8 of NZS 1170.5 using T_p, μ_p, R_p, C_{ph} and $C_f(\xi)_s$ applied through the centre of gravity of the nonbuilding item. WSD design forces = $0.8 F_{ph}$ with F_{ph} obtained using $\mu_p \leq 1.25$	Determined from combined model by either equivalent static analysis or modal response spectrum analysis. WSD design forces obtained from combined model analysed using $\mu \leq 1.25$
Nonbuilding item's anchorages design forces	Forces on nonbuilding item's anchorages are determined from Section 8.7 of NZS 1170.5 using $\mu_p = 1.25, T_p, R_p, C_{ph}$ and $C_f(\xi)_s$ applied through the centre of gravity of the nonbuilding item	Determined from combined model by either equivalent static analysis or modal response spectrum analysis using a design philosophy consistent with Section 12.9.1 of NZS 3404.

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7.2.1 Rigid nonbuilding items – Case 1

The actions on the supporting structure are determined using the procedures outlined in Sections 6.4 or 6.5 with the overall heights adjusted for the masses of the rigid nonbuilding items.

The actions on nonbuilding items are determined using the procedures outlined below in Section 8. This is based on NZS 1170.5 Section 8 for parts and components. The ductility of the supporting structure has no input into the derived horizontal earthquake actions for the nonbuilding items. The chosen value for μ_p affects the magnitude of F_{ph} and the designer needs to assess the maximum value that can be given to the structural ductility factor for the nonbuilding item.

Suggested values for the part structural ductility factor are in NZS 1170.5 Supp 1 Table C8.2. Note 4 to this table states that the designer needs to consider when nonbuilding items can start to sustain damage and whether continuity of operation is required after a ULS event. NZS 1170.5 Supp 1 C8.6 states:

In many instances, especially with mechanical services plant, the design of the part is based on non-structural considerations, and proportioning is such that yielding is unlikely and μ_p should be taken as 1.0.

The actions on the anchorages for rigid nonbuilding items are determined using the procedures outlined in NZS 1170.5 Clause 8.7. Generally anchorages are needed to restrain the item from sliding.

7.2.2 Flexible nonbuilding items – Case 2

The nonbuilding item is included in the analysis model as specific structural members and the masses of the nonbuilding items are kept separate from the rest of the supporting structure. The actions on the combined structure are determined using the procedures outlined in Sections 6.4 or 6.5, with the overall heights including those to the masses of the flexible nonbuilding items. The actions on the nonbuilding item are also determined using the combined model.

The design actions on the anchorages for flexible nonbuilding items should be determined using a connection design philosophy similar to that set out in NZS 3404 Clause 12.9.1. This philosophy ensures the connection between the supporting structure and the nonbuilding item is not the weakest link whose premature failure would invalidate the analysis assumptions.

To derive any benefit from providing a ductile support structure with a $\mu > 3$ for a combination structure to reduce earthquake forces on nonbuilding items, a special study is necessary. See NZS 1170.5:Supp 1 Clause C8.8.

7.3 Example E3 – discussion

This example is for three pressure vessels supported five metres above ground in the Blenheim area. The calculation is an example of a Case 1 analysis:

- The weight of the pressure vessels is more than 2.5 times the weight of the supporting reinforced concrete structure
- The fundamental horizontal period for the vessels (T_p) is less than 0.06 second, i.e. they are rigid.

The seismic coefficients for the reinforced concrete structure have been worked out taking the DWL as 50 years. This is different from the parts analysis used to work out the seismic coefficient for the rigid pressure vessels. This starts with working out the peak ground acceleration for a 250 year return period earthquake corresponding with the DWL of 25 years for the pressure vessels given in AS 1210.

7.3.1 (Left blank intentionally)

7.3.2 Recommendations for the support frame

NZS 3101 permits a maximum structural ductility factor of 6 for a moment resisting frame. As no site specific seismic hazard study is available, the Table H1 minimum seismic coefficients apply for the:

- Pressure vessels (for the WSD case), from Table H1, minimum coefficient = 0.38.
- Supporting structure and their foundations using limit state design, from Table H1, the minimum coefficient = 0.47.

The example in E3 shows the use of $\mu_{uls} \geq 2$ develops a $C_d(T_{1,uls}) < 0.47$ for the supporting structure. Hence a nominally ductile structure with $\mu_{uls} = 1.25$ complies with this Practice Note. However, a design approach according to NZS 3101 Clause 2.6.6 would be required for the reinforced concrete frame design.

To design the reinforced concrete structure as a structure with $2 < \mu_{uls} \leq 6$, a seismic hazard study is required. This would permit an MRSA or ESA to be done and a seismic coefficient less than that set in Table H1 to be used.

If the vessels were flexible instead of rigid, to design the reinforced concrete structure as a ductile structure with $3 < \mu_{uls} \leq 6$, a nonlinear NITHA special study would be required as set out in NZS 1170.5 Clause 6.4.

To progress a simpler design approach and to design the reinforced concrete structure as a ductile structure with $\mu_{uls} = 3$, (the maximum recommended by FEMA 450 without a special study), a seismic hazard study is required. This would permit an MRSA or an ESA to be undertaken and the possible use of seismic coefficients less than those in Table H1.

The total ULS displacements described for the support frame have been checked against Section 5.19 to confirm whether P-Delta effects should be incorporated.

7.3.3 Requirements for pressure vessels

The parts analysis for the seismic forces on the vessels has assumed that although the vessels have wall thickness of 90 mm, there will be some inherent ductility available through their connections to the concrete structure, their shape and material of construction. Therefore μ_p was taken as 1.25. If the vessels were operating at very low temperatures or were made from steel with limited notch ductility, a $\mu_p = 1$ would be more appropriate.

Because the vessels were supported by a slab and not placed directly over the supporting columns, the fundamental vertical period of the structure, T_v was taken as being greater than 0.06 seconds. This gave a ULS $F_{pv} = \pm 0.69W_p$ that would reduce to $\pm 0.26W_p$ if $T_v \leq 0.06$ seconds.

The Part Category was taken as P.1 to give a ULS seismic load for the pressure vessels.

For industrial plants, the notes to NZS 1170.5 Table 8.1 have to be interpreted with the understanding that critical, hazardous or heavy supported above grade non-PEOPR items need to perform for the ULS or SLS2 limit states. All other non-critical nonbuilding and non-PEOPR items may only need to perform satisfactorily in the serviceability limit state SLS1 as regards both strength and displacement.

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7.3.4 *(Left blank intentionally)*

7.3.5 Recommendations for pressure vessels

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Example E3 shows using $\mu_p = 1.25$ develops a $0.8 \cdot F_{ph} = 0.69$. This value is greater than 0.38, the minimum recommended value in Table H1 for pressure equipment.

To use a lower value for the seismic coefficient, a seismic hazard study would be required and used to determine $C(0)$. This would permit an analysis as per Section 8 to be undertaken, and using a seismic coefficient less than that set in Table H1 to be considered.

8.0 GUIDELINES FOR EARTHQUAKE ANALYSIS OF PARTS AND COMPONENTS

8.1 General

The derivation of the applicable seismic forces is undertaken in accordance with NZS 1170.5 Section 8, "Requirements for Parts and Components". This includes deflection induced forces as per NZS 1170.5 Clause 8.5.3 and Section 6.21 above.

Section 8 must be adhered to, even when NZS 1170.5 Clause 6.1.3.1 says the supporting structure has to be analysed by a method other than ESA. It is possible to use the results from a MRSA substituted into NZS 1170.5 Equation 8.2(1) as an alternative to the estimated floor accelerations, the product $C(0) \cdot C_{Hi}$ although there are no specific guidelines given in the standard compared to that given in FEMA 450 and ASCE-07.

Where the examples in Appendix E relate to the guidelines for the seismic design of parts and components, these are highlighted in Section 8.

W_p = weight of subsystem

T_p = fundamental period of part or component

μ_p = structural ductility factor for part or component

$C_f(\xi)_s$ = damping factor for support structure

h_i = height of the attachment of the part or component

h_n = height from base to the uppermost seismic weight

8.2 Floor height coefficient C_{Hi}

Using the three equations to determine the value of the floor height coefficient, C_{Hi} in NZS 1170.5 Clause 8.3 can be confusing. Use of the equation 8.3(3) ($C_{Hi} = 3.0$) when $h_n < 12$ m is not specifically excluded but where items are attached to the top of low level structures it appears unduly onerous. Clarity and serving Clause 8.3's apparent intent would be achieved if NZS 1170.5 equation 8.3(3) was written:

$C_{Hi} = 3.0$ for $h_i \geq 0.2h_n$ and $h_n \geq 12$ m

8.3 Damping

When the supporting structure is analysed using a system damping $\xi < 5\%$, for the design of parts or components attached to such a structure, the designer needs to consider whether the design action on the part or component needs to be adjusted for the likely greater response occurring in the supporting structure. The damping scaling factor $C_f(\xi)_s$ from Section 5.16 above can be added into the NZS 1170.5 Equation 8.5(1) as shown below to account for the greater response:

$$F_{ph} = C_p(T_p) C_{ph} R_p C_f(\xi)_s W_p \leq 3.6 W_p$$

8.4 Analysis method

8.4.1 When supporting structure is analysed using ESA

Appendix C gives the design method for the ULS/SLS2/SLS1 limit states, and details how the seismic coefficients shall be derived. In general, the method of calculating the response for the parts or components is not dependent on whether the part or component is rigid or flexible. The value for the period of the part, T_p only affects the value for the part spectral shape coefficient, $C_i(T_p)$.

The results for the horizontal design earthquake actions on a part or component according to a NZS 1170.5 Section 8 analysis are also not dependent on the supporting structures' chosen or actual structural ductility factor, μ_s . The only way to reduce the F_{ph} design action is to increase the μ_p , the structural ductility factor for the part or component. For items designed using working stress methods, such as pressure retaining systems, this is often not desirable or within the bounds of the design analysis.

As an example, for piping systems, if a $\mu_p = 1.0$ analysis is undertaken and at no location within the piping system does the applied bending moment actions exceed the bending moment where first yield occurs in the pipe, endeavouring to use a μ_p greater than 1.0 cannot be justified.

M_y	= Moment at first yield (kN-m)
M_y	= $S_y Z_e$
S_y	= Specified minimum yield stress of the material at the normal operating temperature (N/mm ²)
Z_e	= Elastic section modulus deducting corrosion/erosion allowance (cm ³)

If the design engineer is satisfied that the pipe support accelerations under the design earthquake is such that yielding of the pipe will occur, the Part Response Factor; C_{ph} or C_{pv} is taken from NZS 1170.5 Table 8.2 using the chosen part ductility factor μ_p .

The values for μ_p in NZS 1170.5 Supp 1 Table C8.2 reflect the ability of the item to carry load in the post-elastic phase of their horizontal force induced stress/strain relationship. Thus yielding and permanent deformations have occurred and if this situation cannot be countenanced, $\mu_p \leq 1.25$.

8.4.2 When supporting structure is analysed using MRSA

FEMA 450 Eqn 6.2-2 outlines how the attachment point accelerations determined from a MRSA can be used to determine F_{ph} . When FEMA 450 Eqn 6.2-2 is compared with the equations in NZS 1170.5 Section 8, it can be seen that the acceleration a_i at Level i determined from a MRSA can be substituted for $C(0) \cdot C_{Hi}$ within NZS 1170.5 Eqn 8.2(1) to determine $C_p(T_p)$.

The MRSA shall be undertaken according to Section 6.5 assuming $\mu = 1$. $C_p(T_p)$ then can be substituted into NZS 1170.5 Eqn 8.5(1) to determine the horizontal design action F_{ph} .

The MRSA software may output a_i , but if not it must be calculated from the deflected shape by established methods of dynamic analysis.

8.4.3 When the supporting structure is base isolated

The analysis of base isolated supporting structures is normally undertaken using overseas standards such as ASCE/SEI 7-10 Chapter 17. For the design of Parts or Components including mechanical items such as pressure equipment within a seismically isolated structure, structural designers should be requested to provide one of:

- A part or component ULS design acceleration equivalent to F_{ph}/W_p that then can be converted to a working stress design coefficient according to Section 5.18.
- A floor acceleration equivalent to $C(0) \cdot C_{Hi}$ that can be fed into NZS 1170.5 Section 8.

8.5 Examples E2, E5 and E6 Discussion

8.5.1 Importance Levels

For the piping in Appendix E2, the IL of 4 given to the piping is derived from the pressure piping hazard level being classed as B according to AS 4343. In addition some form of operational continuity is required during and immediately after a major earthquake.

The DWL is assumed to be the same as the vessel it is attached to.

8.5.2 SLS2 design case

In SLS2 design cases, the chosen displacement ductility factor for the piping of 1.0 as described in NZS 1170.5 Clause 8.6 combined with the required 250 year return period, results in the pipings' loading condition being almost equivalent to the ULS. SLS2 cases need to be investigated by the pipe stress engineer. This is because SLS2 may be the critical design loading case (as in Example E2 for the piping) especially if any over-stress allowance is forgone. However, the B31 piping design standards generally do not distinguish between ULS and SLS earthquakes. They allow the occasional loading from earthquakes to be checked against higher allowable stresses than those for the normal operating case.

The SLS2 inter-storey deflections would have to be determined as per NZS 1170.5 Clauses 7.3.2 and 7.2.2 so the design actions due to seismic anchor motion for the SLS2 case can be analysed.

8.5.3 Deflection induced actions (seismic anchor motion)

NZS 1170.5 Clause 8.5.3 states for items supported on more than one level, the deflection induced actions are determined from Clause 7.3. As the pipe in Example E2 is passing almost to the full height of the vessel, the calculated inter-storey deflections are summed up to the full height. This makes it possible to determine the worst case deflection scenario for the pipe support attachment points. The deflections at the attachment points in the two principal directions have to be fed into the pipe stress analysis as separate loading cases to determine the SAM related actions.

8.5.4 Inertia actions on piping

In Example E2, the elevated locations of the supports ensure the design horizontal accelerations are well above the minimums required by Table H1. In Example E5, the lower elevation of the pipe supports and lower zero period spectral shape factor $C_h(0)$ give a ULS spectral ordinate of the part of 0.92. By contrast, Example E2 has a value of 1.87 for the upper supports.

In Example E6, the design horizontal WSD elastic acceleration of 0.87g for the pressure piping is greater than that required by Table H1 of 0.60g. It is also greater than the Table H1 minimum value of 0.54. If it can be shown that the piping will yield under such a load, the part ductility factor can be increased. If taken as $\mu_p = 2.0$, the design horizontal WSD acceleration reduces to 0.41g. The Table H1 minimum value of 0.54g was used as the final design coefficient.

8.5.5 Combining earthquake actions

In examples E2 and E6, the horizontal inertia effects are combined with the horizontal earthquake induced deflection effects on the piping by the square root of the sum of the squares. This is in line with the ALA (2000) recommendations. The vertical effects on the other hand, are combined in a vectorial fashion. This is because 70% of the peak vertical acceleration is taken as occurring with the peak horizontal acceleration as set out in NZS 1170.5 Clause 3.2.

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8.5.6 Pipe support design

In Example E5, the option to design the pipe supports as a limited ductility structure with $\mu = 3$ is investigated. This was found to have a number of implications for the choice as a limited ductility structure:

- The earthquake design actions can be applied separately without concurrency being considered
- The minimum value for the structural performance factor S_p was taken as 0.7 even though the support is a single vertical cantilever structure with no redundancy
- The ULS damping was taken as 5% of critical, assuming some yielding would occur. This needs to be checked when the final section size and capacity ratio is found. The steel design material standard's stability requirements for ductile members, even though lightly loaded, often force the use of compact steel sections that are in the end not highly stressed; yielding may not occur even under the E_{max} case and the effective damping may be less than 5%
- The ULS $\mu = 3$ seismic design coefficient for the support of 0.29 is significantly less than that permitted by the Table H1. The minimum required = 0.68 and this would be used to design the steel support. Hence there is no advantage in using a limited ductility structure with the additional capacity design work required
- The seismic coefficient required to yield the member at its maximum possible yield strength would be much greater than the maximum likely elastic response
- To design the foundation and the hold down bolts, an elastic seismic coefficient of 0.88 is required. This is greater than the minimum required by this Practice Note's ULS value of 0.68 from Table H1
- To design the foundation for stability, a seismic coefficient of 0.96 is required. The actions have been shown in Figure 24.

The design option to analyse the pipe support as a nominally ductile structure with $\mu = 1.25$ had the following notable points:

- For a $\mu = 1.25$ design, the earthquake directions of action follow NZS 1170.5 Clause 5.3.1.2, 100% on one axis simultaneously with 30% along the perpendicular axis
- The unscaled 5% damped design accelerations were for the ULS = 0.71g and for SLS1 = 0.21g. Thus the assumed levels of damping for the design case were taken to be higher than the minimums recommended in Table 4.
- To design the foundation for bearing and stability and the HD bolts, an elastic seismic coefficient = 1.16 is required. This is greater than the minimum required by this Practice Note's ULS value of 0.68 derived from Table H1.

8.6 Retrofitting or modifying existing plant

The Engineering New Zealand working party notes that existing plant will at best have been designed to comply with an earlier set of seismic codes and standards, and possibly with no recognition of seismic actions. An approach to this problem is set out in Appendix D.

9.0 LOADS AND LOAD COMBINATIONS

9.1 General

Load combinations including seismic and wind forces shall be assessed in terms of AS/NZS 1170.0 Clauses 4.2.1 and 4.2.2 for the ultimate limit states of stability and strength respectively and Clause 4.3 for serviceability limit states. Structures and foundations should be designed for the most critical effects resulting from the combination of all applicable loads that may reasonably occur simultaneously. This shall include all the load combinations stipulated in Sections 4.2, 4.3 and 4.4 of AS/NZS 1170.0.

Suggested load combinations including seismic and wind forces have been given below.

9.2 Loads for industrial and petrochemical plants

In industrial and petrochemical facilities, different types of loadings affect the design of members and structures than those normally considered in building standards. All applicable loads shall be reviewed and their supports designed for their effects. These loads typically include:

1. G = Dead Load – the total weight of materials forming the permanent part of a structure including structural framing, platforms, walls, fixed equipment, vessels, piping, and insulation and fireproofing.
2. DV = Vessel Operating Load – the weight of any liquids, gas or solids within a vessel and any attached equipment and piping during normal operations.
3. DP = Piping Operating Load – the weight of any liquids, gas or solids within piping or in-line equipment during normal operations plus internal pressure effects.
4. Q = Live Load – the weight of all movable loads such as personnel, tools, stored material, movable partitions, cranes and hoists, maintenance loads, impact, surge forces, vibration and horizontal forces from moving equipment. The design loads shall be maximum loads likely to result from the intended use and shall not be less than those given in AS/NZS 1170.1.
5. E = Earthquake Load – the inertia and differential movement effect due to the design earthquake for the chosen limit state.
6. W = Wind Load – the wind force acting on the structure or members. For the determination of wind loads, refer to AS/NZS 1170.2.
7. TF = Support Friction Loads – the loads caused by the sliding of pipes or by the sliding of either horizontal vessels or heat exchangers on their supports in response to thermal expansion.
8. TH = Thermal Load – the self-straining load caused by the restrained thermal expansion in pipes, horizontal vessels and heat exchangers. It results from a change in temperature and may result from both operating and environmental conditions.
9. AF = Pipe anchor and guide forces – the loads from the restraining guides and anchors derived from the pipe stress analyses acting on pipe supports and pipe racks.
10. TE = Test Load – the weight of equipment and hydro-test fluid related to the pre-commissioning test load condition. Also represents the proof testing load of any overhead monorails or cranes required to be undertaken prior to operation, usually 125% of the SWL of the crane.
11. BP = Bundle Pull Load – the force required to extract internal tube bundles from heat exchangers. To be taken as the weight of the removable bundle but not less than 10 kN.

Friction produced by gravity shall not be relied upon to resist seismic forces. However the designer shall consider the loads arising from analyses with and without friction.

9.3 Loads combinations and load factors

9.3.1 General

ULS Strength Load Combinations:

$$E_{d1} = 1.35G + 1.35[TH, TF]_{min}$$

$$E_{d2} = 1.2G + 1.5Q + 1.2[TH, TF]_{min}$$

$$E_{d3} = 1.2G + 1.5\psi_l Q$$

$$E_{d4} = 1.2G + W_u + \psi_c Q$$

$$E_{d5} = 0.9G + W_u$$

$$E_{d6} = G + E_u + \psi_c Q + AF + TH$$

$$E_{d7} = 0.9G + E_u$$

$$E_{d8} = 1.2G + S_u + \psi_c Q + 1.2[TH, TF]_{min}$$

$$E_{d9} = 1.2G + 1.2TE + W_u$$

Where:

E_u Ultimate seismic load

S_u Ultimate load due to snow, liquid, rainwater ponding or earth pressure

W_u Ultimate wind load

ψ_l Imposed Action Long-term Factor

ψ_c Imposed Action Combination Factor

SLS Load Combinations:

$$S1 = G + \psi_l Q + [TH, TF]_{min}$$

$$S2 = G + \psi_s Q$$

$$S3 = G + \psi_l Q + E_s + [TH, TF]_{min}$$

$$S4 = G + \psi_l Q + W_s + [TH, TF]_{min}$$

Where:

ψ_s Imposed Action Short-term Factor

E_s Serviceability seismic load

W_s Serviceability wind load

Specific SLS conditions directly associated with process and operating requirements shall be assessed separately and shall apply where these are more critical.

WSD Method Load Combinations:

$$A_1 = G + [TH, TF]_{min} + AF$$

$$A_2 = G + Q + [TH, TF]_{min} + AF$$

$$A_3 = G + \psi_l Q$$

$$A_4 = G + W_u/1.5 + \psi_c Q + AF + TH$$

$$A_5 = 0.7G + W_u/1.5$$

$$A_6 = G + 0.8E_u + AF + TH$$

$$A_7 = G + 0.8E_u$$

$$A_8 = G + S_u/1.5 + \psi_c Q + [TH, TF]_{\min}$$

In any load combination containing either a wind or earthquake ULS load case, the allowable stress may be increased if permitted by the allowable stress material design standard for such occasional loads.

9.3.2 Vessel and equipment support design

ULS Strength Load Combinations:

$$E_{d1} = 1.35G + 1.35DV + 1.35[TH, TF]_{\min}$$

$$E_{d2} = 1.2G + 1.2DV + 1.5Q + 1.2[TH, TF]_{\min}$$

$$E_{d3} = 1.2G + 1.2DV + 1.5\psi_f Q$$

$$E_{d4} = 1.2G + 1.2DV + W_u + \psi_c Q$$

$$E_{d5} = 0.9G + 0.9DV + W_u$$

$$E_{d6} = G + DV + E_u + \psi_c Q$$

$$E_{d7} = 0.9G + 0.9DV + E_u$$

$$E_{d8} = 1.2G + 1.2DV + S_u + \psi_c Q + 1.2[TH, TF]_{\min}$$

$$E_{d9} = 1.2G + 1.2TE + W_u$$

$$E_{d10} = 1.2G + 1.5BP$$

SLS Load Combinations:

$$S_1 = G + DV + \psi_f Q + [TH, TF]_{\min}$$

$$S_2 = G + DV + \psi_s Q$$

$$S_3 = G + DV + \psi_f Q + E_s + [TH, TF]_{\min}$$

$$S_4 = G + DV + \psi_f Q + W_s + [TH, TF]_{\min}$$

Specific SLS conditions directly associated with process and operating requirements shall be assessed separately and shall apply where these are more critical.

9.3.3 Pipe rack and pipe support design

ULS Strength Load Combinations:

$$E_{d1} = 1.35G + 1.35(DP + AF + TH + TF)$$

$$E_{d2} = 1.2G + 1.2(DP + AF + TH + TF) + 1.5Q$$

$$E_{d3} = 1.2G + 1.2(DP + AF + TF) + 1.5\psi_f Q$$

$$E_{d4} = 1.2G + 1.2(DP + AF) + W_u + \psi_c Q$$

$$E_{d5} = 0.9G + W_u$$

$$E_{d6} = G + DP + AF + E_u + \psi_c Q$$

$$E_{d7} = 0.9G + 0.9(DP + AF) + E_u$$

$$E_{d8} = 1.2G + 1.2(DP + AF) + S_u + \psi_c Q$$

$$E_{d9} = 1.2G + 1.2TE + W_u$$

SLS Load Combinations:

$$S_1 = G + DP + AF + TH + TF + \psi_f Q$$

$$S_2 = G + DP + AF + TF + \psi_s Q$$

$$S_3 = G + DP + AF + TH + TF + \psi_f Q + E_s$$

$$S_4 = G + DP + AF + TH + TF + \psi_f Q + W_s$$

Specific SLS conditions directly associated with process and operating requirements shall be assessed separately and shall apply where these are more critical.

9.3.4 Pressure equipment design

The WSD load combinations for the design of the pressure retaining envelope and integral supports should comply with the applicable design code. To include wind, earthquake and snow loads as appropriate, the ULS effects are modified as follows: $W_u/1.5$, $0.8E_u$ and $S_u/1.5$.

In any load combination containing either a wind or earthquake ULS load case, the allowable stress may be increased if permitted by the allowable stress design standard for such occasional loads.

10.0 GEOTECHNICAL CONSIDERATIONS

10.1 General

Foundations for the items within an industrial plant are generally constructed from reinforced concrete and should comply with NZS 3101.

The design of the foundations shall consider the full set of limit states specified by the Building Code's Verification Method B1/VM4 for short and long term loading effects including:

- Foundation bearing and sliding capacities.
- Pile and pile group vertical and lateral strengths.

The design of the foundations shall also consider the design verification requirements of the PECPR Regulations and industry practice. Appendix F provides guidance, including modified forms of Producer Statements now called Design Statements.

10.2 Geotechnical considerations

Existing geotechnical reports that cover the proposed site may be available from the plant's owner or the local authority. If not available, site investigations are usually required to obtain basic foundation data needed for the foundation design. Existing foundations in the location and their imposed loads offer a guide to what type of foundations may be suitable.

Alternatively, for smaller structures, a conservative estimate of soil properties can be used in design provided these are confirmed after excavations for the item have occurred and before any site concrete is poured.

NZS 3604 Section 3 gives a relatively simple method for determining good ground.

The following is a list of items that ideally are addressed in a site investigation.

1. Evaluations of any existing data or geotechnical reports.
2. Depth to bedrock.
3. Ground water table location.
4. Soil stratification.
5. Unique site conditions such as sloping bedrock, proximity to faults, cliff areas, river beds and coastal areas.
6. Soil physical characteristics such as grain size distribution, index properties, organic content, void ratio, water content.
7. Soil mechanical properties for long and short term loadings.
8. Shear wave properties.
9. Propensity for liquefaction or settlement.
10. CBR values for pavement design.
11. Site soil class as per NZS 1170.5.
12. Recommendations for suitable foundation types for the items to be supported.
13. Recommendations for bulk earthworks excavation and compaction methods and practices.

The site investigation reports should include descriptions of drilling, sampling and in-situ testing methods and photographs of test pits and core samples.

APPENDIX A

IMAGES OF TYPICAL INDUSTRIAL STRUCTURES

BUILDING-LIKE STRUCTURES



Figure 3: Vessel support structure.



Figure 4: Pipe rack and pipe bridge.

NONBUILDING-LIKE STRUCTURES



Figure 5: Rigid structures – pumps skid mounted at grade.



Figure 6: Flat bottomed tanks.



Figure 7: Combination structure.



Figure 8: Combination structure.



Figure 9: Vertical fired heater.



Figure 10: Floor mounted cabinets.



Figure 11: Guyed stack.



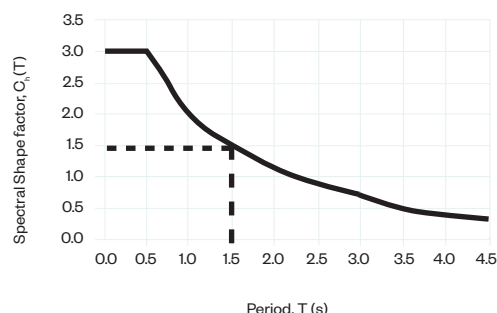
Figure 12: Pipes and pipe supports.

APPENDIX B

EQUIVALENT STATIC ANALYSIS METHOD (GROUND FLOOR) NZS 1170.5

This Appendix should be used with caution. It should be read in conjunction with the Standard and other references, and the other sections of this Practice Note.

	PROCESS	SYMBOL	STANDARD REFERENCE	OTHER REFERENCE
1.0	Parameters relating to structure or plant as a whole. A part supported directly on the ground floor shall be designed as a separate structure in accordance with NZS 1170.5 Section 5 using the structural characteristics determined in NZS 1170.5 Section 4.			
1.1	Determine Importance Level	IL	AS/NZS 1170.0 cl 3.3 Tables 3.1 and 3.2	Section 5.3 and Table 2 of this Practice Note. The AS 4343 Hazard Level is one of the items required to determine the Importance Level.
1.2	Determine Design Working Life	DWL	AS/NZS 1170.0 Table 3.3	Client's brief. See also Section 5.1.1 of this Practice Note on dangers of very short design lives.
1.3	Determine Annual Probability of Exceedance	APE	AS/NZS 1170.0 Table 3.3	
2.0	Parameters relating to structure on the defined site.			
2.1	Determine site subsoil class	A rock B C D E very soft soil	NZS 1170.5 cl 3.1.3	NZS 1170.5 Commentary C 3.1.3 A strong rock B rock C shallow soil D deep or soft soil E very soft soil
2.2	Determine periods of modes of vibration	T	NZS 1170.5 cl 4.1.2.1	Eigenvalues from computer model. $T_1 < 0.4$ s for ESA to be permissible. NZS 1170.5 cl 6.1.3.1.
2.3	Determine spectral shape factor	$C_h(T)$	NZS 1170.5 Table 3.1 or Figure 3.1	
2.4	Determine hazard factor	Z	NZS 1170.5 Table 3.3	
2.5	Determine return period factor	R_u	NZS 1170.5 Table 3.5	Function of APE. Check $ZR_u \leq 0.7$.
2.6	Determine near-fault factor	$N(T,D)$	NZS 1170.5 cl 3.1.6	



	PROCESS	SYMBOL	STANDARD REFERENCE	OTHER REFERENCE
2.7	Calculate horizontal elastic site hazard spectrum	$C(T)$	NZS 1170.5 cl 3.1.1	$C(T) = C_h(T) Z R_u N(T, D)$ Repeat from 2.2 for other modes of vibration.
2.8	Calculate vertical elastic site hazard spectrum	$C_v(T)$	NZS 1170.5 cl 3.2	$C_v(T) = 0.7C(T)$
3.1	Structural ductility factor	μ	NZS 1170.5 cl 4.3.1.1 refers to appropriate material Standard.	NZS 3404 cl 12.2.3.3. $\mu = 1$ elastic $\mu = 1.25$ nominally ductile ¹ NZS 1170.5 Supp 1:2004 Table C8.2 allows $\mu_p = 2$ in some situations. See Section 5.1.8 and example E6 in this Practice Note.
3.2	Structural performance factor	S_p	NZS 1170.5 cl 4.4.2	Despite the notes in the material standards, the lack of structural redundancy in many pressure vessel structures suggests that $S_p \approx 1$. See Section 5.12 of this Practice Note.
4.1	Inelastic spectrum scaling factor	k_μ	NZS 1170.5 cl 5.2.1.1	Adjust ductility for soil and period
4.2	Horiz. design action coefficient	$C_d(T_i)$	NZS 1170.5 cl 5.2.1	$C_d(T_i) = C(T_i) S_p / k_\mu$ Checks for range are required.

¹ NZS 1170.5 C4.3 gives the material Standard as the preferred source for μ . Hence see NZS 3404 cl 12.2.3.3 p 216. Equipment that is to be WSD will be Category 3 if it is nominally ductile, and then $\mu = 1.25$. If ductility cannot be guaranteed, as in refrigeration plant, we have Category 4, which must be designed to be elastic, and then $\mu = 1$.

	PROCESS	SYMBOL	STANDARD REFERENCE	OTHER REFERENCE
4.3	Vert. design action coefficient	C_{vd}	NZS 1170.5 cl 5.4	$C_{vd} = C_v(T_v)S_p$ Some clients require horizontal and vertical actions to be combined, others do not. ASME B31Ea-2010 requires either: SRSS (east-west + north-south + vertical), or Envelope of SRSS (east-west + vertical) and SRSS (north-south + vertical)
4.4	Analysis for P-Delta effects ²		NZS 1170.5 cl 6.5	Not required if $T_{max} < 0.4$ second; or $T_{max} < 0.6$ s and height < 15 m. See Section 6.3.2 and Table 6 of this Practice Note.
5.0	Parameters relating to damping of vibrations.			
5.1	Corrections for damping $\neq 5\%$. See Section 6.3.1 and Tables 4 and 5 of this Practice Note.			
6.0	Parameters relating to working stress design.			
6.1	Factor for working stress design		0.8	NZS 4219 cl 4.3. See Section 5.18 of this Practice Note.
6.2	Factor for working stress design	If NZS 3404 Appendix P Alternative Design Method is used, factors of safety Ω are given where $0.4 \leq \Omega \leq 0.62$. The design requirement then takes the form $M^* \leq \Omega M_{sx}$. See NZS 3404 Appendix P12.2.		

² Second-order effects (P- Δ and P- δ) must always be considered. They must be included in the design only if they are significant. In WSD of the AS 1250 type, they are included by increasing the effective length in member design.

APPENDIX C

EQUIVALENT STATIC ANALYSIS METHOD (ABOVE GROUND FLOOR) NZS 1170.5

SECTION 8

This Appendix should be used with caution. It should be read in conjunction with the Standard and other references, and the other sections of this Practice Note.

	PROCESS	SYMBOL	STANDARD REFERENCE	OTHER REFERENCE
1.0	Parameters relating to structure or plant as a whole.			
1.1	Determine Importance Level	IL	AS/NZS 1170.0 cl 3.3 and Tables 3.1 and 3.2	Section 5.3 and Table 2 of this Practice Note. The AS 4343 Hazard Level is one of the items required to determine the Importance Level.
1.2	Determine Design Working Life	DWL	AS/NZS 1170.0 Table 3.3	Client's brief. See also Section 5.1.1 of this Practice Note.
1.3	Determine Annual Probability of Exceedence	APE	AS/NZS 1170.0 Table 3.3	
2.0	Parameters relating to structure on the defined site.			
2.1	Determine site subsoil class	A rock B C D E very soft soil	NZS 1170.5 cl 3.1.3	NZS 1170.5 Commentary C 3.1.3 A strong rock B rock C shallow soil D deep or soft soil E very soft soil
2.2	Assume period of vibration	T	NZS 1170.5 cl 8.2 $T = 0$	Part is assumed to be rigid, and spectral shape factor is adjusted accordingly.
2.3	Determine spectral shape factors	$C_h(T)$ hori $C(T)$ vert	NZS 1170.5 Table 3.1 (values in brackets) or Figure 3.2.	
2.4	Determine hazard factor	Z	NZS 1170.5 Table 3.3	
2.5	Determine return period factor	R_u	NZS 1170.5 Table 3.5	Function of APE. Check $ZR_u \leq 0.7$.
2.6	Determine near-fault factor	$N(T,D)$	NZS 1170.5 cl 3.1.6	
2.7	Calculate horizontal elastic site hazard spectrum	$C(0)$	NZS 1170.5 cl 3.1.1	$C(T) = C_h(T)ZR_uN(T,D)$

	PROCESS	SYMBOL	STANDARD REFERENCE	OTHER REFERENCE
3.0	Parameters relating to plant at a defined height in a larger structure.			
3.1	Calculate floor height coefficient	C_{Hi}	NZS 1170.5 cl 8.3	$C_{hi} = 3.0$ for $h_i \geq 0.2h_n$ and $h_n \geq 12$ metre.
3.2	Calculate period of first mode of vibration of the part.	T_p	NZS 1170.5 cl 8.2	If part is a pipe, assume elastic beam on simple supports.
3.3	Determine part spectral shape coefficient.	$C_i(T_p)$	NZS 1170.5 cl 8.4	
3.4	Calculate horizontal acceleration coefficient	$C_p(T_p)$	NZS 1170.5 cl 8.2	$C_p(T_p) = C(0) C_{Hi} C_i(T_p)$
3.5	Structural ductility factor for part	μ_p	NZS 1170.5 cl 8.6 $\mu_p = 1$ unless floor acceleration is sufficient to yield the part.	However, NZS 1170.5 Commentary Table C8.2 suggests $\mu_p = 2$ for several types of pipe. See Section 5.18 and example E6 of this Practice Note for guidance.
3.6	Determine part response factor	C_{ph} and C_{pv}	NZS 1170.5 Table 8.2	
3.7	Determine part risk factor	R_p	NZS 1170.5 Table 8.1	
3.8	Determine weight of part	W_p		From manufacturer's data, by calculation, or by weighing.
3.9.1	Calculate horizontal design earthquake actions	F_{ph}	NZS 1170.5 cl 8.5.1	$F_{ph} \leq 3.6 W_p$ $F_{ph} = C_p(T_p) C_{ph} R_p W_p$
3.9.2	Calculate vertical design earthquake actions	F_{pv}	NZS 1170.5 cl 8.5.2	$F_{pv} \leq 2.5 W_p$ $F_{pv} = C_{pv} C_{vd} R_p W_p$ ASME B31Ea-2010 requires either: SRSS (east-west + north-south + vertical), or Envelope of SRSS (east-west + vertical) and SRSS (north-south + vertical) See example E6 of this Practice Note.
4.0	Parameters relating to damping of vibrations.			
4.1	Corrections for damping $\neq 5\%$.			See Table 4 of this Practice Note.
5.0	Parameters relating to working stress design.			
5.1	Factor for working stress design		0.8	NZS 4219 cl 4.3. See Section 5.18 of this Practice Note.

APPENDIX D

GUIDELINES FOR RETROFITTING OR MODIFYING EXISTING PLANT

1.0 Introduction

1.1 Purpose

The purpose of this appendix is to guide controllers, designers, and design verifiers when existing pressure equipment is to be altered.

Unless triggered by some significant change there is no requirement to retrospectively re-design existing equipment. Further, re-design would not usually include the entire plant.

Equipment that requires re-design may not necessarily need to meet the full seismic loading code requirements. This Practice Note recommends the designer follows and documents an engineering safety process for determining low, medium, and high risk, and then choose the appropriate earthquake design action.

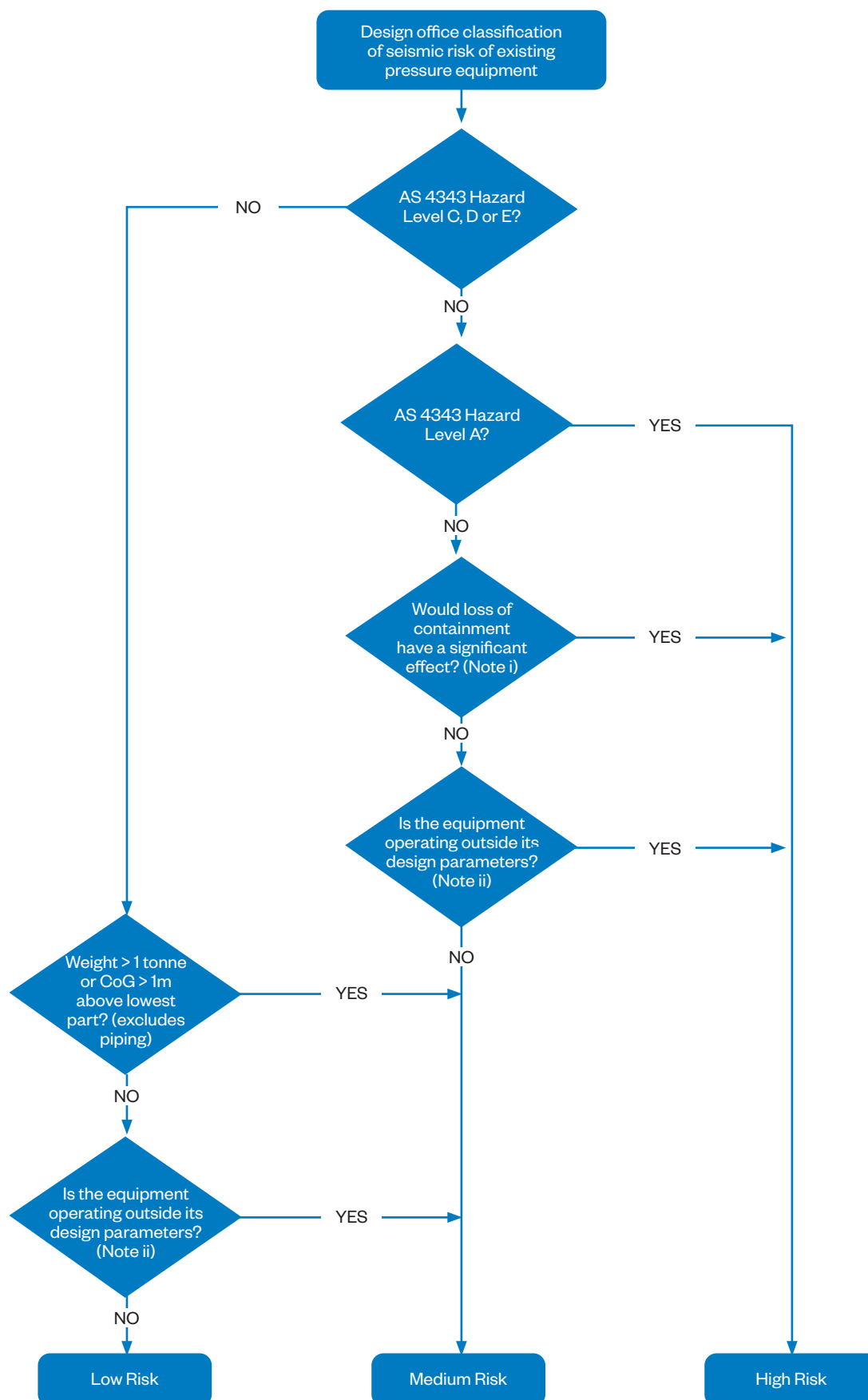
1.2 Scope

This appendix is only intended to cover those alterations affecting seismic resistance.

2.0 Risk evaluation

The HSE Act's Section 7 requirement to identify hazards may initially be complied with by a qualitative process. Figure D1 below shall be used to classify the equipment into low, medium, or high risk categories.

Figure D1: Decision tree for initial risk classification



NOTES FOR FIGURE D1:

- i. A significant effect from a loss of containment would include any of the following:
- Death or permanent injury to members of the public or people other than the plant operators, i.e. would cause death or permanent injury.
 - Long-term damage or a significant clean-up effort in an ecological feature of national significance.
 - Irreparable damage to a facility of national or community significance.
 - Failure of critical systems or post disaster facilities. Critical systems are defined as those which failure of may present a hazard to life outside the boundary of the facility, are required to ensure the safe operation of the facility, or have an emergency or post disaster function. Possible examples of critical systems include deluge water to a hydrocarbon storage tank or vessel, piping containing toxic or lethal gas, flare lines, natural gas supply to a power station with post disaster function.

COMMENT D.2I

Exposure of hazards to members of the public or people other than plant operators would be an increased risk on the basis that they are not trained in dealing with the hazards nor would have access to personal protective equipment etc.

- ii. Equipment operating outside of its design parameters would include:
- Materials operating at temperatures outside levels permitted by the design code.
 - Corrosion levels higher than design criteria.
 - Operating temperature and/or pressure higher than design criteria.
 - Operating past its creep or fatigue life.

COMMENT D.2II

Unsafe equipment should be taken out of service. This Practice Note acknowledges that there is much pressure equipment that is not inspected and would be difficult to take out of service quickly if an issue is identified. The intention of Figure D1 and Note ii is that equipment that is operating outside its design parameters and the issue cannot remedied, should be designated as a hazard which must be managed.

3.0 Unmodified plant

While this Practice Note acknowledges that there is no retrospective requirement to upgrade existing pressure equipment that is not being modified to meet current seismic loading codes, the HSE Act requires employers to take all practicable steps to ensure the safety of employees while at work. It is necessary therefore for employers to evaluate and document the risk posed by existing pressure equipment and make any necessary improvements to reduce any risk posed to acceptable levels.

This Practice Note recommends the following minimum evaluation requirements for low, medium and high risk pressure equipment as classified using the decision tree in Figure D1:

Table D1: Evaluation requirements for unmodified equipment

RISK CATEGORY	EVALUATION REQUIREMENT
Low	None
Medium	Walkdown evaluation ¹ or engineering analysis
High	Engineering analysis and comparison with current codes (Note i)

NOTES FOR TABLE D1:

- i. The designer may use reduced seismic design loads as defined in Table D2 below.

4.0 Procedure for alterations and repairs

Figure D2 below shows the procedure which determines when design and design verification is required.

COMMENT D.4

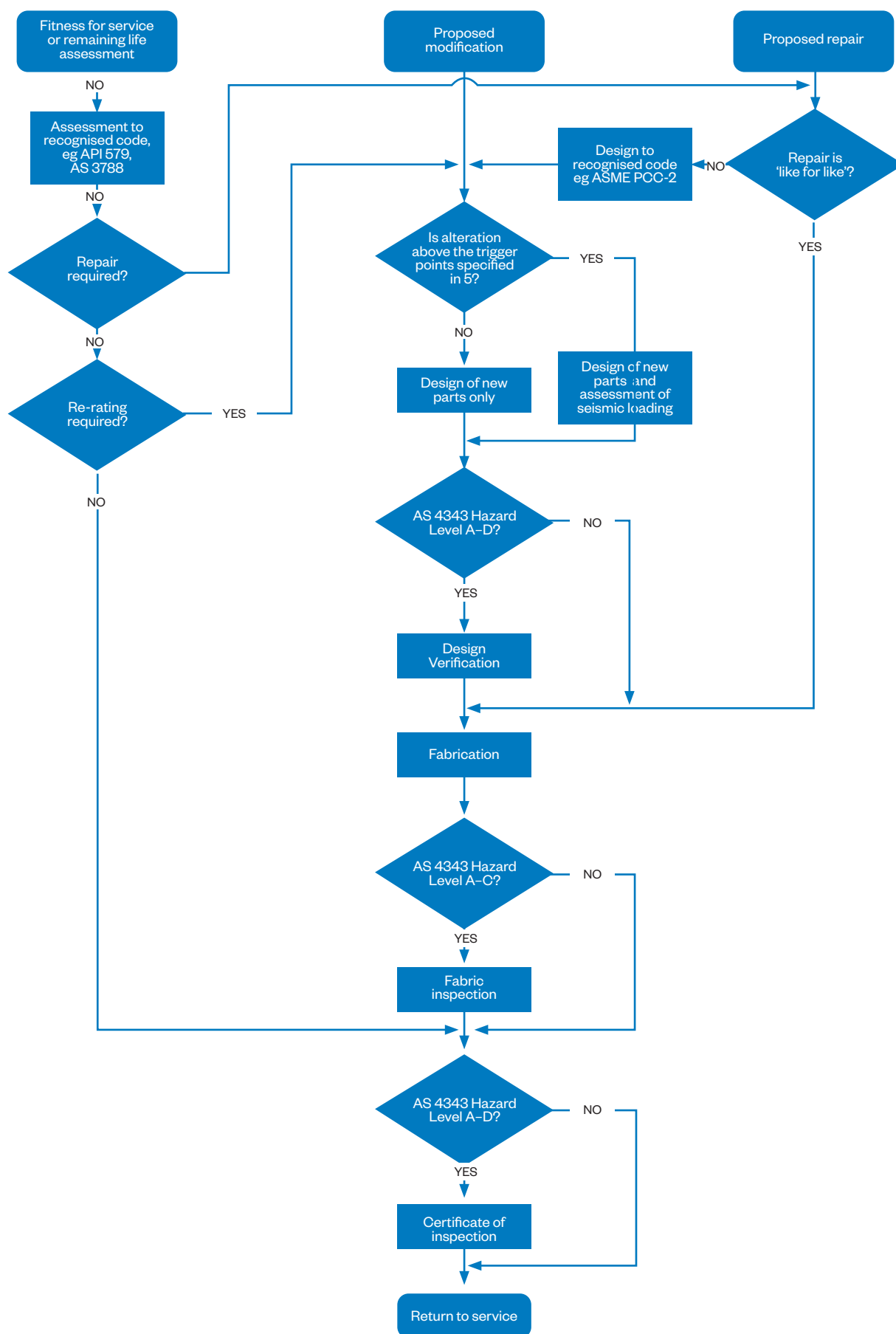
This procedure is intended to supplement Appendix A of the Approved Code of Practice to show the additional steps required to determine the extent of engineering analysis required.

NOTES FOR FIGURE D2:

- i. The equipment controller may choose to remove the equipment from service at any time in the process for economic reasons.
- ii. Exceptions to the requirements for Design Verification, Fabrication Inspection and In-service Inspection are detailed in the Approved Codes of Practice.
- iii. A “Like for like” repair is comparable to the original construction. Comparability is achieved when the repair is made of similar materials and configuration and its performance in terms of PECPR regulations and the design and loading codes, is as good as the original. An example of this would be the replacement of superseded materials with their current equivalents.

¹Walkdown evaluations are described in the ASCE publication Guidelines for Seismic Evaluation and Design of Petrochemical Facilities. They are intended to be a screening exercise conducted by an Engineer or team of engineers and there is no minimum requirements for qualification of the engineer(s) involved.

Figure D2. Procedure for pressure equipment repairs and alterations



5.0 Iterations affecting seismic strength

The Approved Code of Practice for Pressure Equipment (ACPPE) requires alterations to be treated as new design and to be design verified². For practical reasons it is necessary to define what level of alteration is significant in terms of seismic resistance. This Practice Note recommends the designer conducts an initial analysis to determine whether the following “trigger points” apply and consequently to define when a seismic design assessment is required:

A) INCREASES IN WEIGHT OR OVERTURNING MOMENT

A modification which could cause an increase in overall weight or overturning moment by more than 10% from the last design verified design. Modifications include all pressure containing parts, supports and structures such as buildings, access platforms, and pipe bridges which interact with the pressure equipment.

COMMENT D.5(A)

The intention with this limit is to allow minor alterations such as installation of new instruments without triggering a seismic reassessment. The limit of 10% will be reviewed and adjusted if necessary in future revisions of this Practice Note.

B) DECREASES IN SEISMIC STRENGTH

Any modification or repair that could decrease the seismic strength by more than 10% from the last design verified design. Modifications that may decrease seismic strength include change of materials, removal or alteration of stiffening elements or supporting structures, increased stress intensification factors, or changes of pipe diameter.

C) ALTERATIONS TO LOAD OR INTERNAL FORCE DISTRIBUTION

Any modification or repair that could alter the external load or internal force distribution by more than 10% from the last design verified design. Changes of external load or internal force distribution may impose unintended additional load on new or existing areas, e.g. relocation of trays inside a distillation column.

D) DISPLACEMENT-INDUCED FAILURES

Any modification that could cause displacement-induced failure. Displacement-induced failures could be caused by load-carrying attachments between seismically independent bodies, e.g. rigid piping connections, platform connections including permanent structures or equipment permanently installed for the purposes of maintenance or inspection.

E) PROCESS DESIGN CONDITIONS

Changes in process conditions include, for example, increases in pressure, increases or decreases in temperature, increase in fill height, or change in load distribution. All changes in process conditions require design verification. A change in design conditions that decreases the remnant strength and the resulting ability of the altered equipment to withstand seismic effects by more than 10% would trigger re-assessment of the seismic design.

²ACPPE Appendix A Clause 4.5

COMMENT D.5(E)

An increase in corrosion allowance as part of a fitness for service assessment is not included in this clause as AS/NZS 3788 Clause 5.2.4 allows corrosion/erosion levels above those specified in the original design without the need for re-design verification, provided that a remaining life assessment is conducted to calculate the acceptable limits. It is expected that a remaining life assessment would include an assessment on the effect on seismic strength.

Remaining life assessment (to AS3788) is not considered to be an alteration, merely a tool to assess ongoing suitability for service (unless such assessment triggers requirements for alterations or repairs)

Where an item of pressure equipment is being re-purposed, then an increase in corrosion allowance should be considered as an alteration affecting seismic strength.

F) RELOCATION

Relocating existing equipment to a higher level of seismic risk. Relocation could mean a higher elevation in the same building; or a new site with higher Z hazard factor, poorer ground, or proximity to a major fault. Relocation on the same site at ground level would not normally be considered as increasing seismic risk

G) EXTENSION OF DESIGN LIFE

Where the known design life of an item of plant has been exceeded (for items post PECPR 1999), an extension of Design life shall be considered to be an alteration affecting seismic strength. Designers should refer to 6.2 below when selecting the remaining seismic design life in these situations.

A reassessment of seismic design life of existing older plant certified prior to PECPR 1999 could be considered on an individual basis as part of ongoing remaining life assessment.

Site specific assessment of current seismic conditions should be undertaken and compared to known (or standard requirement) design criteria.

COMMENT D.5(G)

Prior to PECPR 1999, a design life was not required to be documented for most pressure equipment (exceptions included boilers operating in the material creep range and pressure vessels with high cyclic loading having a fatigue design life) and hence criterion g) would only apply to items installed after this time.

6.0 Reduced seismic design loading

6.1 General

This Practice Note acknowledges that an increased level of risk of failure of existing structures in an earthquake is considered acceptable in building regulations and society in general. The intention of this appendix is that existing pressure equipment should be required only to meet a loading requirement based on the risk classification and the designer's engineering judgement of the meaning of "all practicable steps".

6.2 Reduced seismic design load

Where the designer is designing an alteration affecting seismic strength, as defined above, and a reduction of the seismic design load below the code requirements appears necessary, the designer may decrease the design seismic load to the appropriate value from Table D2 below provided that the resultant seismic strength is not lower than prior to the alteration.

If a site specific seismic assessment is carried out and results in 100% of the current required seismic load being lower than the original design load, then the current seismic load can be used.

This clause is not intended to apply to equipment that is relocated from another site or to an elevated location on the same site. Relocated equipment shall be assessed against 100% of current requirements and upgraded if necessary.

When conducting a seismic assessment on an existing item of pressure equipment to this appendix, a remaining design life of not less than 25 years shall be used, as recommended in Section 5.1.6 of this Practice Note.

Table D2: Seismic Reduction Factors

PRESSURE EQUIPMENT RISK CATEGORY	MINIMUM SEISMIC DESIGN LOADS FOR EXISTING PRESSURE EQUIPMENT
High	<ul style="list-style-type: none"> • 50% of current requirement • 100% of requirement at the time of installation (note i & ii) • 0.1g
Medium	Maximum of: <ul style="list-style-type: none"> • 34% of current requirement (note ii) • 100% of requirement at the time of installation (note i & ii) • 0.1g
Low	Maximum of: <ul style="list-style-type: none"> • 34% of current requirement (note iii) • 100% of requirement at the time of installation (note i & ii) • 0.1g

NOTES FOR TABLE D2:

- If 100% of current requirement is lower than the requirement at the time of installation, then the current requirement can be used provided it is not less than 0.1g.
- For guidance on seismic design loads that were required at the time of manufacture, the following list shows the appropriate loading codes for New Zealand from 1935. Engineering New Zealand keep a copy of these publications which can be loaned to members.

Prior to 1935	No Requirement
1935–1955	1935 Model Bylaws, Clause 205
1955–1965	NZSS 95, “Model Building Bylaw”, Part IV
1965–1968	NZSS 1900, “Model Building Bylaw”, Chapter 8
1968–1976	NZSS 1900 and M.O.W. Code of Practice for the Design of Public Buildings
1976–1994	NZS 4203 “Code of practice for General Structural Design and Design Loadings for Buildings” (new editions in 1984 and 1992)
1994–2000	NZS 4203 and AS/NZS 1200:1994, Appendix J
2000–2004	NZS 4203 and AS/NZS 1200:2000, Appendix I
2004–2015	NZS 1170.5 “Structural Design Actions Part 5: Earthquake Actions – New Zealand” or NZS 4203 and AS/NZS 1200:2000, Appendix I

iii. This Practice Note acknowledges that while a level of less than 34% for low risk pressure equipment is likely to be acceptable for low risk equipment in the majority of situations, no quantitative justification of the risk has been developed at the time of publication. Should a need arise to use a value of less than 34%, the designer may do so provided that an engineering justification is provided and formally accepted by the controller, design verifier, and equipment inspector.

COMMENT D.6.2

One of the intentions of Table D.2 is to prevent new or second-hand equipment from being installed without design verification and then once operational, having the seismic assessment conducted against the reduced loading requirement.

The minimum requirement of 0.1g is based on the requirement of N.Z.S.S. 95 “Model Building Bylaw”, Part IV, Clause 412, which is effectively the first seismic design load requirement in New Zealand and would therefore be the minimum load that would have been required at the time of construction of any pressure equipment installed in New Zealand.

6.3 Meaning of “all practicable steps”

The HSE Act 1992 gives the following meaning to “all practicable steps”:

In this Act, all practicable steps, in relation to achieving any result in any circumstances, means all steps to achieve the result that it is reasonably practicable to take in the circumstances, having regard to:

- a) the nature and severity of the harm that may be suffered if the result is not achieved; and
- b) the current state of knowledge about the likelihood that harm of that nature and severity will be suffered if the result is not achieved; and
- c) the current state of knowledge about harm of that nature; and
- d) the current state of knowledge about the means available to achieve the result, and about the likely efficacy of each of those means; and
- e) the availability and cost of each of those means.

7.0 Design guidelines for alterations to existing pressure equipment

7.1 Current Standards

The analysis of any proposed significant alteration shall be conducted in accordance with current design and loading standards; however the designer shall use the lower value of the material design strengths from either the original or current design code unless material test certificates for the actual materials used or test results from tokens can be provided that show strengths higher than the current minimum strength requirements. Allowable design strengths can be determined using code rules based on actual material strengths when these are known.

7.2 Alternative analysis methods

When the seismic analysis of a pressure equipment system shows that certain components are overstressed, it is best to modify the design and support arrangement to reduce stresses to within the allowable limits. Where this is not feasible e.g. due to a lack of accessibility and/or when the cost

of modifications would be prohibitive, the designer may consider a more advanced, less conservative, analytical technique. Several advanced techniques are suggested as follows:

1. More accurate Stress Intensification Factors (SIFs). The use of finite element analysis (FEA) software or results of testing and analyses (e.g. to ASME B31J) may show that the SIFs used in the ASME B31 codes are overly conservative.
2. Further geotechnical analysis to determine more accurate soil class and/or advanced ground response modelling to allow the use of a time history analysis.
3. An elastic analysis using traditional software such as Caesar II or AutoPIPE with the allowable stresses as defined in ASME B31.Ea. This method applies only to ductile (as defined in ASME B31.E) piping systems.
4. An elastic analysis using FEA software with methodology as defined by ASME III Div 1 App F, ASME VIII Div 2 or API 579.
5. A limit load (plastic stability analysis) using FEA software with methodology defined by ASME III Div 1 App F, ASME VIII Div 2 or API 579.
6. An elastic-plastic analysis using FEA software with methodology as defined by ASME III Div 1 App F, ASME VIII Div 2 or API 579.

In all methods above, the designer shall use the load cases from the chosen standard and the allowable stress determined from Section 7.1.

Except for methods 1 and 2, the designer shall not use a reduced seismic design factor of less than 66% in conjunction with any of the advanced techniques above.

It is also important to note that the working stress design factor recommended in this Practice Note at Section 5.18 applies to systems analysed using working stress design methods. It is therefore improper to use this factor for systems analysed using the limit load (plastic stability analysis) and elastic-plastic analysis using FEA software methods.

COMMENT D.7.2

Advanced methods can have much lower factors of safety than the traditional code calculations and therefore the lower seismic design load cannot be justified without further work. NZSEE has recommended however, that 66% of current standard is the minimum level of risk that would be acceptable in a societal sense, e.g. for IL=2, an increase from 10% to 20% chance of exceedance. This level therefore, can be justified regardless of the method used.

7.3 Extent of seismic assessment

7.3.1 PIPING SYSTEMS

Where required by Section 5, the analysis shall include a sufficient extent of the existing system to capture its influence on the new/modified sections. Experience has shown that a detailed analysis can sometimes reveal the existing system is seriously deficient in its ability to withstand current seismic design loads or indeed the seismic design loads applicable at the time of first construction. In these situations, it is the designer's duty to advise the controller of the "out of scope" deficiency.

7.3.2 VESSELS, TURBINES AND BOILERS

Where required by Section 5, the seismic assessment may be limited to the seismic load carrying components that are affected by the alteration. In pressure vessel designs conducted to historic codes, it is likely that certain geometry/features may not comply with modern codes. In these situations, non-affected components need not be redesigned, however where components are affected and seismic load carrying, then designer is faced with two options:

- a) Redesign of the component to meet current requirements.

b) Using advanced analysis techniques e.g. FEA

7.4 Interface between new and existing pressure equipment in the alteration

Where new components or fittings are to be retrofitted to existing equipment, the new portion of the alteration shall comply with current pressure equipment Codes when 100% of the current seismic design loading code is applied to both the new and existing portions of the equipment to be altered. This Practice Note permits the seismic load to be factored down for the existing portion of the equipment in accordance with Section 6.2 of this Appendix.

The existing portion of the equipment may have to be retrofitted with additional reinforcing or structures such as compensation pads or supports to prevent excess seismic load being transferred to or from the new portion. The existing portion shall be re-design verified and new and existing supports certified by a design statement as required by the ACPPE and described in Appendix F of this Practice Note. Section 7.6 below has guidelines on assessment of equipment where there is insufficient documentation for design verification.

One common issue with retrofitting new equipment to existing systems is that the relative displacement of the existing system compared to the new system (or vice versa), can cause overstress at the tie-in location (see Section 5.20). Designers may choose to take measures to ensure that the new and existing systems remain independent under seismic actions. In practice, this may involve the use of devices such as flexible couplings and isolators. Flexible joints should be specified and/or designed by a competent designer. There is no requirement for re-design verification for existing pressure equipment that is isolated in this manner, however controllers should bear in mind the obligation to evaluate the risk posed by existing pressure equipment as described in Section 3 above.

Upgrades to existing equipment should be implemented when the initial modification is made, however industry experience has shown that there is often insufficient time and budget to implement these until the next planned shutdown period. In these situations, the controller should agree a risk mitigation plan with the designer and pressure equipment inspector, e.g. restricted access, shrouds etc., subject to an acceptable residual risk being achieved with the interim measures in place.

7.5 Extent of analysis of foundations and supports

The seismic loading analysis shall include the supports and foundations and shall consider their interaction with the pressure equipment.

The extent of analysis for foundations and supports should typically include:

- Where the pressure equipment is 20% or less of the combined weight of the supporting structure and equipment (e.g. a pipe or small vessel supported within a building), this Practice Note recommends the analysis considers only the main supporting elements to demonstrate compliance with the current building code and PECPR regulations.
- Where the pressure equipment is more than 20% of the combined weight of the supporting structure and equipment (e.g. buildings, platforms and towers supporting large vertical pressure vessels such as evaporators and distillation columns, or pipe bridges supporting multiple pressure pipelines), NZS 1170.5 Section 8.1.1 requires a special study. In this case, this Practice Note recommends the analysis extends to the foundations of the main structure which is supporting the pressure equipment through the methods given in the body of this Practice Note.

7.6 Pressure Equipment with no documentation

PECPR Regulation 8(1)(b) requires pressure equipment to possess a valid Design Verification certificate. Engineering New Zealand recognises that there is a significant amount of pressure equipment installed prior to 1999 which is covered by PECPR but is outside of the scope of the previous Boilers, Lifts and Cranes Act 1950. This equipment therefore possesses no certification. Furthermore, experience with newer equipment shows that original design documentation has often been discarded or lost by the controller.

Should the owner wish to modify existing equipment, and the designers have limited or no design data available, they are faced with having to re-design the equipment. The main dilemmas in this situation are:

- Whilst the equipment can be surveyed and stresses calculated, the designer must usually make assumptions to assess the code compliance, e.g. a suitable design code, materials grade, quality of welds and inaccessible geometry.
- Retrospective fabrication inspection is very difficult if not impossible to achieve to the level required by the AOPPE.

Where an alteration to pressure equipment is required and there is no or limited design documentation available, Table D3 below provides a recommended process. These requirements are not intended to apply to alterations that do not trigger a Section 5 seismic assessment.

Designers are also referred to API 510 Section 7.7 for further recommendations.

All assumptions made regarding existing pressure equipment should be accepted by the controller, the pressure equipment inspector and the design verifier.

Table D3: Documentation requirements for alterations requiring a seismic assessment

Requirements for Seismic Assessments			
	High Risk	Medium Risk	Low Risk
Design Conditions Determine the design conditions	If accurate design conditions cannot be determined from control system parameters, then monitoring equipment shall be used to gain accurate data.	If accurate design conditions cannot be determined from control system data, reasonable, conservative assumptions may be used.	
Geometry Survey the equipment	A complete and exhaustive survey shall be conducted and any available documentation shall be verified against the as-built equipment.	The survey may be limited to the components requiring re-design and reasonable, conservative assumptions may be made regarding inaccessible geometry.	
Materials Establish materials of construction	Material types and grades shall be ascertained to a reasonable level of certainty. In practice, this may require tokens to be taken and tested.	Reasonable, conservative assumptions may be made regarding material type and grade.	
Weld Quality Establish weld details and quality.	Retrospective NDT should be undertaken appropriate to the chosen design code and assumed quality of welds.	Reasonable, conservative assumptions may be made regarding weld details and quality.	

Requirements for Seismic Assessments			
Final Documentation Required	Design documentation and verification shall be produced for the entire equipment system as redesigned.	The design documentation and verification shall be produced for the alteration, the components redesigned, the affected components of the existing system and an overall general arrangement showing the design assumptions and enough information to enable the controller to safely operate the equipment.	The design documentation and verification shall be produced for the alteration, the components redesigned and the affected components of the existing system.

COMMENT D.7.5

Some equipment in the medium risk category, e.g. LPG storage tanks, may warrant more certainty in terms of materials, geometry and weld quality. In some circumstances achieving a more accurate assessment may yield a more economic result in terms of the required extent of retrofit work.

APPENDIX E

EXAMPLE CALCULATIONS

Example E1: Skirt mounted vertical pressure vessel at ground level

Three types of analysis could be considered in accordance with NZS 1170.5 Section 6:

1. Equivalent static force analysis.
2. Modal response spectrum analysis.
3. Special study as a supplement to the modal response spectrum analysis, if required by the Purchaser.

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ASSUMPTIONS:

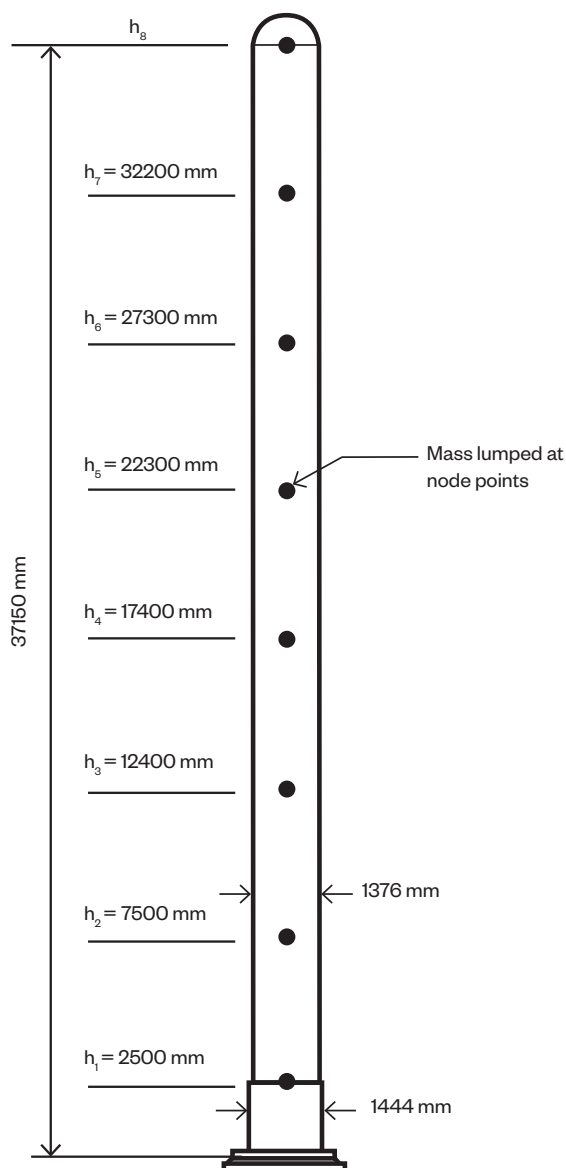
- The design is to use the recommendations of this Practice Note as the preferred work practice.
- As no site specific seismic hazard study has been undertaken, the minimum seismic coefficient is given in Table H1.
- Horizontal torsion effects need not be considered for these types of structures where the geometry and mass distribution is symmetrical about both principal axes at all heights.
- For vessels operating at elevated temperatures, the applicable steel yield stress and modulus of elasticity shall be calculated for the normal operating temperature.
- The vessel is located within a plant that is close to occupied dwellings and has a design pressure such that if released, toxic fumes would spread beyond the plant's boundary. The Importance Level has been set at IL = 4.
- The overall height is 37.15 m and the internal diameter of the pressure retaining vessel is set at 1300 mm .
- The mass distribution of the vessel is irregular according to NZS 1170.5 Clause 4.5.1.1.
- The vessel is to operate at a temperature of 210°C.
- The vessel is to be installed in a site in North Taranaki that has a Site Subsoil Class of C.
- The wall thicknesses for the shell and skirt shall be reviewed for their ability to sustain earthquake induced actions using Tables 12.2.4, 12.2.6, 12.4 and 12.5 of NZS 3404:1997.
- Any modal response spectrum analysis shall comply with Section 6.3 of NZS 1170.5.

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Figure 13: Skirt mounted vertical pressure vessel.



**Table E1: Equivalent static earthquake coefficients calculation
for a vertical pressure vessel**

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
System Information:			
Operating Temperature	T_{op}	210°C	Young's Modulus reduced by 6.7%. No effect on F_y
Overall Height	h_n	37.15 m	
Skirt Internal Diameter	D_s	1372 mm	
Vessel Internal Diameter	D_v	1300 mm	
Skirt thickness	t_{skirt}	36 mm	$F_y = 325 \text{ MPa}$ – from material spec
Shell thickness - bottom strake	t_{shell}	36 mm	$F_y = 315 \text{ MPa}$ – from material spec
			$\therefore F_y > 300 \text{ MPa}$. Skirt or shell cannot be a Category 1 member as per NZS 3404 Table 12.4
Plate Element Slenderness:			Use NZS 3404 Table 12.5 with outside diameters and thicknesses from above
Member Category 2	λ_{e2}	42	$t_{skirt} \geq 45 \text{ mm}$ and $t_{shell} \geq 42 \text{ mm}$
Member Category 3	λ_{e3}	60	$t_{skirt} \geq 32 \text{ mm}$ and $t_{shell} \geq 29 \text{ mm}$
Member Category 4	λ_{e4}	170	$t_{skirt} \geq 11 \text{ mm}$ and $t_{shell} \geq 10 \text{ mm}$
	t_{skirt} & t_{shell}	36 mm	<u>Minimum primary member category = 3</u> <u>Minimum structural ductility category = 3</u> From NZS 3404 Table 12.2.6
Weight of components:			
Weight of support skirt	W_s	35 kN	From vessel vendor data
Weight of vessel	W_p	370 kN	
NZ Building Code Category – Clause A1 Classified Use		<i>Ancillary</i>	May be exempt some amenity provisions but required to comply with structural and safety provisions of NZBC
Structural Type:		Nonbuilding structure not similar to buildings	
Importance Level:	IL	4	3 limit states = ULS, SLS1 & SLS2
Design Working Lives:			Plant's life to be 50+ years – from client
Pressure vessel:			
AS 1210 Paragraph J2		25 years	Use AS 1210 as a guide and Table 1 of this Practice Note.
Foundation:			
NZ Building Code Clause B2		50 years	Indefinite Life to be specified for Building Consent
Fundamental period:			

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Combined skirt + vessel	T_1	1.0 secs	From modal analysis. Assume applicable for all limit states

Determine whether the system meets the requirements for Equivalent Static Analysis (ESA):

Vertical Regularity Check:

NZS 1170.5 Clause 4.5.1.2	No	Assumed as plate thicknesses vary up vessel
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Suitability of Equivalent Static Analysis Method Check:

NZS 1170.5 Clause 6.1.3.1(a)	$h_n < 10$ m	No	
NZS 1170.5 Clause 6.1.3.1(b)	$T_1 < 0.4$ secs	No	
NZS 1170.5 Clause 6.1.3.1(c)	Horizontal regular	Yes	
	Vertical regular	No	
	$T_1 < 2.0$ secs	Yes	May not be able to use ESA
			<u>Modal Response Spectrum Analysis required but do ESA for checking MRSA</u>

Determine the earthquake parameters and site hazard spectra:

Earthquake Return Periods:

AS/NZS 1170.0 Table 3.3

Pressure vessel:

ULS	1000 years	$E_{wsd} = 0.8E_u$ and overstress allowance as per vessel design standard
SLS2	250 years	$E_{wsd} = E_{sls2}$ and no overstress allowance
SLS1	25 years	$E_{wsd} = E_{sls1}$ and no overstress allowance

Foundation:

ULS	2500 years
SLS2	500 years
SLS1	25 years

Site Subsoil Class:

NZS 1170.5 Clause 3.1.3.4	C	NZS 1170.5 Table 3.1 for Spectral Shape Factor $C_h(T)$
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Spectral Shape Factor:

NZS 1170.5 Table 3.1	$C_h(T_1)$	1.19	Site Subsoil Class = C and $T_1 = 1.0$ secs
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Hazard Factor:

NZS 1170.5 Clause 3.1.4	Z	0.18	Suitable for North Taranaki
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Return Period Factors:

NZS 1170.5 Table 3.5

Pressure vessel:

R_u	1.3
R_{sls2}	0.75
R_{sls1}	0.25

Foundation:

R_u	1.8
R_{sls2}	1.0
R_{sls1}	0.25

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
NZS 1170.5 Clause 3.1.1	$Z R_u$	0.32	Check that $Z * R_u = 0.18 * 1.8 < 0.7$. ∴ O.K
Near-fault factor:			
NZS 1170.5 Clause 3.1.6	$N(T, D)$	1.0	Not adjacent to any major faults, i.e. $D > 20$ km
Horizontal Elastic Site Hazard Values:			
NZS 1170.5 Eqn 3.1(1)			$C(T_1) = C_h(T) R Z N(T, D)$
Vessel design:			
ULS	$C(T_1)_{uls}$	0.278	$C(T_1)_{uls} = 1.19 * 1.3 * 0.18 * 1.0$
SLS2	$C(T_1)_{sls2}$	0.161	$C(T_1)_{sls2} = 1.19 * 0.75 * 0.18 * 1.0$
SLS1	$C(T_1)_{sls1}$	0.054	$C(T_1)_{sls1} = 1.19 * 0.25 * 0.18 * 1.0$
Foundation design:			
ULS	$C(T_1)_{uls}$	0.386	$C(T_1)_{uls} = 1.19 * 1.8 * 0.18 * 1.0$
SLS2	$C(T_1)_{sls2}$	0.214	$C(T_1)_{sls2} = 1.19 * 1.0 * 0.18 * 1.0$
SLS1	$C(T_1)_{sls1}$	0.054	$C(T_1)_{sls1} = 1.19 * 0.25 * 0.18 * 1.0$

Determine the design earthquake actions for the pressure vessel:

Minimum earthquake requirements: Use this Practice Note as the preferred work practice

This Practice Note's recommendation:

Earthquake Return Period		1000 years	From above for the vessel's seismic design
Hazard Factor	Z	0.18	From above for North Taranaki
This Practice Note Table H1	$0.8 * C_d(T_1)$	0.30	$0.8 * C_d(T_1) = 0.30$ Value from Table H1
	$C_d(T_1)$	0.38	$C_d(T_1) = 0.30 / 0.8$. For the ULS design of the foundation

Structural Ductility Factors:

ULS - NZS 3404 Table 12.2.4	μ_{uls}	1.25	Vessel and skirt taken as a nominally ductile structure
NZS 1170.5 Clause 4.3.2 SLS2	μ_{sls2}	1.0	Assume no ductility to happen for SLS2 event
NZS 1170.5 Clause 4.3.2 SLS1	μ_{sls1}	1.0	

Structural Performance Factors:

NZS 1170.5 Clause 4.4.2

Pressure vessel:

ULS	S_{pe}	0.925	With $\mu = 1.25$ for pressure envelope
SLS2	S_{pe}	1.0	With $\mu = 1.0$
SLS1	S_{pe}	0.7	Allow code value for SLS1 case

Skirt for pressure vessel:

ULS	S_{ps}	0.925	With $\mu = 1.25$. ∴ $S_{puls} = 0.925$
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DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
SLS2	S_{ps}	1.0	$S_{psls2} = 1.0$
SLS1	S_{ps}	0.7	$S_{psls1} = 0.7$
Inelastic Spectrum Scaling Factors:			
NZS 1170.5 Clause 5.2.1.1 - ULS	$k\mu_{uls}$	1.25	As $T_1 > 0.7$ sec
SLS2	$k\mu_{sl2}$	1.0	
NZS 1170.5 Clause 5.2.1.2 - SLS1	$k\mu_{sl1}$	1.0	
Unscaled 5% Damped Horizontal Design Actions for pressure vessel:			NZS 1170.5 Eqn 5.2(1)
			$C_d(T_1)_{uls} = C(T_1)_{uls} * S_p / k\mu$
ULS	$C_d(T_1)_{uls}$	0.206	$C_d(T_1)_{uls} = 0.278 * 0.925 / 1.25$
NZS 1170.5 Eqn 5.2(2)	$(Z/20 + 0.02)R_u$	0.038	$\therefore C_d(T_1)_{uls} = 0.206$
SLS2	$C_d(T_1)_{sl2}$	0.161	$C_d(T_1)_{sl2} = 0.161 * 1.0 / 1.0$
SLS1	$C_d(T_1)_{sl1}$	0.037	$C_d(T_1)_{sl1} = 0.054 * 0.7 / 1.0$
Review P-delta effects:			
NZS 1170.5 Clause 6.5.2(a)	$T_1 > 0.4$ secs	Yes	Need to consider P-delta effects
NZS 1170.5 Clause 6.5.2(b)	$h_n > 15$ m	Yes	
	$T_1 > 0.6$ secs	Yes	Need to consider P-delta effects
			Therefore include P-delta effects by Method A
ULS P-Delta Effects Factor:			
NZS 1170.5 Eqn 6.5(3)	k_p	0.017	With $\mu = 1.25$
NZS 1170.5 Eqn 6.5(2) - ULS	$C_d(T_1)$	0.273	$C_d(T_1) = 1.19 * 1.3 * 0.18 * 1.0 * 1.32 * 0.925 / 1.25$
	k_1	1.062	$k_1 = [k_p + C_d(T_1)] / C_d(T_1)$
			Increase of 6.2% in base shear to account for P-Delta effects
Material Code Scaling Factor:			
ULS	k_2	1.0	No steel standard requirement to scale ULS earthquake
\therefore	K	1.062	$K = k_1 * k_2 = 1.062 * 1.0$
System Damping Factors:			
ULS	$C_f(2)$	1.32	Chosen values: Damping = 2% of critical
SLS2	$C_f(1)$	1.53	Damping = 1% of critical
SLS1	$C_f(0.5)$	1.67	Damping = 0.5% of critical
Horiz. Design EQ Forces:			
ULS – To avoid collapse	$E_u = V_{uls}$	$0.29 W_t$	$= C_d(T_1)_{uls} * C_f(2) * K * W_t$ $= 0.206 * 1.32 * 1.062 * W_t$
WSD – No loss of contents	$E_{wsc} = 0.8E_u$	$0.23 W_p$	$= 0.8 * 0.29$
			Too low for vessel and support as min. value is 0.30
\therefore For pressure envelope	E_{wsc}	0.30	
\therefore For skirt and foundation	E_u	0.38	

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
SLS2 – To remain operational	$E_{sls2} = V_{sls2}$	$0.25 W_t$	$= C_d(T_1)_{sls2} * C_f(1) * W_t$ $= 0.161 * 1.53 * W_t$
SLS1 – To sustain no damage	$E_{sls1} = V_{sls1}$	$0.06 W_t$	$= C_d(T_1)_{sls1} * C_f(0.5) * W_t$ $= 0.037 * 1.67 * W_t$

Foundation and vessel HD bolts to be designed for the minimum of the over-strength of collapse mechanism or the elastic level earthquake.

Determine Horiz. Elastic Level EQ Forces on foundation and HD bolts:

Horizontal Elastic Site Hazard Value:

ULS Spectral Shape Factor	$C_h(T_1)$	1.19	From above
NZS 1170.5 Table 3.5	R_u	1.8	From above
NZS 1170.5 Clause 3.1.4	Z	0.18	From above
NZS 1170.5 Clause 3.1.6	N(T, D)	1.0	From above
Horizontal Elastic Site Hazard Value:			
NZS 1170.5 Eqn 3.1(1)			$C(T_1) = C_h(T) R Z N(T, D)$
	$C(T_1)$	0.386	$C(T_1) = 1.19 * 1.8 * 0.18 * 1.0$
NZS 3404 Clause 12.9.1.2.2 (4) (b)	μ	1.0	
NZS 1170.5 Clause 4.4.1	S_p	1.0	Can also be used to check overall stability of structure
NZS 1170.5 Clause 5.2.1.1	$k\mu$	1.0	
Unscaled ULS Design Action	$C_d(T_1)^o_{uls}$	0.386	$C_d(T_1) = 0.386 * 1.0 / 1.0$
ULS P-Delta Effects Factor:			
NZS 1170.5 Eqn 6.5(3)	k_p	0.015	$k_p = 0.015 + 0.0075 (\mu - 1)$, where $\mu = 1.0$
ULS damping for elastic case	$C_f(1)$	1.53	Damping = 1% of critical
NZS 1170.5 Eqn 6.5(2) - ULS	$C_d(T_1)$	0.59	$C_d(T_1) = 1.19 * 1.8 * 0.18 * 1.0 * 1.53 * 1.0 / 1.0$
	k_1	1.025	$k_1 = [k_p + C_d(T_1)] / C_d(T_1)$ Increase of 2.5% in base shear to account for P-Delta effects
ULS P-Delta Scaling Factor	k_1	1.025	NZS 1170.5 Eqn 6.5(2)
	K	1.025	$= k_1 * k_2 = 1.025 * 1.0$; $k_2 = 1.0$ from above
	$C_f(1)$	1.53	Damping = 1% of critical for elastic case
ULS – To avoid premature non-ductile failure and overturning or sliding failure	$E_{max} = V^o_{uls}$	$0.61 W_t$	$= C_d(T_1)^o_{uls} * C_f(1) * K * W_t$ $= 0.386 * 1.53 * 1.025 * W_t$ Need to check that any yielding doesn't spread to upper strakes thinner than 36 mm

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
			O.K. as min. AOPPE value = 0.38 from above
			This Practice Note recommended minimum value is 0.30, from Table H1.
Perform a deflection check to see if sideways displacements are within required limits:			
Horizontal Design EQ Displacements:			
No. of storeys assumed for ESA		8	
Storey heights	h_1, \dots, h_8		= (2.5, 7.5, 12.4, 17.4, 22.3, 27.3, 32.2, 37.2) m
Capacity design undertaken	Yes		Static push-over undertaken to determine collapse mechanism for HD Bolt and foundation sizing. Scaled elastic deflections to be used for deflection profiles. As vertical cantilever type structure, no beam sideways mechanisms need be considered
ESA undertaken?	Yes		Deflection scaling factors in NZS 1170.5 Table 6.1 can be used
	k_d	0.85	NZS 1170.5 Table 6.1
Included P-Delta effects in force analysis?	Yes		Elastic deflections from force analysis already scaled for P-Delta effects
Distance to nearest boundary or deflected element within the plant	400 mm		Clearance to adjacent deflected structure using its max ULS deflected shape
Scaling factors:			
P-Delta scaling factor	k_1	1.062	From above for vessel design using 1000 per year RPEQ
ESA Elastic Analysis scaling factor	α	0.800	$\alpha = k_d / k_1 = 0.85 / 1.062$
ESA Inelastic scaling factor	μ	1.25	$\mu = \mu_{uls}$ from NZS 1170.5 Cl. 7.2.1.1
ESA ULS1 deflection scaling factor	β	1.328	$\beta = \mu * k_1$ from NZS 1170.5 Cl. 7.2.1.1(a) and 7.2.1.2
ULS Deflections as per NZS 1170.5 Clause 7.2:			
ESA Force analysis deflections	δ_{iesa}	(8, 28, 53, 81, 111, 146, 186, 225) mm.	Results from EQ analysis $E_u = 0.29 W_t$ $S_p = \mu$ 1.25
Elastic deflections	$\delta_{elastic}$	$\delta_{elastic} = \alpha * \delta_{iesa}$ (6, 22, 42, 65, 89, 117, 149, 180) mm	
Elastic deflection at uppermost level	U_{el}	180 mm	$\delta_{elastic}$ for $h_8 = 37.2$ m
Scaled elastic deflections	$\delta_{scaled elastic}$	$\delta_{scaled elastic} = \mu * \delta_{elastic}$ (8, 28, 53, 81, 111, 146, 186, 225) mm	
ULS deflections	δ_{uls1}	$\delta_{uls1} = \beta * \delta_{elastic}$ (9, 30, 56, 86, 118, 155, 198, 239) mm	As a vertical cantilever, can ignore the beam sideways mechanism deflection calculation

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Maximum ULS deflection	δ_{uls}	$\text{Max}(\delta_{uls}) = 239 \text{ mm}$	$239 \text{ mm} < 400 \text{ mm}$ ∴ ULS deflections OK

Determine Inter-storey deflections as per NZS 1170.5 Clause 7.3:

Structure height	h_n	37.2 m	
Structural Performance Factor	S_{puls}	0.925	
Drift modification factor	k_{dm}	1.50	For $h_n > 30 \text{ m}$ – NZS 1170.5 Table 7.1
Unmodified ULS deflections	δ_{iesa}	(8, 28, 53, 81, 111, 146, 186, 225) mm	
Modified ULS deflections	δ_i	$\delta_i = \delta_{iesa} / K_i$ NZS 1170.5 Cl. 7.3.1.1 (8, 26, 50, 76, 111, 105 137, 175, 212) mm	
ULS inter-storey deflections	$\delta_{inter-storey}$	(8, 19, 24, 26, 28, 33, 38, 37) mm	
Scaled inter-storey deflections	$\delta_{scaled \text{ inter-storey}}$	$\delta_{scaled} = \delta_{inter-storey} * k_{dm}$ (11, 28, 35, 40, 42, 49, 56, 55) mm	
Peak inter-storey deflections	δ_{peak}	$\delta_{scaled} = \delta_{inter-storey} / S_{puls}$ (12, 31, 38, 43, 46, 53, 61, 60) mm	
Maximum inter-storey deflections	$\delta_{max \text{ inter-storey}}$	(12, 31, 38, 43, 46, 53, 61, 60) mm	
Maximum allowable inter-storey deflection	δ_{max}	2.5% storey height	
	h_i	(2.5, 7.45, 12.40, 17.35, 22.30, 27.25, 32.20, 37.15) m	
	δ_{max}	(63, 124, 124, 124, 124, 124, 124, 124) mm	Therefore have no inter-storey deflections that exceed the maximum allowable. Would check the pushover analysis to confirm that no mechanisms formed above the base level, i.e. a column sidesway mechanism forming

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Figure 14: Inter-Story Deflections - Ultimate Limit State

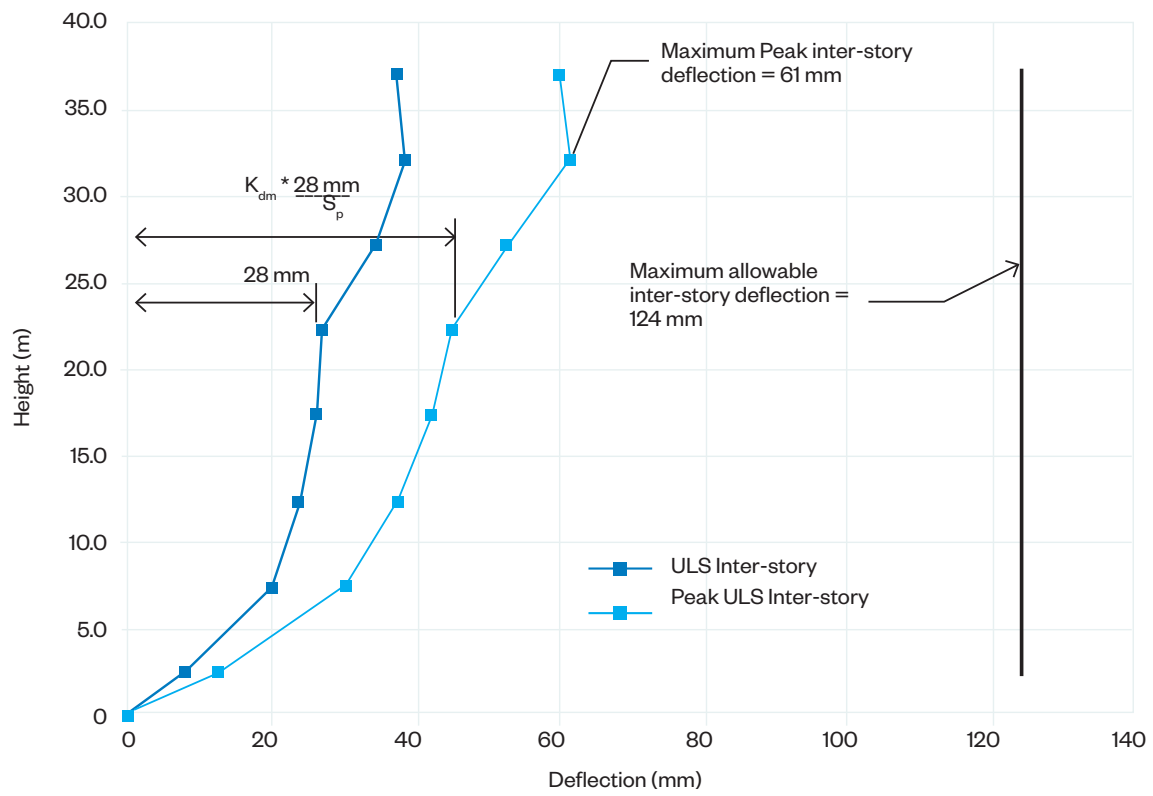
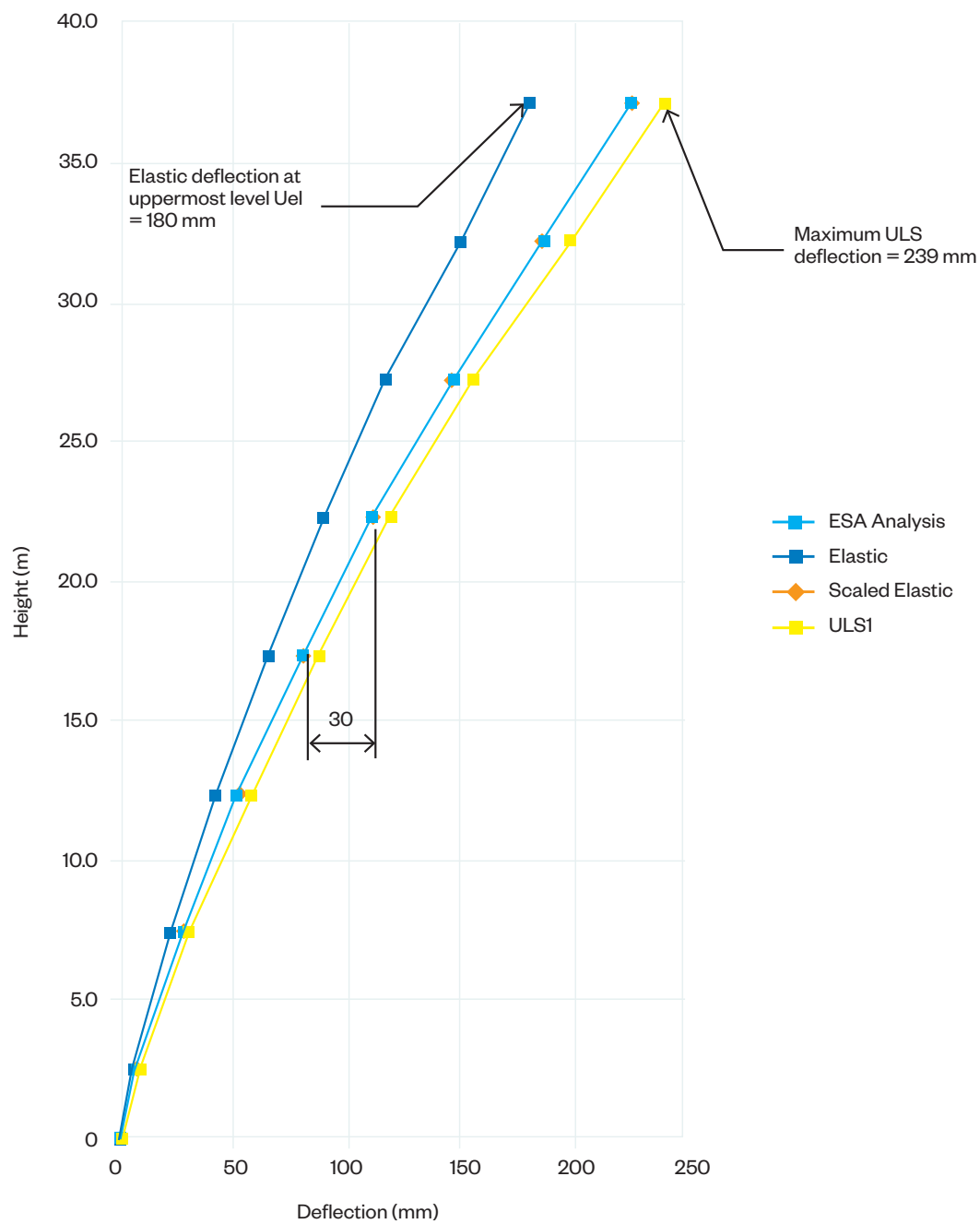


Figure 15: Deflections - Ultimate Limit State



Example E2: Pressure piping supported off the vessel from Example 1

Assume that a 300 NB pressure pipe with an AS 4343 hazard level of B coming off the top of the vessel analysed in Example 1.

It is to be supported vertically and restrained for sideways movement near the top of the vessel ($h_i = 36$ m). Then at 8 m intervals down the vessel, these supports are to restrain the pipe laterally, total of 5 supports up the vessel.

Design by following the preferred work practice, currently the AOPPE (2001). Design by using the recommendations of this Practice Note.

Figure 16: Piping support of a vertical pressure vessel

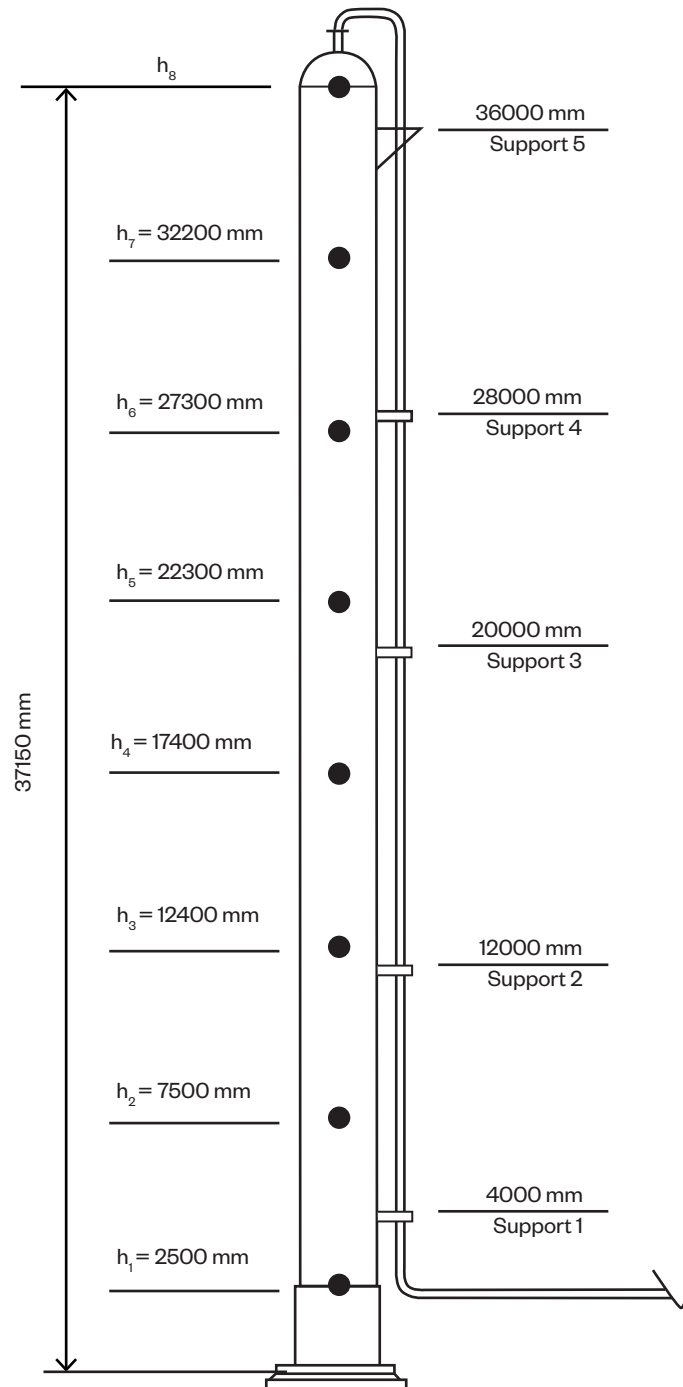


Table E2: Equivalent static earthquake coefficients calculation for pressure piping supported off Example 1

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
System Information:			
Weight of components:			
Weight of support	W_s	405 kN	From Example 1
Weight of piping	W_p	$<0.20 * W_p$	8 m @ 0.95 kN/m $< 0.2 * 405$
Overall Height:	h_n	37.2 metres	
Support Locations:			
Support 5	h_{i5}	36.0 m	Attachment heights
Support 4	h_{i4}	28.0 m	
Support 3	h_{i3}	20.0 m	
Support 2	h_{i2}	12.0 m	
Support 1	h_{i1}	4.0 m	
Level of allowable deformity		Low	Large dia. piping with low tolerance for leaks after an event, $\mu_p \leq 1.25$
Horizontal fundamental periods:			
Support including piping	T_1	1.0 secs	From modal analysis
Piping	T_p	0.06 secs	From pipe stress engineer
Vertical fundamental periods:			
Support including piping	T_{v1}	0.01secs	From modal analysis
Structural Type:		Part as per NZS 1170.5 Section 8	
Design Working Lives:			Plant's life to be 50+ years – from client
Pressure piping:		25 years	Table 1 of this Practice Note
Importance Level:	IL	4	3 limit states = ULS, SLS1 and SLS2
Determine the earthquake parameters and site hazard spectra:			
Earthquake Return Periods:	ULS	1000 years	$E_{wscd} = 0.8 E_u$
	SLS2	250 years	$E_{wscd} = E_{sls2}$ and no overstress allowance
	SLS1	25 years	$E_{wscd} = E_{sls1}$ and no overstress allowance
Site Subsoil Class:			
NZS 1170.5 Clause 3.1.3.4		C	NZS 1170.5 Table 3.1 for Spectral Shape Factor $C_h(T)$
Hazard Factor:			

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
NZS 1170.5 Clause 3.1.4	Z	0.18	Suitable for North Taranaki
Return Period Factors:			
NZS 1170.5 Table 3.5	R_u	1.3	
	R_{sls2}	0.75	
	R_{sls1}	0.25	
Near-fault factor:			
NZS 1170.5 Clause 3.1.6	N(T, D)	1.0	Not adjacent to any major faults, i.e. D > 20 km
Spectral Shape Factors:			
NZS 1170.5 Table 3.1	$C_h(0)$	1.33	Site Subsoil Class = C and $T_1 = 0.0$ secs
	$C(T_{v1})$	1.49	Site Subsoil Class = C and $T_{v1} = 0.01$ secs
5% Damped Elastic spectrum @ T = 0 sec:			
NZS 1170.5 Eqn 3.1(1)			$C(0) = C_h(0) R Z N(0, D)$
ULS	C(0)	0.311	$C(0) = 1.33 * 1.3 * 0.18 * 1.0$
SLS2	C(0)	0.180	$C(0) = 1.33 * 0.75 * 0.18 * 1.0$
SLS1	C(0)	0.060	$C(0) = 1.33 * 0.25 * 0.18 * 1.0$

Determine the design earthquake actions for the pressure piping:

Minimum earthquake requirements:		Use the 2001 DoL Approved Code of Practice for Pressure Equipment (AOPPE) this Practice Note as the preferred work practice	
AOPPE (2001) Clause 3.4.2		Use NZS 1200 for wind, snow and seismic	
NZS 4203 Clause 4.6.2.5	Z	1.0	North Taranaki value. Used in Table H1 of NZS 1200 Appendix I
Zone Factor			
NZS 1200 Appendix I	$0.8 * F_{ph} / W_p$	0.53	Minimum seismic coefficient for pressure equipment
	F_{ph} / W_p	0.53	Minimum seismic coefficient for support design:
This Practice Note's recommendation:			
Earthquake Return Period		1000 years	From above for the vessel's seismic design
Hazard Factor	Z	0.18	From above for North Taranaki
Table H1 this Practice Note	$0.8 * F_{ph} / W_p$	0.30	$0.8 * F_{ph} / W_p = 0.30 + 0.10 * 3/5$. Interpolated Value from Table H1 for 1000-year return period earthquake
	F_{ph} / W_p	0.38	$(F_{ph} / W_p)_{pmin} = 0.30 / 0.8$ - For the ULS design of the support

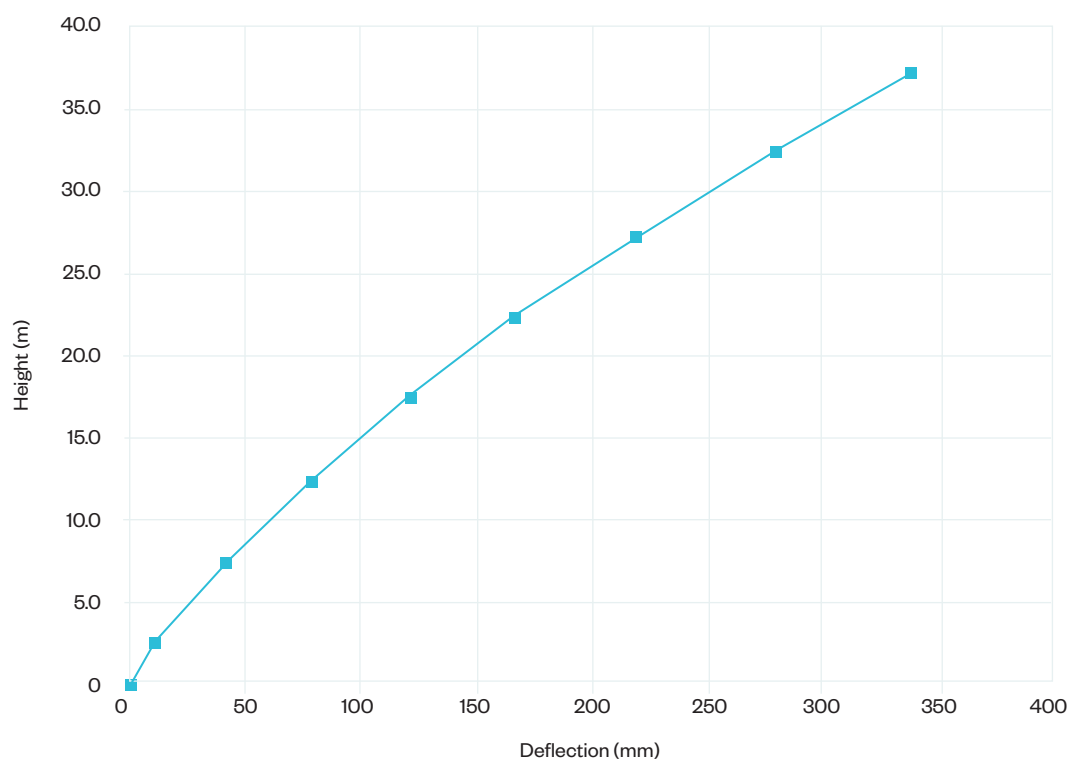
DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Parameters for the Parts and Components analysis:			
Displacement Ductility Factors:			NZS 1170.5 Clause 8.5
ULS	μ_p	1.25	Allow some ductility for ULS event
SLS2	μ_p	1.0	
SLS1	μ_p	1.0	
Part Category:			
NZS 1170.5 Table 8.1	ULS	P. 4	
	SLS2	P. 5	
	SLS1	P. 7	
Risk Factor for Parts:			NZS 1170.5 Table 8.1
ULS	R_{pu}	1.0	
SLS2	R_{psls2}	1.0	
SLS1	R_{psls1}	1.0	
Parts Response Factors:			NZS 1170.5 Table 8.2
ULS	C_{ph}	0.85	For horizontal forces
SLS2	C_{ph}	1.0	
SLS1	C_{ph}	1.0	
Parts Response Factors:			
ULS	C_{pv}	1.0	For vertical forces
SLS2	C_{pv}	1.0	
SLS1	C_{pv}	1.0	
System Damping Factors for supporting structure:			Chosen values from Example 1:
ULS	$C_f(2)$	1.32	Damping = 2% of critical
SLS2	$C_f(1)$	1.53	Damping = 1% of critical
SLS1	$C_f(0.5)$	1.67	Damping = 0.5% of critical
Floor Spectral Shape Factor:			
NZS 1170.5 Clause 8.4	$C_i(T_p)$	2.0	$T_p < 0.75$ secs
Floor Height Coefficients:			NZS 1170.5 Eqns 8.3(1), 8.3(2) and 8.3(3)
Support 5	C_{Hi}	3.0	C_{Hi} constant for $12\text{ m} \leq h_i \leq 37.2\text{ m}$ as per Eqn 8.3(3)
Support 2	C_{Hi}	3.0	Seismic forces will be the same for the upper 4 supports
Support 1	C_{Hi}	1.67	NZS 1170.5 Eqn 8.3(1) governs

	DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
	Horizontal Forces on Part:			
	Spectral Ordinate of the Part:			$C_p(T_p) = C(0) * C_{Hi} * C_i(T_p)$ - NZS 1170.5 Eqn 8.2(1)
	Support 5:			
	ULS	$C_p(T_p)$	1.87	$C_p(T_p) = 0.311 * 3.0 * 2.0$
	SLS2	$C_p(T_p)$	1.08	$C_p(T_p) = 0.180 * 3.0 * 2.0$
	SLS1	$C_p(T_p)$	0.36	$C_p(T_p) = 0.059 * 3.0 * 2.0$
	Support 2:			
	ULS	$C_p(T_p)$	1.87	$C_p(T_p) = 0.311 * 3.0 * 2.0$
	SLS2	$C_p(T_p)$	1.08	$C_p(T_p) = 0.180 * 3.0 * 2.0$
	SLS1	$C_p(T_p)$	0.36	$C_p(T_p) = 0.059 * 3.0 * 2.0$
	Support 1:			
	ULS	$C_p(T_p)$	1.04	$C_p(T_p) = 0.311 * 1.67 * 2.0$
	SLS2	$C_p(T_p)$	0.60	$C_p(T_p) = 0.180 * 1.67 * 2.0$
	SLS1	$C_p(T_p)$	0.20	$C_p(T_p) = 0.060 * 1.67 * 2.0$
	Horizontal Design EQ Actions:			$F_{ph} = C_p(T_p) * C_{ph} * R_p * C_f(\xi) * W_p$ - NZS 1170.5 Eqn 8.5(1)
	Support 5:			
V4 April 2016	ULS - No collapse	$E_u = F_{ph}$	$2.10 W_p$	$F_{ph} = 1.87 * 0.85 * 1.0 * 1.32 * W_p$
	WSD – No loss of contents	$E_{wsd} = 0.8E_u$	$1.68 W_p$	$= 0.8 * 2.10 + \text{overstress allowance as per ASME B31.3}$ No overstress allowance as this is an IL=4 structure
	SLS2 – To remain operational	$E_s = F_{ph}$	$1.65 W_p$	$F_{ph} = 1.08 * 1.0 * 1.0 * 1.53 * W_p$
	SLS1 – To sustain no damage	$E_s = F_{ph}$	$0.60 W_p$	$F_{ph} = 0.36 * 1.0 * 1.0 * 1.67 * W_p$
	Support 2:			
V4 April 2016	ULS - No collapse	$E_u = F_{ph}$	$2.10 W_p$	$F_{ph} = 1.87 * 0.85 * 1.0 * 1.32 * W_p$
	WSD – No loss of contents	$E_{wsd} = 0.8E_u$	$1.68 W_p$	$= 0.8 * 2.10 + \text{overstress allowance as per ASME B31.3}$ No overstress allowance
	SLS2 – To remain operational	$E_s = F_{ph}$	$1.65 W_p$	$F_{ph} = 1.08 * 1.0 * 1.0 * 1.53 * W_p$
	SLS1 – To sustain no damage	$E_s = F_{ph}$	$0.60 W_p$	$F_{ph} = 0.36 * 1.0 * 1.0 * 1.67 * W_p$
	Support 1:			
V4 April 2016	ULS – No collapse	$E_u = F_{ph}$	$1.17 W_p$	$F_{ph} = 1.04 * 0.85 * 1.0 * 1.32 * W_p$
	WSD – No loss of contents	$E_{wsd} = 0.8E_u$	$0.93 W_p$	$= 0.8 * 1.17 + \text{overstress allowance as per ASME B31.3}$ No overstress allowance
	SLS2 – To remain operational	$E_s = F_{ph}$	$0.91 W_p$	$F_{ph} = 0.60 * 1.0 * 1.0 * 1.53 * W_p$
	SLS1 – To sustain no damage	$E_s = F_{ph}$	$0.33 W_p$	$F_{ph} = 0.20 * 1.0 * 1.0 * 1.67 * W_p$
	Piping system analysed for:			

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
WSD			<p>WSD case of 1.68g in two principal directions analysed separately. Internal forces and moments = M_{iux} & M_{iuy}</p> <p>O.K. AS/NZS 1200 appendix 1 min. value = 0.53 Table H1 min. value = 0.30</p> <p>SLS2 and SLS1 cases not normally analysed during a pipe stress analysis unless their inertia loadings are greater than the ULS case.</p> <p>This Practice Note's recommended minimum WSD coefficient is 0.30 and this is exceeded at all the supports.</p> <p>Checking against typical coefficient given in Table H2 for an item at height for a 1000 year return period EQ and Z = 0.18:</p> $0.8E_u = 0.83 + 0.28 * 3/5 = 1.00$ <p>\therefore design coefficient = 1.68 > 1.00 \therefore O.K.</p>
Horizontal Design Actions due to EQ deflections:			
ULS deflections at pipe support locations		From Example E1:	
Inter-storey	$\delta_{\text{Inter-storey}}$	(8, 19, 24, 26, 28, 35, 38, 37) mm	
	δ_{Peak}	(12, 31, 38, 43, 46, 53, 61, 60) mm	
Totals	δ_{Peak}	(12, 43, 81, 124, 170, 223, 384, 240) mm	
Support 5	δ_5	325	In mm, interpolated for the actual support locations using the deflections calculated as per Clause 7.3 as required by NZS 1170.5 Clause 8.5.3
Support 4	δ_4	229	
Support 3	δ_3	146	
Support 2	δ_2	77	
Support 1	δ_1	21	
WSD			<p>Piping system analysed for:</p> <p>ULS deflection case with the above imposed deflections in two principal directions analysed separately.</p> <p>Internal forces and moments = $M_{\delta ux}$ and $M_{\delta uy}$</p>
Piping Horizontal Design Actions due to EQ acting on vessel			
WSD:			
X direction	M_{hux}	$(M_{iux}^2 + M_{\delta ux}^2)^{0.5}$	
Y direction	M_{huy}	$(M_{iuy}^2 + M_{\delta uy}^2)^{0.5}$	

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Vertical Forces on Piping:			
5% Damped Elastic spectrum @ $T = T_{v1}$ sec:			$C_v(T_{v1}) = 0.7 C(T_{v1}) R Z N(T_{v1}, D)$ – NZS 1170.5 Eqns 3.1(1) and 3.2(1)
ULS	$C_v(T_{v1})$	0.244	$C_v(T_{v1}) = 0.7 * 1.49 * 1.3 * 0.18 * 1.0$
ULS	C_{vd}	0.24	Eqn 5.4(2)
Vertical Design EQ Forces:			
ULS - No collapse	$E_u = F_{pv}$	$0.24 W_p$	$F_{pv} = C_{vd} * C_{pv} * R_p * W_p$ - NZS 1170.5 Eqn 8.5(2) $F_{pv} = 0.24 * 1.0 * 1.0 * W_p$
WSD – No loss of contents	$E_{wsd} = 0.8E_u$	$0.20 W_p$	$= 0.8 * 0.24 + \text{possible over-stress allowance}$
			WSD case of 0.20g in vertical direction.
			Internal forces and moments = M_{uv}
Piping Total Design Actions due to EQ			
WSD:			
X direction	M_{xu}	$M_{hux} + M_{uv}$	These earthquake actions need to be combined with normal operating pressure and temperature conditions for pipe stress checking.
Y direction	M_{yu}	$M_{huy} + M_{uv}$	

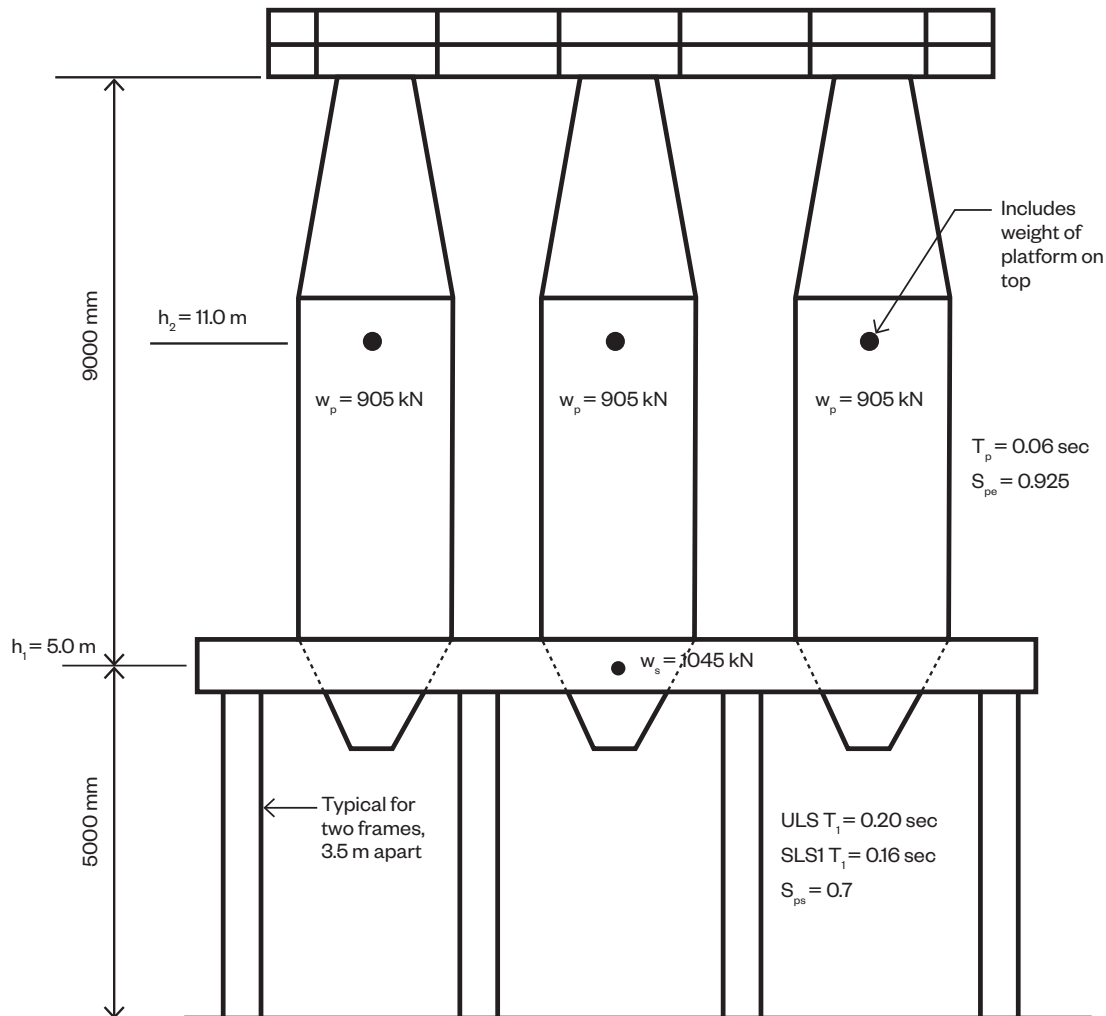
Figure 17: Cumulative Inter-story Peak Deflections



Example E3: Combination structure supporting 3 vertical pressure vessels

This is an adaptation of the example given in ASCE (2010) Appendix 4B. Three heavy pressure vessels are to be supported 5 metres off the ground by reinforced concrete moment resisting frames spanning in both principal directions.

Figure 18: Combination structure supporting three vertical pressure vessels.



ASSUMPTIONS:

- The structure is to be installed on the outskirts of Blenheim on a soft soil site (very deep gravels).
- Design to follow the preferred work practice of the AOPPE (2001), but compare with this Practice Note's recommendations.
- That the structure is located within a plant that is in an area zoned as industrial and the vessels have a design pressure such that if released, toxic fumes are not likely to spread beyond the plant's boundary.
- No site specific seismic hazard study has been undertaken.
- The circular pressure vessel's fundamental horizontal period $T_p < 0.06 \text{ secs}$, therefore Case 1 of Table 11 in this Practice Note has been used.
- The vessels have a mechanical design life of 25 years and a minimum Importance Level of 2.

The minimum seismic coefficient is currently given in AS/NZS 1200 Appendix I and this applies for the pressure envelope, vessel hold down and support, refer to AOPPE (2001) Clause 5.4.4(8). This minimum value is dependent on the NZS 4203:1992 Zone Factor Z .

Table E3: Equivalent static earthquake coefficients calculation for a combination structure

DESCRIPTION	PARAMETER	VALUE	COMMENT
System Information:			
Weight of components:			From Example 4B ASCE (2010)
Weight of support structure	W_s	1045 kN	
Weight of vessels and platform	W_p	2715 kN	
Total	W_t	3760 kN	Seismic weight
Heights to seismic weights:			
	h_1	5 m	Support structure's Centre of Gravity
	h_2	11 m	Vessels' (including platforms) Centre of Gravity
Height to centre of mass for combined system	h	9.332 m	$h = [1045 * 5 + 2715 * 11] / 3760$
Fundamental periods:			
Combined system – ULS	T_1	0.20 sec	From Example 4B ASCE (2010).
Combined system – SLS1	T_1	0.16 sec	From C4.1.2.2 of NZS 1170.5 Supp 1
Vessel – ULS and SLS1	T_p	0.06 sec	Assumed
NZ Building Code Category – Cl. A1 Classified Uses		<i>Ancillary</i>	May be exempt some amenity provisions but required to comply with structural and safety provisions of NZBC
Structural Type	Combination structure		Weight of supported vessels over 70% of total weight, $T_p \leq 0.06$ secs: Case 1 as per Table 11 of this Practice Note
Importance Level:	IL	2	2 limit states to consider; ULS and SLS1
Design Working Lives:			
Pressure vessels:			Plant's life to be 50+ years – from client
AS 1210 Paragraph J2		25 years	Use AS 1210 as a guide and Table 1 of this Practice Note
Support structure and foundation:			
NZ Building Code Clause B2		50 years	Indefinite Life to be specified for Building Consent
Determine whether the system meets the requirements for Equivalent Static Analysis (ESA):			
Vertical Regularity Check:			
Check whether structure has any of the features listed in NZS 1170.5 Cl. 4.5.1 and 4.5.2			Result required for ESA Check
NZS 1170.5 Cl. 4.5.1.1	Weight of vessels	Yes	Weight > 150% of supporting structure

DESCRIPTION	PARAMETER	VALUE	COMMENT
Suitability of Equivalent Static Analysis Method Check:			To be suitable for the Equivalent Static Analysis (ESA) Method, the system must satisfy at least one of the criteria a), b) or c) from Cl. 6.1.3.1 of NZS 1170.5
NZS 1170.5 Cl. 6.1.3.1(a)	$h_n < 10 \text{ m}$	No	May not be able to use ESA
NZS 1170.5 Cl. 6.1.3.1(b)	$T_1 < 0.4 \text{ secs}$	Yes	OK to use ESA
NZS 1170.5 Cl. 6.1.3.1(c)	Horizontal regular	Yes	
	Vertical regular	No	Mass and stiffness distribution between structure and vessels does not comply with NZS 1170.5 cl. 4.5.1.1 and cl. 4.5.1.2
	$T_1 < 2.0 \text{ secs}$	Yes	May not be able to use ESA
			<i>Conclusion: ESA is OK using Cl. 6.1.3.1(b)</i>

Determine the earthquake parameters and site hazard spectra:

Determine Annual Probability of Exceedence (Earthquake Return Periods):

Pressure vessels:

AS/NZS 1170.0 ULS 250 years
Table 3.3

AS/NZS 1170.0 SLS1 25 years
Table 3.3

Support structure:

AS/NZS 1170.0 ULS 500 years
Table 3.3

AS/NZS 1170.0 SLS1 25 years
Table 3.3

Site Subsoil Class:

NZS 1170.5 D NZS 1170.5 Table 3.1 for Spectral Shape Factor $C_h(T)$
Cl. 3.1.3.5

Spectral Shape Factor:

NZS 1170.5 $C_h(T_1)$ 3.0 Site Subsoil Class = D and $T_1 = 0.2 \text{ secs}$ (ULS) and $T_1 = 0.16 \text{ secs}$ (SLS1)
Table 3.1

Hazard Factor:

NZS 1170.5 Z 0.33 Applicable for Blenheim
Cl. 3.1.4

Return Period Factors:

Pressure vessels:

NZS 1170.5 R_u 0.75
Table 3.5

R_{sls1} 0.25

Support structure:

DESCRIPTION	PARAMETER	VALUE	COMMENT
NZS 1170.5 Table 3.5	R_u	1.0	
	R_{slsl1}	0.25	
NZS 1170.5 Cl. 3.1.1	$Z R_u$	0.33	Check that $Z * R_u < 0.7$. ∴ OK
Near-fault factor:			
NZS 1170.5 Cl. 3.1.6.2	$N(T, D)$	$N_{max}(T)$	Adjacent to major fault (Wairau), i.e. $D < 2$ km Period not long enough to be influenced by near-fault effects
NZS 1170.5 Table 3.7	$N_{max}(0.2)$	1.0	

Horizontal Elastic Site Hazard Values:

Support structure design:

NZS 1170.5 Eqn 3.1(1)			$C(T_1) = C_h(T) R Z N(T, D)$
ULS	$C(T_1)_{uls}$	0.990	$C(T_1)_{uls} = 3.0 * 1.0 * 0.33 * 1.0$
SLS1	$C(T_1)_{slsl1}$	0.248	$C(T_1)_{slsl1} = 3.0 * 0.25 * 0.33 * 1.0$

Determine the design earthquake actions:

Minimum earthquake requirements:

Use this Practise Note the AOPPE (2001) as the preferred work practice

AOPPE (2001) Clause 3.4.2

Use NZS 1200 for wind, snow and seismic

NZS 4203 Cl. 4.6.2.5 Zone Factor	Z	1.2	Value for the Blenheim area
--	-----	-----	-----------------------------

NZS 1200 Appendix I	$0.8 * F_{ph} / W_p$	0.60	Minimum seismic coefficient for pressure equipment
	$C_d(T_1)$	0.60	Minimum seismic coefficient for support structure's design

This Practice Note's recommendation:

Hazard Factor	Z	0.33	From above for Blenheim
Table H1	$0.8 * F_{ph} / W_p$	0.46	$0.8 * F_{ph} / W_p = 0.40 + 0.10 * 3/5$
	$C_d(T_1)$	0.58	$C_d(T_1) = [0.40 + 0.10 * 3/5] / 0.8$ Interpolated values from Table H1

Structural Ductility Factors:

Pressure vessels:

NZS 3404 Table 12.2.4 - ULS	μ_p	1.25	Nominally ductile pressure vessels
NZS 1170.5 Cl. 4.3.2 - SLS1	μ_{slsl1}	1.0	

Support structure:

DESCRIPTION	PARAMETER	VALUE	COMMENT
NZS 3101 Table 2.5 - ULS	μ_s	6.0	Ductile moment resisting frame = reinforced concrete support structure
NZS 3101 Cl. 2.6.2.3.1- SLS1	μ_{sls1}	1.0	
Combination structure:			
Design ULS	μ_{uls}	6.0	Can use $\mu_{uls} > 3.0$ as supported vessels are rigid as per FEMA 450 Cl. 14.1.5
Design SLS1	μ_{sls1}	1.0	
Structural Performance Factors:			
Pressure vessels:			
ULS	S_{pe}	0.925	Nominally ductile pressure vessel designed using WSD mechanical codes. Refer Table 7 this Practice Note
SLS1	S_{pe}	0.7	NZS 1170.5 Cl. 4.4.4
Support structure:			
Support Structure - ULS	S_{ps}	0.7	Ductile support structure with 8 columns. Refer Table 7 this Practice Note and NZS 3101 Cl. 2.6.2.2.1
SLS1	S_{ps}	0.7	NZS 1170.5 Cl. 4.4.4
Combination structure:			
Design ULS	S_p	0.7	$S_p = S_{ps}$ Take lower value as the supported vessels are rigid and the response is largely from the ductile frame which has 8 columns i.e. alternative load paths are available
Design SLS1	S_p	0.7	
Inelastic Spectrum Scaling Factors:			
NZS 1170.5 Cl. 5.2.1.1 - ULS	$k_{\mu, uls}$	3.857	$(6 - 1) * 0.4/0.7 + 1$ - with T_1 taken as = 0.4 secs From Table 9 of this Practice Note
NZS 1170.5 Cl. 5.2.1.2 - SLS1	$k_{\mu, sls1}$	1.0	
Unscaled 5% Damped Horizontal Design Action Coefficients for support structure:			
NZS 1170.5 Eqn 5.2(1)			$C_d(T_1)_{uls} = C(T_1) * S_p / k_{\mu}$
ULS	$C_d(T_1)_{uls}$	0.180	$C_d(T_1)_{uls} = 0.99 * 0.7 / 3.857$
NZS 1170.5 Eqn 5.2(2)	$(Z/20 + 0.02)R_u$	0.037	$= (0.33/20 + 0.02) * 1.0$
			$\therefore C_d(T_1)_{uls} = 0.18$
SLS1	$C_d(T_1)_{sls1}$	0.17	$C_d(T_1)_{sls1} = 0.248 * 0.7 / 1.0$
Review P-Delta effects:			

DESCRIPTION	PARAMETER	VALUE	COMMENT
NZS 1170.5 Cl. 6.5.2(a)	$T_1 > 0.4$ secs	No	
NZS 1170.5 Cl. 6.5.2(b)	$h_n > 15$ m	No	
	$T_1 > 0.6$ secs	No	No need to consider P-Delta effects $\therefore k_1 = 1.0$
Material Code Scaling Factor:			
ULS	k_2	1.0	No concrete standard requirement to scale ULS earthquake
\therefore	K	1.0	$K = k_1 * k_2$
System Damping Factors:			Chosen values:
ULS	$C_f(5)$	1.0	Damping = 5% of critical – Table 5 this Practice Note
SLS1	$C_f(1)$	1.53	Damping = 1% of critical – Table 5 this Practice Note

Horizontal Design EQ Forces for Support Structure:

Modify the Horizontal Design Action
Coefficient Using the Additional Factors

$$C_d(T_1)_{\text{modif}} = C_d(T_1) * C_f(\xi) * K$$

$$\begin{aligned} C_d(T_1)_{\text{modif}} &= 0.18 &= 0.180 * 1.0 * 1.0 \\ C_d(T_1)_{\text{slsl,modif}} &= 0.27 &= 0.170 * 1.53 \end{aligned}$$

ULS – To avoid
collapse with
 $\mu = 6$

$$E_u = V_{\text{uls}} \quad 0.18 W_t \quad = C_d(T_1)_{\text{uls,modif}} * W_t$$

Value too low - AS/NZS 1200 Appendix I minimum value = 0.60

This Practice Note minimum value = 0.58

No benefit in using a ductile or a limited ductile structure.
Either:

- Commission a site specific seismic hazard study and use a ductile structure with $\mu = 6$
- Undertake a special study using NITHA methods
- Use a lower structural ductility factor μ_{uls} and accept higher seismic loads

ULS – with $\mu_{\text{uls}} = 3$ as above with
5% damping

Check to see the effect of reducing the structural ductility
factor μ from 6 to 3

Inelastic Spectrum Scaling Factors:

NZS 1170.5 Cl. 5.2.1.1 - ULS	$k\mu_{\text{uls}}$	2.143	$(3 - 1) * 0.4/0.7 + 1$ - with T_1 taken as = 0.4 secs From Table 9 of this Practice Note.
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NZS 1170.5 Cl. 5.2.1.2 - SLS1	$k\mu_{\text{slsl}}$	1.0	
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Elastic Site Hazard Values:

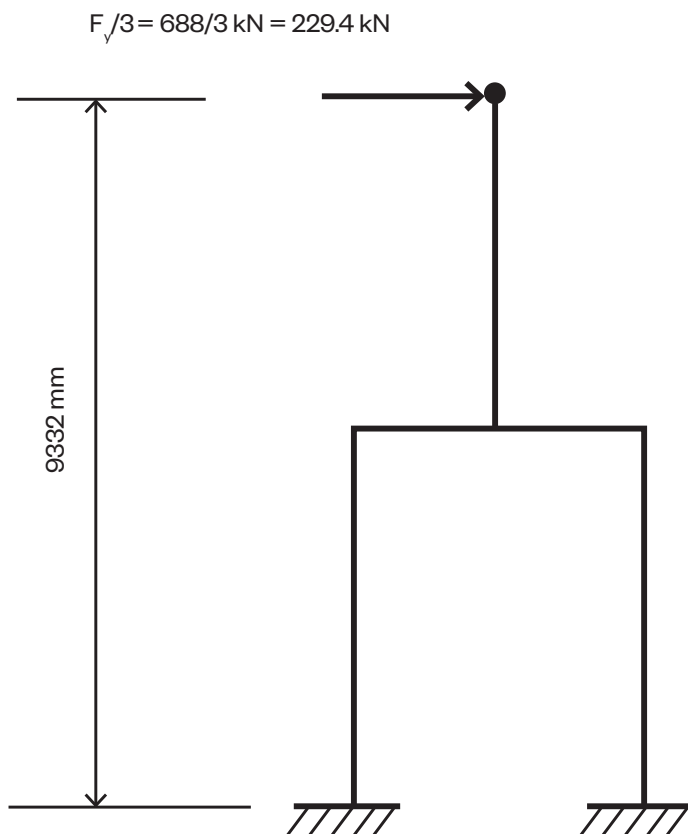
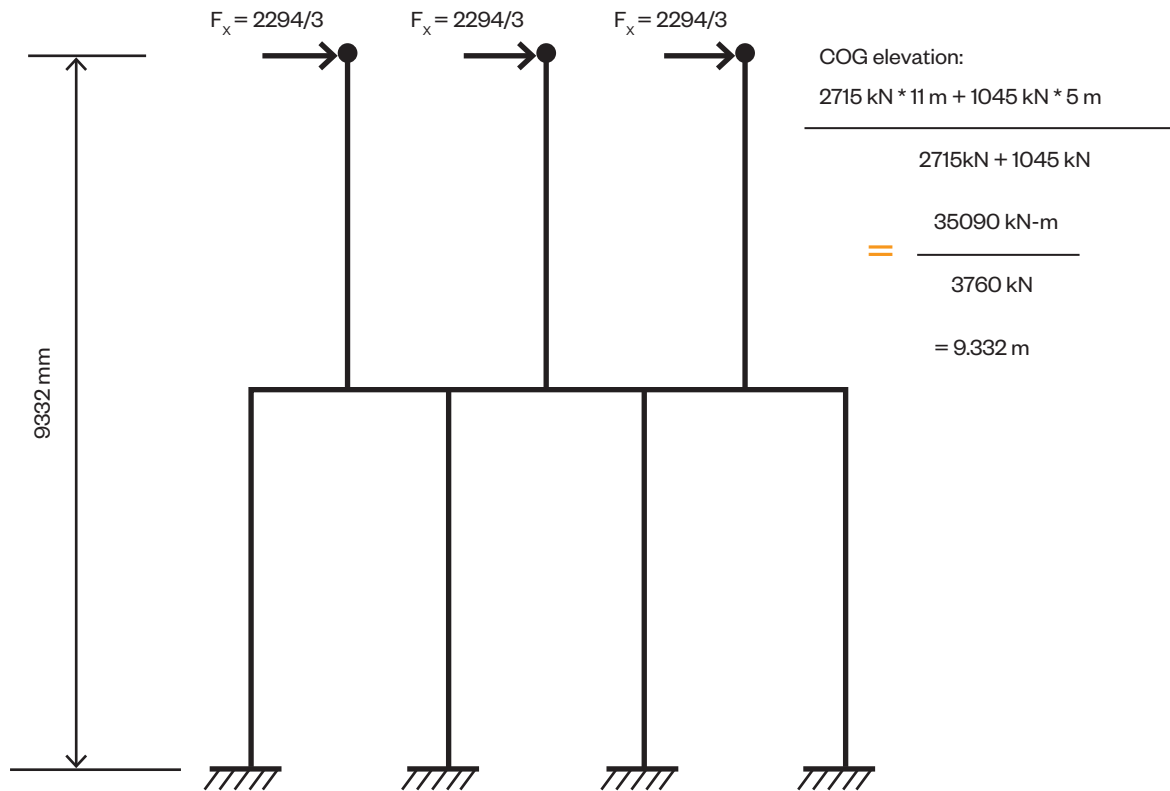
ULS	$C(T_1)_{\text{uls}}$	0.990	From above
SLS1	$C(T_1)_{\text{slsl}}$	0.248	

DESCRIPTION	PARAMETER	VALUE	COMMENT
Unscaled 5% Damped Horizontal Design Action Coefficients:			
ULS	$C_d(T_1)_{uls}$	0.32	$C_d(T_1)_{uls} = 0.99 * 0.7 / 2.143$
SLS1	$C_d(T_1)_{slsl}$	0.17	<p>Would be applied in two horizontal directions separately in accordance with NZS 1170.5 Cl 5.3.1.1 and capacity design as per NZS 1170.5 Cl 5.6.3 required as a ductile structure.</p> <p>However, AS/NZS 1200 Appendix I min. value = 0.60</p> <p>This Practice Note minimum value = 0.58</p>
<p>ULS – with $\mu_{uls} = 2$ as above with 5% damping</p> <p>Inelastic Spectrum Scaling Factors:</p>			
NZS 1170.5 Cl. 5.2.1.1 - ULS	$k_{\mu_{uls}}$	1.571	$(2 - 1) * 0.4/0.7 + 1$ - with T_1 taken as = 0.4 secs. From Table 9 of this Practice Note
NZS 1170.5 Cl. 5.2.1.2 - SLS1	$k_{\mu_{slsl}}$	1.0	
Elastic Site Hazard Values:			
ULS	$C(T_1)_{uls}$	0.990	From above
SLS1	$C(T_1)_{slsl}$	0.248	
Unscaled 5% Damped Horizontal Design Action Coefficients:			
ULS	$C_d(T_1)_{uls}$	0.44	$C_d(T_1)_{uls} = 0.99 * 0.7 / 1.571$
SLS1	$C_d(T_1)_{slsl}$	0.17	<p>Would be applied in two horizontal directions separately in accordance with NZS 1170.5 Cl 5.3.1.1 and capacity design as per NZS 1170.5 Cl 5.6.3 required as a limited ductility structure that is not vertically regular as per NZS 1170.5 Cl 4.5</p> <p>AS/NZS 1200 Appendix I min. value = 0.60</p> <p>This Practice Note minimum value = 0.58</p>
∴ For support structure ULS	$E_u = V_{uls}$	$0.58 W_t$	Apply in two horizontal directions separately in accordance with NZS 1170.5 Cl 5.3.1.1
SLS1 – To sustain no damage	$E_{slsl} = V_{slsl}$	$0.27 W_t$	$= C_d(T_1)_{slsl, modif} * W_t$
<p>ULS – with $\mu_{uls} = 1.25$ as above with 5% damping</p> <p>Inelastic Spectrum Scaling Factors:</p>			
NZS 1170.5 Cl. 5.2.1.1 - ULS	$k\mu_{uls}$	1.143	$(1.25 - 1) * 0.4/0.7 + 1$ - with T_1 taken as = 0.4 secs From Table 9 of this Practice Note
NZS 1170.5 Cl. 5.2.1.2 - SLS1	$k\mu_{slsl}$	1.0	

DESCRIPTION	PARAMETER	VALUE	COMMENT
Elastic Site Hazard Values:			
ULS	$C(T)_{1\text{uls}}$	0.990	From above
SLS1	$C(T)_{1\text{sls1}}$	0.248	
Unscaled 5% Damped Horizontal Design Action Coefficients:			
ULS	$C_d(T)_{1\text{uls}}$	0.61	$C_d(T)_{1\text{uls}} = 0.99 * 0.7 / 1.143$
SLS1	$C_d(T)_{1\text{sls1}}$	0.17	<p>Apply in two horizontal directions concurrently – V_{uls} and $0.3 * V_{\text{uls}}$ in accordance with NZS 1170.5 Cl 5.3.1.2 and capacity design not required as per NZS 1170.5 Cl 5.6 as a nominally ductile structure</p> <p>OK – AS/NZS 1200 Appendix I minimum value = 0.60</p> <p>This Practice Note minimum value = 0.58</p>
∴ For support structure ULS	$E_u = V_{\text{uls}}$	$0.61 W_t$	Analyse further assuming $\mu_{\text{uls}} = 1.25$
SLS1 – To sustain no damage	$E_{\text{sls1}} = V_{\text{sls1}}$	$0.27 W_t$	$= C_d(T)_{1\text{sls1,modif}} * W_t$
Horizontal Design EQ Forces:			
Horizontal shear acting at base	V_u	2294 kN	$= 0.61 * 3760 \text{ kN}$
Equivalent static horizontal forces	F_x	2294 kN	Acting at 9.332 m above base.
	$F_y = 0.3 * F_x$	688 kN	Acting at 9.332 m above base at right angles to F_x .

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Figure 19: WSD forces on the vessel in the transverse direction.



Since the table top structure has a rigid diaphragm, the lateral force is distributed to each bent per its relative stiffness. As the stiffness of each bent is the same, the lateral force to each bent is identical.

Table E3: Equivalent static earthquake coefficients calculation for a combination structure

DESCRIPTION	PARAMETER	VALUE	COMMENTS
Horizontal Elastic Level EQ Forces on foundation:			
			Foundation to be designed for the minimum of the over-strength of collapse mechanism or the elastic level earthquake
Spectral Shape Factor ULS	$C_h(T_1)$	3.0	From above
NZS 3101 Table 2.5	μ	1.0	For elastic design
NZS 1170.5 Cl. 4.4.2	S_p	1.0	Can be used for stability check also
NZS 1170.5 Cl. 5.2.1.1	k_μ	1.0	
	$C_f(5)$	1.0	Damping = 5% of critical
	$C(T_1)$	0.99	$C(T_1) = 3.0 * 1.0 * 0.33 * 1.0$
Unscaled ULS Design Action	$C_d(T_1)^{0.5}_{uls}$	0.99	$= C(T_1) * S_p / k_\mu * K * W_t = 0.990 * 1.0 / 1.0 * W_t$
ULS – To avoid premature non-ductile failure	$E_{max} = V_{uls}^0$	$0.99 W_t$	
Equivalent static horizontal force	F_1	3722 kN	Acting at 9.332 m above base

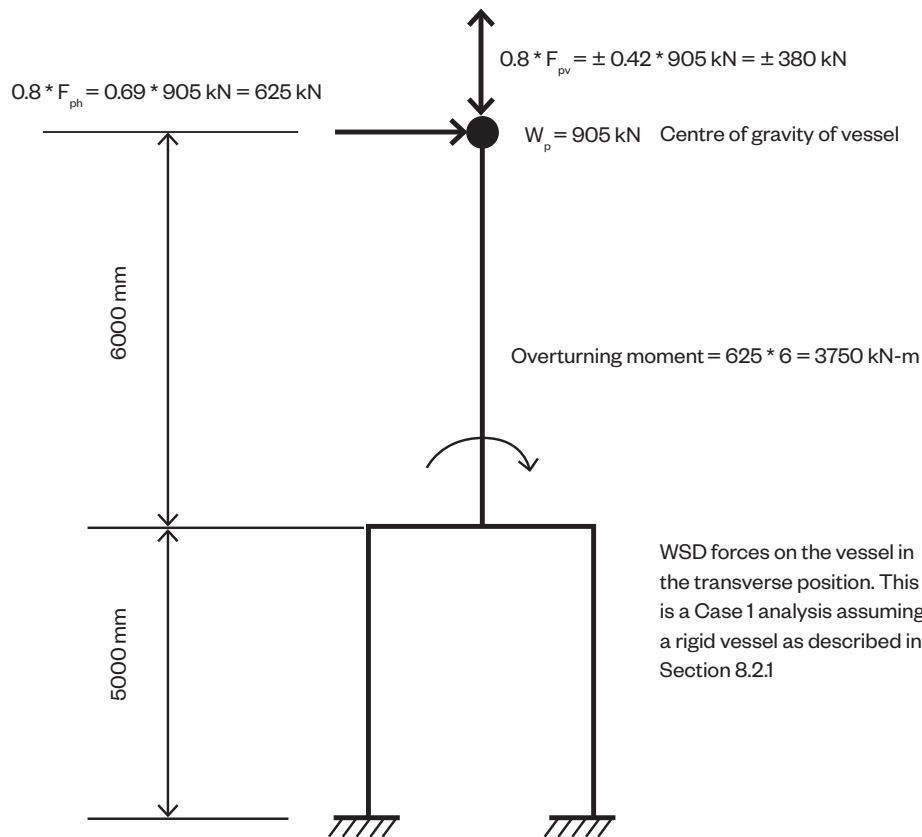
Determine forces on vessels for vessel's stress analysis:

Level of allowable deformity		Low	Rigid heavy walled vessels with some ductility $\therefore \mu_p = 1.25$
Overall Height:	h_n	5.0 metres	
Horizontal Fundamental periods:			
Support including vessels	T_1	0.20 secs	From above for ULS
	T_1	0.16 secs	From above for SLS1
Vessel	T_p	0.06 secs	From mechanical engineer
Vertical Fundamental periods:			
Support including vessels	T_{v1}	0.10 secs	Assumed for structure including vessels
Spectral Shape Factors:			
NZS 1170.5 Table 3.1	$C_h(0)$	1.12	Site Subsoil Class = D and $T = 0.0$ secs. Use value in brackets as per Note 1 to NZS 1170.5 Table 3.1
	$C_h(T_v)$	3.0	Site Subsoil Class = D and $T_v = 0.1$ secs

DESCRIPTION	PARAMETER	VALUE	COMMENTS
Displacement Ductility Factors:			NZS 1170.5 Cl. 8.5
ULS	μ_p	1.25	NZS 1170.5 Supp. 1 Table C8.2
Risk Factor for Parts:			NZS 1170.5 Table 8.1
ULS	R_p	1.0	
Parts Response Factors:			NZS 1170.5 Table 8.2 and Cl. 8.6
ULS	C_{ph}	0.85	For horizontal forces
ULS	C_{pv}	1.0	For vertical forces
System Damping Factors:			
ULS	$C_f(5)$	1.0	Chosen values: Damping = 5% of critical
Part Category:			
NZS 1170.5 Table 8.1	ULS	P.1	High pressure system representing danger to those working within plant. $R_{pu} = 1.0$
Floor Spectral Shape Factor:			
NZS 1170.5 Cl. 8.4	$C_i(T_p)$	2.0	$T_p < 0.75$ secs
Floor Height Coefficients:			
Support	C_{Hi}	1.83	NZS 1170.5 Eqns 8.3(1), 8.3(2) and 8.3(3) As $h_n < 12$ m, Eqn 8.3(1) applies and 8.3(3) not applicable. $C_{Hi} = 1 + 5/6 = 1.83$
Horizontal Forces on Part:			
5% Damped Elastic spectrum @ $T = 0$ sec:			250 year return period earthquake
NZS 1170.5 Eqn 3.1(1)			$C(0) = C_h(0) R Z N(0,D)$
ULS	$C(0)$	0.277	$C(0) = 1.12 * 0.75 * 0.33 * 1.0$
Spectral Ordinate of the Part:			
NZS 1170.5 Eqn 8.2(1)			$C_p(T_p) = C(0) * C_{Hi} * C_i(T_p)$
ULS	$C_p(T_p)$	1.015	$C_p(T_p) = 0.277 * 1.83 * 2.0$
Horizontal Design EQ Actions:			
NZS 117.5 Eqn 8.5(1)			$F_{ph} = C_p(T_p) * C_{ph} * R_p * C_f(\xi) * W_p$
ULS - No collapse	$E_u = F_{ph}$	$0.86 W_p$	$F_{ph} = 1.02 * 0.85 * 1.0 * 1.0 * W_p \leq 3.6 W_p$
WSD – No loss of contents	$E_{wsd} = 0.8 E_u$	$0.69 W_p$	$= 0.8 * 0.862$
			AS/NZS 1200 Appendix I min. value = 0.60
			This Practice Note's Table H1 minimum value = 0.46
\therefore	E_{wsd}	$0.69 W_p$	

DESCRIPTION	PARAMETER	VALUE	COMMENTS
WSD			<p>Vessels to be analysed for:</p> <p>WSD case of 0.69g in any direction as vessels are circular. Internal forces and moments = M_{iu}</p> <p>Check against typical coefficient given in Table H2 for an item at height for a 250 year return period EQ and $Z = 0.33$:</p> <p>$0.8E_u = 0.96 + 0.16 * 3/5 = 1.06$</p> <p>Design coefficient = $0.69 < 1.06$.∴ Check if OK</p> <p>Soil class is D, $C_h(T) = 1.12$ vs. 1.33 for class C</p> <p>Height = 5m vs. 6m for typical example C_{hi} less</p> <p>Part ductility 1.25 vs. 1.0 C_{ph} is less. ∴ OK</p>
Horizontal Design Actions due to EQ deflections:			
ULS deflections at support			Vessels supported at one level. No displacement induced seismic actions.
Vessel Design Horizontal Actions due to EQ:			
WSD:			
Any direction	M_{hu}	3747 kN-m	$= 0.69 * 905 * 6$ – bending moment at vessel base
Vertical Forces on Part:			
5% Damped Elastic spectrum @ $T = T_{v1}$ sec:			
NZS 1170.5 Eqns 3.1(1) and 3.2(1)			$C_v(T_{v1}) = 0.7 C_h(T_v) R Z N(T_v, D)$
ULS	$C_v(T_v)$	0.520	$C_v(T_v) = 0.7 * 3.0 * 0.75 * 0.33 * 1.0$
ULS	C_{vd}	0.520	NZS 1170.5 Eqn 5.4(2)
Vertical Design EQ Forces:			
NZS 1170.5 Eqn 8.5(2)			$F_{pv} = C_{vd} * C_{pv} * R_p * W_p$
ULS - No collapse	$E_u = F_{pv}$	$0.52 W_p$	$F_{pv} = 0.520 * 1.0 * 1.0 * W_p \leq 2.5 W_p$
WSD – No loss of contents	$E_{wsl} = 0.8E_u$	$0.42 W_p$	$= 0.8 * 0.520$
			WSD case of 0.42g in vertical direction
			Internal actions = M_{uv}
Vessel's Total Design Actions due to EQ			
WSD:			
Any direction	M_{xu}		<p>$M_{hu} + M_{uv}$</p> <p>These earthquake actions need to be combined with normal operating pressure and temperature conditions for the vessel's stress checking.</p>

Figure 20: WSD forces on the vessel in the transverse direction



Example E4: Pump mounted at ground level

A water pump with a mass of 1200 kg is located at grade on a concrete slab foundation within a plant that is far from occupied dwellings and has a mechanical design life of 25 years. The 150NB discharge piping has the design conditions of 9 bar at 160° C. The plant is located south of Hastings on the Heretaunga Plains. The design is to follow this Practice Note as the **temperature** is above the Gazetted PECPR exemption for water systems (*NZ Gazette No. 141*, August 2005). As a pressure equipment item, the pump design should also comply with Part 8 of the ACPPE (2001).

Assume no site specific seismic hazard study has been undertaken. Hence the minimum seismic coefficient is given in Table H1 which applies for HD bolts and foundation in a similar fashion to pressure vessels, refer to ACPPE (2001) Clause 5.4.4(8).

This minimum value is dependent on the NZS 1170.5 Hazard Zone Factor Z.

Figure 21: Pump mounted at ground level.

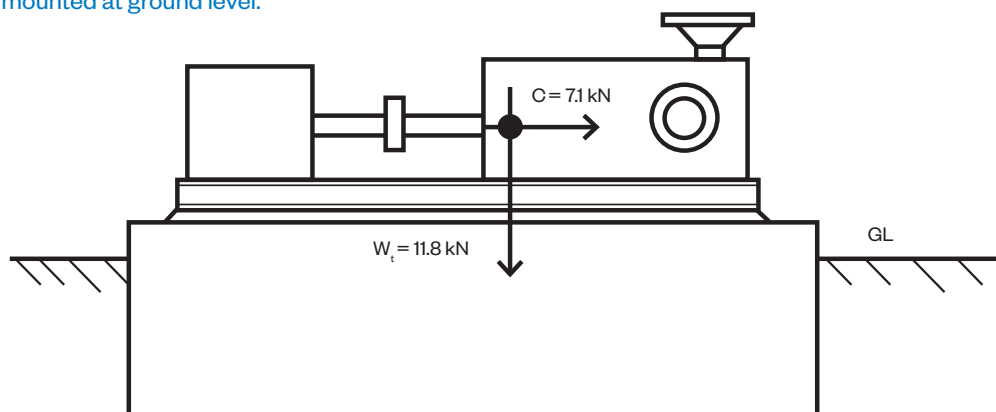


Table E4: Equivalent static earthquake coefficients calculation for a rigid pump at grade

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
System Information:			
Weight of components:			
Weight of pump	W_t	11.8 kN	From pump vendor data
NZ Building Code Category – Clause A1 Classified Use		<i>Ancillary</i>	May be exempt some amenity provisions but required to comply with structural and safety provisions of NZBC
Structural Type:		Nonbuilding structure not similar to buildings	
Design Working Lives:			Plant design life of 25 years – from client
Pressure equipment:			
AS 1210 Paragraph J2		25 years	Table 1 of this Practice Note
Foundation:			
NZ Building Code Clause B2		25 years	Specified Life included in Building Consent application
Hazard level of discharge pipe as per AS 4343:			
Design pressure	p	0.9 MPa	9 barg
Nominal pipe diameter	D	150 mm	From above
	pD	135 MPa-mm	
Water	Fluid Type	HG	AS 4343 Table 2
Hazard Level		D	AS 4343 Table 1 Item 5.3 Pressure Piping with $pD = 135$ and Fluid Type = HG Assume no modifiers to pD value as per the notes to Table 1
Importance Level:	IL	2	From Table 2 of this Practice Note. IL = 1 gives too low a earthquake return period for client 2 limit states = ULS and SLS1
Fundamental period:			
Pump	T_1	0.02 secs	Effectively rigid. Applicable for all limit states.

Determine whether the system meets the requirements for the Equivalent Static Analysis method:

Suitability of Equivalent Static Analysis

Method Check:

NZS 1170.5 Clause 6.1.3.1(a)	$h_n < 10$ m	Yes	Able to use ESA
NZS 1170.5 Clause 6.1.3.1(b)	$T_1 < 0.4$ secs	Yes	Able to use ESA
NZS 1170.5 Clause 6.1.3.1(c)	Horizontal regular	Yes	
	Vertical regular	Yes	
	$T_1 < 2.0$ secs	Yes	Able to use ESA
			Equivalent Static Analysis O.K.

Determine the earthquake parameters and site hazard spectra:

Earthquake Return Periods:

AS/NZS 1170.0 Table 3.3	ULS	250 years	$E_{wsl} = 0.8E_u$ and possible overstress allowance
	SLS1	25 years	$E_{wsl} = E_{sls1}$ and no overstress allowance

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Site Subsoil Class:			
NZS 1170.5 Clause 3.1.3.4		D	Located on deep gravels. NZS 1170.5 Table 3.1 for Spectral Shape Factor $C_h(T)$
Spectral Shape Factor:			
NZS 1170.5 Table 3.1	$C_h(T_1)$	3.00	Site Subsoil Class = D & $T_1 = 0.02$ secs
Hazard Factor:			
NZS 1170.5 Clause 3.1.4	Z	0.39	Applicable for Hastings
Return Period Factors:			
NZS 1170.5 Table 3.5	R_u	0.75	
	R_{sls1}	0.25	
NZS 1170.5 Clause 3.1.1	$Z R_u$	0.29	< 0.7 ∴ O.K.
Near-fault factor:			
NZS 1170.5 Clause 3.1.6	$N(T, D)$	1.0	Not adjacent to any major faults, i.e. $D > 20$ km
Elastic Site Hazard Values:			
NZS 1170.5 Eqn 3.1(1)			$C(T_1) = C_h(T) R Z N(T, D)$
ULS	$C(T_1)_{uls}$	0.878	$C(T_1)_{uls} = 3.0 * 0.75 * 0.39 * 1.0$
SLS1	$C(T_1)_{sls1}$	0.293	$C(T_1)_{sls1} = 3.0 * 0.25 * 0.39 * 1.0$

Determine the design earthquake actions for the pressure equipment:

Minimum earthquake requirements:			Use this Practice Note the AOPPE (2001) as the preferred work practice
AOPPE (2001) Clause 3.4.2			Use NZS 1200 for wind, snow and seismic
NZS 4203 Clause 4.6.2.5	Z	1.2	NZS 1200 Appendix I minimum seismic coefficient for pressure equipment = 0.60
Zone Factor			
NZS 1200 Appendix I	$0.8 \times \Theta_d(T_1)$	0.60	Minimum seismic coefficient for pump. Detailed seismic analysis of pumps and their components is not normally undertaken
	$\Theta_d(T_1)$	0.60	Minimum seismic coefficient for HD bolts and foundation design
This Practice Note's recommendation:			
Earthquake Return Period		250 years	From above for the vessel's seismic design
Hazard Factor	Z	0.39	From above for Hastings
Table H1	$0.8 * C_d(T_1)$	0.54	$0.8 * C_d(T_1) = 0.50 + 0.05 * 4/5$. Interpolated value from Table H1 for 250-year return period earthquake
	$C_d(T_1)$	0.68	$C_d(T_1) = 0.54 / 0.8$. For the ULS design of the foundation and HD bolts

Determine the design earthquake actions:

Structural Ductility Factors:			
ULS	μ_{uls}	1.25	Nominally ductile structure
NZS 1170.5 Clause 4.3.2 - SLS1	μ_{sls1}	1.0	
Structural Performance Factors:			NZS 1170.5 Clause 4.4.2
ULS	S_p	0.925	With $\mu = 1.25$
SLS1	S_p	0.7	Allow code value for SLS1 case

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Inelastic Spectrum Scaling Factors:			NZS 1170.5 Clause 5.2.1.1
ULS	$k_{\mu, \text{uls}}$	1.14	As $T_1 < 0.4$ sec
SLS1	$k_{\mu, \text{sls1}}$	1.0	
Unscaled 5% Damped Design Actions:			Eqn 5.2(1)
ULS	$C_d(T_1)_{\text{uls}}$	0.71	$C_d(T_1)_{\text{uls}} = C(T_1) * S_p / k_{\mu}$
Eqn 5.2(2)	$(Z/20 + 0.02)R_u$	0.03	$C_d(T_1)_{\text{uls}} = 0.878 * 0.925 / 1.14$
SLS1	$C_d(T_1)_{\text{slsl}}$	0.21	$\therefore C_d(T_1)_{\text{uls}} = 0.71$
System Damping Factors:			$C_d(T_1)_{\text{slsl}} = 0.293 * 0.7 / 1.0$
			Chosen values:
ULS	$C_f(2)$	1.0	Damping = 2% of critical
SLS1	$C_f(0.5)$	1.0	Damping = 0.5% of critical
			Effectively rigid so damping has no effect on response
Horizontal Design EQ Forces:			
ULS – To avoid collapse	$E_u = V_{\text{uls}}$	$0.71 W_t$	$= C_d(T_1)_{\text{uls}} * W_t$
WSD – No loss of contents	$E_{\text{wscd}} = 0.8E_u$	$0.57 W_t$	$= 0.8 * 0.71 W_t$
			Too low, AS/NZS 1200 Appendix I minimum value = 0.60
			This Practice Note recommends a minimum value of 0.54
\therefore For pressure envelope	E_{wscd}	0.60	
\therefore For foundation and HD Bolts	E_u	0.71	Minimum value to satisfy NZS 1170.5
			This Practice Note recommends a minimum value of 0.68
SLS1 – To sustain no damage	$E_{\text{slsl}} = V_{\text{slsl}}$	$0.21 W_t$	$= C_d(T_1)_{\text{slsl}} * W_t$
			Foundation and HD bolts to be designed for the minimum of the over-strength of collapse mechanism or the elastic level earthquake
Horizontal Elastic Level EQ Forces on foundation and HD bolts:			
Spectral Shape Factor:			By inspection the elastic level of earthquake lower than the over-strength collapse mechanism (assumed to be failure of HD bolts)
NZS 1170.5 Table 3.1	$C_h(T_1)$	3.00	Site Subsoil Class = D & $T_1 = 0.02$ secs
NZS 3404 Clause 12.9.1.2.2 (4) (b)	μ	1.0	
NZS 1170.5 Clause 4.4.1	S_p	1.0	Can also be used for overall structure's stability check
NZS 1170.5 Clause 5.2.1.1	k_{μ}	1.0	
			$C_f(2)$
Elastic Site Hazard Value	$C(T_1)$	0.878	$C(T_1) = 3.0 * 0.75 * 0.39 * 1.0$
Unscaled ULS Design Action	$C_d(T_1)_{\text{uls}}$	0.88	$C_d(T_1) = 0.878 * 1.0 / 1.0$

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
ULS – To avoid premature non-ductile failure & overturning or sliding failure	$E_{max} = V_{uls}^{\circ}$	$0.88 W_t$	$= C(T_1) * S_p * / k_{\mu} * W_t$
∴ For foundation and HD Bolts	$E_u = E_{max}$	0.88	$C = 0.88 * 11.8 = 10.4 \text{ kN. O.K. as}$ AS/NZS 1200 Appendix I minimum value = 0.60 this Practice Note recommends a minimum value of 0.56

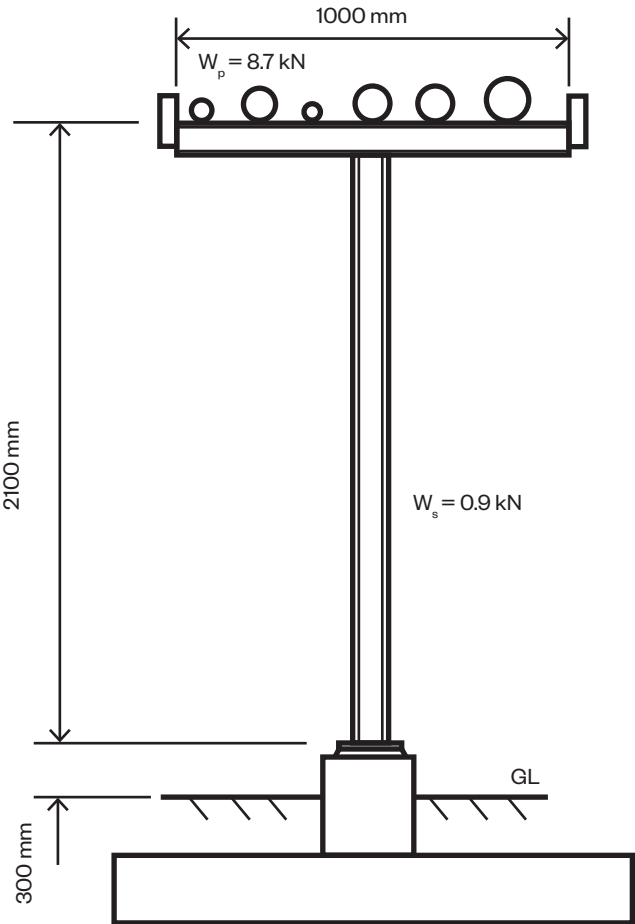
Example E5: Vertical Cantilever Pipe Support

Assume a welded steel cantilever T post supported on a concrete foundation. A 150 NB steel water filled pipe operating at pressures greater than 50 kPa and temperatures greater than 65 °C in the same complex as the pump in Example E4. The pipes are to be supported 2.4 m off the ground at 6 m intervals by individual pipe supports that also support other piping on a 1 m wide support. The water line is not part of fire suppression system or for vital potable water supply or are any other of the items supported on the support but that the client design specification calls for piping to be designed for both horizontal and vertical earthquake design actions. The design is to follow ~~the preferred work practice of the ACPPE (2001)~~ this Practise Note.

From Note 5 of Appendix B of NZS 3404, the maximum ductility factor for a short period Category 2 vertical cantilever = 5 but limit the maximum μ_{uls} = 3 as per Item 1.7 of NZS 3404 Appendix B.

As the 150 NB piping is classed as pressure piping in the DoL ACPPE (2001) and is not exempt as per the 2005 Gazette notice, the design of piping, supports and foundations is governed by this Practise Note and Sections 6 of the ACPPE (2001).

Figure 22: vertical cantilever



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Table E5: Equivalent static earthquake coefficients calculation for a vertical cantilever pipe support and supported piping

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
System Information:			
Weight of components:			
Weight of steel support	W_s	0.9 kN	Assume 150UC30
Weight of piping	W_p	8.7 kN	Assume piping area load = 1.45 kPa
Seismic weight	W_t	9.2 kN	Eqn 4.2(1) . ∴ $W_t = 8.7 + 0.9/2$
Fundamental periods:			
Horizontal	T_1	0.25 sec	Applicable for ULS
	T_2	0.20	Applicable for SLS1
Vertical	T_v	0.01	Applicable for all limit states
NZ Building Code Category – Clause A1 Classified Uses		<i>Ancillary</i>	May be exempt some amenity provisions but required to comply with structural and safety provisions of NZBC
Structural Type:		Nonbuilding structure similar to buildings	
Design Working Lives:		Plant design life of 25 years – from client	
Pressure equipment:			
AS 1210 Paragraph J2		25 years	Table 1 of this Practice Note
Foundation:			
NZ Building Code Clause B2		25 years	Specified Life included in Building Consent application
Importance Level:	IL	2	2 limit states = ULS and SLS1
Determine whether the system meets the requirements for the Equivalent Static Analysis method:			
Vertical Regularity Check:			
NZS 1170.5 Clause 4.5.1.2		N/A	Single storey structure
Suitability of Equivalent Static Analysis Method Check:			
NZS 1170.5 Clause 6.1.3.1(a)	$h_n < 10$ m	Yes	Able to use ESA
	$T_1 < 0.4$ sec	Yes	Able to use ESA
NZS 1170.5 Clause 6.1.3.1(b)	Horizontal regular	Yes	
	Vertical regular	Yes	
NZS 1170.5 Clause 6.1.3.1(c)	$T_1 < 2.0$ sec	Yes	Able to use ESA
			<u>Equivalent Static Analysis O.K.</u>
Determine the earthquake parameters and site hazard spectra:			
Earthquake Return Periods:			
Supports and piping:			
Table 3.3 AS/NZS 1170.0	ULS	250 years	$E_{wcd} = 0.8E_u$ and overstress allowance as per design code for piping
	SLS1	25 years	$E_{wcd} = E_{sls1}$ and no overstress allowance
Site Subsoil Class:			

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
NZS 1170.5 Clause 3.1.3.5 Spectral Shape Factor:		D	NZS 1170.5 Table 3.1 for Spectral Shape Factor $C_h(T)$
NZS 1170.5 Table 3.1	$C_h(T_1)$	3.0	Site Subsoil Class = D & $T_1 = 0.25$ secs
	$C_h(T_2)$	3.0	Site Subsoil Class = D & $T_2 = 0.20$ secs
Hazard Factor:			
NZS 1170.5 Clause 3.1.4	Z	0.39	
Return Period Factors:			
NZS 1170.5 Table 3.5	R_u	0.75	
	R_{sls1}	0.25	
NZS 1170.5 Clause 3.1.1	$Z R_u$	0.29	< 0.7 ∴ O.K.
Near-fault factor:			
NZS 1170.5 Clause 3.1.6	$N(T, D)$	1.0	Not adjacent to any major faults, i.e. $D > 20$ km
Elastic Site Hazard Values:			
NZS 1170.5 Eqn 3.1(1)			$C(T_1) = C_h(T) R Z N(T, D)$
ULS	$C(T_1)_{uls}$	0.878	$C(T_1)_{uls} = 3.0 * 0.75 * 0.39 * 1.0$
SLS1	$C(T_2)_{sls1}$	0.293	$C(T_2)_{sls1} = 3.0 * 0.25 * 0.39 * 1.0$
Determine the design earthquake actions:			
Minimum earthquake requirements:			Use this Practise Note as the preferred work practice
AOPPE (2001)- Clause 3.4.2			Use NZS 1200 for wind, snow and seismic
NZS 4203- Clause 4.6.2.5-	Z	1.2	NZS 1200 Appendix I minimum seismic coefficient for pressure equipment = 0.60
Zone Factor			
NZS 1200 Appendix I	$0.8 * F_{ph} / W_p$	0.60	Minimum seismic coefficient for piping
	$C_d(T_1)$	0.60	Minimum seismic coefficient for pipe support and foundation design
This Practice Note's recommendation:			
Hazard Factor	Z	0.39	From above for Hastings
Piping, pipe supports and foundation:			
Table H1	$0.8 * F_{ph} / W_p$	0.54	$0.8 * F_{ph} / W_p = 0.50 + 0.05 * 4/5$ Interpolated value from Table H1 for 250-year return period earthquake
	$C_d(T_1)$	0.68	∴ $C_d(T_1) = 0.54 / 0.8$ for the ULS design of the pipe support and foundation using steel and concrete Standards
Determine inertia forces on pipe supports and foundation:			
Structural Ductility Factors:			
ULS - NZS 3404 Table 12.2.4	μ_{uls}	1.25	Nominally ductile structure
		3.0	Limited ductility structure
Want to do capacity design?		No	Apply earthquake Horiz:Horiz:Vert = 1.0:0.3:0.0 as per NZS 1170.5 Clause 5.3.1.2(a) and $\mu_{uls} = 1.25$
		Yes	Apply earthquake Horiz:Horiz:Vert = 1.0:0.0:0.0 as per NZS 1170.5 Clause 5.3.1.1(a) and $\mu_{uls} = 3.0$
SLS1 - NZS 1170.5 Clause 4.3.2	μ_{sls1}	1.0	
Structural Performance Factors:			From NZS 1170.5 Clause 4.4.2:

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DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
<u>Nominally ductile structure:</u>			
ULS	S_p	0.925	With $\mu = 1.25$
SLS1	S_p	0.7	Allow code value for SLS1 case
<u>Limited ductility structure:</u>			
ULS	S_p	0.7	With $\mu = 3.0$ Even though no redundancy, as not a critical item use minimum value of S_p
SLS1	S_p	0.7	Allow code value for SLS1 case
Inelastic Spectrum Scaling Factors:			
<u>Nominally ductile structure:</u>			
ULS	$k_{\mu \text{ uls}}$	1.14	NZS 1170.5 Clause 5.2.1.1 with $\mu = 1.25$
SLS1	$k_{\mu \text{ sls1}}$	1.0	
<u>Limited ductility structure:</u>			
ULS	$k_{\mu \text{ uls}}$	2.14	With $\mu = 3.0$
SLS1	$k_{\mu \text{ sls1}}$	1.0	
Unscaled 5% Damped Design Actions:			
			NZS 1170.5 Eqn 5.2(1)
			$C_d(T_1)_{\text{uls}} = C(T_1) * S_p / k_{\mu}$
<u>Nominally ductile structure:</u>			
ULS	$C_d(T_1)_{\text{uls}}$	0.71	$C_d(T_1)_{\text{uls}} = 0.878 * 0.925 / 1.14$
NZS 1170.5 Eqn 5.2(2)	$(Z/20 + 0.02)R_u$	0.03	$\therefore C_d(T_1)_{\text{uls}} = 0.71$
SLS1	$C_d(T_2)_{\text{sls1}}$	0.21	$C_d(T_2)_{\text{sls1}} = 0.293 * 0.7 / 1.0$
<u>Limited ductility structure:</u>			
ULS	$C_d(T_1)_{\text{uls}}$	0.29	$C_d(T_1)_{\text{sls2}} = 0.878 * 0.7 / 2.14$
SLS1	$C_d(T_2)_{\text{sls1}}$	0.21	$C_d(T_2)_{\text{sls1}} = 0.293 * 0.7 / 1.0$
Review P-Delta effects:			
NZS 1170.5 Clause 6.5.2(a)	$T_1 > 0.4 \text{ sec}$	No	No need to consider P-Delta effects
NZS 1170.5 Clause 6.5.2(b)	$h_n > 15 \text{ m}$	No	
	$T_1 > 0.6 \text{ sec}$	No	No need to consider P-Delta effects
	k_1	1.0	<u>Therefore don't include P-Delta effects</u>
System Damping Factors:			Chosen values for fully welded steel structure:
<u>Nominally ductile structure:</u>			
ULS	$C_f(3)$	1.18	Damping = 3% of critical
SLS1	$C_f(1)$	1.53	Damping = 1% of critical
<u>Limited ductility structure:</u>			
ULS	$C_f(5)$	1.0	Damping = 5% of critical as yielding expected with $\mu = 3$
SLS1	$C_f(1)$	1.53	Damping = 1% of critical
Material Code Scaling Factor:			
ULS	k_2	1.0	No steel standard requirement to scale ULS earthquake
System Scaling Factors:			
<u>Nominally ductile structure:</u>			

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
ULS	K	1.18	$K = k_1 * k_2 = 1.18 * 1.0$ See Section 6.4.1 above
SLS1	K	1.53	$K = k_1 * k_2 = 1.53 * 1.0$
<u>Limited ductility structure:</u>			
ULS	K	1.0	$K = k_1 * k_2 = 1.0 * 1.0$
SLS1	K	1.53	$K = k_1 * k_2 = 1.53 * 1.0$

Horizontal Design EQ Forces on pipe supports:

Nominally ductile structure:

ULS – To avoid collapse	$E_u = V_{uls}$	$0.84 W_t$	$= C_d(T_1)_{uls} * K * W_t$ $= 0.71 * 1.18 * W_t$ O.K. AS/NZS 1200 Appendix I minimum value = 0.60 This Practice Note minimum value = 0.68
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∴ For support	E_u V_{uls}	$0.84 W_t$ 7.7 kN	$= 0.84 * 9.2 \text{ kN}$ Apply in two horizontal directions concurrently 7.7 kN and $0.3 * 7.7 = 2.3 \text{ kN}$ Check if any piping guides, line stops or anchor forces apply greater total horizontal force than 7.7 kN. If not: 150UC30 Grade 300+ is OK, load capacity ratio = 0.23
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SLS1 – To sustain no damage	$E_{sls1} = V_{sls1}$	$0.31 W_t$	$= C_d(T_2)_{sls1} * K * W_t$ $= 0.21 * 1.53 W_t$
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Limited ductility structure:

ULS – To avoid collapse	$E_u = V_{uls}$	$0.29 W_t$	$= C_d(T_1)_{uls} * K * W_t$ $= 0.29 * 1.0 W_t$ Too low. AS/NZS 1200 Appendix I minimum value = 0.60 This Practice Note minimum value = 0.68
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∴ For support	E_u V_{uls}	0.68 Wt 6.3 kN	$= 0.68 * 9.2 \text{ kN}$ Apply in two horizontal directions separately Check if any piping guides, line stops or anchor forces apply greater total horizontal force than 6.3 kN. If not: 150UC30 Grade 300+ OK, load capacity ratio = 0.16
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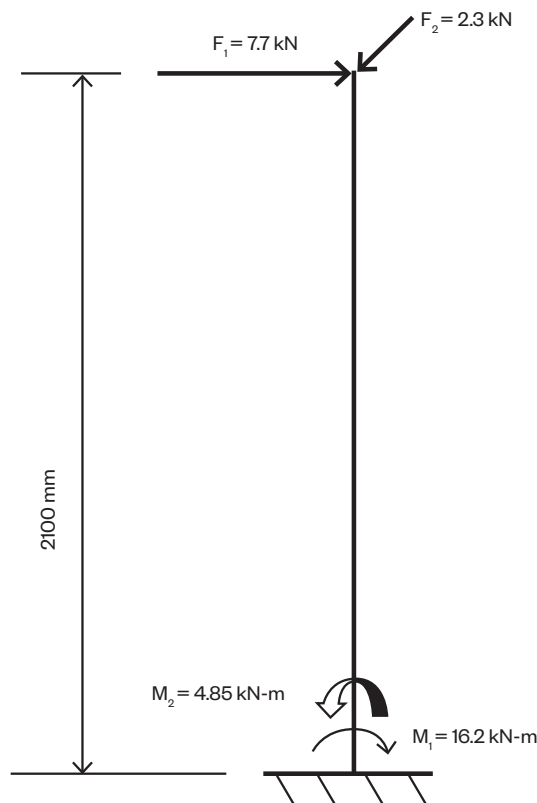
Horizontal Elastic Level EQ Forces on foundation and HD bolts:

Spectral Shape Factor:

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Figure 23: Nominally ductile structure force model

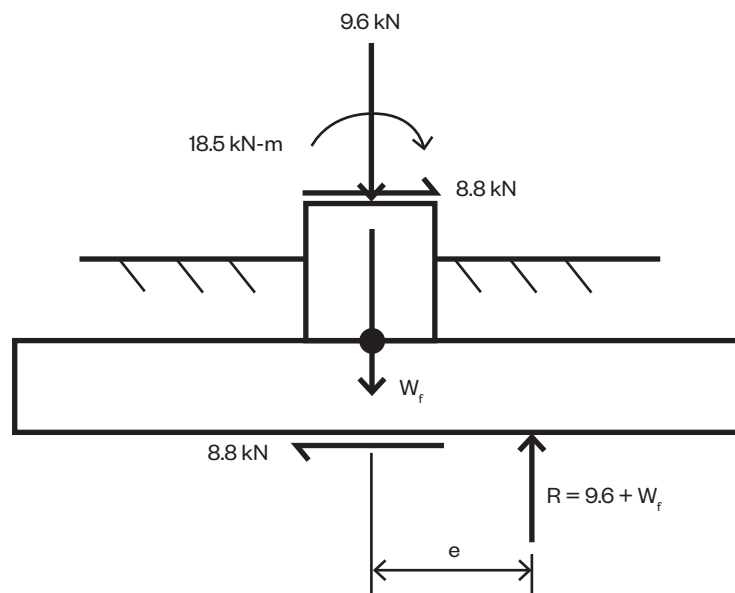


Foundation and support's HD bolts to be designed for the minimum of the over-strength of collapse mechanism or the elastic level earthquake

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
NZS 1170.5 Table 3.1	$C_h(T_1)$	3.0	Site Subsoil Class = D & $T_1 = 0.25$ secs
<u>Nominally ductile structure:</u>			
NZS 3404 Clause 12.9.1.2.2 (4) (b)	μ	1.0	
NZS 1170.5 Clause 4.4.1 and 4.4.2	S_p	1.0	Can be used for design of HD bolts and foundation plus stability check on foundation
NZS 1170.5 Clause 5.2.1.1	$k \mu$	1.0	
	$C_f(2)$	1.32	Damping = 2% of critical
	$C(T_1)$	0.878	$C(T_1) = 3.0 * 0.75 * 0.39 * 1.0$
Unscaled ULS Design Action	$C_d(T_1)^{\circ}_{uls}$	0.878	$C_d(T_1) = 0.878 * 1.0 / 1.0$
ULS – To avoid premature non-ductile failure	$E_{max} = V^{\circ}_{uls}$	$1.16 W_t$	$= C_d(T_1) * C_f(2) * W_t$ $= 0.878 * 1.32 W_t$ O.K. AS/NZS 1200 Appendix I minimum value = 0.60 This Practice Note minimum value = 0.68
<u>Limited ductility structure:</u>			
NZS 3404 Clause 12.9.1.2.2 (4) (b)	μ	1.25	

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
NZS 1170.5 Clause 4.4.1	S_p	1.0	Used for stability check on foundation
NZS 1170.5 Clause 4.4.2	S_p	0.925	Used for design of HD bolts and foundation
NZS 1170.5 Clause 5.2.1.1	k_μ	1.21	
	$C_f(2)$	1.32	Damping = 2% of critical as wanting maximum elastic response.
	$C(T_1)$	0.878	$C(T_1) = 3.0 * 0.75 * 0.39 * 1.0$
Unscaled ULS Design Actions:			
HD bolts and foundation	$C_d(T_1)^o_{uls}$	0.668	$C_d(T_1)^o_{uls} = 0.878 * 0.925 / 1.21$
Foundation stability	$C_d(T_1)^o_{ot}$	0.723	$C_d(T_1)^o_{ot} = 0.878 * 1.0 / 1.21$
ULS – To avoid premature non-ductile failure of HD bolts and reinforced concrete foundation	$E_{max} = V^o_{uls}$	$0.88 W_t$	$= C_d(T_1)^o_{uls} * C_f(2) * W_t$ O.K. AS/NZS 1200 Appendix I minimum value = 0.60
ULS – To avoid sliding or over-turning of the foundation	$E_{max} = V^o_{ot}$	$0.96 W_t$	$= C_d(T_1)^o_{ot} * C_f(2) * W_t$ This Practice Note minimum value = 0.68

Figure 24: Limited ductility structure - foundation stability check force model.



Example E6: Earthquake forces on piping

Figure 25: Earthquake forces on piping.

Assume a welded steel cantilever T post support as in Appendix E5. The design is to follow the preferred work practice of the ACPPE (2001) this Practise Note.

As the 150 NB piping is classed as pressure piping in the ACPPE (2001) and is not exempt as per the 2005 Gazette notice, the design of piping, supports and foundations is governed by this Practise Note and Section 6 of the ACPPE (2001). Pipe stress checking to be in accordance with ASME B31Ea-2010.

Assume the 150 NB pipe (168.3 mm outside dia,) has a wall thickness of 3.4 mm with no corrosion or erosion allowance with a specified minimum yield stress at normal operating temperature $S_y = 241$ MPa. Static weight of pipe plus water is 336 N/m.

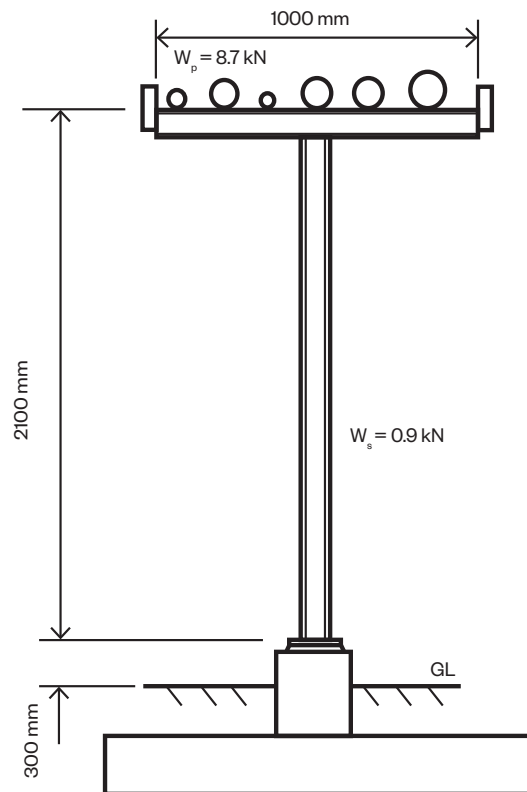


Table E6: Equivalent static earthquake coefficients calculation for supported piping

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Determine inertia forces on piping for pipe stress analysis:			
System Information:			
Level of allowable deformity		Medium	Flange jointed but with tolerance for leaks after an event, $\therefore \mu_p = 2.0$ for ULS assuming pipe will yield
Weight of components:			
Weight of structure	W_s	9.2 kN	From Example E5
Weight of piping	W_p	1.7 kN	$1.7 \text{ kN} < 0.2 * 9.2 \text{ kN} \therefore$ can use NZS 1170.5 Section 8
Overall Height:	h_n	2.4 metres	
Horizontal Fundamental periods:			
Support including piping	T_1	0.25 sec	From Example E5 for ULS
	T_2	0.20 sec	From Example E5 for SLS1
Piping	T_{ph}	0.15 sec	From pipe stress engineer
Vertical Fundamental periods:			
Support	T_{v1}	0.01 sec	From Example E5 for ULS and SLS1

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Piping	T_{pv}	0.15 sec	From pipe stress engineer
Pipe section properties:			
Elastic modulus	Z_e	71.2 cm ³	$Z_e = \pi * (16.83^4 - (16.83 - 2 * 0.34)^4) / (32 * 16.83)$ cm ³
Yield moment	M_{yield}	17.2 kN-m	$M_{yield} = Z_e * S_y = 71.2 * 241$ N-m As the system is supported every 6.0 m (from Example E5) – the applied load w_{yield} to generate such a moment can be estimated assuming the maximum moment is: $w_{yield} * span^2 / 10$ and that pressure and temperature induce negligible bending moments in the system.
	w_{yield}	4.7 kN/m	$w_{yield} \geq 17.2 * 10 / 6^2$ As the static weight is only 0.34 kN/m it is unlikely that seismic will induce enough bending to cause yielding in the pipe unless there is significant bending due to pressure and temperature.

Earthquake parameters:

Spectral Shape Factors:

NZS 1170.5 Table 3.1	$C_h(0)$	1.12	Site Subsoil Class = D & T = 0.0 secs
	$C_h(T_v)$	3.0	Site Subsoil Class = D & T _v = 0.15 secs
5% Damped Elastic spectrum @ T = 0 sec:			
ULS	$C(0)$	0.328	$C(0) = C_h(0) R Z N(0,D)$ $C(0) = 1.12 * 0.75 * 0.39 * 1.0$

Determine the parameters for the Parts and Components analysis:

Part Category:

NZS 1170.5 Table 8.1	ULS	P.1	High pressure system representing danger to those working within plant. $R_{pu} = 1.0$
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Displacement Ductility Factors:

ULS	μ_p	1.0	NZS 1170.5 Clause 8.6 – need to check if system yields under elastic loading
ULS	μ_p	2.0	NZS 1170.5 Supp. Table C8.2
Risk Factor for Parts:			
ULS	R_p	1.0	NZS 1170.5 Table 8.1

Parts Response Factors:

ULS	C_{ph}	1.00	NZS 1170.5 Table 8.2 and cl. 8.6 For horizontal forces, assume elastic response
ULS	C_{ph}	0.55	For horizontal forces, assume yielding will occur in pipe

Parts Response Factors:

ULS	C_{pv}	1.00	For vertical forces, assume elastic response
ULS	C_{pv}	0.55	For vertical forces, assume yielding will occur in pipe

System Damping Factors:

ULS	$C_f(3)$	1.18	Chosen values: Damping = 3% of critical if $\mu_p = 1.0$
ULS	$C_f(5)$	1.0	Damping = 5% of critical if $\mu_p = 2.0$ and have yielding in pipe

Floor Spectral Shape Factor:

NZS 1170.5 Clause 8.4	$C_i(T_p)$	2.0	$T_p < 0.75$ secs
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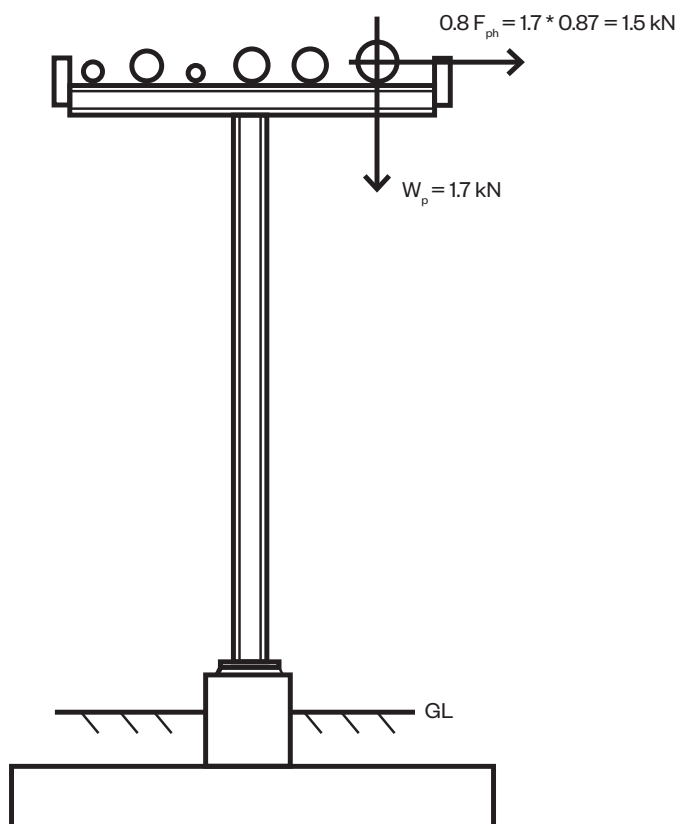
Floor Height Coefficients:

Support	C_{Hi}	1.4	NZS 1170.5 Eqns 8.3(1), 8.3(2) and 8.3(3) As $h_n < 12$ m, Eqn 8.3(1) applies $\therefore C_{Hi} = 1 + 2.4 / 6 = 1.40$
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DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Horizontal Forces on Part:			
Spectral Ordinate of the Part:			$C_p(T_p) = C(0) * C_{Hi} * C_i(T_p)$ NZS 1170.5 Eqn 8.2(1)
ULS	$C_p(T_p)$	0.92	$C_p(T_p) = 0.328 * 1.4 * 2.0$
Horizontal Design EQ Actions:			$F_{ph} = C_p(T_p) * C_{ph} * R_p * C_i(\xi) * W_p$ NZS 1170.5 Eqn 8.5(1)
ULS - No collapse or yield	$E_u = F_{ph}$	$1.09 W_p$	$F_{ph} = 0.92 * 1.0 * 1.0 * 1.18 * W_p$
ULS - No collapse but yielding	$E_u = F_{ph}$	$0.51 W_p$	$F_{ph} = 0.92 * 0.55 * 1.0 * 1.0 * W_p$
WSD - No yield	$E_{wsd} = 0.8 E_u$	$0.87 W_p$	$= 0.8 * 1.09 W_p$ if piping remains elastic
WSD - No loss of contents	$E_{wsd} = 0.8 E_u$	$0.41 W_p$	$= 0.8 * 0.51 W_p$ if piping yields under elastic load levels
			AS/NZS 1200 Appendix I min. value = 0.60
			This Practice Note minimum value = 0.54 from example E5.
\therefore	E_{wsd}	$0.87 W_p$	To check if internal bending actions greater than yield moment M_{yield} Internal forces and moments = M_{iux} and M_{iuy}

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Figure 26: Values for forces acting on the part.



DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
Horizontal Design Actions due to EQ deflections:			
ULS deflections at pipe support	δ_{iesa}	1 mm	<p>Seismic deflections small and within tolerances for pipe guides. Likely displacement induced actions from this support are minimal.</p> <p>Piping system analysed for ULS deflection case with the system's imposed deflections in two principal directions analysed separately.</p> <p>Internal forces and moments = $M_{\delta ux}$ and $M_{\delta uy}$</p>
Piping Design Horizontal Actions due to EQ acting on piping			
WSD:			
X direction	M_{hux}	$(M_{iux}^2 + M_{\delta ux}^2)^{0.5}$	
Y direction	M_{huy}	$(M_{iuy}^2 + M_{\delta uy}^2)^{0.5}$	
Vertical Forces on Part:			
5% Damped Elastic spectrum @ $T = T_{vi}$ sec:			$C_v(T_{vi}) = 0.7 C_h(T_v) R Z N(T_v, D)$ NZS 1170.5 Eqns 3.1(1) and 3.2(1)
ULS	$C_v(T_v)$	0.614	$C_v(T_v) = 0.7 * 3.0 * 0.75 * 0.39 * 1.0$
ULS	C_{vd}	0.614	NZS 1170.5 Eqn 5.4(2)
Vertical Design EQ Forces:			$F_{pv} = C_{vd} * C_{pv} * R_p * C_i(\zeta) * W_p$ NZS 1170.5 Eqn 8.5(2)
ULS - No collapse or yielding	$E_u = F_{pv}$	$0.73 W_p$	$F_{pv} = 0.614 * 1.0 * 1.0 * 1.18 * W_p$
WSD – No yielding	$E_{wsd} = 0.8E_u$	$0.58 W_p$	$= 0.8 * 0.73$ WSD case of 0.58g in vertical direction. Internal forces and moments = M_{uv}
Piping Total Design Actions due to EQ – No yield			
WSD			Piping system to be analysed for: WSD case of 0.87g in two principal directions being analysed separately but to be combined with vertical earthquake 0.58g actions
X direction	M_{xu}	$M_{hux} + M_{uv}$	
Y direction	M_{yu}	$M_{huy} + M_{uv}$	<p>These earthquake actions need to be combined with normal operating pressure and temperature conditions for pipe stress checking.</p> <p>If checked to ASME B31Ea and passes, no need to check with $\mu_p > 1.0$. If does not pass, check whether M_{xu} or $M_{yu} > M_{yield}$ to see if μ_p can be = 2.0</p>
Piping Total Design Actions due to EQ: Yielding occurs			From above:
Horizontal actions:			
WSD – No loss of contents	$E_{wsd} = 0.8E_u$	$0.41 W_p$	$= 0.8 * 0.51 W_p$ if piping yields under elastic load levels
∴	E_{wsd}	$0.54 W_p$	To comply with minimum from this Practice Note
Vertical actions:			
ULS - No collapse but yielding	$E_u = F_{pv}$	$0.34 W_p$	$F_{pv} = 0.614 * 0.55 * 1.0 * W_p$
WSD – No loss of contents	$E_{wsd} = 0.8E_u$	$0.27 W_p$	$= 0.8 * 0.34$
WSD			Piping system to be analysed for: WSD case of 0.54g in two principal directions being analysed separately but to be combined with vertical earthquake 0.27g actions and checking to ASME B31Ea

DESCRIPTION	PARAMETER	VALUE	CONSEQUENCE
WSD:			
X direction	M_{xu}	$M_{hux} + M_{uv}$	<p>These earthquake actions need to be combined with normal operating pressure and temperature conditions for pipe stress checking.</p> <p>The horizontal and vertical actions are combined vectorially as the vertical actions have been reduced to allow for the non-concurrence of peak reactions happening at the same instant.</p>
Y direction	M_{yu}	$M_{huy} + M_{uv}$	

Example E7: Low temperature piping

To be issued.

APPENDIX F

USE OF DESIGN STATEMENTS FOR SUPPORT STRUCTURES

PECPR Regulations require equipment, including foundations, to be designed and constructed in accordance with generally accepted design practice and manufactured according to the verified design¹. The requirements for the certification of the design and construction of supporting structures and foundations are specified in the ACPPE².

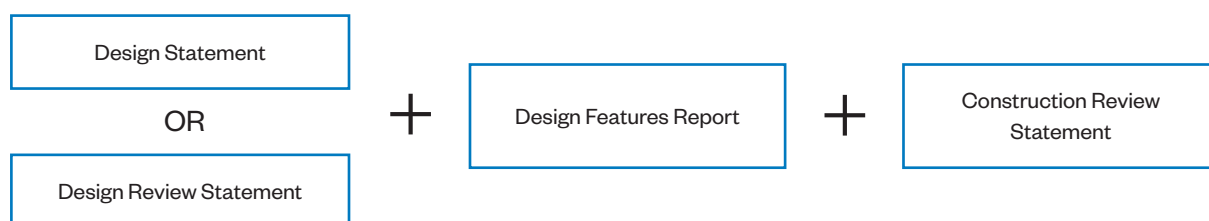
The ACPPE at 3.4.1 (2) requires a CPEng to certify the **design** of the foundations and support structures.

At A4.6 a CPEng must certify the **design and construction** of the foundations and building structures that support equipment.

At 5.4.4 (9) a CPEng must certify the **design and construction** of the seismic features of the building structure and foundations.

Based on the ACPPE, Inspection Bodies require a statement from a Chartered Professional Engineer that the supports and foundations have been designed to an appropriate level. The standard Engineering New Zealand/NZIA/ACENZ Producer Statements have previously been used for this purpose even though they were intended for use for buildings under the Building Act 1991. However, Pressure Equipment Inspection Bodies now no longer accept standard Producer Statements and will only accept modified forms for design, design review and construction review as appropriate forms of certification.

Equipment Inspectors will expect to receive either a **Design Statement** or a **Design Review Statement** accompanied by a **Design Features Report**. Equipment Inspectors will also require a **Construction Review Statement** before they will let the plant go “live” irrespective of the size of the structure.



Example forms for pressure equipment design are given at end of this Appendix.

Note: These forms are examples only. Design professionals (CPEng), will need to create their own forms.

Structures that are designed overseas but are to be installed in New Zealand will need **Design Review Statements** to be signed by a NZ CPEng; an overseas Design Certificate will not be accepted.

The pressure equipment designer will need to provide the following information to the support structure designer in order for the **Design Statement** to be completed:

- The loads applied by the equipment at its supports, ensuring that load combinations are stated and combined loads take proper account of vectors.
- Whether the loads are based on Working Strength Design (WSD) or Ultimate Limit State (ULS).
- Any limits on displacements of the equipment or its supports.

¹ Health and Safety in Employment (Pressure Equipment, Cranes and Passenger Ropeways) Regulations 1999 regulation 4(1)(b). (PECPR regulation 4(1)(b), 18, 19(1)(a)).

² Approved Code of Practice for Pressure Equipment (Excluding Boilers) clauses 3.4.1(2), 3.4.2, 5.4.4 and 6.4.3.

- The specific seismic coefficient(s) used.
- The specific wind and snow loads used to design the equipment.

In addition, the elements that comprise the interface between the pressure equipment and the supports or foundations need to be specified. Examples are given in Table F1.

Table F1

ITEM	RESPONSIBILITY	COMMENT
Reinforcing plates on vessel shell ends for attachment of supports (e.g. saddles, leg)	PE designer	Part of the pressure vessel design
Anchor bolts ⁵	PE designer	Bolt PCD (Pitch Circle Diameter), number, diameter, material. The determination of bolt embedment depth and interaction effects is a specialist area generally undertaken by civil/structural engineers. The PE and concrete designers may need to collaborate to obtain a workable compromise.
Vessel support skirt	PE designer	Co-ordination may be required with structural designer to ensure adequate concrete bearing area.

It is essential that all pressure equipment, support structure and foundation drawings, and all specifications are accurately documented and version controlled. Design parameters must be recorded on the **Design Features Report**. The design parameters given in the Design Features Report should illustrate not just the support structure's parameters but also those for any supported pressure equipment. If possible the Design Verification Certificate should make reference to the Design or Design Review Statement. On occasions the Design/Design Review statements are written after the Design Verification work has been done but this is not accepted as good practice.

SEISMIC COEFFICIENTS

The supports for the pressure equipment have seismic actions derived from NZS 1170.5 Sections 3-6 whereas the majority of the pressure equipment will have its seismic actions determined from Section 8 of NZS 1170.5 or NZS 4219. These seismic coefficients will be different, so the coefficient given on the DV certificate will almost always be different from that given for the supporting structure. Further, for designs made in accordance with capacity design principles, (e.g. NZS 1170.5 clauses 4.4.1 & 5.6) the foundations will have a different seismic design coefficient from that used to design the above ground structure.

The only times the coefficients will be the "same" for supports and pressure equipment are:

- When the pressure equipment is located at the same level and is treated as the structure.
- When the pressure equipment is elevated and so heavy and flexible it is treated as a Combination Structure as shown in Section 7 of PN 19 and included in the structural analysis model.

Even for these, if the foundations and the supports are designed by limit state methods, the coefficients for these items will differ by the 0.8 factor from that used for the pressure equipment.

³ Scarry J: Anchor bolts for steel structures, SESOC Journal 22, Sept 2009 pp 30 – 55.

Holdemup Structures Limited

company logo

DESIGN STATEMENT

**PRESSURE EQUIPMENT SUPPORTS
AND FOUNDATIONS**

Reference Number nnnnnnnnnn

ISSUED BY: Holdemup Structures Ltd

(Structural Engineering Firm)

TO: Titus A. Balloon Ltd

(Client)

IN RESPECT OF: N support structure and foundations

AT: Location

We, Holdemup Structures Ltd, have been engaged to provide design services for

☐ All or ☒ Part only (as specified below), of the proposed structure(s).

Structural/civil design only

The **pressure equipment** covered by this statement is described on drawings:

Titus A. Balloon Ltd

Pressure Equipment Engineering Firm:

Drawing number, Revision number, Title

(Continue on a separate sheet as required)

together with the specification, and other documents set out in the schedule below:

For example, as provided on the drawing: Nnn.

The **proposed foundation and support structures** covered by this statement are described on drawings:

Holdemup Structures Ltd:

Drawing number, Revision number, Title

(Continue on a separate sheet as required)

together with the specification, and other documents set out in the schedule below:

For example, as provided on the drawing: Nnn.

On behalf of Holdemup Structures Ltd, and subject to:

- i. Site verification of the following design assumptions:
 - a. *Ultimate bearing strength of the soil to be no less than xxx kPa*
 - b.
- ii. All proprietary products meeting their performance specification requirements;
- iii. Construction Monitoring specified on separate sheet.

I believe on reasonable grounds that the structure, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code and the Pressure Equipment, Cranes, and Passenger Ropeways Regulations to support the specified pressure equipment against the loads specified by

Titus A. Balloon Ltd

Pressure Equipment Engineering Firm

I, An Nother Engineer

am OPEng #nnnnn

(Name of Design Professional)

I hold the following qualifications: OPEng, B.E.

The Structural Design Firm issuing this statement holds a current policy of Professional Indemnity Insurance no less than \$200,000.

SIGNED BY **An Nother Engineer** ON BEHALF OF Holdemup Structures Ltd

Signed **A.N. Engineer** Date: DD/MM/YYYY

Note: This statement is issued for the purposes of the Health and Safety in Employment (Pressure Equipment, Cranes and Passenger Ropeways) Regulations 1999. Liability under this statement accrues to the Structural Engineering Firm only. The total maximum amount of damages payable arising from this statement and all other statements provided to the Inspection Body in relation to this structure and foundation, whether in contract, tort, or otherwise (including negligence), is limited to the sum of \$200,000.

Holdemup Structures Limited

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DESIGN REVIEW STATEMENT**PRESSURE EQUIPMENT SUPPORTS
AND FOUNDATIONS**

Reference Number nnnnnnnnnn
ISSUED BY: Holdemup Structures Ltd (Structural Engineering Firm)
TO: Titus A. Balloon Ltd (Client)
IN RESPECT OF: N support structure and foundations
AT: Location

We, Holdemup Structures Ltd, have been engaged to provide design review services for:

☒ All or ☐ Part only (as specified below), of the structural/civil design by **Overseas Designers Ltd** of the proposed pressure equipment support structure(s) -.

The **pressure equipment** covered by this statement is described on drawings:

Titus A. Balloon Ltd Pressure Equipment Engineering Firm:
 Drawing number, Revision number, Title
 (Continue on a separate sheet as required)

together with the drawings, specifications and other documents set out in the schedule below:

The **proposed foundation and support structures** covered by this statement are described on drawings:

Overseas Designers Ltd:
 Drawing number Revision number Title
 (Continue on a separate sheet as required)

together with the specification, and other documents set out in the schedule below:

Document number Revision number Title
 (Continue on a separate sheet as required)

On behalf of Holdemup Structures Ltd, and subject to:

- i. Site verification of the following design assumptions:
 - a. Ultimate bearing strength of the soil to be no less than xxx kPa
 - b.
- ii. All proprietary products meeting their performance specification requirements;
- iii. Construction Monitoring specified on separate sheet.

I believe on reasonable grounds that the structure, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code and the Pressure Equipment, Cranes, and Passenger Ropeways Regulations to support the specified pressure equipment against the loads specified by

Titus A. Balloon Ltd Pressure Equipment Engineering Firm

I, An Nother Engineer am CPEng #nnnnn
 (Name of Design Professional)

I hold the following qualifications: CPEng, B.E.

The Structural Design Firm issuing this statement holds a current policy of Professional Indemnity Insurance no less than \$200,000.

SIGNED BY **An Nother Engineer** ON BEHALF OF Holdemup Structures Ltd

Signed **A.N. Engineer** Date: DD/MM/YYYY

Note: This statement is issued for the purposes of the Health and Safety in Employment (Pressure Equipment, Cranes and Passenger Ropeways) Regulations 1999. Liability under this statement accrues to the Structural Engineering Firm only. The total maximum amount of damages payable arising from this statement and all other statements provided to the Inspection Body in relation to this structure and foundation, whether in contract, tort, or otherwise (including negligence), is limited to the sum of \$200,000.

Design Features Report

The design carried out by Holdemup Structures Ltd has been prepared in accordance with loads from supported equipment specified by Titus A. Balloon Ltd, NZ Building Code, good engineering practice and Standards:

AS/NZS 1170 set *Structural Design Actions*
NZS 3404: *Steel Structures*

NZS 3101 *Concrete Structures*
Other (specify)

Titus A. Balloon Ltd has determined the following design actions and supplied them to Holdemup Structures Ltd3:

(Sketch or reference to another drawing)

The unfactored dead weight of the equipment supported $W =$ kN

The AS/NZS 1170.0 Importance Level of the equipment supported IL =

The design working life of the equipment supported DWL = years

The annual probability of exceedance of the Ultimate Limit State earthquake APE =

The subsoil class assumed at the site is

The period of the first mode of vibration $T =$ seconds

The spectral shape factor $C_h(T) =$

The hazard factor $Z =$

The return period factor $R_u =$

The near fault factor $N(T,D) =$

The horizontal elastic site hazard spectrum $C(T) =$

The vertical elastic site hazard spectrum $C_v(T) =$

The structural ductility factor $\mu =$

The structural performance factor $S_p =$

The inelastic spectrum scaling factor $k\mu =$

The damping adjustment factor $C_f(x) =$

The horizontal design action coefficient $C_d(T_p) =$

The vertical design action coefficient $C_{vd} =$

The lateral seismic coefficient used is g, the vertical coefficient is g, added vectorially for design.

The design wind speed $V_{des,0} =$ m/s

The design snow load $s =$ kN/m²

The data reported here is to be sufficient for the Inspection Body to establish a secure audit trail from the foundation to the pressure equipment.

Signed **A.N. Engineer** on behalf of Holdemup Structures Ltd

⁴ The data reported here is to be sufficient for the Inspection Body to establish a secure audit trail from the foundation to the pressure equipment.

Site Verification Structures Limited
company logo

CONSTRUCTION REVIEW STATEMENT
PRESSURE EQUIPMENT SUPPORTS
AND FOUNDATIONS

Reference Number XXXXXXXXXX

ISSUED BY: Site Verification Structures Ltd (Structural Engineering Firm)
TO: Titus A. Balloon Ltd (Client)
IN RESPECT OF: N Support structure and foundations
AT: Location

We, **Site Verification Structures Ltd**, have been engaged to provide construction monitoring services for

☐ All or ☒ Part only (as specified below), of the proposed structure(s).
Structural/civil construction monitoring only.

The foundation and support structures named on the Design/Design Review Statement issued by Holdemup Structures Ltd [Reference No qqqqqqqq, dated dd/mm/yyyy] and described on the drawings, specification, and other documents listed therein have been monitored through construction by **Site Verification Structures Ltd**.

On behalf of Site Verification Structures Ltd, I confirm that the structure was monitored throughout construction by Site Verification Structures Ltd and fully complies with the drawings, specifications and other construction related documents listed in the above mentioned Design/Design Review Statement .

I hold the following qualifications: CPEng (number) B.E.

The structural engineering firm issuing this statement holds a current policy of Professional Indemnity Insurance no less than \$200,000.

I believe on reasonable grounds that the structure, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code and the Pressure Equipment, Cranes, and Passenger Ropeways Regulations to support the specified pressure equipment against the loads specified by

Titus A. Balloon Ltd

SIGNED BY Dedicated Engineer ON BEHALF OF Site Verification Structures Ltd

Signed **Dedicated Engineer** Date: DD/MM/YYYY

Note: This statement shall be relied upon only for the purposes of the Health and Safety in Employment (Pressure Equipment, Cranes and Passenger Ropeways) Regulations 1999. Liability under this statement accrues to the Structural Engineering Firm only. The total maximum amount of damages payable arising from this statement and all other statements provided to the Inspection Body in relation to this structure and foundation, whether in contract, tort, or otherwise (including negligence), is limited to the sum of \$200,000.

APPENDIX G

ROTATING EQUIPMENT SEISMIC ISSUES

General

The ACPPE Parts 7 and 8 refer to various types of new installations of rotating equipment that are subject to the PECPR Regulations. Some of the types included are:

- compressors
- pumps
- gas turbines
- steam engines
- steam turbines.

In order to ensure that the equipment is safe to operate, Equipment Controllers are required to ensure that the equipment is designed, inspected, operated and maintained in accordance with the PECPR Regulations.

For New Zealand the seismic design of the rotating equipment is an essential element of design. Obtaining compliance with seismic loading requirements is commonly not a straight forward or simple task because:

- there are few standards governing rotating equipment
- the standards that do exist are generally silent on seismic design requirements
- the basis or acceptance criteria for seismic design of rotating equipment is not explicitly stated
- it is often difficult to obtain meaningful seismic design and qualification information from equipment suppliers.

Methodology

The methodology described below draws significantly on the American Lifelines Alliance document “*Guide for Seismic Evaluation of Active Mechanical Equipment*”, October 2004.

STEPS TO EVALUATING AND QUALIFYING ROTATING EQUIPMENT FOR SEISMIC ACTION

There are four steps to be followed to evaluate and qualify a piece of rotating equipment for seismic action:

- Determine the required Seismic Function
- Determine the Qualification Method
- Determine the Design Seismic Action to be applied to the equipment
- Evaluate the equipment using the chosen Qualification Method.

DETERMINE SEISMIC FUNCTION

The determination of **Seismic Function** for a piece of rotating equipment firstly requires consideration of the required seismic performance of the overall site or plant in which it is installed. Secondly the part that the equipment plays in this seismic performance must be decided. Thirdly the performance of the specific piece of equipment to meet these requirements must be determined.

(1) State the site or plant **Safety Objective**. For example:

- maintain an operable emergency power supply to a hospital
- safely shutdown a chemical reactor
- avoid spills of flammable materials from a tank
- maintain a leak tight confinement in a building processing toxic gases
- permit the safe evacuation of workers from a process building.

(2) Determine the structures and systems relied upon to accomplish the site or plant **Safety Objective**. For example, in a facility processing toxic materials, the plant safety functions may be to: maintain the leak tight integrity of the building structure,

- shutdown and isolate systems containing toxic materials,
- evacuate personnel,
- assure the integrity of the exhaust fans and air handling units (scrubbers, filters, flare, etc.),
- qualify the interlock between the intake and exhaust fans to assure that the intake fan does not run if the exhaust fan is lost following the earthquake (to avoid pressurizing the building),
- provide operational fire water pumps and piping system, etc.
- Consideration should also be given to the Hazard Level of the system as determined using AS 4343.

(3) Define the post-earthquake condition. For example, consider the following conditions as baseline, unless explicitly established otherwise for the plant:

- offsite power may be lost for up to three days
- the earthquake may cause a fire
- non-seismically qualified equipment may not function and may fail (pipe ruptures, tanks leak, etc.)
- operators may be expected to perform post-earthquake functions provided these are (i) feasible, (ii) documented in emergency response procedure, and (iii) drilled.

(4) Locate the piece of rotating equipment within the plant systems, and determine the required **Seismic Function** to achieve the site **Safety Objective**. The **Seismic Function** of the equipment must be defined as one of three categories:

Position Retention (<i>does not fall or overturn</i>)	The equipment is to serve no active function during or following the earthquake, but it is to remain in position, to be stable, not to fall or overturn (also referred to as structural integrity).
Leak Tightness (<i>does not leak</i>)	The equipment is to remain leak tight during or following the earthquake, not to leak through its pressure boundary.
Operability (<i>operates, functions, delivers or controls flow</i>)	The equipment is to deliver flow. For example a pump is to keep running or be able to start-up and operate for a period of time.

DETERMINE THE QUALIFICATION METHOD

There are three methods of seismic qualification:

Analysis	Loadings are applied to a model of the equipment to calculate movements, stresses or strains using either: <ul style="list-style-type: none">• Static (equivalent static method)• Dynamic (response spectrum or time history)
Testing	Shake table – applying an input excitation and testing to meet a functional requirement (Seismic Function).
Earthquake Experience	Comparison to performance of similar equipment during earthquake

ANALYSIS

Position retention and leak tightness can generally be established by analysis.

Analysis is typically used for:

- evaluation of equipment support structures and anchorage (concrete anchor bolts, welds, braces)
- evaluation of the load path through the equipment (accessories and their attachment to the equipment)
- evaluation of the load path from the centre of gravity of the equipment to its base or supports

Analysis may also be applied to operability, but only to a limited extent. The use of analysis to evaluate operability is limited to the deflection of rotating shafts, compared to manufacturer limits, bearing and seal limits, and clearances between rotating and fixed parts.

As an example, the overload failure of a pump nozzle can be predicted and quantified by analysis; but the operability of the pump after such an overload (whether the moving parts and instrumentation and controls will function correctly) cannot be predicted only by way of analysis.

The calculated reaction loads, movements, stresses or strains obtained by seismic analysis are compared to code and standard limits or manufacturer limits (such as shaft deflection) to determine the seismic adequacy of the equipment.

Table G1 below gives design/stress limits for Nuclear Power Plant components as specified in the ASME Boiler and Pressure Vessel Code, Section III, Division I, Sub-section NC. Plant Condition and Service Limit shall be reconciled with the Seismic Function that has been determined for the equipment.

Table G1: Load Combinations and Stress Limits

PLANT CONDITION	CONCURRENT LOADS	SERVICE LIMIT	DESIGN/STRESS LIMIT
Case I – Design / Normal	PD + OPL	Level A	$\sigma_m \leq 1.0S$ $(\sigma_m \text{ or } \sigma_L) + \sigma_b \leq 1.5S$
Case II – Upset	PD + OPL + SOT	Level B	$\sigma_m \leq 1.1S$ $(\sigma_m \text{ or } \sigma_L) + \sigma_b \leq 1.65S$
Case III – Emergency	PD + OPL + OCL +OBE	Level C	$\sigma_m \leq 1.5S$ $(\sigma_m \text{ or } \sigma_L) + \sigma_b \leq 1.8S$
Case IV – Faulted	PD + OPL + OCL + SSE	Level D	$\sigma_m \leq 2.0S$ $(\sigma_m \text{ or } \sigma_L) + \sigma_b \leq 2.4S$
Loading			
PD	=	design pressure and temperature	
OPL	=	operating - sustained loads including dead weight and sustained nozzle loads	
OCL	=	occasional nozzle loads (not including dead weight and sustained loads)	
SOT	=	system operating transients for upset, emergency, and faulted conditions per customer specification	
OBE	=	Operating Basis Earthquake	
SSE	=	Safe Shutdown Earthquake	
Service Levels			
Level A	=	Those loadings that the pump may be subjected to in the performance of its specified function	
Level B	=	Those loadings that the pump must withstand without damage requiring repair	
Level C	=	Those loadings that may result in large deformations that necessitate the removal of the pump from service for inspection or repair	
Level D	=	Those loadings that result in gross general deformations with some consequent loss of dimensional stability and damage requiring repair, which may require removal of the unit from service.	
Stress Limits			
σ_b	=	General membrane stress; equal to average stress across solid section under consideration; excludes discontinuities and concentrations and is produced only by pressure and other mechanical loads	
σ_L	=	Local membrane stress; same as σ_m except that σ_L includes effects of discontinuities	
σ_m	=	Bending stress; equal to linearly varying portion of stress across solid section under consideration; excludes discontinuities and concentrations and is produced only by mechanical loads	
S	=	Allowable stress, given in Tables I-7.0 and I-8.0 of the ASME code; corresponds to highest metal temperature of section during condition under consideration	
Source: ASME Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, Division I, Sub-section NC, 1992, New York			

Note:

- 1) This table has been adapted from the Pump Handbook, 4th Edition published by McGraw Hill.
See Tables 1 and 2 of Section 12.6.2, Nuclear Pump Seismic Qualifications.

Pressure vessel codes commonly allow stresses to be 1.2 x basic allowable stresses for occasional loadings such as earthquake or wind (for example refer to ASME BPVC Section VIII, Division 1 or PD-5500).

TESTING

In most cases seismic qualification testing consists in anchoring the equipment to a shake table, and exciting the table with a seismic excitation equal to or larger than the design earthquake. Commonly this would be RRS (Required Response Spectrum) testing.

EARTHQUAKE EXPERIENCE

In some cases there may be documented experience with specific rotating equipment that will adequately qualify the equipment for the required application.

Experience based methods are documented in technical reports and standards such as:

- Winston & Strawn, Seismic Qualification Utility Group, 1992. *Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment*. Seismic Qualification Utility Group
- American Society of Mechanical Engineers, 2002. *Qualification of Active Mechanical Equipment Used in Nuclear Power Plants*, New York: ASME QME
- Department of Energy Facilities, March 1997. *DOE/EH-0545 Seismic Evaluation Procedure for Equipment in U.S. DOE*.

DETERMINE THE SEISMIC ACTION

The methods described elsewhere in this document shall be used to determine the seismic action appropriate to the equipment – either an equivalent static acceleration or a response spectrum. The seismic action determined shall take into account the location within New Zealand for the equipment installation, and the location within the plant (for example at ground level or mounted on a structure at elevation).

Example calculations for the determination of seismic action are given in Appendix E.

Care must be taken when documenting the seismic action to qualify the action as either working stress based design or ultimate limit state design. To eliminate confusion it is preferable for both to be provided.

EVALUATE THE EQUIPMENT

Use the chosen Qualification Method to evaluate the equipment against the Design Seismic Action and ensure that it will achieve the required seismic function.

PASS THROUGH TO EQUIPMENT SPECIFICATION

In many cases the evaluation of the equipment will be a task that is carried out by the equipment supplier and therefore it will be necessary to summarise the previously determined requirements and place them in the purchase technical specification for the equipment.

Equipment design and specification

NOZZLE LOADING DISCUSSION

Rotating equipment depends on balanced rotating parts, limited shaft deflections and proper clearances for smooth and reliable operation. Large loads on nozzles can cause deformation of the casing and/or misalignment of the shaft.

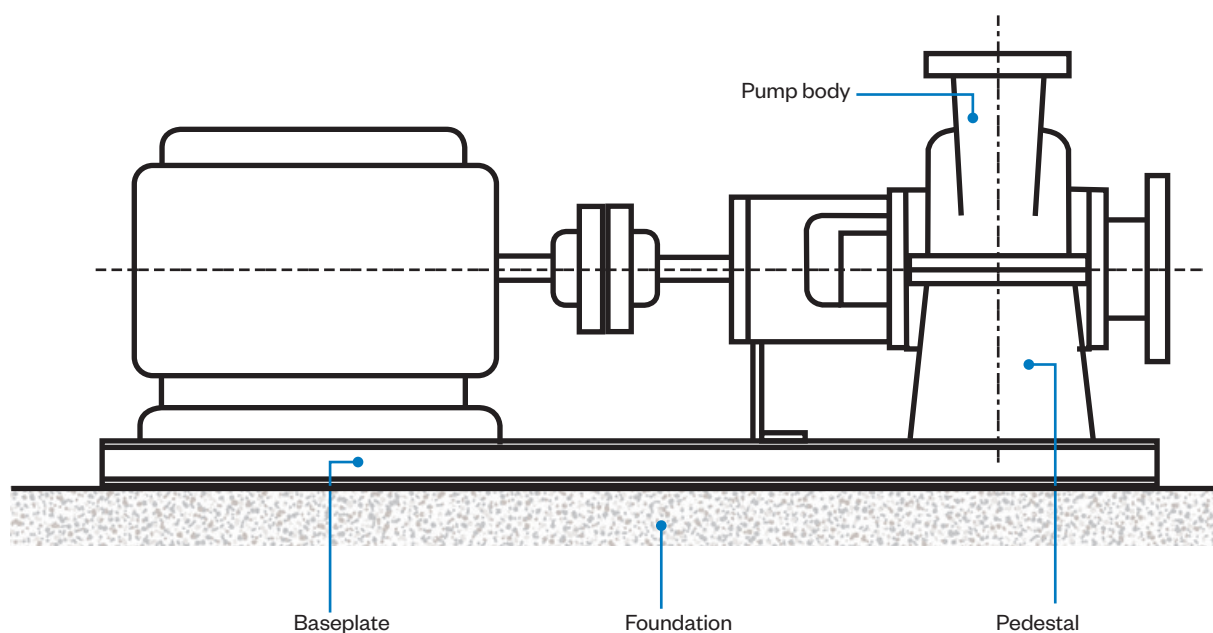
It is therefore common for manufacturers to limit the forces and moments that can be applied to the equipment nozzles. These allowable forces and moments are almost always based on operating or sustained loads and are established in order to prevent damage to the equipment during continuous operation.

PUMP CONSIDERATIONS

Consider the design of a pump assembly – there are three main parts:

- pump body – designed to be as strong, if not stronger, than the piping so that the body can resist the same internal pressure as the piping
- foundation – normally designed with the combined pump and motor effect including piping load and shaft torque. Generally massive and stiff due to the limitation of the soil bearing capacity
- pedestal/baseplate – often not designed for large piping loads.

Figure 27: Pump and motor



On many early pump designs the pedestal and baseplate were only designed for shaft torque and pump weight loads. Loads on the pump nozzles need to be carried through the casing to the mountings, and then to the foundation. This is why some pump designs can only handle very low nozzle loadings.

However this situation can be improved readily and relatively cheaply by strengthening the pump baseplate and pedestal. Therefore it is beneficial to investigate options for pump set strengthening with the equipment supplier.

For example, API 610 Table 4 gives allowable nozzle loads. Loads which are less than the tabulated values will limit casing distortion to less than one-half the pump vendor's design criterion, and ensure pump shaft displacement of less than 250 μm .

These operating limits are generally too small to be applied to nozzle loadings from seismic action. It is rare for equipment suppliers to provide nozzle allowable loads for seismic design that are greater than the standard operating nozzle loadings.

This Practice Note recommends that purchasers specify the required nozzle loadings for rotating equipment under seismic action. At minimum these should be 1.2 x operating loads, but in many cases they can be higher – up to a factor of 2 x operating loads.

Recommendations

RECOMMENDATIONS FOR PURCHASE

This Practice Note recommends that purchasers consider the seismic function of each piece of rotating equipment and include seismic design requirements in the purchase specification.

The seismic design aspects of rotating equipment must include two considerations:

- Equipment Self-Loading
- External Loadings.

EQUIPMENT SELF-LOADING

The piece of equipment itself must withstand the design loadings arising from seismic action. This means that everything from the holding down detail, through the shell or structure itself, and any internal components or structure must all be designed for seismic loadings according to the design code for the equipment.

EXTERNAL LOADINGS

Loadings on any connection points to the equipment (typically nozzles) from piping and/or ducting must not exceed the allowable loads for seismic action.

EXTENT OF SPECIFICATIONS

The extent/complexity of the seismic design requirements needs to be related to the size/cost/importance of the rotating equipment. For example a 10 kW pump for cold water at 6 bar does not warrant a highly complex technical specification, whereas a 750 kW 200 bar boiler feedwater pump does.

OPERATING VERSUS OCCASIONAL

It is essential that the technical specification clearly state the requirements for the sustained or operating condition for the equipment and the requirements for the occasional condition. It must also clearly state whether the loading being specified as occasional is Operating + Occasional, or just the difference between Operating and Operating + Occasional.

COMMERCIAL ARRANGEMENTS

Rotating equipment suppliers are commonly very protective of their equipment. They do not wish it to fail and be the subject of a warranty claim. They will therefore limit the nozzle loadings to the minimum possible values for which they can get agreement.

It is common for rotating equipment suppliers to not have a good understanding of seismic design requirements and concepts.

It is therefore essential that clear seismic design requirements are communicated at the earliest enquiry for equipment. It is also recommended that the satisfactory agreement of seismic design issues be tied into commercial arrangements for the equipment purchase.

STANDARDS AND REFERENCE DOCUMENTS

The following standards and documents provide information relevant to the design of rotating equipment.

- ANSI/API, 2003. *ANSI/API 610 Centrifugal pumps for petroleum, petrochemical and natural gas industries 10th Edition* (ISO 13709: 2003 identical)
- American Society of Mechanical Engineers, *B73.1 Specification for Horizontal End-Suction Centrifugal Pumps for Chemical Process*. New York: ASME
- American National Standards Institute/Hydraulic Institute. *ANSI/HI 9.6.2 Rotodynamic Pumps for Assessment of Applied Nozzle Loads*
- National Electrical Manufacturers Association, 1991. *SM-23 Steam Turbines for Mechanical Drive Service*. Rosslyn, Virginia: NEMA
- National Electrical Manufacturers Association, *SM-24 Land-Based Steam Turbine Generator Sets 0 to 33,000 kW*. Rosslyn, Virginia: NEMA
- American Petroleum Institute, 2002. *API 617 Centrifugal Compressors for General Refinery Service*. API
- American Lifelines Alliance, Oct 2004. *Guide for Seismic Evaluation of Active Mechanical Equipment*
- US Nuclear Regulatory Commission. *Regulatory Guide 1.100, Seismic Qualification of Electrical and Active Mechanical Equipment and Functional Qualification of Active Mechanical Equipment for Nuclear Power Plants*
- K.R.Rao. *Companion Guide to the ASME Boiler & Pressure Vessel Code, Chapter 13 Nuclear Pumps and Valves*. New York: ASME
- LC Peng and AO Medellin. *Rethinking the Allowable Pipe Load on Rotating Equipment Nozzles*. Houston, Texas: The MW Kellogg Company
- LC Peng and TL Peng. *Pipe Stress Engineering, Chapter 9 Interface with Rotating Equipment*. ASME Press
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APPENDIX H

SEISMIC COEFFICIENTS FOR THE DESIGN OF PRESSURE EQUIPMENT

Recommended minimum seismic coefficients for pressure equipment

A table of minimum seismic coefficients for pressure equipment, Table H1, has been developed to replace the table included in NZS 1200 Appendix I.

The table provides the recommended minimum seismic coefficient as a variable that is dependent on the Hazard Factor Z for the site within New Zealand (varies between $Z = 0.10$ and $Z = 0.60$). The return period of the design earthquake has been taken as 250 years.

Use of site specific seismic hazard studies

In the past, for pressure vessels and pressure piping the minimum values from NZS 1200 were disregarded if a site specific seismic hazard study had been undertaken as per ACPPE (2001) Clauses 5.4.4 (10) or 6.4.3 (2) and then used in the determination of the seismic coefficients.

It is this Practice Note's recommendation that this continues to be the case and is extended to all pressure equipment. The site specific seismic hazard study shall include appropriate geological and geotechnical investigations as per accepted guidelines such as the New Zealand Geotechnical Society's "Guidelines for Geotechnical Earthquake Engineering Practice in New Zealand", July 2010.

Table H1 development

Table H1 has been developed to provide a minimum seismic coefficient that would be reviewed against that calculated from NZS 1170.5 using this Practice Note as guidance.

$C(T_1)$ was determined from Equation 3.1(1) of NZS 1170.5 taking $N(T, D) = 1.0$. $C_d(0.15)$ was determined from NZS 1170.5 Eqn 5.1(1) using a Site Soil Class of E, a structural ductility factor $\mu = 1.25$, 5% damping and $T_1 = 0.15$ secs. The working strength method minimum design coefficient for pressure equipment was then taken as $0.8 \cdot C_d(0.15)$.

The values for the Hazard Factor Z were varied to calculate the values shown.

Table H1: Minimum Design Accelerations – Working Stress Based Design ⁽³⁾

Annual Probability of Exceedance		1/250
Return Period Factor, R_u		0.75
Hazard Factor $Z^{(1)}$	0.10	0.30
	0.15	0.30
	0.20	0.30
	0.25	0.35
	0.30	0.40
	0.35	0.50
	0.40	0.55
	0.45	0.60
	0.50	0.70
	0.55	0.75
	0.60	0.80

$0.8 \cdot C_d(0.15)$ is limited to a minimum of 0.30 for APE = 1/250

(1) Refer to Section 3 NZS 1170.5

(2) The values given in this table are minimum values – greater values may be applicable by using inputs specific to the site and using the methods given in NZS 1170.5

(3) If $C(T_1)$ can be determined from a site specific seismic hazard study, the pressure equipment may be designed with seismic coefficients derived from that study.

(4) Values have been rounded to the nearest 0.05

Table H2 development

Table H2 was developed to show the effect of locating pressure equipment at an elevated height above grade. Thus the supporting structure has a major influence on the earthquake accelerations imparted into the pressure equipment.

The Site Subsoil Class has been taken as C which gives the maximum value for $C_h(O)$ from NZS 1170.5 Table 3.1. $C(O)$ was determined from Equation 3.1(1) of NZS 1170.5 taking $N(T, D) = 1.0$. Section 8, Eqn 8.5(1) of NZS 1170.5 was used to develop values for F_{ph} . To obtain values for F_{ph} , the $C(O)$ determined was amplified taking:

- $h_i/h_n = 1.0$ with $h_n = 6.0$ metres
- $T_p = 0.10$ secs
- $\mu_p = 1.0$ for elastic design
- $C_{Hi} = 2.0$ using Equation 8.3(1)
- $C_i(T_p) = 2.0$ using Equation 8.4(1)
- $C_{ph} = 1.0$ from Table 8.2

F_{ph} was based on 5% ULS damping in the supporting structure.

The working strength method design coefficient for items supported above grade was then taken as $0.8 * F_{ph} / W_p$. F_{ph} being calculated for the ULS assuming a Part Category P.4, hence $R_p = 1.0$.

Table H2: Items Elevated above Ground Level – Typical Design Accelerations – Working Stress Based Design

		Design Working Life (yrs)			
		1	2	3	4
Importance Level	1	≤ 25	100+		
	2		≤ 25	≤ 50	100+
	3		≤ 6 months	≤ 25	≤ 50
	4				≤ 25
Annual Probability of Exceedance		1/50	1/250	1/500	1/1000
Return Period Factor, R_u		0.35	0.75	1.0	1.3
Hazard Factor Z	0.10	0.30	0.32	0.43	0.55
	0.15	0.30	0.48	0.64	0.83
	0.20	0.30	0.64	0.85	1.11
	0.25	0.37	0.80	1.06	1.38
	0.30	0.45	0.96	1.28	1.66
	0.35	0.52	1.12	1.49	1.94
	0.40	0.60	1.28	1.70	2.21
	0.45	0.67	1.44	1.92	2.49
	0.50	0.74	1.60	2.13	2.77
	0.55	0.82	1.76	2.34	2.88
	0.60	0.89	1.92	2.55	2.88

$0.8 * F_{ph}$ is limited to a minimum of 0.30 from Table H1

F_{ph} is limited to a maximum of $3.60 W_p$ and $Z * R_u$ is limited to a maximum of 0.70

(1) The values given in this table are typical values – smaller or greater values may be applicable by using inputs specific to the site and using the methods given in this Practice Note and NZS 1170.5 Section 8.

(2) If $C(O)$ can be determined from a site specific seismic hazard study, the pressure equipment may be designed with seismic coefficients ($0.8 * F_{ph} / W_p$) derived from that study. Refer also to Note 3 Table H1.

APPENDIX I

FAQ

1. Can a vertical vessel be an irregular structure?

Clause 4.5 of NZS 1170.5 deals with the regularity of structures analysed using the seismic actions standard. Structures that are irregular have performed poorly during major earthquakes so the NZS 1170.5 standard now requires a more rigorous analysis to determine the applicable seismic actions to be used in the design of an irregular structure.

For a vertical vessel to be an irregular structure, one of the features listed in NZS 1170.5 clauses 4.5.1, 4.5.2 and 4.5.3 must apply to the vertical vessel as it is operating in the plant. Most vertical vessels will have a structural ductility factor $\mu \leq 1.25$, so clause 4.5.3 will not apply. However:

- A vertical vessel with a necked yielding skirt with $\mu > 1.25$ would need to consider clause 4.5.3 (it would probably also fail the regularity checks in clauses 4.5.1.2, 4.1.5.3 or 4.5.1.4). This configuration can be used when an existing foundation with limited room to increase its overturning or sliding capacity is to take a replacement vertical vessel that is taller and/or heavier than what the original foundation was designed for.
- A vertical vessel that reduces or increases in diameter up its height by more than 30% would fail clause 4.5.1.4 (it would probably also fail the regularity checks in clauses 4.5.1.2 and 4.1.5.3).
- A vertical vessel with a secondary pressure vessel that is supported off the shell externally (ie an eccentric mass) would need to consider clauses 4.5.1.1 and 4.5.2.3.
- A vertical vessel with liquid contents that only part fill the pressure envelope would need to consider clause 4.5.1.1.

Consequences for the seismic analysis if a vertical vessel is found to be irregular:

Clause 6.1.3.1 of NZS 1170.5 gives limitations on the use of the equivalent static method of analysis. At least one of the three criteria listed must be satisfied for an equivalent static analysis to be used as the method to determine the seismic actions on a structure.

An equivalent static analysis can be used as the only means of determining seismic actions when any one of the following apply:

- $H_n < 10$ m or,
- $T_1 < 0.4$ sec or,
- Structure regular and $T_1 < 2.0$ sec

So if a vertical vessel is irregular but short (height < 10 m) or stiff ($T_1 < 0.4$ sec), an equivalent static analysis can be used as the only means of determining its seismic actions.

If a modal response spectrum analysis is required, a simple vertical cantilever model can be developed with joints at the base, top of the skirt, changes of wall thickness, changes of diameter, 3.0m intervals or at enough locations to develop sufficient mode shapes to meet the requirements of NZS 1170.5. Structural software such as Microstran or STAAD or mechanical software such as AutoPIPE or Caesar II can be used to create the model using tubular beam elements and to run a modal response spectrum analysis. Modal response spectrum analysis seismic actions are only accurate at the locations of the joints in the model, so choosing sufficient joints throughout the model is critical to obtaining an efficient shell and skirt design. Alternatively, a more complex finite element analysis modal analysis with plate or shell elements could be undertaken to develop the seismic induced stresses in the vessel.

The example of a vertical vessel given in Appendix E, Example E1 was deliberately created with the assumption that the vessel is irregular to show the consequences of the above clauses of NZS 1170.5. The majority of vertical vessels in New Zealand will be short and/or stiff enough to permit an equivalent static analysis as the method to determine seismic actions.

2. Should pipe shoes, vessel supports or a skirt be designed as a structural support or as part of the vessel?

Support structures attached directly to the pressure envelope should be designed as part of the pressure equipment according to the appropriate pressure equipment design standard. It should be noted that the Example E1 does not design the skirt – the calculation only checks whether the thickness of the skirt chosen by the mechanical engineer is suitable for ductile behaviour.

This is essential in determining what structural ductility factor can be used in calculating the seismic loads on the vessel. Mechanical design codes do not often look at ductile behaviour so the use of a standard that does was chosen for the example.

3. Is Table H1 mandatory for all pressure equipment and supports?

Table H1 was intended to provide a minimum standard for new pressure equipment in a similar manner to historic standards, such as the 1981 Ministry of Works document “Seismic Design of Petrochemical Plants” and the Marine Division, MOT minimum seismic requirements, which were later adopted in the post-1990 versions of AS/NZS 1200. Minimum seismic design loads are intended to mitigate the potentially catastrophic effects of very large but rare earthquake events, ie much larger than the design event.

It is not intended to apply in the following situations:

- to a retrospective assessment carried out in accordance with PN19 Appendix D.
- to buildings housing or supporting pressure equipment, beyond the immediate equipment supports (unless the pressure equipment’s weight is more than 20% of the total combined weight of the building).

4. What ductility factor should I use for pressure equipment (mechanical) design?

In most situations, the mechanical engineer will be limited to a ductility factor of 1.0 or 1.25 (nominally ductile) where the equipment is “well-conditioned” (eg distributed piping systems constructed from ductile materials). This is because the working stress design standards require linear elastic behaviour (ie allowable stress is limited to below yield).

When an economic solution cannot be found, the designer may want to consider ductile behaviour in their design. The following procedure would be one example of how higher ductility factors could be used:

1. A piping system attached to equipment with very large seismic displacements cannot feasibly be designed to a normal working stress code and the designer elects to design plastic hinge(s) to absorb the displacements.
2. The designer applies a seismic load calculated with $\mu = 1.0$ and designs the system such that stresses over yield are limited to the plastic hinge component. Note that the working stress seismic reduction factor cannot be applied to the seismic load calculation. The locations of the chosen plastic hinges need to be selected with care to ensure that sufficient energy absorption can occur close to where the high seismic actions in the piping system are occurring.
3. The designer isolates the plastic hinge component(s) and uses FEA software to prove compliance with an advanced analysis standard such as ASME BPVC Sec VIII, Div. 2, limit load or elastic-plastic strain hardening methods. It should also be confirmed that displacements are not so large that the load distribution has significantly changed.
4. The designer then recalculates the seismic load with $\mu \leq 2.0$ (for SLS2 & ULS) and applies that to the rest of the piping system.
5. The designer then ensures that the rest of the system is compliant in the linear elastic analysis and support structures can be designed with loads generated from this model.

It should be noted that the pressure equipment system is proven to be compliant in step 3 above. Steps 4 and 5 are only necessary to design a more cost-effective support structure.

5. What design working life should I use for seismic actions?

PN19 5.1.6 recommends that the minimum design working life that should be used is 25 years for determining the wind, snow, ice and seismic actions. Refer to PN19 Table 1. The designer and owner may choose to apply a longer design working life to minimise the risk of failure in a larger-than-design-level earthquake, particularly where failure would be catastrophic.

For pressure vessels designed to AS 1210:2010, Appendix J2 outlines what the design working life, according to AS/NZS 1170.0, should be. It also provides a relationship between the Importance Level derived from AS/NZS 1170.0 and the Hazard Level from AS 4343. This relationship is reflected in PN19 Table 2.

The design working life is intended only for the purpose of performing calculations according to the AS 1170 series and is not necessarily related to expected service life of the vessel/pressure equipment, ie it should be considered a performance factor not a service life.

6. What happens when my seismic design life elapses?

Unlike creep or fatigue life, which are dependent on operational history, for seismic resilience only the current condition is relevant. The concept of a seismic design life should therefore be considered as a performance requirement rather than a traditional life or expiry date, ie designers, owners and controllers should ensure that the seismic design life is of sufficient duration to present a suitable risk level for potentially affected persons in an earthquake scenario.

When the seismic life has expired, it is therefore recommended that:

1. For equipment understood to be designed and/or compliant with NZS 1170.5 with a seismic design life of ≥ 25 years – this can be considered an indefinite life and therefore no reassessments are required unless the published seismic risk factor has significantly increased since installation.
2. For equipment with shorter seismic design lives or installed prior to 2005 – the equipment should be reassessed to latest loading standards using PN19 Appendix D (or other suitable risk-based approach). For pre-2005-designed equipment where the seismic design life has not been stated, it can be assumed to be 50 years.
3. For equipment already assessed in accordance with PN19 Appendix D (or some other suitable risk-based approach) – the life can be considered indefinite and therefore no further reassessments are required unless the published seismic risk factor has increased significantly since assessment.

7. Can I use the 0.8 working stress factor for SLS1 and SLS2?

The working stress reduction factor can only be used for a ULS analysis where it is being used in a working stress-based assessment. For most mechanical design, the only examples where it should not be used for ULS assessments are where a FEA limit load or elastic-plastic strain hardening analysis is used.

SLS1 and SLS2 are deflection- and lack-of-damage-based limit states and stress checks are usually not required. Young's Modulus and moments of inertia are not dependent on levels of shaking, whereas damping is. Therefore the 0.8 factor is not valid for a stress or deflection check using working stress design methods for SLS1 and SLS2 assessments.

8. Can I use the 0.8 working stress factor in ASME B31.E and/or FEA analyses?

The standard B31.E was developed for use in conjunction with ASCE 7. ASCE 7 uses a working stress factor of 0.7 in conjunction with seismic loads based on an earthquake with a 2500-year return period. It is recommended, however, that a factor of 0.8 is used with loads calculated according to NZS 1170.5 and that careful consideration be given to the earthquake return period to ensure that the levels of shaking will develop the amount of ductility inherent in the B31.E equations.

Refer to FAQ 7 for guidance on FEA analyses.

9. If I am designing to AS1210, AS1228 or AS4041 can I use the working stress factors listed in those standards?

Yes, provided that the design complies in its entirety to that standard.

10. Is Table 2 mandatory for selecting Importance Level?

No, Table 2 is provided for guidance only. The Importance Level should be selected on a case-by-case basis considering the specific risks and performance requirements of the project and always with the acceptance of the pressure equipment controller. In most circumstances, the designer will be the best person to understand and select the importance level and therefore, where the issues are complex, the designer should explicitly explain the performance criteria and also put it in context that a layperson can understand, eg in terms of comparison with related equipment/infrastructure's performance.

A significant number of plant owners will have already selected an Importance Level for their plant and to go to a lower IL for these facilities would require sign off from the owner's technical authority. Often insurance considerations come into the owner's choice of IL.

Also, if the location of the pressure equipment is deemed to be an Upper Tier Major Hazard Facility, there will be considerations that will affect the Hazard Level of the item and owner-decreed Importance Levels.

11. Is there any circumstance where the Importance Level can be reduced?

Following on from FAQ 10, provided that the minimum criteria of AS/NZS 1170.0 Clause 3.3 are applied, the Importance Level could be reduced from the recommended values in PN19 Table 2 in the following circumstances:

1. Where it can be demonstrated that the pressure equipment controller and all affected parties have understood and accepted the performance level, and;
2. The risk posed is congruous with definitions in AS/NZS 1170.0 Tables 3.1 and 3.2

12. Is there more guidance about when a structural performance factor of 1.0 should be applied?

The structural performance factor captures effects that might improve or reduce performance, such as redundant load paths, strain hardening etc. Ductile pressure equipment with many load paths such as distributed piping systems may have a $S_p \geq 0.7$, whereas equipment that is brittle or has no redundant load paths (such as vessels on legs), should have $S_p = 1.0$.

13. Can the occasional load factor (eg 1.33 for ASME B31.3 & 1.2 for ASME B31.1) be used for all seismic limit state loads?

Yes, if it meets the application criteria of the design standard.

14. Do I need to do an SLS1 or SLS2 assessment on my pressure equipment design?

This depends on your project-specific seismic performance criteria. However, in most circumstances, achieving compliance with the working stress design standard for ULS loads will inherently mean compliance for SLS loads – and therefore analysis of the SLS load cases is unnecessary. See FAQ 7.

However, there are some circumstances where SLS1 and/or SLS2 would need to be assessed.

1. When using higher ductility values for ULS load calculation as per FAQ 4.
2. Generally, when ULS is being assessed using limit load or elastic plastic FEA methods.
3. When limited loss of containment would be acceptable through flange connections under your ULS performance criteria (but not in SLS performance criteria).
4. Where there are other (eg performance type) limitations on deflection or load under normal operation that don't apply under your ULS performance criteria; eg piping connected to rotating equipment, vessels with attached platforms.
5. When the plant owner's technical requirements ask for SLS1 and SLS2 assessments; typically these can be called for in plants with an Importance Level of 4.

15. Does the 8% force redistribution to the top of the structure, as specified in NZS 1170.5 clause 6.2.1.3, apply to vertical vessels?

Yes.

16. Does Appendix D cover repairs?

Repair designs are rarely exactly like for like, and the definition of when a repair becomes an alteration is not well defined. The principles of Appendix D were intended to cover repairs so therefore minor repair designs that do not exceed one of the trigger points listed in section D5 should not require a seismic reassessment. Conversely, when a repair design does trigger a seismic reassessment, section D6.2 can be applied to reduce the loads if required.

17. What is the assessment process when the original design code is superseded?

The current design standard should be applied. See Appendix D section 7.1 for more guidance. This also applies to repair designs.

18. Is there a list of code combinations for use with ASME B31.E?

For use in AutoPIPE or Caesar II.

Where:

E1 and E2 are seismic static loads in +X and -X directions

E3 and E4 are seismic static loads in +Y and -Y directions (vertical)

E5 and E6 are seismic static loads in +Z and -Z directions

S1 and S2 are SAM loads in +X and -X directions

S3 and S4 are SAM loads in +Y and -Y directions (vertical)

S5 and S6 are SAM loads in +Z and -Z directions

NAME	LOAD CASE 1	LOAD CASE 2	LOAD CASE 3	LOAD CASE 4	LOAD CATEGORY	OCCASIONAL K FACTOR	SUMMATION METHOD	DESCRIPTION	CODE CASE
Max EX	E1	E2			Occasional	–	ABSMAX	Max Seismic X Dir	N
Max EY	E3	E4			Occasional	–	ABSMAX	Max Seismic Y Dir	N
Max EZ	E5	E6			Occasional	–	ABSMAX	Max Seismic Z Dir	N
Max SX	S1	S2			Occasional	–	ABSMAX	Max SAM X Dir	N
Max SY	S3	S4			Occasional	–	ABSMAX	Max SAM Y Dir	N
Max SZ	S5	S6			Occasional	–	ABSMAX	Max SAM Z Dir	N
SRSS XY	Max EX	Max EY	Max SX	Max SY	Occasional	–	SRSS	Combined XY	N
SRSS ZY	Max EZ	Max EY	Max SZ	Max SY	Occasional	–	SRSS	Combined ZY	N
ENV	SRSS XY	SRSS ZY			Occasional	–	Max	Seismic Envelope	N
Sus + Env	Sus	ENV			Occasional	2.4 ¹	Sum	Sustained + Seismic Envelope	Y

The default code combinations relating to seismic and SAM loads generated in AutoPIPE can be deleted and replaced with the above.

Note that there are other restrictions on the use of B31.E to be comparable with the ASCE 7 methodology including:

- The minimum return period shall be 1/1000
- The maximum ductility factor shall be 1.25
- Part spectral shape coefficient $C_i(T_p) = 2.0$
- Materials must be ductile
- Guide gaps are limited to 12mm (or 50mm if a load impact factor of 2 is used in the support design)
- Shear stress at SAM locations must be manually calculated¹

¹ For high-strength materials and/or high temperatures, the allowable stress may be lower than 2.4S, ie min [2.4S; 1.5SY; 408 MPa.

19. How do I certify new pressure equipment supports that are attached to an existing structure?

This is a complex issue and there are several scenarios:

- a) I am installing new pressure equipment (new design) on existing foundations or structures:
- The certification of pressure equipment supports is a Design Statement as per Appendix F = DS1, (PS1 equivalent). This only needs to cover the primary supporting structures, ie what is directly connected to the equipment if the weight of the new pressure equipment is less than 20% of the weight of the structure supporting the new pressure equipment.
 - Existing foundations and structures beyond the primary supports are deemed to be the responsibility of the owner but may be covered by the Building Code.
 - If the weight of the new pressure equipment is more than 20% of the existing structure's weight that is to support the new pressure equipment, a new Design Statement DS1 will need to cover the primary supports, the rest of the load path to the foundations and the foundations.
 - Where new primary support attachments may be connected to structures that are greater than 34% NBS (New Building Standard) but less than 100% NBS, then the DS1 for the new primary supports may state that they are the same, lower level of NBS (with explanatory notes) provided that it is accompanied by a statement from the pressure equipment controller that accepts the lower performance risk. New components should in and of themselves be designed for 100% NBS.
 - Where the primary support attachments may be connected to structures that are less than 34% NBS, the said structures shall be upgraded. The level of seismic performance upgrade is to be as far as reasonably practicable above 34% NBS.
- b) I am replacing pressure equipment, like for like, on an existing foundation or support structure:
- Provided that the risk of a failure has not increased (eg increased load or moments), there is no need to upgrade or replace existing foundations or support structures unless the system is deemed to be earthquake prone (ie less than 34% NBS).
- c) I am upgrading or replacing the foundations or supports for existing pressure equipment.
- Upgrading of supports and foundations for existing pressure equipment should try and achieve 100% NBS; however, you may take a "so far as reasonably practical" approach where this is not economic to achieve. These should be covered by a new DS1 and also accompanied by a statement from the pressure equipment controller that accepts the lower performance risk where applicable.

20. What happens if I am replacing a pressure equipment on an existing foundation or support structure?

See FAQ 19 b) above.

21. If my building/support is very stiff do I need to use the parts method?

The parts method (NZS 1170.5 Section 8) includes a simplistic approximation of the amplification effect of flexible structures in an earthquake, hence the load applied to the part will be amplified somewhat by ground level shaking. For a structure to have little-to-no amplification effect in a typical earthquake (ie be classified as rigid), the fundamental period in the direction considered would need to be less than 0.06s. Pressure equipment systems that have supports with a fundamental period of <0.06 s may therefore use loads calculated as base shear load (as per NZS 1170.5 section 5) using the fundamental period of the equipment system.

22. Does clause D5(b) contradict clause D6.2?

The intention of Appendix D, section 5 is to define when an alteration triggers a seismic assessment, ie to allow very minor alterations without triggering a full reassessment. The intention of section 6.2 is that if you cannot achieve 100% NBS, then you may use lower loads; however, you shouldn't be systematically making the system weaker.

The wording in section 6.2 should read "... provided the overall resultant seismic strength is not more than 10% lower than prior to the alteration."

In practice, when you need to make such an alteration, you may need to retrofit additional strengthening to ensure this is achieved.

23. Is there more guidance on when and how to assess vertical actions?

There is very little guidance in published standards. However, it is the recommendation of PN19 that pressure equipment systems with sensitivity to vertical accelerations should have both upward and downward vertical actions included in the analysis and that these actions be combined with horizontal actions using the SRSS method, unless a site specific study shows that vertical actions are not expected (see below for further commentary).

Examples of systems sensitive to vertical actions could include:

- Horizontally cantilevered pipe supports.
- Vertical vessels' support and anchorage systems.
- Saddles on horizontal vessels and pipe shoes on large diameter piping.
- Equipment connections with multi-axial allowable load criteria.

If the site has a site-specific probabilistic seismic hazard assessment (PSHA) of recent origin, the report should detail how the uniform hazard spectra for the site is made up. This is usually in a section called "Hazard Deaggregation". It shows the contribution to the seismic hazard from all the potential sources that the study identifies.

Often the greatest contribution comes from either the subduction zones or from potential earthquakes within 50km of the site with a magnitude of less than Mw6.5. Thus the proximity of these sources determines whether the peak vertical earthquake for the site should be combined with the peak horizontal earthquake.

It is likely to be conservative or appropriate to combine the two peaks in the one analysis if the agreed basis of design includes the need to include vertical earthquake effects.

NZS 1170.5: 2004 Clause 3.2 outlines how to develop a site's hazard spectra for vertical earthquake loadings. Clause 8.5.2 details how the vertical earthquake actions on a part can be calculated using both Clauses 3.2 and 5.4 for the period of the system supporting the part. Walls, columns and slabs/foundations at grade are normally stiff in the vertical direction whereas elevated slabs, flexible beams and cantilevers are likely to have a vertical period > 0.06 secs and would not be considered rigid.

This FAQ will be updated when more guidance is available for New Zealand.

24. I have Hazard Level D and E piping; do I need a Design Statement (PS1 equivalent) for their supports? What if the piping or equipment is Hazard Level C or B? Does the requirement change if the piping is only temporary?

The Approved Code of Practice for Pressure Equipment makes no distinction for certifying pressure equipment supports based on hazard levels. In theory, all supports for pressure equipment need to be certified by a CPEng. In practice, certification of supports is undertaken on a risk-based approach where they are Hazard Level E. If piping or equipment is Hazard Levels A, B, C or D, it needs a design statement before being signed off by the authorised inspection body under the PECPR Regs.

Hazard Levels are a coarse method of risk assessment and it is recommended that where there is risk of serious injury or fatality, then certification of both pressure equipment and supports is provided.

APPENDIX J

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DISCLAIMER

While the authors have made every effort to present a carefully considered Practice Note based on their own professional practice, as well as consultation with the wider industry and the sponsors, they accept that what constitutes good practice may alter over time following changes in knowledge, technology and legislation. The authors also acknowledge that differing interpretations of relevant legislation and regulations are possible and that each practitioner will need to confirm requirements with the relevant authorities.

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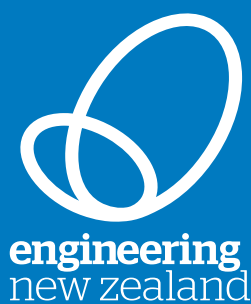
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