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Asset Management Manual

Part 3 - Design Conformance Assessment

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1 Introduction

The main aim in assessing a structure is to ensure that the loads which act on the structure are less than the capacity of the structure. For example, consider the loads acting on a column. The dead loads include the weight of the structure, any supporting equipment and any other permanent loads. Generally dead loads are calculated by measuring the bill of quantities although sometimes dead loads can be measured directly by weighing the structure or its components. For this example, the dead load supported by the column is calculated to be 1000 tonnes.

The capacity of the column is calculated using structural analysis techniques including formulas based on theory and testing and computer modelling techniques such as finite element analysis. For this example, the column capacity is calculated to be 1500 tonnes. The loads on the column are less than the capacity of the column and therefore the column should be “structurally safe”. In fact, we have a “factor of safety” of 1.5.

Unfortunately the world is full of inaccuracies and we cannot calculate the loads or capacity precisely. If we assume that the loads and capacity are not known precisely but have some form of probability distribution then we would give a range and confidence level for the loads and capacity. For example, we may say that there is an 85% probability that the dead load is between 900 tonnes and 1100 tonnes. Similarly there is an 85% probability that the capacity of the column is between 1350 tonnes and 1650 tonnes.

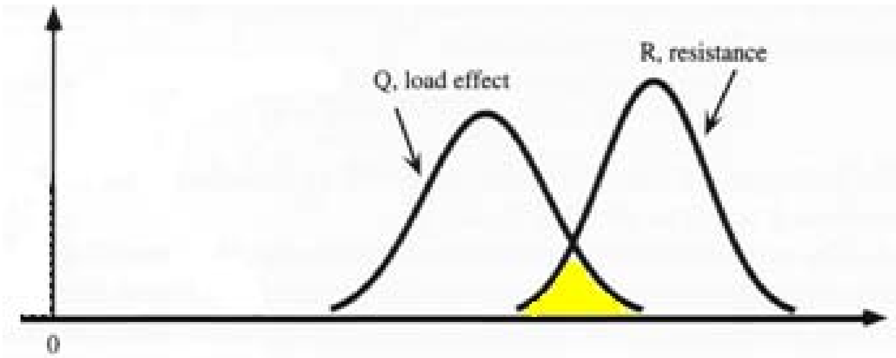


Figure 1 - Probability distribution function of load and resistance

Figure 1 shows this relationship diagrammatically. For our example, the load effect, Q, has an average value of 1000 tonnes and the column capacity (or resistance, R) has an average value of 1500 tonnes. The yellow area shows the overlap of the two probability distribution functions. The overlap area is a measure of the structural reliability or probability of failure.

What does probability of failure mean? From a practical point of view, a probability of failure of 10^{-4}, say, means very little.

In order to illustrate the significance of the probability of failure, let us look at an example such as a weather forecast. If you asked someone what the weather is going to be like tomorrow, they may answer that there is a 30% probability of rain. From a practical point of view, this figure is of little use. It does not say that it will rain for 30% of the time nor does it say it will only rain 30% of a particular intensity. What you really want to know is: do you need to bring an umbrella. The only way to know when to bring an umbrella is to set a target of say, 20%. If the probability of rain is greater than 20%, you bring your umbrella; otherwise you leave your umbrella at home. You hope that over an extended period of time you will not get too wet.

Similarly for structural engineering: a target safety level (or reliability index) is set such that over an extended period of time not too many structures will exhibit distress or failure.

Most building codes deal with the design of new structures only and appropriate safety levels are provided within the design codes to ensure a high level of structural safety. Experience shows that when designing new buildings, very little is gained by reducing safety factors in specific situations in order to save money. This means that the safety criteria contained in design codes are intended to conservatively cover all expected situations. Refer to Appendix A for a discussion of the theory associated with limit state design.

However there are a number of important differences between the assessment of existing structures and new design. Some of these differences include:

- the cost between meeting or not meeting certain code requirements can be high, that is, the cost of minor upgrading can be large;
- material properties can be measured, not assumed;
- geometrical dimensions can be measured;
- loads can be measured, or otherwise determined with some accuracy;
- more realistic estimates of live load can be made;
- degree of deterioration can be assessed;
- consideration of likely future environment is very important; and
- lower “factors of safety” can be used because of the greater knowledge of the structure.

Therefore guidelines are required which provide more flexibility for the assessment of existing structures and require more personal judgment by the structural engineer. These guidelines should provide a consistent methodology for all types of existing structures which can be used to:

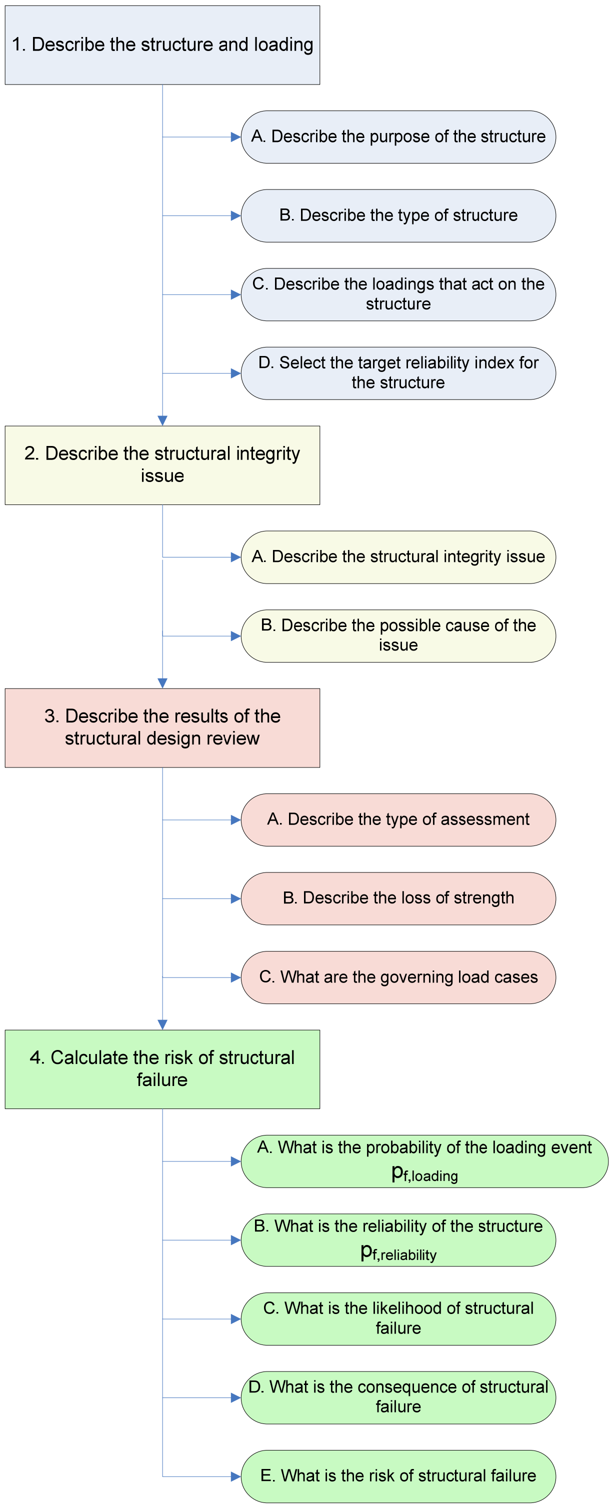
- evaluate the risk of failure (both operational and structural);
- set priorities for remediation work (within a structure, between different structures and between different types of structures); and
- provide a time framework to ensure the most cost-effective remediation options.

Section 2 of this document describes the process which can be used to assess the risk associated with the integrity of existing structures.

Section 3 lists the issues and strength of damaged structures.

2 Structural Integrity Risk Assessment

This section describes the process that can be used to assess the risk associated with the integrity of existing structures.

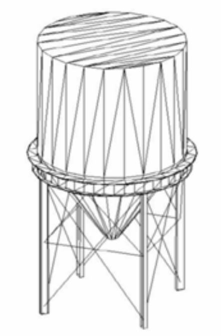


2.1 DESCRIBE THE TYPE OF STRUCTURE AND LOADING

A. Describe the purpose of the structure

Describe the purpose of the structure, what it does and why it is required

B. Select the type of structure

Type of Structure	**Illustration**	**Critical Areas**
Bins		Columns Ring girder connection Stiffeners
Braced structures		Major beams and columns Beam and column connections Braces Crane rail connections
Bridge structures		Major beams and columns Beam and column connections Bearings
Marine structures		Piles Major beams Pile to beam or pile cap connection Bearings

Type of Structure	Illustration	Critical Areas
Materials handling machines		Major beams and columns Connections and pins Ropes and stays Buffers Machine tie-down Anti-collision devices
Sway frame structures		Major beams and columns Beam and column connections Braces Crane rail connections
Trussed structures		Chord and web connections Truss to tower or column connection

C. Describe the loadings that act on the structure

AS5104 classifies loadings according to their variation in time as – permanent actions, variable actions and accidental actions.

Permanent actions are those which are likely to act continuously throughout a given reference period.

Variable actions are those for which the variation in magnitude with time is not negligible.

Accidental actions are those which are unlikely to occur with a significant value on a given structure over a given reference period.

Examples of the types of loadings for each group are presented in the table below.

Permanent actions	Variable actions	Accidental actions
Weight of the structure Weight of equipment Earth pressure Settlement	Live loads due to occupancy Vehicle loads Equipment reactions Conveyor material loads Wind loads Wave loads	Collisions Earthquake Fire Explosion

D. Select the target reliability index for this structure

AS13822, Table C.1 presents a list of the target reliability indices, β, for assessment of existing structures. The reliability index for a structure, β, is usually in the range of 3.0 –

4.0. Most structural design standards correspond to a medium consequence of failure for ultimate limit states.

Limit States	ß	Reference period
Serviceability		
reversible	0,0	remaining working life
irreversible	1,5	remaining working life
Fatigue		
can be inspected	2,3	remaining working life
cannot be inspected	3,1	remaining working life
Ultimate		
very low consequences of failure	2,3	design working life (e.g. 50 years)
low consequences of failure	3,1	design working life (e.g. 50 years)
medium consequences of failure	3,8	design working life (e.g. 50 years)
high consequences of failure	4,3	design working life (e.g. 50 years)

Table C.1: Target reliability indices for assessment of existing structures (ISO 13822)

2.2 DESCRIBE THE STRUCTURAL INTEGRITY ISSUE

A. Describe the issue which causes the integrity of the structure to be considered at risk

Describe the structural integrity issue

B. Describe the cause of the structural integrity issue

Typical causes of structural integrity issues are listed in the table below.

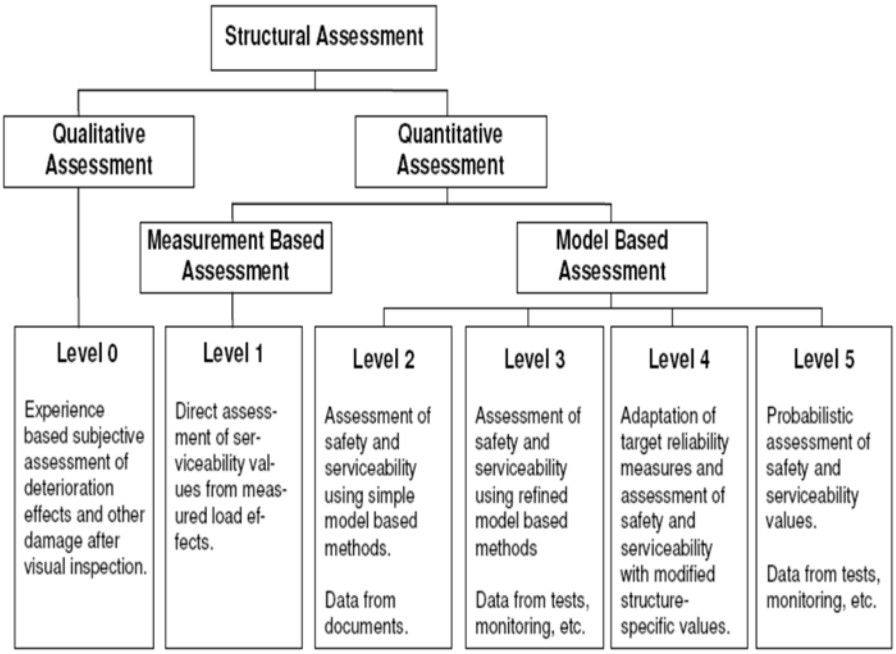
Environmental	**Equipment Overloads**	**Design**	**Human Factors**
Corrosion Fatigue cracking	Vehicle collisions Falling loads	Design fault Inadequate design Poor fabrication/ construction	Unauthorised changes

2.3 DESCRIBE THE RESULTS OF THE STRUCTURAL DESIGN REVIEW

The main aim in assessing a structure is to ensure that the loads (actions) which act on the structure are less than the capacity of the structure (resistance).

A. Describe the type of structural design assessment

Describe the level of the structural assessment from the table below (from *Guideline for the Assessment of Existing Structures* by SAMCO, 2006)



B. Describe the loss of strength (resistance) if applicable

State the percentage loss of strength and describe the reasons for the reduction in strength

C. What are the governing loads types

Typically, the governing loads are one or a combination of the following load types:

- Wind storms
- Excessive material loads – spillage
- Collision due to vehicle impacts
- Mechanical equipment overloads

2.4 CALCULATE THE RISK OF STRUCTURAL FAILURE

A. What is the probability of the loading event – $p_{f,loading}$?

The graphs below provide some guidance for the calculation of the probability of occurrence of some types of loading events. They present indicative return periods for particular load types.

Probability is linked to return period by the following equation:

$$p_{f,loading} = 1 - \left(1 - \frac{1}{T_R}\right)^n$$

where

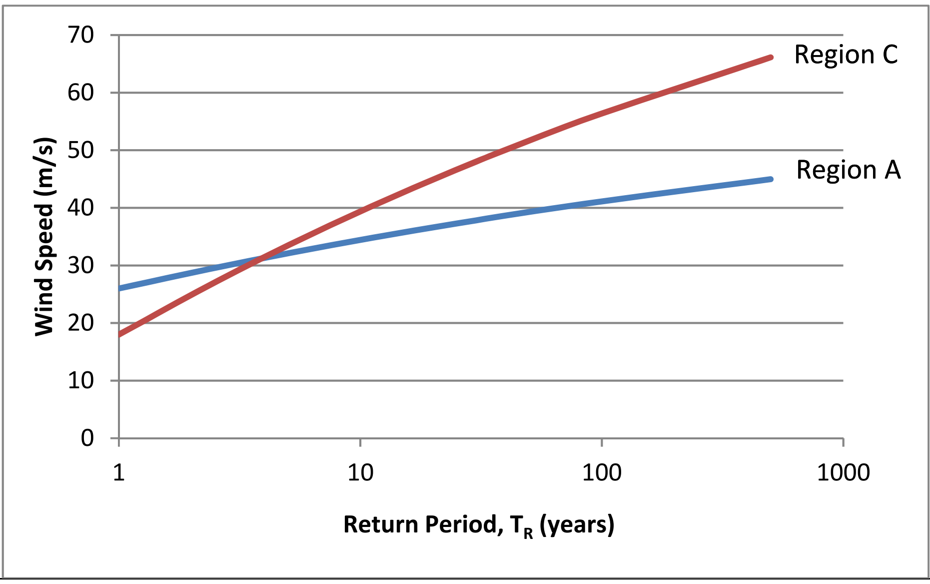
$p_{f,loading}$ is the probability of the loading event occurring
T_R is the return period
n is the reference period

It is assumed that the reference period, n, is one year. The equation is simplified to

$$p_{f,loading} = \frac{1}{T_R}$$

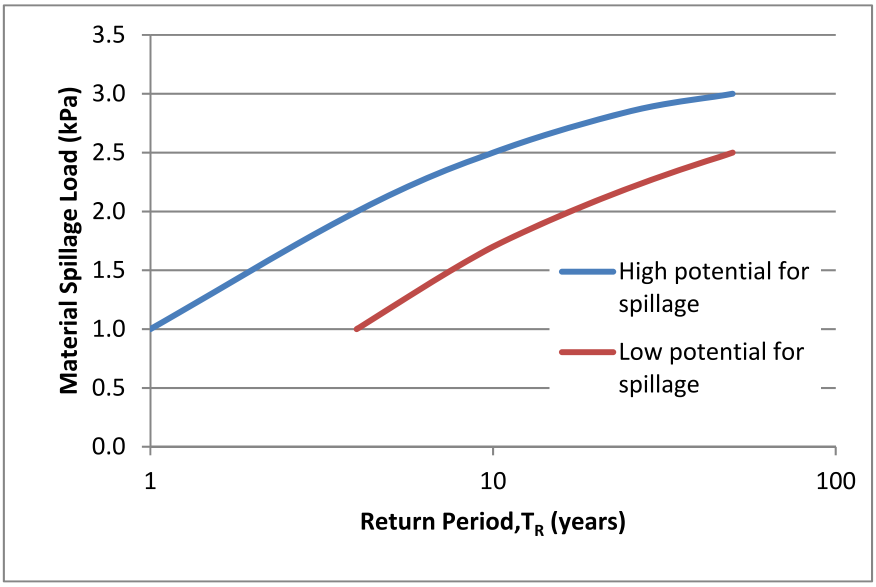
Wind

Wind speeds are taken from AS1170.2 and are presented as ultimate wind speeds. The forces calculated from these wind speeds do not need to be factored.



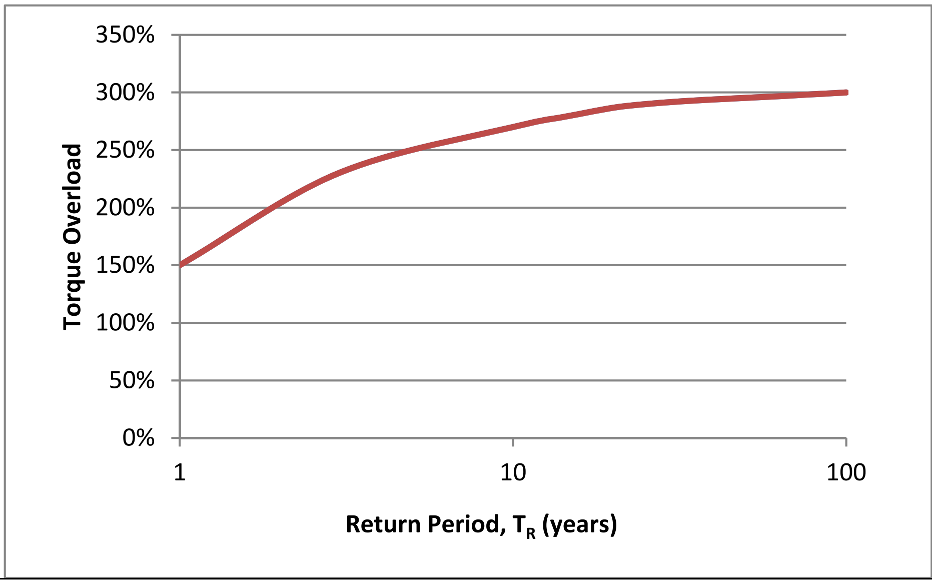
Material Spillage

Spillage is defined as material over the full width and length of the conveyor. This includes spillage on the full length and width of spill trays if they are fitted.



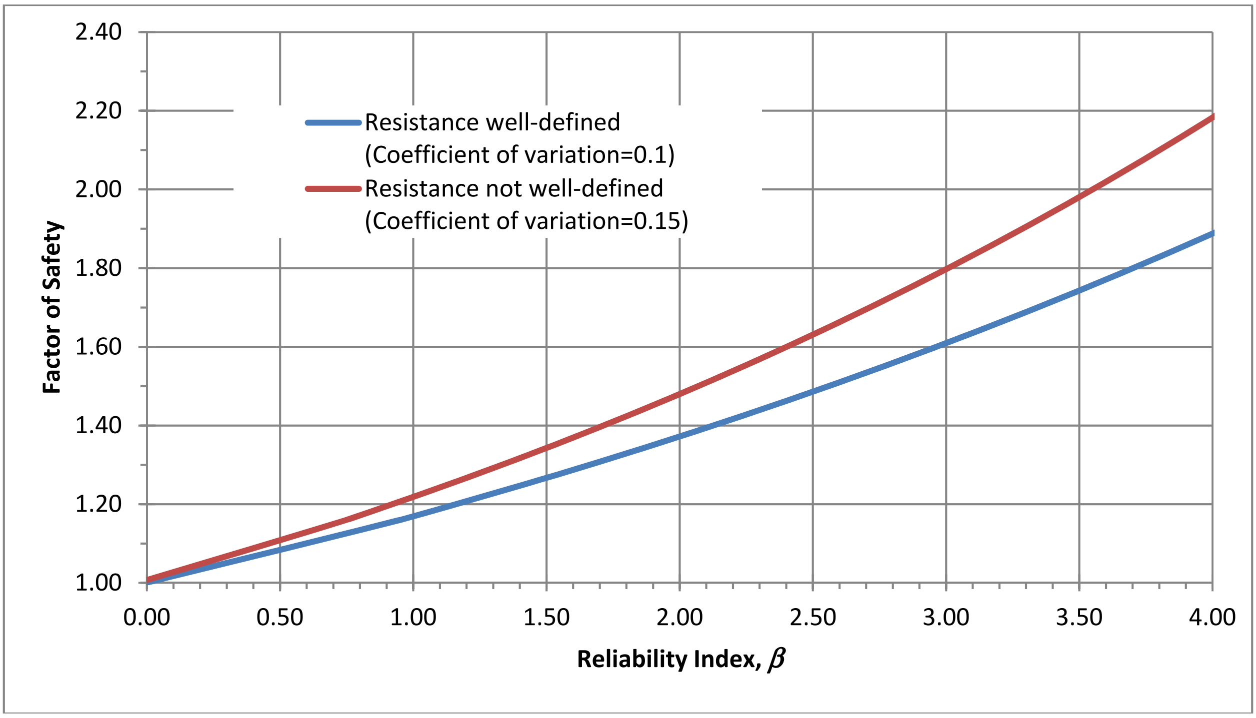
Mechanical Equipment Overload

Mechanical equipment overload is presented as the percentage of motor rated torque. It is assumed that the maximum torque fo the drive is 300% of the rated torque.



B. What is the actual reliability of the structure?

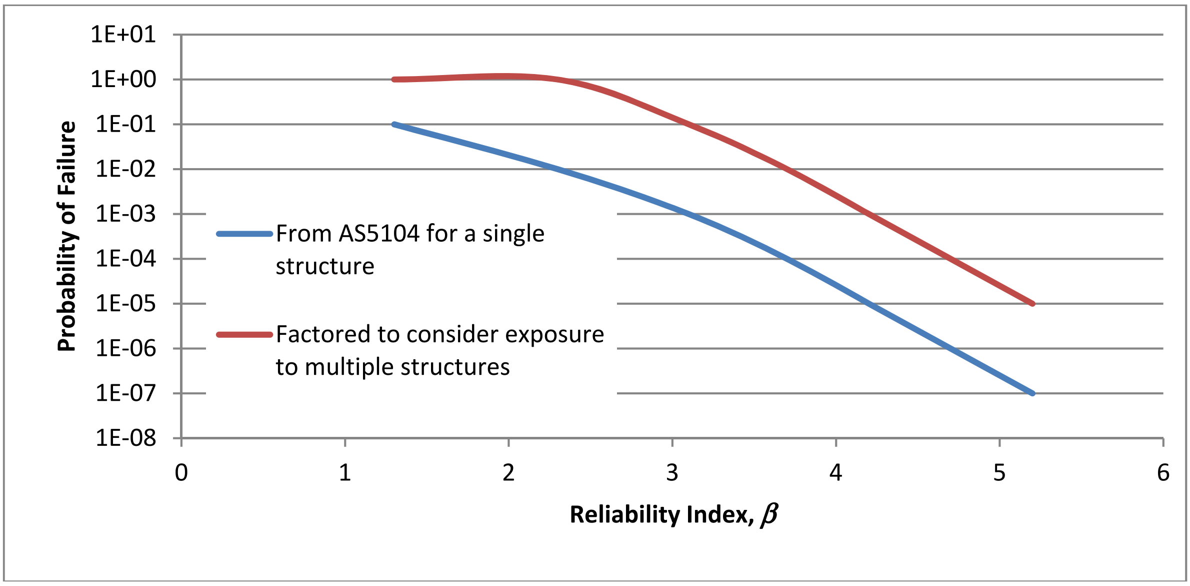
The actual reliability of the structure can be expressed as the Reliability Index, β, and is a function of the loading events (actions) acting on the structure and the strength (resistance) of the structure. As such it is related to the design "factor of safety" as shown in the graph below.



The factor of safety can be established from structural analysis. Typically, for a new structure with little or no corrosion or damage, the resistance is well-defined and the coefficient of variation is equal to 0.1. For older structures, where there is damage such as corrosion, the resistance is less well-defined and the coefficient of variation is equal to 0.15.

The reliability index, β, needs to be expressed as a probability using the graph below. AS5104 gives the relationship between probability, $p_{f,reliability}$, and reliability index, β (AS5104 - Table E.1). The factor $p_{f,reliability}$ is for a single structure and is factored to consider the effects of exposure to multiple structures.

Typically the exposure factor for a large mining site is about 100. The exposure factor is roughly equivalent to the number of structures for a business.



C. What is the likelihood of the structure failing?

The likelihood of structural failure is a function of the probability of the occurrence of the loading event (from Section 4A) times the reliability of the structure (from Section 4B).

$p_f = p_{f,loading} \times p_{f,reliability}$

The return period for the likelihood of failure of the structure is given as:

$T_R = 1 / p_f$

Based on the calculated return period, the likelihood score is evaluated based on the relevant risk rating methodology.

D. What is the consequence of failure?

It is necessary to consider the following:

- Safety
- Business interruption (potential loss of production)
- Replacement cost

Based on the calculated consequences of failure, the consequence score is evaluated based on the relevant risk rating methodology.

E. What is the risk of failure?

The risk of failure is given from the appropriate risk matrix.

3 Strength of Damaged Sections

3.1 CORROSION OF STEEL SECTIONS

Corrosion is defined as the deterioration of materials through chemical or electrochemical attack. It may be caused by:

- wetting and drying of the surface, eg in a splash zone;
- material build-up, eg damp material where there is air and water near the surface; and
- dissimilar metals.

Corrosion can be accelerated in sections which are highly stressed and the rate of deterioration increases with time. As well, if corrosion is present in steel sections which carry cyclic loads, then the fatigue life of these steel sections is shortened. Some of the direct consequences of steel corrosion include:

- a reduction in thickness of section leading to a reduced load carrying capacity of the section and spreading of load to other parts of the structure;
- failure of corroded member and excessive distortion of steel sections in other parts of the structure; and
- loss of contained material.

The options available for the repair of corroded members include:

- Repair. Repair can include isolated repairs such as over plating or inlay plating. These repairs can be expensive and costs should be compared to replacing the whole beam or section. The expense is not so much in the supply of the material but in scaffolding, cutting out the damaged section and weld preparation and welding. The quality of the repair can also be questionable.
- Replacement of the corroded section. This can include splicing a new section in or replacing the whole beam or column. This requires the loads on the beam to be isolated through either propping or supporting from above. It may be very expensive if there are numerous loads to support or if the beam / column is heavily loaded.
- Blast and paint either locally or extensively. Sections blasted should be inspected for the extent of loss of section prior to painting.
- Boxing-in or encapsulation. Boxing-in is useful for corrosion to webs of beam or for sections in shear but may not be beneficial for sections in bending where the flange is the critical component. Encapsulation in concrete may be useful for the base of columns for protecting the hold down bolts and keeping spillage and buildup away. Care should be taken to ensure that the top of the concrete is tapered away web and flanges to prevent a repeating the existing problem in a new location. Reinforcement should also be added to the concrete where necessary.
- Do nothing. This may be a viable option depending on the circumstances.

3.2 CORROSION OF BOLTS

The corrosion of bolts may be caused by temperature variations, wetting and drying of the surface, eg areas in a splash zone, damp material build-up, ie air and water near surface and dissimilar metals. The direct consequences of bolt corrosion include:

- excessive wear on non-aligned mechanical components;
- reduction in the load carrying capacity of joint and spreading of the load to other parts of the section and other parts of the structure;
- excessive distortion of the section;
- failure of the section and excessive distortion or failure of other parts of the structure; and
- increased vibration.

3.3 CRACKING

The causes of steel section cracking include poor detailing/design, excessive loads, cyclic loads, corrosion, poor workmanship, defective materials and changes in the loads. The rate of growth of a crack increases with time. Some of the direct consequences of cracking include:

- reduction in strength of the section;
- spreading of load to other sections; and
- failure of the section and excessive distortion of other sections.

The main causes of steel section cracking include cyclic loads (leading to metal fatigue), poor detailing and design, excessive loads, corrosion, poor workmanship and defective materials. Examples of assets that may be subject to fatigue cracking include bridges, cranes, dump stations, stackers, reclaimers, ship loaders and unloaders. A recent investigation into the failures of over sixty materials handling machines found that about ten percent of the failures can be attributed to fatigue failure. In most cases these failures were unexpected and lead to catastrophic consequences.

Overseas research on steel box girder bridges has found that over 70 percent of fatigue cracking occurs in secondary members which were not part of the original design check. Although not critical initially, this secondary member cracking can propagate into the main structural member. Therefore practices such as welding brackets to suspend cable trays or pipe brackets should be avoided. As well, welding stiffeners to cracked sections can actually make the situation worse and increase the likelihood of failure.

Guidelines for fatigue design of steel structures can be found in Australian Standard AS4100 - Steel Structures. A more detailed approach is found in the British Standard BS7608 - Fatigue Design.

3.4 EXCESSIVE VIBRATION

The causes of excessive vibration include large vibrating loads caused by new equipment or poor running of existing equipment, poor design, deterioration and damage to sections and/or joints and poor foundations.
The direct consequences of excessive vibration include health and safety hazard for operators, excessive wear of machines, cracking of welds, loosening of bolts, and a reduction of the fatigue life due to increased number of loading cycles. It may also result in failure of holding down bolts, grout and foundations and failure of secondary members, welds and connections, for example, handrails, walkways, and platforms.
There are three main sensitivity criteria to be considered for assets. These are:-

- Human sensitivity is a function of the amplitude, frequency and direction of vibration.
- Machine Vibration is a function of the amplitude and frequency of vibration.
- Structural Vibration is a function of the amplitude and frequency of vibration as well as the span of the section and its span to depth ratio.

The causes of excessive vibration include:

- large vibrating loads caused by new equipment or poor running of existing equipment;
- poor design;
- deterioration and damage to sections and/or joints; and
- poor foundations.

The direct consequences of excessive vibration include:

- health and safety hazard for operators;
- excessive wear of machines;
- cracking of welds, loosening of bolts;
- reduction of fatigue life due to increased number of loading cycles;
- failure of holding down bolts, grout and foundations; and
- failure of secondary members, welds and connections, ie handrails, walkways, platforms, etc.

Appendix A

Limit State Concepts

LIMIT STATES CONCEPTS

Modern code design procedures for structures are based on the limit states method, which recognises that many different design requirements must be met concerning both strength and serviceability. According to limit states philosophy, a structure can become unfit for service in many different ways. These include complete or partial failure, excessive deflection or unacceptable cracking. The structure is said to enter a limit state if it cannot perform its intended function. There are many different limit states but they generally fall into the two categories of strength and serviceability.

Strength or ultimate limit states are mostly related to the loss of load-carrying capacity of a member. Some examples of the modes of failure or entry into the strength limit state include:

- Exceeding the moment carrying capacity
- Exceeding the shear capacity
- Crushing of concrete in compression
- Loss of the overall stability
- Fatigue resulting from cyclic loading

The traditional notion of the safety margin is associated with the ultimate limit states. If we use a simple short column as an example, the member resistance will be the axial load capacity R, and the axial load due to the loading (or load effect) will be Q. The performance function for the mode of failure then, can be defined as:

$$g(R,Q) = R - Q$$

When $g(R,Q) = 0$, i.e. when $R = Q$, is the limit state which corresponds to the boundary between acceptable performance and underperformance of the structure. Similarly if $g(R,Q) \geq 0$, the structure is safe (acceptable or desired performance) and if $g(R,Q) < 0$, the structure is not safe (underperformance or undesired performance). In general, the two states of a structure are:

- Safe (Load effect ≤ Resistance)
- Failure (Load effect > Resistance)

The probability of failure, P_f, then in probabilistic terms of the performance functions can be shown mathematically as:

$$P_f = P(R - Q < 0) = P(g(R,Q) < 0)$$

As was shown earlier, both R and Q are random variables and hence each has a probability density function (PDF) as shown in Figure A.1 below. Also shown in Figure A.1, the safety margin (R – Q) also has it's own PDF as it is a random variable quantity. The probability of failure P_f is shown as the shaded region.

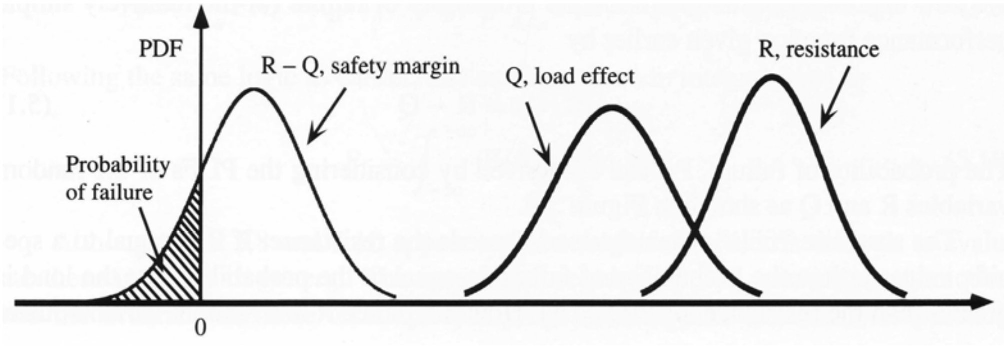


Figure A.1 – PDF's of load, resistance and safety margin

A simple example is to use a simple steel beam in bending with a resistance, R (moment capacity) which is known to be, $R = F_yZ$, where F_y is the yield stress and Z is the plastic section modulus. The performance function g(R,Q) can then be redefined to:

$$g(F_y,Z,Q) = F_yZ - Q$$

The load Q also can be further defined as follows, Q = D + L, where D and L are the contributions to the total demand or load effect by the dead and live loads respectively. Again the performance function can be further refined to give:

$$g(F_y,Z,D,L) = F_yZ - D - L$$

Generally, the limit state function can be a function of many different variable parameters such as material properties, load components, dimensions, and so on. The calculation of the probability of failure, P_f, is extremely difficult, almost impossible. Therefore it is more convenient to ascertain the safety of a structure using the safety (or reliability) index that is discussed in the next section.

RELIABILITY INDEX

As was mentioned in the previous section the safety or reliability index, β can be directly related to the probability of failure of a structural component. A target β value can then be chosen to reflect a probability of failure dependent on the acceptable risk for the mode of failure and is usually in the range of 3.0 – 4.0. Higher target β values are usually chosen for members subject to sudden, brittle failure modes, such as the collapse of a column, to reflect the increased danger to occupants due to lack of warning of failure (lack of ductility). Similarly, lower target β values can be chosen for members such as beams that display ductile failure to reflect less danger. By applying probability and reliability theory for the chosen target β values, the load and resistance factors γ and ϕ can be computed and used in design codes to reflect the chosen safety levels.

If we define a new random variable Z (performance function g(R,Q)) as, Z = R – Q, failure therefore occurs when Z is less than or equal to zero mentioned earlier. If we consider R and Q as having normal independent PDF's then the random variable Z has a normal probability density function, as it is a linear combination of the two normally distributed random variables. It follows that the PDF's are given by:

$$g(r) = \frac{1}{\sigma_R \sqrt{2\Pi}} \exp\left\{ -\frac{1}{2}\left(\frac{r-\mu_R}{\sigma_R} \right)^2 \right\}$$

and $$g(q) = \frac{1}{\sigma_Q \sqrt{2\Pi}} \exp\left\{ -\frac{1}{2}\left(\frac{q-\mu_Q}{\sigma_Q} \right)^2 \right\}$$

and since $\mu_Z = \mu_R - \mu_Q$

and $\sigma_Z^2 = \sigma_R^2 + \sigma_Q^2$

then $$g(z) = \frac{1}{\sigma_z \sqrt{2\Pi}} \exp\left\{ -\frac{1}{2}\left(\frac{z-\mu_Z}{\sigma_Z} \right)^2 \right\}$$

where: μ_x = mean of variable "x"

σ_x = standard deviation of variable "x"

The resultant function can be seen in Figure 2.2. The shaded area indicating the probability of failure is found using the following equation:

$$P_f = \int_{-\infty}^{0} g(z)dz$$

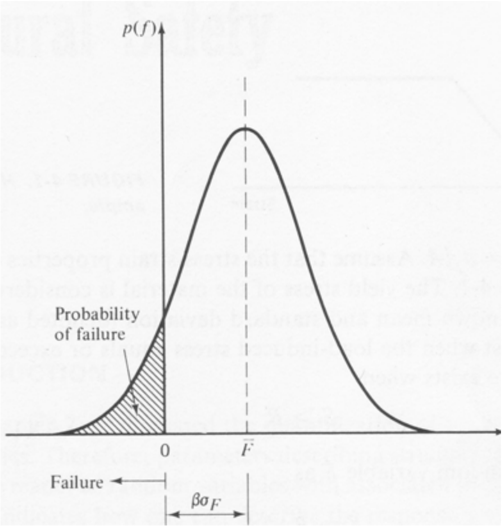


Figure A.2 – Failure probability

As was mentioned in the previous section though, the calculation of P_f is almost impossible, which requires the utilisation of the safety index. The interval between the mean of Z and the value zero is a useful measure of safety. Therefore it can also be seen that the distance from the limit state (Z = 0) to μ_Z is β (safety index) standard deviations. Mathematically:

$$Z = 0 = \mu_Z - \beta\sigma_Z$$

Rearranging and using previous equalities:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma^2{}_R + \sigma^2{}_Q}} = \frac{\mu_Z}{\sigma_Z}$$

The safety index β is then defined as the inverse of the coefficient of variation of Z. If the safety index is known then the probability of failure can be evaluated using the following relationship:

$$P_f = \Phi(-\beta)$$

Therefore values for P_f can be evaluated using standard tables (where Φ represents the normal distribution). Table A.1 shows some typical values.

P_f	β
10^{-1}	1.28
10^{-2}	2.33
10^{-3}	3.09
10^{-4}	3.71
10^{-5}	4.26
10^{-6}	4.75
10^{-7}	5.19
10^{-8}	5.62
10^{-9}	5.99

Table 2.1 – Safety index β and probability of failure P_f

Appendix B

Description of the Corrosion Process

DESCRIPTION OF THE CORROSION PROCESS

Primarily corrosion is an electrochemical process similar to the operation of a car battery. Two important requirements for the electrochemical process to function are oxygen and water. Oxygen is readily available from the air and the main sources of water can be rainfall, dew and high relative humidity. Rainfall may not contribute to the corrosion process as would be expected; although providing a source of water, rainfall may also wash away contaminants on the metal surfaces. Dew and high relative humidity however form a layer of moisture on the metal surface which can significantly influence the corrosion process.

After being exposed to air for a period of time, unpainted steel surfaces initially form a protective thick oxide film (the familiar brown rust) on their surface. This is the initiation of the corrosion process. As long as the film remains intact, further corrosion is prevented. However if some form of chemical attacks the “protective” coating, the steel will continue to corrode.
Steel structures in industrial areas are prone to chemical attack and hence the rate of corrosion in these areas is higher compared to other types of environments. The primary contaminants leading to corrosion include sulphur dioxide and salt from the sea. Sulphur dioxide forms a corrosive acid which increases the rate of corrosion. Salt also influences the rate of corrosion which usually decreases rapidly with increasing distance from the sea.

TYPICAL PATTERNS AND RATES OF CORROSION

Corrosion is defined as the deterioration of materials through chemical or electrochemical attack. Corrosion requires the presence of both oxygen and water. Oxygen is available from the air but levels may be increased, especially in water by agitation such as at the outlet of a drain. Water can be supplied by immersion, washing down, rainfall, dew and high relative humidity. Rainfall may not contribute to the corrosion process as would be expected; although providing a source of water, rainfall may also wash away contaminants on the metal surfaces. Dew and high relative humidity however form a layer of moisture on the metal surface that can significantly influence the corrosion process.
The rate of corrosion depends on what is happening on both a macro and micro climatic level. On a macro scale, the rate of corrosion of an atmosphere can be classified into categories, eg ISO 9223 uses five categories or AS/NZS 2312 uses six. The six classifications within AS/NZS2312 are:

Environment	Description	Corrosion Rate / year.
Mild (ISO Category 2)	All areas remote from the coast, industrial activity and tropics.	Up to 10 μm.
Moderate (ISO category 2)	Light industrial pollution, very light marine influence.	10 to 25 μm.
Tropical (ISO category 2)	Coastal areas of north Queensland, Northern Territory, north-west Western Australia except where directly affected by salt spray	25 to 50 μm
Industrial (ISO category 3-4)	Areas around major industrial complexes such as Port Pirie and Newcastle	25 to 50 μm
Marine (ISO	Areas influenced by coastal salts. The extent depends on such factors	25 to 50 μm

Environment	Description	Corrosion Rate / year.
category 3)	as winds topography and vegetation. It may extend anywhere from 100 m to 10 km inland	
Severe Marine (ISO category 4-5)	Areas off shore and on the coast subject to wave action and heavy salt loads. Generally extends 100 m to 1 km inland.	> 50 μm

In addition to the above macro effects, local effects can significantly affect the rate of corrosion and convert a mild category into an aggressive corrosive environment. These can include:-

- Areas of poor housekeeping where there is a build up of material and rubbish that remains moist for long periods.
- Dripping taps, fire hydrants and wash down hoses. Prolonged periods of wetness can increase the corrosion rate by more than a factor of two.
- Areas that do not allow water to drain or are shaded from sunlight.
- Surfaces that are exposed to contaminants such as coastal salt but are protected from cleansing rain.
- The interior of large hollow sections which can experience very high relative humidity and temperatures due to the ingress of moisture through bolted connections and hatches and often are not inspected due to lack of access.
- Pollutants and other fallout such as industrial chemicals and solvents, fertilisers and other chemicals.
- Industrial processes such as coal wash-plants, fertiliser production, and other industries which produce corrosive materials.
- Dissimilar metals.
- Areas subject to wear and abrasion.
- Highly stressed can corrode more rapidly.

Typical patterns of corrosion which occur in process plants are shown in Figures B.1 and B.2. As can be seen, corrosion occurs where there are surfaces which allow build-up of material, such as the top and bottom flanges of beams, where the material is caught in “pockets”, such as beam to column connections, or where localised or pitting corrosion occurs due to localised splashing of contaminated liquid.

Appendix C

CHS Sections with Local Imperfections

CHS SECTIONS WITH LOCAL IMPERFECTIONS

The paper "Moment-Curvature Relationships for Dented Tubular Sections", Duan L, Loh J T, and Chen W F describe M-P-□ relationships for tubular sections that are dented. Empirically based Moment-Thrust-Curvature Curves are developed and these are utilised in a program, BCDENT to compute axial load deformation curves for dented tubular columns.

Our interest is not in the load deformation curves, but in the capacity and reduction in capacity as dents increase. The paper defines the Moment Capacity of a Dented Tubular Section by means of a design interaction curve;

$$\frac{M_{pud}}{M_{ud}} + \left(\frac{P}{P_{ud}}\right)^{\alpha} = 1$$

where $\alpha = 1.75 - 0.1\frac{dd}{t}\left(1 - 2\frac{\beta}{\pi}\right) \geq 1.0$

The denominator, $M_{ud} = M_u e^{-0.06\frac{dd}{t}cos\beta}$, where M_p is the plastic moment capacity and

$$\frac{M_u}{M_p} = 1.0 \text{ for } 0 < \frac{F_y D}{t} \leq 17240$$

$$\frac{M_u}{M_p} = 1.13 - 1.54\frac{F_y D}{Et} \text{ for } 17240 < \frac{F_y D}{t} \leq 44820$$

$$\frac{M_u}{M_p} = 0.96 - 0.77\frac{F_y D}{Et} \text{ for } 44820 < \frac{F_y D}{t} \leq 137900$$

$$\frac{P_{ud}}{P_u} = e^{-0.08\frac{dd}{t}}$$

Where P_u= mean compression yield strength including the effect of local buckling for an undented tubular section,

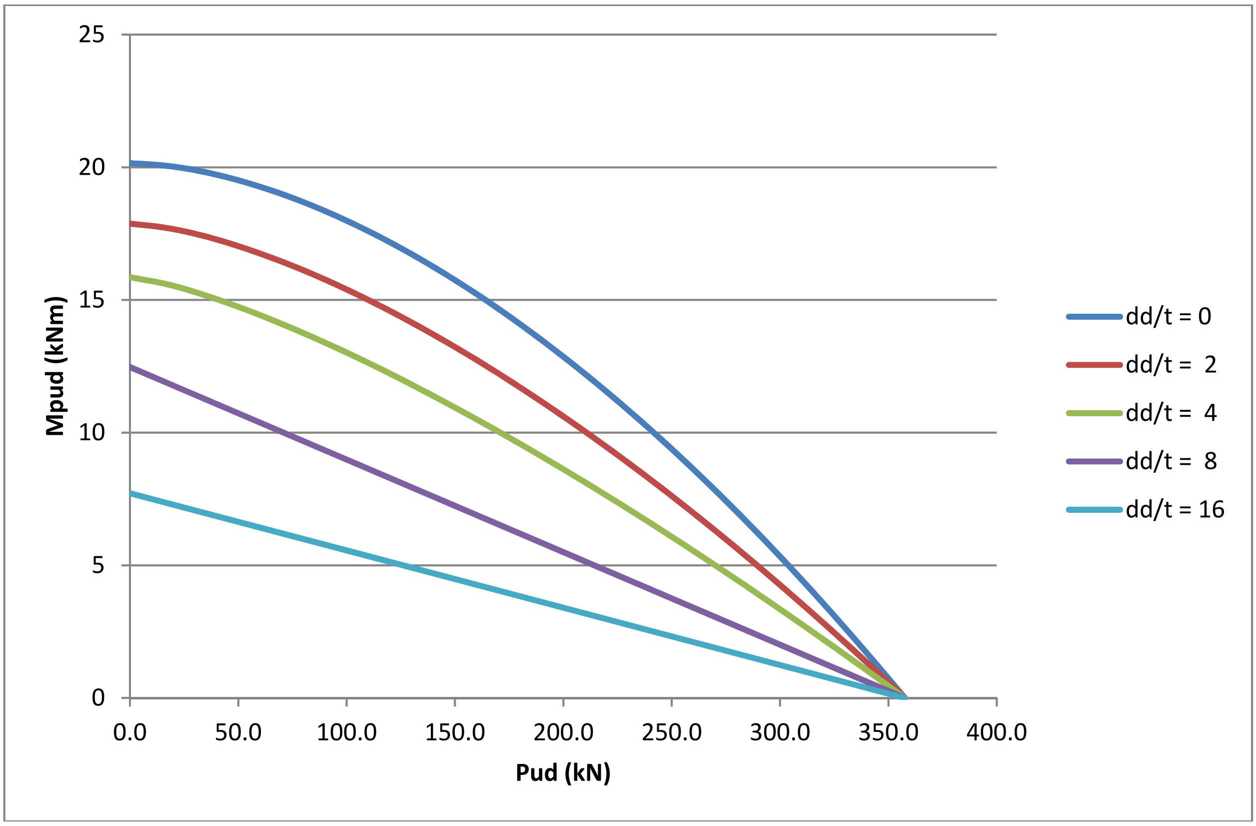
$$\frac{P_u}{P_y} = 1.0 \text{ for } \frac{D}{t} \leq 100, \text{ and}$$

$$\frac{P_u}{P_y} = 1.95 - 0.3\left(D/t\right)^{0.25} \text{ for } \frac{D}{t} > 100$$

And P_y is the axial yield strength of the section.

The empirical relationships for the curvature used in a programme, BCDENT define the moment curvature relationship along the column or beam section being investigated. Axial forces applied to a displaced column increase the moments and deflections and an iterative algorithm that uses Newmark's integration method solves for the final displaced shape. This program takes into account the slenderness of the column and limits the axial loads that can be applied.

The interaction curves alone are capable of predicting the capacity of a dented section, but are unable to do so for a member where the slenderness may dictate its behaviour. The plot below shows the interaction curves for a Grade 350 114.3x4.8 CHS with dd/t ratios up to 16.



An approach is proposed where the theory presented in Timoshenko and Gere's "Theory of Elastic Stability", the AS4100 axial capacity equations and the design interaction curve for dented tubular sections are combined. Firstly, the term P_u is replaced with the limiting axial force, N_c as defined in AS4100. Next, the limiting moment is defined as a function of the axial force and the inherent "out of straightness" ($a = {}^{l_e}/_{450}$) implied in the AS4100 design calculations.

$M_{pud} = \frac{Pa}{1-\frac{P}{P_{cr}}}$ where P is the axial force, and $P_{cr} = \frac{\pi^2 EI}{{l_e}^2}$. Substituting these equations into the interaction curve equations with

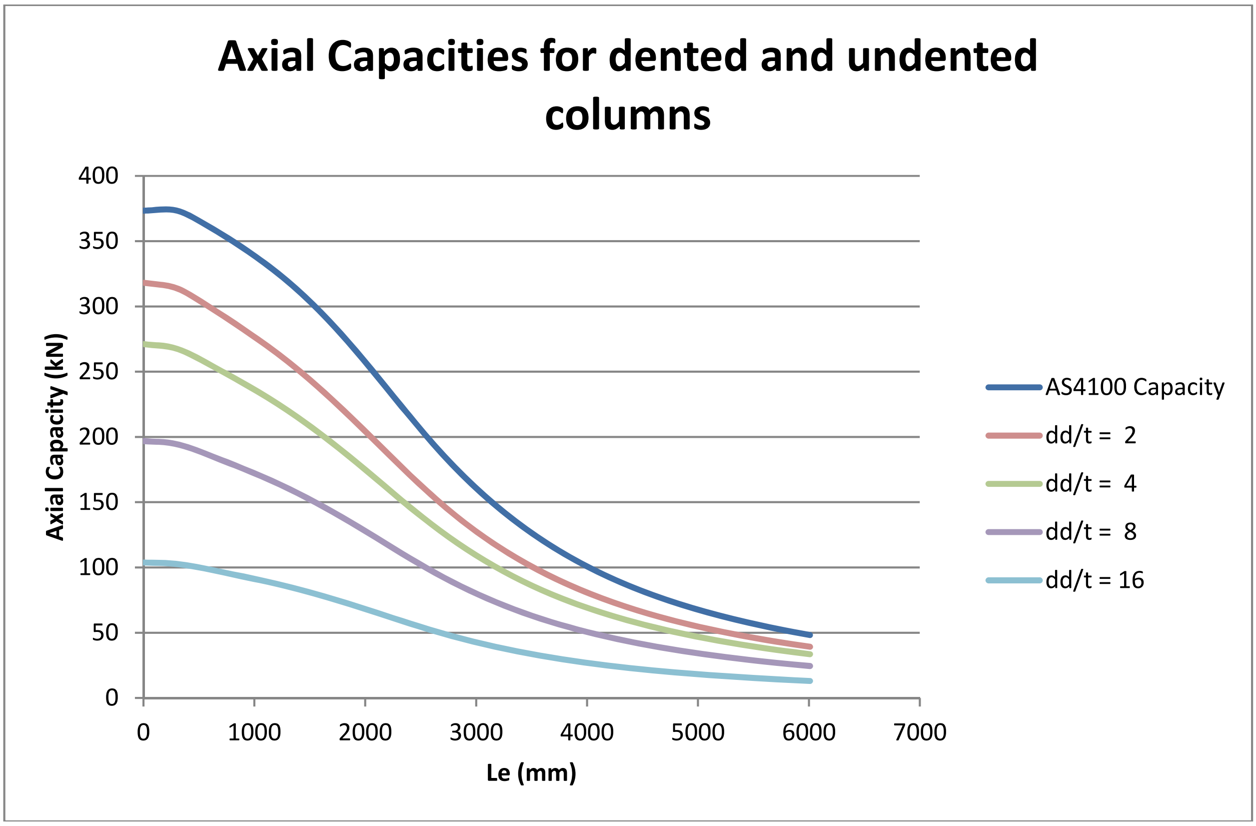
$$P_{ud} = N_c e^{-0.08\frac{dd}{t}}$$

results in a revise interaction equation that accounts for the slenderness of the axial member.

$$\frac{\left(\frac{Pa}{1-\frac{P}{P_{cr}}}\right)}{M_{ud}} + \left(\frac{P}{P_{ud}}\right)^{\alpha} = 1$$

The only unknown in the above equation is the axial force, P and solving for it provides an estimate of the capacity of a dinted section. Substituting dd/t = 0 into these equations shows that the capacities are conservative by at most about 10%.

The chart below is for a Grade 350 114.3x4.8 CHS.


Axial Capacities for dented and undented columns
Axial Capacity (kN)
400
350
300
250
200
150
100
50
0
0
1000
2000
3000
4000
5000
6000
7000
Le (mm)
AS4100 Capacity
dd/t = 2
dd/t = 4
dd/t = 8
dd/t = 16

Appendix D

Columns with Overall Imperfections

COLUMNS WITH OVERALL IMPERFECTIONS

The design codes AS4100 and AS3990 have an implied lack of straightness or eccentricity built into the capacity formulae.

The plots below show the major and minor axis column capacities for a typical UB and UC section. For the major (x) axis the capacities are calculated using three different approaches:

1. AS3990 equations which use the Perry Robertson formula as a basis,
2. AS4100 equations, and
3. The formula presented on page 45 of Timoshenko and Gere's monograph entitled "Theory of Elastic Stability" and assuming an imperfection of $\ell_e/450$

The same is calculated for the minor (y) axis, but assuming an imperfection of $\ell_e/(450 \times c_y/r_y \times r_x/c_x)$ where r and c are the radius of gyration and distance to the extreme fibre respectively. The three approaches are outlined in more detail below.

AS3990 equations

In this case the limiting stress is given by

$$F_{ac} = \frac{1}{\Omega}\left(\frac{F_Y + (\eta + 1)F_{oc}}{2} - \sqrt{\frac{F_Y + (\eta + 1)F_{OC}}{2} - F_Y F_{oc}}\right)$$

l/r = slenderness ratio

Ω = load factor,

$\eta = 0.00003\left(\frac{l}{r}\right)^2$

$F_{oc} = \frac{\pi^2 E}{(l/r)^2}$ = Euler critical stress

AS4100 equations
In this case the limiting axial force is given by

$$N_c = \alpha_c N_s \leq N_s$$

where

N_s = the nominal section capacity, determined in accordance with Clause 6.2

α_c = the member slenderness reduction factor

$$= \xi \left\{ 1 - \sqrt{\left[1 - \left(\frac{90}{\xi \lambda} \right)^2 \right]} \right\}$$

$$\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2}$$

$$\lambda = \lambda_n + \alpha_a \alpha_b$$

$$\eta = 0.00326(\lambda - 13.5) \geq 0$$

$$\lambda_n = \left(\frac{l_e}{r}\right) \sqrt{(k_f)} \sqrt{\left(\frac{f_y}{250}\right)}$$

$$\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\lambda_n + 2050}$$

α_b = the appropriate member section constant given in Table 6.3.3(1) or 6.3.3(2)

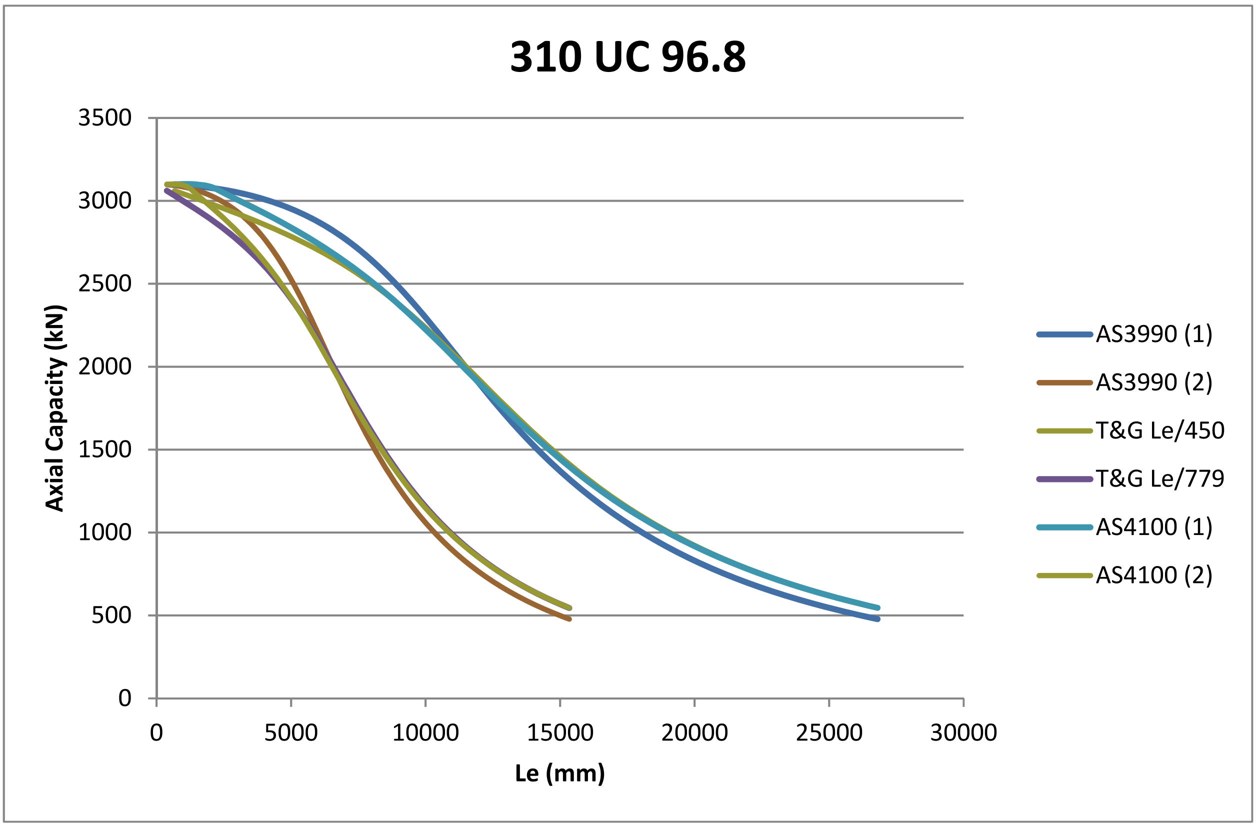
k_f = the form factor determined in accordance with Clause 6.2.2.

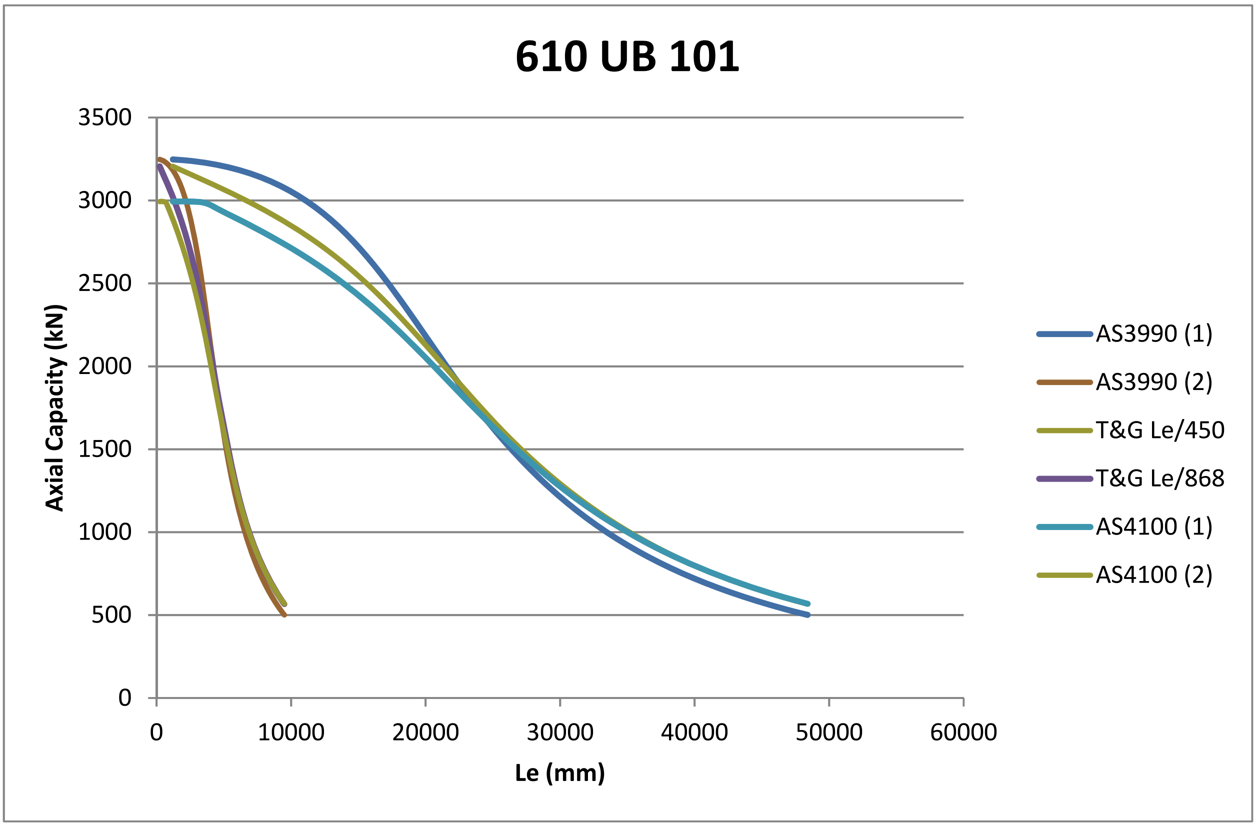
Timoshenko and Gere equations
In this case the axial stress is obtained by solving for F_{ac} the following quadratic equation. In the equation Ω = 1 and the same stress definitions as used in AS3990 are adopted.

$$F_Y = F_{ac}\left(1 + \frac{a}{s}\frac{1}{1 - \frac{F_{ac}}{F_{oc}}}\right)$$

Where $s = \frac{Z}{A} = \frac{I}{Ac} = \frac{r^2}{c}$ and

$a =$ magnitude of the imperfection.





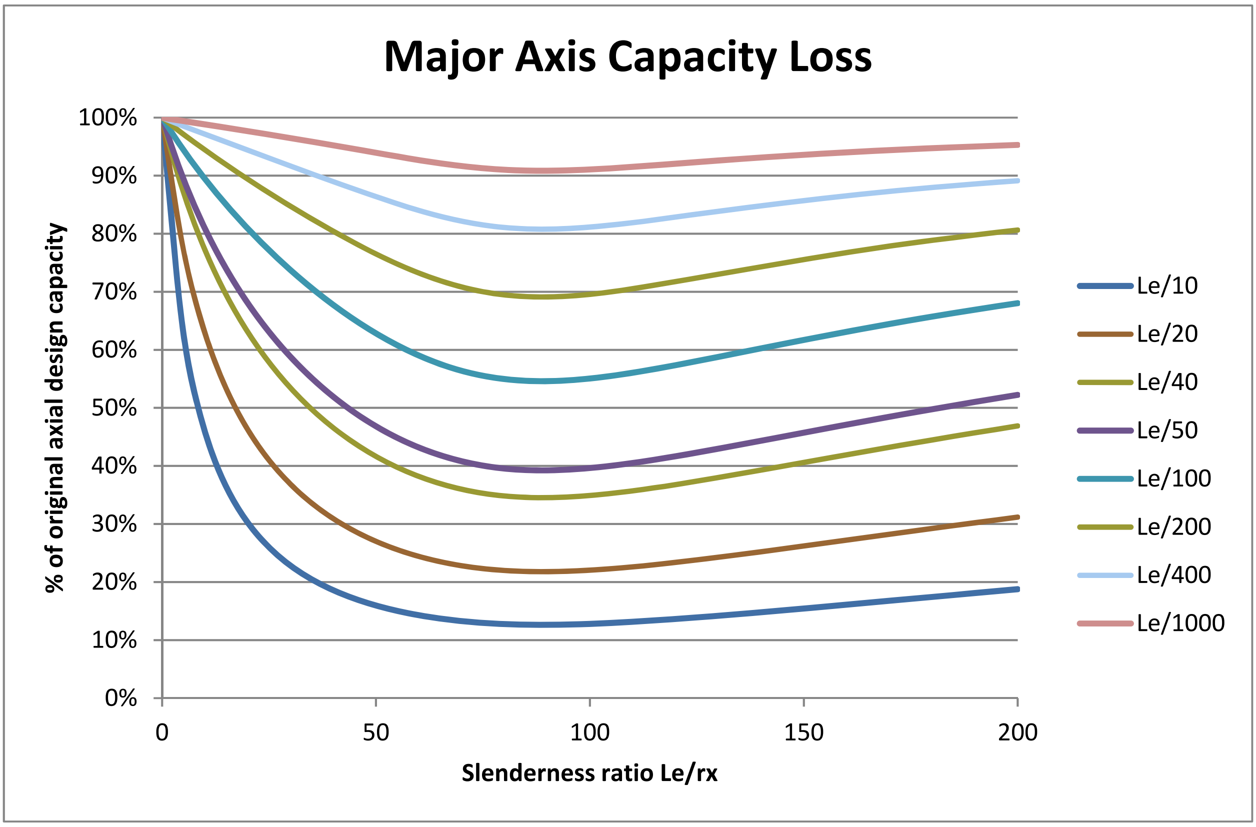
The plots above clearly show that the Timoshenko and Gere equation with a = ℓe/450 for the major axis and = ℓe/(450×cy/ry×rx/cx) for the minor axis matches the AS4100 equations almost exactly.

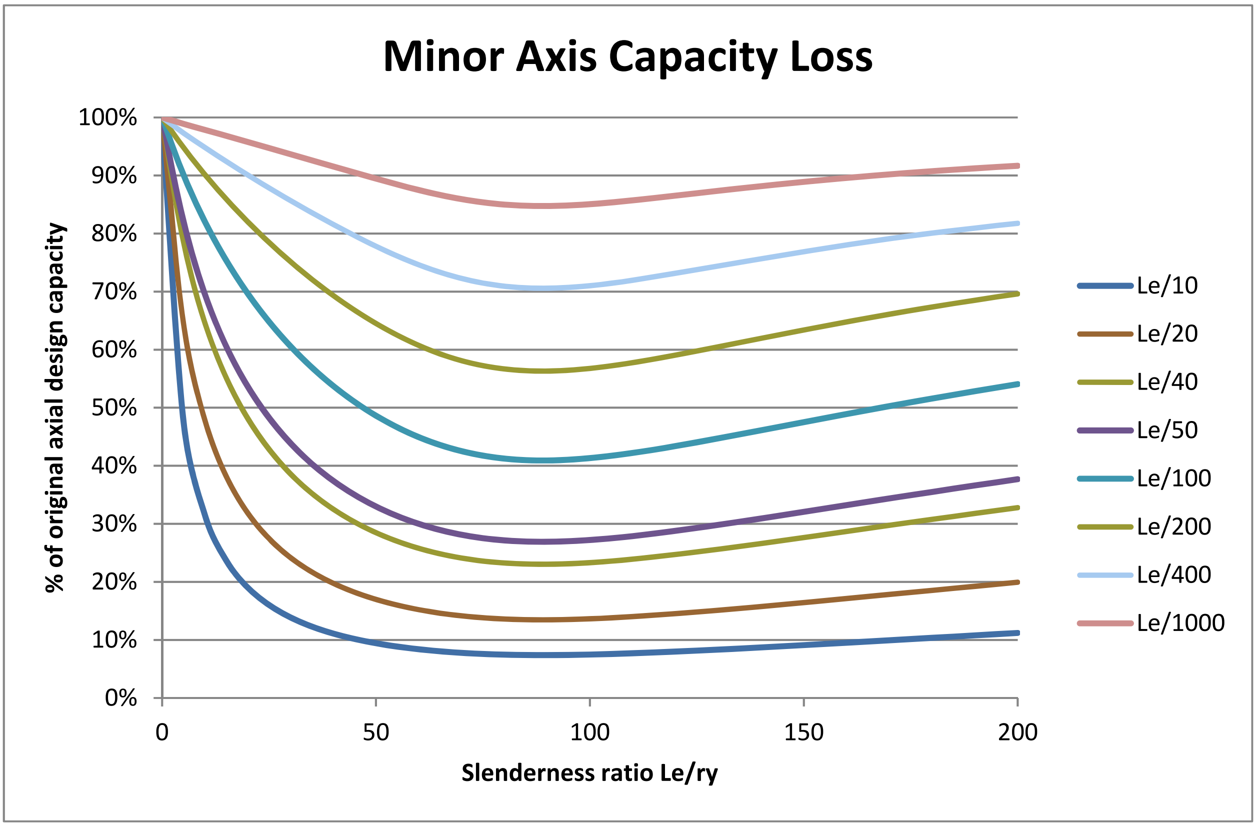
The Perry Robertson equations used by AS3990 give greater capacities for smaller slenderness ratios and lesser capacities for the higher slenderness ratio. In order to estimate the effects of overall imperfections the Timoshenko and Gere formula is used and the measured imperfection is superimposed on the $a = \ell e/450$ for the major axis and the $a = \ell e/(450 \times cy/ry \times rx/cx)$ for the minor axis to compute the reduced capacity.

The following capacity plots for differing magnitudes of imperfections expressed as a ratio of Le are plotted below for a 360UB50.7 section. It is noted that these plots are very similar for all the UB sections with the "percentage error" for the minor axis plots being roughly the same the percentage difference between the factor $cy/ry \times rx/cx$ for the desired section and 1.85, the factor for a 360UB50.7 section.

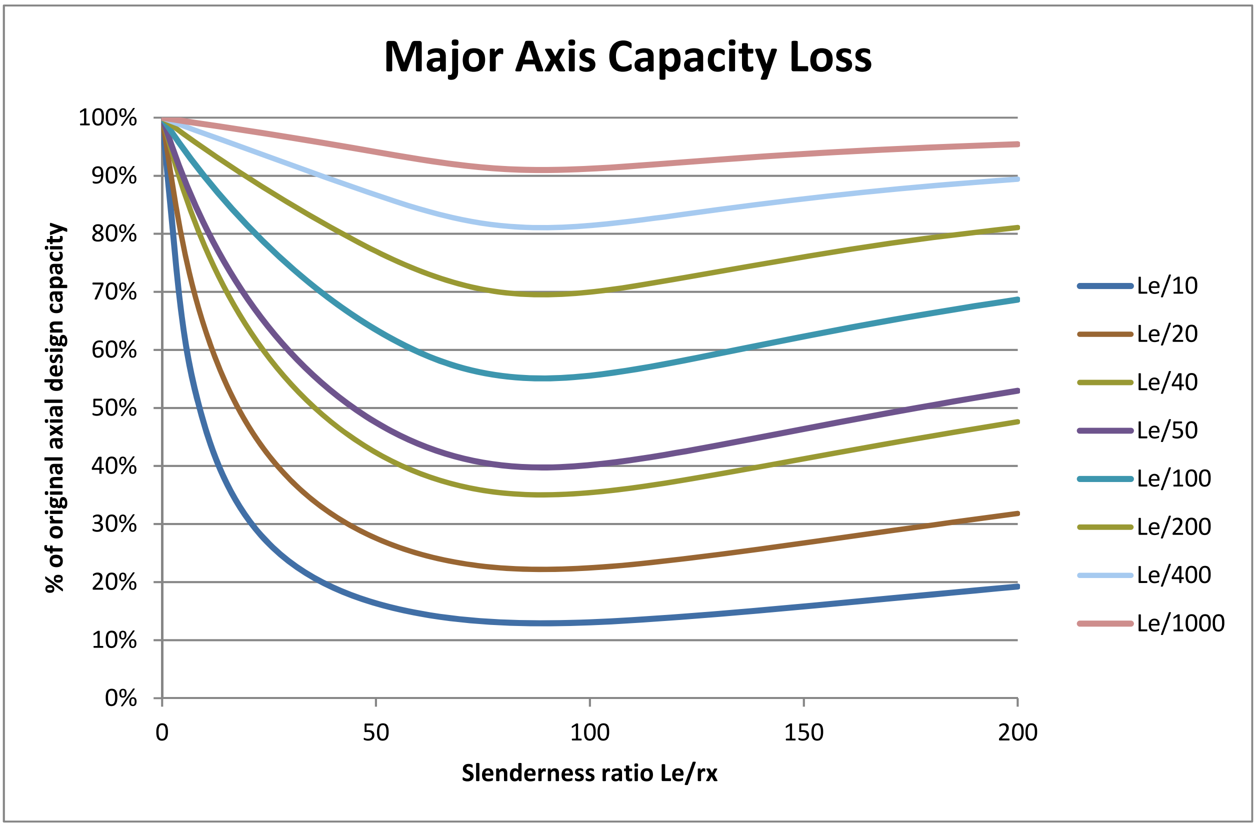
The plots are very similar for UC sections as is borne out by the capacity plots for a 200UC52.2 section.

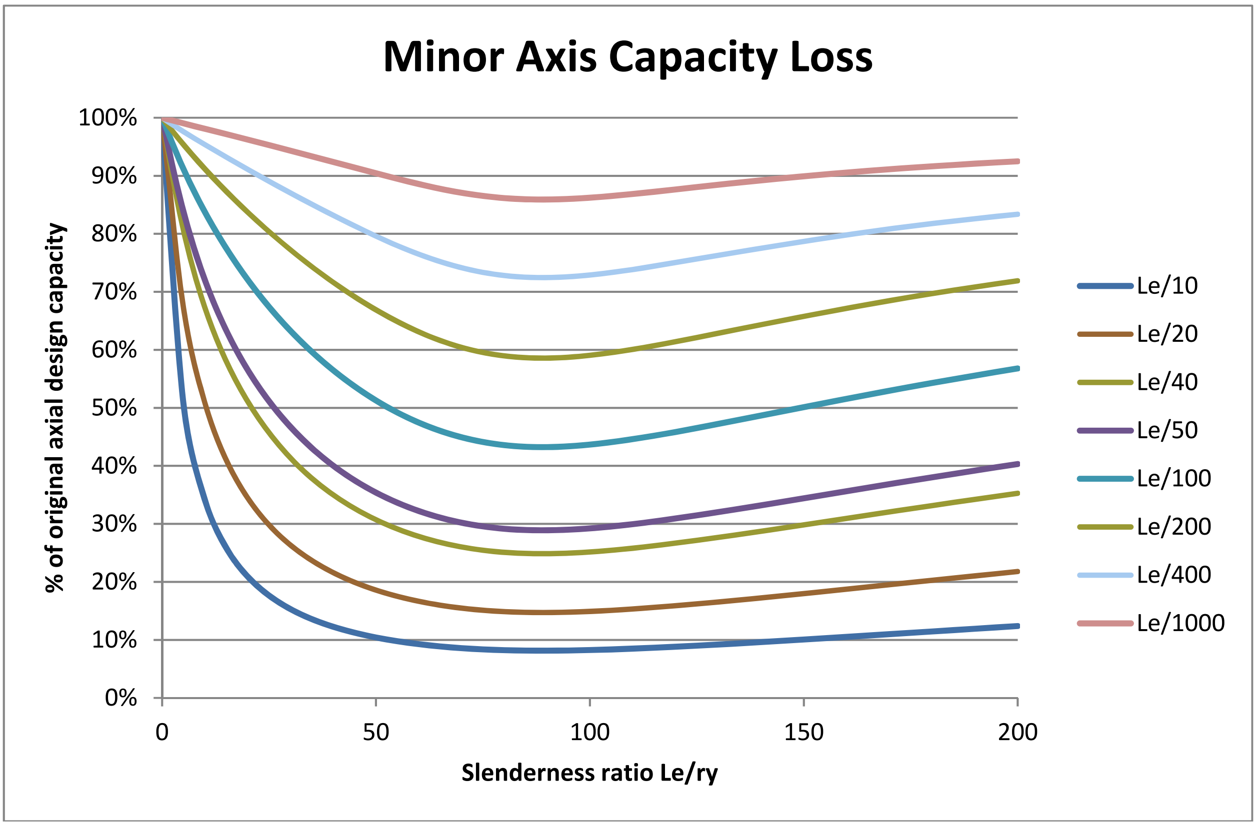
For a 360UB50.7 section





For a **200UC52.2** section





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