TALAT Lecture 2204

Design Philosophy

30 pages, 10 Figures

Advanced Level

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Objectives:

- to establish an understanding of the requirements on load bearing structures with respect to safety against failure
- to introduce the design analysis process with methods of verification and partial safety factors
- to describe the characteristic of loads and load combinations on structures
- to introduce the subject of load and resistance factors in the verification methods
- to describe the basic structural design properties of aluminium alloys vs. steel

Prerequisites:

- background and experience in structural engineering and design calculations
- basic understanding of the physical and mechanical properties of aluminium

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2204 Design Philosophy

Contents

2204.01 Notations

E	-	Youngs modulus of elasticity
F	-	load
F_d	-	design load
F_k	-	characterictic load
G_k	-	characteristic value of permanent load
Q_k	-	characteristic value of variable load
Q_{ak}	-	characteristic value of accidental load
R	-	resistance
R_m	-	tensile strength of test specimen
$R_{p0,2}$	-	0,2 proof strength of test specimen
S	-	load effect
S_k	-	characteristic value of snow load
W_k	-	characteristic value of wind load
f	-	strength
f_d	-	design strength
f_k	-	characteristic strength
l	-	dimension
l_d	-	deisgn dimension
l_k	-	characteristic dimension
s	-	safety factor
t_q	-	duration of variable loads
t_{tot}	-	service life of structure
δ	-	deflection
γ_F	-	load factor
γ_G	-	load factor for permenent loads
γ_M	-	resistance factors
γ_Q	-	load factor for variable loads
γ_{Qi}	-	load factor for variable loads except the main variable load
η	-	factor for transforming strength of test specimen to strength of structure
η_d	-	relative duration
σ	-	stress
σ_{all}	-	allowable stress
σ_y	-	yield stress
ψ	-	reduction factor for loads in combination

2204.02 Introduction and Definition

The procedure, starting from general information concerning the use, location etc. of the structure to be built, and leading to complete design documents sufficient for the manufacture and erection/installation, is referred to as the design procedure.

The course of the design procedure naturally depends on the type of structure, the purchaser, future proprietor etc. In most cases the various phases within the procedure may in general be described in the following manner.

- The purchaser initiates the project and provides the conditions and general requirements.
- The design engineer formulates the conditions and requirements in technical terms guided by the regulations given by authorities.
- The designer selects the structural system and materials in cooperation with the purchaser and based on a preliminary design analysis.
- The designer performs the design analysis which includes dimensioning by structural analysis, preparation of drawings, specifications and descriptions.
- The design documents are approved by the purchaser, authorities and, possibly, by a responsible designer.
- The manufacture and erection/installation can be commenced supervised by the purchaser, the authority and the designer.

It should be pointed out that all phases of the procedure are of importance in order to arrive at an adequate design implying good quality and acceptable economy. There may be a tendency to underestimate the responsibilities of the designer in the early stage of the procedure.

In summary, the objectives of the design procedure are:

- to produce design documents (drawings, descriptions, specifications etc.) suitable as a basis for fabrication of the structure,
- to verify that the documents are in agreement with the purchaser's requirements according to the given design conditions and valid regulations, and
- to ensure, as far as possible, that the documents specify a structure satisfactory from an economical point of view.

2204.03 Requirements on the Load Carrying Structure

- Specification of requirements
- Requirements on safety against failure
- Requirements on the serviceability of structures in normal use
- Limit state
- Safety classes
- Economic considerations on the formulation of requirements

Specification of Requirements

Requirements here and in the following sections denote expressions of expectations defined by the purchaser, future proprietor, utilizers, authorities, etc. concerning the function of the structure. The requirements may to some extent be varied with respect to the balance between quality level and cost.

The requirements on a load carrying structure may be specified as follows:

- requirements on safety against failure,
- requirements on serviceability in normal use,
- requirements on durability.

Requirements on Safety Against Failure

The concept of failure may imply anything from destruction of a structural element to collapse of the entire structural system. The cause of a failure may be of various kinds and can be classified in three categories:

1. Unfavourable combinations of factors affecting the resistance.

An unfavourable combination of critical parameters has occurred. These parameters may be interpreted as loads, strength of the material, dimensions, imperfections and minor damages. They possess values which may be extreme, but do not deviate significantly from normal conditions.

2. Unforeseen loads.

An event (explosion, fire, ship impact etc.) not considered in the design has appeared as a single occurrence with such a magnitude that the consequence was failure of the structure. The load may either be of a character entirely different from those considered in the design, or it may be of the same character but of a magnitude not foreseen.

3. **Gross errors.**
   A gross error has been committed in the design work, material production, or construction. A gross error implies that the structure has received some material or geometrical property of a character entirely different from what was intended.

The requirement on safety against failure means that the structure shall be designed and fabricated in such a way that the probability of failure becomes sufficiently low. The concept "sufficiently low" also implies that the probability has to be lower the more serious the consequences would be of a failure happening.

The measures to be taken to ensure a sufficiently low probability of failure should in principle be adapted to all categories mentioned above.

When the cause of a failure is attributed to the first category, the risk of failure can be sufficiently reduced at the design level by choosing sufficiently large factors of safety, which can also be dependent with regard to the consequences of a possible failure.

The measures which can be taken against failure occurring because of an unforeseen load are more difficult to quantify. Some loads of that kind may be known to a certain degree through experience from earlier incidents. This is, for instance, the case with loads arising as a consequence of a collision or an explosion. Other kinds of loads may be possible but so far unknown. A reasonable step may be to design a structure with respect to a few known loads of the kinds mentioned above and further assume that it will also be able to resist other types of loads of a similar category. As a complement, or an alternative, it is possible to select a structure of such a type and perform a detailed design in such a way that the carrying system becomes highly insensitive to local damage, which may arise from loads of the kinds mentioned. Unforeseen loads may, for example, be caused by impact of various kinds, flood and earthquake. The character of these loads implies that the probability of their occurrence is small. Therefore, they need to be considered only for those types of structures where the consequence of a possible failure may be expected to be very serious. Structures of a vital importance should thus, if possible, be designed according to damage tolerance criteria.

Gross errors can, for example, be caused by the designer in miscalculating a wall thickness by a factor of 2, or in the manufacture of a metal structure by forgetting to define the characteristics of a welded joint or a similar operation. Such errors can not be compensated for by choosing a larger safety factor in the design analysis. Measures to be taken to decrease the frequency of gross errors are:

- improved training and information,
- improved organization at the building site,
- more effective quality control.

In summary it may be stated that the measures which can be taken in order to keep the probability of failure at a low level do not only apply to the choice of safety factors but include also training, information, organization and quality control.

Requirements on the Serviceability of Structures in Normal Use

If a load carrying structural member is, in normal use, subjected to damage or causes damage to other members and, if the damage is unacceptable, the function or serviceability of the structural member can be considered to be unsatisfactory. The damage may be permanent or occasional. The word damage is used here in a wider sense and can be the cause of, for instance, some kind of inconvenience.

Examples of *permanent damages* may be open cracks in the structural member, cracks in other building components, e.g. partition walls, and disturbing permanent deflections of beams. If such damage has occurred and involves inconvenience, it will continue to bring the same or about the same inconvenience until repaired. In this case the requirements given and the measures taken to avoid the inconveniences should be aimed at reducing the *risk of generation* of the damage. In principle, the problem is equivalent to that concerning safety against failure. Even if no well-defined limit exists between these cases, the risk which can be accepted for a minor damage to occur to the structure in normal use, is normally higher, however, than the acceptable risk of failure. This implies that it is, in general, only necessary to consider causes of damage corresponding to the category in the preceding paragraph.

Examples of *occasional damages* are occasional large deflections of beams and occasional vibrations. The inconvenience of such damages will only appear during those periods when the load or other actions occur which cause the damage. The requirements and measures to reduce the inconveniences should, in this case, be concentrated to the *duration of the damage*. Vibrations of a certain intensity may be acceptable from a comfort point of view if they appear infrequently and only during short periods of time. On the other hand vibrations of the same intensity may be entirely unacceptable if they are effective during longer periods.

The requirements on the serviceability of a structure in normal use apply, in most cases, to deformations including oscillations and vibrations (considered as time dependent deformations). The inconveniences resulting from large deformations can be the following: they

- can cause damage to other building components,
- may convey a feeling of discomfort to people in the building,
- can disturb and impair the function of machines, instruments and similar objects supported by the structure,
- may be disturbing from an aesthetic point of view.

Further cases of damage or poor function in normal use may refer to

- abrasion,
- leakage, e.g. in liquid tanks,
- surface finish, e.g. roughness or discoloration etc.

It is not possible to express generally valid requirements concerning the function of a structure in normal use by numerical values. The requirements which should be formulated are too much dependent on the situation to which the requirement applies. Usually, the future proprietor/utilizer may establish the requirements after consultation with the design engineer. Moreover, the requirements must be expressed with due regard to the situation. A requirement concerning limitations of the deformations can thus be formulated in one of the following ways:

- limitation of absolute values of displacements,
- limitation of the mutual displacements between the nodes of e.g. a frame system,
- limitation of the deflection of a structural component (e.g. a beam) in proportion to the span,
- limitation of the angular deformation of a structural component.

Specific recommendations are given in the different national codes concerning limitation of angular deformations in order to avoid damage in adjacent building components. Furthermore, recommendations are provided concerning the bending stiffness of beams required to guarantee that deflections do not cause discomfort for people walking on a floor or over a bridge, or that the structure is not operationable at this deflection (crane beams).

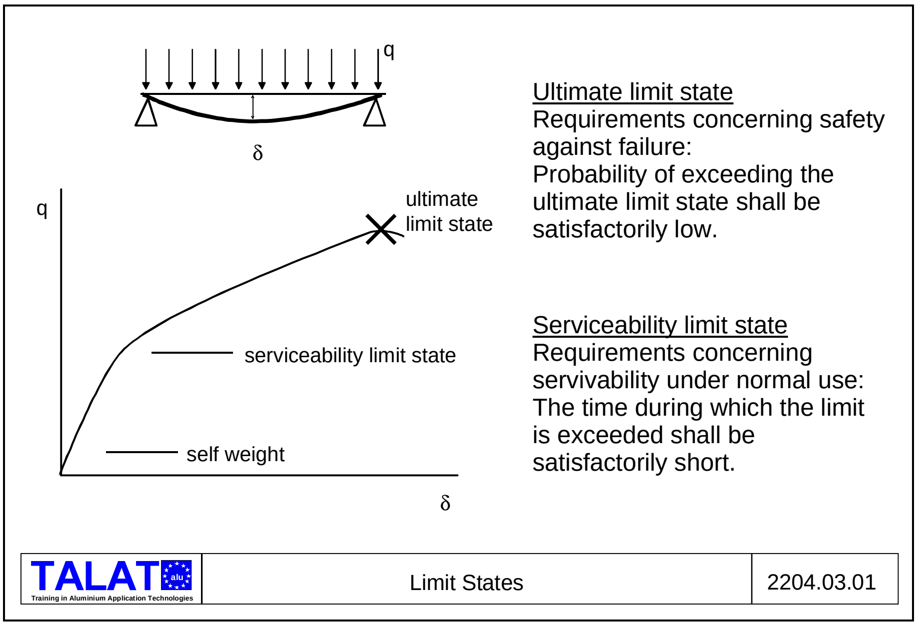
Limit States

The requirements on the load carrying function of a structure apply to both safety against failure and to serviceability in normal use. These two requirements are, at least in some cases, quite different in nature and should thus be separated in their formulation. This can be achieved by performing the design analysis at two limit states with regard to the function of the structure:

- ultimate limit state, which is a state where the structure is at the limit of failure,
- serviceability limit state, which is a state where the structure is at the limit of not satisfying the requirements for normal use.

The implication of the limit states is illustrated in **Figure 2204.03.01**, which shows the deflection versus load for a simply supported beam. The serviceability limit state and the ultimate limit state are indicated by their upper limits.

The limit states are thus conceivable states of the structure. The requirements concerning safety against failure are, in principle, formulated such that the probability that any of the possible ultimate limit states is exceeded is satisfactorily low. The requirements with regard to serviceability in normal use are established in a corresponding way, or such that the time during which the limit is exceeded, will be satisfactorily short.



Economic Considerations on the Formulation of Requirements

Some of the requirements which apply to a structure - in particular those concerning the safety against failure - constitute the requirements of the society. They are given in the national codes and standards and should be regarded as minimum requirements. Therefore, they cannot be modified in an alleviating direction.

The remaining requirements are given by the purchaser/future proprietor and utilizer (tenant). This means, that in the early phase of the design procedure, the cost of future maintenance and repair during the service life of the structure have been determined to a certain degree. There are thus good reasons to consider, at an early stage, the formulation of the requirements from an economic point of view.

2204.04 The Design Analysis Process

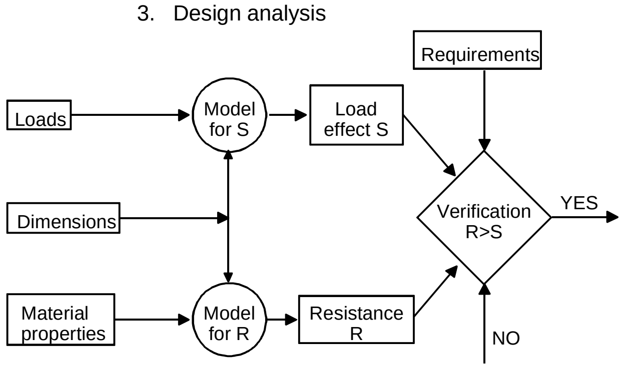
- Introduction
- Methods of verification
- The load and resistance factor method
- Method of allowable stresses

Introduction

After the formulation of requirements follows the selection of systems and materials. At this point the design analysis begins, which involves a detailed determination of dimensions and strength of structural components. The methods of analysis can often be decided by the designer himself. It is essential that the verification of the structure, with the chosen dimensions and the properties of the materials selected, satisfies the requirements established. The procedure can be described according to **Figure 2204.04.01** for a simple case. With the assumptions stated concerning loads, dimensions and material properties, calculation models are applied which provide the load effect *S* (Solicitation, in ENV 1999-1-1, called E) and carrying capacity *R* (Resistance). The load effect may be expressed as a section quantity (e.g. a bending moment in a beam) caused by the load.

The resistance is the capacity of the structure to resist a load effect of the same kind (the capacity of the beam to transfer a moment). The verification implies that the resistance R has to be higher than the load effect *S*.

1. Formulation of requirements
2. Selection of system and material
3. Design analysis



Assumptions on loads, dimensions and material properties, and calculation models provide the load effect (solicitation) S and resistance R.
The load effect may be expressed as section quantity, e.g. bending moment caused by the load.

TALAT	Schematic Description of the Design Analysis Process	2204.04.01

The case described concerns safety against failure, but the procedure of verification that the requirements on the serviceability of the structure in normal use are satisfied will in principle be the same. In many cases the procedure is more complicated. Several different kinds of load effects and resistance (e.g. normal forces and bending moment) may act at the same time. The verification analysis provides an answer, yes or no. In case the answer is no, the procedure has to be repeated with updated dimensions and material properties.

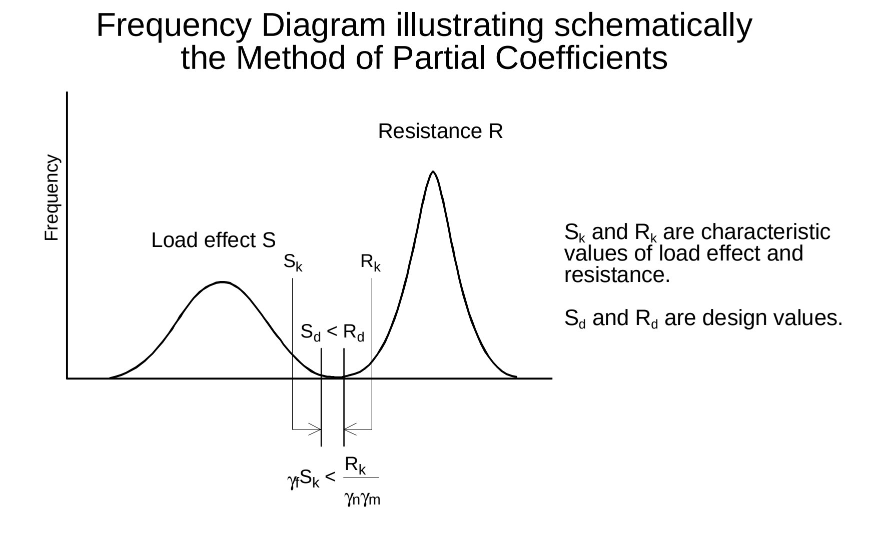
Applications of the design analysis process will be found in lecture 2204.07 (Design Criteria).

Methods of Verification

The quantities which describe the load effect *S* and the resistance *R* (e.g. load values *F*, strength values *f* and dimensions *l*) are stochastic variables which can be represented in a simplified manner by frequency curves according to **Figure 2204.04.02**.

The verification consists of demonstrating that the resistance *R* is greater than the load effect *S*. This can be done by use of a number of methods, listed in historical order:

- The safety factor method (method of allowable stresses)
- The load factor method with one single load factor (often used in plastic design)
- The load and resistance factor design method (method of partial coefficients),
- Probabilistic methods



Frequency Diagram Illustrating Schematically the Method of Partial Coefficients 2204.04.02

The first method has been used earlier, and is still being used in design codes in many countries but it is being replaced by the third method.

Probabilistic methods have to be based on statistical data for loads, strength properties etc. which, so far, are available only on a very limited scale. The methods are, therefore, only used in very special cases.

The load and resistance factor method and the method of allowable stresses are briefly described below. A more comprehensive discussion of the methods will be found in chapters **2204.05** and **2204.06** and in the lecture series **2400** (Fatigue).

The Load and Resistance Design Factor Method

The load and resistance factor method (often called the method of partial coefficients) is a verification method which is accepted in many countries. In the following, the method is described as it is applied in the Eurocodes. The formulation is very similar to that used in the different national codes and standards.

The basis is formed by the so called characteristic values:

F_k	for loads (called «actions» in Eurocodes)
f_k	for strength
l_k	for dimensions where, in most cases, l_k is equal to the nominal value, i.e. the value given in drawings and descriptions.

The calculation of F_k and f_k is indicated in chapter **2204.05** and chapter **2204.06**. From the characteristic values the design values are deduced:

$$F_d = \gamma_F F_k \qquad \text{for loads} \qquad (4.1)$$

$$f_d = \frac{f_k}{\gamma_M} \qquad \text{for strength} \qquad (4.2)$$

$$l_d = l_k + \Delta l \qquad \text{for dimensions} \qquad (4.3)$$

γ_F and γ_M are called partial coefficients. The partial coefficient γ_F for load is in the following referred to as the load factor, and the partial coefficient γ_M is named resistance factors. Δl is an additive quantity by which deviations from the ideal dimensions are considered. In most cases Δl can be set to zero. The partial coefficients are discussed in more detail in **2204.05** and **2204.07**.

The design values are used in the calculation models for load effect and resistance and provide the design criteria.

$$R(f_d, 1_d) \geq S(F_d, 1_d) \qquad (4.4)$$

The load and resistance factor method is illustrated in **Figure 2204.04.02**. Since the load factor can be given different values for different kinds of loads a more consistent design for a low risk of failure can be attained. For example, γ_F = 1.1 is adopted for gravity loads and 1.5 for environmental loads, such as snow and wind loads in load combinations see **2204.05**.

Method of Allowable Stresses

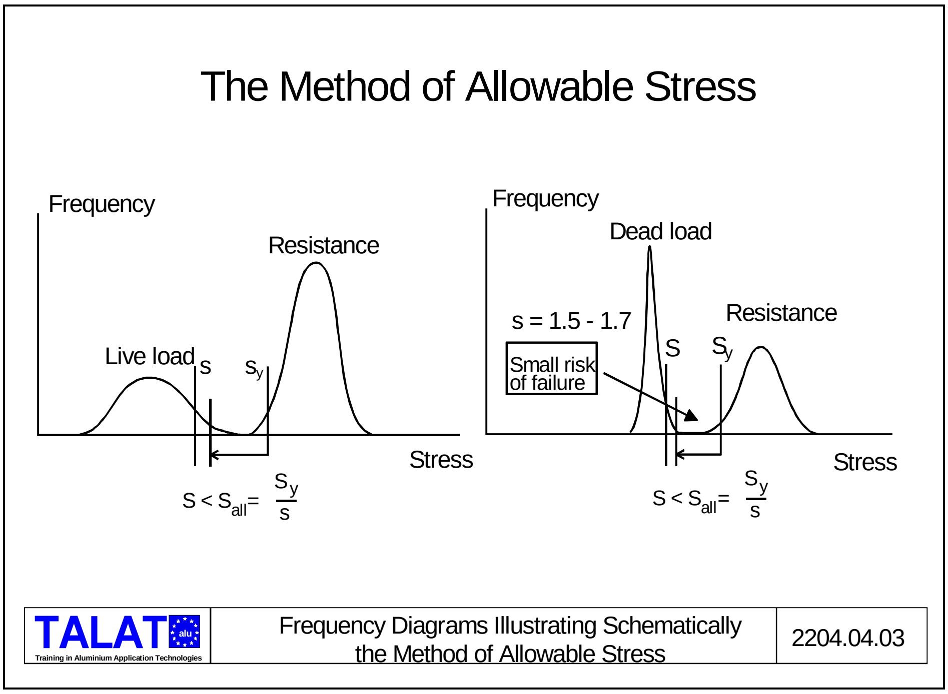
In some design codes the scatter in loads, resistance etc. is covered by one single safety factor *s*. The verification consists of demonstrating that

$$\sigma \leq \sigma_{all} \tag{4.5}$$

where σ is the stress determined from the loads and, for instance when designing against yield failure (plastic deformations),

$$\sigma_{all} = \frac{\sigma_y}{s} \tag{4.6}$$

The safety factor *s* may vary within rather wide limits (1.3 - 3.5) depending on what elements of uncertainity have to be considered. In design against buckling, safety factors to the order of magnitude 10 are found in older codes. It should be noted, however, that the analysis in this course provides lower limit values of the carrying capacity, for instance with respect to buckling and a safety factor of the order of 1.5 to 2 would be appropriate.



2204.05 Loads and Load Factors

- Introduction
- Classification of loads
- Characteristic loads, normal loads and long-term loads
- Load combinations, design value of the load
  - Examples
- Loads on buildings, bridges and hydraulic structures

Introduction

The following discussion on loads is, primarily, applicable to the construction sector, i.e. for buildings, bridge and hydraulic constructions, and for scaffoldings in installation and erection, cranes, masts, power-line pylons, lighting posts and similar load carrying structures.

The discussion will, however, be of interest also to design engineers working with other types of structures such as cisterns, pressure vessels, tanks, transportation vehicles etc.

Classification of Loads

Loads are in the present publication used as a common name for effects due to forces and deformations. A force effect is primarily caused by external forces on a structure, while the deformation effect is primarily caused by a forced displacement, e.g. a support settlement, change of temperature or humidity.

Loads may be classified with respect to their variation with time as

- permanent load — approximately constant in time
- variable load
  - static load
  - dynamic load — which causes additional forces due to acceleration including resonance
  - fatigue load — load with so many load cycles that fatigue failure can occur
- accidental load — e.g. impact, explosion

Loads can also be classified with respect to variation in space

- fixed load — the load distribution over the structure is uniquely defined

- free load — has an arbitrary distribution over the structure within possible limits

The duration t_q of variable loads (**Figure 2204.05.01)** is the time during which the magnitude of the load amounts to at least the value q within the service life t_{tot} of the structure. The relative duration is defined as

$$\eta_q = t_q / t_{tot} \qquad (5.1)$$

It is assumed that the variations of the load are similar during the entire service life t_{tot}. The reduction factor Ψ, which defines a normal load value of ΨQ_k, is derived from the relative duration η_q.

In ENV 1991-1 the Ψ - factor (combination value) is divided into 3 factors:

Ψ_0 = coefficient for combination value of a variable load
Ψ_1 = coefficient for frequent value of a variable load
Ψ_2 = coefficient for quasi-permanent value of a variable load

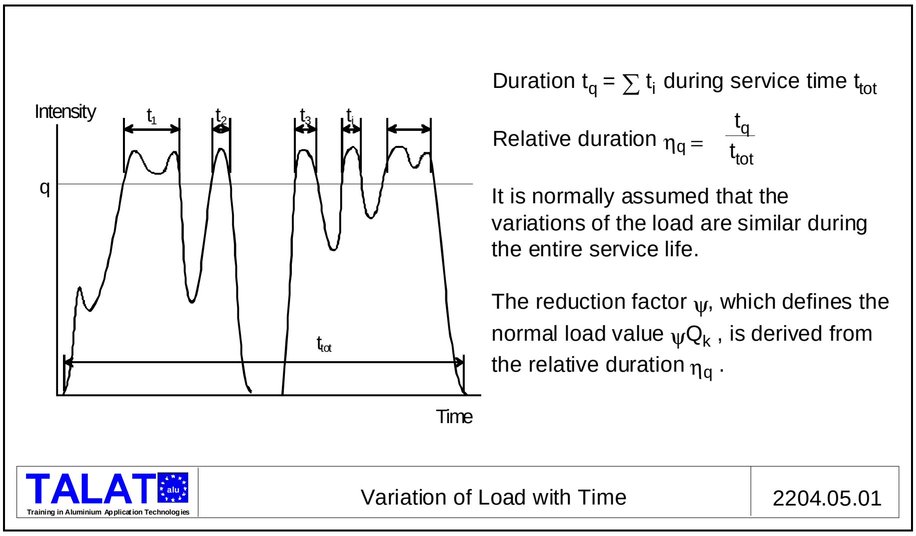
The combination values (Ψ_0) are associated with the use of combinations of loads, to take account of a reduced probability of simultaneous occurence of the most unfavourable values of several independent loads.

The frequent value (Ψ_1) is determined such that the total time, within a chosen period of time, during which it is exceeded for a specified part, or the frequency with which it is exceeded, is limited to a given value. The part of the chosen period of time or the frequency should be chosen with due regard to the type of construction works considered and the purpose of the calculations. Unless other values are specified the part may be chosen to be 0,05 or the frequency to be 300 per year for ordinary buildings.

The quasi-permanent value (Ψ_2) is so determined that the total time, within a chosen period of time, during which it is exceeded is a cinsiderable part of the chosen period of time. The part of the chosen period of time may be chosen to be 0,5. The quasi-parmanent value may also be determined as athe value averaged over the chosen period of time.

These representative values and the characteristic value are used to define the design values of the loads and the combination of loads. The combination values are used for the verification of ultimate limit states and irreversible serviceability limit states. The frequent values and quasi-permanent values are used for the verification of ultimate limit states involving accidental loads and for the verification of reversible serviceability limit states. The quasi-permanent values are also used for the calculation of long term effects of serviceablilty limit states.

In structures subjected to fatigue loading, the load range, the load level, and the number of load cycles are usually of importance. (For the design of aluminium alloys structures with regard to fatigue see **lecture 2400**).



Characteristic Loads, Normal Loads and Long-Term Loads

According to most national codes, loads are defined as follows:

- the characteristic value G_k of a permanent load shall be assumed to be the mean value.
- the characteristic value Q_k of a variable load shall be a value with the probability 0.02 of being exceeded at least once during one year.
- the normal value $\Psi_i Q_k$ of a variable load shall be determined considering the relative duration $\eta_q = t_q/t_{tot}$,
- characteristic value Q_{ak} of an accidental load shall be determined with respect to the nature of the load.

Further below it is indicated where G_k, Q_k, Ψ_i and Q_{ak} for normal loads on buildings, bridges and hydraulic structures are defined. If the characteristic value is not available in a load standard, the value of Q_k may in principle be estimated by use of the following procedure (determination of G_k usually does not present a problem).

1. Several observations, about 50, of the yearly maximum load are available. Fit a reasonable distribution function F_Q to measured values and determine Q_k from the condition $F_Q = 0.98$.

2. A smaller number of observations are available. The problem consists of finding a conservative distribution. A lognormal distribution function complies with this

requirement in most cases, and for such a distribution, Q_k can be determined by computing:

a) the mean value μ of log χ

b) the standard deviation σ of log χ_i

c) log $Q_k = \mu + 2.05\sigma$, or $Q_k = \exp(\mu + 2.05\ \sigma)$, where $2.05 = \Theta^{-1}(0.98)$ and Θ is the distribution function of the standardized normal distribution.

3) No observations of the yearly maximum load are available. In this case it is in principle not possible to determine Q_k. The situation is not unusual, however, and it is thus often necessary to make an estimate of Q_k.

a) Compare with other similar loads for which Q_k is known.

b) Guess the mean m and the standard deviation s. Adopt $Q_k = \exp(\log m + 2.05\sigma)$ where δ = s/m, compare 2) above. It is normally easier to make a reasonable guess of *m* and *s* than to guess directly the 98 per cent fractile.

c) Assume Q_k to be equal to the physical upper limit of the load. It is sometimes possible to indicate an upper limit. For instance, a reservoir or a tank can only be filled to its capacity.

Load Combinations, Design Value of the Load

For each critical load case, the design values of the effects of loads should be determined by combining the values of loads which occur simultaneously, as follows:

a) Persistent and transient situations: Design values of the dominant variable loads and the combination design values of other loads.

b) Accidental situations: Design values of permanent loads together with the frequent value of the dominant variable load and the quasi-permanent values of other variable loads and the design value of one accidental load.

Seismic situations: Characteristic values of the permanent loads together with the quasi-permanent values of the other variable loads and the design value of the seismic loads.

When the dominant load is not obvious, each variable load should be considered in turn as the dominant load.

Design situation	Permanent actions G_d	Single variable actions Q_d		Accidental actions or seismic actions A_d
		Dominant	Others	
Persistent and transient	$\gamma_G\ G_k\ \ (\gamma_P\ P_k)$	$\gamma_{Q1}\ Q_{k1}$	$\gamma_{Qi}\ \Psi_{0i}\ Q_{ki}$	
Accidental	$\gamma_{GA}\ G_k\ (\gamma_{PA}\ P_k)$	$\Psi_{11}\ Q_{k1}$	$\Psi_{2i}\ Q_{ki}$	$\gamma_A\ A_k$ or A_d
Seismic	G_k		$\Psi_{2i}\ Q_{ki}$	$\gamma_I\ A_{Ed}$

In general, the design value of the loads is a load combination as follows:

$$\sum_{j\geq 1}\gamma_{Gj}\cdot G_{kj}+\gamma_{Q1}\cdot Q_{k1}+\sum_{i>1}\gamma_{Qi}\cdot\Psi_{0i}\cdot Q_{ki}$$

where γ_{Gj} = partial factor for permanent load j

G_{kj} = characteristic value of a permanent loads

γ_{Qi} = partial factor for for variable load i

Q_{k1} = characteristic value of the variable load 1

Q_{ki} = characteristic value of the variable load i

Ψ_{0i} = combination coefficients

γ_P = partial factor for prestressing loads

P_k = characteristic value of prestressing load

In the relevant load cases, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values, those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values.

Where the results of a verification may be very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and the favourable parts of this action shall be considered as individual actions. This applies in particular to the verification of static equilibrium.

For building structures, the partial factors according to ENV 1991-1 for ultimate limit states in the persistent, transient and accidental design situations are given in table below. The values have been based on theoretical considerations, experience and back calculations on existing designs.

Case[1)]	Action	Symbol	Situations P/T	Situations A
Case A Loss of static equilibrium; strength of structural material or ground insignificant	Permanent actions: self weight of structural and non-structural components, permanent actions caused by ground, ground-water and free water - unfavourable - favourable	 γ_{Gsup} γ_{Ginf}	 1,10 0,90	 1,00 1,00
	Variable actions - unfavourable	 γ_Q	 1,50	 1,00
	Accidental actions	γ_A		1,00
Case B Failure of structure or struc tural elements, including those of the footing, piles, basement walls etc., governed by strength of structural material	Permanent actions (see above) - unfavourable - favourable	 γ_{Gsup} γ_{Ginf}	 1,35 1,00	 1,00 1,00
	Variable actions - unfavourable	 γ_Q	 1,50	 1,00
	Accidental actions	γ_A		1,00
Case C Failure in the ground	Permanent actions (see above) - unfavourable - favourable	 γ_{Gsup} γ_{Ginf}	 1,00 1,00	 1,00 1,00
	Variable actions - unfavourable	 γ_Q	 1,00	 1,00
	Accidental actions	γ_A		1,00
P: Persistent situation	T: Transient situation	A: Accidental situation		
1) The design should be verified for each case A, B and C separately as relevant				

Recommended Ψ factors for buildings according to ENV 1991-1 are given in the table below. In ENV 1991-1 the values are boxed. For other applications see relevant parts of ENV 1991.

Action	Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings [1)]			
category A: domestic, residential	0,7	0,5	0,3
category B: offices	0,7	0,5	0,3
category C: congregation areas	0,7	0,7	0,6
category D: shopping	0,7	0,7	0,6
category E: storage	1,0	0,9	0,8
Traffic loads in buildings			
category F: vehicle weight: ≤ 30kN	0.7	0,7	0,6
category G : 3OkN < vehicle weight ≤ 160kN	0,7	0,5	0,3
category H: roofs	0	0	0
Snow loads on buildings	0,6	0,2	0
Wind loads on buildings	0,6	0,5	0
Temperature (non-fire) in buildings [3)]	0,6	0,5	0)

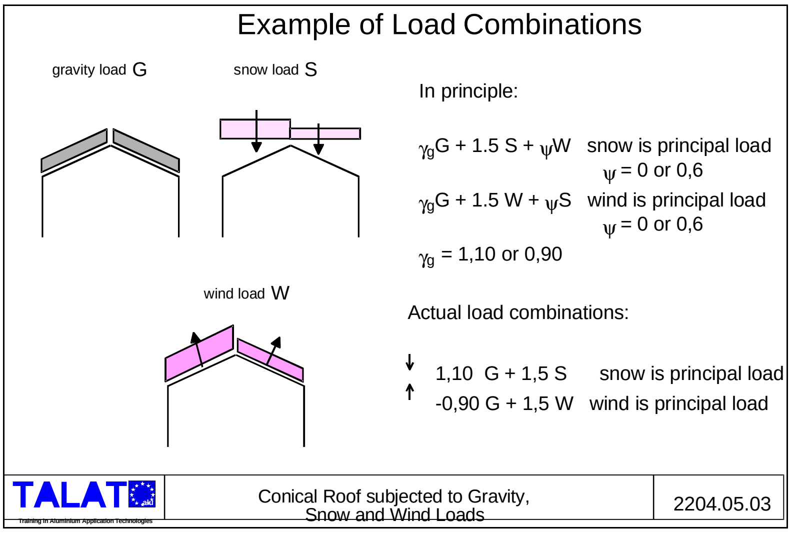
1) For combination of imposed loads in multistorey buildings, see ENV 1991-2-1.
2) Modification for snow loads for different geogaphical regions may be required.
3) See ENV 1991-2-5.

The combination of actions to be considered for serviceability limit states depends on the nature of the effect of actions being checked, e.g. irreversible, reversible or long term. Three combinations designated by the representative value of the dominant action are given in the following table.

Combination	Permanent actions G_d	Variable actions Q_d	
		Dominant	Others
Characteristic (rare)	G_k (P_k)	Q_{k1}	Ψ_{0i} Q_{ki}
Frequent	G_k (P_k)	Ψ_{11} Q_{k1}	Ψ_{1i} Q_{ki}
Quasi-permanent	G_k (P_k)	Ψ_{21} Q_{k1}	Ψ_{2i} Q_{ki}
For serviceability limit states, the partial factors (serviceability) γ_G and γ_Q are taken as 1,0 except where specified otherwise.			

Example

Indicate the load combinations in the ultimate limit state which have to be considered in the design analysis of the tank roof in **Figure 2204.05.03**. The roof shall be designed for gravity load G, snow load S and wind load W. (Other loads may occur but are not included for the sake of simplicity).



In general, four different alternatives must be investigated:

1. $\gamma_g G_k + 1{,}5W_k$ — wind is the principal load
2. $\gamma_g G_k + 1{,}5W_k + 0{,}6S_k$ — wind is the principal load
3. $\gamma_g G_k + 1{,}5S_k$ — snow is the principal load
4. $\gamma_g G_k + 1{,}5S_k + 0{,}6W_k$ — snow is the principal load

The load factor γ_g may, according to ENV 1991-1, assume the values 0,90 and 1,10, respectively. Due to symmetry, only one wind direction has to be investigated, but the wind load may have two different distributions, corresponding to two load cases.

The snow load also provides two load cases, either a uniform or a triangular distribution over the roof surface.

The four alternatives thus result in a large number of possible load combinations. Many of these are not critical, however, and may be sorted out at an early stage.

The cistern roof will probably be dimensioned by either

a) $1{,}10\ G_k + 1{,}50\ S_k$ or

b) $0{,}90\ G_k - 1{,}50\ W_k$

Since two snow and two wind load cases must be examined, a) and b) will result in four load combinations.

In an actual situation where G_k, S_k and W_k are known, the number of load combinations may often be further reduced, which implies that an individual structural element normally needs to be examined only for one or a couple of load combinations.

In certain types of structures, e.g. an unsymmetrical framework truss, a general application of the rules for selection of design load combinations leads to an overwhelming number of load cases, most of which are critical only for some elements. It should be noted, however, that the designer is free to perform an analysis on the safe side which, in many cases, will lead to a drastic reduction of the load combinations which must be considered. The increase in weight, for instance, that results is often marginal.

Loads on Buildings, Bridges and Hydraulic Structures

Frequently occurring loads on buildings, bridges and hydraulic structures are given in national or international specifications. Loads on overhead cranes are stated by the suppliers. Loads on power-line pylons are chosen according to special standards, etc.

2204.06 Resistance and Resistance Factors

- Assumptions concerning strength properties
- Models of analysis

Assumptions Concerning Strength Properties

The material strength properties are the yield and ultimate strength limits in compression and tension, the modulus of elasticity and the shear modulus. Other material properties related to strength are Poisson's ratio, fatigue strength, fracture toughness, creep properties and thermal expansion.

The requirements for design analysis of a structure indicate the strength class of the material to be used. In the analysis, then, various kinds of strength values are introduced which apply to the strength class selected. The strength values introduced in the design analysis are sometimes based on results from tests performed in advance. The producer of the material certifies that the strength properties are according to the requirements specified. Alternatively, the strength properties are checked at the delivery.

The procedure used to verify that the strength of the material meets the given requirements normally includes tests with special test specimens and a specified procedure. In certain cases the results of these tests cannot be considered to be directly representative for the strength of the material in the actual structure and, thus, have to be corrected. This may be performed by dividing the strength values obtained in the tests by a number η, normally greater than 1, such that:

$$f_{structure} = \frac{1}{\eta} f_{testspecimen} \tag{6.1}$$

The factor η should not be mistaken for the reduction factor with respect to buckling. *For metals*, the value of η should be *close to one*.

The characteristic value of strength f_k should be interpreted as a condition for the analysis which refers to the expected results of actual or imagined tests. It thus applies to the strength of the test specimen and not to that of the actual construction. The characteristic value is defined somewhat differently for different materials.

The design value for strength should, naturally, be valid for the material of the structure. This means a certain deviation from the basic presentation in **2204.04** in such a manner that the coefficient η should be entered into the equation below which translates characteristic values into design values. With this modification the formula for computation of the design value f_d from the characteristic value f_k becomes

$$f_d = \frac{f_k}{\eta \gamma_M} \tag{6.2}$$

The value of η depends on factors quite different for different materials, and no generally valid figures can be given. For metals, η = 1.0 may be used and η may, therefore, be omitted in the above equation.

By introducing the partial coefficient γ_M, uncertainties in the strength of the material are taken into consideration as caused by:

- the normal scatter of the material strength,
- the variability of the factor or function η which translates the strength of test specimens into strength of the structure.

For practical reasons other factors not directly related to the strength of the material are taken into consideration by γ_M. Such factors are:

- deviations of dimensions and geometry from the nominal values assumed in the design analysis, if such deviations are not considered elsewhere,
- unreliability of the model of analysis, if kept within reasonable limits.

The partial coefficients used in ENV 1999-1-1 is different for resistance of members and connections. They are, however, boxed values.

Resistance of class 1 cross sections:	$\gamma_{M1} = 1{,}10$
Resistance of class 2 or 3 cross sections:	$\gamma_{M1} = 1{,}10$
Resistance of class 4 cross sections:	$\gamma_{M1} = 1{,}10$
Resistance of member to buckling:	$\gamma_{M1} = 1{,}10$
Resistance of net section at bolts holes:	$\gamma_{M2} = 1{,}25$

Resistance of bolted connections:	$\gamma_{Mb} = 1{,}25$
Resistance of riveted connections:	$\gamma_{Mr} = 1{,}25$
Resistance of pin connections:	$\gamma_{Mp} = 1{,}25$
Resistance of welded connections:	$\gamma_{Mw} = 1{,}25$
Slip resistance connections:	
- ultimate limit state:	$\gamma_{Ms,ult} = 1{,}25$
- serviceability limit state	$\gamma_{Ms,ser} = 1{,}10$
Adhesive bonded connections:	$\gamma_{Ma} \geq 3{,}0$

Models of analysis

The calculations used in the design are based on models by means of which the behavior of the structure is described. The models of analysis may be more or less complicated and provide a more or less accurate description of the function of the structure. Often a model giving a higher accuracy turns out to be more complicated. In certain cases the nature of the problem demands a more sophisticated model, e.g. for stress analysis in structures subjected to fatigue. Usually, there is an option, however, between different models and the choice has to be made on an economic basis, which applies to the cost of material/construction in relation to the cost of the design analysis.

Models of analysis should be considered as approximate descriptions of the function of a structure. Even the most advanced models are thus subject to some uncertainties. With regard to this fact numerical values of coefficients etc. should be chosen in such a way that the model gives results on the safe side. But it is often not feasible to enter such values of the coefficients that the results are conservative in all conceivable cases.

Probabilistic aspects may be introduced, choosing the strength coefficients in such a way that the model gives results on the unsafe side only in a small fraction of the cases. This fraction should not exceed 5 per cent. The resulting resistance may thus be interpreted as a characteristic value.

2204.07 Design Criteria

The load and resistance factor method

The load and resistance factor method is briefly described in **2204.04.** The method is applied in many design specifications and is sometimes referred to as the method of partial coefficients. According to this method the characteristic values of loads and resistance are first determined. Then the design values are obtained by:

- multiplying the the characteristic values of the loads by the load factor γ_F,
- dividing the characteristic values of the resistance by the resistance factors γ_M,.

The design analysis should verify that the stresses caused by design loads σ_{Sd} (or section forces M_{Sd}) are smaller than the design value of the resistance expressed in terms of the same quantity (σ_{Rd}, or M_{Rd}), i.e.

$$\sigma_{Sd} < \sigma_{Rd} \qquad (7.1)$$

where σ_{Sd} = stress caused by the load: $\Sigma\gamma_G G_k + \Sigma\gamma_{Qi} \Psi_{0i} Q_{ki}$

$$\sigma_{Rd} = \frac{f_k}{\gamma_M} \qquad (7.2)$$

f_k = characteristic strength, refering to a limit state

γ_M = resistance factor considering uncertainties in the material parameters and tolerances for dimensions.

Method of allowable stresses

A safety factor should consider the unreliability of load assumptions as well as the unreliability of resistance values. Since uncertainties of the methods of analysis are included in the estimation of the resistance, a moderately low safety factor may be chosen, normally 1.5 for normal types of loading.

The allowable stress σ_{all} is thus determined as

$$\sigma_{all} = \frac{f_k}{s} \tag{7.3}$$

where f_k = the resistance according to this course.

s = safety factor, normally 1.5.

The allowable stress shall be higher than the stress determined from loads without load factors i.e.

$$\sigma < \sigma_{all} \tag{7.4}$$

2204.08 Aluminium Alloys as a Structural Material

Most of the structural aluminium alloys have relatively high strength compared to the modulus of elasticity. A comparison between different aluminium alloys and tempers and some other materials shows the table in **Figure 2204.08.01**.

Strength ($R_{p0.2}$) and Modulus of Elasticity (E) for Some Metals

Material	$R_{p0.2}$	E	$E/R_{p0.2}$
AA 5083-0	125	70000	560
AA 5083-H321	220	70000	318
AA 6082-T6	270	70000	259
AA 7108-T6	360	70000	194
St 42	260	210000	808
St 52	360	210000	583
Concrete C45	28	28000	994
Timber	20	9000	450

- Aluminium has high strength compared to modulus of elasticity, especially strain hardened and heat treated alloys
- Steel structures are often designed in the ultimate limit state
- Aluminium structures are mostly designed in the serviceability state (deflections)

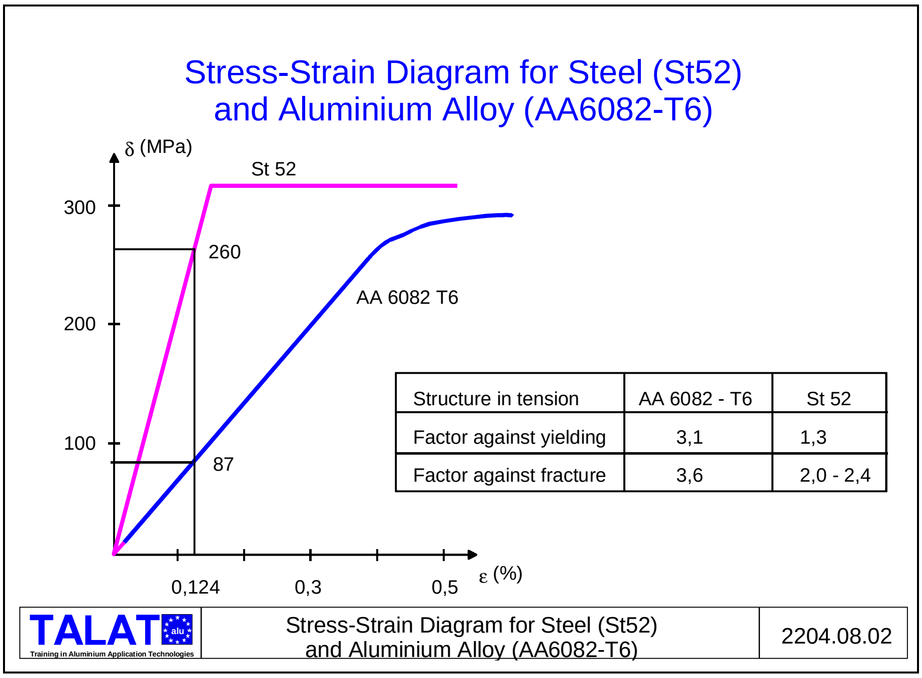
TALAT Training in Aluminium Application Technologies | Strength ($R_{p0.2}$) and Modulus of Elasticity (E) for Some Metals | 2204.08.01

This effect is especially clear when the aluminium alloy is strain-hardened or heat-treated. Structural aluminium alloys have roughly twice the strength of steel compared to the modulus of elasticity.

Steel designers often use the strength of the material when designing a steel structure, and than check if the deflection is within the requirement.

When designing an aluminium alloy structure, it will often be the deflection criteria which is governing. The design procedure will for that reason be designing according to the deflection criteria or stability and than check the stress or the bearing capacity of the structure.

Comparing steel and aluminium alloy members in tension with the same elastic strain, the steel member will have 3 times the stress of the aluminium alloy member, see **Figure 2204.08.02**.



The stress in an aluminium alloy structure designed according to the deflection criteria is very often low. A steel structure will usually be designed according to strength criteria. **Figure 2204.08.02** shows stress strain curves for an aluminium alloy member of 6082-T6-alloy and a steel member of St 52. The example shows different stress in the members for the same strain, caused by the difference in the modulus of elasticity.

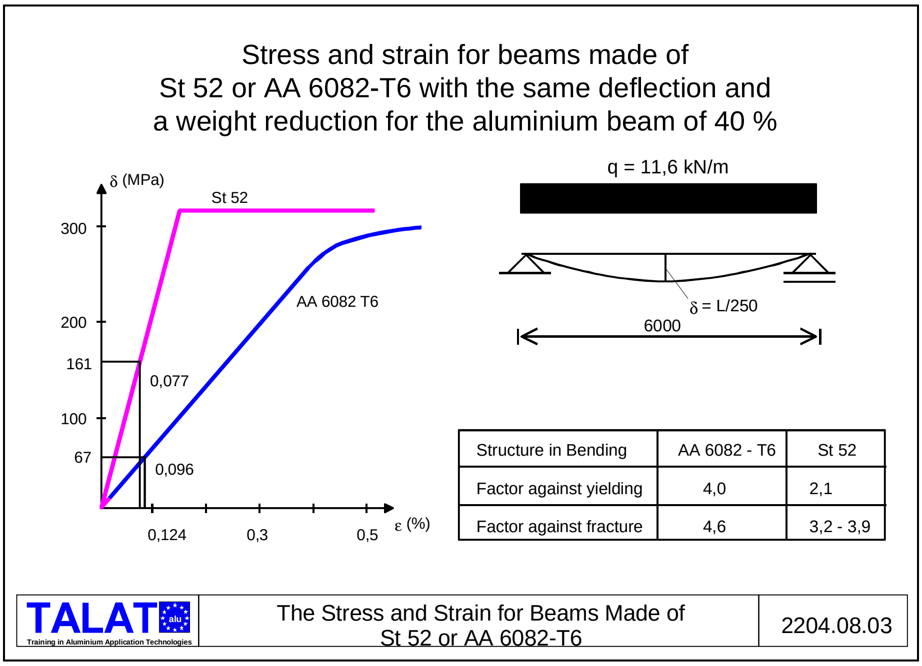
A structure or member in tension designed according to the deflection criteria will usually be in this situation. The safety against yielding and fracture will in this example be:

	AA 6082 - T6	St 52
Factor against yielding	3,1	1,3
Factor against fracture	3,6	2,0 - 2,4

Comparing members in steel and aluminium in bending, the shape of the members will be different. At the same deflection, the strain will be different. In **Figure 2204.08.03** this is illustrated for a 6,0 m long beam with a distributed load of 11,6 kN/m and a deflection of l/250. For this example we will have the following factors against yielding and fracture:

	AA 6082-T6	St 52
Factor against yielding	4,0	2,1
Factor against fracture	4,6	3,2 - 3,9

Because of the relatively low modulus of elasticity of aluminium alloys compared to their strength, the safety of designing an aluminium alloy structure to the deflection criteria, is very high and usually higher than a steel structure.



The Stress and Strain for Beams Made of St 52 or AA 6082-T6 2204.08.03

The deflection of members in bending are dependent on the modulus of elasticity (E) and on the moment of inertia (I) together with the load and the span. With the same span and load, it will be the product E • I which will determine the deflection.

To get the same deflection of steel and aluminium alloy beams in bending, the moment of inertia of the aluminium alloy beam must be three times that of steel. If the increase in the moment of inertia is to be done only by increasing the thicknesses of the web and flanges the aluminium alloy beam will have the same weight as the steel beam. To save weight the aluminium alloy beams in bending have to be higher. An example will illustrate this:

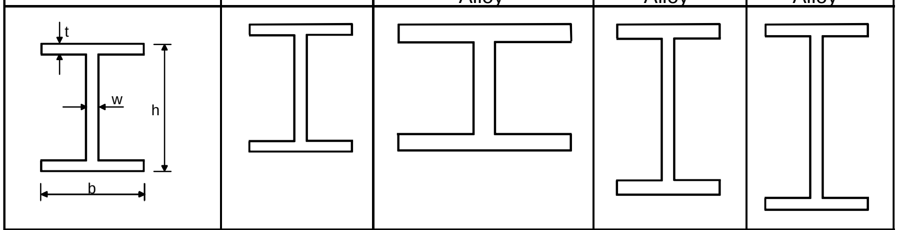
An aluminium alloy beam shall have the same deflection as an IPE 240 steel beam. The moment of inertia of the IPE 240-beam is $38{,}9 \cdot 10^6$ mm^4 about the strong axis. The weight of this beam is 30,7 kg/m. The aluminium alloy beam must have a moment of inertia of 116,7 · 10^6 mm^4 to get the same deflection.

If the height of the aluminium alloy beam shall be 240 mm, this will be satisfied by an I-beam of I240 x 240 x 12 x 18,3, which has a moment of inertia of I = 116,6 · 10^6 mm^4 and a weight of 30,3 kg/m (approximately the same weight as the steel beam). If the height of the aluminium alloy beam can be 300 mm, the deflection criteria will be satisfied by an I300 x 200 x 6 x 12,9 which has a moment of inertia of 116,7 · 10^6 mm^4 and a weight of 18,4 kg/m which is a weight saving of 40%.

An I330 x 200 x 6 x 10 will have a moment of inertia of 117,3 · 10^6 mm^4 and a weight of 15,8 kg/m which give a weight saving of 49%.

These three different aluminium alloy beams will give the same deflection as an IPE 240 steel beam. It will be the shape and stability of the beam which will determined the weight of the beam. **Figure 2204.08.04** shows the beams and the weight savings.

Comparison between four beams which will give the same deflection

	Steel	Aluminium Alloy	Aluminium Alloy	Aluminium Alloy
				
Moment of inertia in mm^4	38,9 E 6	116,6 E6	116,7 E6	117,3 E6
EI (N/mm^2)	8,17 E12	8,16 E12	8,17 E12	8,21 E12
h (mm)	240	240	300	330
b (mm)	120	240	200	200
t (mm)	9,8	18,3	12,9	10
w (mm)	6,2	12	6	6
g (kg/m)	30,7	30,3	18,4	15,8

TALAT Training in Aluminium Application Technologies | Comparison between four beams which will give the same deflection | 2204.08.04

2204.09 References/Literature

[1] : Lars Østlund. Handboken Bygg (in Swedish)

[2] : CEN/TC 250/SC 1: ENV 1991-1. Eurocode 1 Basis of design and actions on structures. Part 1: Basis of design. 1994

[3] : CEN/TC 250/SC 9: ENV 1999-1-1. Eurocode 9 Design of aluminium structures. Part 1-1. General rules. 1997

2204.10 List of Figures