Uncertainties and Human Errors in the Design and Execution of Steel Structures

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Abstract

This paper contains a compilation of results from some 50 000 steel specimen tests and close to 5 000 measurements of the cross-sectional properties of rolled steel members. Based on the statistical distribution of these properties the statistical distribution of the sectional capacity of such steel members is evaluated using a numerical integration procedure. For standard structural steel members the variations of the strength properties are reasonably well-known and may be used in reliability assessment methods for the design of structures. However, it has been observed in many actual failures with steel structures that the cause of such failures normally is one gross human error, rather than a combination of "normal" variations in parameters affecting the actions and response of the structures. Another observation from failures experienced with steel structures is that gross human errors in execution are more critical than gross errors in the design process.

Keywords: Steel structures; variations in strength; cross-sectional properties; structural failures; gross human error; design; execution; inspection.

1 Introduction

This paper will treat uncertainties in the parameters affecting the load-carrying capacity of structural steel members. Results for the yield strength from about 53 000 tensile specimen tests made as mill tests at SSAB and elsewhere are reviewed [1]. Cross-sectional dimensions of hot-rolled I- and H-shapes produced according to European standards were measured for a large number of different members. Variations of various cross-sectional properties of such members, for instance, the plastic modulus Z (W_{pl}), have been derived from the measured cross-sectional dimensions.

From the statistical variations of the steel strength and of the cross-sectional dimensions the statistical variation of cross-sectional capacity, for instance, the plastic moment capacity M_{pl}, has been derived using a simple numerical integration procedure. Since no mention of such a simple procedure has been found in the literature the procedure is outlined in this paper.

The data for structural member capacity summarized in this paper may be used in reliability assessment methods for the design of structures. For design situations with more complicated relationship the data contained here may be use to facilitate structural design employing for instance Monte Carlo simulation, such as the SBRA technique derived by professor Pavel Marek and others [2]. The advances in computer technology makes possible practical design applying more probabilistic methods than presently used.

However, it has been observed in most actual failures with steel structures that the cause is a gross human error, rather than a combination of "normal" variations in parameters affecting the actions and response of the structures [3, 4].

Thus, in order to arrive at safe structures it is necessary to limit the risk of failure due to an unfavorable combination of variation in load action and structural capacity. However, this is not sufficient since effects of gross errors normally may not be modeled in the variation of the relevant parameters in a reliability assessment. For a truly probabilistic design of structures we have to address the matter of gross human errors in a more rational way. The most rational manner to account for the effect of gross errors in steel construction is to improve means and procedures for a knowledgeable and effective check of design endeavors and, in particular, inspection of the execution of steel structures. Some comments on such means and procedures are given in the paper.

2 Uncertainties in structural steel member properties

2.1 Variations in steel strength

For a probabilistic assessment in the structural design of real structures it is important to have a fair knowledge about the variation of relevant variables affecting the strength and serviceability criteria of the structure. The results of even a very advanced simulation using finite-element computer software are not more reliable than the input data supplied by the user.

Much data for the variables entering the design of load-carrying steel structures has compiled in the literature. It appears to the writer that much data furnished in various textbooks on the use of simulation techniques are fine for demonstration and educational purposes, but they may not be appropriate for accurate design of real structures. The writer would like to draw attention to some data for European structural steel grades and hot-rolled members produced according to European standards which has been published previously [1] but may not be easily found in the readily available literature.

A compilation of data on the yield strength from some 53 000 tension tests of structural steel from various steel mills is summarized in Figure 1 for four different steel grades, corresponding to the present S235X grade up to and including S420X. The yield strength as defined here is the yield strength level on the yield plateau, which is more

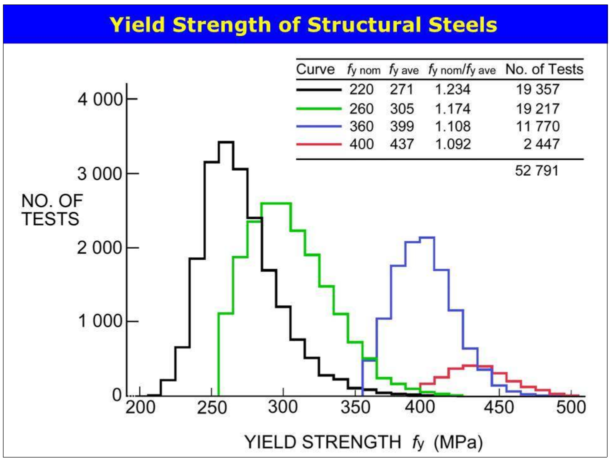

Figure 1 Distribution of yield strength of about 53 000 tension specimen tests for four steel grades approximately equivalent to S235X, S275X, S355X and S420X according to EN 10025-2

adequate for most design purposes than the upper yield point often reported in conventional tensile test results.

As can be seen in the histograms and the table in Figure 1 there is on the average a considerable reserve in the strength over the nominal (specified) value. For the lowest steel grade in Figure 1, equivalent to S235X according to EN 10025-2, the average of close to 20 000 tensile tests is 23 percent above the nominal value. This reserve is smaller for higher strength steels. For the steel comparable to the mostly used steel grade today, S355X, the average is 11 percent above the nominal value.

It is also evident from Figure 1 that the variation of yield strength is skewed to the right for all grades shown. That is, the variation may not be accurately represented by a normal distribution. A principal reason for the skewness of the actual strength distributions is the fact that the process of manufacturing structural steels in a modern steel mill is kept under strict control, for instance by adding alloying elements such as manganese etc. as required to fulfill specifications. It will normally be more economical for the steel producer to add a little extra alloying elements than to risk having a complete charge not fulfilling criteria.

The skewness of the distribution and the properties at the lower tail of the distribution are of particular importance with respect to the safety of steel structures and reliability assessment methods. These properties are connected to various statistical distribution types. It may be demonstrated that a lognormal distribution as well as an extreme-value type I distribution will give good fit to the recorded date [3]. Figure 2 shows an example for the probability density function PDF and the cumulative distribution function CDF assuming a lognormal distribution fitted to the recorded data for the steel grade equivalent to S235X as shown in Figure 1. The scales of the CDF have been transformed in such a way that the lognormal distribution would plot as a straight line.

In the following will be demonstrated the evaluation of member properties for H- and I-shapes as a combined effect of variation in yield strength and variation in cross-sectional members. Variation of yield strength for such members has been extracted from the data in Figure 1 and is shown in Figure 3. It appears from the diagram that the size of the rolled member, here signified by the flange thickness, is a statistically significant parameter for the variation in the yield strength of the members.

2.2 Variations in cross-sectional properties of rolled H- and I-shapes

The capacity of a structural member under various types of loading is dependent upon its cross-sectional properties. Figure 4 shows the variation of cross-sectional dimensions as measured on 4 816 hot-rolled H- and I-shapes produced at several European steel mills [1].

The variation in thickness of the flange and web is much greater than the relative variation in height and depth of H- and I-shapes measured. Contrary to the variation of yield strength, it appears that the variation in thickness of flanges and web could be approximated by a normal distribution.

There is a tendency of the flanges on the average being thinner and the web being thicker than the nominal. The reason for this is probably an economical optimization in the adjustment and replacement of the rolls at the rolling mill due to wear, within the specified geometrical tolerances for the produced members. Rolling tolerances are specified both for individual cross-sectional dimensions and for cross-sectional area of the member, which means that smaller thickness than nominal of the flanges may to some extent be matched with greater thickness of the web, creating a member cross section with a larger web-to-flange thickness ratio.

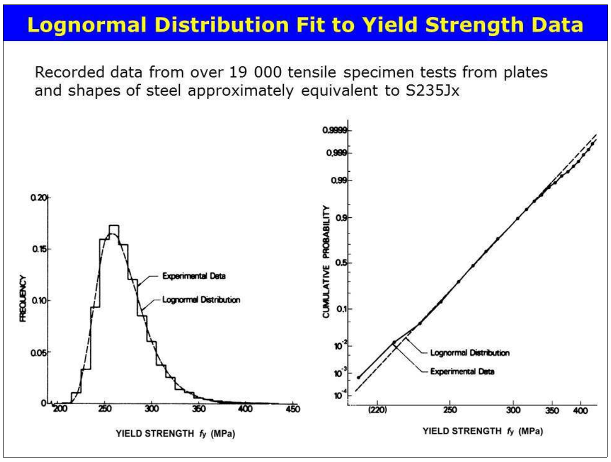

Figure 2 Example of probability density function PDF and cumulative distribution function CDF of a lognormal distribution fitted to the recorded data for steel equivalent to S235X

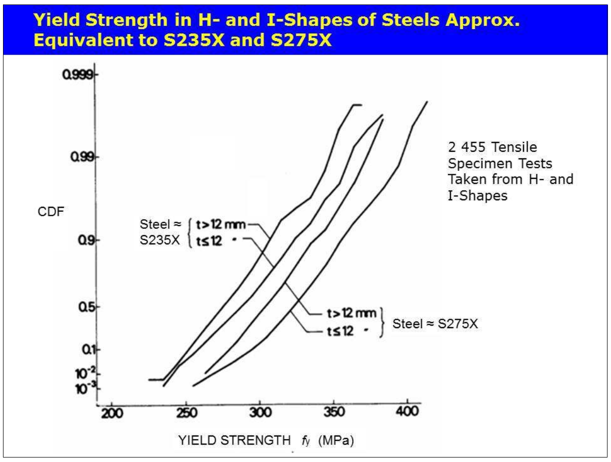

Figure 3 Data for yield strength in hot-rolled H- and I-shapes in structural steel grades equivalent to S235X and S275X

Figure 5 shows the cross-sectional properties *A*, *I*, *W* and *Z* (W_{pl}) calculated from the measured dimensions of 4 816 hot-rolled H- and I-shapes as given in Figure 4. The properties for *I*, *W* and *Z* have been calculated for the strong axis (bending about an axis parallel to flanges) as well as the weak axis (bending about an axis parallel to web).

All cross-sectional properties in Figure 5 show a similar variation, which is mainly caused by the variation in the thickness of the flanges. The variation of the properties about the weak axis falls in all cases somewhat below those of the strong axis. The variations in cross-sectional properties fit reasonably well a normal distribution.

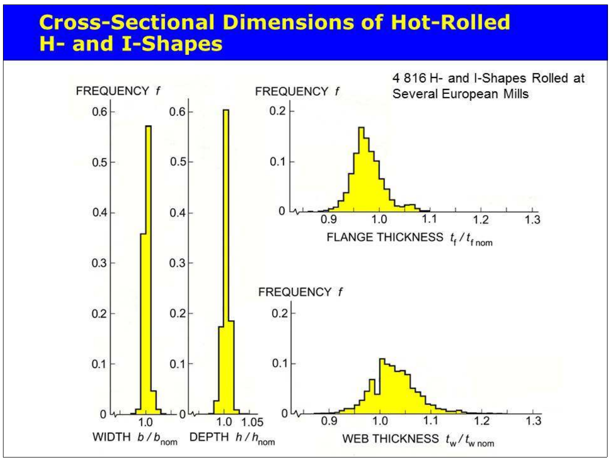

Figure 4 Variation of cross-sectional dimensions measured in 4 816 hot-rolled H- and I-shapes of structural steel produced at various European steel mills

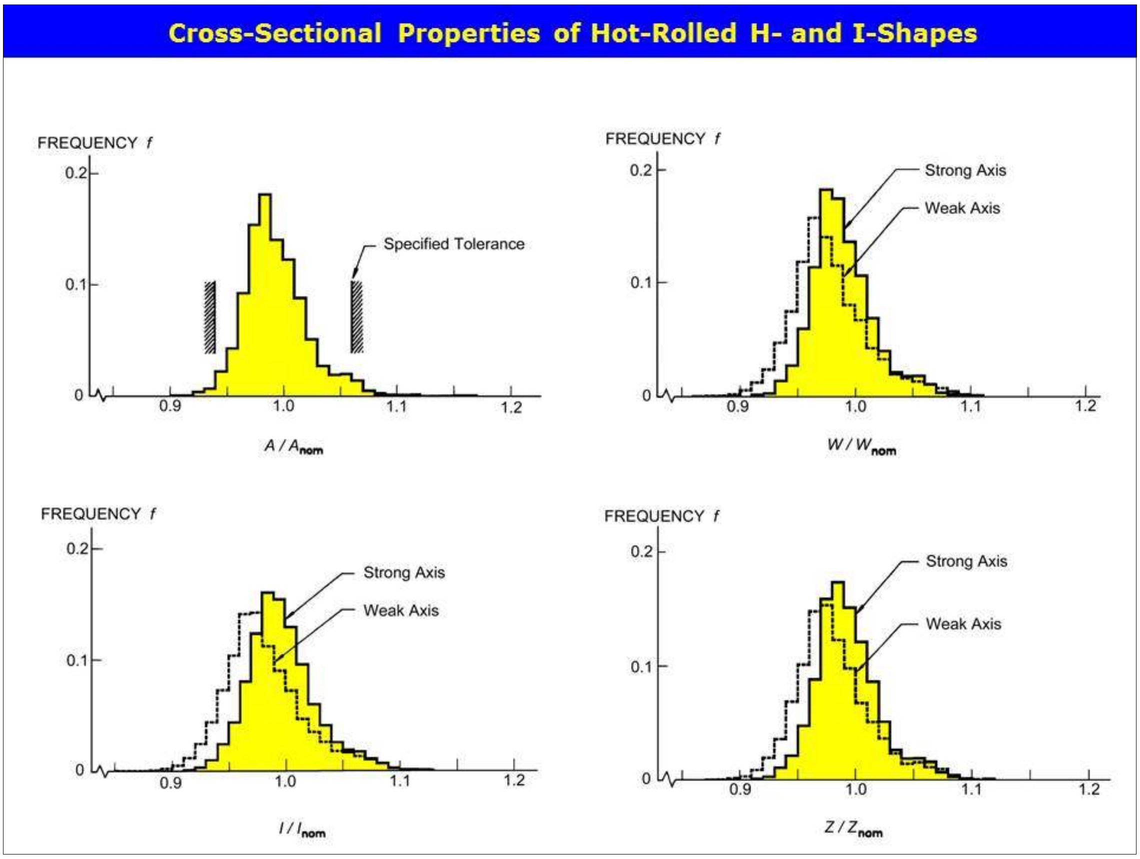

Figure 5 Statistical variation of cross-sectional properties for area A, moment of inertia I, elastic modulus W and plastic modulus Z (W_{pl}) of hot-rolled H- and I-shapes, determined from the measured dimensions shown in Figure 4.

2.3 Variation of sectional capacity

The variation of the sectional capacity of a member is a function of the variation of the yield strength and in cross-sectional properties. For instance, the plastic moment capacity of a member is $M_{pl} = f_y \cdot W_{pl}$. Assuming the variation of the yield strength and of the various cross-sectional properties to be independent stochastic variables, the variation in the plastic moment capacity M_{pl} could be assessed for instance using a Monte Carlo simulation.

However, for a simple relationship like Z = X Y (or Z = X / Y, Z = X + Y or Z = X - Y), where X and Y are independent stochastic variables, the distribution of Z could easily be determined by a direct numerical integration technique of histogram data for X and Y. This would require much less computing resources than for instance a Monte Carlo simulation. Since this simple technique has not been found referred to in the literature, it will be demonstrated here.

Consider the relationship Z = X Y, where X and Y are independent statistical variables. The data on the variables X and Y are grouped into histograms, Figure 6, where each variable value x_i has a frequency value of $f_{x\ i}$ and each variable value y_j has a frequency value of $f_{y\ j}$, the area of each histogram being equal to 1. X might represent for instance the normalized variable yield strength $f_y / f_{y\ nom}$ and Y the normalized variable for the plastic modulus $W_{pl} / W_{pl\ nom}$. The resulting variable Z will then represent the variation of the normalized variable $M_{pl} / M_{pl\ nom}$.

The data in the histograms for X and Y should be grouped in such a manner that the class middle values and the class limits for X and Y are chosen on a logarithmic scale with a suitable class width dependent of the number of records available and the required accuracy. For instance, the class middle values of variables X and Y might be chosen
... ln 0.98, ln 0.99, ln 1, ln 1.01, ln 1.02 ...
and the corresponding class limits in-between. For simplicity the class number corresponding to the value 1 for the variables X and Y is set to i = 0 and j = 0.

A histogram defining the statistical distribution of

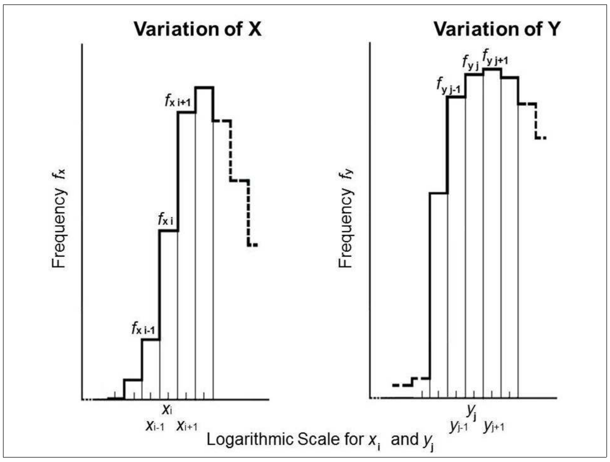

Figure 6 Numerical technique to determine the distribution of the variable Z in the relationship Z = X Y, where X and Y are independent stochastic variables

Z can now be arrived at by a summation over all values i with j = k - i according to the relationship

$$f_{z\,k} = \sum f_{x\,i} \cdot f_{y\ k-i} \quad \ldots\ldots\ldots\ldots \quad (1)$$

The class number k = 0 for the variable Z corresponds to the value 1 of the variable.

If the variables X and Y are not completely independent, then it is possible to divide the data into groups, see further below.

A similar technique may be employed for determining the distribution of the variable Z in the relationship Z = X / Y, where X for instance might be the distribution of the load action and Y the distribution of the capacity of a structural member.

The technique outlined here has been used to determine the distribution of the plastic moment capacity for strong axis bending of hot-rolled H- and I-shapes of structural steel equivalent to S275X. Because the yield strength and the sectional properties (in this case Z, that is, W_{pl}) are not statistically independent variables the data has been grouped here in three different categories for the flange thickness of the shape. The results are given in Figure 7.

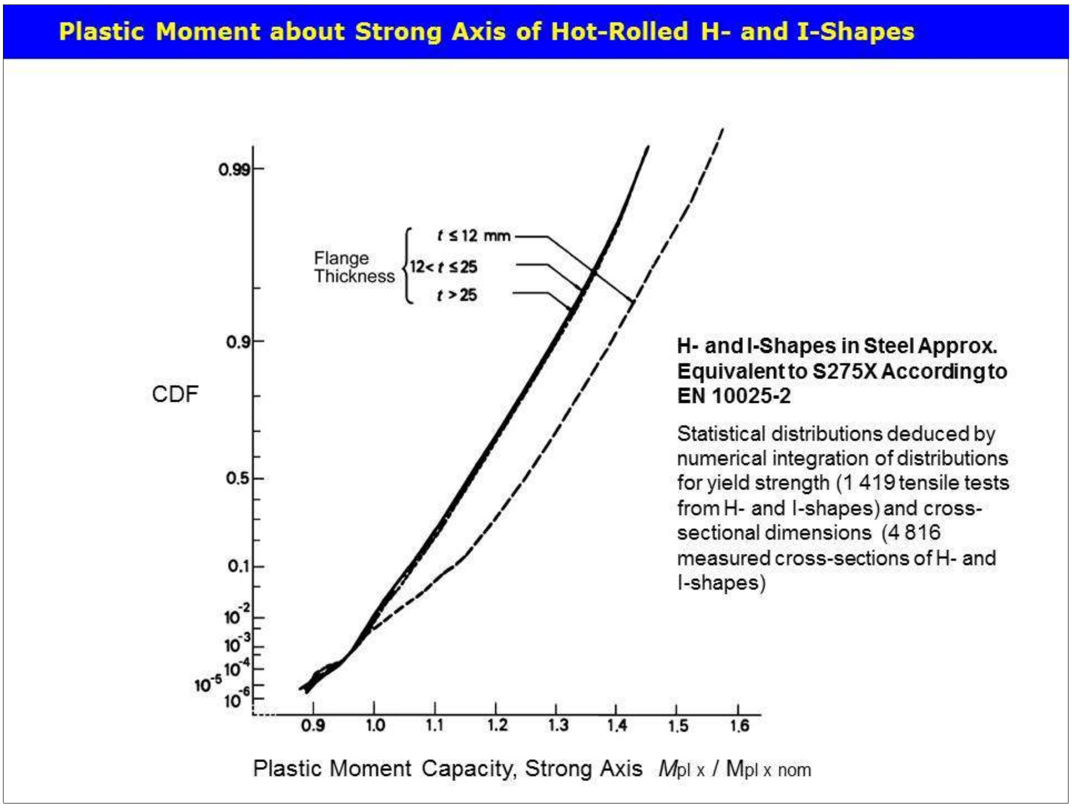

Figure 7 Variation of plastic moment capacity $M_{\mathrm{pl\,x}}$ about the strong axis of hot-rolled H- and I-shapes in structural steel equivalent to S275X, deduced from the data for variation of yield strength and the plastic modulus shown in Figures 3 and 5. Ordinate scale transformed in such a way that an extreme-value distribution would plot as a straight line

The diagram in Figure 7 indicates that there is not much deviation between the three curves for the different flange-thickness groups in the left-hand area of the diagram for $M_{\mathrm{pl\,x}}$ / $M_{\mathrm{pl\,x\,nom}}$ values below or approximately equal to 1. The probability of variable values being lower than 1 is in the range of $5{\cdot}10^{-3}$ to 10^{-2}. Thus, the great probability of the cross-sectional variable Z_x being less than its nominal value is to a considerable extent matched by a great tail to the right of the strength variable f_y. On the other hand there is a great difference between the three curves in the right-hand side of the diagram, which will be of significant importance when determining the probability of failure considering also the variation of the load action. The variable value corresponding to 50 percent probability in the distribution is about 15 to 25 percent larger than the nominal value, with the highest value for small shapes with flange thickness up to 12 mm.

2.4 Other uncertainties in properties of steel structures

The load-carrying capacity of steel structures is dependent on many other variations in the actual properties of the structure and its members. These include geometrical deviations, such as curvature of members, slope of columns and eccentricities at joints, and residual stresses.

Considerable data is available for geometrical deviations of structural steel members, as well as for residual-stress distributions in various hot-rolled as ell as welded steel members. It is however beyond the scope of this paper to review such data.

2.5 Uncertainties in structural models

Uncertainties may also be introduced in the design of steel structures from simplifications in the modelling of the structure. The effect of such uncertainties may be reduced by applying a more rational structural model and assessment of the structural behavior by simulation techniques, for instance, using Monte Carlo simulation.

The use of simulation techniques is possible through the advances in computer technology. All major structures today are designed using computers, in one way or another. While today's computers present a formidable tool for the design of structures there is also a potential danger that the experience and engineering judgment of a skilled designer may be replaced by an uncritical adoption of nicely presented results from a computer program, the assumptions and limitations of which the designer is not completely aware of. The writer has seen too many examples of this in practice, in particular in relation to the use of computer software for design and presentation on drawings and project specifications of steel structures.

So why use simplistic structural models of the slide-rule era, such as simplified interaction formulae in regulations, when computers are at hand? Numerical models for simulation of, for instance, the buckling behavior of a steel column with actual distribution of residual stresses, variation of yield strength over the cross section, out-of-straightness of the column (which may not always be best represented by a perfect sine curve), end eccentricities etc. are available but appear presently not to be extensively used in the structural design of actual structures. Combining finite-element models, consideration of uncertainties affecting the capacity and load effects and Monte Carlo simulation should allow practical probability-based design of structures using realistic models for the structural behavior. The advantages would include a more economical use of material and other resources and a better understanding of the structural behavior and the influence of various uncertainties. A spin-off effect could be that especially disadvantageous uncertainties could be identified and hopefully attended to.

3 Human errors in design and execution

A general conclusion drawn from the investigation of actual failures with steel structures is that a gross human error, in a few cases a combination of two gross human errors, is the major cause of collapses, incidents and serious structural damage cases occurring in reality [3, 4]. Thus, gross human error is the main cause for collapses of such structures rather than a combination of unfortunate variations in parameters affecting the actions and response of the structures, as may be considered in the probabilistic and semi-probabilistic design of structures.

It is usually not possible or realistic to include the effects of gross errors in the variation of variables in a semi-probabilistic or probabilistic assessment of the capacity of a structure. A safe structure would require that the design is based on reasonable assumptions on the variables affecting the capacity, and that the effects of gross human error is treated separately.

During the winter 2009/2010 about 3 000 roofs collapsed in Sweden because of unusually heavy snow loads [5]. Most of the collapses were in the agricultural industry. About 40 percent of the collapses were in buildings with steel structures. The writer participated in the investigation of a few of these roof collapses with steel structures, including a big hall building with a soccer field. As far as known to the writer no collapse involved snow loads greater than the nominal snow load times the partial load factor to be considered by the structural designer. The major cause to the collapses were probably that the load-carrying structure had not been professionally checked by a structural designer at all. In other cases gross human error was the main cause.

3.1 Design

Of the over 400 structural collapses, incidents and serious structural damage cases investigated by the writer [4], less than 10 percent could be attributed to incorrect structural design. However, although the total number of such events is relatively few, the consequences in these particular instances is very serious. Of the few collapses involving loss of human life or serious human injuries grave design errors is a major cause.

The most effective way to reduce the risk for gross human errors in the design process is to introduce a knowledgeable and careful scrutiny of the endeavors of the structural designer, such as the structural calculations, drawings and project specifications. Such a check should include the following items:

- that basic assumptions made are in accordance with the actual conditions and requirements
- that choice of materials and assumptions of their properties are relevant
- that assumptions on the loading and other actions are relevant
- that the structural models adopted corresponds to the real structure, are relevant and correctly used
- that numerical calculations, including those applying computers, are relevant and correctly executed
- if the design has incorporated any tests, that such tests are relevant for the actual conditions
- that the results of the structural calculations are correctly transferred to the drawings and other documents used for the production.

3.2 Execution

Human errors in the execution of steel structures are more prevalent as cause for failures than errors in the design process [4]. Human errors appear particularly serious in causing collapses from structural instability during the construction phase.

The most effective way to reduce the risk for gross human errors in the execution of steel structures is to introduce a knowledgeable and careful general control and inspection of the complete process applied in producing the structures, that is, in planning, fabrication in the workshop, assembling and erection on the site.

4 Summary and conclusions

Some statistical data on the variation of parameters affecting the strength of steel structures has been discussed in the paper. From a compilation of the yield strength of steel (yield strength at the yield plateau level) of about 53 000 steel specimen tests it may be concluded that the average value of the yield strength of normal structural steel grades is 9 to 23 percent higher than the nominal strength. The statistical distribution is skewed to the right and a log-normal distribution fits well to the recorded data.

Results from a large number of measurements of the cross-sectional dimensions of hot-rolled steel H- and I-shapes have been reviewed and the corresponding cross-sectional properties calculated. There is a tail with property values less than the nominal value, mainly because of low values for the flange thickness of such members.

For the statistical evaluation of the variable Z in a simple relationship like $Z = X\,Y$, where X and Y are stochastic variables, it is possible to make a simple numerical procedure, which is outlined in the paper. This procedure is simpler and more direct than for instance applying a Monte Carlo simulation, but only applicable to simple relationships like $Z = X\,Y$, $Z = X / Y$, $Z = X + Y$ or $Z = X - Y$.

Based on the statistical distribution of the yield strength and the cross-sectional properties the sectional capacity of steel H- and I-shapes has been evaluated for the plastic moment capacity about the strong axis using this numerical integration procedure. Although there is a reasonably large tail of the cross-sectional properties with values smaller than the nominal value, there is only a small risk that the sectional capacity such as the plastic moment capacity falls short of the nominal value. The mean value (50 percent probability) is 15 to 25 percent larger than the nominal value.

For standard structural steel members the variations of the strength properties and of the capacity of member cross section are reasonably well-known and may be used in reliability assessment methods for a probabilistic design of steel structures.

With the advances of computers today it is possible to perform practical design analysis of load-carrying structures considering the variability of parameters affecting the capacity and load action. Such reliability-based assessment methods can be based on Monte Carlo simulation, such as the SBRA technique developed by Professor Pavel Marek and others [2].

However, it has been observed in many actual failures with steel structures that the cause of such failures normally is one gross human error, or in some cases a combination of two gross errors, rather than a combination of “normal” variations in parameters affecting the actions and response of the structures, as discussed above [3, 4].

Another observation from failures experienced with steel structures is that gross human errors in execution are more critical than gross errors in the design process [4]. Human errors appear particularly serious in causing collapses from structural instability during the construction phase.

The rational way to account for this finding is to implement in all construction projects a carefully designed and knowledgeable quality control and inspection of the execution of steel structures, instead of just a routine inspection by someone not well informed about the background and consequences of deviations with respect to the safety of structures.

Based on experience from failures several changes would be required in regulation and rules for the design and execution of steel structures.
Since most failures in steel structures are caused by gross human errors, in particular, in the construction phase, it seems imperative to extend in regulations and rules the requirements for competence of the individual performing checking of design and inspecting execution of steel structures.

In the current European standard for execution of steel structures, EN 1090-2 [6], there is a requirement for 100 percent visual inspection of welds. However, there are no detailed and relevant competence requirements specified for the individual performing such tasks. Other deficiencies than weld discontinuities are of great concern with respect to the safety of steel structures with (quasi-)static loading, especially parameters affecting structural instability. It seems important to widen requirements and the competence of the inspectors to cover not only the welds of the structure but the complete technology of steel structures.

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