Assessment of Existing Structures
according to the JCSS recommendations

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Abstract

The Joint Committee on Structural Safety (JCSS) has published a document on the reliability assessment of existing structures, which has been worked out by the members of the working party of the JCSS. The scope of the document is to provide guidelines and relevant information on how a reliability based assessment of an existing structure should be performed. The present paper gives an introduction to the document which includes general guidelines on reassessment, methodologies for reliability updating, acceptability and target safety criteria, examples and case studies.

1. Introduction

In 1971 the Liaison Committee which co-ordinates the activities of six international associations in Civil Engineering FIB, CIB, ECCS, IABSE, IASS and RILEM, created a Joint Committee on Structural Safety (JCSS), with the aim of improving the general knowledge in structural safety. After a reorganisation in 1992 the JCSS set as a long term goal the development of a probabilistic model code for new and for existing structures.

This paper gives an introduction to the JCSS document on existing structures which is published under RILEM [1] and has been worked out by the members of the working party of the JCSS. The contents of the document such as general guidelines on reassessment, methodologies for reliability updating, acceptability and target safety criteria, examples and case studies, are briefly described. The scope of the document and the benefits related to its applicability are outlined. Examples and a case study are included in this contribution. It is noted here that the probabilistic model code and the associated stochastic models is published in the internet [2].

2. Objective

The need to assess the reliability of an existing structure may arise from a number of causes among which the most common are:

- deviations from the original project description;
- adverse results of a periodic investigation of its state;
- doubts about the structural safety caused by evidence of damage;
- unusual incidents during use (such as impact of vehicles, avalanches, fire in the building, earthquakes), which could have damaged the structure;
- inadequate serviceability (for example large deformations);
- suspicion of possible impairment of the structural safety related to building materials, to construction methods or to the statical system;
- the discovery of design or construction errors;
- a planned change of the use of the structure;
- expiry of a residual service life granted on the basis of an earlier assessment of the structure.

A typical actual example is the reassessment of roofs under the extreme snow load. During the reassessment procedures typical questions which need to be answered are:

- What type of inspections are necessary?
- What analyses shall be performed?
- What are the risks involved in further using the structure?
- What are the risk acceptance criteria to be considered?
- What type of measures shall be taken?

Such answers cannot be given by using a classical code approach. In addition one key point is that new information becomes available related to the state of the existing structure. Therefore there is an increasing need and consequently an increasing tendency to use probabilistic methods in the assessment of existing structures. The scope of the JCSS document is to provide such reliability based procedures and to illustrate them in characteristic examples and case studies.

3. Contents

The JCSS document is of educational type and provides reliability methods to be used in the structural reassessment. Tutorial examples and practical case studies are included as shown in the contents, which are as follows:

The document provides relevant information on how to process specific information about an existing structure, how to update its reliability based on such information, how to base decisions regarding maintenance, strengthening, upgrading etc. It is generally applicable for various materials and various structure types. The chapters of the JCSS document and the associated guidelines are summarized in the following paragraphs.

4. Part 1: General

This part includes basic definitions such as residual service life, hazard scenarios, safety plan, information updating, etc. Basic concepts on inspection and maintenance are described. Two types of inspection are in general distinguished:

- qualitative inspection: this type of information is related to the observation of parameters such as surface characteristics, visible deformations, cracks, spalling, corrosion etc.
- quantitative inspection: this type of information results in a set of values of parameters that characterize the condition of the structural elements, such as crack widths.

For both inspection types the related uncertainties such as the probability to detect some damage and/or the accuracy of the results are addressed.

Maintenance is defined as a set of activities that are carried out to retain or restore a structure in an operable state. The following types of maintenance are distinguished:

- Corrective maintenance: no inspection is carried out and repair is done after partial failure has occurred.
- Preventive maintenance: no inspection is carried out but replacement or maintenance at a time that no failure has occurred.
- Condition based maintenance: inspections are planned in advance and when measured parameters no longer meet prescribed criteria repair or replacement must be carried out.

Decision criteria which serve as a basis of the decision regarding the requalification of an existing structure are analysed. Decision criteria may be absolute but, normally, are relative in a sense that they allow an ordering of states or possible solutions. Possible decision criteria are reviewed such as:

- Target reliability
- Economical considerations
- Time constraints
- Socio-economical and political preference
- Codes and Standards
- Complexity of analysis

Three basic phases of the reassessment procedure depending upon the degree of the sophistication of the assessment are distinguished and discussed. Finally codification aspects on the reassessment procedure are proposed.

5. Part 2: Reliability Updating

Assessment of existing structures using methods of modern reliability theory should be seen as a successive process of model building, consequence evaluation and model updating by introduction of new information, by modification of the structure or by changing the use of the structure. The principle may be illustrated schematically as shown in Figure 1.

The analyses to be performed involve various steps:

- Formulation of *prior* uncertainty models
- Formulation of limit state functions
- Establishing *posterior* probabilistic models
- Performing *prior*, *posterior* and *pre-posterior* decision analysis
- Setting acceptable levels for the probability of failure.

The two first steps are briefly addressed together in order to introduce the philosophy of Bayesian probabilistic modelling in the assessment of existing structures. The next two points, however, are essential for the understanding of the framework of structural reassessment and are described in detail. The respective methodological aspects are provided and applied in an educational example. The issue of setting acceptable failure probabilities is central both for reliability based design and reliability based assessment of structures. This issue is considered in part 3 of the JCSS document.

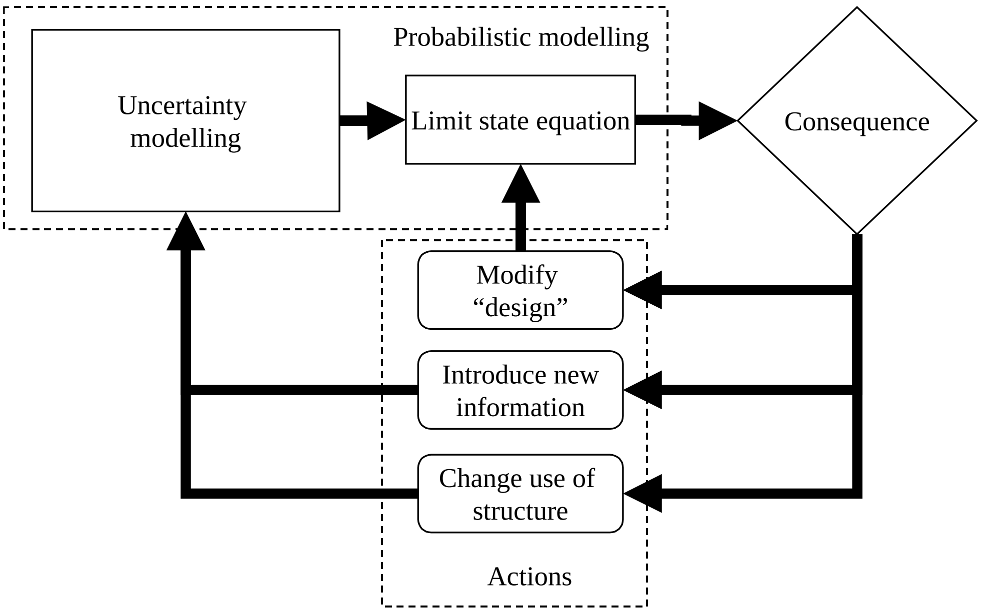

Figure 1: Bayesian probabilistic assessment of structures

6. Part 3: Acceptability and target criteria

For reliability-based design and reliability assessment of existing structures acceptability limits or targets have to be set. Both quantities are not necessarily the same as they may result from partially different criteria. Also, they are not necessarily the same for structures to be designed and structures which already exist because the decision point (point in time where a decision is made whether some requirements are fulfilled or not) and thus the degree of information, the relative effort to control reliability and potential failure consequences is changed. Acceptability limits or targets may also differ depending on whether one considers an entire building facility including other than structural failure modes or the structure itself in the narrow sense. It is further necessary to distinguish between limits or targets set for facilities including human error in its various forms (design error, failure of quality management, operation failure, ignorance, etc.) and limits or targets where such failure causes are not included.

It should be considered also whether limits or targets are related to individual failure modes or the failure modes of a system and, in accordance with present practice, in relation to the failure consequences. Such failure consequences may include direct financial losses due damage and for demolition and reconstruction, injuries or even loss of human lives but also so-called intangibles like loss of future opportunities (for example, loss of public welfare, professional reputation, and the like. Limits or targets may be different depending on whose behalf (for example, user, builder and public) decisions are to be made.

Finally, in a probabilistic context, such limits or targets are not independent of the set of probabilistic models used to verify them. This yet incomplete list of aspects when setting limits or targets indicates that the question of setting targets or limits is all but trivial. They are nevertheless mandatory to render probabilistic design and/or reliability assessment of existing structures operational.

Such limits or targets have been developed for structural components and systems in the narrow sense by not including non-structural failures modes and by not including failures due to human error or ignorance as a function of relative cost of safety measure and of degree of failure consequences.

Much debate has been thereby going on whether to include human lives into cost benefit analyses and whether it is at all admissible to perform cost benefit analysis when human lives or injuries are involved in case of structural failures. This requires to introduce a monetary equivalent to save human life and limb into the analysis. More recent studies on behalf of the public use so-called compound social indicators. Social indicators are statistics that reflect some aspect of the quality of life in a society or group of individual. More specifically, they aim to reflect broadly accepted goals that may carry labels such as national development, high expectancy of quality-adjusted life, the common good or the public interest. Any undertaking (project, program or regulation, adoption of new therapy, etc.) that affects the public by changing health or risk and expenditure will have an expected impact on a compound social indicator. A positive net impact of an undertaking on the accepted social indicator will lend to support the undertaking.

For example, the Life Quality Index (LQI) is intended as an indicator for "quality-adjusted life expectancy. It is a function of the real gross national product (GNP) per person and year and the life expectancy at birth. If applied to the fatality risk for structural failure in developed countries it can be shown that in the nineties of the 20th century expenditures for the safety a human life have approximately a value of US$ 100000 per year or about US$ 4000000 per average life time. By using the LQI it is possible to include human losses when deriving optimal target reliability indices.

The target values for the ultimate limit states related to failure of structural members are presented in Table 1. The values correspond to individual structural elements and to one year reference period and reflect as well code calibration experience and the aforementioned cost-benefit considerations. These values shall be considered in reliability analyses in association with the stochastic models for the influencing variables as described in the probabilistic model code [2]. In case of structures with extreme failure consequences the target values shall be defined based on risk-benefit studies. For existing structures the costs of achieving a higher reliability level are usually high compared to structures under design. For this reason the target level for existing structures usually should be lower.

Table 1: Tentative target reliability indices β (and associated target failure probabilities) related to one year reference period and ultimate limit states

Relative Cost of Safety Measure	Minor consequences of failure	Moderate consequen-ces of failure	Large consequences of failure
Large	β=3.1($p_F \approx 10^{-3}$)	β=3.3($p_F \approx 5x10^{-4}$)	β=3.7($p_F \approx 10^{-4}$)
Normal	β=3.7($p_F \approx 10^{-4}$)	β=4.2($p_F \approx 10^{-5}$)	β=4.4($p_F \approx 5x10^{-6}$)
Small	β=4.2($p_F \approx 10^{-5}$)	β=4.4($p_F \approx 5x10^{-5}$)	β=4.7($p_F \approx 10^{-6}$)

The grading for both the relative effort to achieve reliability and the expected failure consequences agrees also well with calculations provided in various studies.

It is further noted here that the relationship between the failure probability and the reliability index is expressed as:

$$\beta = - \Phi^{-1} (p_F) \qquad (1)$$

where p_F is the probability of failure

$\Phi^{-1}(\cdot)$ is the inverse Gaussian distribution

7. Part 4: Examples and case studies

Two educational examples and a case study are presented next.

Example 1: Timber Beam

Consider a timber beam as presented in Figure 1. First the reliability of this beam without any inspection will be estimated. Then the updating of this reliability will be demonstrated if the beam deflection is measured.

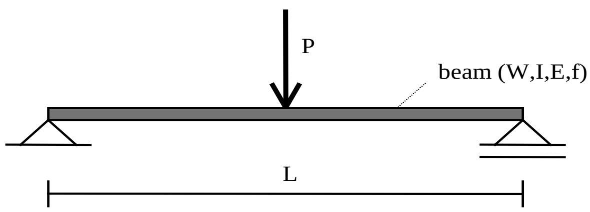

Figure 1: Simply supported timber beam with concentrated load

The limit state function for failure is defined as:

$$g = W f - 0.25\, P\, L \qquad (2)$$

For the meaning of the variables and their respective probability models reference is made to Table 2. All random variables are assumed to be normal for simplicity. The yield stress f and the modulus of Elasticity E are correlated with a correlation coefficient of $\rho(E,f) = 0.5$.

Table 2: Input data for example 1

Variable	**Designation**	**μ (mean)**	**v (c.o.v.)**
L	Span	4 m	-
W	plastic section modulus	0.01 m^3	-
I	moment of inertia	0.0002 m^4	-
P	load (annual maximum)	100 kN	0.20
P_t	test load	50 kN	-
f	yield stress	20 MPa	0.15
E	modulus of elasticity	30 GPa	0.20

Calculation of the failure probability

Given the data we may calculate the failure probability according to the classical method for linear limit state functions:

$\mu(g) = W\,\mu(f)$ - 0.25 $\mu(P)\,L$ = 0.01 x 20000 - 0.25 x 100 x 4 = 100 kNm

$\sigma(g) = \sqrt{}(W^2\,\sigma^2(f)+0.25^2\,\sigma^2(P)\,L^2) = \sqrt{}\,(0.01^2 \text{ x } 3000^2 + 0.25^2 \text{ x } 20^2 \text{ x } 4^2)$ = 36.1 kNm

$\beta = \mu(g)/\,\sigma(g)$ = 100/36 = 2.77

P_F = 0.0026 (see Eq. 1)

Measurement

Assume next that we do a measurement of the deflection u under a deterministic load of P_t = 50 kN. The expectation of the deflection is then approximately equal to:

$\mu(d) = P_t\,L^3/48\mu(E)I$ = 0.0011m = 11mm

Suppose the test gives $d = d_m$ = 9 mm. In that case we may conclude that the beam is better than expected. Given the positive correlation between stiffness and strength this should lead to an increase of the beam reliability. We will make this calculation by the two possible alternative procedures.

Procedure (1) Direct updating

For the direct calculation we introduce a so called “artificial limit state function" h for the measurement event, which is given by:

$$h = 48\,EI\,d_m - P_t\,L^3 \qquad (3)$$

If h = 0 the E-value corresponds exactly to the situation that $d = d_m$. The corresponding β_h can be calculated as follows:

$\mu(h) = 48\,\mu(E)I\,d_m - P_t\,L^3$ = 48 x 30000000 x 2.10^{-4} x 0.009 - 50 x 4^3 = - 608 kNm^3

$\sigma(h) = 48\,\sigma(E)I\,d_m$ = 518 kNm^3

β_h = -1.17

The negative β_h corresponds to the fact that beam behaves better than expected.

We now use the standard formulas for direct updating:

$$\mu(g \mid h = 0) = \mu(g) + \rho(g,h)\sigma(g)\frac{0-\mu(h)}{\sigma(h)} \qquad (4a)$$

$$\sigma(g \mid h = 0) = \sigma(g)\sqrt{}\{1-\rho(g,h)^2\} \qquad (4b)$$

The basic data, which follow from the previous calculations, are:

$\mu(g)$ = 100 kNm $\sigma(g)$ = 36.1 kNm, $\mu(h)$ = -608 kNm^3 , $\sigma(h)$ = 518 kNm^3

In order to find the coefficient of correlation we first calculate the covariance using a standard first order approximation:

$$\text{cov}(g,h) = \{W\}\{48 I u_m\}\sigma(f)\sigma(E)\rho(E,f) = 7776 \text{ (kN)}^2 \text{ m}^3$$

Note that in this case gand h are linear functions of the random variables. And so finally:

$$\rho(g,h) = \frac{\text{cov}(g,h)}{\sigma(g)\,\sigma(h)} = \frac{7776}{36.1 \cdot 518} = 0.42$$

Inserting the numbers in the basic equations for direct updating we obtain:

$\mu(g \mid h=0)$ = 117.6 *kNm*

$\sigma(g \mid h = 0)$ = 32.8 *kNm*

and this leads to an updated reliability index $\beta(g|h)$ equal to:

$\beta(g \mid h = 0)$ = 117.6/32.8 = 3.59

This means that the good test result has increased β from 2.77 to 3.59. If, for instance, we would have started from d_m = 14 mm we would have found the updated β to be 2.4. In that case the beam has low E and probably a corresponding low f, leading to a reduction of the reliability.

Procedure (2) Updating of individual random variables

As an alternative we could also update the random variables with f instead of M and E in stead of h. From d_m = 0.009 m we may derive that E = 37037 MPa deterministically. We now may update the mean and standard deviation of f according to:

$$\mu(f \mid E = 37037 \text{ MPa}) = \mu(f) + \rho\sigma(f)\frac{37037 - \mu(E)}{\sigma(E)} = 21.76 \text{ MPa}$$

$\sigma(f \mid E\text{=37000 MPa}) = \sigma(f)\sqrt{1-\rho^2}$ = $3\sqrt{1-0.5^2}$ = 2.60 MPa

If we redo the limit state reliability analysis using this new model for f we find:

$\mu(g|E\text{= 37 GPa}) = W\,\mu(f\,|E) - 0.25\,\mu(P)\,L$ = 0.01 x 21800 - 0.25 x100 x 4 = 117.6 kNm

$\sigma(g|E\text{= 37 GPa}) = \sqrt{(W^2\sigma^2(f\,|E)+0.25\,\sigma^2(P)\,L^2)} = \sqrt{(0.012 \times 2600^2+0.25^2 \times 20^2 \times 4^2)}$=32.8 kNm

β = 117.6/32.8 = 3.59

In this case the procedure is relatively easy, because only one variable is involved. In general, however, the first procedure is to be preferred.

Semi-probabilistic verification

We could even have a semi-probabilistic updating and telling that the characteristic value (5% fractile) for the strength has increased from:

$fc = \mu(f) - 1.645\,\sigma(f)$ = 20 - 1.645 x 3 = 15.1 MPa

to

$fc_{|E} = \mu(f\,|E = 37 \text{ GPa}) - 1.64\,\sigma(f\,|E = 37 \text{ GPa})$ = 21.8 - 1.64 x 2.6 = 17.5 MPa

and perform an updated level I analysis.

Example 2: Inspection of fatigue cracks

Consider a steel structure where various nodes are inspected for fatigue cracks. Failure in those cases will happen if the inspection results are considered as satisfactory but the failure event (nevertheless) occurs, assuming that some adequate action is taken if the inspection is not satisfactory.

Let the fatigue crack for some selected node grow as indicated in Figure 3. Fatigue failure will occur as soon as the crack $a(t)$ reaches a random critical length a_{crit}, so the failure probability for a period t can be written as:

$$P_F(t) = P\{M_f < 0\} = P\{a_{crit} - a(t) < 0\} \qquad (5)$$

Note that a_{crit} is considered as time-independent; if a_{crit} is considered as time dependent (as it is in reality) this equation becomes more complex. Note also that $a(t)$ is an increasing stochastic process as cracks do not get smaller.

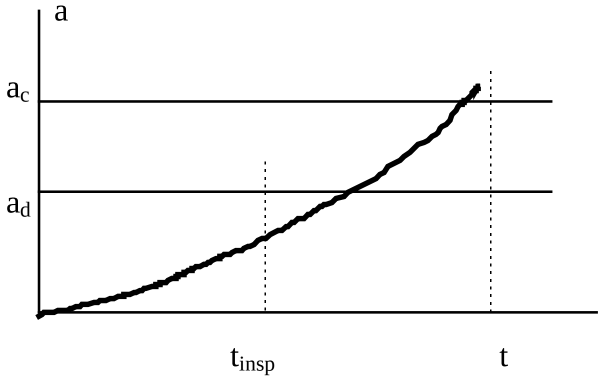

Figure 3:Fatigue failure before time t occurs if at inspection the crack length is smaller than a_d and at time t the crack length is larger than a_{crit}

Let the reliability be considered as inadequate: that is, $P_F(t) > P_{Ft}$ when P_{Ft} is the target probability for fatigue failure. For this reason an inspection is planned at some point t_{insp} during the life time. Let the decision rule be that the structure will be repaired if a crack is detected, that is if a crack $a(t_{insp})$ larger than random detection limit a_d is detected.

Updated reliability analysis

The probability of failure, given a positive inspection can be written as:

$$P_F(t) = P\,(a(t) > a_{crit} \mid a_m(t_{insp}) < a_d) \qquad (6)$$

The first event represents “failure” and the second one "fit at inspection". In terms of limit state functions this may be rewritten as:

$$P_F(t) = P(g < 0 \mid h_i < 0) \qquad (7)$$

with:

$g = a_{crit} - a(t)$ and $h_i = a(t_{insp}) - a_d$

So one can obtain based on reliability updating methods:

$P_F(t) = P(g < 0 \mid h < 0) = P(g < 0 \cap h < 0) / P(h < 0) =$

$$= \Phi\left(-\frac{\beta_h - \rho\beta_g}{\sqrt{1-\rho^2}}\right) \Phi(-\beta_g) / \Phi(-\beta_h) \tag{8}$$

In order to find β_h, β_g and ρ we need a calculation model and statistical parameters for all random variables. We will not go into details here, but assume that a calculation leads to:
$\beta g = 2$, $\rho = -0.8$ and $\beta_h = 1$

So, the value of $\beta_g = 2$ is considered as inadequate and inspection is planned at t_{insp}. The value of β_h corresponds to 16% probability of finding a crack larger than the detection limit a_d at time t_{insp}. Then:

$$P_F(t) = \Phi\{\frac{1-0.8\cdot 2}{0.6}\}\Phi(-2)/\Phi(1) = \Phi(-1.0)\Phi(-2)/\Phi(1)) = 0.16 \cdot 0.0228/0.84 = 0.0043$$

So the inspection raises the reliability index from 2.0 to 2.6 (approximately).

Complete event tree
It is also interesting to observe the total event tree for this case, standing at t=0. This tree is given in Figure 4

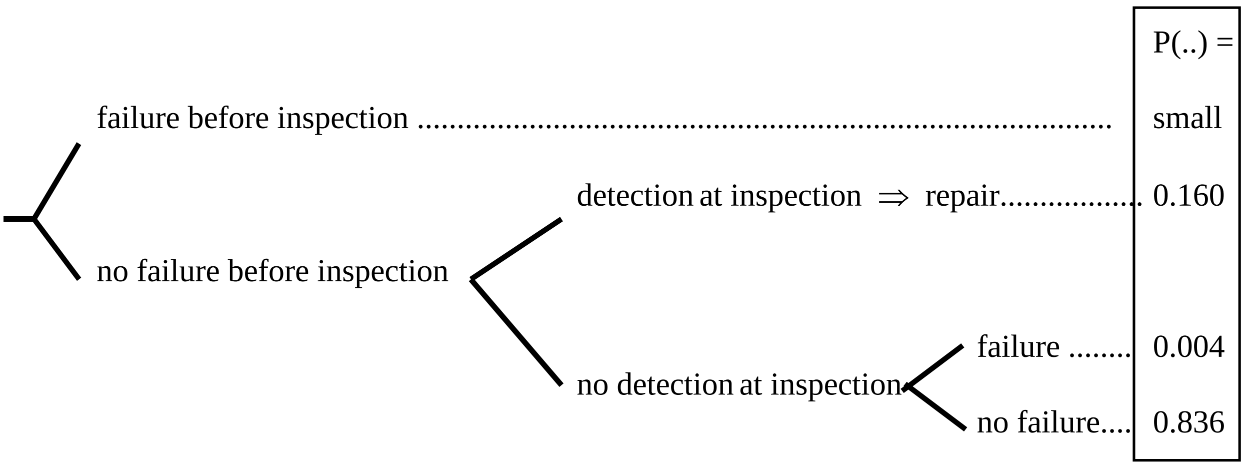

Figure 4: Event tree for inspection and failure events

At $t = 0$ we have first the possibility that failure occurs before the inspection is planned. Let us assume here that the time of inspection has been chosen in such a way that this probability is small. If no failure prior to inspection occurs, this inspection may reveal a defect ($a\ (t_{insp}) > a_d$), which leads to a repair action. In this example the probability for this branch in the event tree is $\Phi(-1) = 0.16$. It is assumed that the probability of failure after repair is negligible. If the inspection provides satisfactory results ($a\ (t_{insp}) > a_d$), we then may have either failure or no failure in the period between inspection and the desired life time (or next inspection in a more advanced example). the probability of failure is 0.004. If also cost values are attached to inspection and failure, the optimal time of inspection and repair level a_d can be found.

Case Study

The third case study related to the pile capacity is summarised here. It presents the deterministic and probabilistic analyses of an offshore pile foundation at two times in the platform lifetime.

1. In 1975, before platform installation, when limited information and limited methods of interpretation of the soil data were available.

2. In 1999, after a reinterpretation of the available data using the geotechnical improvements attained in the interim additional and more advanced laboratory tests, a reanalysis of the loads, and an analysis of the installation records.

The reanalysis in 1999 was prompted because the environmental loads had been revised and the operators hoped to increase the gravity loads on deck. The structure consists of a steel jacket installed in 110 m of water in the North Sea. The jacket was installed in 1976. The jacket rests on four pile groups, one at each corner. Each pile group consists of six piles. The piles in the groups are 60" diameter tubulars, with wall thicknesses of 3 and 2.5". The soil profile consists of mainly stiff to hard clay layers, with relatively thinner layers of very dense sand in between.

The results of the analyses are summarised next. In 1975, only deterministic calculations were carried out. The 1975-probabilistic calculations were run in 1999 for the purpose of this example calculation.

Table 3: Results of the case study

Soil Profile	Deterministic factor of safety*	β Reliability index	Probability of failure, P_F
1975	1.73	2.06	2.0×10^{-2}
1999	1.39	2.41	0.8×10^{-2}

We may conclude that the factor of safety is not a sufficient indicator of safety margin because the uncertainties in the analysis parameters affect probability of failure, but these uncertainties do not intervene in the deterministic calculation of safety factor. The safety of the foundation is higher in the reanalysis phase.

8. Concluding remarks

This paper gives an introduction to the JCSS document on existing structures which is published under RILEM [1]. The contents of the document such as general guidelines on reassessment, methodologies for reliability updating, acceptability and target safety criteria, examples and case studies, are briefly described. The scope of the document and the benefits related to its applicability are outlined.

The JCSS document is of educational type and provides relevant information on how to process specific information about an existing structure, how to update its reliability based on such information, how to base decisions regarding maintenance, strengthening, upgrading etc. It is generally applicable for various materials and various structure types and therefore of general use.

9. References

[1] Joint Committee on Structural Safety (JCSS), *Assessment of Existing Structures*, RILEM Publications S.A.R.L., 2000.

[2] Joint Committee on Structural Safety (JCSS), *Probabilistic Model Code*, www.jcss.ethz.ch